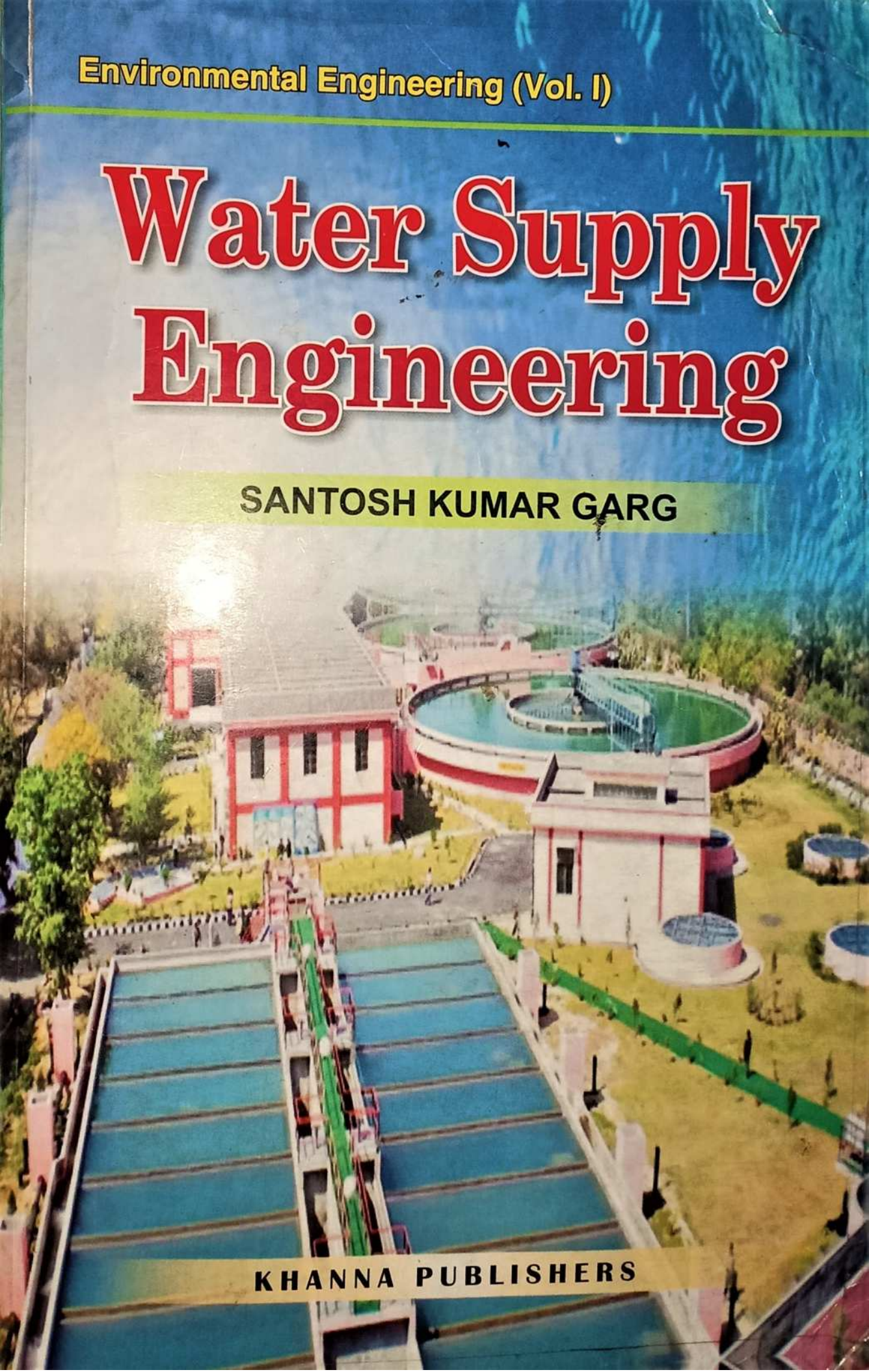
Environmental Engineering (Vol. I)

Water Supply Engineering

SANTOSH KUMAR GARG



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WATER SUPPLY ENGINEERING

[For Civil, Environment, and Agriculture Engineering Degree & Post Graduate Courses ; MSc (Environmental Science) Courses ; Engg. Services and Civil Services Competitive Examinations of UPSC ; GATE examinations for admissions to M. Tech. Courses ; AMIE (Section B) Examinations ; and for Professional Field Engineers.]

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(Revised 28th Edition)

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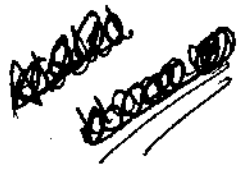

Dedication

This book is dedicated to Pandit Jawaharlal Nehru, who was the pioneering visionary, responsible for the large scale exploitation of our available water supplies

PREFACE TO THE 28th REVISED EDITION

The first edition of this book was published in October 1977, and during the intervening 38 years, more than 3 lakh copies have been sold through its twenty seven editions and their reprints. Additional 11000 copies of the present edition are being printed. The content matter of the book and its size have also undergone vast changes during these years, since the author has done his very best, to keep pace with the advancing world, and to include in the book, all what is required, atleast for the undergraduate courses, without making the text difficult and boring.

The 11th edition of the book included three new chapters on *Water Supply Plumbing Systems in Buildings ; Rural Water Supplies* ; and *Water Treatment in Swimming Pools*. Besides this, a **full fledged design of a water treatment plant project** was given in Chapter 13, on specific demand from *Prof. A.M. Malek*, Asst. Prof. at L.D. Engg. College, Ahmedabad. The request of Mr. Malek in the interests of the students, was highly appreciated and acknowledged. In the 12th edition, an article on "*Packaged Mineral Waters*", was added. In the 13th edition, one new chapter on **Laboratory Experiments in Water Supply Engineering (Chapter No. 15) was added.**

Roof top rain water harvesting was added and explained again in the 14th edition, besides making several other changes and additions.

In the 15th edition, large scale additions and changes were made in the text, by completely rewriting the topics on *'Desalination, Fluoridation and Defluoridation'* etc., besides including and explaining *Arsenic Contamination and its removal.*

In the 16th edition, several changes were again affected in the chapter of Purification of Water Supplies, to explain the design of Stock sand for filters. Latest amendments in prescribed standards of Drinking Water and of Packaged water were also incorporated in this edition.

Certain more changes were brought out in the 17th edition, and an article on pricing of water and existing water rates for domestic and industrial supplies in various States of the country were also given.

The 21st edition was further updated and improved on a large scale, besides being recomposed in a **new size of 24 cm × 16 cm**, as against the old size of 20 × 13 cm, to make the book more appealing and easily handleable, and for reducing the total number of pages.

This revised 28th edition now includes the latest Conventional questions of the *Engineering Services Exams*, upto the year 2015; and of the *Civil Services Exams*, upto the years 2014, duly supported with Hints and Solutions where required. The *Objective Section* has also similarly been updated, by including the latest objective questions of the Engineering Services Exams upto the year 2015.

The author feels delighted to know about the growing popularity of the book, not only amongest the average students, but also among the toppers of the various engineering colleges of the country as well as in foreign countries. *It is*

also a matter of great pleasure to the author as well as to the students to find questions being directly set from this book year after year, in the prestigious "Civil Services" and "Engineering Services" Exams. The author expresses his gratitudes to all such learned *Professors, Examiners* and *Paper Setters*, who have valued this book worthy of such a high acclaim.

Since my wife, **Rajeshwari Garg,** has largely contributed and helped in causing large scale revisions and improvements of this book, edition after adition, she has been associated as a junior co-author in this book.

ACKNOWLEDGMENTS

In going through the twenty six editions of the book, the author has accumulated an enormous amount of intellectual debts from students and teachers over the years, who had written to him about their satisfaction with the book, and extended suggestions for the improvement of its various editions. *If the book today is so strikingly popular, they all deserve much of the credit.* The author certainly wants them to know of his gratitudes and openness to further suggestions, and is hence enlisting some of the names below :

Prof. S.C. Prasad of Moti Lal Nehru Regional Engg. College Allahabad ; *Sh. P. Sen*, the then Senior Manager NHPC ; *Mr. Umesh Mishra*, a student of Allahabad Engg. College ; *Sh. Satish Aggrawal*, a student of M.R. Engg. College, Jaipur ; *Sh. Ashfaq Ahmed* of Shri Ram Rayon's Kota ; *Sh. H.S. Bhatia*, Professor of Delhi College of Engineering ; *Sh. A.M. Malek* of S.S. College Sidsar Gujarat ; *Sh. A.P. Thankhavel* of A.K. College of Engg. Krishna Kovil, Tamilnadu ; *Sh. G. Sankara*, a student of S.V. College of Engg. Machllipatnam, Karnataka ; **Sh. Biswaroop Ghosh**, an ex-student of Bengal Engg. College, Calcutta ; *Sh. Krishan Mohan Kansal*, a student of Delhi College of Engineering ; *Miss Shanta W. Kulkarni*, a student of Regional Engineering College Hulkoti, Karnataka ; **Sh. Pawan Goyal**, an ex-student of MBM Engineering College Jaipur, Rajasthan ; Mr. Palash Benerjee, an M. Tech. student of BIT Mesra, Ranchi etc., etc.

The excellent comments and suggestions received on the 10th edition from various professors had, in fact, caused the author to affect large scale revision and additions in the book. The suggestions furnished by these professors are hence especially acknowledged, by listing their names below :

(*i*) **Mr. R.K. Rai**—Lecturer in Govt. College of Engg. Karad, Maharashtra-415124.

(*ii*) **Dr. S.C. Prasad**—Professor at Motilal Nehru Regional Engg. College, Allahabad, U.P.-211004.

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(*v*) **Prof. V.S. Prasad**—Prof. at Regional Engg. College, Calicut, Kerala-713601.

The author also takes this opportunity to express his sincere gratitudes to one of his old professors, **Prof. R. Narayanan**, the then Head of the Civil Engineering Deptt. at Delhi College of Engineering, who encouraged this

author to take up writing work, when he was highly depressed in life, at its prime stage, due to his medical problems, resulting in the loss of his class I appointment through the Engineering Services Competition, 1970.

The author also expresses his thanks to *Sh. Kratu Khanna S/o Vineet Khanna* of Khanna Publishers, and *Mr. Sunder Goswami* of Goswami Printers, for ensuring good and prompt composing of this revised edition. Since the entire book in its 21st edition was largely revised and entirely recomposed in a new size, and all the figures were redrawn on computer, new mistakes of printing might have crept in. The author shall be grateful, if such mistakes are brought to his knowledge for making amendments and changes in the subsequent editions.

New Edition
August 11, 2015

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Introduction to S.I. Units

Different countries have been following different systems of units, like f.p.s. system, c.g.s. system, m.k.s. system, etc. However, in order to avoid consequent confusion in international trade and to introduce uniformity over the entire world, it was decided during the 11th General Conference on Weights and Measures held in Paris in the year 1960, that a new system called **S.I. (System International)** be adopted all over the globe. Confirming to this international agreement, India has also now largely shifted from the *mks* units to the S.I. units.

Be that as it may, the S.I. unit system is largely similar to the m.k.s. system, because of the fact that the basic units of length, mass, and time are the same in both the systems ; *i.e.*, the length is measured by metre (m), the mass is measured by kilogram (kg), and the time in seconds (s). This system is, rather a rationalisation of m.k.s. system, as it removes confusion between kg and kg wt (also usually exressed as kg), as adopted in m.k.s. system.

S.I. Unit of Force. The basic difference between m.k.s. units and S.I. units is in the unit of force. Whereas, in the m.k.s. system, the unit of force is kg wt or kgf (kg force) usually written as kg ; in S.I. units, the unit of force is *Newton* (*N*).

One Newton (N) *is defined as the force required to move a body of mass equal to 1 kg, as to generate in it, an acceleration equal to 1 m/sec*^{2}. Since

Force = mass × acceleration, we have by definition of Newton

$$1 \text{ Newton} = 1 \text{ kg} \times 1 \text{ m/sec}^2$$

or $$\boxed{1 \text{ N} = 1 \text{ kg m/sec}^2}$$

Hence, Newton (N) will have the dimensions of kg m/sec^2.

Moreover, the relation between two force units *i.e.* kgf and N would be worked out as ;

$$1 \text{ kgf} = 1 \text{ kg wt}$$
$$= 1 \text{ kg}. g$$

where g is acceleration due to gravity
$= 9.807 \text{ m/sec}^2$

$$= 1 \text{ kg} \times 9.807 \text{ m/sec}^2$$
$$= 9.807 \text{ kg m/sec}^2$$
$$= 9.807 \text{ N}$$

or $$\boxed{1 \text{ kgf} = 9.807 \text{ N}}$$

Newton is, thus, a very small unit of force (approximately 1/10th of kg wt) and hence, it is customary to use a bigger unit, such as, **kilo-Newton**

(kN) or mega newton (MN) for expressing force ; kilo newton being 10^3 N, and mega newton being 10^6 N, as usual.

S.I. Unit of Stress or Pressure. Since the force per unit area is *stress* or *pressure*, we would evidently have S.I. unit of pressure as Newton/m^2 (N/m^2), and this is known as **Pascal P_a**).

The m.k.s. unit for pressure being kgf/m^2, usually written as kg/m^2) or its bigger unit of kg/cm^2. The relation between these two units can be derived as follows :

$$kg.\ f/m^2 = 9.807\ N/m^2$$

or $$9.807\ N/m^2 = 1\ kg/m^2$$

(kgf/m^2 being usually expressed as kg/m^2)

or $$9.807\ N/m^2 = \frac{1}{10^4}\ kg/cm^2$$

or $$1\ kg/cm^2 = 9.807 \times 10^4\ N/m^2$$

or $$\boxed{1\ kg/cm^2 = 98.07\ k\ N/m^2}$$

Kilo Pascal (kP_a), the bigger S.I. unit of pressure would then be

$$kP_a = kN/m^2 = \frac{1}{98.07}\ kg/cm^2$$

or $$100\ kP_a = \frac{100}{98.07}\ kg/cm^2$$

$$= 1.02\ kg/cm^2$$

$$= 1.02 \times 10\ m\ head\ of\ water$$

$$= 10.2\ m\ of\ water\ head.$$

$\therefore$ $$\boxed{100\ kP_a = 1.02\ kg/cm^2 = 10.2\ m\ of\ water\ head}$$

S.I. Unit of Energy. *When the applied force of 1 newton displaces a body by 1 metre in the direction of application of the force, then work is said to be done equal to 1 newton metre* (1 N-m), called 1 *Joule* (J). The S.I. Unit of work or energy is, thus, **Joule** (J), and has the dimensions of N. m *i.e.* $kg\ \frac{m}{sec^2}\ m$, *i.e.* $kg\ m^2/sec^2$.

or $$1\ Joule = 1\ Nm = 1\ kg\ m^2/sec^2$$

The m.k.s. unit for energy is kgf. m (commonly written as kg-m)

$\therefore$ $$1\ kg\ .\ m = 9.807\ N.\ m = 9.807\ J$$

$\therefore$ $$\boxed{1\ kg\ m\ of\ energy = 9.807\ N.m = 9.807\ J}$$

kilo joule (kJ) will also be used as unit of energy and would be equal to 1000 J = 1 kN. m.

S.I. Unit of Power. Power, as we know is the rate of doing work, *i.e.* energy per second. Its m.k.s. unit being kg-m /sec (strictly speaking kgfm/sec). The S.I. unit for power would evidently be Joule per second (j/s), or N.m/s, and this is called **watt** (W).

$$1 \text{ watt} = 1 \text{ j/s} = 1 \text{ Nm/s}$$

It may also be pointed out here, that the term *horse power,* being used as the unit of power in m.k.s. system and fps system, is not at all used in S.I. units. The relation between j/s (*i.e.* watt) and kgf.m/sec can be worked out as below :

$$1 \text{ kgf m/sec} = 9.807 \text{ N.m/sec}$$
$$= 9.807 \text{ j/s} = 9.807 \text{ W}$$

or $\boxed{1 \text{ kgm/sec} = 9.807 \text{ W}}$

$$1 \text{ H.P. (metric)} = 75 \text{ kgm/sec}$$
$$= 75 \times 9.807 \text{ W} = \mathbf{735 \text{ watts}}$$
$$1 \text{ H.P. (fps)} = 550 \text{ ft lb/sec}$$
$$= 550 \times 0.138 \times 9.807 \text{ watts} = \mathbf{746 \text{ watts.}}$$

S.I. Unit of Temperature. The unit of temperature in S.I. units is **kelvin** (K), and is equal to the temperature interval which is equal to temperature interval corresponding to 1°C. The relation between K and °C is given as :

$$°C + 273° = K$$

—1

Introduction

1.1. Importance and Necessity for Planned Water Supplies

Water is a chemical compound and may occur in a liquid form or in a solid form or in a gaseous form. All these three forms of water are extremely useful to man, providing him the luxuries and comforts, in addition to fulfilling his basic necessities of life. Everyone of us knows how important and precious the water is. Whenever there is no water in our taps, we become helpless. No life can exist without water, since water is as essential for life as air is. It has been estimated that two-third of human body is constituted of water. Water is absolutely essential not only for survival of human beings, but also for animals, plants and all other living beings. Further, it is necessary that the water required for their needs must be good, and it should not contain unwanted impurities or harmful chemical compounds or bacteria in it. Therefore, in order to ensure the availability of sufficient quantity of good quality water, it becomes almost imperative in a modern society, to plan and build suitable water supply schemes, which may provide potable water to the various sections of community in accordance with their demands and requirements. The provision of such a scheme shall ensure a constant and a reliable water supply to that section of the people for which it has been designed. Such a scheme shall not only help in supplying safe ***wholesome**** water to the people for drinking, cooking, bathing, washing, etc. so as to keep the diseases away and thereby promoting better health ; but would also help in supplying water for fountains, gardens, etc. and thus helping in maintaining better sanitation and beautification of surroundings, *thereby reducing environmental pollution*. Besides promoting overall hygiene and public health, it shall ensure safety against fire by supplying sufficient quantity of water to extinguish it. The existence of such a water supply scheme shall further help in attracting industries (since industries require large amounts of water) and thereby helping in industrialisation and modernisation of the society, consequently reducing unemployment and ensuring better living standards. Such schemes shall, therefore, help in promoting wealth and welfare of the entire humanity as a whole.

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*The water which is safe and potent for drinking to human health is referred to as **wholesome water**. Such a water will *neither* contain any chemicals or impurities in excess of which they may harm a human body *nor* any harmful bacteria or microorganisms. A water which is **safe** due to absence of harmful bacteria may or may not be wholesome, but wholesome water will always be safe.

1.2. Financing of Water Supply Schemes and Their Development in India

Huge amounts of money is required for planning, designing and executing a city water supply scheme. The first investment as well as the RMO (*i.e.* Running, Maintenance and Operation) charges, along with depreciation and interest charges, must be returned by the beneficiaries in the form of ***revenue returns*** from the sale of water, which is generally measured by meters. It is, therefore, necessary to work out fair and reasonable charges for the sale of treated water, in addition to designing the project.

In a developing country like India, the problem of availability of funds for the first investment is sometimes very difficult to be solved, as money is required for financing various other important projects such, as those of flood control, education, defence, health, etc. Inspite of such handicaps, the Indian Govt. has done fairly well by spending about 1—3% of their total plan outlays on "Water Supply and Sanitation Sector', as reflected in Table 1.1.

As a result of such investments, India has been able to cover about 84% of its urban population (which is about 30% of the total population of the country) with safe drinking water supplies, although such supplies are quite limited*. Moreover, the scene on the rural sector still remains grim and at many of our villages, the village folk are till today fetching and bringing water on their heads from distant wells or ponds, and there is no provision for testing or purifying those waters. Even in towns, where municipal committees do exist, there are no sufficient provisions even for testing and purifying the drinking water supplies except using chlorination at certain places.

Let us, therefore, in all sincerity, appeal to the Indian Government that it should try to further curtail expenses on luxurious livings and should invest that money on improving the town and village water supplies, as also the sewerage and drainage arrangements, so that our villages, towns and cities no longer remain heaps of filth, dung and garbage. The Indian Socialistic Government under the leadership of Late Pt. Jawahar Lal Nehru and his daughter Smt. Indira Gandhi, had definitely achieved some remarkable successes in the fields of irrigation, education, atomic energy, defence, etc. but unfortunately the field of public health engineering remains somewhat neglected. Let us hope that this sector will also get its enhanced due share and a good treatment from the hands of our planners, as most of the money being spent on medicines and under health sector etc. can be saved, if unadulterated food, proper sanitation, better hygiene, and above all, *safe and wholesome water*, are made available to the general public.

1.3. Pricing of Municipal Water Supplies

Water supplied by the municipalities to urban and rural areas for domestic, commercial, or industrial uses has to be priced in such a way that the RMO (Running, Maintenance and Operation) costs as well as the depreciation and interest charges on the capital investment is returned by the beneficiaries in

*None of the metropolitan cities of India (including Delhi) is being supplied water for 24 hours of the day. Even Kolkatta, where the first modern water supply system was installed in 1870, faces shortage of water and supplies water only for 10 hours a day ; while Delhi supplies hardly for 6 hours a day.

Table 1.1. Planwise Investment in Water Supply and Sanitation Sector in India

Plan Period		*Total Public Sector Plan Outlay/ Expenditure Rs. Crores*	*Plan Outlay/Expenditure under Water Supply and Sanitation Sector*		*Plan Outlay/Expenditure for Urban Water Supply Sanitation*		*Plan Outlay/Expenditure for Rural Water Supply and Sanitation*	
			Amount Rs. Crores	*Public Sector Outlay, %**	*Amount Rs. Crores*	*Public Sector Outlay**, %*	*Amount Rs. Crores*	*Public Sector Outlay***, %*
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
First Plan (1951-56)	Outlay	3360.00	49.00	1.46	43.00	1.28	6.00	0.18
	Expenditure	**1960.00**	**11.00**	**0.56**	**8.00**	**0.41**	**3.00**	**0.15**
Second Plan (1956-61)	Outlay	6750.00	72.00	1.07	44.00	0.65	28.00	0.41
	Expenditure	**4672.00**	**74.00**	**1.58**	**44.00**	**0.94**	**30.00**	**0.64**
Third Plan (1961-66)	Outlay	8573.00	105.70	1.23	89.37	1.04	16.33	0.19
	Expenditure	**8576.00**	**110.17**	**1.28**	**91.34**	**1.07**	**18.83**	**0.22**
Annual Plans (1966-69)	Outlay	6664.97	106.42	1.60	N.A.	N.A.	N.A.	N.A.
	Expenditure	**6625.40**	**102.70**	**1.55**	**73.53**	**1.11**	**29.17**	**0.44**
Fourth Plan (1969-74)	Outlay	15902.00	437.00	2.75	282.00	1.77	155.00	0.99
	Expenditure	**15778.80**	**458.90**	**2.91**	**250.90**	**1.59**	**208.00**	**1.32**
Fifth Plan (1974-79)	Outlay	39303.49	1030.68	2.62	549.44	1.40	481.24	1.22
	Expenditure	**39426.20**	**1091.60**	**2.77**	**539.51**	**1.37**	**552.09**	**1.40**
Annual Plan (1979-80)	Outlay	12549.63	430.22	3.43	197.93	1.58	232.29	1.85
	Expenditure	**12176.50**	**395.28**	**3.25**	**148.99**	**1.22**	**240.39**	**1.97**
Sixth Plan (1980-85)	Outlay	97500.00	4047.00	4.15	1766.68	1.18	2280.32	2.34
	Expenditure	**1,09,291.70**	**3997.78**	**3.66**	**2334.53**	**2.14**	**1663.45**	**1.52**

$*\frac{\text{Col. 4}}{\text{Col. 3}} \times 100$; $**\frac{\text{Col. 6}}{\text{Col. 3}} \times 100$; $***\frac{\text{Col. 8}}{\text{Col. 3}} \times 100$.

Table 1.1. Contd.

Plan Period		*Total Public Sector Plan Outlay / Expenditure Rs. Crores*	*Plan Outlay / Expenditure under Water Supply and Sanitation Sector*		*Plan Outlay / Expenditure for Urban Water Supply Sanitation*		*Plan Outlay / Expenditure for Rural Water Supply and Sanitation*	
			Amount Rs. Crores	*Public Sector Outlay, %**	*Amount Rs. Crores*	*Public Sector Outlay**, %*	*Amount Rs. Crores*	*Public Sector Outlay***, %*
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
Seventh Plan (1985-90)	Outlay	1,80,000.00	6522.47	3.62	2965.75	1.65	3556.72	1.98
	Expenditure	**2,19,029.23**	**7093.13**	**3.24**	**2557.81**	**1.17**	**4535.32**	**2.07**
Annual Plans (1990-92)	Outlay	1,37,033.15	4427.29	3.23	1721.37	1.26	2705.92	1.97
	Expenditure	**1,33,834.85**	**4391.48**	**3.28**	**1693.69**	**1.27**	**2698.29**	**2.02**
Eighth Plan (1992-97)	Outlay	4,34,100.00	16711.03	3.85	5982.28	1.38	10728.79	2.47

$*\frac{\text{Col. 4}}{\text{Col. 3}} \times 100$; $**\frac{\text{Col. 6}}{\text{Col. 3}} \times 100$; $***\frac{\text{Col. 8}}{\text{Col. 3}} \times 100$.

the form of revenue returns from the sale of water. From good economics point of view, there should be no under-pricing and subsidies involved in such water supply schemes. However, in a poor and developing country, the rates for domestic supplies are to be kept lower than the rates for commercial uses. Similarly, the rates for commercial uses should be lower than the rates for industrial uses. Although, fixation of such ***differential rates for various uses*** is permissible, yet the rates for water supplied should not be too low, as not to offer any incentive for saving and conserving water. When water rates are heavily under-priced, the consumers waste water by litres, as no one has the real idea of the real cost spent by the Govt. in supplying water. The Govt's accounts also don't reflect the true value of water lost in transits and thefts ; and hence when measures are taken to plug the losses by preventing supply loopholes, it appears that the cost spent on these measures is more than the cost of water saved, which, in fact, is a total misnomer. Low water rates, thus, bring water management on a low priority area, and leads to alround wastage and mismanagement of water supply schemes.

The actual prevailing water rates in India for municipal supplies are, in fact, abysmally low, when compared to the global standards. There are no regulators to compute and fix the economical and justified water tariffs, and the State Govts. do fix the water rates on adhoc basis, without any rational computations for affecting recovery of O & M charges & capital investments. What to talk of recovery of depreciation & interest costs on capital costs, the rates are usually too low, as even not to recover the O & M charges alone. Although, no official estimates have been made to compute the tariff levels, which will allow full recovery of operation, maintenance and capital costs ; yet various expert analysis have suggested a tariff of ₹ 15/- ($ 0.3) per kL (cu. m) for the treated water supplied through piped supplies. As compared to this economic rate of ₹ 15/- per kL, the existing rates in the country vary from about ₹ 0.01 per kL to about ₹ 3.60 per kL over most of the States of India, except for Mizoram (where minimum rate is ₹ 10/- per kL), as can be seen from the prevailing domestic water rates given in Appendix Table A-1. From this table, it can be seen and inferred that the rates fixed for domestic water supplies in most of our States are on extremely low side. Even for the metro cities, the water rates, on an average, are fixed at not more than about ₹ 1.50 per kL, *i.e.* @ about $ 0.03 per kL. Although the rates fixed for industrial water supplies are higher (Appendix Table A-2), yet even this rate averages to around ₹ 12/- per kL in metros and ₹ 5/- per kL in smaller cities, much below the **cost recovery tariff of about ₹ 15/- per kL** ($ 0.3). Appendix table A-3 reflects the water rates prevailing in various countries of the World in U.S. dollars ($) per kL, which reflects as to how abysmally low Indians pay for their water supplies, in comparison to other countries.

There, therefore, exists an urgent necessity in India to rationalise the water rates, and provide meters to bill the consumers on their actual consumption, rather than charging them an adhoc monthly basis, which is being done in several States. A minimum economical rate of ₹ 6/- per litre has been suggested to be fixed by NGO—PPIAF (*Public-Private Infrastructure Advisory Facility*), which can be afforded even by the poorest of the families. In any case, there exists an urgent need to fix **regulators** to work out economic water

terrifs, keeping into consideration the cost of providing public water supplies, and economic conditions of households. Higher water prices will not only encourage all users to use water more efficiently, but will also generate funds to maintain the existing water infrastructure and to build new infrastructure. The State Governments will have to show political courage to do so, before the water completely runs out, and people become totally dependent upon packaged waters, which is available at around ₹ 15/- per litre bottle.

1.4. Planning and Execution of Modern Water Supply Schemes

In planning a water supply scheme, it is essential, to first of all, search a source of water in the vicinity of the town or the city for which the scheme is to be designed. Sometimes, the water may be available nearby, and sometimes it may be far away. Further it may be an underground well, or it may be a river, stream or a lake. It is, therefore, necessary to seek out all the possible sources and evaluate each in terms of quantity, quality and cost, and then to take a final decision regarding the utilisation of a particular source or sources depending upon the availability of water in those sources and the water demand of the town or the city.

Suitable systems should then be designed for *collecting, transporting*, and *treating* this water. The treated water is finally distributed to the residents and industries depending upon their requirements, through a network of *distribution system*. The essential elements of a public water supply scheme may, therefore, consist of *Intakes and reservoirs* ; *a water treatment plant* having screening, sedimentation, filtration, disinfection units, etc. ; elevated *tanks and stand pipes* which provide storage to meet peak demands occurring for limited periods ; *valves* which control the flow of water in the pipe system ; *hydrants* which provide a connection with the water in the mains for fighting fires, flushing streets, etc. ; *mains, sub-mains* and *branch lines* which carry the water to the streets ; *services* which carry the water to the individual homes, etc. etc. All these essentials and components of a water supply scheme are described in details in the subsequent chapters of this volume.

PROBLEMS

1. (*a*) What is the importance of public water supply schemes in the present day civil life ?
   (*b*) Draw the flow diagram of a typical water supply scheme using an impounded reservoir as the source of supply and show therein the different works involved.
   [*Note* : Pl. see Fig. 2.2]
2. Describe the necessity of having a planned water supply scheme for a town. How are such schemes financed, planned and executed ? Discuss with special reference to the circumstances prevailing in India.
3. Why is it necessary to control the water quality in nature and also in community ?
4. Write briefly the historical background of water treatment and water supply engineering.
5. Explain the connection between 'Public Health' and 'Engineering', and also between 'Water Supply' and 'Diseases'.
6. "There is no shortage of water in India, yet the water supplies are very poorly developed here." Discuss critically the above statement, and suggest methods for improvements.
7. Explain what is meant by 'Healthy Environment' and how is it achieved in practice ?

—2

Water Demands

Whenever an engineer is given the duty to design a water supply scheme for a particular section of the community, it becomes imperative upon him, to first of all, evaluate the amount of water *available* and the amount of water *demanded* by the public. *In fact, the first study is to consider the demand, and then the second requirement is to find sources to fulfil that demand.* Many a times, a compromise is sought between the two.

2.1. Various Types of Water Demands

While planning a water supply scheme, it is necessary to find out not only the total yearly water demand but also to assess the required average rates of flow (or draft) and the variations in these rates. The following quantities are, therefore, generally assessed and recorded :

(*i*) Total annual volume (V) in litres or million litres.

(*ii*) Annual average rate of draft in litres per day, *i.e.* $V/365$.

(*iii*) Annual average rate of draft in litres per day per person (*i.e.* litres per capita per day or lpcd), called ***per capita demand (q).***

(*iv*) Average rate of draft in litres per day per service, *i.e.*

$$\frac{V}{365} \times \frac{1}{\text{No. of services}}$$

(*v*) Fluctuations in flows expressed in terms of percentage ratios of maximum or minimum yearly, monthly, daily or hourly rates to their corresponding average values.

It is very difficult to precisely assess the quantity of water demanded by the public, since there are many variable factors affecting water consumption. Certain thumb rules and empirical formulas are, therefore, generally used to assess this quantity, which may give fairly accurate results. The use of a particular method or a formula for a particular case has, therefore, to be decided by the intelligence and foresightedness of the designer. The various types of water demands, which a city may have, may be broken down into the following classes :

(1) *Domestic water demand* ;

(2) *Industrial water demand* ;

(3) *Institution and commercial water demand* ;

(4) *Demand for public uses* ;

(5) *Fire demand* ; and

(6) *Water required to compensate losses in wastes and thefts.*

In order to estimate, as correctly as possible, the total water demand of a particular section of the community, all these demands must be considered

and suitable provision made, depending upon the needs of those people for whom the water supply scheme is to be designed. These demands are described below :

2.1.1. Domestic Water Demand. This includes the water required in residential buildings for drinking, cooking, bathing, lawn sprinkling, gardening, sanitary purposes, etc. The amount of domestic water consumption per person shall vary according to the living conditions of the consumers. As per IS : 1172-1993*, the minimum domestic consumption for a town or a city with full flushing system should be taken at 200 l/h/d ; although it can be reduced to 135 l/h/d for economically weaker sections and LIG colonies depending upon prevailing conditions. The break up of 200 l/h/d and of 135 l/h/d are given in table 2.1 and table 2.2, respectively.

Table 2.1. Minimum Domestic Water Consumption (Annual Average) for Indian Towns and Cities with Full Flushing Systems as per IS : 1172-1993

Use	*Consumption in litres per head per day (l/h/d)*
Drinking	5
Cooking	5
Bathing	75
Washing of clothes	25
Washing of utensils	15
Washing and cleaning of houses and residences	15
Lawn watering and gardening	15
Flushing of water closets, etc.	45
Total	200

Table 2.2. Minimum Domestic Water Consumption (Annual Average) for Weaker Sections and LIG Colonies in Small Indian Towns and Cities

Use	*Consumption in l/h/d*
Drinking	5
Cooking	5
Bathing	55
Washing of clothes	20
Washing of utensils	10
Washing and cleaning of houses and residences	10
Flushing of water closets, etc.	30
Total	135

*IS Code on "Basic Requirements of Water Supply, Drainage and Sanitatation".

The IS code infact lays down a limit on domestic water consumption between 135—225 l/h/d (with 200 l/h/d being minimum under ordinary circumstances with flushing system).

In a developed and an effluent country like U.S.A., this figure usually goes as high as 340 l/h/d. This is because more water is consumed in rich living in air-cooling, bathing in bath-tubs, dish washing of utensils, car washing, home laundries, garbage grinders, etc.

The total domestic water consumption usually amounts to 50—60% of the total water consumption.

The total domestic water demand shall be equal to the total design population multiplied by per capita domestic consumption. The methods to compute the design population (*i.e.* the expected population at the end of design period), are discussed in articles 2.7, 2.8 and 2.9.

2.1.2. Industrial Water Demand. The 'industrial water demand', represents the water demand of industries, which are either existing or are likely to be started in future, in the city for which water supply is being planned. This quantity will thus vary with the number and types of industries present in the city. *The ordinary per capita consumption on account of industrial needs of a city is generally taken as 50 litres / person / day*, which may suffice only to meet the water demand of small scattered industries, without catering to larger industries. Separate provision will have to be made to meet the water demand of such specific industries. Their requirement will have to be approximated on the basis of the nature and magnitude of each industry, and the quantity of water required per unit of production. The potential for industrial expansion should also be investigated, so that the availability of water supply may attract such industries, and add to economic prosperity of the community. Some of the industries may develop their own water supplies, and may place a very little or virtually no demand on the public supplies.

In industrial cities, the per capita water requirement may finally be computed to be as high as 450 litres/person/day or so, as compared to the normal industrial requirement of 50 litres/person/day.

The approximate quantities of water required by different industries per unit of their production, are shown in table 2.3, which may serve as a rough guide in estimating the total industrial demand.

2.1.3. Institutional and Commercial Water Demand. The water requirements of institutions, such as hospitals, hotels, restaurants, schools and colleges, railway stations, offices, factories, etc. should also be assessed and provided for, in addition to *domestic* and *industrial* water demands, discussed above. This quantity will certainly vary with the nature of the city and with the number and types of commercial establishments and institutions present in it. *On an average, a per capita demand of 20 litres / head / day is usually considered to be enough to meet such commercial and institutional water requirements, although of course, this demand may be as*

Table 2.3. Water Demand of Certain Important Industries*

S. No.	*Name of industry and product*	*Unit of production or raw material used*	*Approximate quantity of water required per unit of production / raw material in kilo litres*
1.	Automobiles	Vehicle	40
2.	Distillery (Alcohol)	kilo litre	122—170
3.	Fertilizer	Tonne	80—200
4.	Leather (Tanned)	Tonne	40
5.	Paper	Tonne	200—400
6.	Special Quality Paper	Tonne	400—1000
7.	Straw board	Tonne	75—100
8.	Petroleum Refinery	Tonne (Crude)	1—2
9.	Steel	Tonne	200—250
10.	Sugar	Tonne (crusted cane)	1—2
11.	Textile	Tonne (goods)	80—140

high as 50 l/h/d for highly commercialised cities. The individual approximate water requirements of such institutions/commercial units, are shown in table 2.4, and these values may be used for better assessment of their water needs.

Table 2.4. Water requirements of Individual Institutions and Commercial Establishments

S. No.	*Type of Institution or Commercial Establishment*	*Average water consumption in litres / head / day*
1.	*Offices*	45—90
2.	*Factories*	
	(*a*) where bath rooms are provided	45—90
	(*b*) where no bath rooms are provided	30—60
3.	*Schools*	
	(*a*) day scholars	45—90
	(*b*) residential	135—225
4.	*Hostels*	135—180
5.	*Hotels*	180 (per bed)
6.	*Restaurants*	70 (per seat)
7.	*Hospitals* (including laundry)	
	(*a*) number of beds not exceeding 100	340 (per bed)
	(*b*) number of beds exceeding 100	450 (per bed)
8.	*Nurses homes and medical quarters*	135—225

(*Contd.*)

*Due to continuous modernisation and reuse of water in industrial processes, the industrial water requirements are getting reduced than those shown in table 2.3.

S. No.	Type of Institution or Commercial Establishment	Average water consumption in litres/head/day
9.	*Railway Stations*	
	(*a*) junctions and intermediate stations where mail and express trains stop	70 (with bathing facilities) 45 (without bathing facilities)
	(*b*) intermediate stations where mail and express trains do not stop	45 (with bathing facilities) 23 (without bathing facilities)
	(*c*) terminal railway stations	45
10.	*Airports—International and domestic*	70
11.	*Cinema Halls and Theatres* (per seat)	15

2.1.4. Demand for Public Uses. This includes the quantity of water required for public utility purposes, such as watering of public parks, gardening, washing and sprinkling on roads, use in public fountains, etc. For most of the water supply schemes in India, these needs are not believed as essentials, and a nominal amount not exceeding 5% of the total consumption may be added to meet this demand on an arbitrary basis. A figure of 10 l/h/d is usually added on this account, while computing total water requirement, as shown in Table 2.5.

2.1.5. Fire Demand. In thickly populated and industrial areas, fires generally break out and may lead to serious damages, if not controlled effectively. Big cities, therefore, generally maintain full fire-fighting squads. Fire-fighting personnel require sufficient quantity of water, so as to throw it over the fire at high speeds. A provision should, therefore, be made in modern public water supply schemes for fighting fires. The quantity of water required for extinguishing fires should be easily available and kept always stored in storage reservoirs. Fire hydrants are usually fitted in the water mains at about 100 to 150 metres apart, and fire-fighting pumps are immediately connected into them by the fire brigade personnel, as soon as a fire breaks out. These pumps then throw water on the fire at very high pressures, so as to bring it under control. The minimum water pressure available at fire hydrants should be of the order of 100 to 150 kN/m^2 (10 to 15 m of water head) and should be maintained even after 4 to 5 hours of constant use of fire hydrant.

Generally, in a moderate fire break out, three *jet streams* are simultaneously thrown from each hydrant ; one on the burning property, and one each on adjacent property on either side of the burning property. The discharge of each stream should be about 1100 litres/minute. Hence, in a big city having a population of say 50 lakhs, if six fires break out in a day and each fire stands for 3 hours, the total amount of water required shall be given as :

$$= 6 \times (3 \times 1100) \times (3 \times 60)$$

[*i.e.* No. of fires × Discharge × Time of each fire]

$$= 35,64,000 \text{ litres/day}$$

The amount of water required per person

$$= \frac{35,64,000}{50 \text{ lakhs}} = \frac{35,64,000}{50,00,000} < 1 \text{ litre/person/day}$$

From the above example, it therefore, follows that although the rate at which water is required for fire-fighting is very large, the total amount of water consumption hardly amounts to 1 litre/head/day. *The per capita fire demand is thus generally ignored while computing the total per capita water requirement of a city.* However, for cities having populations exceeding 50,000, the water required in kilo litres may be computed by using the relation

$$\text{Kilo litre of water required} = 100\sqrt{P} \qquad ...(2.1)$$

where P is population in thousands.

The high rate of water consumption during a fire considerably affects the design of *distribution system*, and hence while designing public water supply schemes, the **rate of fire demand** is sometimes treated as a function of population, and is worked out on the basis of certain empirical formulas, which are given below :

(1) **Kuichling's formula.** It states that

$$Q = 3{,}182\sqrt{P} \qquad ...(2.2)$$

where Q = Amount of water required in litres/minute

P = Population in thousands.

(2) **Freeman formula.** It states that

$$Q = 1136\left[\frac{P}{10} + 10\right] \qquad ...(2.3)$$

where Q and P have the same meaning as in Eq. (2.2).

(3) **National Board of Fire Under Writers Formulas.** According to the recommendations given by the Board (now known as American Insurance Association), the fire requirements are as follows :

(*a*) ***For a central congested high valued city*** **:**

(*i*) When population is less than or equal to 2,00,000

$$Q = 4{,}637\sqrt{P}\,[1 - 0.01\sqrt{P}] \qquad ...(2.4)$$

(*ii*) When population is more than 2 lakhs, a provision for 54,600 litres/minute may be made with an extra additional provision of 9,100 to 36,400 litres/minute for a second fire.

(*b*) ***For a residential city :***

The required draft for fire-fighting may be as follows :

(*i*) Small or low buildings = 2,200 litres/minute.

(*ii*) Larger or higher buildings = 4,500 litres/minute.

(*iii*) High value residences, apartments, tenements = 7,650 to 13,500 litres/minute.

(*iv*) Three storeyed buildings in densely built-up sections = upto 27,000 litres/minute.

(4) **Buston's formula.** It states that

$$Q = 5{,}663\sqrt{P} \qquad ...(2.5)$$

where P and Q have the same meaning as given above.

The formulas at (2), (3) and (4) above, give higher results and, therefore, only 2 hours storage with these rates may be assumed as fairly good allowance, while working out the total stand-by storage capacity or the volume of water required for fire demand, as for as Indian conditions are concerned ; although 5-10 hr storage is considered as minimum requirement in USA.

All the above formulas suffer from the drawback that they are not related to the type of district served. These formulas, therefore, give equal results for industrial as well as non-industrial areas, although the possibility of occurrence of a fire with a given duration is more for an industrial area as compared to a non-industrial area.

This probability of occurrence of a fire, which, inturn, depends upon the type of the city served, has been taken into consideration in developing a formula (eq. 2.6), on the basis of actual water consumption in fire fighting for Jabalpur city of India. The formula is given as :

$$Q = \frac{4360\, R^{0.275}}{(t+12)^{0.757}} \qquad ...(2.6)$$

where R = Recurrence interval of fire, *i.e.* period of occurrence of fire in years, which will be different for residential, commercial and industrial cities. Its recommended minimum value is 1 year.

t = Duration of fire in minutes, Its recommended min. value is 30 min.

Thus, if fire fighting provision is to be made for fighting a fire occurring once in 3 years, and lasting for 2 hours, then

$$Q = \frac{4360\,(3)^{0.275}}{(2\times 60+12)^{0.757}}\ l/\text{min} = 146.4\ l/\text{min}.$$

2.1.6. Water Required to Compensate Losses in Thefts and Wastes. This includes the water lost in leakage due to bad plumbing or damaged meters, stolen water due to unauthorised water connections, and other losses and wastes. These losses should be taken into account while estimating the total requirements. These losses can be reduced by careful maintenance and universal metering. Even in the best managed water works, this amount may, however, work to be as high as 15% of the total consumption.

2.2. Total Requirement of Water For a Town or a City

The ***annual average daily draft**** in litres/day required by a town can be worked out by multiplying the probable number of the persons who are going to use the facilities provided by the scheme, and the annual average daily consumption of each person (for all uses) called ***per capita demand*** (q).

This estimated quantity of water required for municipal uses for which the water supply scheme has to be designed, should be worked out with due provision for the estimated requirements of the future. The *future period* for

*Total annual draft divided by 365.

which a provision is made in the water supply scheme is known as *the* **design period**. Say for example, let the population of a certain area be 1,00,000 in 1986 ; but in the year 2016, it may rise to 2,00,000. In order to ensure a reliable water supply even after 30 years (*i.e.* up to 2016) it is, therefore, necessary, that the scheme be designed for a population of 2,00,000 and not for 1,00,000. But at the same time, it is also possible that the population of that particular region or town may not increase at all. In that case, it will be very unwise to design the scheme for a higher population, because it will result in unnecessary heavy investments and financial burdens. It is, therefore, necessary to estimate *this* ***future population*** as accurately as possible. Depending upon the possibilities of future development of the town, the possible future population of the region just by the end of the design period (say y years) is generally estimated, using different methods, such as :

(*i*) *Arithmetical increase method* ;
(*ii*) *Geometrical increase method* ;
(*iii*) *Incremental increase method* ;
(*iv*) *Decreasing rate method* ;
(*v*) *Simple graphical method* ;
(*vi*) *Comparative graphical method* ;
(*vii*) *Master plan method* ;
(*viii*) *The apportionment method* ; and
(*ix*) *The logistic curve method.*

The *design period* chosen, should neither be so long as to throw unnecessary extra financial burden on the present generation, nor it should be so short as to make the design insufficient for future needs. These aspects are discussed under **'Population Forecasts'** a little later, under articles 2.8 to 2.10.

2.3. The Per Capita Demand (q)

It is the annual average amount of daily water required by one person, and includes the domestic use, industrial and commercial use, public use, wastes, thefts, etc. It may, therefore, be expressed as

Per Capita Demand (q) in litres per day per head

$$= \frac{\text{Total yearly water requirement of the city in litres } (i.e.\ V)}{365 \times \text{Design population}} \quad ...(2.7)$$

Total yearly water requirement of the city can, therefore, be worked out by using Eq. (2.7), provided the per capita demand is known or assumed. The per capita demand depends upon various factors (enumerated in the next article) and will vary according to the living conditions of the consumers and also with the extent and type of industries developed or likely to be developed in that region. For an average Indian city, as per recommendations of I.S. code, the per capita demand (q) may be taken as given in table 2.5.

This figure of 335 litres/head/day when multiplied by the prospective population at the end of the design period, shall give the annual average water requirement of the city per day. When multiplied by 365, it will give the volume of the yearly water requirement in litres.

Table 2.5. Break up of Per Capita Demand (q) for an Average Indian City

Use	*Demand in l/h/d*
(*i*) Domestic use	200
(*ii*) Industrial use	50
(*iii*) Commercial use	20
(*iv*) Civic or public use	10
(*v*) Wastes and thefts, etc.	55
	Total = 335 = Per Capita Demand (*q*)

2.4. Factors Affecting Per Capita Demand

The annual average demand for water (*i.e.* per capita demand) considerably varies for different towns or cities. This figure generally ranges between 100 to 360 litres/capita/day for Indian conditions. These variations in total water consumption of different cities or towns depend upon various factors, which must be thoroughly studied and analysed before fixing the per capita demand for design purposes. These factors are discussed below :

(1) Size of the City. The per capita demand for big cities is generally large as compared to that for smaller towns. This is because of the fact that in big cities, huge quantities of water are required for maintaining clean and healthy environments. For example, big cities are generally sewered, and as such require large quantities of water (a sewered house requires four to five times the water required by an unsewered home). Similarly, in a big city, commercial and industrial activities are generally more, thus requiring more water. Affluent rich living in air cooled homes may also increase the water consumption in cities.

In fact, the effect of population on the size of the city is an indirect one, because even a smaller town may have high water consumption, if it is fully industrialised or is having some industry requiring tremendous quantities of water, or if rich affluent people are living in it. On an average, the per capita demand for Indian towns may vary with the population, as shown in Table 2.6.

(2) Climatic Conditions. At hotter and dry places, the consumption of water is generally more, because more of bathing, cleaning, air cooling, sprinkling in lawns, gardens, roofs, etc., are involved. Similarly, in extremely cold countries, more water may be consumed, because the people may keep their taps open to avoid freezing of pipes, and there may be more leakage from pipe joints, since metals contract with cold.

(3) Types of Gentry and Habits of People. Rich and upper class communities generally consume more water due to their affluent living standards. Middle class communities consume average amounts, while the poor slum dwellers consume very low amounts. The amount of water consumption is thus directly dependent upon the economic status of the consumers.

Table 2.6. Variations in Per Capita Demand (q) with Population in India

S. No.	*Population*	*Per Capita Demand in litres/day/person**
1.	Less than 20,000	110
2.	20,000—50,000	110—150
3.	50,000—2 lakhs	150—240
4.	2 lakhs—5 lakhs	240—275
5.	5 lakhs—10 lakhs	275—335
6.	Over 10 lakhs	335**—360

(4) Industrial and Commercial Activities. The pressure of industrial and commercial activities at a particular place increases the water consumption by large amounts. Many industries require really huge amounts of water (much more than the domestic demand), and as such, increase the water demand considerably. As pointed out earlier, the industrial water demand is having no direct connection with the population or the size of the city, but more industries are generally situated in big cities, thereby increasing the per capita demand for big cities. However, for a properly planned zoned city, the water requirement can be more accurately predicted by estimating the industrial and commercial demands separately.

(5) Quality of Water Supplies. If the quality and taste of the supplied water is good, it will be consumed more, because in that case, people will not use other sources such as private wells, hand pumps, etc. Similarly, certain industries such as boiler feeds, etc., which require standard quality waters will not develop their own supplies and will use public supplies, provided the supplied water is up to their required standards.

(6) Pressure in the Distribution System. If the pressure in the distribution pipes is high and sufficient to make the water reach at 3rd or even 4th storey, water consumption shall definitely be more. This water consumption increases because of two reasons :

(*i*) People living in upper storeys will use water freely as compared to the case when water is available scarcely to them.

(*ii*) The losses and wastes due to leakage are considerably increased if this pressure is high. For example, if the pressure increases from 20 m head of water, (*i.e.* 200 kN/m^2) to 30 m head of water (*i.e.* 300 kN/m^2), the losses may go up by 20 to 30 per cent.

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*The figures shown here are liable to variation upto about 25%.

**Break up given in table 2.5, by taking domestic demand as 200 l/h/d, which may go as high as 225 l/h/d, thereby raising the total demand to 360 l/c/d. As against the ideal per capita water requirement of about 335—360 l/c/d for cities exceeding 10 lakh in population, the present water consumption of various cities are approximated as follows :

Delhi = 244 l/c/d ; **Bombay = 296 l/c/d** *; Kolkatta = 226 l/c/d ; Chennai (Madras) = 115 l/c/d ; Lucknow = 253 l/c/d ; Bhopal = 271 l/c/d ; Hongkong = 168 l/c/d ; Singapur = 111 l/c/d ; Rochester (New York) = 450 l/c/d ; Syracuse (New York) = 728 l/c/d ; Camden (New Jealand) = 640 l/c/d ; Passaic (New Jealand) = 807 l/c/d ; South Gate (California) = 550 l/c/d, St. Cloud Minn (USA) = 271 l/c/d ; etc.*

(7) Development of Sewerage Facilities. As pointed out earlier, the water consumption will be more, if the city is provided with 'flush system' and shall be less if the old 'conservation system' of latrines is adopted.

(8) System of Supply. The water may be supplied either continuously for all the 24 hours of the day, or may be supplied only for peak periods during the morning and evening. The second system, *i.e.* the intermittent supplies, may lead to some saving in water consumption due to losses occurring for lesser time and a more vigilant use of water by the consumers. But at many places, the intermittent supplies may not give much saving over the continuous supplies, because of the following reasons :

(*i*) In intermittent supply system, water is generally stored by consumers in tanks, drums, utensils, etc., for non-supply periods. This water is thrown away by them even if unutilised as soon as the fresh supply is restored. This increases the wastage and losses considerably.

(*ii*) People have a general tendency to keep the taps open during non-supply hours, so that they may come to know of it as soon as the supply is restored. Many a times, water goes on flowing unattended even after the supply is restored, thus resulting in wastage of water.

(9) Cost of Water. If the water rates are high, lesser quantity may be consumed by the people. This may not lead to large savings as the affluent and rich people are little affected by such policies.

(10) Policy of Metering and Method of Charging. Water tax is generally charged in two different ways :

(*a*) On the basis of meter reading (meters fitted at the head of the individual house connections and recording the volume of water consumed).

(*b*) On the basis of certain fixed monthly flat rate.

In the 2nd case, *i.e.* when the supplies are unmetered and the charges are fixed, people generally do not practice economy in the use of water, because they think that they have to pay only a fixed amount irrespective of the quantity of water used by them. They are, therefore, generally liberal in consuming water, and many a times, leave their taps flowing unused. All this leads to high wastage and high consumption of water. However, when the supplies are metered, people use only that much of water as much is required by them. Although metered supplies are preferred because of lesser wastage, they generally lead to lesser water consumption by poor and low income groups, leading to unhygienic conditions. Moreover, meters put unnecessary hinderance to the flow, resulting in loss of pressure and increased cost of pumping. Meters are also liable to be stolen and the cost of installing, repairing and reading the meters is generally high. In a good water works management, it is, therefore, generally desired to work out the economics of metering and to balance the cost of meterage against the value of water conserved by reduction of wastages. At certain places, the saving in water due to meterage has really been large enough, as to permit postponing the otherwise needed extensions of supplies.

25. Factors Affecting Losses and Wastes

As pointed out earlier, enormous amount of good water is lost in leakages, wastes, thefts, etc., in every water supply scheme. In order to control these

losses and to reduce them to minimum, it is desirable to thoroughly study the factors on which these losses depend. The various factors on which these losses depend and the measures to control them are summarised below :

(1) **Water Tight Joints.** The joints in the water mains and pipes generally leak due to bad plumbing, leading to high wastage of water. The leakage of water can be reduced by careful and better plumbing with constant maintenance, and thereby keeping the joints water tight. The meters when installed in individual house connections should also be periodically checked, so as to ensure that they do not leak.

(2) **Pressure in the Distribution System.** As pointed out earlier, high pressure in the distribution pipes leads to higher leakage losses. So much so that an increase in pressure from 20 m head of water to 30 m head of water may increase the leakage by about 20 to 30 per cent. Hence, in order to keep the leakage less, it is desirable to keep the pressure in the pipes to a minimum possible value (say upto two or at the most three storey height). Multistoreyed buildings may be encouraged to develop their own storage.

(3) **System of Supply.** In intermittent system of supplies, the leakage losses are reduced since they do not occur for all the 24 hours but occur only for a few hours during the period the supply is restored. However, it cannot be said with certainty that intermittent supply reduces losses, because in that case, people generally waste more water by throwing the old stored water and also by leaving their taps open, as explained earlier.

(4) **Metering.** When the supplies are metered, wastage is considerably reduced, because people become more careful in using water, as they have to pay for the volume of water consumed by them.

(5) **Unauthorised Connections.** Water is sometimes stolen through unauthorised connections, which must be detected and checked by heavily punishing the defaulters, so as to keep the losses to minimum.

VARIATIONS IN DEMAND (OR DRAFT) AND THEIR EFFECTS ON THE DESIGN OF VARIOUS COMPONENTS OF A WATER SUPPLY SCHEME

2.6. Variations in Demand

The per capita demand (q), so far discussed, has been based upon the annual consumption of water. It was, therefore, defined as the ***annual average daily consumption*** per person. As pointed out in the beginning of this chapter, this annual average demand is not sufficient, although very useful, for the design of various components of a water supply scheme. There are wide variations in the use of water in different seasons, in different months of the year, in different days of the month, in different hours of the day, and even in different minutes of the hour.

Seasonal variations occur due to larger use of water in summer season, lesser use in winter, and much less in rainy season. These variations may also be caused by seasonal use of water in industries such as processing of cash crops at the time of harvesting, etc. Day to day variations (called ***daily variations***) reflect household and industrial activity. For example, the water

consumption is generally more on Sundays and holidays, on days of dust storms, etc. Again there are variations in hour to hour demand (called ***hourly variations***). For example, the consumption in the early hours of morning (0 to 6 hours—say) is generally small, increases sharply as the day advances, reaching a peak value between about 8 to 11 AM, then decreases sharply upto about 1 PM, remains constant upto about 4 PM, again increases in the evening reaching a peak between 7 to 9 PM, finally falling to a low value in the late hours of night, as shown in Fig. 2.1. The night flow (excluding industrial consumption) generally represent the magnitude of losses and wastes, since there is no appreciable domestic consumption during this time.

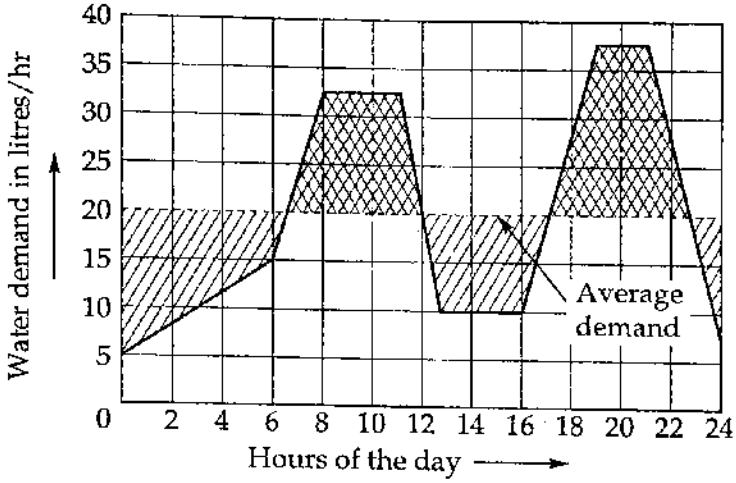


Fig. 2.1. Showing hourly rate of water consumption.

These ***normal variations in the demand*** or draft should generally be assessed and known in order to design supply pipes, service reservoirs, distributary pipes, etc. Moreover, a suitable allowance must also be made for sudden and heavy drafts required for fire-fighting.

2.6.1. Assessment of Normal Variations. The smaller the town, the more variable is the demand. The shorter the period of draft, the greater is the departure from the mean. The maximum demands (monthly, daily or hourly) are generally expressed as ratios of their means. These ratios may vary considerably for different communities. The following figures are generally adopted :

(*i*) *Maximum daily consumption* is generally taken as 180 per cent of the average daily consumption

Therefore, Maximum daily demand

$$= 1.8 \text{ (}i.e.\text{ 180 per cent)} \times \text{Average daily demand}^*$$

$$= 1.8\, q \qquad \ldots(2.8)$$

(*ii*) *Maximum hourly consumption* is generally taken as 150 per cent of its average hourly consumption.

Therefore *Maximum hourly consumption of the maximum day i.e.*, **Peak demand**

*This average is done over an year, and hence the average daily demand means *annual* average daily demand.

= 1.5 (*i.e.* 150 per cent) × Average hourly consumption of the max. day

$$= 1.5 \times \left(\frac{\text{Maximum daily demand}}{24}\right)$$

$$= 1.5 \times \left(\frac{1.8 \times q}{24}\right) = 2.7\left(\frac{q}{24}\right)$$

= 2.7 (Annual average hourly demand) ...(2.9)

Many a times, the following formula given by Goodrich is also used for finding out the ratios of peak demand rates to their corresponding means :

$$p = 180.t^{-0.10} \quad ...(2.10)$$

where p = Per cent of the annual average draft for the time t in days

t = Time in days from $\frac{1}{24}$ to 365.

When t = 1 day (for daily variations)

$$p = 180 \times (1)^{-0.10} = 180$$

$$\therefore \quad \frac{\text{Max. daily demand}}{\text{Average daily demand}} = 180\% \quad ...(2.11)$$

When t = 7 days (for weekly variations)

$$p = 180 \times (7)^{-0.10} = 148\%$$

$$\therefore \quad \frac{\text{Max. weekly demand}}{\text{Average weekly demand}} = 148\% \quad ...(2.12)$$

When t = 30 days (for monthly variations)

$$p = 180\ (30)^{-0.10} = 128\%$$

$$\therefore \quad \frac{\text{Max. monthly demand}}{\text{Average monthly demand}} = 128\% \quad ...(2.13)$$

The GOI Manual on water supply has recommended the following values of the **Peak factor**, depending upon the population :

Table 2.7. Peak Factors

S. No.	*Population / etc.*	*Peak factor**
1. (*i*)	Up to 50,000	3.0
(*ii*)	50,001—2,00,000	2.5
(*iii*)	Above 2 lakh	2.0
2.	For Rural water supply schemes, where supply is effected through standpost for only 6 hours	3.0

Evidently, the peak factor tends to reduce with increasing population, since the different habits and customs of several groups in larger population, tend to minimise the variation in demand pattern.

*As against the general value of 2.7, suggested above.

2.6.2. Coincident Draft. It is extremely improbable that a fire may break out when water is being drawn by the consumers at maximum hourly draft. Therefore, for general community purposes, the *total draft* is not taken as the sum of maximum hourly demand and fire demand, but is taken as the sum of maximum daily demand and fire demand, or the maximum hourly demand, whichever is more. The maximum daily demand (*i.e.* 1.8 times the average daily demand) when added to fire draft for working out total draft, is known as *coincident draft*.

2.7. Effects of Variations in Demand on the Design Capacities of Different Components of a Water Supply Scheme

Two simple sketches showing the possible layouts of water supply schemes are given in Fig. 2.2. In Fig. 2.2 (*a*), a dam and a reservoir are used, and thus, there is generally no need of pumps ; while in Fig. 2.2 (*b*), a well or a river is used as the source, needing pumping equipment.

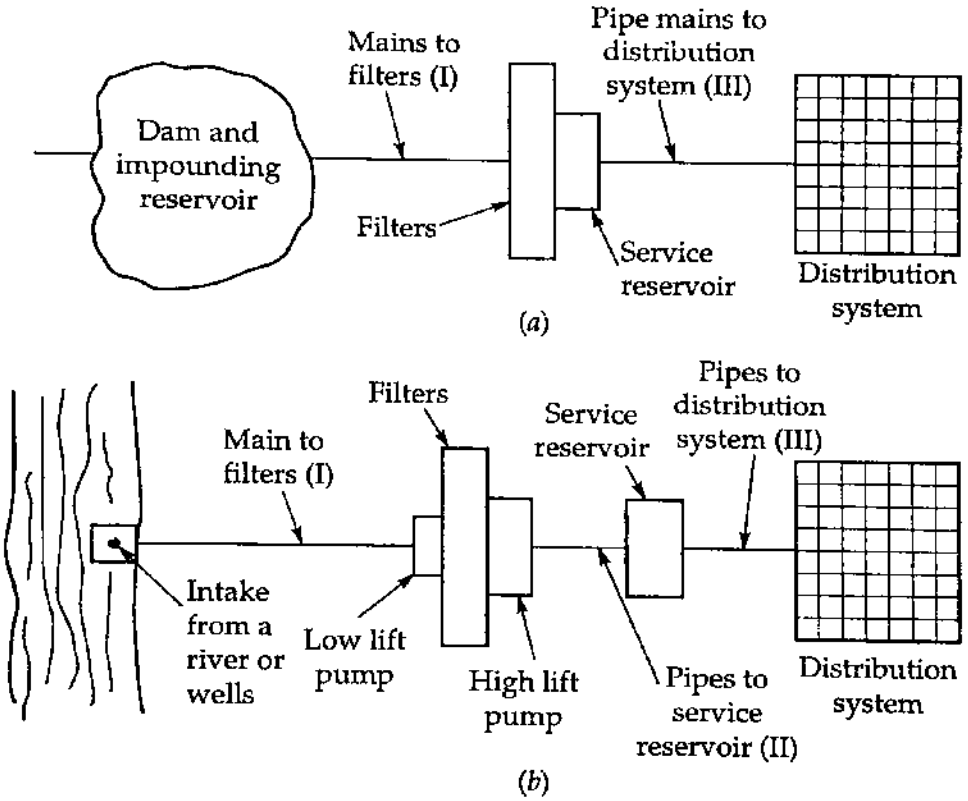


Fig. 2.2. Layouts of water supply schemes.

The various units involved in such schemes should be designed not only to serve the average daily demand but to serve the maximum demand as and when it arises and also the variations in the demand. The following recommendations may be adopted for designing the capacities of different components.

(1) The **sources of supply** such as wells, etc., may be designed for *maximum daily* consumption or sometimes for average daily consumption.

(2) The **pipe mains** (type I and II, Fig. 2.2) taking the water from the source upto the service reservoir may be designed for *maximum daily consumption*.

(3) The **filter and other units** at water treatment plant may also be designed for maximum daily draft. Sometimes, an additional provision for reserve is also made for break-downs and repairs. Therefore, they may be designed for *twice the average daily* instead of 1.8 times the average daily.

(4) The **pumps** lifting the water may be designed for *maximum daily draft* plus some additional reserve for break-downs and repairs ; say, for *twice the average daily* instead of 1.8 times the average daily.

Note. When the pumps do not work for all the 24 hours, such as in small town supplies, the design draft should be multiplied by

$$\left[\frac{24}{\text{No. of hours in the day for which the pumps are running}}\right].$$

(5) The **distribution system** (including the pipes carrying water from service reservoir to distribution system (*i.e.* Type III, Fig. 2.2) should be designed for *maximum hourly draft of the maximum day or coincident draft* with fire, whichever is more. Generally, no provision for reserve is made.

(6) The **service reservoir** is designed to take care of the hourly fluctuations, fire demands, emergency reserve, and the provision required when pumps have to pump the entire-day's water in fewer hours than 24 hours. Only 2 hours storage may be considered for fire allowance as sufficient. Ordinarily, the required storage approximates a day's consumption.

Example 2.1. *A water supply scheme has to be designed for a city having a population of 1,00,000. Estimate the important kinds of drafts which may be required to be recorded for an average water consumption of 250 lpcd. Also record the required capacities of the major components of the proposed water works system for the city using a river as the source of supply. Assume suitable figures and data where needed.*

Solution. (*i*) *Average daily draft*

$$= \begin{pmatrix}\text{Per capita average consumption}\\ \text{in litres/person/day}\end{pmatrix} \times \text{population}$$

$$= 250 \times 1{,}00{,}000 \text{ litres/day} = 250 \times 10^5 \text{ litres/day}$$

$$= 25 \text{ million litres/day} = 25 \text{ Mld.} \quad \textbf{Ans.}$$

(*ii*) *Maximum daily draft.* May be assumed as 180% of annual average daily draft (see Eq. 2.7).

$\therefore$ Maximum daily draft

$$= \frac{180}{100}\,[25 \text{ Mld}] = 45 \text{ Mld.} \quad \textbf{Ans.}$$

(*iii*) *Maximum hourly draft of the maximum day.* It may be assumed as 270 per cent of annual average hourly draft (see Eq. 2.8).

$\therefore$ Maximum hourly draft of maximum day

$$= \frac{270}{100}\,[25 \text{ Mld}] = 67.5 \text{ Mld.} \quad \textbf{Ans.}$$

(*iv*) *Fire flow.* May be worked out from Eq. (2.4), *i.e.*

$$Q = 4{,}637\sqrt{P}\left[1 - 0.01\sqrt{P}\right]$$

Using population in thousands as equal to $P = 100$ thousands (*i.e.* 1 lakh), we get

$$Q = 4637\sqrt{100}\left[1 - 0.01\sqrt{100}\right]$$

$$= 4637\,(10)\,(1 - 0.1) = 41733 \text{ litres/minute}$$

$$= \frac{41{,}733 \times 60 \times 24}{10^6} \text{ million litres/day}$$

$$= 61 \text{ Mld.} \quad \textbf{Ans.}$$

Coincident draft = Max. daily draft + Fire draft

$$= 45 + 61 = 106 \text{ Mld.} \quad \textbf{Ans.}$$

(which is greater than the max. hourly draft of 67.5 Mld)

Note. It shows that the distribution system has to be designed for 106 mld instead of 67.5 mld, which proves that *the fire allowance considerably affects the design of distribution system.*

Capacities of various components in the above example

(1) *The intake structure* for fetching water from the stream may be designed for maximum daily draft, *i.e.* for 45 Mld. **Ans.**

(2) *The pipe mains* carrying the water from the intake to the treatment plant and then to service reservoir may be designed for maximum daily draft.

∴ Required capacity = Maximum daily draft = 45 Mld. **Ans.**

(3) *The filters and other units* at the treatment plant may be designed for maximum daily demand plus some reserve (say equal to twice the average instead of 1.8 times the average).

∴ Required capacity = 2 × 25 Mld = 50 Mld. **Ans.**

(4) The *lift pumps* may be designed for twice the average daily

$$= 2 \times 25 = 50 \text{ Mld.} \quad \textbf{Ans.}$$

Note. If the pumps are not operated for all the 24 hours, say they are operated for 8 hours, then the design draft

$$= \frac{24}{8} \times \text{Normal design draft for pumps}$$

$$= \frac{24}{8} \times 50 = 3 \times 50 = \textbf{150 Mld.}$$

In this case, the capacity of service reservoir will also go up.

(5) The *distribution system* including the pipes carrying water from service reservoir to the distribution system may be designed for *coincident draft with fire* or *maximum hourly*, whichever is more.

∴ The required capacity = 106 Mld. **Ans.**

DESIGN PERIODS AND POPULATION FORECASTS

2.8. Design Periods

A water supply scheme includes huge and costly structures (such as dams, reservoirs, treatment works, penstock pipes, etc.) which cannot be replaced or increased in their capacities, easily and conveniently. For example, the water mains including the distributing pipes are laid underground, and cannot be

replaced or added easily, without digging the roads or disrupting the traffic. In order to avoid these future complications of expansions, the various components of a water supply scheme are purposely made larger, so as to satisfy the community needs for a *reasonable number of years* to come. This future period or the number of years for which a provision is made in designing the capacities of the various components of the water supply scheme is known as ***design period***. Such a scheme which is designed for a design period of say y years is supposed to satisfactorily serve the community needs up to the end of y years.

The design period should neither be *too long* nor should it be *too short*. The design period cannot exceed the useful life of the component structure, and is guided by the following considerations :

2.8.1. Factors Governing the Design Period

(*i*) Useful life of component structures and the chances of their becoming old and obsolete. Design periods should not exceed those respective values.

(*ii*) Ease and difficulty that is likely to be faced in expansions, if undertaken at future dates. For example, more difficult expansions mean choosing a higher value of the design period.

(*iii*) Amount and availability of additional investment likely to be incurred for additional provisions. For example, if the funds are not available, one has to keep a smaller design period.

(*iv*) The rate of interest on the borrowings and the additional money invested. For example, if the interest rate is small, a higher value of the design period may be economically justified and, therefore, adopted.

(*v*) Anticipated rate of population growth, including possible shifts in communities, industries and commercial establishments. For example, if the rate of increase of population is less, a higher figure for the design period may be chosen.

2.8.2. Design Period Values. Water supply projects, under normal circumstances, may be designed for a *design period of 30 years*. This 30 years period is to be counted after the completion of the project. Hence, a completion time of about 2 years may also be added to this design period. The 30 year period may, however, be modified in regard to certain components of the project, depending upon : (*i*) the useful life of the component facility, (*ii*) ease in carrying out extensions when required, and (*iii*) rate of interest so that expenditure, far ahead of utility, is avoided. Necessary land for future expansion/addition of components should, however, be acquired at the start of the project. Expansive components like tunnels and large aqueducts, entailing large capital outlays for duplication, may however, be designed for ultimate project requirements.

The design period recommended by the GoI Manual on water supply, for designing the various components of a water supply project, are given in table 2.8.

Table 2.8. Design Periods for Different Components of a Water Supply Scheme

S. No.	*Item*	*Design period in years*
1.	Storage by dams	50
2.	Intake works	30
3.	Pumping	
	(*i*) Pump house	30
	(*ii*) Electric motors and pumps	15
4.	Water treatment units	15
5.	Pipe connections to the several treatment units and other small appurtenances	30
6.	Raw water and clear water conveying units	30
7.	Clear water reservoirs at the head works, Balancing tanks, and Service reservoirs (over-head or ground level)	15
8.	Distribution system	30

2.9. Population Data and Population Growth

2.9.1. Population Data. The present population of a town or a city can be best determined by conducting an official enumeration, called *census*. The government of every country generally carries these official surveys at intervals of say about 10 years (called ***decennial census***). Sometimes, smaller period surveys are conducted by State governments or local bodies. All the useful data available from census department should, therefore, first of all, be collected by the planners of water supply schemes. The data is then used for predicting the future populations of the city at the end of the design periods. The present population, if needed, is generally determined by extending the line of the last two preceding enumerations upto the year in question.

2.9.2. Population Growth. In order to predict the future population, as correctly as possible, it is necessary to know the factors affecting population growth. There are three main factors responsible for changes in populations. They are :

(*i*) *Births* ;

(*ii*) *Deaths* ; and

(*iii*) *Migrations*.

All these factors are influenced by social and economic factors and conditions prevailing in the various communities. For example, ***birth rates*** may decrease due to excessive family planning practices and legalised abortions. Spread of education and development of extra recreational facilities for the people, also tend to reduce the birth rates.

The ***death rates*** may decrease with the development and advancement of medical facilities, thereby controlling infant mortality rates and adult death rates due to control of infections and other diseases.

The ***migrations*** are dependent upon the industrialisation and commercialisation of the particular cities or towns. People generally migrate

from villages to cities and towns, where more opportunities for earning livelihood are available. The migration rates, therefore, tend to increase sharply with the development of industries and commerce in the city or in the nearby areas. The migration rates may decrease when immigration restrictions are imposed.

Besides these three main factors, some other factors like wars, natural havocs and disasters may also bring about sharp reductions in the populations.

All these varying influences make the task of predicting future population, very very difficult and highly unexact, as it is very difficult and time consuming, especially for the engineers, to evaluate all these economic and social factors. It is, therefore, more common to rely upon mathematical formulas and graphical solutions based upon previous population records.

Ideal Population Growth curve. When all these varying influences do not produce extraordinary changes, the population would probably follow the growth curve characteristics of living things within limited space or with limited economic opportunity. The curve is S-shaped, as shown in Fig. 2.3, and is known as ***logistic curve***.

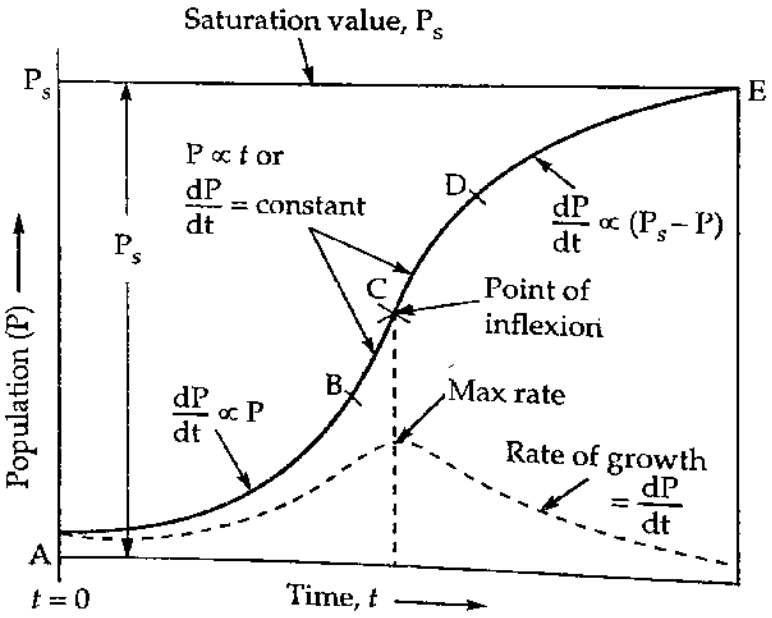


Fig. 2.3. Ideal population growth curve, called logistic curve.

The curve represents early growth AB at an increasing rate (*i.e.* geometric or log growth, $\frac{dP}{dt} \propto P$) ; and late growth DE at a decreasing rate [*i.e.* first order curve $\frac{dP}{dt} \propto (P_s - P)$] as the saturation value (P_s) is approached. The transitional middle curve BD follows an arithmetic increase $\left[i.e. \frac{dP}{dt} = \text{constant}\right]$. What the future holds for a given population, depends upon, as to where the point lies on the growth curve at a given time.

2.10. Population Forecasting Methods

The various methods which are generally adopted for estimating future populations by engineers are described below. Some of these methods are used when the design period is small, and some are used when the design period is large. The particular method to be adopted for a particular case or for a particular city depends largely upon the factors discussed in these methods, and the selection is left to the discretion and intelligence of the designer. However, as pointed out earlier, none of these methods is exact, and they are all based on the laws of probability, and thus, only approximate estimates for the possible future populations can be made.

2.10.1. Arithmetic Increase Method. This method is based upon the assumption that the population increases at a constant rate ; *i.e.* the rate of change of population with time $\left(i.e.\ \frac{dP}{dt}\right)$ is constant. Thus

$$\frac{dP}{dt} = \text{Constant} = K$$

or $$dP = K \,.\, dt$$

or $$\int_{P_1}^{P_2} dP = K \,.\, \int_{t_1}^{t_2} dt$$

or $$P_2 - P_1 = K \,.\, (t_2 - t_1)$$

where suffixes 1 and 2 represent the last and first decades or census, respectively. Thus, $t_2 - t_1$ = No. of decades.

The population data for the last 4 to 5 decades, is, therefore, obtained and the population increase per decade (x) is calculated ; the average of which ($\bar{x}$) is then used as the ***design growth rate*** for computing future population.

Thus, P_1 = Population after 1 decade from present

$$= P_0 + 1 \,.\, \bar{x}$$

P_2 = Population after 2 decades from present

$$= P_1 + 1 \,.\, \bar{x}$$

or $$P_2 = [P_0 + 2\bar{x}] \qquad ...(i)$$

Similarly $$P_3 = P_2 + 1 \,.\, \bar{x}$$

or $$P_3 = [P_0 + 3\bar{x}] \qquad ...(ii)$$

Writing in this fashion, we get

$$\boxed{P_n = [P_0 + n \,.\, \bar{x}]} \qquad ...(2.14)$$

where P_n = Prospective or forecasted population after n decades from the present (*i.e.* last known census)

P_0 = Population at present (*i.e.* last known census)

n = No. of decades between now and future.

$\bar{x}$ = Average (arithmetic mean) of population increases in the known decades.

Eqn. (2.14) represents the requisite mathematical form of this arithmetic progression method.

Example 2.2. *The population of 5 decades from 1930 to 1970 are given below in table 2.9. Find out the population after one, two and three decades beyond the last known decade, by using arithmetic increase method.*

Table 2.9

Year	*1930*	*1940*	*1950*	*1960*	*1970*
Population	*25,000*	*28,000*	*34,000*	*42,000*	*47,000*

Solution. The given data in table 2.9 is extended in table 2.10, so as to compute the increase in population (x) for each decade (col. 3), the total increase, and average increase per decade ($\bar{x}$), as shown.

Table 2.10

Year (1)	*Population* (2)	*Increase in population (x)* (3)
1930	25,000	
		3000
1940	28,000	
		6000
1950	34,000	
		8000
1960	42,000	
		5000
1970	47,000	
Total		22,000
Average increase per decade ($\bar{x}$)		$\bar{x} = \frac{22000}{4} = \mathbf{5{,}500}$

The future populations are now computed by using Eqn. (2.14) as :

$$P_n = P_0 + n \,.\, \bar{x}$$

$\therefore$ (*a*) *Population after 1 decade beyond 1970*

$$= P_{1980} = P_1 = P_{1970} + 1 \,.\, \bar{x}$$
$$= 47{,}000 + 1 \times 5500 = \mathbf{52{,}500.} \quad \textbf{Ans.}$$

(*b*) *Population after 2 decades beyond 1970*

$$= P_{1990} = P_2 = P_{1970} + 2 \,.\, \bar{x}$$
$$= 47{,}000 + 2 \times 5{,}500 = \mathbf{58{,}000.} \quad \textbf{Ans.}$$

(*c*) *Population after 3 decades beyond 1970*

$$= P_{2000} = P_3 = P_{1970} + 3 \,.\, \bar{x}$$
$$= 47{,}000 + 3 \times 5{,}500 = \mathbf{63{,}500.} \quad \textbf{Ans.}$$

2.10.2. Geometric Increase Method. In this method, the per decade percentage increase or ***percentage growth rate (r)*** is assumed to be

constant, and the increase is compounded over the existing population *every decade*. This method is, therefore, also known as **uniform increase method**.

The basic difference between arithmetic and geometric progression methods of forecasting future populations is that : *whereas, in arithmetic method, no compounding is done ; in geometric method, compounding is done every decade*. The computations in two methods are, thus, comparable to simple and compound interest computations, respectively.

Hence, if population increases from 1 lakh to 1.1 lakh in a past decade *i.e.* @ 10% ; then the population in the next decade will increase from 1.1 lakh to 1.21 lakh (*i.e.* 10% increase over the present decade population of 1.1 lakh). In simple arithmetic method, this value would have been 1.20 lakh only (*i.e.* 10% simple increase over 1 lakh for 2 decades).

Hence, the assumed *constant value of percentage growth rate per decade* (r) *is analogous to the 'rate of interest per annum'*.

The above geometric increase can be expressed as :

P_1 = Population after 1 decade

$$= P_0 + \frac{r}{100} \cdot P_0$$

where r is in per cent.

$$= P_0 \left(1 + \frac{r}{100}\right)$$

Similarly,

P_2 = Population after 2 decades

$$= P_1 + \frac{r}{100} \cdot P_1$$

$$= P_1 \left(1 + \frac{r}{100}\right) = P_0 \left(1 + \frac{r}{100}\right)^2$$

Similarly,

P_3 = Population after 3 decades

$$= P_2 + \frac{r}{100} \cdot P_2$$

$$= P_2 \left(1 + \frac{r}{100}\right) = P_0 \left(1 + \frac{r}{100}\right)^3$$

Proceeding in this way, we can write

$$\boxed{P_n = P_0 \left(1 + \frac{r}{100}\right)^n} \qquad \text{...(2.15)}$$

where P_0 = Initial population ; *i.e.* the population at the end of last known census.

P_n = Future population after n decades.

r = Assumed growth rate (%).

This **assumed growth rate** (r) can be computed in several ways from the past known population data. One method is to compute, r, as :

$$r = \sqrt[t]{\frac{P_2}{P_1}} - 1 \qquad ...(2.16)$$

where P_1 = initial known population

P_2 = final known population

t = No. of decades (period) between P_1 and P_2.

The other method to determine r, is to compute the average of the percentage growth rates of the several known decades of the past. The growth rates ; *i.e.* $\frac{\text{increase in population}}{\text{original population}} \times 100$ values, are computed for each known decade, and their average may be taken as the assumed *constant per decade increase (r)*.

The average may again be either the *arithmetic average i.e.*

$$\frac{r_1 + r_2 + r_3 + \ldots r_t}{t} \qquad ...(2.17)$$

or the *geometric average i.e.* $\sqrt[t]{r_1 \cdot r_2 \cdot r_3 \ldots r_t}$ ...(2.18)

The design engineers in the field generally consider the arithmetic mean here, because it is slightly higher than the geometric mean, and hence gives conservative higher value of forecasted population. However, the "*GoI Manual on Water and Water Treatment*" recommends the use of geometric mean here ; and hence, we can safely use that value.

Example 2.3. *Solve example 2.2 by geometric increase method.*

Solution. The given population data of table 2.9, is used to compute the growth rates (%) for each decade, as in Table 2.11.

Table 2.11. Computations by Geometric Mean Method

Year (1)	*Population* (2)	*Increase in population in each decade* (3)	*Percentage increase in population i.e. growth rate (r)* = $\frac{Col.\ (3)}{Col.\ (1)} \times 100$ (4)
1930	25,000		
		3000	$\frac{3000}{25000} \times 100 = 12\%$
1940	28,000		
		6000	$\frac{6000}{28000} \times 100 = 21.4\%$
1950	34,000		
		8000	$\frac{8000}{34000} \times 100 = 23.5\%$
1960	42,000		
		5000	$\frac{5000}{42000} \times 100 = 11.9\%$
1970	47,000		

The geometric mean of the growth rates (r)

$$= \sqrt[4]{12 \times 21.4 \times 23.5 \times 11.9}$$

$$= 16.37\% \text{ per decade.}$$

Now, assuming that the future population increases at this constant rate (16.37%), we have

$$P_n = P_0 \left(1 + \frac{r}{100}\right)^n$$

or $$P_n = P_0 (1 + 0.1637)^n$$

$$= P_0 (1.1637)^n$$

Using n = 1, 2, 3 decades, we have

The population after 1 decade *i.e.* for the year 1980

$$= P_{1980} = 47{,}000\ (1.1637)$$

$$= \mathbf{54{,}694.} \quad \textbf{Ans.}$$

Similarly P_{1990} = Population after 2 decades

$$= 47000\ (1.1637)^2 = \mathbf{63{,}647.} \quad \textbf{Ans.}$$

Similarly P_{2000} = Population after 3 decades

$$= 47000\ (1.1637)^3 = \mathbf{74{,}066.} \quad \textbf{Ans.}$$

Example 2.4. *Compute the population of the year 2000 and 2006 for a city whose population in the year 1930 was 25,000, and in the year 1970 was 47,000. Make use of geometric increase method.*

Solution. In this question, the intermediate census data between 1930 to 1970 is not given, and hence geometric mean method of all known decades is not possible. The growth rate per decade (r) can, however, be computed by using Eqn. (2.16) as :

$$r = \sqrt[t]{\frac{P_2}{P_1}} - 1 = \sqrt[4]{\frac{47000}{25000}} - 1$$

$$= 0.17095$$

$$= \mathbf{17.095\% \text{ per decade.}}$$

Now, using Eqn. (2.15) as :

$$P_n = P_0 \left(1 + \frac{r}{100}\right)^n, \text{ we have}$$

Hence, $P_{2000} = P_3$ (after 3 decades from 1970 onward)

$$= P_{1970} \left(1 + \frac{r}{100}\right)^3$$

$$= 47{,}000\ (1 + 0.17095)^3$$

$$= 47{,}000\ (1.17095)^3$$

$$= \mathbf{75{,}459.} \quad \textbf{Ans.}$$

Population for the year 2006, means that it is after 36 years (3.6 decades) from 1970 onward

$$\therefore \quad P_{2006} = P_{3.6} = P_{1970}\,(1 + 0.17095)^{3.6}$$
$$= 47000\,(1.17095)^{3.6} = \mathbf{82{,}954.}\quad \textbf{Ans.}$$

Example 2.5. *Determine the future population of a satellite town by the Geometric increase method for the year 2011, given the following data :*

Year	*1951*	*1961*	*1971*	*1981*	.....	*2011*
Population in thousand	*93*	*111*	*132*	*161*	.....	?

(Civil Services, 1982)

Solution. The given data is analysed in table 2.12, to determine growth rates for each decade.

Table 2.12

Year	*Population in thousands*	*Increase in population in thousands*	*%age increase in population = growth rate* $= \frac{Col.\,(3)}{Col.\,(2)} \times 100$
(1)	*(2)*	*(3)*	*(4)*
1951	93		
		18	19.35
1961	111		
		21	18.92
1971	132		
		29	21.97
1981	161		

Constant growth rate, assumed for future

r = geometric mean of past growth rates

$= \sqrt[3]{19.35 \times 18.92 \times 21.97}$

= 20.03% per decade.

The population after n decades is now given by Eqn. (2.15) as :

$$P_n = P_0\left(1 + \frac{r}{100}\right)^n$$

$\therefore$ P_{2011} = Population after 3 decades from 1981

$$= P_{1981}\left(1 + \frac{20.03}{100}\right)^3$$
$$= 1{,}61{,}000\,(1.2003)^3$$
$$= \mathbf{2{,}78{,}417.}\quad \textbf{Ans.}$$

2.10.3. Method of Varying Increment or Incremental Increase Method. In this method, the per decade growth rate is not assumed to be constant as in the arithmetic or geometric progression methods ; but is progressively *increasing* or decreasing, depending upon whether the average of the *incremental increases** in the past data is *positive* or negative.

The population for a future decade is worked out by adding the mean arithmetic increase (say $\bar{x}$) to the last known population as in "arithmetic increase method", and to this is added the average of the *incremental increases* $\bar{y}$, once for the first decade, twice for the second decade, thrice for the third decade, and so on.

The method, thus, assumes that the growth rate in the first decade is @ $(\bar{x} + \bar{y})$, in the second decade @ $(\bar{x} + 2\bar{y})$, and in the nth decade @ $(\bar{x} + n.\bar{y})$. *Thus, the growth rate is assumed to be varying.*

As per the above discussion, we can mathematically write as

P_1 = Population after 1 decade from the present (*i.e.* last known census)

$$= P_0 + (\bar{x} + 1.\bar{y}) \qquad ...(i)$$

P_2 = Population after 2 decades

$$= P_1 + (\bar{x} + 2.\bar{y})$$

$$= P_0 + (2\bar{x} + 3.\bar{y}) \qquad ...(ii)$$

$$= \left[P_0 + 2\bar{x} + 2.\frac{(2+1)}{2}\bar{y}\right] \qquad ...(iii)$$

Similarly,

$$P_3 = P_2 + (\bar{x} + 3.\bar{y}) \qquad ...(iv)$$

$$= P_0 + 3\bar{x} + 6.\bar{y} \qquad ...(v)$$

$$= \left[P_0 + 3\bar{x} + 3.\left(\frac{3+1}{2}\right)\bar{y}\right] \qquad ...(vi)$$

Similarly,

$$P_4 = P_3 + (\bar{x} + 4.\bar{y})$$

$$= P_0 + 4\bar{x} + 10.\bar{y}$$

$$= \left[P_0 + 4\bar{x} + 4.\left(\frac{4+1}{2}\right)\bar{y}\right] \qquad ...(iii)$$

Proceeding in this way, we can write :

$$\boxed{P_n = P_0 + n\bar{x} + \frac{n(n+1)}{2}.\bar{y}} \qquad ...(2.19)$$

which is the requisite equation for this method

*Increment over the increase.

where P_n = Population after n decades from present (*i.e.*, last known census)

$\bar{x}$ = Average increase of populations of known decades

$\bar{y}$ = Average of incremental increases of the known decades.

This method will give end results, somewhere between the results given by 'arithmetic increase method' and 'geometric increase method', and is, thus, considered to be giving quite satisfactory results.

Example 2.6. *Solve example 2.2 by Incremental Increase method.*

Solution. The given population data of table 2.7 is extended in table 2.13, so as to compute the values of per decade *population increases*, and *incremental increases* for all the given decades, as in col. (3) and (4), respectively of this table, which is self explanatory.

Table 2.13

Year (1)	*Population* (2)	*Increase in population* (3)	*Incremental increase ; i.e. increment on the increase* (4)
1930	25,000		
		3000	
1940	28,000		(+) 3000
		6000	
1950	34,000		(+) 2000
		8000	
1960	42,000		(–) 3000
		5000	
1970	47,000		
Total		22,000	(+) 2000
Average per decade		$\bar{x} = \frac{22,000}{4} = \mathbf{5500}$	$\bar{y} = \frac{2000}{3}$ = (+) **667** (say)

The future population P_n is now given by eqn. (2.19) as :

$$P_n = P_0 + n\bar{x} + n \cdot \frac{(n+1)}{2} \cdot \bar{y}$$

Hence,
$$P_{1980} = P_{1970} + 1 \cdot \bar{x} + \frac{1(1+1)}{2} \cdot \bar{y}$$

$$= 47,000 + 1 \times 5500 + \frac{(1 \times 2)}{2} \times 667$$

$$= \mathbf{53,167.} \quad \textbf{Ans.}$$

$$P_{1990} = P_{1970} + 2 \cdot \bar{x} + \frac{2(2+1)}{2} \cdot \bar{y}$$

$$= 47{,}000 + 2 \times 5500 + 3 \times 667$$

$$= \mathbf{60{,}001.} \quad \textbf{Ans.}$$

$$P_{2000} = P_{1970} + 3\bar{x} + \frac{3(3+1)}{2} \cdot \bar{y}$$

$$= 47{,}000 + 3 \times 5500 + 6 \times 667$$

$$= \mathbf{67{,}502.} \quad \textbf{Ans.}$$

Note. By comparing the results of Example 2.2, 2.3 and 2.6, it can be seen that the 'Incremental method' gives results somewhere between the results given by the Arithmetic increase method and the Geometric increase method.

The geometric progression method, since gives highest values of forecasted population, evidently gives higher results for developed cities which do not expand in future at compound rates ; although it may be suitable for new younger cities expanding at faster rates. For old cities, the arithmetic method may be better, although, incremental method is considered to be the best for any city, whether old or new.

Example 2.7. *The population of a locality, as obtained from a census report, is given below :*

Table 2.14

Census year (1)	*Population* (2)
1880	*8,000*
1890	*12,000*
1900	*17,000*
1910	*22,500*
1920	*29,000*
1930	*37,500*
1940	*47,000*
1950	*57,000*
1960	*66,500*

Estimate the population of the locality in 1980 by the Incremental method.
(*Engg. Services, 1967*)

Solution. The given table 2.14 is extended to table 2.15 as shown. The table is otherwise self-explanatory.

Table 2.15

Census year (1)	*Population* (2)	*Increase in population* (3)	*Increment over the increase, i.e. Incremental Increase* (4)
1880	8,000		
		4,000	
1890	12,000		(+) 1,000
		5,000	
1900	17,000		(+) 500
		5,500	
1910	22,500		(+) 1,000
		6,500	
1920	29,000		(+) 2,000
		8,500	
1930	37,500		(+) 1,000
		9,500	
1940	47,000		(+) 500
		10,000	
1950	57,000		(–) 500
		9,500	
1960	66,500		
Total		58,500	(+) 5,500
Average		$\bar{x} = \frac{58,500}{8}$ = 7313 (say)	$\bar{y} = \frac{5,500}{7}$ = (+) 786 (say)

Population at the end of year 1980

$$P_2 = P_0 + 2.\ \bar{x} + \frac{2(2+1)}{2} \cdot \bar{y}$$

$$= 66,500 + 2(7,313) + 3(786) = \mathbf{83,484.}\ \textbf{Ans.}$$

Example 2.8. *With the help of the following data ; estimate by Incremental increase method, the population of a city for the year 2000 A.D.*

Table 2.16

Year	*Population*
1880	*25,000*
1890	*27,500*
1900	*34,100*
1910	*41,500*
1920	*47,050*
1930	*54,500*
1940	*61,000*

(Engg. Services, 1970)

Solution. The given table 2.16 is extended to table 2.17 so as to work out the average increase per decade and average of the net incremental increases, as shown. The table is otherwise self-explanatory.

Table 2.17

Year (1)	*Population* (2)	*Increase in population* (3)	*Increment over the increase, i.e. Incremental increase* (4)
1880	25,000		
		2,500	
1890	27,500		(+) 4,100
		6,600	
1900	34,100		(+) 800
		7,400	
1910	41,500		(–) 1,850
		5,500	
1920	47,050		(+) 1,900
		7,450	
1930	54,500		(–) 950
		6,500	
1940	61,000		
Total		36,000	6,800 – 2,800 = 4,000
Average per decade		$\bar{x} = \frac{36,000}{6} = 6,000$	$\bar{y} = \frac{4,000}{5} = (+)\ 800$

The expected population at the end of year 2000

$$P_6 = P_0 + 6 \,.\, \bar{x} + \frac{6 \times 7}{2} \bar{y}$$

$$= 61,000 + 6(6,000) + 21(800)$$

$$= \mathbf{1,13,800.}\ \ \mathbf{Ans.}$$

Example 2.9. *In a town, it has been decided to provide 200 l per head per day in the 21st century.*

Estimate the domestic water requirements of this town in the year AD 2000 by projecting the population of the town by incremental increase method :

Year	*Population*
1940	*2,37,98,624*
1950	*4,69,78,325*
1960	*5,47,86,437*
1970	*6,34,67,823*
1980	*6,90,77,421*

(Engg. Services, 1987)

Solution. Thy given population data is analysed, as shown in Table 2.18.

Table 2.18

Year (1)	*Population* (2)	*Increase in population* (3)	*Increment over the increase, i.e. Incremental increase* (4)
1940	2,37,98,624		
		2,31,79,701	
1950	4,69,78,325		(–) 1,53,71,589
		78,08,112	
1960	5,47,86,437		(+) 8,73,274
		86,81,386	
1970	6,34,67,823		(–) 30,71,788
		56,09,598	
1980	6,90,77,421		
Total		4,52,78,797	(–) 1,75,70,103
Average per decade		$\bar{x}$ = 1,13,19,699	$\bar{y} = (-)\frac{1,75,70,103}{3}$ = (–) 58,56,701

Expected population at the end of year 2000 (*i.e.* after 2 decades from 1980)

$$= P_2 = P_0 + 2 \,.\, \bar{x} + \frac{2 \times .3}{2} \,.\, \bar{y}$$

$$= 6,90,77,421 + 2(1,13,19,699) - 3(58,56,701)$$

$$= 7,41,46,716$$

$\therefore$ Water requirement in AD 2,000 @ 200 *l*/head/d

$$= \frac{200 \times 7,41,46,716}{10^6} \text{ M.}l\text{/day}$$

= 14,829 ML/day. Ans.

2.10.4. Decreasing Rate of Growth Method. Since the rate of increase in population goes on reducing, as the cities reach towards saturation, a method which makes use of the decrease in the percentage increase, is many a times used, and gives quite rational results. In this method, the average decrease in the percentage increase is worked out, and is then subtracted from the latest percentage increase for each successive decade, as explained in example 2.10. This method is however, applicable only in cases, where the rate of growth shows a downward trend.

Example 2.10. *Solve example 2.2 by decreasing rate method.*

Solution. The given table 2.9 is extended to table 2.19 as shown. The table is otherwise self-explanatory.

Table 2.19

Year	*Population*	*Increase in population*	*Percentage increase in population*	*Decrease in in the percentage increase*
1930	25,000			
		3,000	$\frac{3000}{25,000} \times 100 = 12\%$	
1940	28,000			(–) 9.40%
		6,000	$\frac{6000}{28,000} \times 100 = 21.4\%$	
1950	34,000			(–) 2.1%
		8,000	$\frac{8000}{34,000} \times 100 = 23.5\%$	
1960	42,000			(+) 11.6%
		5,000	$\frac{5000}{42,000} \times 100 = 11.9\%$	
1970	47,000			
Total		22,000		11.5 + 11.6 = 0.1%
Average per decade				$\frac{0.1}{3} = 0.03\%$ decrease

(*a*) The expected population at the end of year 1980

$$= 47,000 + \frac{11.9 - 0.03}{100} \times 47,000$$

$$= 47,000 + \frac{11.87}{100} \times 47,000$$

$$= 47,000 + 5,570$$

$$= \mathbf{52,570.} \quad \textbf{Ans.}$$

(*b*) The expected population at the end of year 1990

$$= 52,570 + \frac{11.87 - 0.03}{100} \times 52,570$$

$$= 52,570 + \frac{11.84}{100} \times 52,570$$

$$= 52,570 + 6,230$$

$$= \mathbf{58,800.} \quad \textbf{Ans.}$$

(*c*) The expected population at the end of year 2000

$$= 58,800 + \frac{11.84 - 0.03}{100} \times 58,800$$

$$= 58{,}800 + \frac{1181}{100} \times 58{,}800$$

$$= 58{,}800 + 6{,}950$$

$$= \mathbf{65{,}750.} \quad \textbf{Ans.}$$

Note. Strictly speaking, this method is not applicable in this case, because here, the rate of growth is not falling regularly, but has fallen only in the last decade.

Example 2.11. *Given the following data, calculate the population at the end of next three decades by decreasing rate method.*

Table 2.20

Year	*Population*
1940	*80,000*
1950	*1,20,000*
1960	*1,68,000*
1970	*2,28,580*

Solution. The given table 2.20 is extended to table 2.21, as shown below. The table is otherwise self-explanatory.

Table 2.21

Year	*Population*	*Increase in population*	*Percentage increase in population*	*Decrease in in the percentage increase*
1940	80,000			
		40,000	$\frac{40{,}000}{80{,}000} \times 100 = 50\%$	
1950	1,20,000			10%
		48,000	$\frac{48{,}000}{1{,}20{,}000} \times 100 = 40\%$	
1960	1,68,000			4%
		60,580	$\frac{60{,}580}{1{,}68{,}000} \times 100 = 36\%$	
1970	2,28,580			
Total				14%
Average per decade				$\frac{14}{2} = 7\%$

The expected population at the end of year 1980

$$= 2{,}28{,}580 + \left[\frac{36 - 7}{100}\right] 2{,}28{,}580$$

$$= 2{,}28{,}580\ [1.29] = \mathbf{2{,}94{,}870.} \quad \textbf{Ans.}$$

(*b*) The expected population at the end of year 1990

$$= 2{,}94{,}870 + \frac{29-7}{100} \times 2{,}94{,}870$$

$$= 2{,}94{,}870\ (1.22) = \mathbf{3{,}59{,}740.}\quad \textbf{Ans.}$$

(*c*) The expected population at the end of year 2000

$$= 3{,}59{,}740 + \frac{22-7}{100} \times 3{,}59{,}740$$

$$= 3{,}59{,}740[1.15] = \mathbf{4{,}13{,}700.}\quad \textbf{Ans.}$$

2.10.5. Simple Graphical Method. In this method, a graph is plotted from the available data, between time and population. The curve is then smoothly extended up to the desired year. The method, however, gives very approximate results, as the extension of the curve is done by the intelligence of the designer.

Example 2.11. *Work out the population of the year 2,000 from the data given below, using simple graphical method.*

Table 2.22

Year	*Population*
1880	*25,000*
1890	*27,500*
1900	*33,000*
1910	*39,000*
1920	*45,000*
1930	*54,500*
1940	*61,000*

Solution. The graph between time and population is plotted from the data given in table 2.22, as shown in Fig. 2.4.

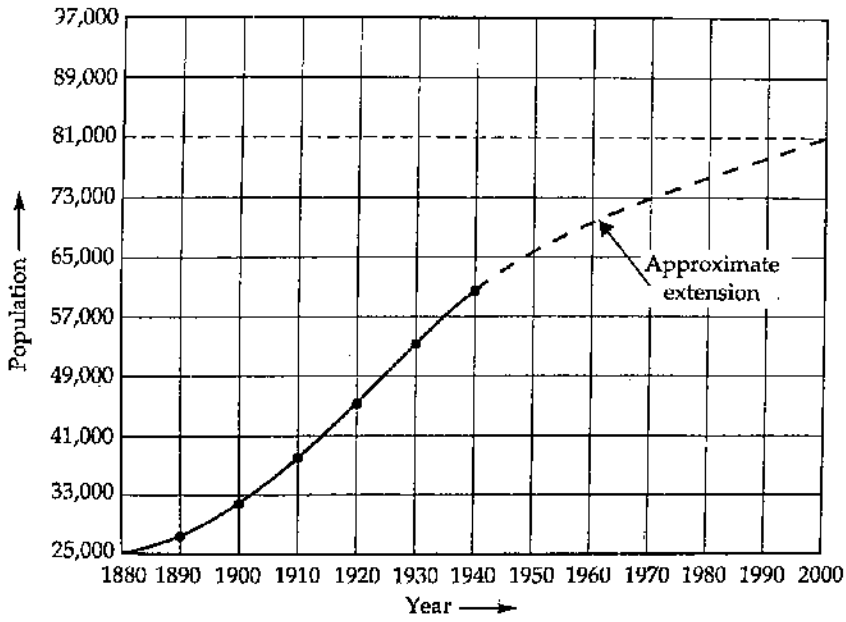


Fig. 2.4. Graph between population and time for example 2.11.

The curve obtained is smoothly extended upto year 2000, and to read the corresponding population as **81,000. Ans.**

All the five methods described upto now are based on the assumption that factors and conditions which were responsible for population increase in the past will continue even in future also, with the same intensity. That is a vague assumption and may or may not be satisfied. For example, a city or a community may suddenly impose immigration restrictions, or may impose compulsory family planning methods for eligible couples, or may develop enhanced medical facilities thereby reducing death rates, etc. Due to such reasons, the results obtained from these methods may or may not be precise. However, they are less time consuming and are used by engineers. They are also useful for providing a check on the results obtained by some other advanced and time consuming methods of population forecasting, a few of which are described below :

2.10.6. Comparative Graphical Method. In this method, the cities having conditions and characteristics similar to the city whose future population is to be estimated are, first of all, *selected.* It is then assumed that the city under consideration will develop, as the *selected similar* cities have developed in the past. This method has a logical background, and if statistics of development of similar cities are available, quite precise and reliable results can he obtained. However, it is rather difficult to find identical cities with respect to population growth. The use of this method is explained below :

Consider Fig. 2.5. Let the population of a city X be given for 4 decades (say 1940, 1950, 1960 and 1970). The population-time curve is then plotted.

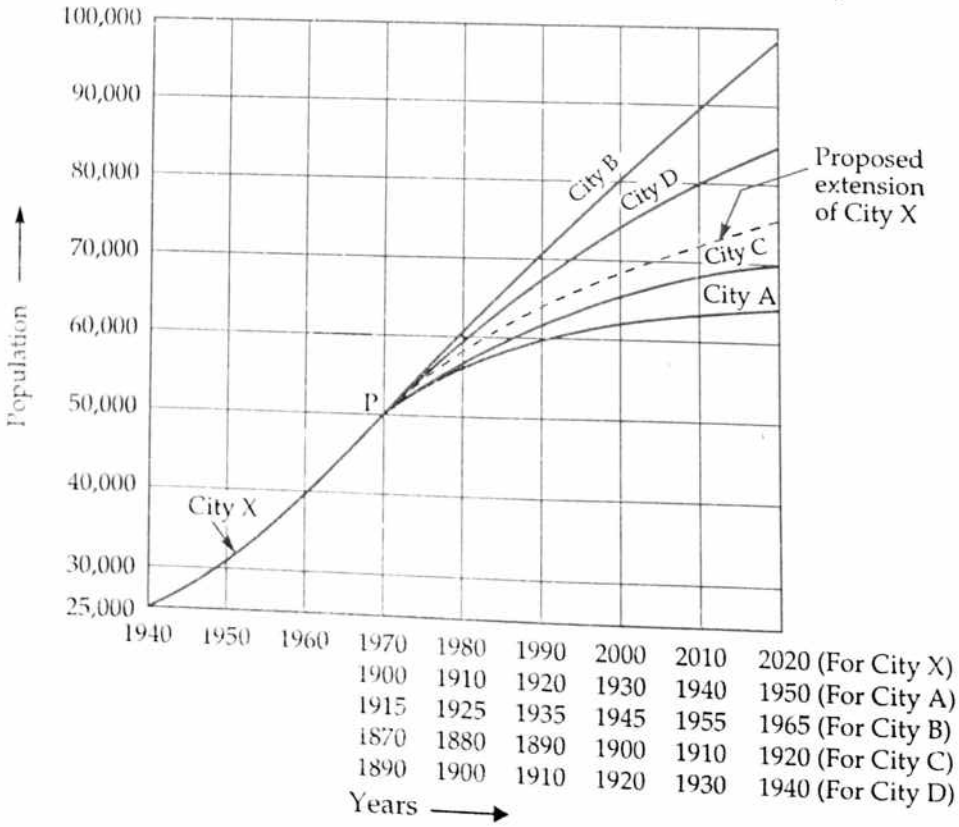


Fig. 2.5

Now, suppose it is required to estimate the population of the city X at the end of year 2020. And let the available data show that this city X has reached the present population of 50,000 in the year 1970. Then the available data of *similar cities* (say city A, B, C and D) is analysed. Let it be found that city *A* has reached 50,000 in the year 1900, then its curve is plotted beyond the year 1900 onward. However, this curve for city A should start from point *P* which represents the present population of city X. Similarly, the populations of cities B, C and D are plotted from the year they have reached 50,000, onwards. Now, the curve for city X is carefully extended between the curves of cities A, B, C and D, allowing it to be influenced more by the rates of growth of cities, which are more 'similar' to the city in consideration.

2.10.7. Master Plan Method or Zoning Method. Big and metropolitan cities are generally not allowed to develop in haphazard and natural ways, but are allowed to develop only in planned ways. The expansions of such cities are regulated by various by-laws of corporations and other local bodies. Only those expansions are allowed, which are permitted or proposed in the master plan of that city.

The master plan prepared for a city is generally such, as to divide the city in various zones, and thus to separate the residence, commerce, and industry from each other. The population densities are also fixed. Say for example, there may be 5 persons living in a residential plot and there may be 10,000 plots in a zone. Then, the total population of this zone, when fully developed, can be easily worked out as $5 \times 10{,}000 = 50{,}000$. Hence, when the development is regulated by such a scheme, it is very easy to access precisely the design population, *because the master plan will give us as to when and where the given number of houses, industries and commercial establishments would be developed.*

The common figures for population densities which may be taken in master plan preparations or in population estimates are shown in table 2.23.

Table 2.23. Population Densities

S. No.	*Type of Area*	*No. of persons per hectare*
1.	Residential areas	
	(*a*) Single family dwellings, large lots	10—40
	(*b*) Single family dwellings, small lots	40—90
	(*c*) Multiple family dwellings	90—250
	(*d*) Apartments or tenement houses	250—2500 or more
2.	Commercial trade areas	40—75
3.	Industrial areas	10—40
4.	Total, exclusive of parks, playgrounds, etc.	25—125

2.10.8. The Ratio Method or the Apportionment Method. In this method of forecasting future population of a city or a town, the city's census population record is expressed as the percentage of the population of the whole country. In order to do so, the local population and the country's population for the last

four to five decades is obtained from the census records. The ratios of the local population to national population are worked out for these decades. A graph is then plotted between time and these ratios, and extended up to the design period, so as to extrapolate the ratio corresponding to the future design year. Sometimes, the last census ratio or the average of the past few years may be used. This ratio is then multiplied by the expected national population at the end of the design period, so as to obtain the required city's future population. The expected national population in different future years are generally available, as they are worked out precisely by the Census Deptt. by some rational recommended method. This method is, therefore, very suitable for areas whose growths are parallel to the national growth. However, this method does not take into consideration, the abnormal conditions that may prevail in certain local areas.

2.10.9. The Logistic Curve Method. It was explained earlier that under normal conditions, the population of a city shall grow as per the *logistic curve*, shown in Fig. 2.3. P.F. Verhulst has put forward a mathematical solution for this logistic curve. According to him, the entire curve AD (Fig. 2.3) can be represented by an autocatalytic first order equation, given as :

$$\log_e\left(\frac{P_s - P}{P}\right) - \log_e\left(\frac{P_s - P_0}{P_0}\right) = -K P_s . t \qquad ...(2.20)$$

where P_0 = The population at the start point of the curve A

P_s = Saturation population

P = Population at any time t from the origin A

K = Constant.

From Eq. (2.20), we get

$$\log_e\left[\left(\frac{P_s - P}{P}\right)\times\left(\frac{P_0}{P_s - P_0}\right)\right] = -K P_s.t$$

or $$\left(\frac{P_s - P}{P}\right)\left(\frac{P_0}{P_s - P_0}\right) = \log_e^{-1}(-K.P_s.t)$$

or $$\frac{P_s - P}{P} = \left[\frac{P_s - P_0}{P_0}\right]\log_e^{-1}(-K.P_s.t)$$

or $$\frac{P_s}{P} - 1 = \left[\frac{P_s - P_0}{P_0}\right]\log_e^{-1}(-K.P_s.t)$$

or $$\frac{P_s}{P} = 1 + \left[\frac{P_s - P_0}{P_0}\right]\log_e^{-1}(-K.P_s.t)$$

or $$P = \frac{P_s}{1 + \frac{P_s - P_0}{P_0}\log_e^{-1}(-KP_s.t.)} \qquad ...(2.21)$$

Substituting $\frac{P_s - P_0}{P_0} = m$ (a constant)

and $-K.P_s = n$ (another constant)

we get

$$P = \frac{P_s}{1 + m \cdot \log_e^{-1}(nt)} \qquad ...(2.22)$$

This is the required equation of the logistic curve. McLean further suggested that if only three pairs of characteristic values P_0, P_1, P_2 at times $t = t_0 = 0, t_1$, and $t_2 = 2t_1$ extending over the useful range of the census populations, are chosen, the saturation value P_s and the constants m and n can be evaluated from three simultaneous equations, as follows :

$$P_s = -\frac{2P_0P_1P_2 - P_1^2(P_0 + P_2)}{P_0P_2 - P_1^2} \qquad ...(2.23)$$

$$m = \frac{P_s - P_0}{P_0} \qquad ...(2.24)$$

$$n = \left(\frac{1}{t_1}\right)\log_e\left[\frac{P_0(P_s - P_1)}{P_1(P_s - P_0)}\right] \qquad ...(2.25)$$

$$= \frac{2.3}{t_1}\log_{10}\left[\frac{P_0(P_s - P_1)}{P_1(P_s - P_0)}\right] \qquad ...(2.25a)$$

Knowing P_0, P_1 and P_2 from census data and using them in these equations, the values of P_s, m and n are known, and the equation of the logistic curve (Eq. 2.22) is thus known. From that, the population P at any time t can then be obtained, as explained in the example given below.

Example 2.12. *In two periods of each of 20 years, a city has grown from 30,000 to 1,70,000 and then to 3,00,000. Determine, (a) the saturation population ; (b) the equation of the logistic curve ; (c) the expected population after the next 20 years.*

Solution. In this question, we have

$P_0 = 30{,}000$ $\quad t = 0$

$P_1 = 1{,}70{,}000$ $\quad t_1 = 20$ yrs.

$P_2 = 3{,}00{,}000$ $\quad t_2 = 40$ yrs.

Using Eq. (2.23), we have

$$P_s = \frac{2P_0P_1P_2 - P_1^2(P_0 + P_2)}{P_0P_2 - P_1^2}$$

$$\therefore \quad P_s = \frac{2 \times 30{,}000 \times 1{,}70{,}000 \times 3{,}00{,}000 - (1{,}70{,}000)^2[30{,}000 + 3{,}00{,}000]}{30{,}000 \times 3{,}00{,}000 - (1{,}70{,}000)^2}$$

$$= \frac{[2 \times 30 \times 170 \times 300 - (170)^2(330)]\,10^9}{[30 \times 300 - (170)^2]\,10^6}$$

$$= \frac{170\,[(18{,}000) - 170 \times 330]\,10^3}{9{,}000 - 28{,}900}$$

$$= \frac{(-)\,170 \times 38{,}100}{(-)\,19{,}900} = 3{,}26{,}000$$

(a) Hence, the saturation population = **3,26,000. Ans.**

Using Eq. (2.24), we get

$$m = \frac{P_s - P_0}{P_0}$$

$$= \frac{3,26,000 - 30,000}{30,000} = \frac{2,96,000}{30,000} = \mathbf{9.87}$$

Using Eq. (2.25a), we get

$$n = \frac{2.3}{t_1} \log_{10} \left[\frac{P_0(P_s - P_1)}{P_1(P_s - P_0)}\right]$$

$$= \frac{2.3}{20} \log_{10} \left[\frac{30,000(3,26,000 - 1,70,000)}{1,70,000(3,26,000 - 30,000)}\right]$$

$$= \frac{2.3}{20} \log_{10} \left(\frac{30 \times 156}{170 \times 296}\right) = \frac{2.3}{20} \log_{10} 0.093$$

$$= \frac{2.3}{20} \times (\bar{2}.968) = \frac{2.3}{20} (-1.032)$$

$$= \mathbf{-0.119.}$$

(b) Substituting the values of P_s, m and n in Eq. (2.22), we get the required equation of logistic curve, as

$$\left[P = \frac{3,26,000}{1 + 9.87 \log_e^{-1} (-0.119\, t)}\right] \quad \textbf{Ans.} \qquad \ldots(2.26$$

(c) When t = 60 yrs., what is P ?

Substituting t = 60 in Eq. (2.26), we get

$$P = \frac{3,26,000}{1 + 9.87 \log_e^{-1} (-0.119 \times 60)}$$

or $$P = \frac{3,26,000}{1 + 9.87 \log_e^{-1} (-7.14)} = \left[\frac{3,26,000}{1 + 9.87\, x}\right]$$

where, $x = \log_e^{-1} (-7.14)$.

Now, to evaluate $\log_e^{-1} (-7.14)$, we proceed as follows :

$$\log_e^{-1} (-7.14) = x$$

or $$\log_e^x = -7.14$$

or $$2.3 \log_{10}^x = -7.14$$

or $$\log_{10}^x = \frac{-7.14}{2.3} = -3.1 = \bar{4}.9$$

Taking antilog, we get

$$x = 0.000795$$

$\therefore$ $$P = \frac{3,26,000}{1 + 9.87 \times 0.000795}$$

$$= \frac{3,26,000}{1+0.000776} = \frac{3,26,000}{1.00776} = \mathbf{3,23,000}$$

Hence, the population after 20 more years will be = **3,23,000. Ans.**

PROBLEMS

1. How will you estimate the quantity of water required by a town while arranging a water supply scheme for the same ?
2. State how the quantity of water for which a water supply scheme is to be designed, is estimated. Explain the purpose of the considerations involved in such an estimate.
3. What are the various purposes for which provision should be made in the average daily per capita demand of water in a water supply scheme ? Give the approximate breakdown for each of these purposes if the average daily demand is 270 litres.
4. Describe how you would arrive at the total quantity of water to be supplied to a metropolitan area.
5. (*a*) A town water supply is given at 150 litres per head per day. Suggest a suitable break up of the consumption on different heads.
   (*b*) What allowance should be made in India for loss and wastage in water supply ? Give reasons.
6. Discuss briefly and critically the statement. "The amount of water used annually for fire fighting is small compared with yearly consumption for domestic purposes. Therefore, the determination of pipe sizes in a distribution system is based largely on an analysis of the domestic flow".
7. What is meant by the term "per capita demand" ? How is it estimated ? What values of per capita demand would you recommend for :
   (*i*) a small town with a population of 50,000.
   (*ii*) a small city with a population of 1—2 lakhs.
   (*iii*) a large city with a population of 5 lakhs.
   State the reasons in support of your answers.
8. (*a*) Mention and discuss the factors that influence per capita demand.
   (*b*) Give a reasonable estimate of average daily consumption of water for a residential town of 50,000 population. Suggest a possible break-up of consumption on different heads.
9. Write a short note on provision for "fire demand" in water supply.
10. (*a*) Write short note on the wastage of water in public supplies.
    (*b*) State various methods of detection and prevention of wastage. Enumerate the causes of such wastage.
    [**Note.** Please see chapter 10 for "Methods of Detection and Prevention of Wastage".]
11. What is meant by the variations in the rate of demand ? What are the effects of these variations on the design of various units of a water supply scheme ?
12. What is peak-hour demand and how does it affect the design of water supply system ? Sketch the fluctuations in demand for typical Indian conditions.
13. (*a*) What do you understand by continuous and intermittent system of water supply ? What are their relative advantages and disadvantages ?
    (*b*) Discuss the steps to be taken to prevent waste and leakage of water in the distribution system.
14. (*a*) Discuss the following as they relate to water supply :
    (*i*) Intermittent supply *versus* continuous supply
    (*ii*) Single supply *versus* dual supply
    (*iii*) Metered supply *versus* non-metered supply.

(*b*) The average increase in the population of a town per decade over a period of 6 decades was 4100 and the average percentage increase was 12%. If the population at the end of the sixth decade was 2,20,000, estimate the population two decades later by (*i*) the arithmetic increase and (*ii*) the geometric increase methods.

15. (*a*) Discuss briefly as to how the requirements of water vary from season to season, day to day, and hour to hour, and in what way does it affect the design capacities of various components of a water supply scheme.

(*b*) A water supply scheme has to be designed for a city having a population of 2 lakhs. Estimate the important kinds of drafts which may be required to be recorded for an average water consumption of 180 lphd. Also record the required capacities of major components of the proposed water works system for the city using a river as a source of supply. Assume suitable data, wherever needed.

16. What is meant by coincident draft and how is it used in estimating design draft for the design of distribution system ?

17. For a city of a population 1,00,000, find the following in connection with the water distribution system :

(*i*) Daily water demand

(*ii*) Daily variation in water demand

(*iii*) Monthly variation in water demand

(*iv*) Hourly variation

(*v*) Fire demand.

18. What is meant by "design periods" and "population forecasts" ? Why is the population forecast necessary in the design of public water supply schemes ? Discuss the different methods employed for the purpose and their comparative merits and demerits.

19. (*a*) Explain the different methods of forecasting future population of a city for which a water supply scheme is to be planned.

(*b*) The population statistics pertaining to a town are given below. Estimate the population expected in 1980 by arithmetical and geometrical increase methods.

Year	1930	1940	1950	1960	1970
Population	70,000	1,00,000	1,50,000	2,00,000	2,40,000

20. Describe the 'incremental method' by which the population of a locality on a future date can be estimated. Also state its advantages over arithmetical and geometrical methods.

21. (*a*) Explain any three methods of estimating the future population of a city. What are their relative merits ?

(*b*) Assuming the geometric rate of growth of population of a town, calculate the population of the town in the year 1991 with the help of the following census records of the population.

Year	1951	1961	1971
Population in thousand	258	495	735

22. (*a*) Discuss the 'Logistic curve method' for determining the future populations of a locality. Derive a standard equation for such a curve and explain its use for determining the future population.

(*b*) In two periods each of 20 years, a city has grown from 40,000 to 1,60,000 and then 2,80,000. Determine : (*a*) the saturation population ; (*b*) the equation of the logistic curve ; (*c*) the expected population after the next 15 years.

23. (*a*) "The incremental increase method of estimating population combines the advantages of other two methods *i.e.* arithmetical increase and geometrical increase". Discuss the statement.

(*b*) The population of a locality as obtained from census report is as follows :

Census year	1911	1921	1931	1941	1951
Population	3,50,000	4,66,000	9,94,000	15,60,000	16,23,000

Estimate the population of the locality in the year 2001 by using Incremental method.

24. Give reasons for the following :

(*i*) The drinking water should possess a high degree of purity.

(*ii*) The quantity of water required for industries can not the connected to the density of population.

(*iii*) The water demand changes from season to season and from hour to hour.

(*iv*) The climate of a city affects its water requirements.

(*v*) "Intermittent supply system" and "policy of metering" are debatable and should be used only after full considerations of the particular project.

(*vi*) It is desirable to pump the water in shifts.

—3

Sources of Water, Hydrological Concepts, and Study of Surface Sources

3.1. General Introduction

After estimating the water requirement for the proposed water supply scheme, as discussed in the previous chapter, the planners of the scheme must go in for search of nearby water sources, which may be able to supply the required amount of water. If the available supplies of a particular source or sources of water chosen in the vicinity of the area, are not sufficient to meet the demand, then it becomes imperative upon the planners to search some other distant source or sources and to make arrangements for bringing the water from there.

The various sources of water available on the earth can be classified into the following two categories :

(A) **Surface sources,** such as

(*i*) *Ponds and lakes* ;

(*ii*) *Streams and rivers* ;

(*iii*) *Storage reservoirs* ; and

(*iv*) *Oceans*, generally not used for water supplies, at present.

(B) **Sub-surface sources** *or* **Underground sources,** such as :

(*i*) *Springs* ;

(*ii*) *Infiltration galleries* ;

(*iii*) *Infiltration wells* ; and

(*iv*) *Wells and tube-wells.*

Most of the earth's water sources get their water supplies from precipitation, which may fall in various forms, such as, rain, snow, hail, dew, etc. Rains, no doubt, form the principal and the major part of the resultant supplies. Since precipitation is the principal source of water on the earth, and is responsible for the resultant yields of various sources, it will be worth while to discuss the *hydrologic principles* underlying precipitation, rainfall, run-off, yields of surface sources, etc., before discussing these sources of water. The principles have, however, been discussed only in brief and for a detailed description, if needed, readers may refer to "*Water Resources Engg. Vol. I–Hydrology and Water Resources Engineering*" by the same author.

HYDROLOGIC PRINCIPLES

3.2. Hydrologic Cycle

As stated earlier, the various earth's water-sources get their supplies from precipitation, while the precipitation in itself is the evaporation from these sources. Water is lost to the atmosphere as vapour from the earth, which is then precipitated back in the form of rain, snow, hail, dew, sleet or frost, etc. This precipitation and evaporation continue for ever, and thereby a balance is maintained between the two. This *process* is known as *hydrologic cycle* and is shown in Fig. 3.1.

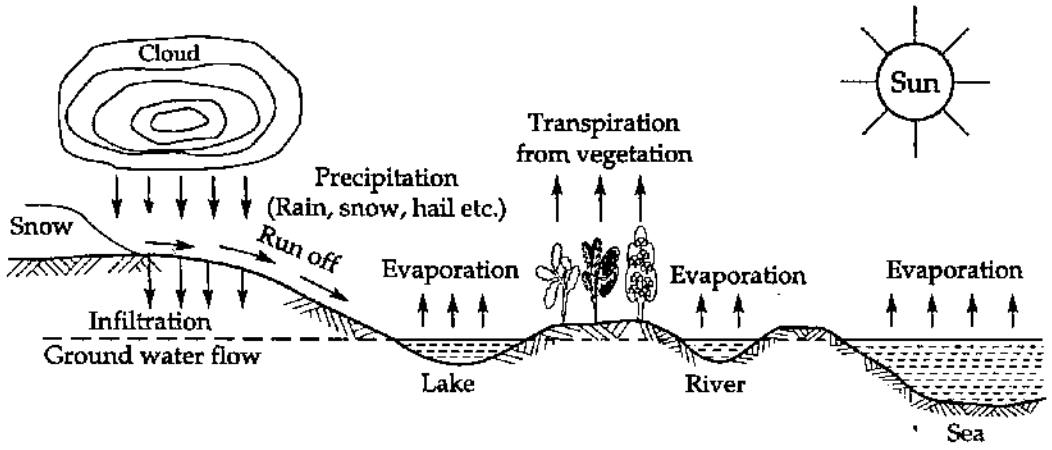


Fig. 3.1. Hydrologic cycle.

3.3. Precipitation

Water evaporates from water surfaces like streams, rivers, oceans, ponds, etc., and also from the land and plants in the form of water vapour. The water vapour get collected in the atmosphere and behave like a gas. Under a normal range of temperature and pressure, the water vapour obey the various gas laws (*i.e.* Boyle's Law, Charles' Law, etc.). As the evaporation continues, the amount of atmospheric vapour goes on increasing. But since a space can hold only a certain fixed amount of water vapour in the presence of a solid or a liquid surface, a stage is reached when any further addition of vapour will get condensed on the surfaces. The vapour may get condensed in different forms, such as mist, rain, hail, snow, sleet, etc. The evaporated water, thus, returns to the earth's surface in any of these forms. *This water which comes back to the surface of the earth in its various forms like rain, snow, hail, etc., is known as precipitation*. A major part of the precipitation occurs in the form of rain, and a minor part occurs in the form of snow. Other forms of precipitation such as hail, sleet, mist, etc., are all very small, and generally ignored in the design of most of the hydrological works, and, therefore, they are not of much importance to us.

3.4. Types of Precipitation

Although the moisture is always present in the atmosphere, but it is condensed only when the air is cooled, so that it becomes saturated with the same water vapour. The usual mechanism by which the air is cooled to cause precipitation is the lifting of the air mass. There are three different methods

by which the air mass gets lifted, so as to cause cooling and condensation of the atmospheric water vapour and the subsequent precipitation mainly in the form of rain or sometimes under special conditions as snow, hail, sleet, etc. Depending upon the way in which the air is cooled so as to cause precipitation, we can have three kinds of precipitation, as given below :

(1) **Cyclonic Precipitation.** Cyclonic Precipitation is caused by the lifting of an air mass due to the pressure difference. If low pressure occurs in an area, air will flow horizontally from the surrounding area, causing the air in the low pressure area to lift. The precipitation that results, is called *non-frontal cyclonic precipitation*. If one air mass lifts over another air mass, the precipitation is called *frontal cyclonic precipitation*. The boundary between these two air masses of different temperatures and densities (one warm air mass and the other colder) is known as a *front or a frontal surface*. The large whirling mass of air, at the centre of which the barometric pressure is low is known as a *cyclone*. The air that rushes horizontally into the low pressure area changes into a whirling mass, because of the rotary motion of the earth about its own axis. The cyclone is a very large mass of air ranging from 800 to 1600 km. in diameter and moving with a velocity of about 50 km/hr. The central portion of this cyclone, where the pressure is low, acts like a chimney, through which the air gets lifted, expands, cools and finally gets condensed, causing precipitation.

Cyclonic precipitation can occur in the form of drizzle, intermittent rain, or steady rain. If the precipitation is caused by a cold front, it is very intense and of short duration, while that caused by a warm front is more continuous. Third possibility is that of an occluded front. An *occluded front* occurs when a cold front over-takes a warm front. The precipitation pattern is a combination of both cold and warm frontal distributions. A cold front is that in which the warm air replaces the colder air, whereas in a warm front, the case is opposite.

Cyclones are responsible for most of the rains in the Central part of the United States, and for most of the winter rains in Haryana and Punjab in India.

(2) **Convective Precipitation.** Convective Precipitation is due to the upward movement of the air that is warmer than its surroundings. Generally, this kind of precipitation occurs in tropics, where on a hot day, the ground surface gets heated unequally, causing the warmer air to lift up as the colder air comes to take its place. The vertical air currents develop tremendous velocities, and are hazardous to aircrafts. Precipitation occurs in the form of showers of high intensity and short duration.

(3) **Orographic Precipitation.** Orographic Precipitation is the most important precipitation and is responsible for most of the heavy rains in India. Orographic precipitation is caused by air masses which strike some natural topographic barriers like mountains, and cannot move forward, and hence rise up, causing condensation and precipitation. The greatest amount of precipitation falls on the windward side, and the leeward side often has very little precipitation. Orographic barriers tend to increase both cyclonic and orographic precipitation because of the increased lifting involved. The rainfall is composed of showers and steady rainfall. A striking example of this kind of natural barriers is the southern slope of Himalayas in India. The winds,

heavily laiden with moisture from the Bay of Bengal strike the southern slope of Himalayas, causing intense rains ; so much so that near Cherrapunji, the average annual rainfall is of the order of 1270 cm. Similarly, the winds, coming from the Pacific Ocean strike the western slopes of the coastal ranges in Washington (U.S.A.) causing heavy rains.

3.5. Rainfall and its Distribution

When rain starts falling, it is first of all intercepted by buildings and other objects, thus preventing it from reaching the ground. This quantity is known as **rainfall interception**. If the total rain is denoted by the letter P, the rainfall interception may be denoted by $P_{(interception)}$ or P_i. Although it is not of much importance, but many a times, the large portions of lighter rains are disposed of in this manner. This difference between the total rainfall and that which is intercepted is called the **ground rainfall** and is denoted by P_g where, $P_g = P - P_i$. Secondly, when the rainfall rate (p) exceeds the interception rate (p_i), water starts reaching the ground and infiltration starts. This quantity is known as **infiltration** and denoted by the letter F. The maximum rate at which the soil in a given condition can absorb water is known as the *infiltration capacity* and may be denoted by the letter f. Thirdly, it finds its way into the innumerable small or large depressions, filling them to their overflow level. This quantity is termed as the **depression storage** (S_d). All this storage is either evaporated or used by vegetation or infiltrates into the soil. None of it, appears as surface run-off.

If after the depression storage is filled, the rain intensity exceeds the infiltration capacity of the soil, the difference appears as **rainfall excess** and denoted by P_e, and *excess rainfall* rate is denoted by p_e

$$\text{where } p_e = p - f$$

This excess water, first of all, accumulates on the ground as **surface detention** (D) and then flows overland, towards the stream channel. This movement is called the *overland flow*, and the water that reaches the stream channel in this manner is called the **surface run-off (S.R.O.)** or **direct run-off (D.R.)**. The entire amount of water contained within the permanent stream channel at any instant is known as *Channel Storage* (S_c).

The rain that falls in the beginning of a storm before the depression storage is just completely filled is called the **initial rain**. And the rain that falls near the end of a storm at a rate less than the infiltration capacity is called the **residual rain**. Both these rains will not contribute to surface run-off. The intervening period is known as the **net supply interval**. The infiltration occurring after the end of the net supply interval is called the **residual infiltration**. It consists of residual rainfall plus that portion of surface detention which remains on the ground by the end of the net supply interval and later infiltrates. The total rainfall excess is, therefore, equal to the surface run-off plus the difference between the residual infiltration and residual rainfall. However, because the latter quantity is usually small, the rainfall excess is generally assumed to be equal to surface run-off.

3.6. Measurement of Rainfall

In order to estimate the effects of precipitation, it is necessary to measure the precipitation, and to find out its distribution at various places on the earth.

All forms of precipitation are measured as the vertical depth of water that would accumulate on a level surface, if the entire precipitation remained where it fell. The total amount of precipitation falling on earth in a given period, will hence be expressed as the depth to which it would accumulate on the *horizontal projection* of the Earth's surface, if there were no losses by evaporation or run-off, and if any part of precipitation falling as snow or ice were melted. The two types of precipitation (*i.e.* rain and snow) are measured separately by measuring devices called rain gauges and snow gauges, respectively.

Since the amount of precipitation varies from place to place, it is necessary to install measuring devices at key points.

The simplest method of measuring rainfall is by setting up a rain gauge with a horizontal circular aperture of known area, and collecting and measuring at regular intervals, the rainfall collected in that gauge. It is assumed that the amount of rainfall collected in the gauge is representative of a certain area around the point where the measurement is made.

3.6.1. Types of Rain Gauges. Any open receptacle with vertical sides can be used as a gauge for measuring rainfall. These refined receptacles which are used for measuring rainfall are called *rain gauges*. Two kinds of rain gauges are generally used. First is the 'non-recording type' rain gauges, and the second is the 'recording type' rain gauges. These two types of rain gauges are summarised below :

(1) **Non-recording rain gauges.** Non-recording rain gauges are most widely used in India. They are known as 'non-recording type', because they do not record the rain, but only collect the rain. The collected rain is then measured by means of graduated cylinders, so as to directly represent the rainfall volume in cm of water depth :

$$\left[i.e. \frac{\text{Volume of water collected in cm}^3}{\text{Area of aperture of the gauge in cm}^2} = \text{cm depth of rain fallen} \right]$$

Various models have been designed for such gauges. Out of them, the **Symon's type** (Fig. 3.2) was mostly used in India till the year 1969 or so. However, since then, the Indian Meteorological Department (IMD) has adopted another model called **Standard gauges** (Fig. 3.3) for their use at all rainfall stations in this country. Symon's and other types of rain gauges have, therefore, been replaced in India, and have become obsolete.

The complete installation of the new standard type of a non-recording rain gauge, which is now being used in India, is shown in Fig. 3.3. Such a rain gauge consists of a collector with a gun metal rim, a base, and a polythene bottle. Both the collector and the base are made of fibre glass reinforced polyster. The collector has a deep set funnel, and the complete rain gauge has a slight taper with the narrower portion at the top, as shown. The collector and the base are locked to each other by means of two complementary locking rings, fixed inside the collector at its lower end and the base at its top end.

The collectors can have apertures of either 100 sq-cm or 200 sq-cm area. The polythene bottles are available in three sizes having capacities of 2, 4 and 10 litres of water. The rain gauge with different combinations of aperture size

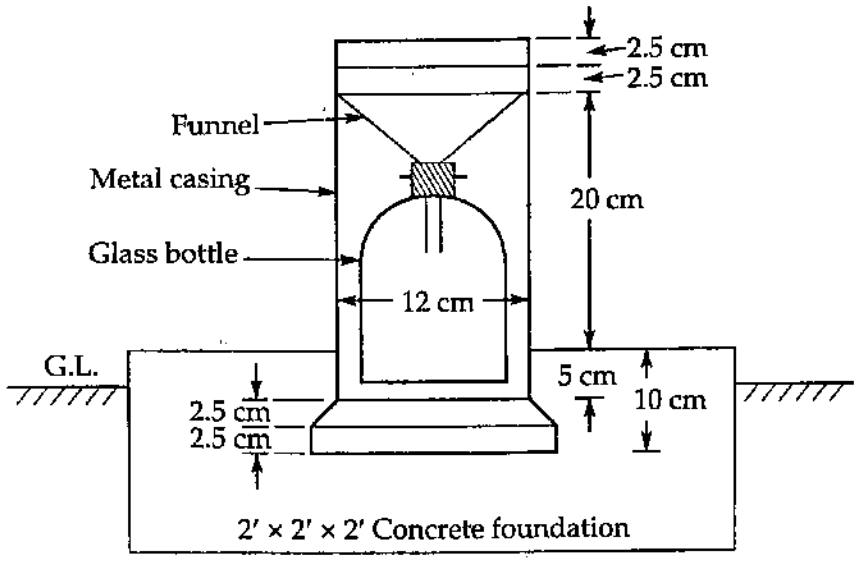


Fig. 3.2. Simon's non-recording rain gauge.

and bottle size, may have capacities of 100, 200, 400 and 1000 mm of rainfall. The 200 mm capacity rain gauge with 200 sq-cm collector and 4 litre bottle, however, is most widely used.

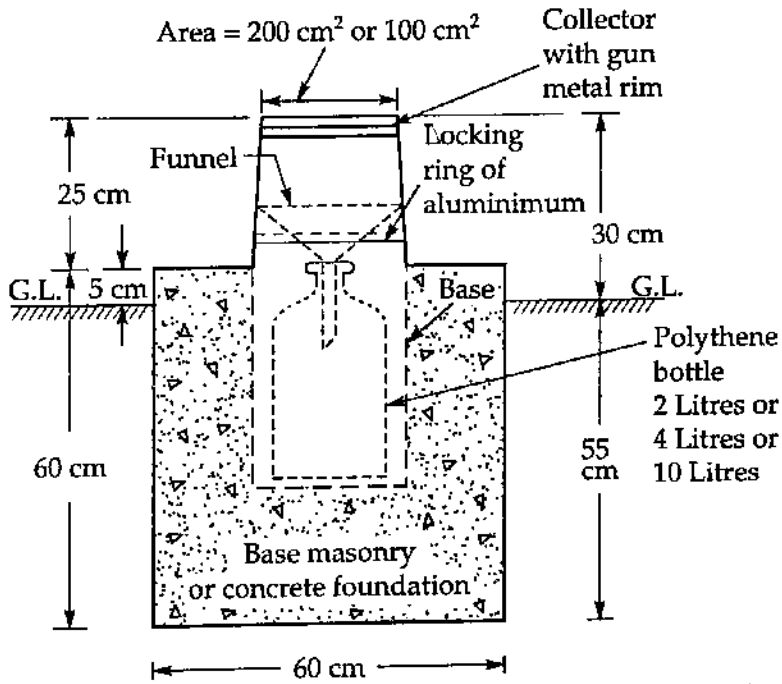
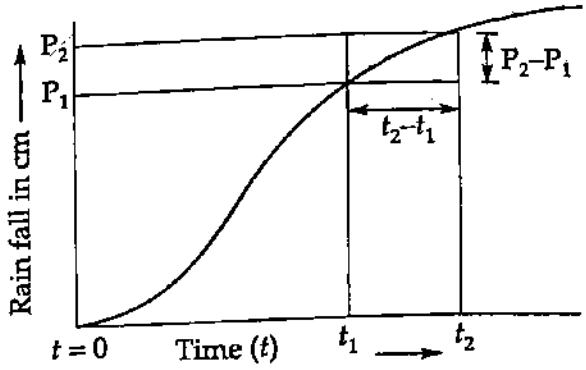


Fig. 3.3. Standard non-recording rain gauge.

When the rain falls, it is collected in the bottle. A man comes daily at 8.30 A.M. (in India) and records the amount of water collected in the bottle, and thus the rainfall is recorded. The process is common throughout this country and rain is recorded at 8.30 A.M. every day. It the rainfall is too much and is likely to exceed the capacity of the bottle, then two or three intermediate readings are taken, and their sum will have to be recorded as the rainfall of the past 24 hours of the day on which the final reading is taken.

(2) **Recording type rain gauges.** Recording type rain gauges are those rain gauges which can give us a permanent automatic rainfall record without any bottle reading. In this type of a gauge, a man has not to go to the gauge to measure or to read the amount of rain fallen. There is a mechanical arrangement by which the total amount of rain, fallen since the record was started, gets recorded automatically on a graph paper. The gauge thus produces a record of cumulative rain *vs* time in the form of a graph, which is known as the *mass curve* of rain fall, and is shown in Fig. 3.4.

Besides giving the total amount of rain fallen at a station, such a curve will also help in indicating the times of onset and cessation of a rain, and thereby indicating its duration. Moreover, the slope of the curve gives us the intensity of rainfall for any given period, as indicated in Fig. 3.4.



$$\frac{P_1 - P_1}{t_2 - t_1} = \frac{dP}{dt} = \text{Rate of rainfall or Intensity of rainfall}$$

Fig. 3.4. Mass curve of Rainfall.

Since such gauges represent the cumulative rain, they are sometimes called the *Integrating rain gauges*. Moreover, such gauges can provide us with continuous recorded measurements for a number of days. They are of great utility in hilly and far off areas, where it is not practically feasible to daily visit the gauge stations. Such gauges are, therefore, also sometimes known, as *Continuous rain gauges*.

A recording rain gauge is generally installed in conjunction with an ordinary non-recording gauge exposed closeby. The nearby installed ordinary gauge will serve as an aid and a standard for checking and adjusting the readings of the recording gauge.

Various models have been designed for recording gauges ; on the basis of which, various gauges, such as "Tipping bucket type", "Weighing type", and "Floating type", etc. are available in the market. Out of these three important types, floating type is most widely used in India at almost all its meteorological stations. The tipping bucket type is, however, used at remote hilly stations, because it can transmit its records, through telecommunication system, directly to the control room at the main meteorological station (Poona in India).

A **float type gauge** (Fig. 3.5), in general, consists of a rotating drum with a graph paper fixed around it. There is a pen point in contact with the graph paper, which moves up with the rise of the float. The float, in turn, rises up with the rain water collected in the gauge chamber, and entering through a funnel. The moving pen thus goes on recording the cumulative rain with the passage of time.

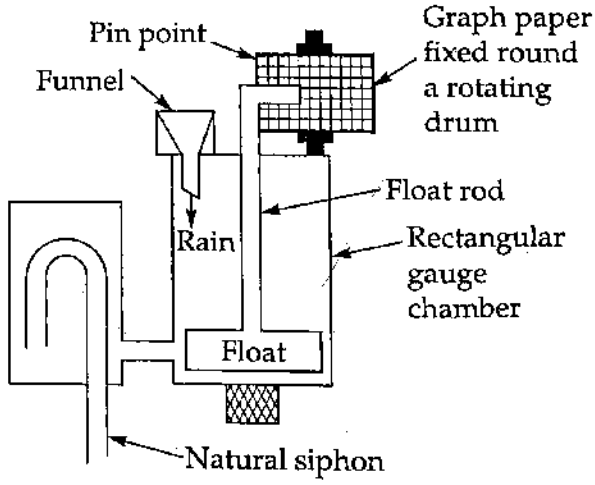


Fig. 3.5. Float type automatic rain gauge.

When the chamber gets filled with water to such an extent that the float touches the top, a natural siphon starts working, and the rain water collected

in the chamber is drained out.

In a **tipping-bucket gauge,** the rain water is first caught in a collector and then passed through a funnel. The funnel discharges the water into a two-compartment bucket. When 0.1 mm of rain water gets filled up in one compartment, the bucket tips, emptying into a reservoir, and moving the second compartment into place beneath the funnel, as shown in Fig. 3.6. The tipping of the bucket completes an electric circuit, causing a pen to mark on a revolving drum. Since the movement of the tipping of the bucket can be transmitted electronically over distances, such gauges are generally installed in hilly and inaccessible areas, from where they can supply measurements directly in the control room, as pointed out earlier.

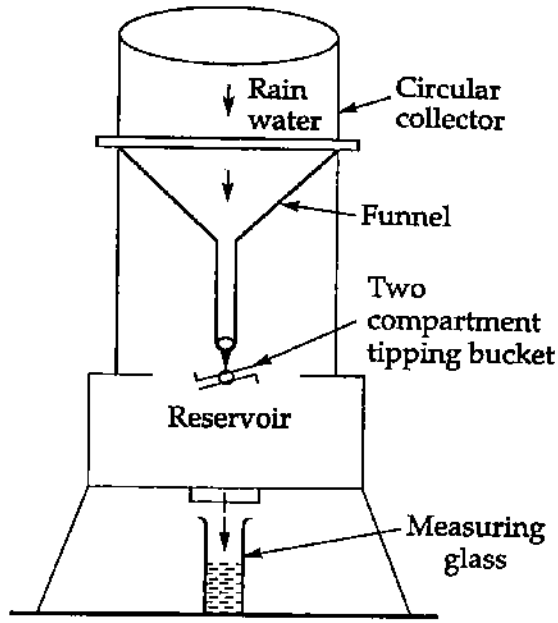


Fig. 3.6. Tipping Bucket type of recording rain gauge.

3.7. Measurement of Snow

Snow is the second important part of precipitation. Although it does not yield immediate run-off, but is generally responsible for delayed supplies of water. Thus, most of our Himalayan rivers are snow fed and are, therefore, in a position to provide continuous perennial supply of water. Snowfall is generally measured by a non-recording type of rain-gauge. The snow is melted and measured. To measure the snow, snow surveys are also conducted. In India, snow surveys are generally conducted in March or April, *i.e.*, at the start of summer season, when snow starts melting. These surveys are useful in estimating the amount of available water expected due to the melting of snow in summer.

3.8. Average Annual Rainfall, Minimum Annual Rainfall, and Index of Wetness

The amount of rain collected by a given rain gauge in 24 hours is known as *daily rainfall*, and the amount collected in one year is known as *annual rainfall*. This annual rainfall at a given station should be recorded over a number of years, say 35 years to 50 years or so. In India, this rainfall cycle period is now a days taken to be 50 years, although earlier it used to be taken as 35 years. The mean of the annual rainfalls over a period of 35-50 years or so, is therefore, known as the *average annual rainfall* at the given station.

Whenever we talk of the rainfall at any given place, we generally refer to the average annual rainfall of that place. Thus, when we speak that Cherrapunji gets 1270 cm of yearly rain, it means that this figure has been averaged over a long period of 35-50 years. This is known as the average rainfall. But in any given particular year, the rain may not fall equal to this amount. It may be less than this average value or may exceed this average

value. The ratio of the actual rainfall in a given particular year at a given place to the normal rainfall of that place is known as the *index of wetness*.

∴ **Index of wetness**

$$= \frac{\text{Actual rainfall in a given year at a given place}}{\text{Normal rainfall of that place}} \quad ...(3.1)$$

Index of wetness, thus gives us an idea of the wetness of the year, and hence, it indicates the deficiency of rain. For example, a 60% index of wetness means a rain deficiency of 40%. A deficiency of about 30 to 45% is known as *large deficiency*, 45 to 60% as *serious deficiency*, and more than 60% as *disastrous deficiency*.

The year in which the rainfall is less than the average annual, is called a ***bad year*** or a *sub-normal year* ; and the year in which the rainfall is more than the average, is called a ***good year***. If the rainfall in a particular year is approximately equal to the annual average value, then it is known as a ***normal year***.

The average annual rainfall of a given place is very useful for planning water supply schemes at that place, because it directly gives us an idea of the probable amount of water that may be available at that place.*

3.8.1. Minimum Annual Rainfall. From the available rainfall data of 35-50 years or so, the minimum annual value is found out. This minimum annual rainfall and the nature of drought conditions are necessary to be determined by the designer of water supply scheme, so as to decide the storage capacity of the reservoir. By studying the rainfall record over a number of years, it is found that the bad years are generally followed or preceded by good years, and it is extremely rare for a bad period to last for more than three years. Due to this reason, while designing storage reservoirs, a period of three successive bad years may be considered sufficient.

3.9. Water Budget of India

The *average annual rainfall of India*, distributed over the entire Indian geographical area, comes out to about 1.19 m* (119 cm), as its total geographical area is 328 million hectares, and total rainfall is about 392 million hectare metres (M. ha-m). This may be rounded off to 400 M. ha-m. including snowfall, which is of the order of 10 M. ha-m, though not fully assessed. It has also been estimated that the moisture present in the atmosphere of this sub-continent (India) is about 770 M. ha-m during monsoons, and 340 M. ha-m during the rest of the year. The total precipitation of 400 M. ha-m, as against the total moisture of 1110 M. ha-m, proves that only about 30 to 35% of moisture present in the atmosphere comes back as rainfall over the sub-continent.

Out of this 400 M. ha-m of total precipitation, it has been estimated that 115 M. ha-m appears as true surface run-off; 70 M. ha-m is lost in evaporation

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*The average annual rainfall values and normal monsoon rainfall values in India, are shown in Fig. 3.7 and 3.8 respectively.

*The corresponding figure for the entire world is 0.86 m.

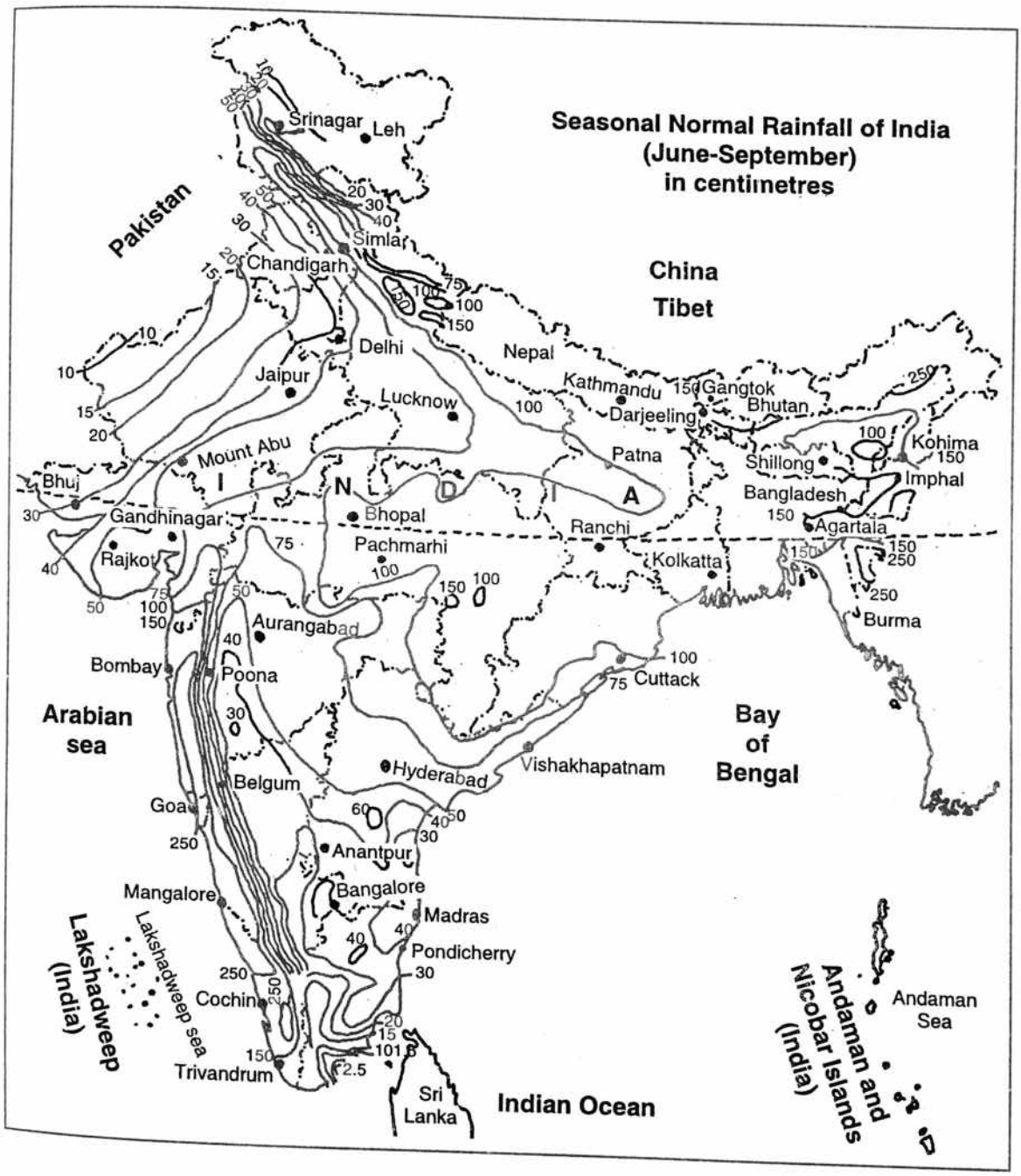

Fig. 3.7. Average annual rainfall in India (in centimetres).

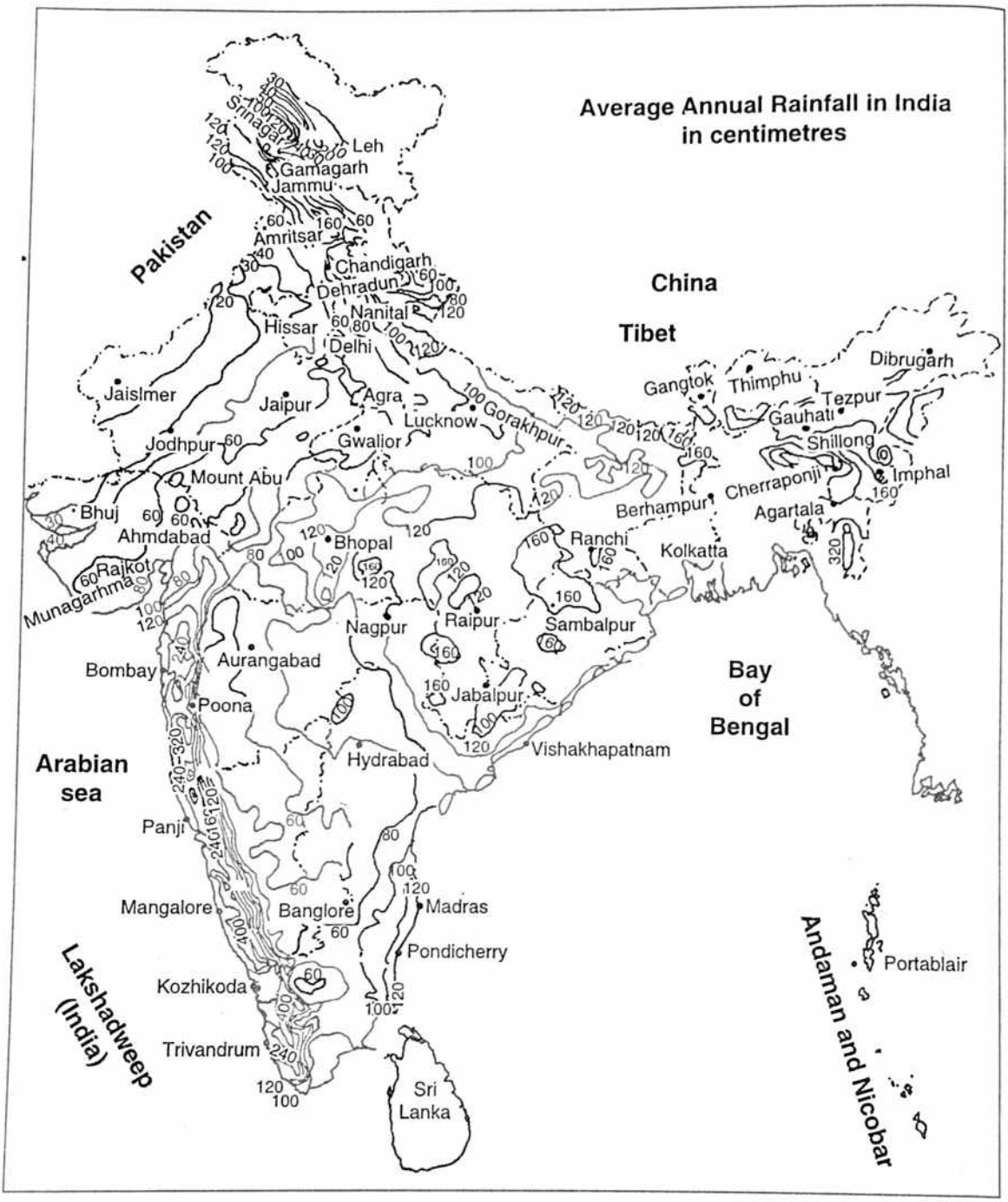

Fig. 3.8. Seasonal normal rainfall of India (June-September) in centimetres.

and transpiration ; and 215 M. ha-m infiltrates into the ground. Out of this 215 M. ha-m of infiltration, 45 M. ha-m regenerates as base flow in rivers ; 50 M. ha-m goes and actually joins the ground-water-table ; and 120 M. ha-m is absorbed as soil-moisture.

In addition to this precipitation of 400 M. ha-m, 10 M. ha-m of runoff comes from areas outside India, thus producing a total runoff or stream flow in India = 115 (SRO) + 45 (Base flow) + 10 (outside India) = 170 M. ha-m. Out of this, only 31 M. ha-m is utilised through pumping/storages in dams, lakes, etc., and 139 M. ha-m goes as waste to join the sea*.

3.10. Characteristics of a Rain Storm

A rainfall at a place can be completely described if its intensity, duration and frequency are known. The intensity of a rain is the rate at which it is falling, the duration is the time for which it is falling with that given intensity, and frequency is the number of times it falls. The relationship and the studies involving intensity, duration and frequency of a storm are a little complex and not of much concern to a water supply engineer and, therefore, have not been dealt here. Their detailed description is however, available in "*Water Resources Engg. Vol. I—Hydrology and Water Resources Engineering*" by the same author. It may, however, be pointed out here that these studies help us in determining peak flood flows in streams or rivers, which may be required to be worked out by the water supply engineers also, in order to design the dam, reservoir, and other hydraulic structures connected with the storage. The foundations of these structures should be designed safe for the maximum scour that is likely to be caused by the peak flows, and the top of the dam should have sufficient freeboard above the high flood level of the expected peak flow. Generally, 50 to 100 years flood discharges (*i.e.* flows occurring with frequencies of 50 years or 100 years) are considered in the design, and the peak discharges are estimated either by empirical formulas or *unit hydrograph method*, as explained in that volume.

3.11. Run-off and Estimation of Run-off

3.11.1. Run-off. As pointed out earlier, the entire rain water falling over the surface of the earth does not reach the surface streams and river channels. A certain part of this water is lost to the atmosphere through evaporation or transpiration**, and certain part which infiltrates into the soil is lost to the ground water reservoir. The remaining water, called the *surface run-off*, flows and joins the surface streams. However, the total flow or run-off in a surface stream is the combination of surface run-off and the run-off derived from the underground water, called ***base flow***. This total flow in the stream called run-off or stream flow or discharge is very important, and has to be evaluated,

*It has been estimated that out of this 139 M. ha-m, which at present goes as waste, 45 M. ha-m can be easily utilised by us by developing the necessary infrastructure.

**Transpiration loss is the water utilised by the plants for their growth, and it virtually evaporates to the atmosphere.

because it directly tells us the amount of water available in the surface sources, and hence their yields.

3.11.2. Yield of a Drainage Basin. The run-off or streamflow is generally expressed as the instantaneous rate or the average rate of flow for short periods (such as cubic metres per sec), but when it is expressed over long periods, such as total volume of water flowing per year (*i.e.* million cubic metres per year) or yearly run-off in cm depth over the catchment (Q_y), it is termed as the yield of the drainage basin. The figure Q_y when multiplied by the catchment area of a stream will, therefore, directly give us the annual volume of water available in that stream. This will be useful in assessing the storage potentials of different catchments for deciding the location of a dam for the construction of storage reservoirs.

3.11.3. Estimating Run-off and Yield of a Basin. As pointed out earlier, the run-off is nothing but the *rainfall minus losses*. These losses include :

(*i*) *Evaporation losses* including transpiration losses, and

(*ii*) *Percolation losses*, called *watershed leakage*.

If these losses can be estimated with a fair degree of accuracy, the run-off and the yield of the basin can be easily obtained by subtracting the losses* from the precipitation.

On the basis of this theory, several investigators have evolved their empirical formulas for finding out the quantity of monthly or yearly run-off. The use of these formulas is, however, restricted only to those areas for which they have been evolved, and cannot be applied to any catchment of choice. Some of these important formulas are given below :

(1) **Inglis formula.** This formula is generally useful for southern parts of India, and states that

(*a*) $Q_y = 0.85\,P_y - 30.4$ for hilly catchments ...(3.2)

(*b*) $Q_y = P_y \left[\dfrac{P_y - 17.8}{254}\right]$ for plain areas ...(3.3)

where Q_y = yearly run-off in cm depth over the basin.

P_y = yearly precipitation or Annual rainfall in cm.

(2) **Khosla's formula.** This general formula can be used for any catchment, and is applicable to both India as well as U.S.A., where the rainfall is fairly uniformly distributed throughout the year. This formula is very useful, and states that

$$Q = P - 0.4813\,T_m \text{ (when } T_m > 4.5°\text{C)} \qquad ...(3.4)$$

where Q = Yearly or monthly run-off in cm.

P = Yearly or monthly rainfall in cm.

T_m = Mean yearly or monthly temperature in degree centigrades.

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*A detailed description and estimation of these losses is available in "*Water Resources Engg. Vol. I—Hydrology and Water Resources Engineering*" by the same author.

(3) **Justin's formula.** This formula is useful for the eastern parts of the U.S.A., and states that

$$Q_y = 0.368 \left[\frac{P_y^2}{1.8T_y + 32}\right]\left[\frac{H}{\sqrt{A}}\right]^{0.155} \quad ...(3.5)$$

where Q_y = yearly run-off in cm

P_y = yearly precipitation in cm

T_y = mean annual temperature in °C

H = Maximum difference of elevation within the catchment in metres

A = the area of the catchment in sq. metres.

Note : $\frac{H}{\sqrt{A}}$ represents the slope in the catchment.

(4) **Vermule's formula.** This formula is useful for the mountainous districts along the Atlantic Coast of U.S.A. It states that

$$Q_y = P_y - \left[0.279 - 0.116\, P_y\right]\left[0.063\, T_y + 0.47\right] \quad ...(3.6)$$

where Q_y, P_y and T_y have the same meaning as given above.

(5) **Run-off Co-efficient formula.** The run-off can be determined by using the direct rainfall run-off formula, *i.e.* $Q = KP$. For yearly values, this formula would become

$$Q_y = K.P_y \quad ...(3.7)$$

where Q_y = yearly run-off in cm

P_y = yearly precipitation in cm

K = a constant, the usual values of which are given in Table 3.1.

Table 3.1. Values of Run-off Coefficient (K)

Types of area,	*Values of K*
Single houses	0.3
Garden apartments	0.5
Commercial and Industrial areas	0.9
Forested areas depending on soil	0.05—0.20
Parks, farm land, pastures, etc.	0.05—0.30
Asphalt or concrete pavements	0.85

(6) **Barlow's Tables.** Based upon the investigation carried out on the catchments under 140 sq kilometres in United Provinces, T.B. Barlow has given the values of K (in percentage) for various classified catchments, as follows :

Table 3.2. Values of K given by Barlow to be used In Eq. (3.7), i.e. $Q_y = KP_y$

S. No.	*Class of the catchment*	*Description of the catchment*	*Runoff per cent, i.e. K*
1.	*A*	Flat, cutlivated and black cotton soils	10%
2.	*B*	Flat, partly cultivated, various soils	15%
3.	*C*	Average	20%
4.	*D*	Hills and plain with little cultivation	35%
5.	*E*	For hilly and steep areas with hardly any cultivation	45%

The above values of run-off percentages are for average monsoons, and will help in yearly run-off determination. However, when monthly or daily values are desired, then the above value of K should be multiplied by the coefficient given in table 3.3, which depends on the type of rainfall.

Table 3.3

S. No.	*Nature of season*	*Class of catchment*				
		A	*B*	*C*	*D*	*E*
1.	Light rain, no heavy downpour	0.7	0.8	0.8	0.8	0.8
2.	Average or varying rainfall, no continuous downpour	1.0	1.0	1.0	1.0	1.0
3.	Continuous downpour	1.5	1.5	1.6	1.7	1.8

(7) **Lacey's formula.** It states that

$$Q_y = \left[\frac{P_y}{1 + \frac{304.8\, m}{P_y\, n}} \right] \qquad ...(3.8)$$

where Q_y = yearly run-off in cm

P_y = yearly precipitation in cm

m = a constant called ***monsoon duration factor***, the values of which are given in Table 3.4

n = a constant called ***catchment factor***, the values of which are given for different types of catchments in Table 3.5.

Table 3.4. Values of m to be used in Eq. (3.8)

S. No.	*Duration of Monsoon,*	*Monsoon duration factor (m)*
1.	Very short *i.e.* bad year	0.5
2.	Standard length *i.e.* normal year	1.2
3.	Very long *i.e.* good year	1.5

Table 3.5. Values of n to be used in Eq. (3.8)

S. No.	*Class of catchment*	*Value of n*
1.	*A*	0.25
2.	*B*	0.60
3.	*C*	1.00
4.	*D*	1.70
5.	*E*	3.45

3.11.4. Streamflow Measurements and the Yield of Surface Streams. The various empirical formulas discussed above give us the yearly run-off, and are, therefore, helpful in evaluating the amount of water available in the river over long periods. They are, therefore, useful for estimating the storage capacity of the reservoir, and thus fixing the height of the dam required to be constructed for that storage. These determinations are hence useful when a storage reservoir is to be used as a source of water. *However, when a river or a stream is to be directly used as a source of water (without any storage reservoir), we must determine the day to day quantities of flow in that river or stream.* These daily flows and their adjustments are determined by actual observations and measurements over long periods. The detailed description of this is again available in "*Water Rsources Engg. Vol. I—Hydrology and Water Resources Engg.*" in the chapter on "Stream-flow and Stream Gauging". However, a few important points are reproduced here.

The daily discharges of the stream at the intended site of the water supply intake point, are determined and recorded over a period of number of years. These discharge measurements can be carried out by various methods, such as

(*i*) Velocity area method or Stage-discharge relationship method (generally adopted).

(*ii*) By the use of a weir or a spillway of any existing dam.

(*iii*) By the use of control meters, such as venturiflumes, etc. (this is quite useful).

(*iv*) By using power plant records (this is an indirect method and may have to be adopted when no velocity area station, control meter, etc. is available, but some power plant does exist on the stream or river in question).

The first method, *i.e.* the ***velocity area method*** is the most important, and can be easily adopted for planning water supply projects. In this method, the velocity of water is measured by a current meter, etc., and the river cross-section is measured up to the existing water level (gauge) in the river. Velocity multiplied by area gives us the discharge.

The daily flow measurements recorded by any of these methods are properly corrected and adjusted for the possible changes or shifts in the river control or stage discharge curve, etc. The total weekly, monthly or yearly yields of the streams can then be computed by adding the daily discharges.

SURFACE SOURCES OF WATER SUPPLIES

Surface sources are those sources of water in which the water flows over the surface of the earth, and is thus directly available for water supplies. The important of these sources are :

(*i*) *Natural ponds and lakes* ;

(*ii*) *Streams and rivers* ; and

(*iii*) *Impounding reservoirs.*

These sources are discussed below :

3.12. Ponds and Lakes as Surface Sources of Supplies

A natural large sized depression formed within the surface of the earth, when gets filled up with water, is known as a pond or a lake. The difference between a pond and a lake is only that of size. If the size of the depression is comparatively small, it may be termed as a *pond,* and when the size is larger it may be termed as a *lake.* The flow of water in a lake is just like the flow in a stream channel. Generally, the surface run-off from the catchment area contributing to the particular lake, enters the lake through small drains or streams. Sometimes, the underground water through some spring, also enters natural depressions and gets collected there, forming ponds and lakes.

The *quality* of water in a lake is generally good and does not need much purification. Larger and older lakes, however, provide comparatively purer water than smaller and newer lakes. Self-purification of water due to sedimentation of suspended matter, bleaching of colour, removal of bacteria, etc. makes the lake's water purer and better. On the other hand, in still waters of lakes and ponds, the algae, weed and vegetable growth take place freely, imparting bad smells, tastes and colours to their waters.

The *quantity* of water available from lakes is, however, generally small. It depends upon the catchment area of the lake basin, annual rainfall, and geological formations. Due to the smaller quantity of water available from them, lakes are usually not considered as principal sources of water supplies. They are, therefore, useful for only small towns and hilly areas. However, when no other sources are available, larger lakes may become the principal sources of supplies. For example, in Bombay city, water is supplied and brought from lakes about 70 km from there.

3.13. Streams and Rivers as Surface Sources of Supplies

Small stream channels feed their waters to the lakes or rivers. *Small streams* are, therefore, generally not suitable for water supply schemes, because the quantity of water available in them is generally very small, and they may even sometimes go dry. They are, therefore, useful as sources of water only for small villages, especially in hilly regions. Larger and perennial streams may, however be used as sources of water, by providing storage reservoirs, barrages, etc. across them.

Rivers are the most important sources of water for public water supply schemes. It is a well known fact that most of the cities are settled near the rivers, and it is generally easy to find a river for supplying water to the city. Rivers may be perennial or non-perennial. Perennial rivers are those in which

the water is available throughout the year. Such rivers are generally fed by rains during rainy season and by snow during summer season. Perennial rivers can be used as sources of public supplies directly, whereas the non-perennial rivers can be used as sources of public supplies by providing storage on the upstream of the intake works. The construction of a dam is generally adopted on a highly non-perennial river and may be adopted even on a perennial river when water is used for multiple uses such as irrigation, hydropower, etc. The head works, such as a barrage or a weir, may also be constructed on those perennial rivers, where supplies are considerably reduced during dry weather periods.

The *quality* of water obtained from rivers is generally not reliable, as it contains large amounts of silt, sand and a lot of suspended matter. The disposal of the untreated or treated sewage into the rivers is further liable to contaminate their waters. *The river waters must, therefore, be properly analysed and well treated before supplying to the public.*

3.14. Storage Reservoirs as Surface Sources of Supplies

A water supply scheme drawing water directly from a river or a stream may fail to satisfy the consumers demands during extremely low flows ; while during high flows, it may again become difficult to carry out its operations due to devastating floods. A barrier in the form of a dam may, therefore, sometimes be constructed across the river, so as to form a pool of water on the upstream side of the barrier. This pool or artificial lake formed on the upstream side of the dam is known as the storage reservoir. The quality of this reservoir water is not much different from that of a natural lake.

The water stored in the reservoir can be used easily not only for water supplies but also for other purposes. Generally, multipurpose reservoirs are planned these days and operated so as to get optimum benefits. The subject of design and planning of dams and reservoirs is a big topic in itself, and is generally dealt under the subject of Irrigation.* However, its salient features such as : Selection of dam site and types of dam ; Storage capacity of reservoirs, Reservoir sedimentation, Reservoir losses, etc. are, however being reproduced here.

3.14.1. Selection of Dam Site. The selection of a site for constructing a dam should be governed by the following factors :

(1) Suitable foundations must be available.

(2) For economy, the length of the dam should be as small as possible, and for a given height it should store the maximum volume of water. It, therefore, follows that the river valley at the dam site should be narrow but should open out upstream to

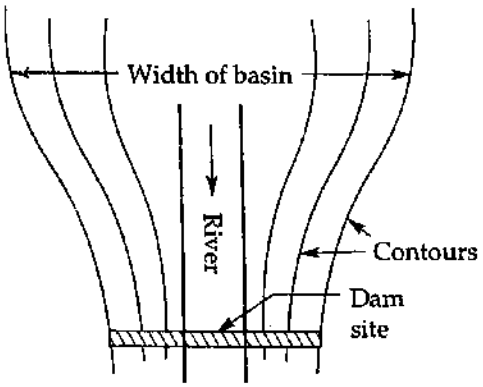


Fig. 3.9. Contour map of a suitable site for a dam.

*For detailed description please refer "*Water Resources Engg. Vol. II—Irrigation Engineering and Hydraulic Structures*" by the same author.

provide a large basin for a reservoir. A general configuration of contours of a suitable site is shown in Fig. 3.9.

(3) The general bed level at dam site should preferably be higher than that of the river basin. This will reduce the height of the dam and will facilitate the drainage problem.

(4) A suitable site for the spillway should be available in the near vicinity. If the spillway is to be combined with the dam, the width of the gorge should be such as to accommodate both.

The best dam site is the one, in which a narrow deep gorge is separated from the flank by a hillock with its surface above the dam, as shown in Fig. 3.10.

If such a site is available, the spillway can be located separately in the flank, and the main valley is spanned by an earthen or a similar dam. Sometimes, the spillway and concrete masonry dam may be compositely spanned in the main gorge, while the flanks are built in earth at low cost.

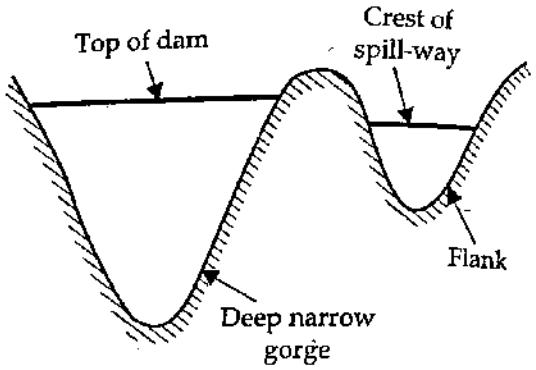


Fig. 3.10. Best dam site having a narrow gorge with a side flank.

(5) Materials required for the construction should be easily available, either locally or in the near vicinity, so that the cost of transporting them is as low as possible.

(6) The reservoir basin should be reasonably water-tight. The stored water should not escape out through its side walls and bed.

(7) The value of land and property submerged by the proposed dam should be as low as possible.

(8) The dam site should be easily accessible, so that it can be economically connected to important towns and cities by rails, roads, etc.

(9) Site for establishing labour colonies and a healthy environment should be available in the near vicinity.

3.14.2. Various Kinds of Dams. Most engineers recognise seven general types of dams. Three of them are ancient in origin and four have come into general use only in the last about 100 years or so.

The three older types of dams are :

1. *Earth dams* ;
2. *Rock-fill dams* ; and
3. *Solid masonry gravity dams* ; as discussed below :

(1) **Earth dams** are made of soil that is pounded down solidly. They are built in areas where the foundation is not strong enough to bear the weight of a concrete dam, and where earth is more easily available as a building material compared to concrete or stone or rock.

Some important *earth dams* of the world are :

(*i*) *Green mountain dam* of Colorado in U.S.A.

(*ii*) *Swift dam* in Washington in U.S.A.

(*iii*) Side flanks of *Nagarjun Sagar dam* in India.

(*iv*) *Trinity dam* in California in U.S.A.

(*v*) *Maithan dam* in India (it is partly earthen and partly rock fill).

(2) **Rock fill dams** are formed of loose rocks and boulders piled in the river bed. A slab of reinforced concrete is often laid across the upstream face of a rock fill dam to make it water-tight.

Some important *rock-fill dams* are :

(*i*) The *Salt Springs dam* in California (345′ high) in U.S.A.

(*ii*) The *San Gabriel No. 1 dam* (321′ high) in U.S.A.

(*iii*) *Cougar dam* on McKnezie River in Oregon (445′ high) in U.S.A.

(3) **Solid-masonry gravity dams** are made of stone masonry or concrete. The examples are : *Aswan, Roosevelt, Hoover*, and above all *Bhakra dam.*

These big dams are expensive to build but are more durable and solid than earth and rock dams. They can be constructed on any dam site, where there exists a natural foundation strong enough to bear the great weight of the dam.

These three types of dams were all found in ancient days. In recent times, *four other types of dams have come into practice.* They are :

Hollow masonry gravity dams ;

Timber dams ;

Steel dams ; and

Arch dams, as discussed below :

(4) **The hollows masonry gravity dams** are essentially designed on the same lines on which the solid masonry gravity dams are designed. But they contain less concrete or masonry ; about 35 to 40% or so. Generally, the weight of water is carried by a deck of R.C.C. or by arches that share the weight burden. They are difficult to build, and adopted only if very skilled labour is easily available, otherwise the labour cost is too high to build its complex structure.

(5) **Steel dams** are not used for major works. Today, steel dams are used as temporary coffer dams needed for the construction of permanent dams. Steel coffer dams are usually reinforced with timber or earth fill.

(6) **Timber dams** are short lived, since in a few years time, rotting sets in. Their life is not more than 30 to 40 years, and must have regular maintenance during that time. However, they are valuable in agricultural areas, where a cattle raiser may need a pool for his live stock to drink from, and for other such low-level needs.

(7) **Arch dams** are somewhat complex. They make use of the horizontal arch action in place of weight, to hold back the water. They are best suited at sites where the *dam must be extremely high and narrow, with the availability of very strong side walls of the canyon.* Some examples are :

(*i*) *Sautet dam* on the Drac River in France, 414′ high, but only 230′ long at top and 85′ long at bottom of the gorge, 56′ thick at bottom and 8′ thick at top.

(*ii*) *The Tignes dam* in the France (592′ high).

(*iii*) *Mautvoisin dam* on the Drause River in Switzerland (780′ high).

(*iv*) The *Idukki dam in India*—the only arch dam in our country (1200′ long and 560′ high) on the Periyar river in Kerala State of India.

3.14.3. Storage Capacity of Reservoirs. Whatever be the size or use of a reservoir, the main function of a reservoir is to store water, and thus to stabilize the flow of water. Therefore, the most important physical characteristic of a reservoir is noting but is it *storage capacity*. The capacity of reservoirs on dam sites is determined from the contour maps of the area. A topographic survey of the dam site is carried out and a contour map, such as shown in Fig. 3.11 is drawn. The area enclosed within each contour within the

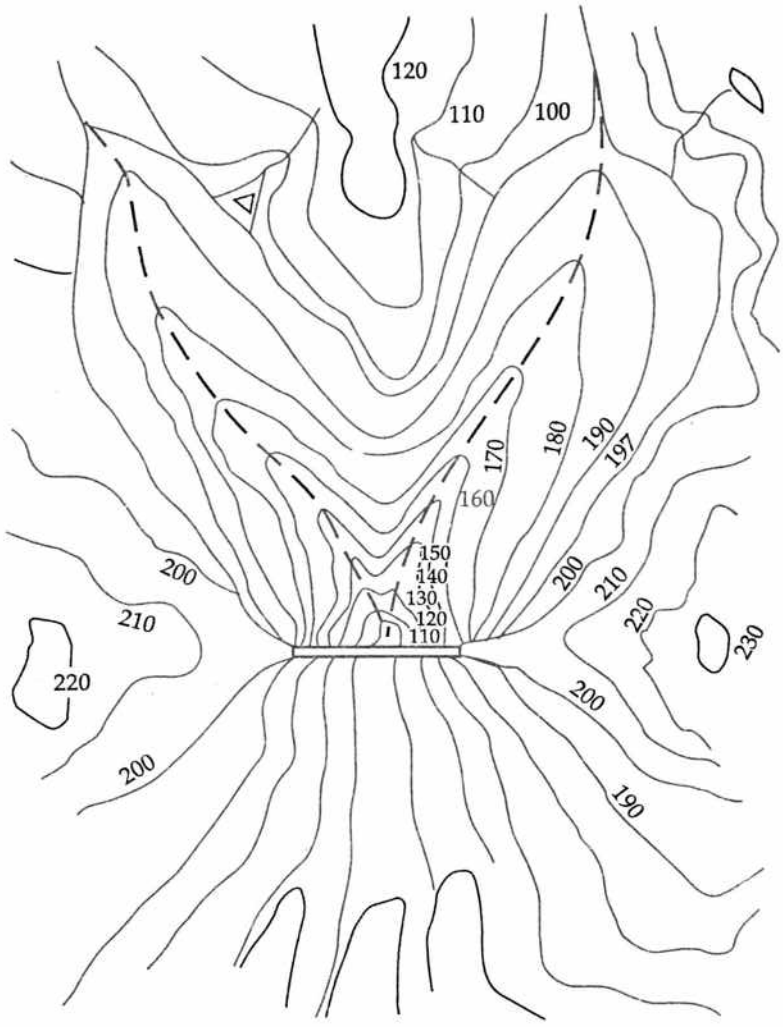


Fig. 3.11. A typical contour map of a dam site.

reservoir site can be measured with a planimeter. The contour elevations and the areas enclosed at respective elevations can be plotted in the form of a curve, called ***Area-Elevation Curve*** (Fig. 3.12). The integral of the area-elevation curve is nothing but ***elevation-storage*** or ***elevation capacity curve*** as shown (Fig. 3.10). The volume of water that can be stored by the reservoir at a certain water surface elevation can be computed after determining the increment of storage between two elevations (ΔS). The increment

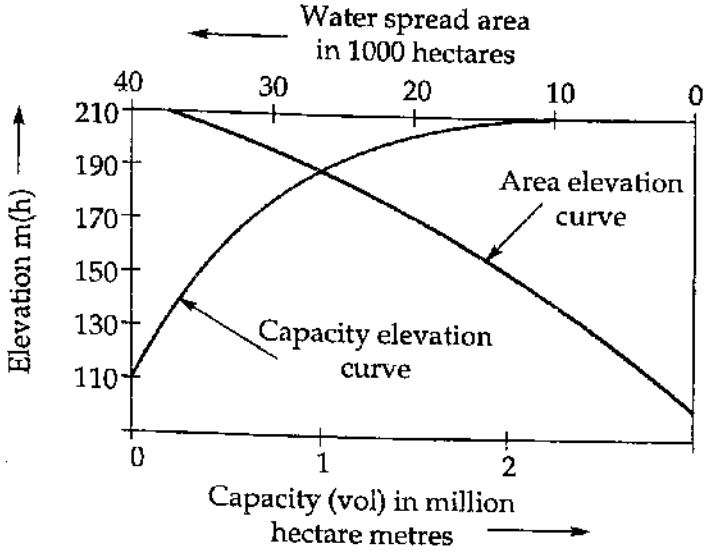


Fig. 3.12. The Area-Elevation Curve.

of storage (ΔS) between two elevations is usually computed by multiplying the average of the area at the two elevations, by the elevation difference (Δh). The summation of these increments below any elevation is the storage volume below that level.

In the absence of topographic maps, the cross-sections of the reservoir can be surveyed, and the capacity computed from these vertical cross-sections by using Trapezoidal or Prismoidal formulas ; preferably by Prismoidal formula, as follows :

$$\text{Volume } (V) = h\left[\frac{A_1 + A_n}{2} + A_2 + A_3 + \ldots + A_{n-1}\right]$$

...(Trapezoidal formula) ...(3.9)

$$= \left[(A_1 + A_n) + 4(A_2 + A_4 + \ldots) + 2(A_3 + A_5 + \ldots)\right]$$

...(Prismoidal formula) ...(3.10)

where A_1, A_2, A_3, A_4, ... are the areas enclosed between successive elevation lines having a height interval of h.

The storage capacities for various water surface elevations can be worked out and plotted, so as to obtain *elevation-capacity* or *elevation-storage* curve.

3.14.4. Storage Zones of a Reservoir. Various storage zones of a reservoir are described below w.r. to Fig. 3.13.

Normal pool level or Maximum conservation level is the maximum elevation to which the reservoir water surface will rise during normal operating conditions. It is equivalent to the elevation of spillway crest or the top of the spillway gates for most of the cases.

Minimum pool level. The lowest water surface elevation which has to be kept under normal operating conditions in a reservoir, is called the minimum pool level. This level may be fixed by the elevation of the lowest outlet in the dam, may be guided by the minimum head required for the efficient functioning of turbines.

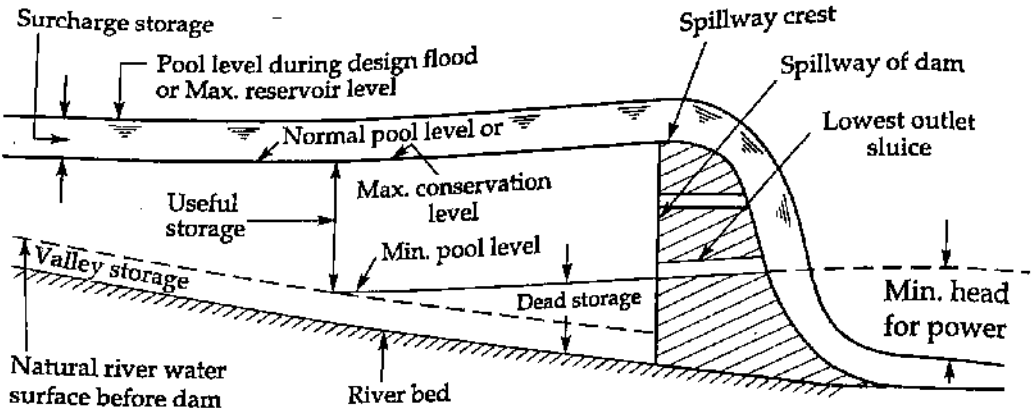


Fig. 3.13. Storage zones in a reservoir.

Useful and Dead storage. *The volume of water stored in the reservoir between the minimum pool level and the normal pool level is called the Useful Storage.* Water stored in the reservoir below the minimum pool level is known as *Dead Storage*, and it is not of much use in the operation of the reservoirs. The useful storage may be subdivided into **conservation storage** and **flood mitigation storage**, in a multipurpose reservoir.

Maximum pool level. During high floods, water is discharged over the spillway, but will cause the water level to rise in the reservoir above the normal pool level. The max. level to which water rises during the worst design flood is known as the *maximum pool level*.

Surcharge storage. The volume of water stored between the normal pool level and the maximum pool level is called the *Surcharge* storage. Surcharge storage is an uncontrolled storage, in the sense, that it exists only till the flood is in progress and cannot be retained for later use.

Bank storage. When the reservoir is filled up, certain amount of water seeps into the permeable reservoir banks. This water comes out as soon as the reservoir gets depleted. This volume of water is known as the *Bank storage* and may amount to several per cent of the reservoir volume depending upon the geological formations. The bank storage effectively increases the capacity of the reservoir above that indicated by the elevation capacity curve of the reservoir.

Valley storage. Even before a dam is constructed, certain variable amount of water is stored in the stream channel called the *Valley storage*. After the reservoir is formed, the storage increases and the actual net increase in the storage is equal to the storage capacity of the reservoir minus the natural valley storage. The valley storage thus reduces the effective storage capacity of a reservoir. It is not of much importance in conservation reservoirs, but the available storage for flood mitigation is reduced, as given by the following relation.

Effective storage for flood mitigation

= Useful storage + Surcharge storage

– Valley storage corresponding to the rate of inflow to the reservoir.

3.14.5. Catchment Yield and Reservoir Yield. Long range runoff from a catchment is known as the yield of the catchment. Generally, a period of one year is considered for determining the yield value. *The total yearly runoff, expressed as volume of water entering/passing the outlet point of the catchment, is thus known as the catchment yield, and is expressed in M m^3 or M. ha.m.*

The annual yield of the catchment upto the site of a reservoir, located at the given point along a river, will thus indicate the quantum of water that will annually enter the reservoir, and will thus help in designing the capacity of the reservoir. This will also help to fix the *outflows* from the reservoir, since the outflows are dependent upon the inflows and the reservoir losses.

The amount of water that can be drawn from a reservoir in any specified time interval, called the **reservoir yield**, naturally depends upon the inflow into the reservoir, and the *reservoir losses* consisting of *reservoir leakage* and *reservoir evaporation.*

The annual inflow to the reservoir, called the **catchment yield**, is represented by the *mass curve of inflow* ; whereas, the outflow from the reservoir, called the **reservoir yield**, is represented by the *mass demand line* or *mass curve of outflow*. Both these curves decide the reservoir capacity, provided the reservoir losses are separately accounted.

The inflows to the reservoir are however, quite susceptible of variation in different years, and may therefore vary throughout the prospective life of the reservoir. *The past available data of rainfall or runoff in the catchment is therefore used to work out the optimum value of the catchment yield.* Say for example, in the past available records of say 35 years, the minimum yield from the catchment in the worst rainfall year may be as low as say 100 M. ha.m ; whereas, the maximum yield in the best rainfall year may be as high as say 200 M. ha.m. The question which then arises would be as to whether the reservoir capacity should correspond to 100 M. ha.m yield or 200 M. ha.m yield. If the reservoir capacity is provided corresponding to 100 M. ha.m yield, then eventually the reservoir will be fully filled up every year with a dependability of 100% ; but if the capacity is provided corresponding to 200 M. ha.m yield, then eventually the reservoir will be filled up fully only in the best rainfall year (*i.e.* once in 35 years or so) with a dependability of about $\frac{1}{35} \times 100 = 3\%$.

In order to obtain a sweet agreement, a via media is generally adopted and an intermediate *dependability percentage value* (p), such as 50% to 75%,* may be used to compute the **dependable yield** or **design yield.** The yield which correspond to the worst or most critical year on record is however, called the **firm yield** or the **safe yield**. Water available in excess of the firm yield during years of higher inflows, is designated as the **secondary yield.** Hydropower may be developed from such secondary water, and sold to industries *'on and when available basis'*. The power commitments of domestic consumers must,

*This percentage will depend upon the risk which can be absorbed for the proposed uses of water. Say for example, city water supply projects can absorb lesser risk as compared to irrigation projects, and hence should consider higher dependability percentage value.

however, be based on firm basis, and should not exceed the power which can be produced with the firm yield, unless thermal power is also available to support the hydroelectric power.

The arithmetic average of the firm yield and the secondary yield is called the **average yield.**

3.14.5.1. Computing design or dependable yield value. The dependable yield, corresponding to a given dependability percentage p, is determined from the past available data of the last 35 years or so. The yearly rainfall data of the reservoir catchment is generally used for this purpose, since such long runoff data is rarely available. *The rainfall data of the past years is therefore used to work out the dependable rainfall value corresponding to the given dependability percentage p, and this rainfall value is then converted into runoff value by using the available empirical formulas (connecting yearly rainfall with yearly runoff), already given in article 3.11.3.*

The procedure which is adopted to compute the ***dependable rainfall value*** for a given dependability percentage p is explained below, and has been further used in solving example 3.1.

(*i*) The available rainfall data of the past N years is first of all arranged in descending order of magnitude.

(*ii*) The *order number* m, given by the equation

$$m = N \cdot \frac{p}{100} \text{ (where } p \text{ is the dependability percentage)} \quad ...(3.11)$$

is then computed, and rainfall value corresponding to this order number in the tabulated data, will represent the required dependable rainfall value.

(*iii*) If the computed value of m is a fraction, then the arithmetic mean of rainfall values corresponding to whole number m values above and below this fraction value is taken as the dependable rainfall value.

Example 3.1. *The yearly rainfall data for the catchment of a proposed reservoir site for 35 years is given in Table 3.6. Compute from this data, the value of dependable rainfall for a 60% dependability percentage.*

Table 3.6

Year	*Rainfall in cm*	*Year*	*Rainfall in cm*
1956	98	1974	88
1957	100	1975	94
1958	101	1976	107
1959	99	1977	110
1960	85	1978	208
1961	112	1979	114
1962	116	1980	104
1963	78	1981	102
1964	160	1982	108
1965	66	1983	102
1966	184	1984	80
1967	90	1985	109

Contd...

Table 3.6 Contd.

Year	*Rainfall in cm*	*Year*	*Rainfall in cm*
1968	76	1986	122
1969	118	1987	115
1970	86	1988	140
1971	92	1989	138
1972	96	1990	60
1973	93		

Solution. The given data of table 3.6 is arranged in descending order, mentioning serial number (order number m) infront of each, as shown in table 3.7.

Table 3.7

S. No. i.e. order number (m)	*Rainfall in descending order in cm*	*S. No. i.e. order number (m)*	*Rainfall in descending order in cm*
1	208	19	101
2	184	20	100
3	160	21	99
4	140	22	98
5	138	23	96
6	122	24	94
7	120	25	93
8	118	26	92
9	116	27	90
10	115	28	88
11	114	29	86
12	112	30	85
13	110	31	80
14	109	32	78
15	108	33	76
16	107	34	66
17	104	35	60
18	102		

Now, using equation (3.11), we compute order number (m) corresponding to given dependability percentage p = 60%, as :

$$m = N \cdot \frac{p}{100} = 35 \times \frac{60}{100} = 21.$$

The rainfall value tabulated above in table 3.7 at order no. 21 is 99 cm ; and hence the required dependable rainfall = **99 cm. Ans.**

Example 3.2. *The design annual rainfall for the catchment of a proposed reservoir has been computed to be 99 cm. The catchment area has been estimated to have the mean annual temperature of 20°C. The catchment area contributing to the proposed reservoir is 1000 sq. km. Calculate the annual design yield for this reservoir. Make use of Khosla's formula.*

Solution. Use Khosla's formula connecting design rainfall P = 99 cm with design yield (Q) by Eq. (3.4) as :

$$Q = P - 0.4813\, T_m \text{ (when } T_m > 4.5°C)$$

where P = 99 cm (given)

$$T_m = 20°C \text{ (given)}$$

$$\therefore \quad Q = 99 - 0.4813 \times 20$$

$$= 89.4 \text{ cm} = 0.894 \text{ m}.$$

The total yield produced in m^3 from the given catchment of 1000 sq. km (*i.e.* 1000×10^6 m^2)

$$= 0.894 \times 1000 \times 10^6 \text{ m}^3 = 89400 \text{ ha.m.}$$

$\therefore$ Annual design yield = 89400 ha.m. **Ans.**

3.14.6. Fixation of Reservoir Capacity and Maximum Water Level (Full Tank Level) From Dependable Yield. After computing the dependable annual yield of a reservoir (ha.m) as above, the annual water demand (ha.m) is assessed from the irrigation, water supply, and other needs of the area to be served by the proposed reservoir. The estimated dead storage for the reservoir in ha.m is also assessed and added to the water demand to obtain gross storage required to fulfil the demand. The storage capacity of the reservoir is now fixed at lesser of the :

(*i*) dependable yield ; and

(*ii*) gross storage required to fulfil demand and dead storage.

The MWL of the reservoir can now be fixed to provide the above capacity from the elevation-capacity curve of the reservoir.

3.14.7. Relation between Inflow, Outflow, and Capacity of a Storage Reservoir. The *catchment yield* (inflow to the reservoir) and the *outflow* from the reservoir (reservoir yield) are the only two factors which govern the storage capacity of a reservoir. Since the inflow to the reservoir is variable, water is stored in the reservoir to cater to the required outflow from the reservoir, particularly during the critical periods in non-monsoon season. Naturally, if more outflow is required, more capacity has to be provided. The provided capacity is, however, further governed by the inflow (yield), since the reservoir must receive enough discharge to fill the provided storage. If the inflow is insufficient, even the huge capacity, if provided for a reservoir, may go waste. In order to ensure a realistic design of the reservoir capacity, the actual inflows at the site of the proposed reservoir should be assessed over a number of years, rather than basing the design on the empirical formulas (as done in the previous article). The annual inflows (minimum of each year) are thus tabulated along with assessing the annual demand. The reservoir capacity is then designed to meet the above inflow and demand, under the basic storage equation, given as :

$$\text{Inflow} - \text{Outflow} = \text{Increase in storage } i.e. \text{ storage created} \qquad ...(3.12)$$

3.14.8. Capacity of a Storage Reservoir for the given Inflow and the given outflow. The capacity of a reservoir is determined by determining the storage needed to accommodate the given inflow with the given outflow, as governed by the above equation. However, this study involves numerous factors, as discussed below :

Streamflow data at the reservoir site must be known. Monthly inflow rates are sufficient for large reservoirs, but daily data may be required for small reservoirs. When the inflow data at the dam site are not known, the data at a station elsewhere on the stream or on a nearby stream may be collected and

adjusted to the dam site. The available data may sometimes be extended, so as to include a really critical drought period.

Besides determining the streamflow data at the dam site, an adjustment has to be made for the water required to be passed from the reservoir to satisfy the prior water rights and to obey the agreements between the various sharing States through which the river is passing.

Moreover, the construction of a reservoir increases the exposed area of the water surface than that of the natural stream, and thus, increases the evaporation losses. There is, thus, a net loss of water occurring due to reservoir construction. Sometimes, the losses may be so huge that the entire purpose of the reservoir may be defeated. Seepage from the reservoir may also add to the loss resulting from the reservoir.

All these factors make this study very very complex. An approximate easy solution for determining the reservoir capacity may be obtained graphically with the help of *mass curves* as explained in the next article. On the other hand, a tabular solution is necessary in order to account for all important factors. For more precise results and for all complex systems, computers may be used for programming the analysis (called operation study).

Example 3.3. *Monthly inflow rates during a low-water period at the site of a proposed dam are tabulated in Col. (2) table 3.8. The corresponding monthly pan evaporation and precipitation at a nearby station are also tabulated in Col. (3) and Col. (4) of the same table. Prior water rights make it obligatory to release the full natural flow or 15 hectare-metres per month, whichever is minimum. If the estimated monthly demands are as given in Col. (5) table 3.8, and the net increased pool area is 400 hectares, find the required storage capacity for the reservoir. Assume pan evaporation coefficient = 0.7 and also assume that only 28% of the rainfall on the land area to be flooded by the reservoir has reached the stream in the past.*

Table 3.8

Month (1)	*Inflow at dam site in hectare-metres* (2)	*Pan evaporation in cm* (3)	*Precipitation in cm* (4)	*Demand in hectare-metres* (5)
January	*1.2*	*1.8*	*1.3*	*15.8*
February	*0.0*	*1.8*	*1.7*	*14.3*
March	*0.0*	*2.6*	*0.6*	*9.6*
April	*0.0*	*10.2*	*0.0*	*4.8*
May	*0.0*	*15.4*	*0.0*	*3.5*
June	*0.0*	*16.6*	*1.1*	*3.4*
July	*240.0*	*10.8*	*16.1*	*5.0*
August	*480.0*	*11.7*	*16.4*	*5.0*
September	*1.0*	*10.8*	*2.2*	*10.0*
October	*0.6*	*9.6*	*0.8*	*15.6*
November	*0.5*	*7.8*	*0.0*	*16.8*
December	*0.2*	*2.0*	*0.0*	*16.8*
Σ	*723.5*	*101.1*	*40.2*	*120.6*

Table 3.9

Month	*Inflow at dam site in hectare metres*	*Pan evaporation in cm*	*Precipitation in cm*	*Demand in hectare-metres*	*Requirement due to prior rights, equal to Col. (2) or 15 ham, whichever is minimum*	*Evaporation in hectare-metres* = $\frac{400 \times \text{Col. (3)}}{100} \times 0.7$ = 2.8 × Col. (3)	*Precipitation in hectare-metres* = $\frac{400 \times \text{Col. (4)}}{100} \times 0.72$ = 2.88 × Col. (4)	*Adjusted inflow* : Col. (2) + Col. (5) − Col. (6) − Col. (7) *in hectare-metres*	*Water required from Storage in ha-m* : Col. (5) − Col. (9) (Only + ve values)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	10
January	1.2	1.8	1.3	15.8	1.2	5.03	3.74	(−) 1.29	17.09
February	0.0	1.8	1.7	14.3	0.0	5.03	4.9	(−) 0.13	14.43
March	0.0	2.6	0.6	9.6	0.0	7.28	1.73	(−) 5.55	15.15
April	0.0	10.2	0.0	4.8	0.0	28.56	0.0	(−) 28.56	33.36
May	0.0	15.4	0.0	3.5	0.0	43.1	0.0	(−) 43.1	46.60
June	0.0	16.6	1.1	3.4	0.0	46.4	3.17	(−) 43.23	46.63
July	240.0	10.8	16.1	5.0	15.0	30.3	46.4	(+) 241.1	Nil
August	480.0	11.7	16.4	5.0	15.0	32.8	47.3	(+) 479.5	Nil
September	1.0	10.8	2.2	10.0	1.0	30.3	6.34	(−) 23.96	33.96
October	0.6	9.6	0.8	15.6	0.6	26.9	2.31	(−) 24.39	39.99
November	0.5	7.8	0.0	16.8	0.5	21.8	0.0	(−) 21.8	38.60
December	0.2	2.0	0.0	16.8	0.2	5.6	0.0	(−) 5.6	22.40
Σ	723.5	101.1	40.2	120.6	33.5	283.1	116.49		308.21

Solution. The given table 3.8 is extended as shown in Table 3.9. The 28% precipitation is already reaching and is included in the given inflow [Col. (2)] and hence, only 72% of the precipitation is to be included as worked out in Col. (8). The table is self-explanatory and the monthly water drawn from the reservoir are worked out in Col. (10). Finally, their summation (*i.e.* **308.21 hectare metres**) works out to be the required storage capacity of the reservoir.

—Ans.

3.14.8.1. Fixation of reservoir Capacity with the help of mass curves of inflow and outflow. After a flow hydrograph for the stream at the dam site has been plotted for a large number of years (say 25 to 30 years) ;the required storage capacity for a reservoir with the given outflow pattern can be approximately calculated with the help of flow mass curves. A **hydrograph** is a plot of discharge V_s time, while a **flow mass curve** is a plot of accumulated flow V_s time. Since the area under the hydrograph between the times $t = 0$ and $t = t$ will represent the accumulated flow upto the time t, it will hence, represent the ordinate of the flow mass curve at time t.

A typical inflow hydrograph is shown in Fig. 3.14 (*a*) and the mass curve for this inflow hydrograph is plotted in Fig. 3.14 (*b*). The area under the first curve up to a time (t) is equal to the ordinate of the second curve at the same time (t). Adjustments for the scales and unit of the two curves must be made while plotting.

It is evident that an ***inflow mass curve*** will continuously rise as it is the plot of accumulated inflow. Periods of no inflow would be represented by horizontal lines on the mass inflow curve.

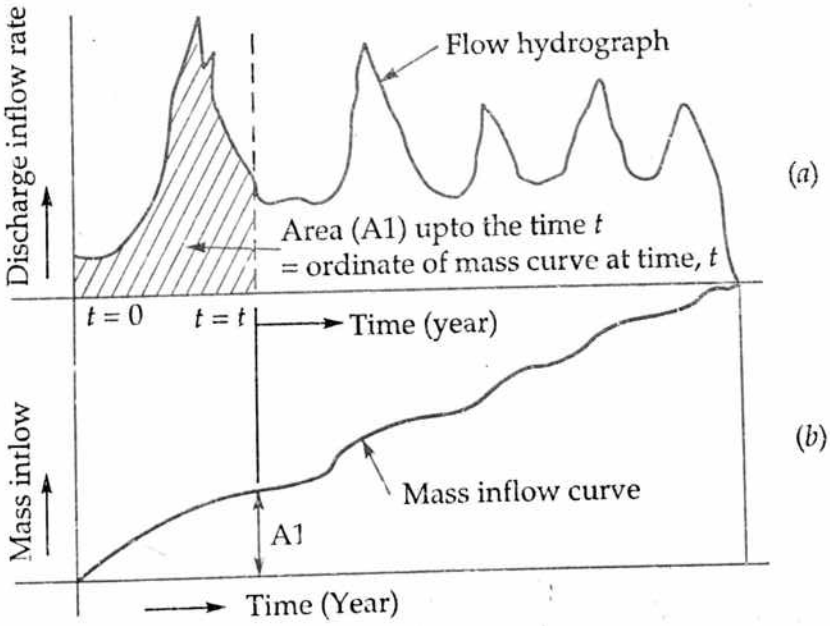
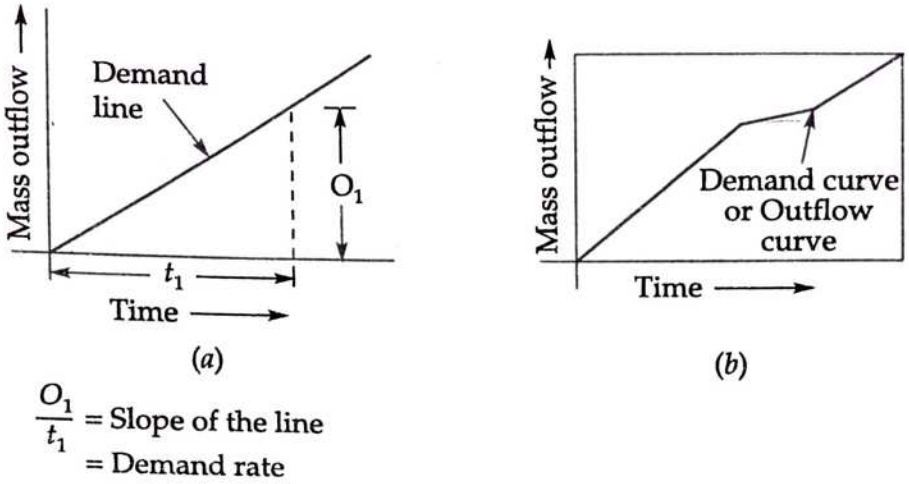


Fig. 3.14 (*a*) and (*b*) Flow hydrograph and flow mass curve.

The mass curve may also be called the **ripple diagram**. The slope of the inflow mass curve at any time is a measure of the inflow rate at that time.

After the inflow mass curve has been plotted, the mass curve of demand *i.e.* ***outflow mass curve*** may also be plotted by accumulating the required outflows. *If a constant rate of withdrawal is required from the reservoir, the*

mass curve of demand will be a straight line having a slope equal to the demand rate. Demand curves or demand lines are generally straight lines (representing uniform withdrawal) although, in practice they may be curved also [See Fig. 3.15 (*a*) and (*b*)].



$\frac{O_1}{t_1}$ = Slope of the line
= Demand rate

Fig. 3.15

Fixing Reservoir Capacity for the Given Inflow and the Given Demand

The mass curve of inflow and the demand line can be used to determine the required storage capacity. In Fig. 3.14, it is evident that demand lines drawn tangent to the high points A_1, A_2, ... of the flow mass curve, represent rate of withdrawal from the reservoir. Assuming the reservoir to be full whenever the demand line intersects the mass curve (points F_1, F_2 ...), the maximum departure (B_1C_1, B_2C_2 ...), between the two curves represents the reservoir capacity just required to satisfy the demand.

Fig. 3.16 has been drawn for a demand of 2,000 hectare-metres/year. The biggest departure ordinate (*i.e.* maximum of B_1C_1, B_2C ...) works out to be 1,950 hectare-metres/year, which represents the required storage capacity for the reservoir.

The vertical distance between successive tangents A_1B_1, A_2B_2 ..., represents the water wasted over the spillway. The spillway must have sufficient capacity to discharge this flood volume.

For Fig. 3.16, the spillway capacity works out to be 600 hectare metres, and reservoir capacity as 1,950 hectare-metres. It can also be observed that :

(*i*) Assuming the reservoir to be full at A_1, it is depleted to 1,950 – 1,500 = 450 ha-m at C_1 and is again full at F_1.

(*ii*) The reservoir is full between F_1 and A_2 and the quantity of water spilled over spillway is equal to 600 ha.m.

(*iii*) From A_2, the water starts reducing in the reservoir till it becomes fully empty at C_2.

(*iv*) The water again starts collecting in the reservoir and it is again full at F_2.

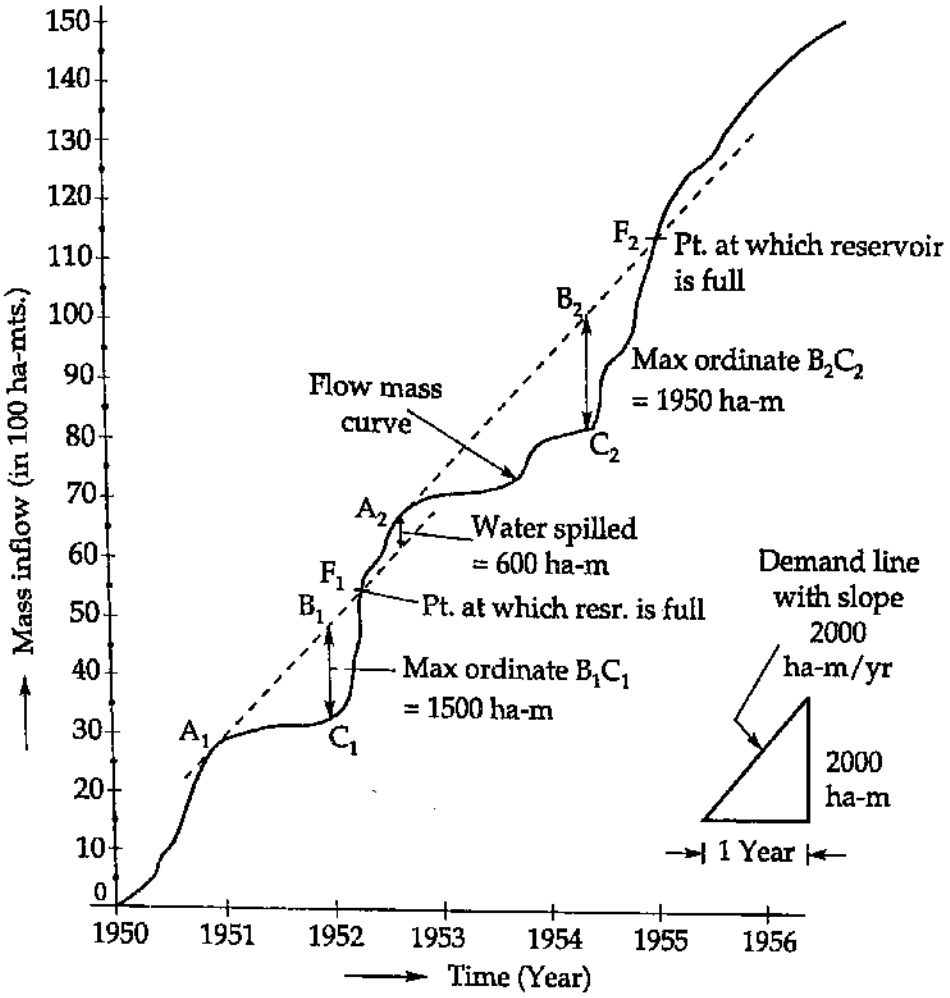


Fig. 3.16

Note 1. It may also be noted that a demand line, when extended must intersect the flow mass curve. If it does not, the reservoir will not refill.

Note 2. When the demand curve is not a straight line, then the two mass curves are superimposed over each other in such a way that their origin and axis coincide. The largest ordinate between the two, gives the required storage capacity, as shown in Fig. 3.17.

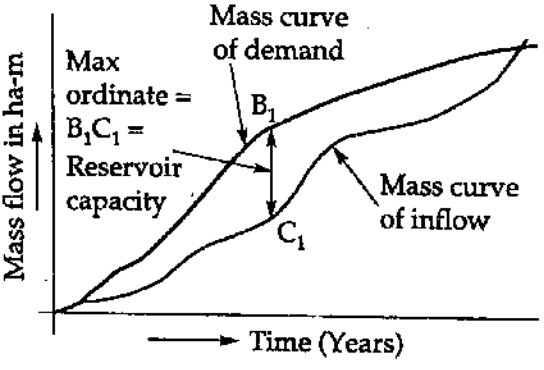


Fig. 3.17

Fixing the Demand from a Reservoir having the Given Inflow and the Given Capacity

In the previous article, we have explained as to how the reservoir capacity can be determined for fulfilling a given demand. The reverse, *i.e.*, fixing the demand from a given reservoir capacity may also be done with the help of mass curves.

In this case, the tangents are drawn to the high points (A_1, A_2...) of the mass inflow curve in such a way that the maximum departure from the mass inflow curve is equal to the reservoir capacity. The slopes of the lines so drawn

represent the supplies, which can be obtained with this capacity during different periods. The minimum value of these slopes will represent the guaranteed firm supply which can be obtained continuously from this reservoir.

For example in Fig. 3.18, *a mass inflow curve is given. It is further required to find out the possible supplies and safe withdrawal for a reservoir capacity of 750 ha-m.* To determine these values, we shall proceed as follows :

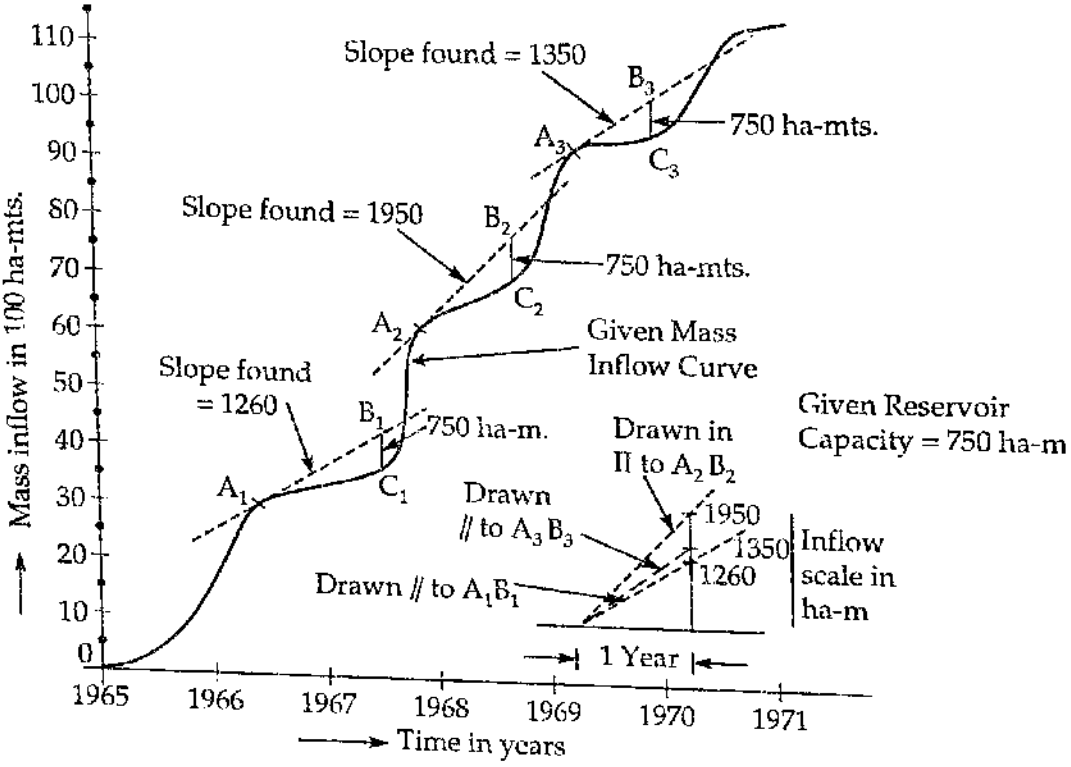


Fig. 3.18

First of all, the high points A_1, A_2 and A_3 are marked on the given curve. The points B_1, B_2 and B_3 are determined in such a way that their maximum departure from the curve is equal to 750 ha-m. The tangent lines A_1B_1, A_2B_2 and A_3B_3 are then drawn. The slopes of these lines are determined and are found to be 1,260, 1,950, 1,350 ha.m/year, respectively. These values represent the possible supplies in different periods. *The minimum of them, i.e., 1,260 ha-m/year represents the safe withdrawal or firm supply.* **Ans.**

3.14.9. Reservoir Sedimentation and its Control. Every river carries certain amount of sediment load. The sediment particles try to settle down to the river bottom due to the gravitational force, but may be kept in suspension due to the upward currents in the turbulent flow which may overcome the gravity force. Due to these reasons, the river carries fine sediment in suspension as suspended load, and large solids along the river bed as bed load. When this silt laiden water reaches a reservoir in the vicinity of a dam, the velocity and turbulence are considerably reduced. The bigger suspended particles and most of the bed load, therefore, gets deposited in the head reaches of the reservoir. Finer particles may travel some more distance and may finally deposit farther down in the reservoir, as shown in Fig. 3.19. Some

very fine particles may remain in suspension for much longer periods, and may finally escape from the dam along with the water discharged through spillway or sluiceways.

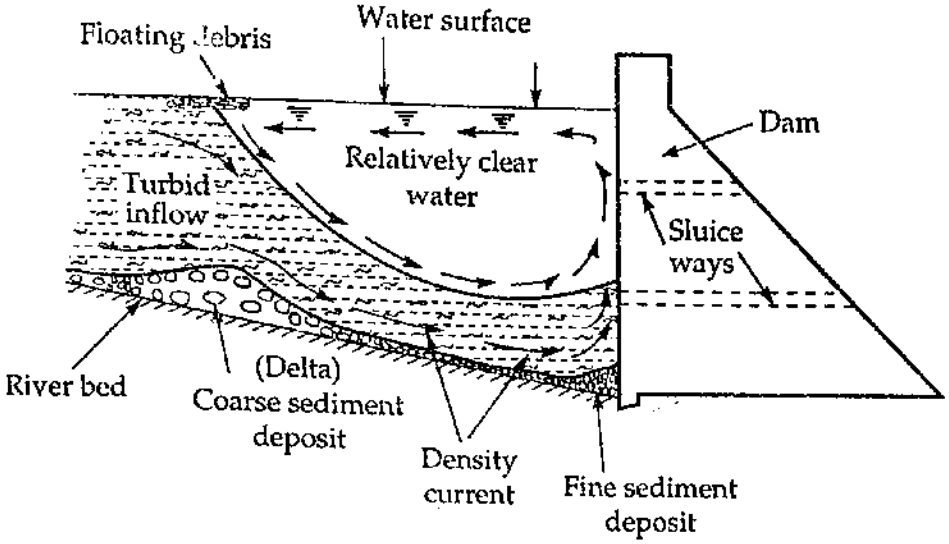


Fig. 3.19

This deposition of sediment in the reservoir is known as ***Reservoir Silting*** or ***Reservoir Sedimentation.***

The deposition of the sediment will automatically reduce the water storing capacity of the reservoir, and if this process of deposition continues longer, a stage is likely to reach when the whole reservoir may be silted up and become useless.

Moreover, with the passage of time, its capacity will go on reducing. Thus, if today at the time of construction, a reservoir can store 10,000 cubic metres of water, tomorrow after five years say, it may be able to store only say 8,000 cubic metres of water. Therefore, in order to see that the capacity does not fall short of requirement ever during the design period, we must take this silting into account. The total volume of silt likely to be deposited during the designed life period of the dam is, therefore, estimated ; and approximately that much of volume is left unused to allow for silting, and is known as **dead storage.** The remainder is known as ***Effective storage*** or **live-storage.** The dead storage is generally kept at about 20 to 25% of the total storage. For example, in Bhakra dam, the gross capacity of the dam is 9,344 million cubic metres and the dead storage provided is 2,054 million cubic metres. All the outlets fetching water from the reservoir are provided above the dead storage level.

The importance of silting can be understood by considering the following example :

Let the total capacity of a reservoir be 30 million cubic metres and the provision of dead storage be 6 million cubic metres. Let the average volume of sediment deposition be 0.15 million cubic metres per year. Then it is evident, that the dead storage will be filled up in $\frac{6}{0.15} = 40$ years, and the total storage in about $\frac{30}{0.15} = 200$ years.

Hence, the usefulness of the reservoir will start reducing after 40 years ; and after 200 years, it will be nothing but a collection of sand and sediment with no water in it.

Density Currents. The water stored in the reservoir is generally free from silt, but the inflow during floods is generally muddy. There are, thus, two fluids having different densities, resulting in the formation of **density currents** ; which may be defined *as the gravitational flow of one fluid under another fluid of approximately equal density*. The density currents thus separate the turbid water from the clearer water and make the turbid water flow along the river bottom in the vicinity of the dam.

The rate of reservoir silting can be reduced by venting the density currents by properly locating and operating the outlets and sluice-ways.

Trap Efficiency. Now, we introduce another very important term called trap efficiency. Trap efficiency is defined as the percentage of the sediment deposited in the reservoir, even in spite of taking precautions and measures to control its deposition.

Therefore, **trap efficiency** (η)

$$= \frac{\text{Total sediment deposited in the reservoir}}{\text{Total sediment flowing in the river}} \qquad ...(3.13)$$

Most of the reservoirs trap 95 to 100% of the sediment load flowing into them. Even if various feasible silt control measures are adopted, it has not been possible to reduce this trap efficiency below 90% or so.

Capacity-Inflow Ratio. The ratio of the reservoir capacity to the total inflow of water in it, is known as the capacity-inflow ratio. It is a very important factor, because the trap efficiency (η) has been found to be a function of **capacity-inflow ratio**.

$$\therefore \qquad \eta = f\left(\frac{\text{Capacity}}{\text{Inflow}}\right).$$

The graph obtained for existing reservoirs between trap efficiency and log of $\left(\frac{\text{Capacity}}{\text{Inflow}}\right)$ has been found to be of the type shown in Fig. 3.20.

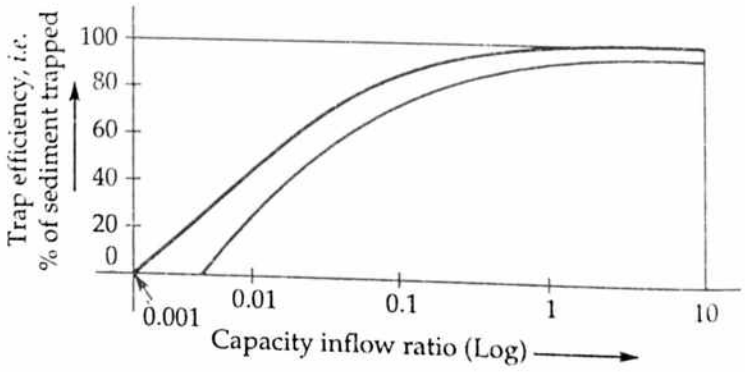


Fig. 3.20

It is evident from the above curve that if the capacity reduces (with constant inflow), trap efficiency reduces, and hence, lesser sediment is trapped. *Therefore, the silting rate in a reservoir shall be more in the beginning, and as its capacity reduces due to silting, the silting rate will reduce.* Hence, the complete reservoir-silting shall take a very long period.

It can also be concluded that for small reservoirs (having small capacities) on large rivers (having large inflow rates), the trap efficiency is extremely low because the capacity-inflow ratio is very small. Such reservoirs silt very little

and most of their sediments are passed downstream. On the other hand, large reservoirs on smaller rivers shall silt tremendously and almost complete deposition of sediment may take place.

Silting of Power Reservoirs. In case of reservoirs constructed solely for the purpose of power generation, the silting is comparatively less important. This is because of the fact, that for the proper and efficient functioning of a power reservoir, only a certain minimum head is necessary. This head remains available even after some silt is deposited. So, only if sufficient water required for power generation remains available, the reservoir's efficiency remains un-affected by silting. But due to silted water, the abrasion of the blades of the turbines may occur very soon, and power production may get stopped over considerable length of time.

Silting Control in Reservoirs. In order to increase the life of a reservoir, it is necessary to control the deposition of sediment. Various measures are undertaken in order to achieve this aim. The various methods which are adopted can be divided into two parts :

(1) *Pre-constructing measures* ; and

(2) *Post-constructing measures.*

Both these types of measures are discussed below :

(1) *Pre-constructing Measures.* These are those measures which are adopted before and during the execution of the project. They are enumerated below :

(*a*) *Selection of Dam Site.* The silting depends upon the amount of erosion from the catchment. If the catchment is less erodible, the silting will be less. Hence, the silting can be reduced by choosing the reservoir site in such a way as to exclude the run-off from the easily erodible catchment.

(*b*) *Construction of the Dam in Stages.* The design capacity plays an important role in the silting of the reservoir. When the storage capacity is much less than the average annual run-off entering the reservoir, a large amount of water will get out of the reservoir, thereby, reducing the silting rate compared to what it would have been if the entire water would have been stored. Therefore, the life of a reservoir can be prolonged by ***constructing the dam in stages***. In other words, first of all, the dam is built lower, and raised subsequently when some of its capacity has been silted up.

(*c*) *Construction of Check Dams.* The sediment inflow can be controlled by building ***check dams*** across the river streams contributing major sediment load. These are smaller dams and trap large amounts of coarser sediments. They are quiet expensive.

(*d*) *Vegetation Screens.* This is based on the principle that vegetations trap large amounts of sediment. The vegetation growth is, therefore, promoted at the entrance of the reservoir as well as in the catchment. These vegetative covers, through which flood waters have to pass before entering the reservoirs, are known as ***vegetation screens***, and provide a cheap and a good method of silt control.

(*e*) *Construction of Under-sluices in the Dam.* The dam is provided with openings in its base, so as to remove the more silted water on the downstream side. The sediment concentration will be more at some levels than at others. Therefore, sluices are located at the levels of higher sediment concentration.

Hence, the usefulness of the reservoir will start reducing after 40 years ; and after 200 years, it will be nothing but a collection of sand and sediment with no water in it.

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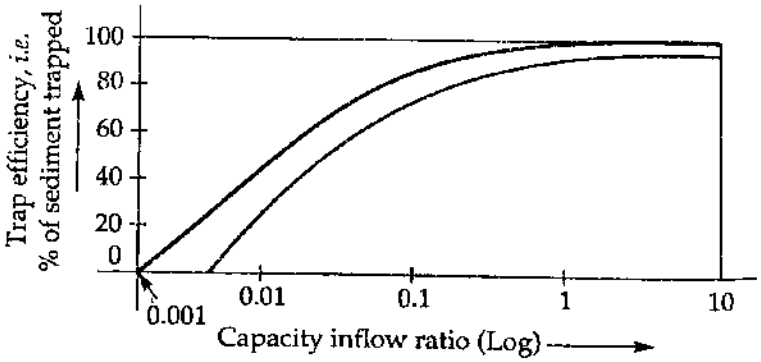

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This method, in itself, is not sufficient, because the water digs out a channel behind the sluice for its movement and leaves most of the sediment undisturbed. Therefore, this is simultaneously supplemented by mechanical loosening and scouring of the neighbouring sediment in order to increase its effectiveness. But provision of large sluices near the bottom of the dam is again a structural problem. The use of this method is therefore, limited.

(2) *Post-constructing Measures.* These measures are undertaken during the operation of the project. They are given below :

(*a*) *Removal of Post Flood Water.* The sediment content increases just after the floods ; therefore, attempts are generally made not to collect this water. Hence, the provision should be made to remove the water entering the reservoir at this time.

(*b*) *Mechanical Stirring of Sediment.* The deposited sediment is scoured and disturbed by mechanical means, so as to keep it in a moving state and thus help in pushing it towards the sluices.

(*c*) *Erosion Control and Soil Conservation.* This includes all those general methods which are adopted to reduce erosion of soil and to make it more and more stable. When the soil erosion is reduced, automatically, the sedimentation problem is reduced. But the methods of treating the catchment in order to minimise erosion are very costly. It has been estimated that the investment required for treating 15% of the Indian catchment area would be around ₹ 2,000 crores. In India, only about 1.5% of the catchment area has been treated to minimise silting.

Example 3.4. *The following information is available regarding the relationship between trap efficiency and capacity inflow ratio.*

Table 3.10

Capacity inflow ratio	*0.1*	*0.2*	*0.3*	*0.4*	*0.5*	*0.6*	*0.7*	*0.8*	*0.9*	*1.0*
Trap efficiency per cent	*87*	*93*	*95*	*95.5*	*96*	*96.5*	*97*	*97*	*97*	*97.5*

Find the probable life of the reservoir with an initial reservoir capacity of 30 million cubic metres, if the average annual flood inflow is 60 million cubic metres and the average annual sediment inflow is 2,00,000 tonnes. Assume a specific weight of the sediment equal to 1.2 gm per c.c. The usual life of the reservoir will terminate when 80% of its initial capacity is filled up with sediment.

Solution.

Average annual sediment inflow = 2,00,000 tonnes

$$= 2 \times 10^5 \text{ tonnes} = 2 \times 10^{11} \text{ gm}$$

Volume of average annual sediment inflow

$$= \frac{2 \times 10^{11}}{1\,2} \text{ c.c.} = \frac{2 \times 10^{11}}{1.2 \times 10^6} \text{ m}^3$$

$$= \frac{0.2}{1.2} \times 10^6 \text{ cubic metres}$$

$= \frac{1}{6}$ million cubic metres

$= \frac{1}{6}$ m.c.m.

Initial Reservoir Capacity = 30 m.c.m.

Annual flood inflow = 60 m.c.m.

Let us assume that 20% of the capacity, *i.e.* 6 m.c.m. is filled up in the 1st interval.

Capacity inflow ratio at the start of the interval $= \frac{30}{60} = 0.5$

Trap efficiency at the start of interval = 0.96

Capacity inflow ratio at the end of the interval $= \frac{24}{60} = 0.4$

Trap efficiency at the end of interval = 0.955

Average trap efficiency during this interval

$$= \frac{0.96 + 0.955}{2} = \mathbf{0.9575}$$

Volume of sediment deposited annually till the 20% capacity is filled

$= \frac{1}{6} \times 0.9575$ m.c.m./year.

∴ No. of years during which 20% of the capacity *i.e.* 6 m.c.m. shall be filled up

$$= \frac{6}{\frac{1}{6} \times 0.9575} \text{ years} = \frac{36}{0.9575} = \mathbf{37.6 \text{ years.}}$$

Similarly in the 2nd interval

Capacity inflow ratio at the start $= \frac{24}{60} = 0.4$

Capacity inflow ratio at the end $= \frac{18}{60} = 0.3$

Trap efficiency at start = 0.955

Trap efficiency at end = 0.95

Average trap efficiency = **0.9525**

∴ No. of years during which the next 20% of capacity shall be filled up

$$= \frac{6}{\frac{1}{6} \times 0.9525} = \frac{36}{0.9525} = \mathbf{37.8 \text{ years.}}$$

Similarly in the 3rd interval

Capacity inflow ratio at start $= \frac{18}{60} = 0.3$

Capacity inflow ratio at end $= \frac{12}{60} = 0.2$

Trap efficiency at start = 0.95

Trap efficiency at end = 0.93

Average trap efficiency during the interval = 0.94.

∴ No. of years during which the next 20% of the capacity shall be filled up

$$= \frac{6}{\frac{1}{6} \times 0.94} = \frac{36}{0.94} = \mathbf{38.3\ years.}$$

Similarly in the 4th interval

Capacity inflow ratio at start $= \frac{12}{60} = 0.2$

Capacity inflow ratio at end $= \frac{6}{60} = 0.1$

Trap efficiency at start $= 0.93$

Trap efficiency at end $= 0.87$

Average trap efficiency during the interval = 0.90

∴ No. of years during which the next 20% of the capacity shall be filled up

$$= \frac{6}{(\frac{1}{6} \times 0.9)} = \frac{36}{0.9} = \mathbf{40\ years}$$

Total probable life till 80% capacity gets filled up

$$= 37.6 + 37.8 + 38.3 + 40.0 = \mathbf{153.7\ years.}$$ **Ans.** –

The above calculations of dividing the entire capacity into intervals (20% each in the above case) can also be carried out in a tabular form, as shown in table 3.11.

Table 3.11

Capacity		*Capacity/ inflow* $= \frac{Col.(2)}{60\ Mcum}$	*Trap eff.* η	*Av. trap. eff.* η_{av} *during the interval*	*Sediment trapped per year* Col. (5) × Av. annual sediment inflow $= Col.(5) \times \frac{1}{6}$ M.cum	*Years reqd. to fill up 20% capacity (6 M.cum)* $= \frac{6}{Col.(6)}$ *in years*
%	*Capacity Vol. in M.cum.*					
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*	*(7)*
100	30	0.5	0.96			
				0.9575	0.1596	37.6
80	24	0.4	0.955			
				0.9525	0.1588	37.8
60	18	0.3	0.95			
				0.94	0.1566	38.3
40	12	0.2	0.93			
				0.90	0.15	40.0
20	6	0.1	0.87			
						Σ = 153.7 yrs

Example 3.5. *A proposed reservoir has a capacity of 500 ha-m. The catchment area is 125 km², and the annual streamflow averages 12 cm of runoff. If the annual sediment production is 0.03 ha-m/km², what is the probable life of the reservoir before its capacity is reduced by 10% of its initial capacity by sedimentation ? The relationship between trap efficiency η (%) and capacity inflow ratio C/I, is as under :*

Table 3.12

C/I,	*0.01*	*0.02*	*0.04*	*0.06*	*0.08*	*0.1*	*0.2*	*0.3*	*0.5*	*0.7*
η (%)	*43*	*60*	*74*	*80*	*84*	*87*	*93*	*95*	*96*	*97*

(Civil Services, 1987)

Solution. Av. annual streamflow

= 12 cm of runoff

Area of catchment = 125 km^2 = 125 × 10^6 m^2

∴ Annual flood inflow

$$= (125 \times 10^6) \cdot \frac{12}{100} \text{ m}^3 = 15 \times 10^6 \text{ m}^3$$

$$= \mathbf{15\ M\text{-}m^3\ (Mcum).}$$

Annual sediment inflow = 0.03 ha-m/km^2 of catchment

$$= (0.03 \times 125) \text{ ha-m} = 0.03 \times 125 \times 10^4 \text{ m}^3$$

$$= 3.75 \times 10^4 \text{ m}^3 = \frac{3.75}{100} \times 10^6 \text{ m}^3$$

$$= \mathbf{0.0375\ M\text{-}m^3\ (Mcum).}$$

Reservoir capacity = 500 ha-m

$$= 500 \times 10^4 \text{ m}^3 = 5 \times 10^6 \text{ m}^3$$

$$= \mathbf{5\ M\text{-}m^3\ (Mcum).}$$

It means that 0.0375 Mcum of sediment flows every year into the dam/ reservoir site, but the quantum of this, which is trapped in the reservoir, depends upon the average trap efficiency (η) during that year, and this trap efficiency, inturn, depends upon capacity/inflow ratio.

In the question, the total capacity to be filled up by sediment is 10% of the initial reservoir capacity, *i.e.*, 10% × 5 Mcum = 0.5 Mcum.

Now, we have to calculate the time during which this 0.5 Mcum of sediment will get deposited in the reservoir, as follows :

Capacity of reservoir at the start = 5 Mcum

Capacity of reservoir at the end

i.e., when 0.5 Mcum of sediment is filled up

= 4.5 Mcum

$$\therefore \text{ Capacity/inflow at start} = \frac{5 \text{ Mcum}}{15 \text{ Mcum}} = 0.333$$

$$\eta \text{ at start} = 95\%$$

$$\text{Capacity/inflow at end} = \frac{4.5}{15} = 0.30$$

η at the end of interval = 95%

Average η = 95%

$\therefore$ Sediment load trapped/yr, = 0.0375 × 95% = 0.035625 Mcum/yr

$\therefore$ No. of years during which 0.5 Mcum of sediment will get trapped

$$= \frac{0.5}{0.035625} \text{ years}$$

$$= 14.04 \text{ years ; say 14 years}$$

Hence, after 14 years, 10% reservoir capacity will get filled up. **Ans.**

3.14.10. Reservoir Losses. The important reservoir losses are :

(*i*) ***Evaporation Losses.*** These losses mainly depend upon the reservoir surface area. The other factors influencing these losses are temperature, wind velocity, relative humidity, etc. Standard pan evaporation can be measured and when multiplied by pan coefficient, gives the reservoir evaporation loss. The pan coefficients and various types of pans in use are given in the Chapter on "Evaporation and Transpiration Losses from a Watershed", in "*Hydrology and Water Resources Engineering*" by the same author.

The evaporation losses sometimes become enormous, and certain chemical compounds such as Hexadecanol (acetyl alcohol) are sprinkled over the surface of water, so as to reduce the activity of evaporation.

These losses are expressed in cm of water depth and may vary from month to month. Certain average values of these losses for North and South India are given in Table 3.13.

Table 3.13. Evaporation losses (Average values for Indian Reservoirs)

Month	*Losses in cm*	
	North India	*South and Central India*
January	7	10
February	9	10
March	13	18
April	16	23
May	27	25
June	24	18
July	18	15
August	14	15
September	14	15
October	13	13
November	9	10
December	8	10
Total	172	182

(*ii*) ***Absorption Losses.*** These losses do not play any significant role in planning, since their amount, though sometimes large in the beginning, falls considerably as the pores get saturated. They certainly depend upon the type of soil forming the reservoir.

(*iii*) ***Percolation Losses or Reservoir Leakage.*** For most of the reservoirs, the banks are permeable, but the permeability is so low that the leakage is of no importance. But in certain particular cases, when the walls of the reservoir are made of badly fractured rocks or having continuous seams of porous strata, serious leakage may occur. Sometimes, pressure grouting may have to be used to seal the fractured rock. The cost of grouting has to be accounted for, in the economic studies of the project, if the expected leakage is large.

3.14.11. Reservoir Clearance. The removal of trees, bushes and other vegetation from the reservoir area is known as reservoir clearance. It is an expensive operation and difficult to be justified on cost-benefit considerations. Non-clearance of such vegetation may create the following difficulties :

(*i*) decay of organic material may create undesirable odours and tastes, and hence, important for water supply reservoirs.

(*ii*) trees projecting above the water surface may create an undesirable appearance when the reservoir is to be used for recreation and tourism purposes.

(*iii*) bushes, trees, etc. will float and may create debris problem at the dam site.

3.14.12. Selection of a Suitable Site for a Reservoir. It is almost impossible to select a perfect ideal reservoir site, but its selection is guided by the following factors :

(*i*) A suitable dam site should be available. The cost of the dam is generally a controlling factor in the selection of a reservoir site.

(*ii*) The geological formations for the reservoir banks, walls, etc. should be such as to entail minimum leakage.

(*iii*) The geology of the catchment area should be such as to entail minimum water losses through absorption and percolation.

(*iv*) The site should be such that a deep reservoir is formed. A deep reservoir is preferred to a shallow one, because of lower land cost per unit of capacity, less evaporation loss, and less possibility of weed growth.

(*v*) The reservoir site must have adequate capacity.

(*vi*) Too much silt laiden tributaries should be avoided as far as possible.

(*vii*) The reservoir basin should have a deep narrow opening in the valley, so that the length of the dam is minimum.

3.14.13. Fixing the Height of the Dam. After determining the gross useful storage capacity required for fulfilling the demand, the height of the dam is fixed by providing that much of storage between minimum pool level (or dead storage level) and normal pool level with the help of *elevation-storage* curve. The water surface elevation or F.S.L. or normal pool level of the reservoir is thus fixed. The max. pool level or H.F.L. of the reservoir is determined by using "flood routing"* methods. A suitable free-board is then added for fixing the top of the dam. The provision for the depth of

*If needed, please refer "Hydrology and Water Resources Engg." by the same author.

foundation is made in deciding the overall height of the dam, as shown in Fig. 3.21.

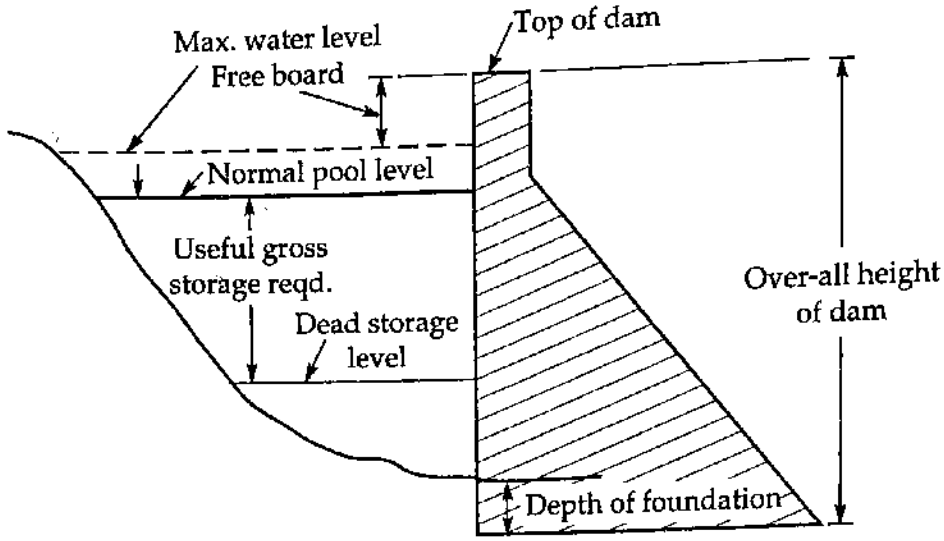


Fig. 3.21

Economic Height of a Dam. The economic height of a dam is that height of the dam, corresponding to which, the cost of the dam per unit of storage is minimum. For this purpose, estimates are prepared for construction costs, for several heights of dam, somewhat above and below the level at which the elevation-storage curve shows a fairly high rate of increase of storage per unit rise of elevation, keeping the length of the dam moderate. The construction cost increases with the dam height, as shown in Fig. 3.22.

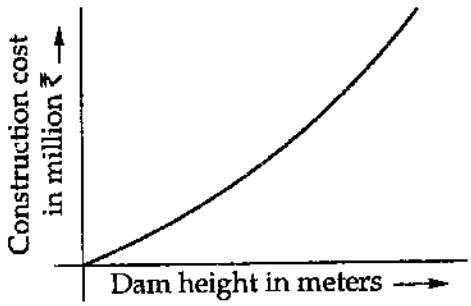


Fig. 3.22

For each dam height, the reservoir storage is known from the reservoir-capacity curve. The construction cost per unit of storage for all the possible dam heights can then be worked out and plotted as shown in Fig. 3.23.

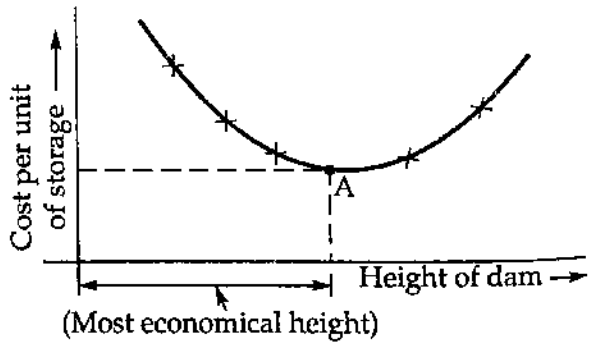


Fig. 3.23

The lowest point *A* on this curve, gives the dam height for which the cost per unit of storage is minimum, and hence most economical.

Example 3.6. *The construction costs for certain possible heights of a dam at a given site have been estimated and are tabulated below in table 3.14. The storage capacity for all these dam heights are also given.*

Table 3.14

S. No. (1)	*Height of the dam in metres* (2)	*Construction cost in million* (3)	*Storage in million cubic metres (m.c.m.)* (4)
1	*10*	*4*	*50*
2	*20*	*8*	*110*
3	*30*	*12*	*180*
4	*40*	*18*	*250*
5	*50*	*27*	*350*
6	*60*	*39*	*500*
7	*65*	*50*	*600*

Determine the most economical height of the dam from purely construction point of view.

Solution. Table 3.14 is extended so as to work out the cost per million cubic metre of storage, as shown in Col. (5) of Table 3.15.

Table 3.15

S. No. (1)	*Height of the dam in metres* (2)	*Construction cost in m.* ₹ (3)	*Storage in m.c.m.* (4)	*Cost per unit of storage* $= \frac{\text{Col. 3}}{\text{Col. 4}}$ (5)
1	10	4	50	0.080
2	20	8	110	0.073
3	30	12	180	0.067
4	40	18	250	0.072
5	50	27	350	0.077
6	60	39	500	0.078
7	65	50	600	0.083

The cost per unit of storage (col 5) is plotted against the height of the dam (col 2) as shown in Fig. 3.24. The most economical height is the lowest point of this curve, and it works out to be 30 metres. **Ans.**

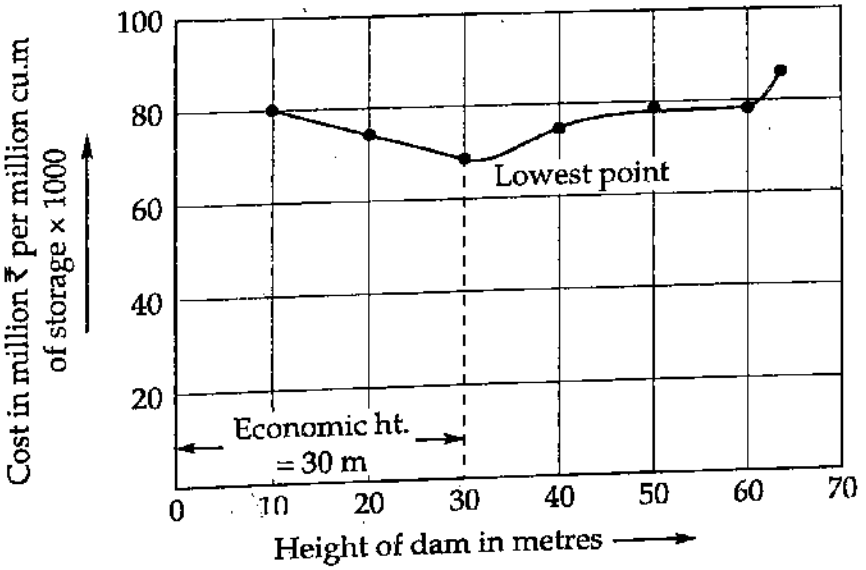


Fig. 3.24

3.15. Quality and Quantity of Surface Waters and Their Usefulness for Public Water Supplies

Quantity. The quantities of available surface waters are dependent upon rainfall. Since on an average, in India, rainfall is sufficient and considerable, there should, therefore, be no scarcity of water in these surface sources. But since the rainfall is not uniformly and regularly spread throughout the year, considerable variations in the available flows do occur during the year or years. Thus, the available flow in a stream channel or a river may be too high to be controlled or may be too less to fulfil the demand. Storage reservoirs, therefore, provide good means of storing and utilising rain waters.

Quality. The rain water, though pure in the beginning, gets considerably polluted till it reaches the river streams. The gases, dusts, etc., from the atmosphere, get added to the rain water till it reaches the ground ; from where onward, it flows on earth's surface and also through drains, channels, etc., which add a lot of organic as well as inorganic impurities to it. Many a times, sewage and industrial wastes get added to this water, making it contaminated.

Inorganic impurities like silt, clay, etc., get added due to erosion from the beds of the stream channels. The organic impurities get added in the form of vegetable washings, dead organic matters, and dead animals, etc.

The inorganic suspended matter, though largely present in direct river or stream waters, get settled considerably in still waters of lakes, ponds and reservoirs. However, the algae, weed and plant growth in still waters increase enormously, thus giving colours and tastes to these waters. Surface waters are, however, generally soft and less corrosive than ground waters.

On the whole, it can be stated that the *surface supplies are generally contaminated* and cannot be used with minor treatment or without any treatment. They, therefore, need building up of proper water treatment plants (WTPs) and other connected works before being used for public supplies. They are useful for big cities and large industrial towns where huge quantities of water are required by the public.

SUB-SURFACE OR UNDERGROUND SOURCES

The water which gets stored in the ground water reservoir through infiltration, is known as the underground water. This water is generally pure, because it undergoes natural filtration during the percolation through the soil pores. Moreover, these waters are less likely to be contaminated by bacteria. However, they are generally rich in dissolved salts, minerals, gases, etc. The extent of the salts and minerals present in the ground water depends upon the type and extent of geological formations through which the water is passing before joining the watertable.

Sometimes, the ground water is brought to the surface by some natural processes like springs, and sometimes these waters are tapped by artificial means by constructing wells, tube wells, infiltration galleries, etc.

The replenishment (*i.e.*, filling up) and drainage (*i.e.*, tapping out) of the ground water reservoir is a full topic in itself, involving the hydrological concepts of ground water flow, the possible yields, the construction details of

wells, tubewells, galleries, etc. *Since the ground water is largely tapped in our country for water supplies and there is a scope for its development in future also, the next full chapter has, therefore, been devoted to the same.* The surface and sub-surface supplies have also been compared there itself.

FACTORS GOVERNING THE SELECTION OF A PARTICULAR SOURCE OF WATER

The following important factors are generally considered in selecting a particular source for supplying water to a city or a town.

(*i*) **The Quantity of Available Water.** The quantity of water available at the source must be sufficient to meet the various demands during the entire design period of the scheme. Sometimes the water sources may be mobilised for the present day demand, and extra units added with the passage of time. If sufficient quantity of water is not available in the vicinity of the area, we may have to think of bringing water from distant sources.

(*ii*) **The Quality of Available Water.** The water available at the source must not be toxic, poisonous or in any other way injurious to health. The impurities present in the water should be as less as possible, and should be such as to be removed easily and economically by normal treatment methods. The quality standards required for public water supplies are discussed in chapter 8.

(*iii*) **Distance of the Source of Supply.** The source of water must be situated as *near the city* as possible. Because when the distance between the source and the city is less, lesser length of pipe conduits and fewer number of associated appurtenances are required, thereby reducing the cost of the project.

(*iv*) **General Topography of the Intervening Area.** The area or land between the source and the city should not be highly uneven. In other words, it should not contain deep valleys or high mountains and ridges. In such uneven topographics, the cost of trestles for carrying water pipes in valleys and that of constructing tunnels in mountains, shall be enormous.

(*v*) **Elevation of the Source of Supply.** The source of water must be on a high contour, lying sufficiently higher than the city or town to be supplied with water, so as to make the gravity flow possible. When the water is available at lower levels than the average city level, pumping has to be resorted to, which involves huge operational cost and frequent possible breakdowns.

PROBLEMS

1. What are the common sources of water for a water supply scheme ? State the factors that govern the final selection. Explain how you would determine the yield from any three of such sources. (*Engg. Services, 1971*)
2. Give a list of different sources of water, which can be considered for large supplies, bringing out the quality that can be expected in each case, and the treatment that may have to be undertaken in each case. (*Engg. Services, 1973*)
3. (*a*) Discuss the various methods to control the evaporation losses of a reservoir.
   (*b*) Explain with the help of a sketch the utility of a mass curve.

4. Write brief notes on any four of the following :
   (*i*) Hydrologic cycle and its importance ;
   (*ii*) Precipitation and its types ;
   (*iii*) Rainfall and its distribution ;
   (*iv*) Rainfall measurement, rain gauges and their types ;
   (*v*) Average annual rainfall and Index of wetness ;
   (*vi*) Determination of yearly run off from a catchment with the help of empirical formulae, and their usefulness in planning a storage water supply scheme ;
   (*vii*) Determination of river discharge, and its use in designing and planning a river water supply scheme.
5. Enumerate the various surface sources of water, and discuss and compare the quality and quantity of water supplies that may be available from these sources.
6. (*a*) What are the two major types of sources of water supplies ?
   Discuss the comparative merits and demerits of both these types of sources.
   (*b*) Discuss the various factors that govern the selection of a particular source of water in formulating a town/city water supply scheme.
7. Write short notes on any four of the following :
   (*i*) Dams, their types and selection of site ;
   (*ii*) Storage capacity of reservoirs ;
   (*iii*) Yield of a storage reservoir ;
   (*iv*) Flow mass curves and their use in determining storage capacity of a dam reservoir ;
   (*v*) Reservoir sedimentation and its control ;
   (*vi*) Reservoir losses and their control ;
   (*vii*) Selecting a suitable site for a storage reservoir, and fixing the height of the required dam.

—4

Development of Ground Water

4.1. Occurrence of Ground Water

The rainfall that percolates below the ground surface, passes through the voids of the rocks, and joins the watertable. These voids are generally interconnected, permitting the movement of the ground water. But in some rocks, they may be isolated, and thus, preventing the movement of water between the interstices. Evidently, the mode of occurrence of ground water depends largely upon the type of formation, and hence upon the geology of the area. In fact, all the materials of variable porosity (or interstices) near the upper portion of the earth's crust, can be considered as the place of potential storage for the ground water, and may be referred to as the *ground water reservoir*.

The volume of water contained in the ground water reservoir in any localised area, *i.e.* **the water storage capacity of the ground water** is dependent upon (*i*) *the porosity and permeability of the rocks* ; (*ii*) *the rate at which water is added to it by infiltration* ; and (*iii*) *the rate at which water is lost from it by evaporation, transpiration, seepage to surface courses, and withdrawal by man*.

4.2. Geological Factors Governing the Occurrence of Ground Water

As stated above, the possibility of occurrence of ground water mainly depends upon two properties of the underground soil, *i.e.*

(*i*) *the porosity of the soil,* and

(*ii*) *the permeability of the soil.*

Both these properties are discussed below :

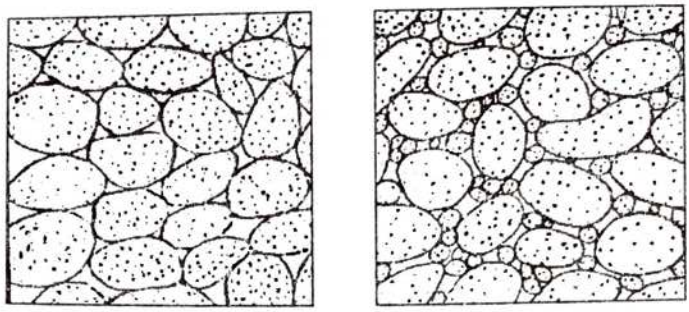
4.2.1. Porosity. The porosity of the soil, which is the major geological criteria for occurrence of ground-water, is a quantitative measurement of the interstices or voids present in the soil. It is generally defined as *the percentage of the voids present in a given volume of soil aggregate*. Mathematically, it can be expressed as :

$$\text{Porosity} = \frac{\text{Total volume of voids in the soil aggregate } i.e., \text{ the volume of water required to saturate the dry sample } (V_v)}{\text{Total volume of the soil aggregate } (V)}$$

It is generally denoted by the letter neeta (η).

$$\text{Therefore} \quad \eta = \frac{V_v}{V} \times 100 \text{ (per cent)} \qquad ...(4.1)$$

Porosity, in fact, depends upon the shape, packing, and degree of sorting of the component grains in a given material. Uniform and well sorted grains (Fig. 4.1 A) give rise to higher porosity ; whereas, heterogeneous grains with irregular arrangement (Fig. 4.1 B) reduce the porosity.



(A) (B)

Fig. 4.1. Diagrammatic sketch of well sorted (*A*) and poorly sorted (*B*) deposits. Porosity of *A* is greater than that of *B*, because, the grains of *A* are all about equal in size. In *B*, smaller grains fill the spaces between the larger grains, thus reducing the volume of void space.

The porosity of rocks and unconsolidated materials may vary considerably. It may be less than a per cent or more than 50%. But generally, it does not exceed 40%, except in very poorly compacted materials. In general, a porosity greater than 20% is considered to be large, and below 5% as small, and between 5 to 20% as medium. The porosity values for a few common types of rock formations are given in table 4.1.

Table 4.1. Porosity Values of a few Rock Formations

S. No.	*Type of rock formation*	*Porosity*
1.	Granite, Quartzite	1.5%
2.	Slate, Shale	4%
3.	Limestone	5 to 10%
4.	Sandstone	10 to 15%
5.	Sand and Gravel	20 to 30%
6.	Only Gravel	25%
7.	Only Sand	35%
8.	Clay and soil	45%

4.2.2. Permeability and Transmissibility. As stated above, the ground-water can get stored in the under-ground rocks, only if, they are sufficiently porous, because the water is actually stored in the pores (voids) of the rocks. *The Porosity of the rock, thus, defines the maximum amount of water that can be stored in the rock.* This porosity, however, in itself, does not ensure the storage of under-ground water, because the water can enter into a rock (with any amount of porosity) only if the rock permits the flow of water through it, *i.e.*, it depends on whether the rock is *permeable* or not. It may be clarified here that a rock which is porous, may or may not be permeable. For example, Shale is a porous rock, but its pore spaces are so minute that the rock remains impermeable. *The size of the pores* is, thus quite an important factor, and hence, the pore size should be sufficiently large to make the rock permeable.

The permeability is, therefore, defined as the ability of a rock or unconsolidated sediment, to transmit or pass water through itself. *Transmissibility* is another term which represents the same physical meaning, but only differing mathematically, as explained below.

The capability of the entire soil of full width and depth, (*i.e.* area *b.d.*) is represented by *permeability* ; while that of the soil of unit width and full depth, (*i.e.*, $b = 1$ and $d = d$. *i.e.*, $A = d$) is known as *transmissibility*.

The permeability is measured in terms of *coefficient of permeability* which will be defined a little later in article 4.4. Various methods including constant head permeameter and variable head permeameter are used to measure permeability.

4.3. Zones of Under-ground Water

As we move down below the surface of the earth towards its centre, water is found to exist in different forms in different regions.

With regard to the existence of water at different depths, the earth's crust can be divided into various zones, namely,

(*i*) *Zone of rock fracture* ; and

(*ii*) *Zone of rock flowage.*

These zones are shown in Fig. 4.2, and explained below :

The depth of the **zone of rock flowage** (*i.e.*, the zone in which the rocks undergo permanent deformation) is not accurately known but is generally estimated as many kilometres. Interstices are probably absent in this zone, because the stresses are beyond the elastic limits, and the rocks remain in a state of plastic flow. Water present in this zone is known as **internal water,** and a hydraulic engineer has nothing to do with this water.

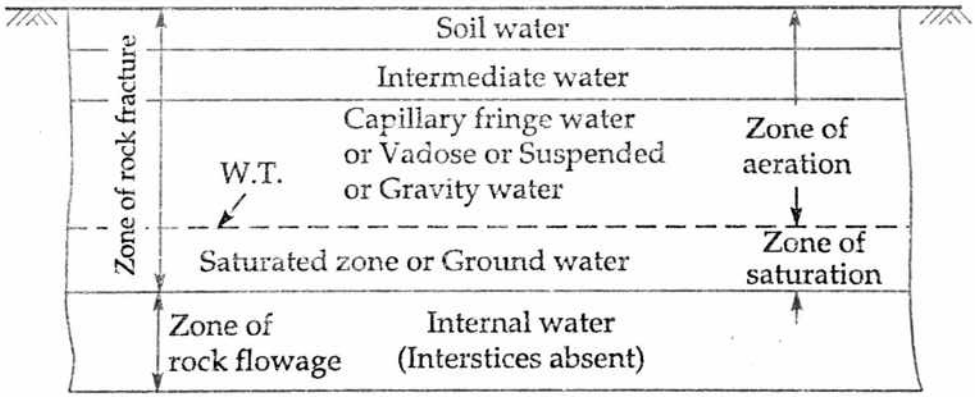


Fig. 4.2. Zones of underground water.

Above the *zone of rock flowage*, there lies the **zone of rock fracture**. In this zone, the stresses are within the elastic limit, and the interstices do exist. Water is stored in the voids, the amount of which depends upon porosity. The maximum depth of this zone below the ground surface, varies in the range of about 100 metres or less to 1,000 metres or more.

In crystalline rocks, most of the water is met within 100 m of the surface, while in the sedimentary rocks, it is found upto depths of 1,800 m or so, although very less quantity is found below 1,000 m or so.

The zone of the rock fracture can be further sub-divided into two zones. One is the *zone of saturation, i.e.*, the zone below the watertable, and the other is the *zone of aeration, i.e.*, the zone above the watertable.

In the **zone of saturation,** water exists within the interstices and is known as ground-water. This is the most important zone for a ground water hydraulic

engineer, because he has to tap out this water. Water in this zone is under hydrostatic pressure.

The space above the watertable and below the surface is known as the **zone of aeration**. Water exists in this zone by molecular attraction. The gravity water moves through this zone, and the water in this zone is not at hydrostatic pressure. The thickness of this zone varies from almost none in marshy and low lying areas to about 300 metres or so in arid regions.

This zone is also divided into three classes depending upon the number of interstices present. The *capillary fringe* is the belt overlying the zone of saturation, and it does contain some interstitial water, and is thus a continuation of the zone of saturation ; while the depth from the surface which is penetrated by the roots of vegetation is known as the *soil zone*. The remainder intermediate part is the *intermediate zone*.

4.4. Movement of Ground Water and its Velocity

The watertable is generally not horizontal, and has high and low points in it, *i.e.* it is not in equilibrium. In order that the equilibrium is approached, water moves inside the ground from the high points on the watertable to the points lower down, as shown in Fig. 4.3. The rate at which such movement occurs is

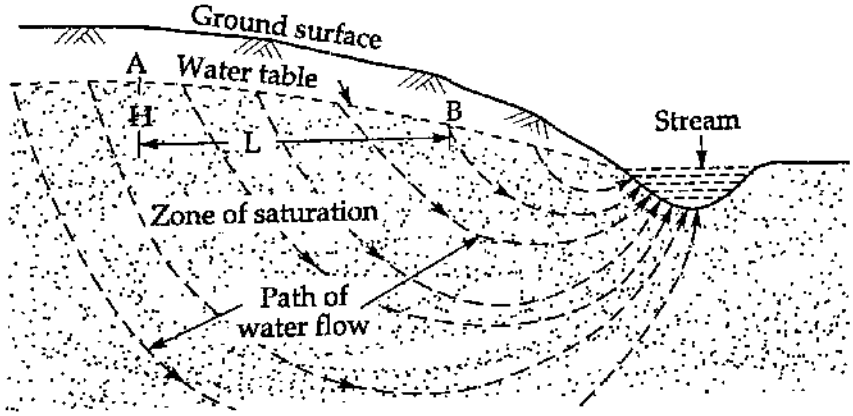


Fig. 4.3. Cross-sectional diagram showing the flow of ground water in a uniformly permeable material. The difference in elevation between points *A* and *B* of the watertable is *H*, and the distance between them is *L*. The hydraulic gradient is *H*/*L*.

dependent upon two factors ; *i.e.*, (*i*) the ability of the porous medium to pass water through it *i.e.*, on the *permeability* ; and (*ii*) the driving force or the *hydraulic gradient* (*i*) usually expressed as the ratio between the difference in elevation (*H*) of the two points on the watertable (in the direction of flow), and the distance between them (*L*), as shown in Fig. 4.3. This was stated as such, by Darcy, as the important *Darcy's law*, which is discussed below :

4.4.1. Darcy's Law for Determining Velocity of Ground Water. On the basis of experimental evidence, Mr. H. Darcy, a French scientist, enunciated in 1856, a law governing the flow (*i.e.* discharge) through soils. According to him, the discharge was proportional to the head loss (ΔH), and area of cross-section (*A*) of the soil, and inversely proportional to the length of the soil sample (*L*). In other words,

$$Q \propto \frac{\Delta H}{L} . A^*$$

But $\frac{\Delta H}{L}$ represents the rate of loss of head, *i.e.* hydraulic gradient (i).

$$Q \propto i.A$$

or $$\boxed{Q = KiA} \qquad ...(4.2)$$

where K is the proportionality constant, and was found to be changing with the type of soil and hence represents a property of the soil, called ***permeability*** or ***coefficient of permeability.***

The above equation becomes dimensionally compatible only if K has the units of $\frac{L}{T}$ *i.e.* say cm/sec (*i.e.* the units of velocity).

Dividing both sides of eqn. (4.3) by A, we get

$$\frac{Q}{A} = K . i$$

where A is the area of the soil medium, through which the flow is taking place.

or $$v = Ki \qquad ...(4.3)$$

where v is the discharge velocity (into the well), and not the actual flow velocity (of ground water) through the soil medium. Since the actual flow velocity (v_a) occurs through the voids of cross-sectional area A_v and not through the area A, the permeability of the soil can be viewed as this superficial flow velocity under a unit hydraulic gradient. Also

$$A_v \times v_a = A . v$$

or $$v = \left(\frac{A_v}{A} v_a\right) \qquad ...(1)$$

when A is large in comparison, we can safely assume that the ratio of the area of the voids (A_v) to the total area (A) is the same as the ratio of the volume of the voids (V_v) to the total volume V

$$\therefore \quad \frac{A_v}{A} \approx \frac{V_v}{V} = n \text{ (Porosity)}$$

Substituting in (1) above, we get

$$\boxed{v = (n . v_a)} \qquad ...(4.4)$$

Knowing the value of v from Darcy's eqn. (4.3), and dividing it by porosity (n), the actual velocity of flow v_a through the soil can be worked out.

4.4.2. Empirical Formulas for Estimating Ground Water Velocity. Before Darcy came into picture, certain empirical formulae, based upon the

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*This Darcy's law, applicable to general viscous fluid flows, has now been demonstrated to be valid only for laminar flows [Reynold No. (R_e) value upto 1] which, as for as soils are concerned, is not at all a serious problem, because the flow in sands, silts, and clays is invariably laminar.

experimental results, were being used to find out the velocity of the ground water flow. The formulae which were in common use, are given below :

(1) **Slichter's formula.** According to which

$$v_a = K' i \frac{D_{10}^2}{\mu} \qquad ...(4.5)$$

where v_a = velocity of ground-water flow in m/day

K' = a non-dimensional constant, whose value is about 400 for adopted units

i = slope of the hydraulic gradient line

D_{10} = effective size of the particles in the aquifer in mm (*i.e.*, the hypothetical size which is larger than 10% of the particles in the sample, *i.e.*, only 10% of the particles will pass through this size)

μ = viscosity of water depending on temperature in centi pose*. Its value at 20°C = 1.

(2) **Hazen's formula.** Formula in M.K.S. system is

$$v_a = \frac{K''.i.D_{10}^2}{60} \times (1.8T + 42) \qquad ...(4.6)$$

where V = velocity in m/day

T = temperature in °C

K'' = a constant whose value is about 1000

Example 4.1. *Find out the velocities of the ground water flow with the following data using Slichter and Hazen's constants as 400 and 800, respectively.*

Viscosity coefficient of water at ground-water temperature of 10°C = 1.

Effective size of the particles in the aquifer = 0.1 mm.

Hydraulic gradient = 1 in 80.

Solution. (*a*) Using *Slichter's formula, i.e.*, Eq. (4.5), we have

$$v_a = \frac{K' i . D_{10}^2}{\mu} = \frac{400 \times \frac{1}{80} \times (0.1)^2}{1}$$

$$= \frac{400 \times 0.01}{80} = \mathbf{0.05\ m/day.}\quad \mathbf{Ans.}$$

(*b*) Using *Hazen's formula, i.e.*, Eq. (4.6), we have

$$v_a = \frac{K''.i.D_{10}^2}{60} (1.8T + 42) \text{ where } T \text{ is in °C}$$

---

*Poise is the CGS unit of dynamic viscosity, and

1 Poise = 10^{-1} Ns/m^2

1 Centipoise = $\frac{1}{100}$ Poise = 10^{-3} Ns/m^2 = 10^{-6} kNs/m^2

$$= \frac{800 \times \frac{1}{80} \times (0.1)^2}{60} (1.8 \times 10 + 42) \qquad [\because \quad T = 10°C]$$

$$= \frac{800 \times \frac{1}{80} \times \frac{1}{100} \times 60}{60} = \mathbf{0.1\ m/day.}\ \textbf{Ans.}$$

4.4.3. Field Measurement of the Velocity of Ground Water. The actual flow velocity of ground water (v_a) can be computed by using the above methods, or it can be measured in the field by using **chemical tracers** or **electrical resistivity methods.**

The known time (t) taken by a chemical tracer, such as a dye, to travel a given known distance (S) between the two observation wells, will directly indicate the ground flow velocity as equal to

$$v_a = \frac{S}{t} \qquad ...(4.7)$$

Such a test, in which a chemical tracer is introduced into one observation well, and its entry into the second observation well is observed, will also help to compute K, since

$$v = K \,.\, i = K \,.\, \frac{H_L}{S}$$

where H_L is the difference of water levels in the two observation wells, and

S is the distance between them.

or $$v = n \,.\, v_a = K \,.\, \frac{H_L}{S}$$

or $$K = n \,.\, v_a \,.\, \left(\frac{S}{H_L}\right) \qquad ...(4.8)$$

Example 4.2. *In a field test, a time of 6 hours was required for a tracer to travel through an aquifer from one well to another. The observation wells were 42 m apart and the difference in their water levels was found to be 0.42 m. Compute (i) the discharge velocity ; (ii) the coefficient of permeability (K). Given the porosity of the soil medium as 20% (iii) Also compute the value of coefficient of intrinsic permeability for the aquifer in Darcy's if viscosity of water* $\nu = 0.01\ cm^2/s$.

Solution. Velocity of ground water

$$= v_a = \frac{S}{t} = \frac{42\ \text{m}}{6\ \text{h}} = \frac{4200}{6 \times 60 \times 60}\ \text{cm/s}$$

$$= \mathbf{0.194\ cm/s}$$

Discharge velocity (v) = $\eta \,.\, v_a = 0.2 \times 0.194$ cm/s

$$= \mathbf{0.0388\ cm/s.}\ \textbf{Ans.}$$

Hydraulic gradient between the wells

$$= i = \frac{H_L}{S} = \frac{0.42\ \text{m}}{42\ \text{m}} = \frac{1}{1}$$

Using $v = K\,i = K \times \frac{1}{100}$

or $$0.0388 = \frac{K}{100}$$

$$K = \mathbf{3.88\ cm/s.}\quad \textbf{Ans.}$$

Intrinsic permeability coefficient is given by eqn. (4.12) as :

$$K_0 = \frac{K\,\nu}{g}$$

$$= \frac{3.88\,\frac{\text{cm}}{\text{s}} \times 0.01\,\frac{\text{cm}^2}{\text{s}}}{981\ \text{cm/sec}^2}$$

$$= \mathbf{3.96 \times 10^{-5}\ cm^2}$$

Since $\boxed{9.87 \times 10^{-9}\ \text{cm}^2 = 1\ \text{Darcy},}$ we have

$$K_0 \text{ in Darcy} = \frac{3.96 \times 10^{-5}}{9.87 \times 10^{-9}} \text{ Darcy's}$$

$$= \mathbf{4007\ Darcy's.}\quad \textbf{Ans.}$$

4.5. Coefficient of Permeability

The *coefficient of permeability*, is defined from Darcy's law, as the rate of flow of water through a unit cross-sectional area of the water bearing material under a unit hydraulic gradient, and at a temperature* of 20°C.

As a matter of fact, the *coefficient of permeability* or ***hydraulic conductivity***, reflects the combined effects of the porous medium and the fluid properties. From an analogy of laminar flow through a conduit (Hagen-Poiseuille flow), the coefficient of permeability can be expressed as :

$$K = C \,.\, d^2 \,.\, \frac{\gamma}{\mu} \qquad \ldots(4.9)$$

$$= \frac{C \,.\, d^2 \,.\, (\rho . g)}{\mu} = \frac{C \,.\, d^2 \,.\, g}{\left(\frac{\mu}{\rho}\right)}$$

or $$K = \frac{(C \,.\, d^2) \,.\, g}{\nu} \qquad \ldots(4.10)$$

where C = a constant, called **shape factor,** and depends on porosity, packing and grain size distribution of the porous medium

d = mean particle size of porous medium

γ = unit weight of water flowing through soil

μ = dynamic viscosity of water.

ν = kinematic viscosity of water.

In the above equation, C and d represents the properties of the porous medium, while ν represents the property of the fluid passing through the porous medium.

---

*This accounts for viscosity of water, which is found to be inversely proportional to K, as shown in eqn. (4.10).

The component of the permeability which represents only the properties of the porous medium is called **intrinsic permeability** *or* ***specific permeability***, and is represented by K_0, given by :

$$K_0 = C \,.\, d^2 \qquad ...(4.11)$$

The units of K_0 are m^2 or Darcy, where *1 darcy* = 9.87×10^{-13} m^2

The relation between K_0 and K works out from eqn. (4.10) and (4.11) as :

$$\boxed{K_0 = K \,.\, \frac{\nu}{g}} \qquad ...(4.12)$$

The coefficient K_0 is generally used when the water/fluid temperature varies considerably, or where more than one fluid is involved. However, in ground water flows, since the temperature variation is small and only water is involved, the coefficient K is more conveniently used.

The values of K for different types of soils are given in table 4.2.

Table 4.2. Permeability Coefficient (K) Values

S. No.	*Type of rock formation*	*App. av. value of K in cm/sec*
1.	Granite, quartzite	0.6×10^{-5}
2.	Slate, Shale	4×10^{-5}
3.	Limestone	4×10^{-5}
4.	Sandstone	0.004
5.	Sand and Gravel	0.4
6.	Only Gravel	4.0
7.	**Only Sand**	**0.04**
8.	Clay and Soil	0.04×10^{-5}

4.5.1. Relation between Coefficient of Permeability and Co-efficient of Transmissibility. The term *transmissibility* introduced by Theis is measured by the **coefficient of transmissibility (T)** which is defined as the rate of flow of water through a vertical strip of the water bearing material (*i.e.*, aquifer) of unit width and full depth (d), under a unit hydraulic gradient and at a temperature of 20°C.

The relation between K and T is simple, being given by

$$T = K.d \qquad ...(4.13)$$

4.6. Drainage of Ground Water

By the term 'drainage of ground water', we generally mean extracting the water from below the watertable through wells, infiltration galleries, springs, etc. The water is, thus, drained from the ground-water reservoir, either under a natural phenomenon (like spring), or it can be drained artificially by constructing wells and lifting the water from them. The water so drained, may then be used to fulfil irrigation or municipal needs.

4.7. Ground Water Yield

The interstices present in the given formation get filled up with water during the process of ground-water replenishment. If all these voids are completely

filled with water, then it is known as saturated formation. The water contained in these voids is drained by digging wells under the action of 'gravity-drainage' (explained later). When these saturated formations are drained under the action of 'gravity drainage', it is found that the volume of water so drained is less than the volume of the void space as indicated by its porosity. This is because of the fact, that the entire water contained in these voids cannot be drained out by the mere force of gravity. Some of the water is being retained by these interstices due to their molecular attraction. The water so retained is known as ***pellicular water.***

4.7.1. Specific Yield (S_y). The volume of ground-water extracted by gravity-drainage from a saturated water bearing material is known as the yield, and when it is expressed as the ratio of the volume of the total material drained, then it is known as *specific yield.*

$$\therefore \quad \text{Sp. yield } (S_y) = \frac{\text{Volume of water obtained by gravity drainage}}{\text{Total volume of the material drained or dewatered}} \times 100 \qquad ...(4.14)$$

In order to give a higher specific yield, the acquifer material must be porous and should have high permeability. Porosity, being the percentage of voids (pores), represents the maximum amount of water that a formation can store. However, porosity along cannot ensure higher specific yield (of ground water), since ground water cannot get stored (or withdrawn) from a formation with any greater amount of porosity, unless and until its permeability is also high, as to permit flow of water through it. Since a rock or a soil, which is porous, may or may not be permeable ; the higher porosity alone cannot give higher specific yield. For example, shale rock has a medium porosity (1 to 10%), but its pore spaces are so minute that the rock remains impermeable with permeability of the order of 10^{-10} to 10^{-14} cm/sec. Due to its low permeability, its specific yield is hardly 0.5 to 5%.

Typical range values of specific yields, porosity and permeability of different types of soils or rock formations are given in table 4.3.

Table. 4.3. Typical Range Values for Specific yield, Porosity and Permeability for Different types of soil/rock formations

Material	*Porosity* (%)*	*Coefficient of Permeability (cm/sec)*	*Specific yield (%)*
(1)	*(2)*	*(3)*	*(4)*
Clay	45–55	10^{-5}–10^{-9}	1–10
Silt	35–50	0.001–10^{-7}	5–10
Sand fine	30–35	0.001–0.05	10–20
Sand mixed	20–35	0.005–0.01	15–30
Gravel	30–40	1–100	15–30
Sand stone	10–20	0.001–0.0001	5–15
Shale	1–10	10^{-10}–10^{-14}	0.5–5
Lime stone	1–10	0.001–0.00001	0.5–5

*Porosity between 0–5% is known as *low*; between 5–20% as *medium* ; and between 20–40% as *high.*

4.7.2. Specific Retention (S_r) or Field Capacity. On the other hand, the quantity of water retained by the material against the pull of gravity is termed as *the specific retention or the* ***field capacity***, and this is also expressed as percentage of the total volume of the material drained.

Specific retention or Field capacity

$$= \frac{\text{Volume of the water held against gravity drainage}}{\text{Total volume of the material drained}} \times 100 \quad ...(4.15)$$

It is evident that the sum of the specific yield and the specific retention is equal to its porosity; *i.e.*

$$S_y + S_r = n \quad ...(4.16)$$

4.7.3. Specific Retention of Different Kinds of Formations. As has been said earlier, the specific retention is the amount of the water held between the grains due to molecular attraction. This film of water is thus held by molecular adhesion on the walls of the interstices. Therefore, the amount of this water will depend upon the total interstitial surface in the rock. If the total interstitial surface is more, the specific retention will be more and *vice versa*.

Now, if the effective size of the grains decreases, the surface area between the interstices will increase, leading to more specific retention and less specific yield.

It, *therefore, follows that, in fine soils like clay, the specific retention would be more. and hence, such soils would result in very small specific yields.*

The reverse is also true when the grain size increases ; the interstitial surface area reduces, and, therefore, sp. retention reduces and hence sp. yield increases. *It, therefore, follows that in large particle soils like coarse gravels, the specific retention would be small and it would result in large specific yields.*

This conclusion is very important from practical stand point, because it follows from this *that a water bearing formation of coarse gravel would supply large quantities of water to wells, whereas, clay formations although saturated and of high porosity, would be of little value in this respect.* Hence, the location of the wells depends considerably upon the type of neighbouring formations.

4.7.4. Determination of Specific Yield of a Formation. It is difficult to determine accurately the specific yield in the laboratory, because it is difficult to obtain an undisturbed soil sample, and also the short sample columns used in the laboratory cannot duplicate the very long capillary tubes actually existing in the field. And hence, the field observations are made through the medium of pumping tests at site, in order to determine accurate and reliable values of specific yields, as will be explained later.

4.8. Aquifers and Their Types

A permeable stratum or a geological formation of permeable material, which is capable of yielding appreciable quantities of ground-water under gravity, is known as an ***aquifer***. The term 'appreciable quantity' is relative, depending upon the availability of the ground-water. In the regions, where ground-water is available with great difficulty, even fine-grained materials containing very less quantities of water may be classified as principal aquifers.

When an aquifer is overlain by a confined bed of impervious material, then this confined bed of overburden is called an **aquiclude,** as shown in Fig. 4.4.

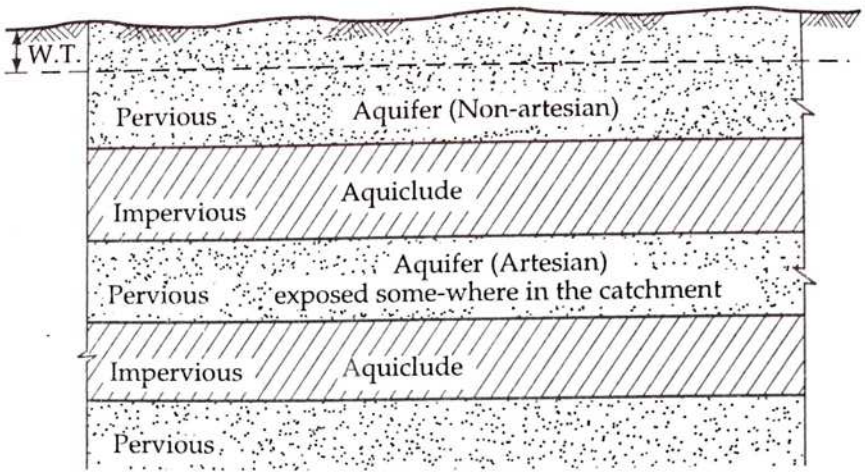


Fig. 4.4. Aquifers and Aquicludes.

The amount of water yielded by a well excavated through an aquifer, depends on many factors ; some of which, such as well diameter, are inherent in the well itself. But all other things being equal, the permeability and the thickness of the aquifer are the most important.

Aquifers vary in depth, lateral extent, and thickness ; but in general, all aquifers fall into one of the two categories, *i.e.*,

1. *Unconfined* or *Non-artesian aquifers* ; and
2. *Confined* or *Artesian aquifers*, as discussed below :

4.8.1. Unconfined Aquifers or Non-artesian Aquifers. The top most water bearing stratum having no confined impermeable over burden (*i.e.*, aquiclude) lying over it, is known as an *unconfined* aquifer or *non-artesian* aquifer (Refer Fig. 4.5).

The ordinary gravity wells of 2 to 5 m diameter, which are constructed to tap water from the top most water bearing strata, *i.e.*, from the unconfined aquifers, are known as *unconfined* or *non-artesian* wells. *The water levels in these wells will be equal to the level of the watertable.* Such wells are, therefore, also known as ***gravity wells or water-table wells***.

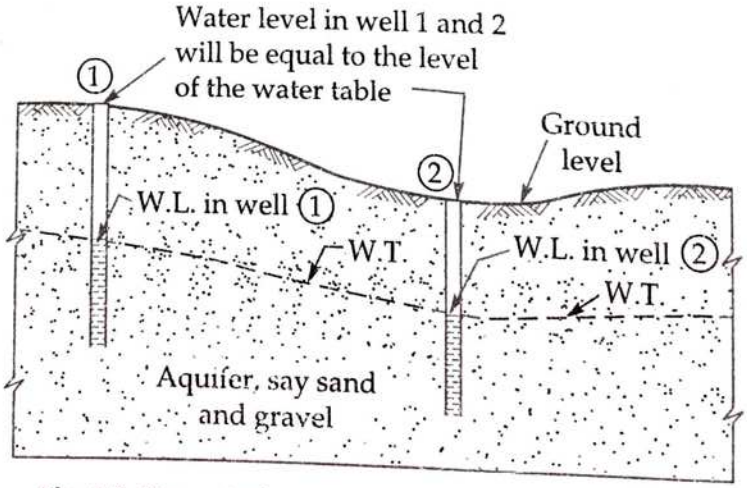


Fig. 4.5. Non-artesian or Unconfined aquifers and wells.

4.8.2. Confined Aquifers or Artesian Aquifers. When an aquifer is confined on its upper and under surface, by impervious rock formations (*i.e.*, aquicludes), and is also broadly inclined so as to expose the aquifer somewhere to the catchment area at a higher level for the creation of sufficient hydraulic head, it is called a *confined aquifer* or an *artesian aquifer*. A well excavated through such an aquifer, yields water that often flows out automatically, under the hydrostatic pressure, and may thus, even rise or gush out of surface for a reasonable height. However, where the ground profile is high, the water may remain well below the ground level. The former type of artesian wells, where water gushes out automatically, are called **flowing wells.**

The level to which water will rise in an artesian well is determined by the highest point on the aquifer, from where it is fed from the rains falling in the catchment (*i.e.*, by discharge). However, the water will not rise to this full height, because the friction of the water moving through the aquifer uses up some of the energy.

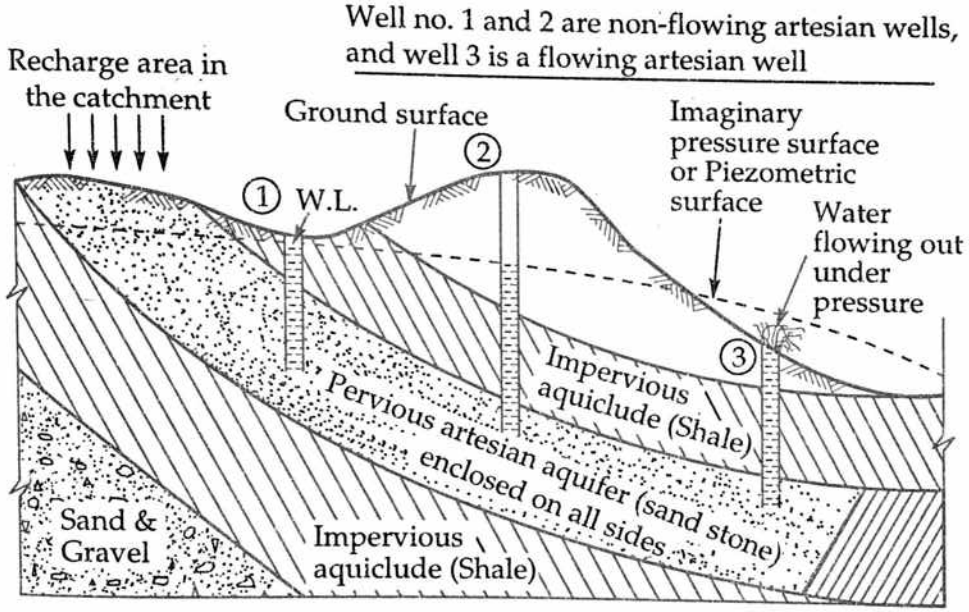


Fig. 4.6. Confined or Artesian aquifers and wells.

The question whether it will be a *flowing artesian well* or a *non-flowing artesian well* depends upon the topography of the area, and is not the inherent property of the artesian aquifer. In fact, if the ***pressure surface**** lies above the ground surface, the well will be a flowing artesian well, whereas, if the pressure surface is below the ground surface, the well will be artesian but non-flowing, and will require a pump to bring water to the surface, as shown in Fig. 4.6. Such *non-flowing artesian wells* are sometimes called ***sub-artesian wells.***

4.8.3. Perched Aquifers. Perched aquifer is a special case which is sometimes found to occur within an unconfined aquifer.

*Piezometric surface is the imaginary surface which represents the magnitude of hydrostatic pressure available along the artesian aquifer. It is represented by the line joining the various piezometric heads in various tightly cased wells tapping the aquifer.

Sometimes, a lens or a localised patch of an impervious stratum can occur inside an unconfined aquifer in such a way that it retains a water table above the general water table, as shown in Fig. 4.7. Such a water table retained around the impervious material is known as a ***perched water table***, while the supported body of the saturated material is known as a *perched aquifer*. Usually, the perched water table is of limited extent, and the ground water yield from such geological occurrence is very small. In ground water exploration, a perched water table is quite often confused as the general water table.

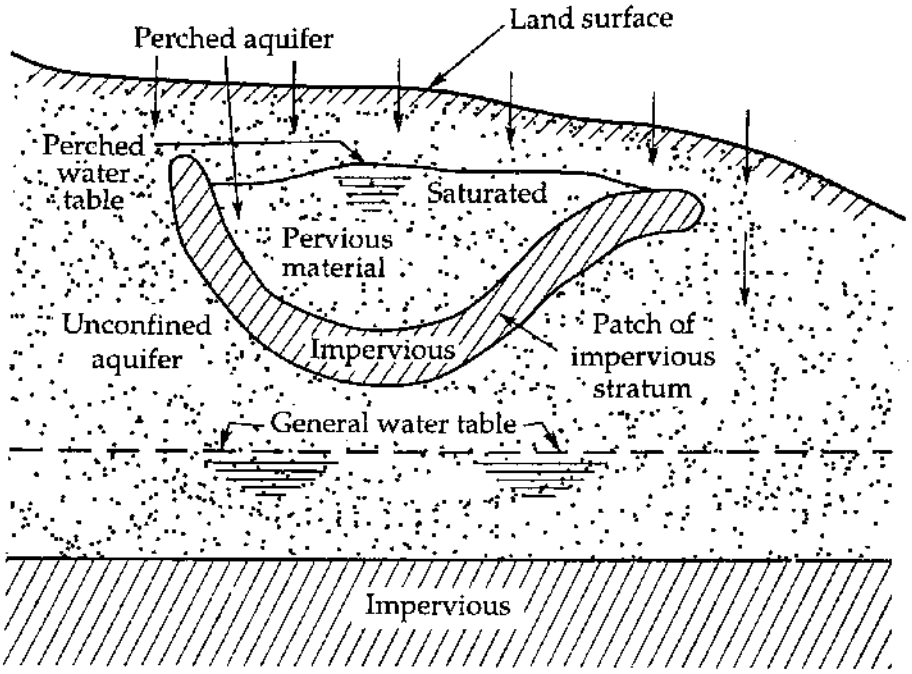


Fig. 4.7. Perched aquifer.

4.8.4. Difference between Aquifuge, Aquitard and Aquiclude

Aquifuge is that geological formation, which is, neither porous nor permeable ; and hence it neither contains nor yields ground water. *Granite rock is an example of aquifuge.*

Aquitard is that geological formation, which does not yield water freely to wells due to its lesser permeability, although seepage is possible through it. The yield from such a formation is, thus, insignificant. *Sandy clay is an example of aquitard.*

Aquiclude is highly porous, containing large quantities of water, but essentially impervious, as not to yield water. *A clay layer is an example of aquiclude.*

4.9. Certain Other Important Terms

4.9.1. Specific Capacity. Specific capacity of a well is the rate of flow from a well per unit of drawdown. It should be determined for the fall of the first metre, as it is not the same for all the drawdowns.

4.9.2. Coefficient of Storage or Storage Coefficient or Specific Storage or Storativity (S). In general, *storage coefficient (S) is defined as the volume of water that an aquifer releases or stores per unit surface area per unit decline or rise of water table.*

For an unconfined aquifer, storage coefficient is generally taken as equal to the specific yield S_y – *i.e.* volume of water drained out by the force of gravity per unit volume of the fully saturated aquifer, because most of the water from storage is released by the action of gravity with a negligible part from the compression of the aquifer or the expansion of the water.

Hence, for an unconfined aquifer, $S = S_y$

For a confined aquifer (*i.e.* artesian aquifer), however, compression of the aquifer and expansion of water comes into play, as and when pumping is resorted to.

In such a confined aquifer, the hydrostatic pressure within an aquifer partially supports the over burden, while the remainder is supported by the soil structure of the aquifer. When the hydrostatic pressure is reduced by pumping from a well penetrating the aquifer, the aquifer load increases. The aquifer gets compressed and forces some water from it. In addition, lowering of the pressure causes a small expansion and release of water. *The water yielding capacity of such an aquifers can be best expressed by its storage coefficient (S), which for artesian aquifers may be defined as equal to the volume of water released from the aquifer of full height and unit cross-sectional area when the piezometric surface declines by unity.*

Storage coefficient for confined aquifers is given by the equation :

$$S = \gamma_w H \left[\alpha + n\beta\right] \qquad ...(4.17)$$

where S = storage coefficient

γ_w = Unit weight of water = $\rho_w . g$ kN/m^3

H = Confined aquifer thickness in m

α = Compressibility of the aquifer

$= \dfrac{1}{E_s}$, where E_s is the elasticity of the aquifer

n = Porosity of the aquifer material

β = Compressibility of water

$= \dfrac{1}{E_w}$, where E_w is the elasticity of water.

$$\therefore \qquad S = \gamma_w . H \left[\frac{1}{E_s} + n\frac{1}{E_w}\right] \qquad ...(4.17a)$$

The value of E_w at about 10°C is 2.1×10^{-6} kN/m^3. The typical values of E_s for some important types of aquifer materials are indicated in table 4.4 below :

Table 4.4. Typical Values of Elasticity (E_s) for Certain Aquifer Materials

S. No.	*Aquifer material*	*Bulk modulus of elasticity (E_s) in kN/m^2*
1.	Loose sand	10^4 to 2×10^4
2.	Dense sand	5×10^4 to 8×10^4
3.	Dense sandy gravel	10^5 to 2×10^5
4.	Loose clay	10^3 to 5×10^3
5.	Stiff clay	10^4 to 10^5

Typical values of S for confined aquifers varies from 5×10^{-3} to 5×10^{-5}.

Example 4.3. (*a*) *For an aquifer located at a certain place, a zone of 900 sq km is bounded by a confined aquifer of 20 m thickness. The average maximum and minimum piezometric level variations range between 4–10 m. Calculate the annual rechargeable ground water storage from this area, assuming the value of the storage coefficient = 0.0005.*

(*b*) *If the area is having about 40 wells, each pumping water for irrigation for about 200 days in a year, calculate the average well yield.*

Solution. (*a*) Annual Rechargeable ground water storage (m^3)

= Area in m^2 × Variation in piezometric head in m × Storage coefficient

= $(900 \times 10^6\ m^2) \times (10 - 4)\ m \times 0.0005$

= $900 \times 10^6 \times 6 \times 5 \times 10^{-4} = 270 \times 10^4\ m^3$

= $2.7\ Mm^3$

(*b*) Annual storage fluctuation

= Average well yield/day × Pumping days × No. of wells

$\therefore$ $2.7\ Mm^3$ = Average well yield/day × 200 days × 40 wells

$$\therefore \text{ Average well yield} = \frac{2.7 \times 10^6\ m^3/day}{200 \times 40}$$

$$= 337.5\ m^3/d = \frac{337.5 \times 1000\ l}{24 \times 3600\ s} = 3.8 \text{ litres/sec.}$$ **Ans.**

VARIOUS FORMS OF UNDERGROUND SOURCES AND THEIR EXPLOITATION

The underground water is generally available in the following forms :

(1) Infiltration galleries ; (2) Infiltration wells ;

(3) Springs ; and (4) Wells including tubewells.

These forms are discussed below :

4.10. Infiltration Galleries

Infiltration galleries are the horizontal or nearly horizontal tunnels constructed at shallow depths (3 to 5 metres) along the banks of rivers through the water bearing strata, as shown in Fig. 4.8. They are sometimes called *horizontal wells*.

An infiltration gallery is generally constructed of masonry walls with roof slab, and extracts water from the aquifer by various porous lateral drain pipes located at suitable intervals along the length of the gallery. These pipes are generally covered with gravel, so as to prevent the entry of the fine sand particles into the pipes. These tunnels or galleries are generally laid at a slope, and the water collected in them is taken to a sump well, from where it is *pumped*, treated and distributed to the consumers. These infiltration galleries are quite useful when water is available in sufficient quantity just below the ground level or so.

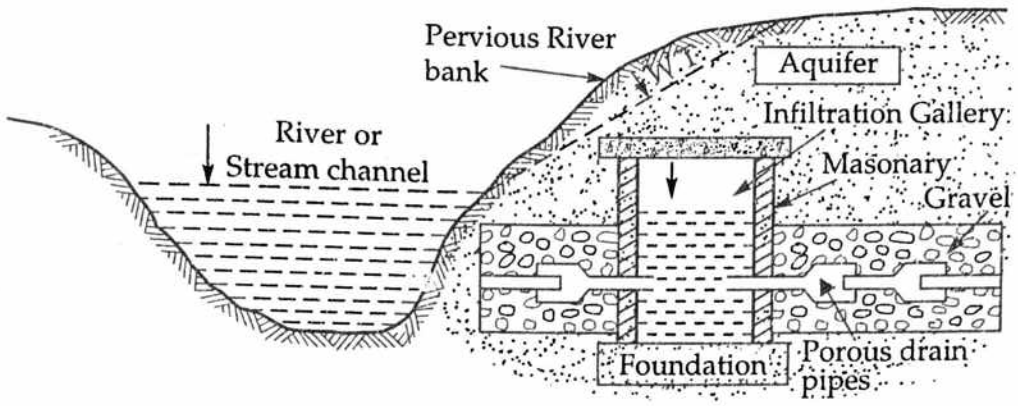


Fig. 4.8. Section of an Infiltration gallery.

The infiltration galleries may have width of about 1 m, depth of about 2 m and length varying from 10 m to as long as 100 m, depending on the extent of the water field. They are seldom deeper than 6 m below the ground level. The shaft or gallery of sufficient length is fed at intervals by lateral pipes, as stated above.

The flow pattern into an infiltration gallery under pumping equilibrium is shown in Fig. 4.9, as the drawdown curve follows the Dupuit's drawdown

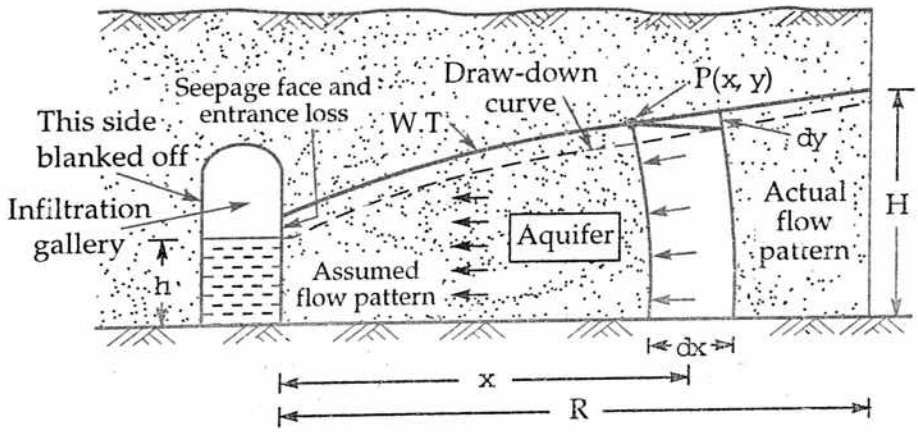


Fig. 4.9. Flow pattern in an Infiltration gallery.

curve*. The discharge from the gallery can be computed by using Darcy's law as :

$$Q = KiA$$

Now, consider a point P on the drawdown curve having coordinates (x, y). On considering a small element of width dx, the head gradient will be $\frac{dy}{dx}$ and the area of soil through which flow will occur will be $y \times L$, where L is the length of the gallery ($\perp$ to paper).

$$\therefore \quad \text{discharge } Q = \int K \cdot \left(\frac{dy}{dx}\right)(y \cdot L) \qquad \ldots(4.18)$$

*Explained under Article 4.15

or $$Q \cdot \int_{x=0}^{x=R} dx = KL \int_{y=h}^{y=H} y \cdot dy$$

where x varies between O to R

where R is the radius of influence*

and y varies between h and H

where h = depth of water in the gallery on pumping at equilibrium

H = Static head *i.e.* height of initial water level above the bottom of gallery

$$\therefore \quad Q \left| x \right|_{x=0}^{x=R} = K \cdot L \cdot \left| \frac{y^2}{2} \right|_{y=h}^{y=H}$$

or $$Q \cdot R = K \cdot L \cdot \left(\frac{H^2 - h^2}{2} \right)$$

or $$\boxed{Q = KL \cdot \left(\frac{H^2 - h^2}{2R} \right)} \qquad ...(4.19)$$

where K = Permeability coefficient of the aquifer

L = Length of gallery ($\perp$ to paper)

R = radius of influence

H = static water level above the bottom of the gallery

h = depth of water in the gallery on pumping equilibrium.

Sometimes, horizontal perforated pipes are laid in place of rectangular tunnels, and their perforations are covered with gravel so as to prevent sand entry. These pipes may be called *infiltration pipes* and are useful when the available ground water is small in quantity.

Example 4.4. *600 m^3/day of water is to be obtained from a proposed infiltration gallery, which is placed at 6 m depth from sub-surface water-table. The coefficient of permeability of the soil aquifer is 100 m/day. Find the length of the gallery if the drawdown in the gallery on pumping is not to exceed 4 m. The radius of influence may be assumed to be 100 m.*

Solution. Using Eq. (4.19) for discharge of an infiltration gallery, we have

$$Q = K \cdot L \cdot \left(\frac{H^2 - h^2}{2R} \right)$$

where $Q = 600$ m^3/d, $H = 6$ m,

$K = 100$ m/day $R = 100$ m,

$h = 6 - 4 = 2$ m,

*Defined under Article 4.13.1.

Substituting values, we get

$$600 \text{ m}^3/\text{d} = 100 \text{ m/d} \times L \text{ in m} \times \left(\frac{6^2 - 2^2}{2 \times 100}\right)\frac{\text{m}^2}{\text{m}}$$

or $$600 = \left(100 \times L \times \frac{32}{200}\right)$$

or $$L = \frac{600 \times 200}{100 \times 32} = 37.5 \text{ m.} \quad \textbf{Ans.}$$

4.11. Infiltration Wells

Infiltration wells are the shallow wells constructed in series along the banks of a river, in order to collect the river water seeping through their bottoms, as shown in Fig. 4.10.

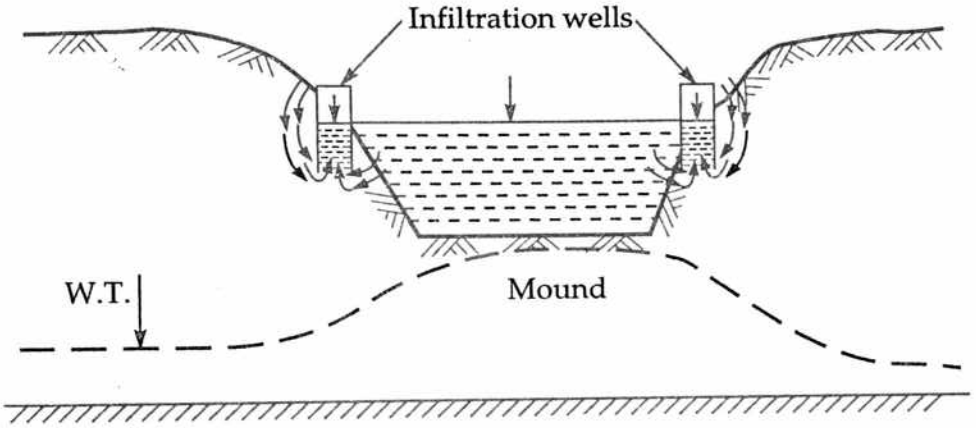


Fig. 4.10. Locations of Infiltration wells.

These wells are generally constructed of brick masonry with open joints. They are generally covered at the top and kept open at the bottom, as shown in Fig. 4.11. For inspection purposes, manholes are provided in the top slab cover.

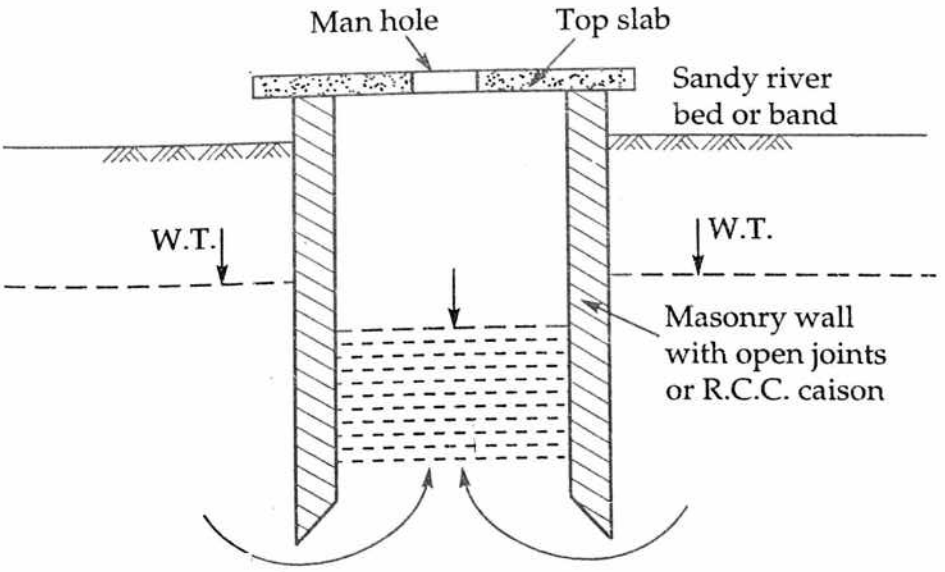


Fig. 4.11. Section of an Infiltration well.

The various infiltration wells are connected by porous pipes to a *sump well,* called **jack well,** as shown in Fig. 4.12. The water reaching the jack well from the different infiltration wells is lifted, treated and distributed to the consumers.

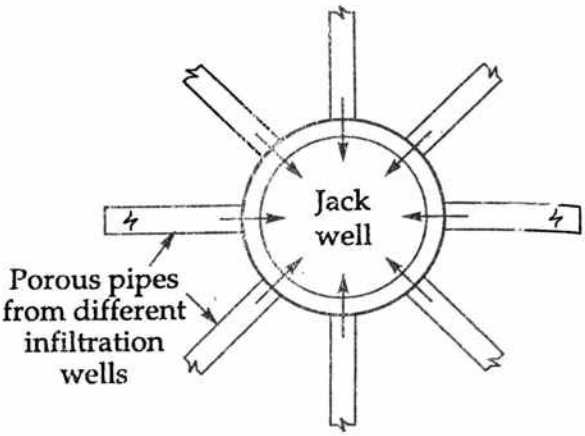


Fig. 4.12. Plan of a Jack well receiving water from a number of infiltration wells.

Ranney wells. A new technique that has recently started is to construct a vertical well 3 to 6 m in diameter with horizontal radial collectors, and is known as **radial well.** Such a well is sunk to the required level and *plugged at the bottom.* Horizontal perforated steel pipes are then driven just at the level of the aquifer in the well by powerful hydraulic jacks. The length of these pipes or *radial collectors* may be of the order of 60 to 80 m or so. About 10 radial collectors can be installed at one level. Some other set or sets of such radial collectors can be installed at other levels, if possible, so as to increase the yield. The inner end of each collector pipe is fitted with a sluice valve, which can be operated from the pump house above. The inflow of water into the well is thus controlled. The water from the well can possibly be pumped directly into the mains, as the water obtained by this method is generally clean, fresh and free from bacterial contamination. These wells are, therefore, very useful for drawing water from polluted streams. This type of well construction is very common in *France,* and is sometimes referred to as *French system of tapping underground water.*

A patented type of a radial well is known as a **ranney well** or a *ranney collector.* It consists of a R.C.C. caisson of 4.3 m (13′) in diameter, 0.45 m (18″) thick, which is sunk into the ground up to the required level, and from which radial collectors are projected, as explained above.

4.12. Springs

The natural outflow of ground water at the earth's surface is said to form a *spring.* A pervious layer sandwiched between two impervious layers, give rise to a natural spring. A spring indicates the outcropping of the watertable.

The springs are generally capable of supplying very small amounts of waters, and are, therefore, generally not regarded as sources of water supplies. However, good developed springs may sometimes be used as water supply sources for small towns, especially in hilly areas. Certains springs, sometimes discharge hot water due to the presence of sulphur in them. These hot springs (such as the one in Sohana in Haryana ; a group on the bed and banks of Sutlej river at Tattapani near Simla ; and also another at Manikaran near Manali on Parvati river in H.P. state) usually emit sulphur mixed water (warm to boiling), and hence cannot be used for water supplies, though sometimes useful for taking dips for the cure of certain skin ailments.

4.12.1. Formation and Types of Springs. Springs are usually formed under three general conditions of geological formations, as explained below :

(*a*) **Gravity springs.** When the ground watertable rises high and water overflows through the sides of a natural valley or a depression (as shown in Fig. 4.13), the spring formed is known as a *gravity spring*. The flow from such a spring varies with the rise or fall of the watertable.

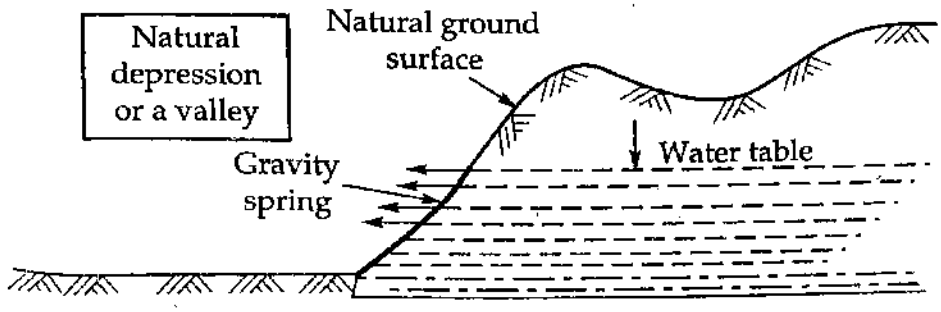


Fig. 4.13. Gravity spring.

(*b*) **Surface springs.** Sometimes, an impervious obstruction or stratum supporting the underground storages becomes inclined (such as shown in Fig. 4.14), causing the watertable to go up and get exposed to the ground surface. This type of spring is known as *surface spring*. The quantity of water available from such a spring is quite uncertain.

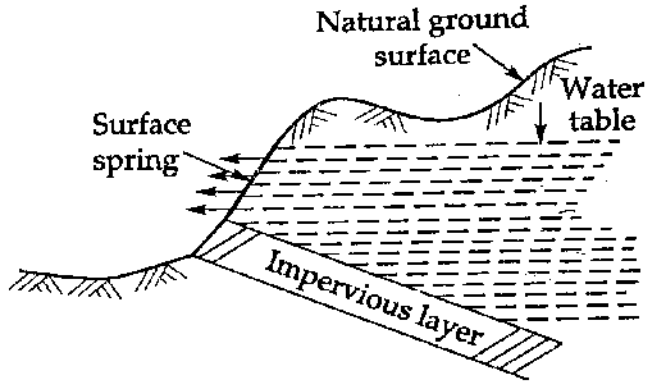


Fig. 4.14. Surface spring.

(*c*) **Artesian springs.** When the above storage is under pressure (*i.e.* the water is flowing through some confined aquifer), such as shown in Fig. 4.15, the spring formed is known as *artesian spring*. This type of springs are able to provide almost uniform quantity of water. Since the water oozes out under pressure, they are able to provide higher yields and may be thought of as the possible sources of water supply.

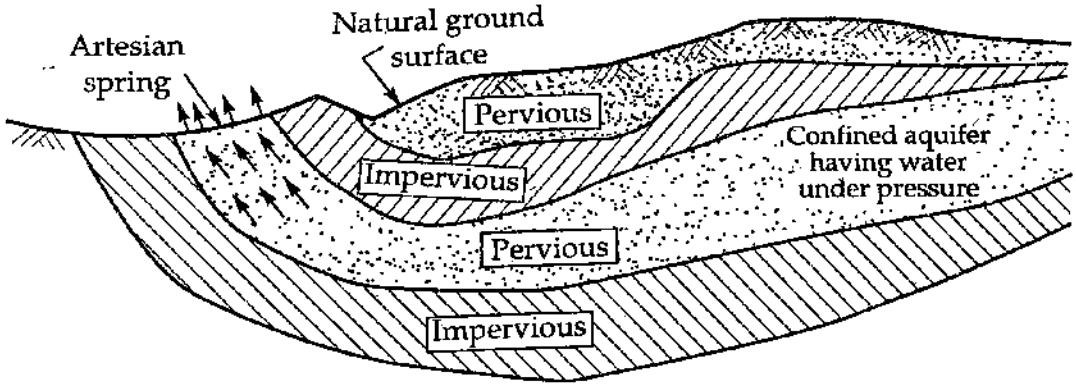


Fig. 4.15. Artesian spring.

When a spring issues out of the ground, a lake or a pond gets formed. Pucca masonry walls are, therefore, constructed on four sides of this tank or pond which is covered at top. Proper cleaning of the area and arrangements for

excluding the surface waters from entering the spring should be properly made, so as to avoid contamination of spring water. The water collected in the tank or pond can be carried through pipes into the downstream reservoir for supplies.

WELLS

A water well is a hole usually vertical, excavated through the earth strata for lifting ground water to the surface. The wells may be classified into two types :

(1) *Open wells* ; and

(2) *Tube wells.*

4.13. Open Wells or Dug Wells

Smaller amount of ground water has been utilised from ancient times by open wells. Open wells are generally open masonry wells having comparatively bigger diameters, and are suitable for low discharges of the order of 18 cubic metres per hour (*i.e.* about 0.005 cumecs). The diameters of the open wells generally vary from 2 to 9 m, and they are generally less than 20 m in depth. The walls of an open well may be built of precast concrete rings or in brick or stone masonry, the thickness generally varies from 0.5 to 0.75 m, according to the depth of the well (See Fig. 4.16).

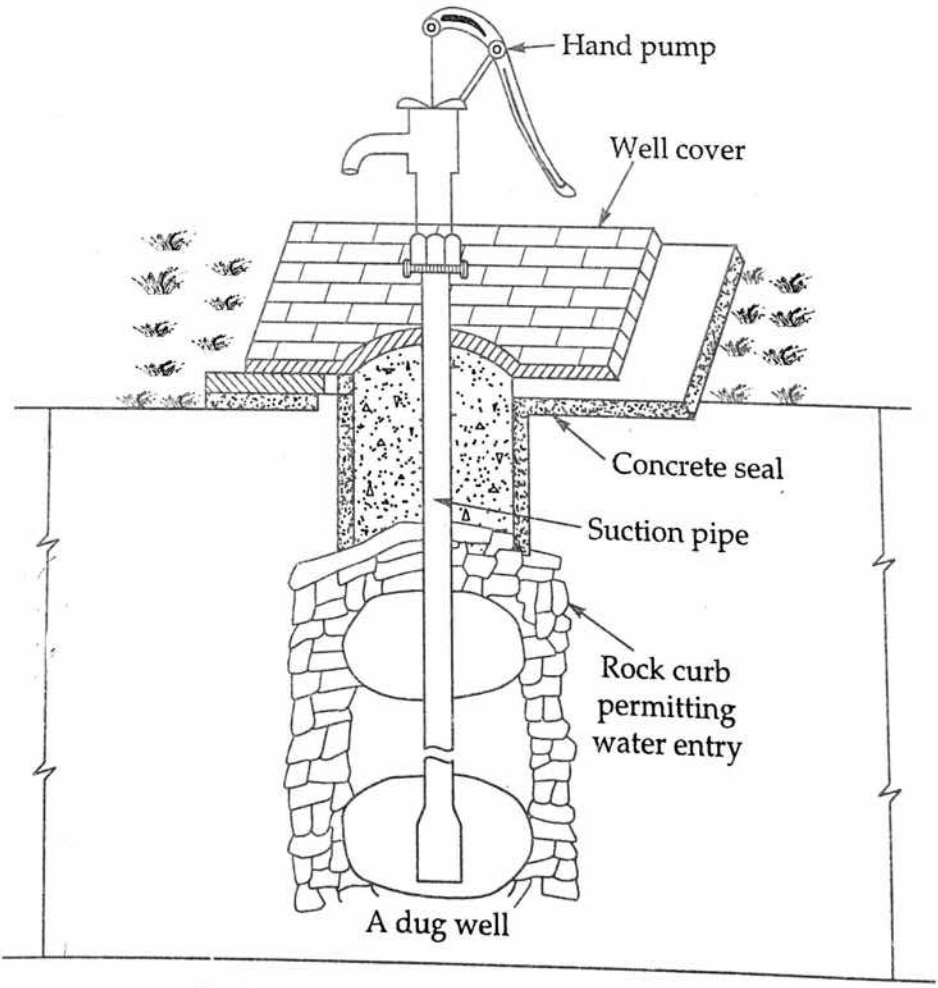


Fig. 4.16. Open well fitted with a hand pump.

The yield of an open well is limited because such wells can be excavated only to a limited depth, where the ground water storage is also limited.

Moreover, in such a well, the water can be withdrawn only at the critical velocity for the soil. Higher velocities cannot be permitted, as that may lead to the disturbance of the soil grains, and the consequent subsidence of the well lining in the hollow so formed. The limit placed on velocity, therefore, also limits the maximum possible safe discharge of an open well.

One of the recent methods used to improve the yield of an open well is to put in a 8 to 10 cm diameter bore hole in the centre of the well, so as to tap the additional water from an aquifer or from fissures in the rock. If a clay or a kankar layer is available at a smaller depth so as to support the open masonry well, a bore hole can be made in its centre so as to reach the sand strata. Such an arrangement will not only give a structural support to the open well but will also considerably increase its yield. Depending upon the availability of such a provision, the open wells may be classified into the following two types :

(*a*) *Shallow open wells* ; and

(*b*) *Deep open wells.*

A shallow open well is the one which rests in a pervious strata and draws its supplies from the surrounding materials. On the other hand, a deep open well is the one which rests on an impervious '*mota*' layer and draws its supply from the pervious formation lying below the mota layer, through a bore hole made into the '*mota*' layer, as shown in Fig. 4.17. The term "mota layer", also sometimes known as "Matbarwa" or "Magasan", refers to a layer of clay, cemented sand, kankar or other hard materials, which are often found lying a few metres below the watertable in the sub-soil. These names are not applied to the layers of hard material lying above the watertable. The main advantage of such a mota later lies in giving structural support to the open well resting on its surface. It is useful for unlined and partly lined wells, and is indispensable for a heavy masonry well which would not remain stable under steady use without such a support. The mota layer is generally found throughout the Indo-Gangetic plain. These mota layers may either be continuous or may be localised, and are generally found in different thicknesses and depths at different places.

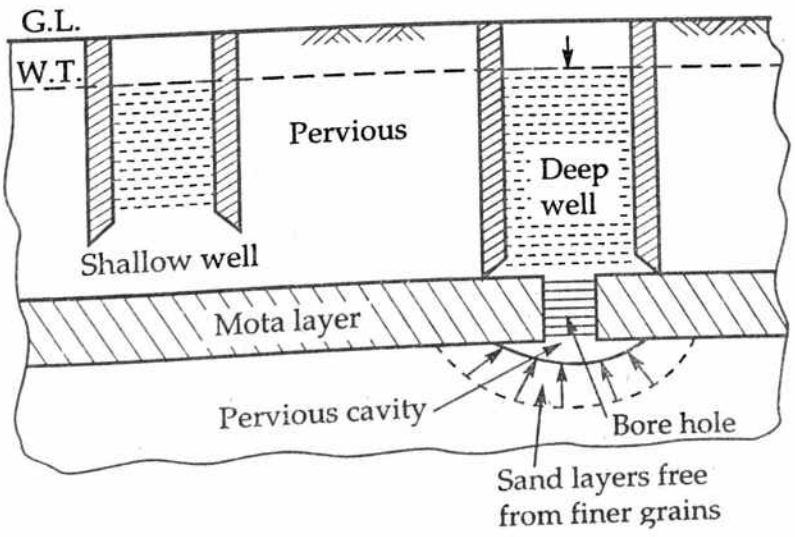


Fig. 4.17. Shallow and Deep open wells.

The nomenclature of the shallow and deep open wells is purely technical, and has nothing to do with the actual depth of the well. A "shallow open well" might be having more depth than a "deep open well"

Since a shallow open well draws water from the topmost water bearing stratum, its water is liable to be contaminated by the rain water percolating in the vicinity and taking with it minerals or organic matter such as decomposing animals and plants, etc. The water in a deep open well, on the other hand, is not liable to get such impurities and infections. Secondly, the pervious formations below the mota layer generally contain greater quantities of ground water, yielding higher specific yields. Hence, larger discharge and larger supplies can be obtained from a deep open well as compared to those from a shallow open well.

Water is generally drawn from dug or open wells by means of a bucket and a rope. However, due to the possible surface contamination of water in an uncovered well and also the individual buckets adding contamination to the water, such open wells have been covered in many parts of India and fitted with *hand pumps* (Fig. 4.16). A hand pump uses a reciprocating type of a pumping arrangement to lift water and is described in details in chapter 7.

4.13.1. Cavity Formation in Open Wells. Consider a well from which no water is being withdrawn. The water level in such a well will obviously be the same as is the static watertable outside the well. Now, if a discharge is withdrawn from this well at a constant rate, the level in the well will go down and stabilise at a lower level than that of the outside watertable. The head difference between these two levels is called the **depression head** (Fig. 4.18). Under the influence of this head difference, water enters the well from outside, so as to fill the gap created by the withdrawn water. As the water from the surrounding soil travels towards the well, there is a gradual loss of head, and water surface drops towards the well. Since the same discharge is passing through the reducing soil areas as it approaches the well, there is a gradual increase in the flow velocity towards the well. Now according to Darcy's law, this velocity can gradually increase only if the hydraulic gradient gets gradually increased. Hence, the water surface will fall gently in the beginning and will fall more and more rapidly as it approaches the well. The surface of watertable surrounding the well, therefore, takes up a curved shape, and is called **cone of depression.** At a certain distance from the well, there is no appreciable depression of water-table. This distance from the central line of the well is called the ***radius of influence*** of the well.

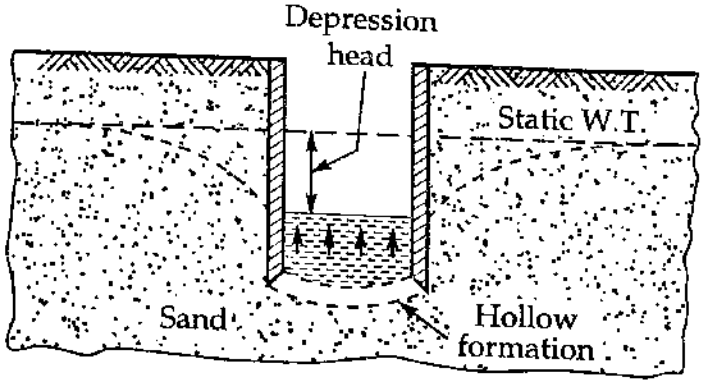


Fig. 4.18. Cavity formation in Dug wells.

The velocity of percolating water into the well depends upon the depression head. If more amount of water is withdrawn from the well, thereby increasing the depression head, higher flow velocities will prevail in the vicinity of the well. Thus, at a certain rate of withdrawal, it is very much possible that the

flow velocity may exceed the critical velocity for the soil, thereby causing the soil particles to lift up. As more and more sand particles are lifted, a hollow is created in the bottom of the well, resulting in the increased effective area, so that ultimately, the velocity falls below the critical value and then no further sand goes out of the well.

As pointed out earlier, the formation of such hollows beneath the wells are dangerous in shallow open wells, because there is always a danger of subsidence of the well lining. *The maximum rate of withdrawal from such wells is, therefore, limited.*

In case of a deep open well resting on mota layer, the cavity or hollow formation below the bore hole (Fig. 4.17) is not dangerous, because the well lining remains supported on the mota layer. Hence, a hollow, much larger in area than the cross-sectional area of the well, may safely form in deep wells, and thereby giving higher yields. In a shallow open well of an equivalent yield, the well area will have to be increased equal to the area of the cavity under the deep well, which would make it costlier.

4.13.2. Construction of Open Wells. From the construction point of view, the open wells may be classified into the following three types :

Type I. Wells with an impervious lining, such as masonry lining, and generally resting on a mota layer.

Type II. Wells with a pervious lining, such as dry brick or stone lining, and fed through the pores in the lining.

Type III. No lining at all *i.e.*, a Kachha well.

Type I. *Wells with impervious lining.* They provide the most stable and useful type of wells for obtaining water supplies. For constructing such a well, a pit is first of all excavated, generally by hand tools, up to the soft moist soil. Masonry lining is then built up on a *kerb* upto a few metres above the ground level. A "kerb" is a circular ring of R.C.C., timber or steel having a cutting edge at the bottom and a flat top, wide enough to support the thickness of the well lining called "steining". The kerb is then descended into the pit by loading the masonry by sand bags, etc. As excavation proceeds below the kerb, the masonry sinks down. As the masonry sinks down, it is further built up at the top. To ensure vertical sinking, plumbobs are suspended around the well steinning, and if the well starts tilting, it may be corrected by adjusting the loads or by removing the soil from below the kerb which may be causing the tilt. The well lining (steinning) is generally reinforced with vertical steel bars.

After the well has gone up to the watertable, further excavation and sinking may be done either by continuously removing the water through pumps, etc., or the excavation may be carried out from top by *Jhams*. A *Jham* is a self-closing bucket which is tied to a rope and worked up and down over a pulley. When the *Jham* is thrown into the well, its jaws strike the bottom of the well, dislodging some of the soil material. As the *Jham* is pulled up, the soil cuttings get retained but the water oozes out. The sinking is continued till the mota layer is reached. A smaller diameter bore hole is then made through the mota layer in the centre of the well, which is generally protected by a timber lining.

Sometimes, when mota layer is not available, shallow wells may be sunk as described above upto a required depth, and partly filled with gravel or broken

ballast. This will function as a filter through which water will percolate and enter the well but the sand particles will be prevented from rising up.

In a pucca well, lined with an impervious lining on its sides, the flow is not radial. The water enters only from the bottom and the flow becomes spherical when once the cavity has been formed at the bottom.

Type II. ***Wells with pervious lining.*** In this type of wells, dry brick or stone lining is used on the sides of the well. No mortar or binding material is used. The water, thus enters from the sides, through the pores in the lining. The flow is, therefore, radial. Such wells are generally plugged at the bottom by means of concrete. If the bottom is not plugged, the flow pattern will be a combination of radial flow and a spherical flow. Such wells are generally suitable in strata as of gravel or coarse sand. The pervious lining may have to be surrounded by gravel, etc., when such a well is constructed in finer soils, so as to prevent the entry of sand into the well along with the seeping water.

Type III. ***Kachha wells.*** These are temporary wells of very-shallow depths, and are generally constructed by cultivators to obtain irrigation supplies in their fields. Such wells can be constructed in hard soils, where the well walls can stand vertically without any support. They can, therefore, by constructed only where the watertable is very near to the ground. Though they are very cheap and useful, yet they collapse after some time, and may sometimes prove to be dangerous.

4.13.3. Yield of an Open Well. The yield of an open well can be determined with the help of theoretical method, or with practical methods, or by carrying out practical tests and then calculating it from the observations. The third method is useful for calculating the yields of open wells, as well as those of tubewells penetrating through the confined aquifers.

(1) ***Theoretical method.*** If a well is penetrated through an aquifer, water will rush into it with a velocity v. If A_s is the surface area of the aquifer opening into the well, then

$$Q = v \, . \, A_s$$

where $v = \eta . v_a$, where v_a is the actual flow velocity of the ground water, and v is the velocity with which water rushes into the well, and n is the porosity.

$$\therefore \qquad Q = n \, v_a \, . \, A_s \qquad \ldots(4.20)$$

In the above equation, the velocity of ground water flow (v_a) can be found by using Slichter's or Hazen's formula, or by conducting actual measurements by chemical or electrical methods, as discussed in article 4.4.2 and 4.4.3.

A_s, the area of the aquifer opening into the well, can be found by knowing the diameter of the well (d) and the depth (H) of the porous strata as $\pi \, . \, d \, . \, H$.

n the porosity, can be found by studying the sample of the soil in the laboratory.

Knowing n, v_a and A, the discharge (Q) can be easily calculated.

(2) ***Field Methods or Yield Tests.*** There are two yield tests *i.e.*, (*a*) *Pumping test* ; and (*b*) *Recuperating test.* They are described below :

(*a*) **Pumping test.** A pump is, first of all, installed, so as to draw sufficient supplies of water from the open well, and to cause heavy drawdown in its water level. The rate of pumping is then changed and so adjusted that the

water level in the well becomes constant. In this condition of equilibrium, the rate of pumping will be equal to the rate of yield, and hence, the rate of pumping will directly give us the yield of the well at a particular drawdown. Knowing this yield say Q_1 at a certain known drawdown say s_1, the yield (Q) at any other drawdown (s) can be evaluated as follows :

By Darcy's law

$$Q = K.i.A$$

$$= K\left(\frac{s}{L}\right)A$$

$$= \frac{K}{L}.A.s$$

or $$Q = C.A.s. \qquad ...(4.21)$$

where s is the depression head or drawdown in the well.

c = a constant

If Q_1 is the known discharge at a certain known drawdown s_1, we have

$$Q_1 = C.A.s_1 \qquad ...(i)$$

Q_1 and s_1 are known and A is the area of cross-section of the well. In case a cavity is formed, the area A is taken as $\frac{4}{3}$ times the actual cross-sectional area of the bottom of the well. Knowing Q_1, A and s_1 in Eq. (i) above, value of C can be calculated. Hence, the discharge Q at any other value of depression head (s) can be easily worked out.

From the above equation, it is also evident that the velocity increases with the depression head (s). The value of s, for which the velocity becomes equal to the critical value, is called the ***critical depression head.*** Generally, the depression head is kept equal to $\frac{1}{3}$ times the critical depression head, and this head is known as the ***working head.***

Hence, the maximum yield or critical yield corresponds to critical depression head, and maximum safe yield corresponds to working head.

(*b*) **Recuperating test.** Although the pumping test gives accurate value of safe yield, it sometimes becomes very difficult to adjust the rate of pumping, so as to keep the well water level constant. In such circumstances, recuperating test is adopted.

In this method, the water is first of all drained from the well at a fast rate so as to cause sufficient drawdown. The pumping is then stopped. The water level in the well will start rising. The time taken by the water to come back to its normal level or some other measured level is then noted. The discharge can then be worked out as below :

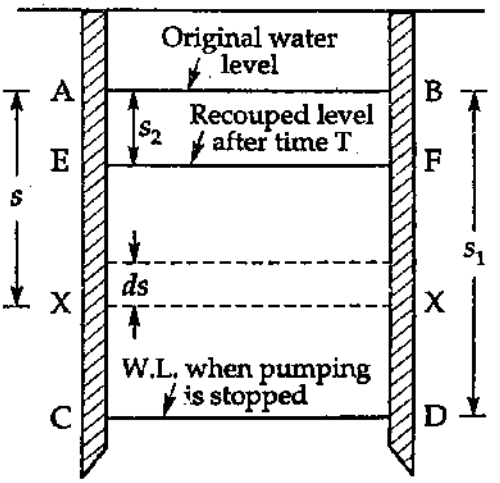


Fig. 4.19

With reference to Fig. 4.19.

Let AB = Static water level in the well before the pumping was started

CD = Water level in the well when the pumping was stopped

s_1 = Depression head in the well at the time the pumping was stopped

EF = Water level in the well at the noted time (say after a time T from when the pumping is stopped)

s_2 = Depression head in the well at time T after the pumping is stopped.

Let X-X be the position of water level at any time t after the pumping was stopped, and let the corresponding depression head be s. Let ds be the decrease in depression head in a time dt after the time t. Hence, in a time t after the pumping is stopped, the water level recuperates by $(s_1 - s)$. It again recuperates by ds in a time dt after this.

$\therefore$ Volume of water entering the well in the small interval of time (dt)

$$= dV = A.ds \qquad ...(1)$$

where A is the cross-sectional area of the well at the bottom.

Also, if Q is the rate of recharge into the well at the time t under a depression head s, then the volume of water entering the well in this small time interval is

$$= dV = Q.dt$$

But $Q \propto s$

$\therefore$ $Q = C'.s$ ...(2)

when C' is a constant depending on the soil through which the water enters the well.

$\therefore$ $dV = C'.s.dt$ ...(3)

Equating (1) and (3), we get

$$-A.ds = C'.s.dt$$

(The – ve sign indicates that s decreases as t increases)

or $$\frac{C'.dt}{A} = -\left(\frac{ds}{s}\right)$$

Integrating between the limits

$t = 0, \quad s = s_1$

$t = T, \quad s = s_2$

we get, $$\frac{C'}{A}\int_0^T dt = -\int_{s_1}^{s_2} \frac{ds}{s}$$

or $$\frac{C'}{A}\left| t \right|_0^T = -\left| \log_e s \right|_{s_1}^{s_2}$$

or $$\frac{C'}{A}(T) = -\log_e \frac{s_2}{s_1}$$

$$= -2.3 \log_{10} \frac{s_2}{s_1}$$

$$= 2.3 \log_{10} \frac{s_1}{s_2}$$

$$\therefore \quad \frac{C'}{A} = \frac{2.3}{T} \log_{10} \frac{s_1}{s_2} \qquad ...(4.22)$$

Knowing the values of s_1, s_2 and T from the above test, the value of $\frac{C'}{A}$ can be calculated. $\frac{C'}{A}$ is called the ***specific yield*** or the ***specific capacity*** of the open well in cumecs per sq-m of area under a unit depression head. Knowing the value of $\frac{C'}{A}$, the discharge Q for a well under a constant depression head H can be calculated as follows :

$$Q = C'.s$$

or $$Q = \left(\frac{C'}{A}\right) A.s \qquad ...(4.23)$$

or $$Q = \left(\frac{2.3}{T} \log_{10} \frac{s_1}{s_2}\right) A.s \qquad ...(4.24)$$

A and s are known, the discharge for any amount of drawdown (s) can be easily worked out.

In the absence of recuperation test, the following rough values of $\frac{C'}{A}$, as given by Marriot, can be used.

Table 4.5

Type of soil	$\frac{C'}{A}$ *(i.e. specific capacity or specific yield) in cubic metres per hour per sq. metre of area under unit drawdown*
Clay	0.25
Fine sand	0.50
Coarse sand	1.00

(3) The third method, which is useful for determining the yields of open wells (*i.e.* for unconfined aquifers) as well as those of tube wells (*i.e.* for confined aquifers*) is based on Dupuit's, Thiem's, and Thies' formulas, and are described after the description of the tube-wells.

Example 4.5. *Design on open well in coarse sand for a yield of 0.004 cumec when operated under a depression head of 3 metres.*

Solution.

The discharge required from the well

$$= 0.004 \text{ cumec}$$

*Generally, the tubewells are constructed to tap the lower confined aquifers ; although they may sometimes tap the upper unconfined aquifers also.

$= 0.004$ cubic metre per second

$= 0.004 \times 60 \times 60 \times$ cubic metres per hour

$= 14.4$ m^3/hr.

From Table 4.5, the value of $\frac{C'}{A}$ for coarse sand may be taken as 1.00 m^3/hr/m^2/m of depression head.

Also, Depression head = $s = 3$ m.

Now form Eq. (4.23)

$$Q = \left(\frac{C'}{A}\right) A.s$$

$$\therefore \quad 14.4 = 1.00 \times A \times 3$$

or $$A = \frac{14.4}{3} = 4.8 \text{ m}^2.$$

If d_w is the diameter of the well, then

$$\frac{\pi d_w^2}{4} = 4.8$$

or $$d_w = \sqrt{\frac{4.8 \times 4}{\pi}} = 2.48 \text{ m ; } \textbf{Say 2.5 m} \quad \textbf{Ans.}$$

Example 4.6. *During a recuperation test, the water level in an open well was depressed by pumping by 2.5 metres and is recuperated by an amount of 1.6 metres in 70 minutes.*

(a) Determine the yield from a well of 3 m diameter under a depression head of 3.5 metres.

(b) Also determine the diameter of the well to yield 10 litres / second under a depression head of 2.5 metres.

Solution.

From Equation (4.22)

$$\frac{C'}{A} = \frac{2.3}{T} \log_{10} \frac{s_1}{s_2}$$

where s_1 = Initial drawdown = 2.5 m

s_2 = Final drawdown = 2.5 – 1.6 = 0.9 m

T = Time = 70 minutes = $\frac{70}{60}$ hr = 1.167 hr

$$\frac{C'}{A} = \frac{2.3}{1.167} \log_{10} \frac{2.5}{0.9} = 0.875 \text{ m}^3/\text{hr/m}^2/\text{m of depression head}$$

(a) Yield from a well of 3 m diameter, under a depression head of 3.5 m is given by Equation (4.23) as

$$Q = \left(\frac{C'}{A}\right) A.s$$

$$= 0.875 \times \left(\frac{\pi}{4} \times 3^2\right) \times 3.5 = 21.65 \text{ m}^3/\text{hr} = 6.02 \text{ litres/sec.} \quad \textbf{Ans.}$$

(*b*) If $Q = 10$ litres/sec,

$$= \frac{10 \times 60 \times 60}{1000} \text{ m}^3/\text{hr} = 36 \text{ m}^3/\text{hr}$$

$s = 2.5$ m

$$Q = \left(\frac{C'}{A}\right).A.s$$

$$\therefore \quad 36 = (0.875) \times A \times 2.5$$

or $$A = \frac{36}{0.875 \times 2.5} = 16.46 \text{ m}^2$$

or $$\frac{\pi}{4} d^2 = 16.46$$

or $$d = 4.58 \text{ m ; Say } 4.6 \text{ m}$$

Hence, the diameter of the required well = 4.6 m. **Ans.**

4.14. Tubewells

The discharge from an open well is generally limited to 3 to 6 l/s. Mechanical pumping of small discharges available in open wells is therefore not economical. To obtain large discharge mechanically, *tubewell, which is a long pipe or a tube, is bored or drilled deep into the ground, intercepting one or more water bearing strata.* The discharge of an open well is smaller, because : (*i*) open wells can tap only the topmost or at the most the next lower water bearing stratum ; (*ii*) water from open wells can be withdrawn only at velocity equal to or smaller than the critical velocity for the soil, so as to avoid the danger of the well subsidence. But in the tubewells, larger discharge can be obtained by getting a larger velocity as well as a larger cross-sectional area of the water bearing strata. Since, we have a large storage of ground water in India, the tubewells provide excellent method of obtaining water supplies, although they are generally used for irrigation.

4.14.1. Tubewells in Alluvial Soils. Most of our land, especially the entire area from Himalayas to Vindhya mountains (such as the Indo-Gangetic plain), Coastal areas, Narmada valley, etc., consist of deep alluvial soils. The subsoil water slowly penetrates and is stored in the porous sand and gravel beds which are extensively found in India, except that in the desert areas. Tubewells can be easily installed in such soils and are very useful for irrigation. It is in this context that the tubewells are assuming greater and greater importance for tapping our ground water resources, especially in alluviums.

Deep Tubewells are as deep as 70 to 300 m, and tap more than one aquifer. They are usually constructed in our country by the State Governments, and are called the State tubewells. Such wells may yield as high as 200 to 220 litres/sec. The general average yield from the standard tubewell is however of the order of 40 to 45 litres/sec. A 300 m deep tubewell has been constructed at Allahabad (UP) at the edge of the river Ganga and is yielding at about 140 litres/sec. The diameter of this hole is 0.6 m upto 60 m depth, and then 0.56 m below 60 m. The diameter of the strainer is 0.25 m and draw-down

is 10 m. Such deep tubewells are drilled by *heavy duty rotary drilling rigs* (direct rotary as well as reverse rotary). *Percussion drilling rigs* may also be used in hard boulder areas. About 48,000 deep tubewells are existing in our country, and every year about 1000 such wells are being added. Most of our deep tubewells have been constructed by using mild steel slotted pipe screens with gravel packing.

Besides deep tubewells, *shallow tubewells*, having 20 to 70 m depth and tapping only one aquifer, are also constructed, usually by private individual farmers. Such wells may yield as high as 15-20 litres/sec, if located at proper places. Each well irrigates about 8 hectares. Such shallow tubewells are usually drilled by *light rigs, cable tool drills, water jet methods,* and *hand boring devices*. About 48 lakh shallow tubewells are existing in our country, and every year about 2.5 lakh new wells are added.

4.14.2. Tubewells in Hard Rocky Soils. It is very difficult to construct tubewell irrigation in rocky areas. Therefore, in rocky areas, tubewells are constructed only when no alternative water supply sources are available. Hence, in rocky areas, only isolated holes of 10 to 15 cm diameter are drilled using *Down the hole hammer rigs (DTH rigs)*. These wells are usually in the depth range of 100 m, although tubewells upto 300 m depths have been successfully bored. Such tubewells are called **bored wells**, because the bore hole is able to withstand on its own in the bottom portion, and a pipe is pushed only in the upper weathered zone. These wells usually depend on joints and fissures in the rocks for their water supply. Even with a heavy drawdown of 20 to 30 m, such wells are generally not able to yield more than 5-10 litres/sec, except when they tap some embedded aquifer. Such tubewells have mostly been constructed in Southern States of our country.

4.14.3. Types of Tubewells. Depending upon the entry of water through a *cavity* or a *screen*, the tubewells can be broadly classified into two categories, *viz.* :

1. *Cavity type tubewells* ; and
2. *Screen type tubewells.*

Both these types of tubewells are discussed below :

(1) Cavity Type Tubewells. A cavity type tubewell draws water from the bottom of the well, and not from the sides as is done by a screen well. The flow in a cavity well, therefore, is essentially spherical, and not radial like that of a screen well. Moreover, since in a gravity well, the water is drawn from the bottom of the well, such a well can tap only one water bearing stratum. Such a tubewell, however, is very economical, as it requires only plain well pipe, which is lowered into the bore, made through the ground strata upto the required depth, so as to tap the requisite aquifer.

The principle behind the working of a cavity-type tube-well is essentially similar to that of a deep open well, with the only difference that whereas a deep open well taps the first aquifer just below the mota layer, a cavity tubewell need not do so, and may even tap the lower or still lower stratum, as shown in Fig. 4.20.

A cavity type tubewell essentially consists of a pipe, bored through the soil and resting on the bottom of a strong clay layer. A cavity is formed at the bottom, and the water from the aquifer enters the well pipe through this cavity, as shown in Fig. 4.20. In the initial stages of pumping, fine sand comes out with water and consequently a hollow or a cavity is formed. As the spherical area of the cavity increases outwards, the radial critical velocity decreases for the same discharge, thus reducing the flow velocity and consequently stopping the entry of sand. Hence, the flow in the beginning is sandy but becomes clean with the passage of time.

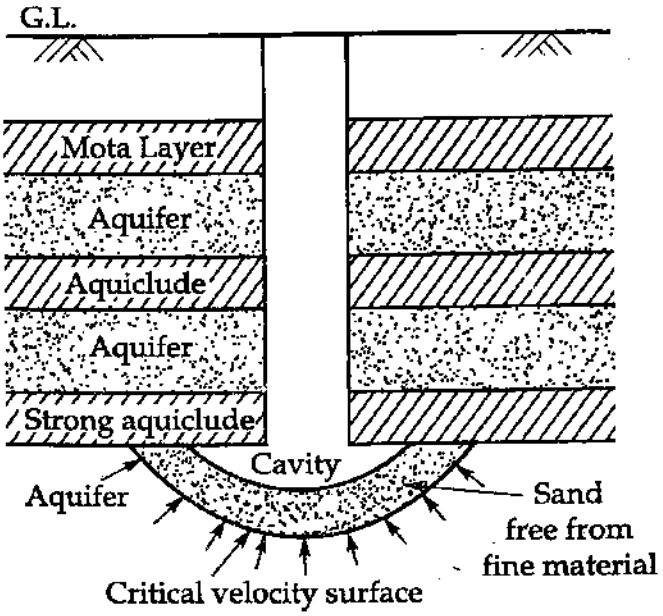


Fig. 4.20. Cavity type tubewell.

The cavity of such a tubewell, however needs to be **developed** carefully and slowly by using a *centrifugal pump* rather than a *compressor* or a *turbine pump*.

To begin with, the water is pumped from the well at a low discharge rate. When the discharging water becomes clear, the drawdown may be increased slightly, which may result in further sand being drawn out. The process is repeated till the normal drawdown and clear discharge is obtained. The pumping is then stopped for an hour or so, and then resumed again. The discharge after restarting may again contain sand. The pumping is continued till the water is clear again. The procedure may be repeated till the well is fully developed, which is shown by the sand free discharge even on resumption of pumping after a closing interval.

Cavity type tubewells can however be used only for small supplies, particularly for domestic water supplies.

(2) Screen Type Tubewells. Screen type tubewells are most widely adopted and have been extensively constructed in our country particularly for irrigation purposes. So much so that whenever we refer to a tubewell, we generally mean a "screen well". All the State tubewells constructed in U.P., from where the technique of tubewell construction started in 1931, are exclusively of this type. Such a well can easily tap a number of aquifers, and hence does not depend only on one aquifer like a cavity well.

Screen type tubewells can be further subdivided into the following two types :

(*i*) *Strainer tubewells* ; and

(*ii*) *Slotted pipe gravel-pack tubwells.*

Both these types of screen wells are discussed below :

(*i*) A **strainer tubewell** uses assembled strainer lengths, which are lowered into the bore hole and located opposite the water bearing formations,

whereas plain pipe lengths (*blind pipes*) are located opposite the non-water bearing formations, as shown in Fig. 4.21. A bail plug is provided at the bottom. Water enters into the well through these strainers from the sides, and the flow in the well is thus *radial*.

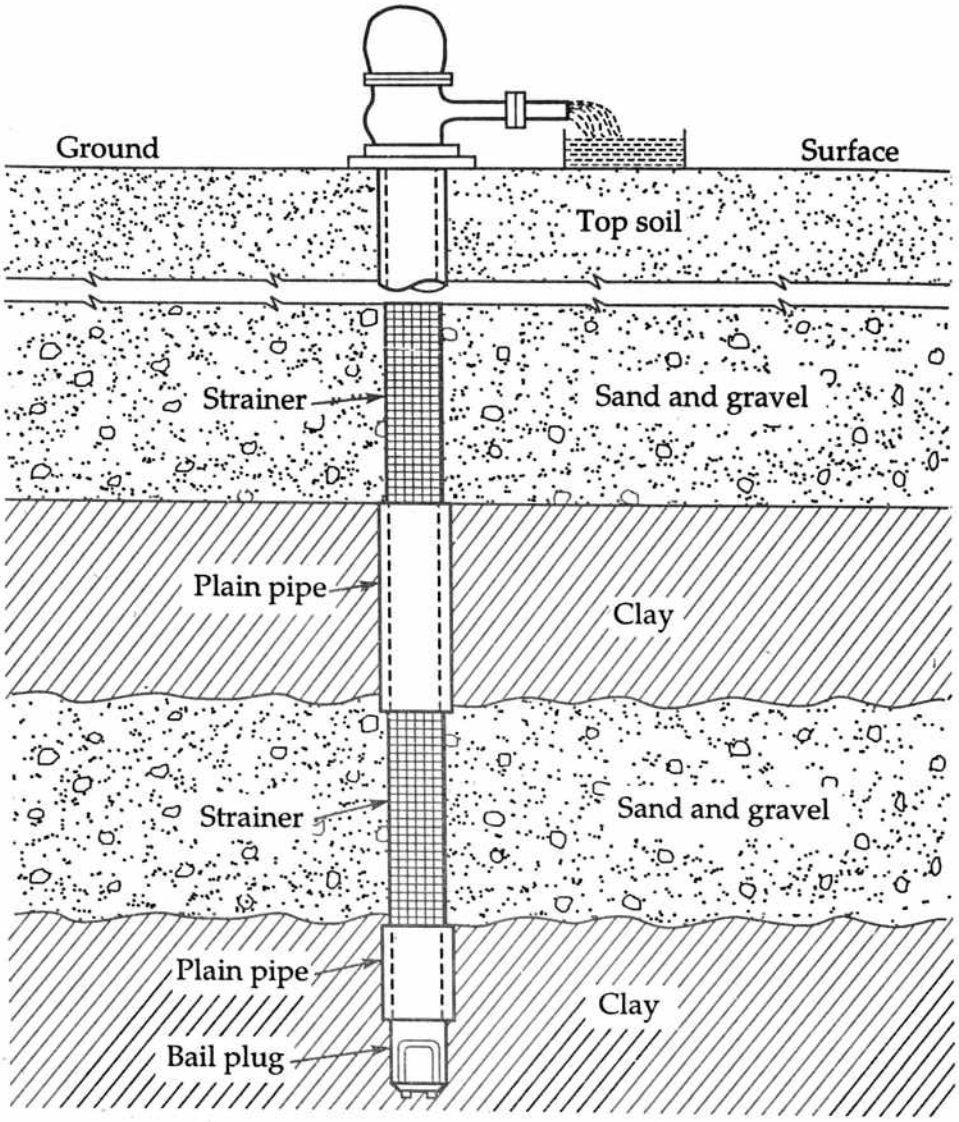


Fig. 4.21. A strainer tubewell.

A *strainer* (Fig. 4.22) essentially consists of a *perforated or a slotted pipe with a wire mesh wrapped round the pipe with a small annular space between the two*. The wire screen prevents the sand particles from entering the well. The water, therefore, enters the well pipe through the fine mesh, and the sand particles of size larger than the size of the mesh are prevented from entering the well. This reduces the danger of sand removal, and hence larger flow velocities can be permitted.

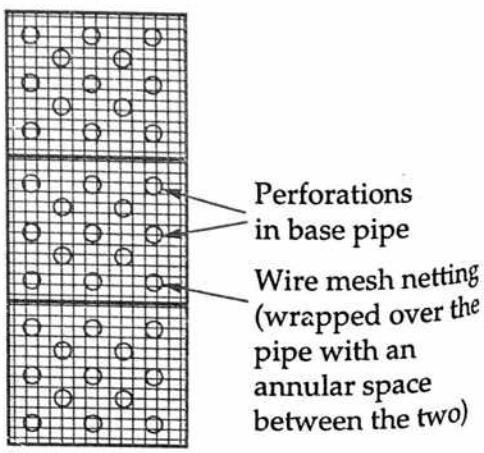


Fig. 4.22. A typical strainer.

The perforated pipe is made to have the cross-sectional area of its openings equal to that in the wire mesh, so that no change of velocity occurs between the two. The annular space between the pipe and the wire mesh is required, as otherwise the wires of the mesh will cover a large part of the area of the pipe openings.

A strainer type tubewell is generally unsuitable for fine sandy strata, because in that case, the size of the mesh openings will have to be considerably reduced, which may result in choking of the strainer ; and if the mesh openings are kept bigger, the well will start discharging sand.

(ii) **A gravel pack slotted pipe tubewell,** on the other hand, uses *a slotted pipe without being covered by any wire mesh.* S h slotted pipe lengths are located opposite the water bearing formation as is done with the strainers in a strainer tubewell. These slotted pipes should strictly be referred

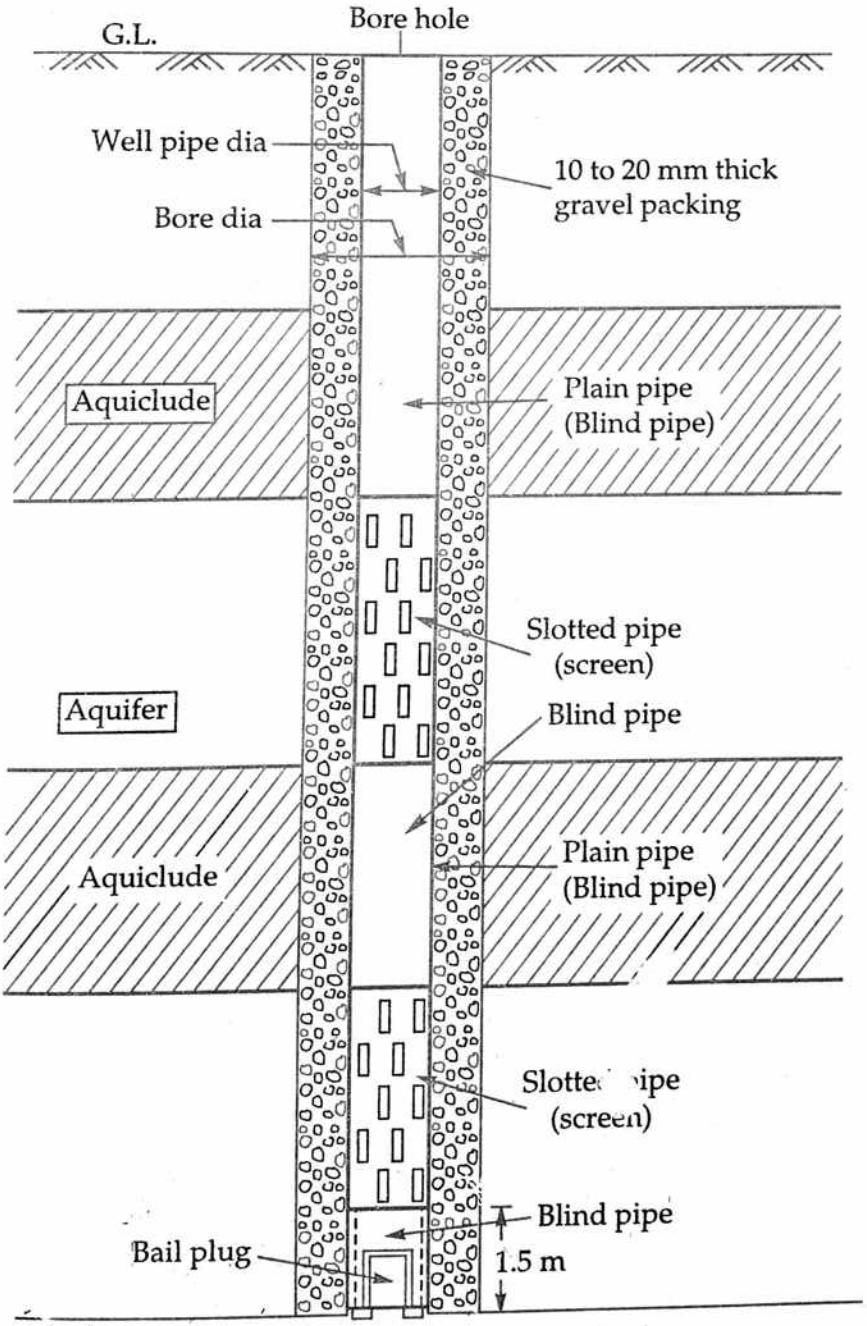


Fig. 4.23. Slotted pipe gravel pack tubewell.

to as *"screen pipes"*, although, however, they are often called as *"slotted pipe strainers"* or *"pipe strainers"*, thereby leaving little scope to differentiate between such slotted pipes (*i.e. screen pipes*) and the perforated pipes with mesh coverings (*i.e. strainers*).

After placing the assembly of the plain and slotted pipes in the bore hole, a mixture of gravel and bajri (called ***gravel shrouding***) is poured into the bore hole between the well pipe assembly and the casing pipe, so as to surround the well pipes by a designed 10-20 mm thickness of gravel pack. The gravel packing is specifically required around the screen pipes, but since the material is poured from the ground level into the bore hole, the pack shall be installed in the entire depth of the well below the top level of the shallowest screen. Usually, the gravel pack is extended even above the top level of the shallowest screen, and up to the ground level, so as to obtain a stable and an efficient tubewell, as shown in Fig. 4.23.

Gravel pack wells are generally provided in *fine aquifers*, where the effective grain size (D_{10}) may be less than 0.25 mm, and uniformity coefficient (C_u) may be 2 or less. *Gravel packing also enables the use of single size slots in the screen pipes, irrespective of the fact that fine as well as coarser aquifers are tapped simultaneously and in alternating layers.* Gravel pack is therefore highly preferred for *deep tubewells*, which tap more than one aquifer. Due to this reason, most of the State deep tubewells have been constructed in our country in fine alluvial soils by providing *cast in situ* gravel packs around the slotted mild steel pipes.

Precast gravel packs have also come into the market, where the slotted pipes are mounted with *prepack gravel filters*, popularly called *Ashim filters*. In these filters, graded hard gravel grains are coated and bonded together with water proof chemicals over the outer surface of the slotted pipe, providing a highly permeable screen. Such prepack gravel filters are found suitable for shallow and medium deep tubewells in aquifers composed of find sand, coarse sand and gravel. Prepacking is claimed to be providing economy in the quantity of gravel used and protects against dislodging of the gravel envelope. However, such filters are so far not being used in our country on a large scale because of their high costs.

Abyssinian tubewell is a special type of a strainer tubewell in which the diameter of the well pipe is kept equal to 3.8 cm ($1\frac{1}{2}''$) and the strainer is provided only for a length of about 1 to 1.7 m.

4.14.4. Design of a Well Screen. A well screen constitute the most important part of a tubewell, since it serves as the *intake structure* for the entry of water into the well. The design of the screen is largely influenced by the characteristics of the water bearing formations. *Dry sieve analysis* of the aquifer sample obtained during drilling of the bore hole, is therefore carried out to plot the *grain size distribution curve* for the aquifer. The results will help us to determine the design specifications of the well screen as well as to design the gravel pack for the slotted pipe screen.

From the grain size distribution curve of the aquifer, the following characteristics of the aquifer are determined :

(*i*) effective size (D_{10}) ;

(*ii*) uniformity coefficient (C_u) ; and

(*iii*) mean particle size (D_{50}) of the aquifer.

The ***effective size*** (D_{10}) is that sieve size through which 10% of the particles shall pass, and 90% are retained. 90% of the particles in the aquifer are thus coarser than this size, and hence retained on a sieve of this size.

The ***uniformity coefficient*** (C_u) is defined as the ratio of sieve size passing 60% of the aquifer material, to the sieve size passing 10% of the aquifer material. In other words,

$$C_u = \frac{D_{60}\ (40\%\ \text{retained})}{D_{10}\ (90\%\ \text{retained})} \qquad ...(4.25)$$

This ratio was proposed by Hazen (1893) to be a quantitative expression of the degree of the assortment of water bearing sand, as an indicator of *porosity*. The value of C_u for complete assortment (one grain size) is 1 ; while for fairly even grained sand, it ranges between 2 to 3. For heterogeneous sand, this value will be higher. *Generally, a material is classified as uniform, if the uniformity coefficient* $C_u \leq 2.0$.

Designing the Size of the Slot Openings of a Screen. Fixing an optimum size for the slot openings of a screen is an important component of the screen design, and it depends upon the size of the aquifer material. *Oversized slots will pump finer material indefinitely, and it will be difficult to obtain clear water. Undersized slots will provide more resistance to the flow of ground water into the well, resulting in more head loss and corrosion.* Fine slots are also blocked by small sand and silt particles. The problem of clogging is reduced as the size of openings is increased. *Therefore, theoretically, the slot openings should be as wide as possible.* The optimum value is determined by matching the size of the opening with the grain size distribution of the material surrounding the screen. *In practice, the slot size varies from values as low as 0.2 mm to as large as 5 mm.* In India, however, two sizes *i.e.* 1.6 mm and 3.2 mm are generally available in the market.

The criteria for selecting the size of the slot openings for non-gravel pack and gravel pack well is given below :

(*a*) **Slot openings for non-gravel pack wells.** The optimum slot opening size is chosen as the one which retains 40% of the sand (*i.e.* equal to D_{60}) size of the aquifer if the ground water is non- corrosive ; and equal to the D_{50} size if the ground water is corrosive.

The above selected slot size may therefore vary for different aquifers, when more than one aquifer is to be tapped, thereby indicating the use of different sizes for slots to be made in the screen lengths to be rested against different aquifers. Practically, this is a difficult proposition, and hence uniform widths of slots designed on the basis of the finest aquifer material are often provided.

The above adoption of D_{60} size for the slots indicates that 60% of the formation material shall be pumped out during the development of the well, which will result in the removal of all the fine particles from around the well screen.

(*b*) **Slot openings for gravel pack wells.** The slot size for well screens for gravel pack wells is determined on the basis of the grain size distribution curve of the gravel material used for the gravel pack. On this curve, a point is located indicating the 90% size of gravel to be retained (*i.e.* D_{10} size for the gravel material), which indicates the optimum slot size for the well screen. ***The actual size of the slots may thus be fixed at ± 8% of the D_{10} size of the gravel pack.***

Gravel pack design. The gravel pack should however also be designed before designing the size of the slots to be made in the well screens. The gravel pack is usually designed on the basis of *Pack Aquifer ratio (PA ratio)*, which is usually defined as the ratio of D_{50} size of the gravel pack material to the D_{50} size of the aquifer material.

$$\therefore \quad \textit{Pack Aquifer (PA) ratio} = \frac{D_{50} \text{ of the gravel}}{D_{50} \text{ of the aquifer}} \qquad ...(4.26)$$

Several recommendations have been made by different investigators for optimum values of PA ratio. The Central Board of Irrigation and Power (1967) has for Indian conditions, suggested the following criteria for PA ratio.

(*i*) PA ratio should lie between 9 and 12.5
for uniform aquifers having $C_u \leq 2.0$ ...(4.27)

(*ii*) PA ratio should lie between 12 and 15.5
for graded aquifers having $C_u > 2.0$ ...(4.28)

The uniformity coefficient of the gravel material is also preferably kept to be ≤ 2.5, because its higher value shall cause segregation of the gravel shrouding during pouring, which will result in poor efficiency of operation of the well. Such gravel materials, having $C_u \leq 2.5$ are difficult to obtain, and hence grading and preparing the requisite gravel material requires a lot of field control. A numerical example on the design of the gravel pack has been solved in the next article to make its design very clear.

The *thickness of the gravel pack* should normally be fixed from practical considerations at about 7.5 cm, and in no case should exceed about 20 cm. A thicker envelope does not materially increase the well yield, nor will it reduce the possibility of sand pumping, because the controlling factor is the ratio of the grain size of the pack material and that of the aquifer. Too thick a gravel pack, instead of giving any advantages, may make the final development of the well more difficult.

Example 4.6. *The results of sieve analysis test carried out on a 500 gm sample of underground aquifer proposed to be tapped for installation of a tubewell, are given in the table below :*

Size of the sieve in mm	*Mass of material retained in gm*
> 2.54	*0.0*
1.80	*6.0*
0.30	*15.0*
0.25	*320.0*

(Contd.)

0.21	*5.0*
0.16	*50.0*
0.12	*34.0*
< 0.12	*70.0*
	500.0

Design the size of the gravel pack, and the slot size for the slotted mild steel screen pipes.

Solution. The given sieve analysis data is analysed in table 4.6. A particle size distribution curve, is then plotted between grain size in mm on log X-axis, and %age finer (p) on Y-axis, as shown in Fig. 4.24.

Table 4.6

Size of the sieve in mm (1)	*Mass of the material retained in gm* (2)	*%age of material retained* $\frac{col.(2)}{500} \times 100$ (3)	*Cumulative %age retained* (4)	*%age finer (p) 100–col. (4)* (5)
> 2.54	0	0.0	0	100
1.80	6	1.2	1.2	98.8
0.30	15	3.0	4.2	95.8
0.25	320	64.0	68.2	31.8
0.21	5	1.0	69.2	30.8
0.16	50	10.0	79.2	20.8
0.12	34	6.8	86.0	14.0
< 0.12	70	14.0	100.0	0.0
Σ	500 gm	100%		

From the drawn curve, the following characteristics of the aquifer material are read out, as :

$$D_{60} = 0.27$$
$$D_{50} = 0.265$$
$$D_{10} = 0.102$$

$$\therefore \quad C_u = \frac{D_{60}}{D_{10}} = \frac{0.27}{0.102} = \mathbf{2.64}$$

Since $C_u \geq 2.0$, we should use P.A. ratio for designing the gravel pack lying between 12 and 15.5 (Refer eq. 4.28).

$$\text{Hence, P.A. ratio} = \frac{D_{50} \text{ of gravel pack}}{D_{50} \text{ of aquifer}} = 12, \text{ and } 15.5$$

Using PA ratio as 12, we have

$$12 = \frac{D_{50} \text{ of gravel pack}}{0.265}$$

$$\therefore \quad D_{50} \text{ of gravel pack} = 12 \times 0.265 = \mathbf{3.18 \text{ mm}} \qquad ...(i)$$

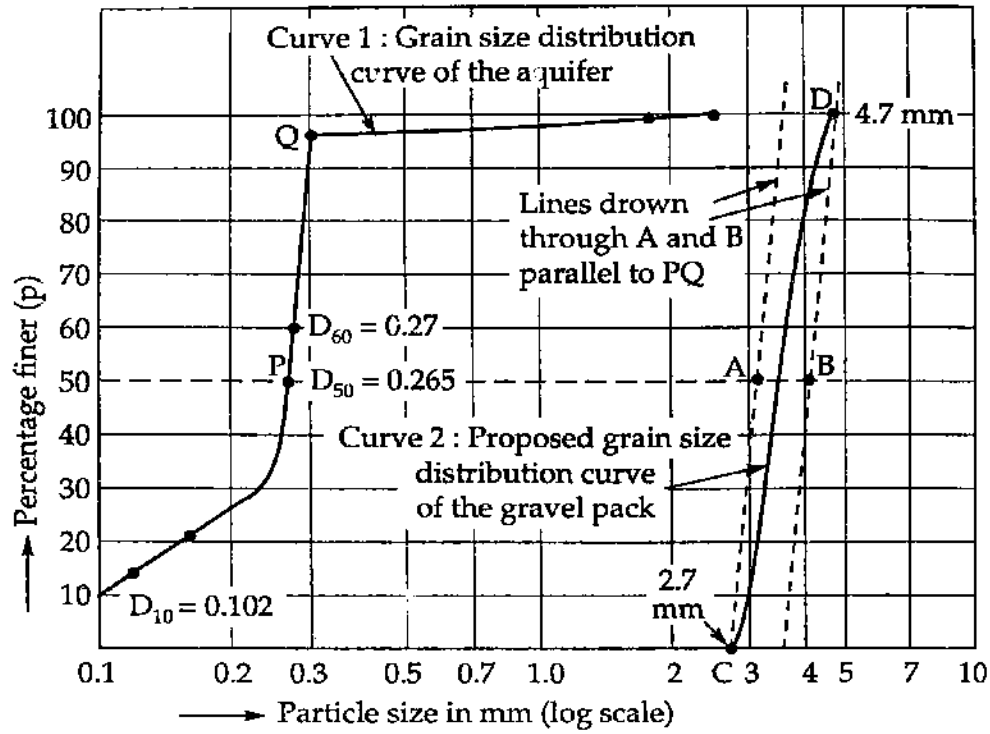


Fig. 4.24. Grain size distribution curves for the aquifer and gravel pack.

Using PA ratio as 15.5, we have

$$15.5 = \frac{D_{50} \text{ of gravel pack}}{0.265}$$

$\therefore$ D_{50} of gravel pack = $15.5 \times 0.265 = \mathbf{4.11\ mm}$ ...(*ii*)

In other words, D_{50} of the gravel pack should lie between these two limiting values ; *i.e.* 3.18 mm and 4.11 mm. These values are marked on p = 50% horizontal line, as A and B. Dotted lines are now drawn through these two points, parallel to the central portion of the grain size curve of the aquifer material (PQ). These two enveloping curves are the limiting curves for the grain size distribution curve of the gravel pack. The minimum size of the gravel between these two curves is represented by point C as 2.7 mm ; and the maximum size is represented by point D as 4.7 mm. *Hence, the gravel pack size should vary between 2.7 mm and 4.7 mm.* The gravel should therefore, be screened, such that the gravel size ranges between 2.7 mm and 4.7 mm. The proposed grain size curve for the gravel shall be somewhat as shown by firm line CD. D_{10} size of this curve is approximately equal to 3 mm, and hence the screen pipes having standard 3.2 mm wide slots, may be used.

C_u for the gravel is also computed as $\frac{D_{60}}{D_{10}} = \frac{3.6 \text{ mm}}{3 \text{ mm}} = 1.2$ (O.K.). This value should be ≤ 2.5 for good design and the drawn curve may be adjusted between two envelope curves to achieve such a satisfactory value of C_u.

Designing the Length and Size of the Screen. The total length of the screen to be provided for a tubewell shall be primarily controlled by the available thickness of the aquifers, since this length cannot exceed the aquifers thickness. It shall further be governed by the *area of the screen openings*, because to pass a given discharge, the screen length will be less if the area of its openings per m length is more, and *vice versa*.

The area of the screen openings per m length of the screen usually vary between 15 to 22% of the screen area, which equals $\pi \cdot d$, where d is the dia of the screen pipe. If the area of the openings is kept more than about 15 to 22%, the structural strength of the well screen reduces, which consequently reduces the life of the screen.

The diameter of the screen is selected to satisfy the essential basic requirement that sufficient open area be provided in the screen, so as to limit the entrance velocity to a safe permissible limit. The optimum safe entrance velocity for a given aquifer is related to the coefficient of permeability (K) value of the aquifer, as shown in table 4.7.

Table 4.7. Optimum Screen Entrance Velocity (v_e)

Coefficient of permeability (K) of the aquifer in cm/s	*Optimum entrance velocity (v_e) in cm/s*
0.02	1.5
0.05	2.0
0.09	3.0
0.14	4.0
0.18	4.5
0.24	5.0
0.28	5.5

For most of the available sandy aquifers, K value varies between 0.05 to 0.10, and hence v_e generally varies between 2 cm/s to 3 cm/s. When entrance velocity exceeds this safe optimum value, the frictional loss through the screen openings shall become higher, and there will be more incrustation and corrosion.

The entrance velocity is therefore calculated by dividing the design discharge per m length of screen by the total area of the openings in the screen per m length of screen. If the value works out to be more than the safe optimum value, the dia of the screen is increased, so as to increase the opening area to achieve the optimum value of the entrance velocity. The guiding values of the screen diameter for different discharges, as suggested by USBR (Ahrens, 1970), are given in table 4.8.

Table 4.8. Recommended values of screen diameter

Discharge in l/min	*Recommended screen diameter in mm*
0 to 475	10
475 to 1125	15
1125 to 3000	25
3000 to 5250	30
5250 to 9500	35
9500 to 13,300	40

The minimum length of the screen can finally be designed by using the following equation :

Minimum length of the screen

$$= h = \frac{Q}{A_0 \cdot v_e} \qquad ...(4.29)$$

where Q = Design discharge of tubewell

A_0 = Area of the screen openings per m length of the screen

= $\pi \cdot d \times$ percent of openings (as fraction)

v_e = optimum entrance velocity assumed for the K value of the aquifer for non-gravel wells. For gravel pack wells, the mean K value for the aquifer and for the gravel pack is considered for assuming the value of v_e from table 4.7.

The above calculated minimum strainer length should be adjusted within the available aquifer depths by screening the available depth (s) by about 75 to 90% for the confined aquifers. The percentage should be increased with the increase in thickness. Say for example, 75% screening is satisfactory for 8 m thick aquifers, and 90% for 20 m thick aquifers. Atleast 0.3 m aquifer depth at top as well as bottom of the aquifer should be left unscreened to safeguard against the error in the placement of screen during installation. The pumping water level should never fall below the top of the aquifer. The screen is usually located at the centre of the aquifer.

In unconfined aquifers, the bottom $\frac{1}{3}$ to $\frac{1}{2}$ depth of the aquifer is usually screened.

Example 4.7. *A fully penetrating well in a confined sandy aquifer has a maximum discharge capacity of 1200 l/min. The aquifer is overlain and underlain by impervious formations. The thickness of the aquifer is 20 m. Design the length of the well screen, assuming the percentage of open area in the available strainer to be 15%, and bore hole dia as 15 cm.*

Solution.

$$Q = 1200 \text{ l/min} = \frac{1.2}{60} \text{ m}^3\text{/s} = 0.02 \text{ m}^3\text{/s}$$

A_0 = Area of opening per m length of screen

$= (\pi \cdot d)\, p = \pi(0.15 \text{ m}) \times 15\%$

$= \pi \times 0.15 \times 0.15 \text{ m}^2\text{/m}$

$= \mathbf{0.071}$ m^2/m length of screen

v_e = safe entrance velocity

= may be assumed to be 2 cm/s, since K for sands is usually 0.04 cm/s

= 0.02 m/s

Using eqn. (4.29), we have

$$h = \text{Minimum length of the screen} = \frac{Q}{A_0 \cdot v_e}$$

$$= \frac{0.02}{0.071 \times 0.02} = \mathbf{14.2\ ;\ 15\ m}\ \text{(say)}$$

Since the aquifer thickness is 20 m, and the minimum reqd. screen length is 15 m, it would be prudent to use 18 m length of the screen, which shall be about 90% of the aquifer depth. The screen may be provided in the central 18 m depth of the aquifer, leaving one metre depth of aquifer unscreened at both ends.

4.14.5. Types of Well Screens

(1) Slotted pipe screens. As stated earlier, all the gravel pack wells normally utilise *slotted pipe screens*. The slot size is also decided somewhere near the D_{10} size of the designed gravel material. The slotted pipes to be used are generally made of mild steel. IS 8110—1976 provides details in respect of such screens. The standard slot size commonly adopted in India are 1.6 mm and 3.2 mm wide, and 10 cm long. The minimum spacing between slots is 3 mm. The slots are so arranged as to obtain an even distribution of flow over the entire periphery of the screen. They are distributed in groups of 3 or 4, and arranged so that the slots of one group are not in line with those of the adjacent row, so as to maintain adequate strength in the well pipe. A typical view of the arrangement of slots is shown in Fig. 4.25.

(*a*) General view

(*b*) Layout of slots as per IS 8110—1976

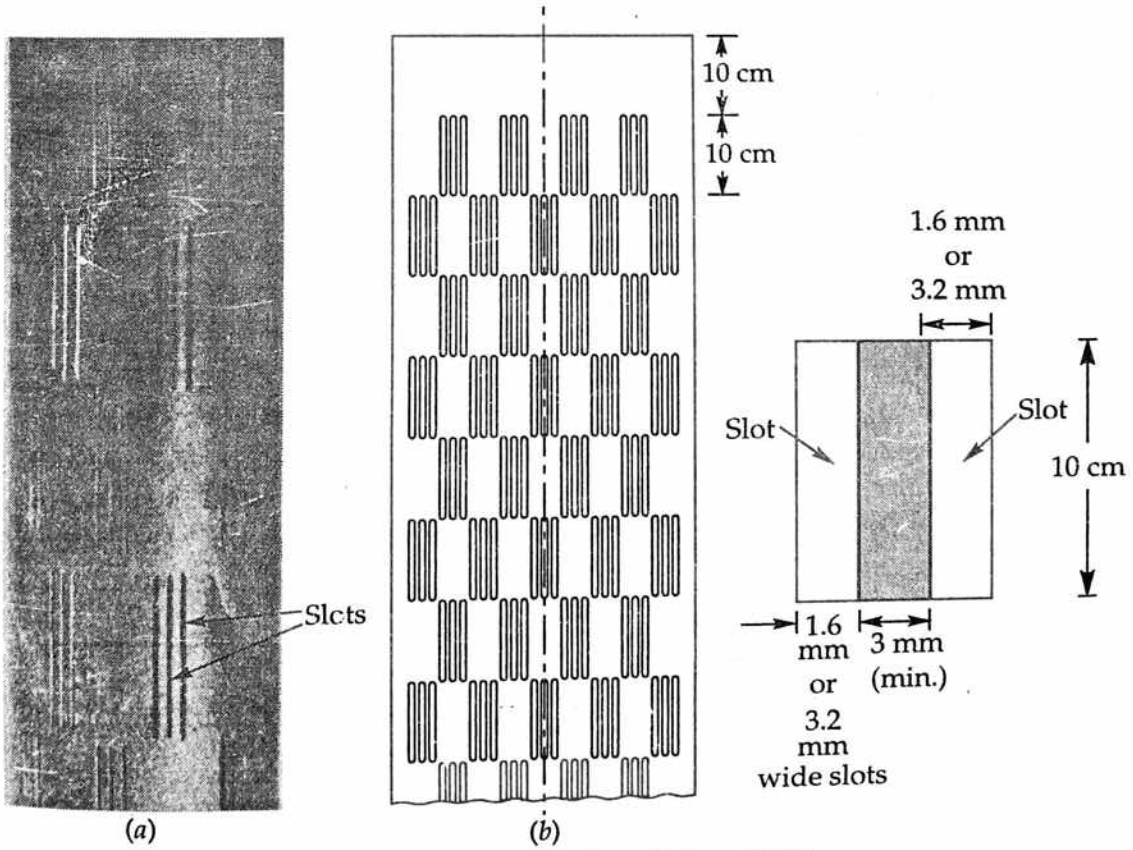


Fig. 4.25. Mild steel slotted pipe screen.

The slotted pipe is threaded at both ends. The bottom end of the slotted pipe is fitted with a blind pipe of 1.25 m length, with a cap called ***bail plug*** at the bottom. The bail plug has an *eye* fixed inside, which facilitates the extraction of the tubewell assembly in case of failure.

Slotted pipe screens with prepack filters, called **Ashim filters**, are also available in the market, and may be used for comparatively smaller and medium depth wells. A photographic view of such a screen is shown in Fig. 4.26. In such screens, mild steel slotted pipes are coated with 15-20 mm thick precast gravel layers, as to provide highly permeable screens. The mild steel pipe is usually slotted to have an open area of 25% with slots of 3.2 mm width. Different grades of filters are available to suit different sized aquifer particles. Such screens are, however, very costly, and hence adopted only for smaller individual wells.

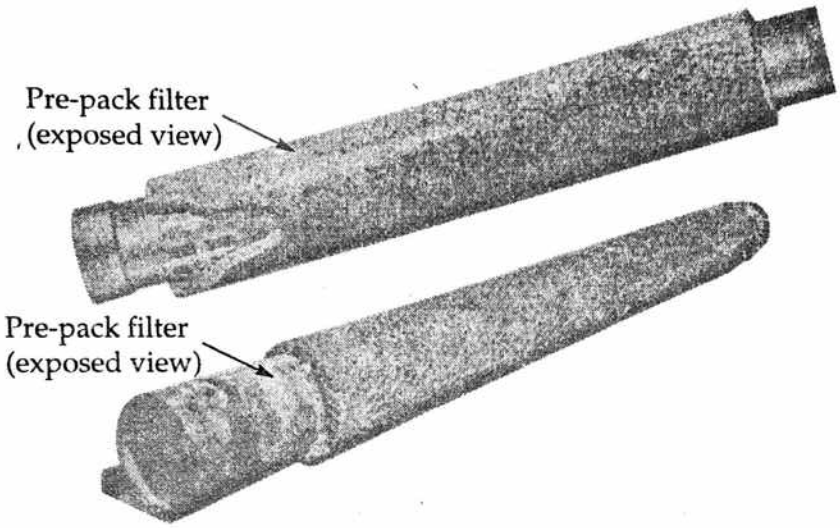


Fig. 4.26. Ashim slotted pipe prepack filter screens.

Brass screens and **stainless steel screens**, utilising brass or stainless steel pipes, with slots as in mild steel pipes, are also available in the market. These screens are quite costly but are more durable and less liable to corrosion, incrustation and consequent contamination of water due to rusting. Hence, such screens may be preferred for tubewells to be used for water supplies. Such screens are manufactured from 4 mm thick copper or stainless steel sheets, provided with designed openings (slots), which is rolled and welded to make the circular tube. The ends of the tube length (unslotted) are provided with outer threads for joining the tubes through a socket. Screens with different slot sizes and diameters are available in the market to suit the requirement of the design based on grain size distribution curve of the aquifer.

(2) **Strainer type of well screens.** Aquifers have traditionally been screened by utilising the strainer type of well screens. Such screens contain very small and narrow openings, to exclude the removal of those aquifer particles which are larger than the screen openings. Such screens usually have a double system of openings, consisting of slotted or perforated cylindrical pipes or shells, which are covered by fine wire mesh, wound round the pipe. The water has to initially enter the openings of the *outer wire jacket*, and then pass through the openings of the *pipe shell*. *Such strainer wells are*

usually provided without gravel packs, although it is not an absolute necessity, and even strainer wells can be gravel packed, like those using slotted pipe screens.

Several types of strainers, which are generally used in tubewell construction, are :

(*i*) *continuous slot type of strainers* ;

(*ii*) *louver or shutter type of strainers* ;

(*iii*) *pipe strainers with a cover of wire jacket, like in an agricultural strainer* ; and

(*iv*) *coir rope strainers.*

These types are briefly discussed below :

(*i*) **Continuous slot type of strainers** consists of cold drawn wire wound round a suitable cylindrical frame made of rods of iron, steel, brass, etc. At each point where the wire crosses the rods, the two are welded and jointed to produce a one piece rigid unit. In order to avoid clogging, V-shaped openings are usually provided by using triangular shaped wire in the outer jacket. Such screens can be made practically in any width from 1.5 mm onward.

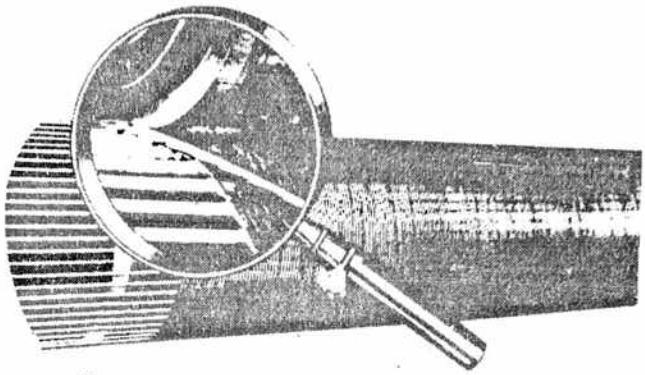


Fig. 4.27. Photoview of a continuous slot type of well strainer.

The well loss in such strainers is much lower than that in the slotted pipe well screen, but they are much costlier. A photographic view of such a strainer is shown in Fig. 4.27.

(*ii*) **Louver type of strainers** have openings, which are rows of louvers. The openings may be oriented either at right angles or parallel to the axis of the screen. The openings are produced in the wall of the welded tube by stamping process, using a die. A view of such a screen is shown in Fig. 4.28. Such a screen provides lesser choice of opening sizes, because die sizes are generally limited. Such a screen also provides lesser percentage of open area than the one provided by the continuous slot type. The openings of such a screen also tend to clog during the development of the well, where the aquifer contains appreciable quantity of sand. These screens are, however, best suited when provided with gravel pack, as is done with slotted pipe screens.

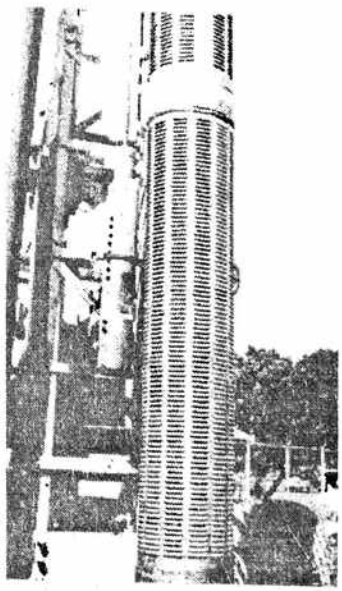


Fig. 4.28. Photoview of a louver type of well screen.

The continuous slot type of strainers as well as the louver type of strainers, though are hydraulically more efficient, but have not become popular in developing countries like India, owning to their high costs.

(*iii*) **Pipe strainers with outer fine mesh jackets**, utilise perforated metal or pvc pipes surrounded by fine wire mesh with some annular space between the two. Several designs are available in the market, and the most commonly adopted type is known as the **agricultural strainer**. A photoview of such a strainer is shown in Fig. 4.29. It consists of a perforated galvanised iron pipe, on which steel or iron rods, or strips of 1 cm width and 3 mm thickness, are welded at fixed intervals around the circumference of the pipe. The perforations are circular and made with a drilling machine. The perforations are arranged in such a manner as not to be covered by the rods. Trapezoidal shaped bronze, brass or copper wire is wound round the series of rods, and securely shouldered, to provide a jacket of non-corrosive alloys.

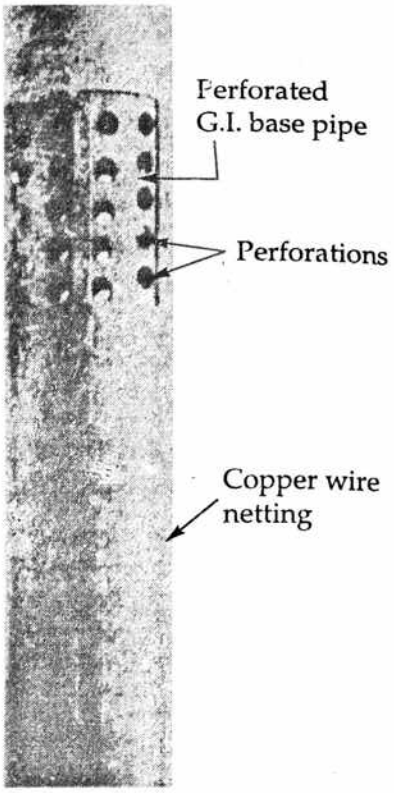


Fig. 4.29. Partially exposed photoview of an agricultural strainer.

The type of wire mesh to be used depends upon the aquifer particle size. Usually, three types of wire nettings suitable for fine, medium and coarse aquifers are available in the market.

In the above design, since the wire netting is not in direct contact with the perforated pipe over which it is wrapped, the area of perforations is not decreased by the netting infront of the perforations. The arrangement, therefore, provides good hydraulic efficiency. Such a strainer could last for 10 years or so.

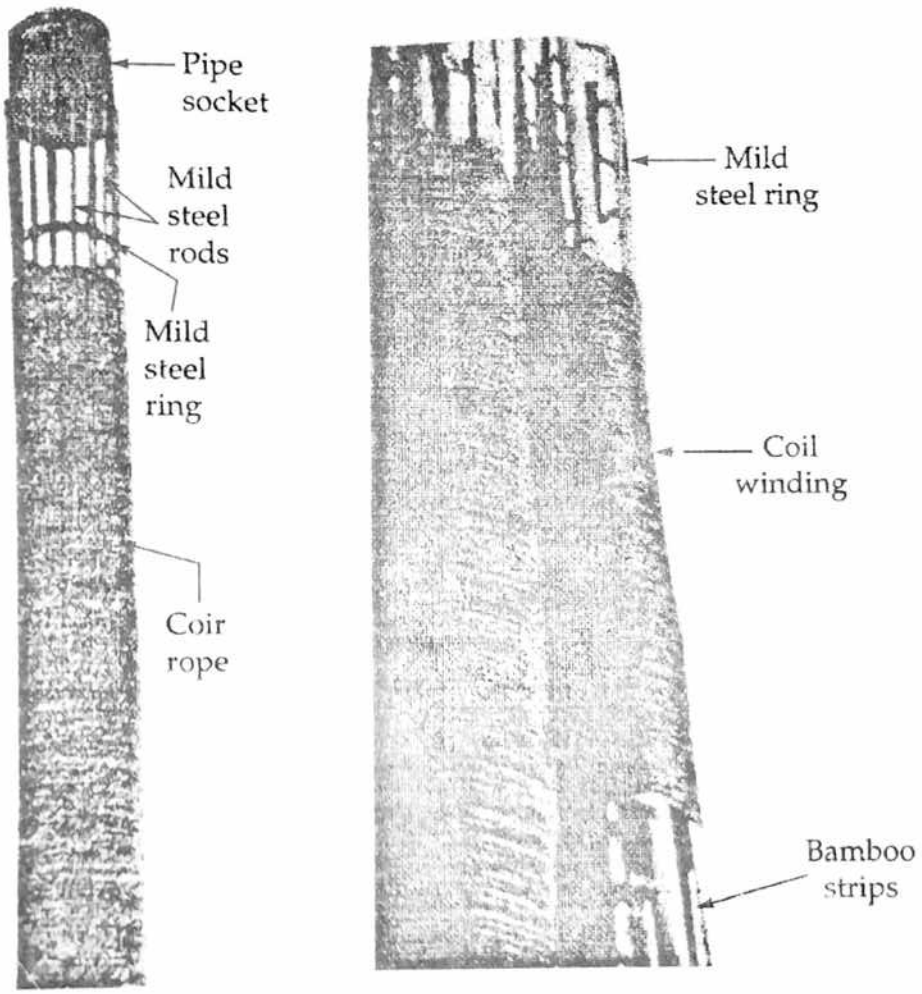
The biggest disadvantage of such a strainer, however, is that it involves bimetallic construction, since the pipe base is of steel and the outer mesh. Jacket is of brass or bronze, etc. Such bimetallic construction causes electrolytic action and corrosion of the steel pipe. To overcome this difficulty, several designs by using the same type of metal in the base pipe as well as in the net jacket have been commercially produced, but are generally very costly, since single metal construction in brass or copper considerably increases the cost of the strainer. Such costly single metal strainers are, however, widely used in developed countries, and are sold under several brand names.

(*iv*) **Coir rope strainers** are low cost strainers, generally used for shallow irrigation tubewells, particularly in deltaic regions of developing countries. Such a strainer essentially consists of a cylindrical frame made of iron rods or bamboo strips, wound round by coir rope of 3 to 5 mm diameter, as shown in Fig. 4.30.

The disadvantage of such a strainer is its short life of 3 to 5 years. The cause of its failure is the rusting of iron bars of the supporting frame, and the loosening of the coir rope which expands on wetting. Use of *nylon rope*, and

coating the iron rods with bitumen have been found to increase its life to a good extent. Since the coir rope strainers cannot withstand the high pressure created during the development of the tubewell by an air compressor, wells using such screens shall have to be developed only by pumps.

Closely mounted *polythene rings* of 6 to 10 mm width have also been used in place of coir rope to make such a strainer to be more resistant to corrosion and deterioration.



(*a*) Coir rope strainer with mild steel frame

(*b*) Coir rope strainer with bamboo frame

Fig. 4.30. Coir rope strainers.

4.14.6. Construction and Boring of Tubewells. With respect to the adopted method of boring, the tubewells can be classified into the following three categories :

(1) *Driven tubewells* ;

(2) *Jetted tubewells* ; and

(3) *Drilled tubewells*.

These three types of borings are discussed below :

4.14.6.1. Driven Tubewells. A driven tubewell consists of a pipe and a well point fixed to the lower end of the pipe as shown in Fig. 4.31. The assembly is forced down the ground to penetrate into the water bearing

formation, by driving it with a wooden maul, drop hammer, or other suitable means.

If the water level is 7 m or more below the land surface, the well pipe should be more than 5 cm in diameter, so that a jet or a cylinder may be inserted and submerged to permit the pump to function.

A special device, called a *cap* or a *drive head* is provided at the top of the pipe assembly during the driving operation, so as to protect the pipe during hammering. After each length of the pipe is hammered into the ground, the cap is removed and additional sections are attached, and driven as required.

The pipe is kept full of water at all times. In its descent through the aquicludes, the water in the pipe shall not flow out through the screen, since the screen is sealed by the impervious formation, but as the well point reaches the water bearing formation, some water will flow out of the pipe, and water remaining in the pipe drops to a static level. This is a signal to the driller that water bearing formation has been reached.

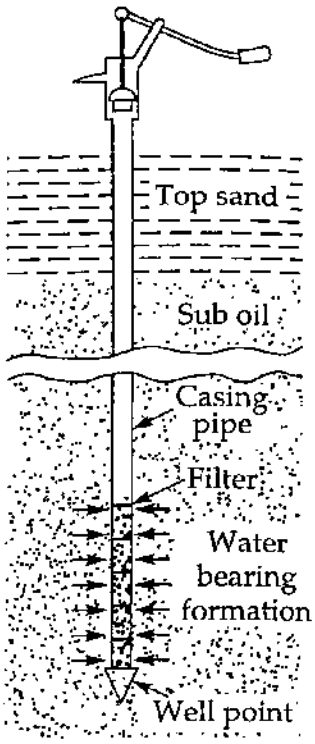


Fig. 4.31. Driven Tubewell.

To develop the well, the driller attaches a small pitcher pump to the pipe. A considerable amount of the sandy water may be pumped for a short time, but if a good aquifer has been tapped, continued pumping should result in clear sand free water.

Driven tubewells yield very small discharges, suitable for individual domestic supplies. Their construction is limited to shallow depths in soft unconsolidated formations free from boulders and other obstructions.

4.14.6.2. Jetted tubewells. A Jetted tubewell may be constructed either with a hand operated equipment or power driven machines, depending upon the soil formation, and size and depth of the well bore.

A hole in the ground is made by the cutting action of a stream of water, which is pumped into the well through a small dia pipe, and forced against the bottom of the hole through the nozzle of a jetting bit (Fig. 4.32). A casing pipe is provided during boring to prevent the caving of the bore hole.

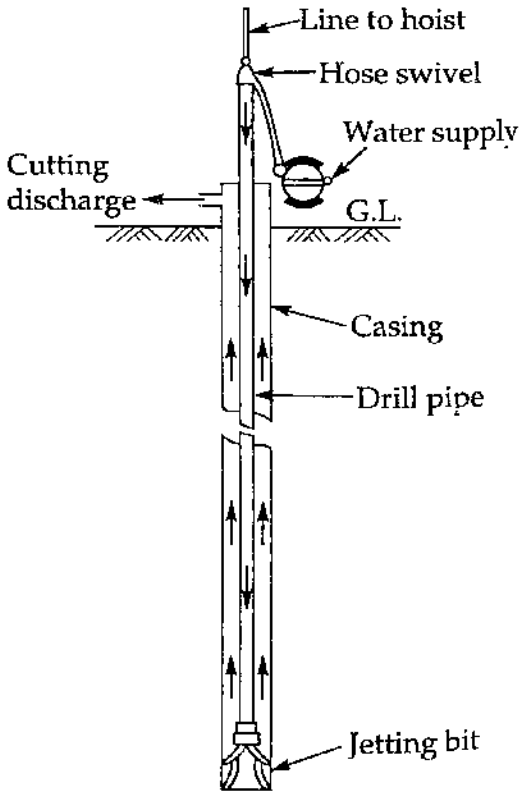


Fig. 4.32. Jetting method of drilling shallow tubewells.

After the hole has been jetted down to the desired depth, the well assembly consisting of blank pipes and screen, is lowered, and *the outer casing is pulled out.*

Jetted tubewells have small yields, and their construction is feasible only in unconsolidated formations.

4.14.6.3. Drilled tubewells. Deep and high capacity wells are constructed by drilling. Various different techniques are employed in drilling the well hole. Different techniques have comparative advantages and disadvantages over each other, depending upon the type of formation to be drilled. Therefore, each well should be treated as an individual project, and one particular method adopted, depending upon its suitability. Some of the drilling methods, commonly used, are described below :

(1) **Standard method or Cable tool method of drilling.** This method of drilling the well hole is also known as **percussion drilling**, because in this method, the well hole is made by percussion, (*i.e.*, by hammering and cutting).

This method is very useful for cutting consolidated rocks from soft clay to hardest rocks, and is generally unsuitable in loose formations, such as unconsolidated sands and gravels or quick sands.

This method becomes ineffective in loose materials, because the loose material slumps and caves around the *drilling bit.* The drill bit has a chisel sharp edge, which breaks the rocks by impact when alternately lifted and dropped. This drilling bit is connected at the lowest end of the entire 'falling and rising arrangement' known as *string of tools.* [Refer Figs. 4.33 (*a*) and (*b*)]. From top to bottom, the string of tools consists of a rope socket, a set of jars, a drill stem, and the drilling bit.

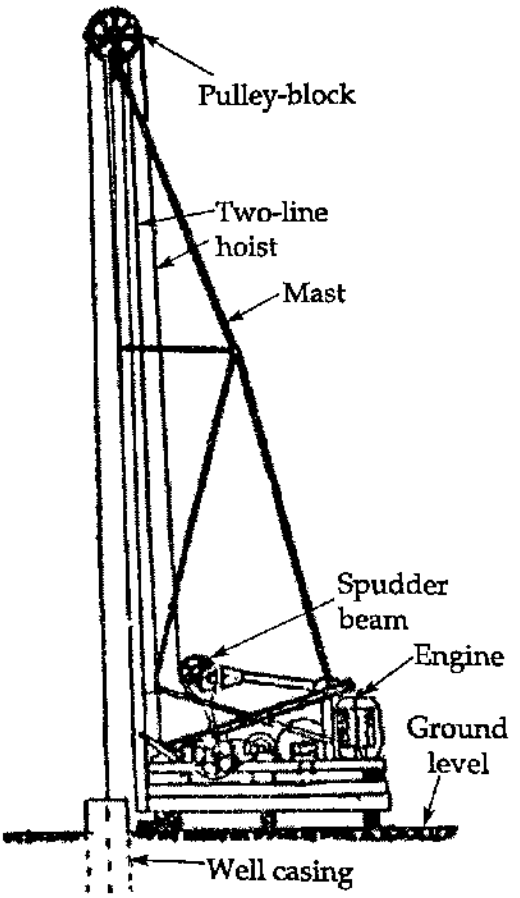


Fig. 4.33 (*a*) Basic components of Percussion Drilling Rig.

Tools are made of steel and joined with tapered box and pin screw joints. The entire assembly weighs several tonnes. The most important tool of the entire assembly is the drilling bit (or drill) as it does the actual rock cutting. The drill stem is the long steel bar which adds weight and length to the drill, so that it can cut rapidly and vertically.

The set of jars have no direct effect on the drilling. They only loosen the tools when they stick in the hole. A rope or a cable is fastened at the upper end to the rope socket and to a dead man (or a heavy weight) at the lower end.

The entire assembly of tools is suspended from an assembly of a mast and a walking beam, etc. This assembly, known as *drilling rig*, in turn, is generally mounted on a truck, so as to make it easily portable. The mast should be sufficiently high, so as to allow the longest of tools to be hoisted.

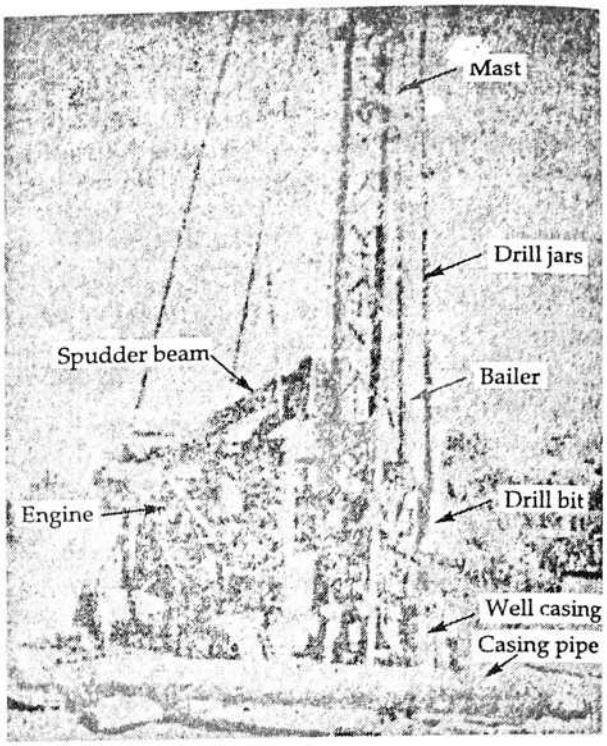


Fig. 4.33 (*b*) Photographic view of a Percussion Drilling Rig. The string of tools is seen to the right of the mast.

As the drilling proceeds, the tools make 40 to 60 strokes per minute, from a height of 0.4 to 1 m. Water is sometimes added in the hole, so as to form a paste with the cuttings, thus reducing friction on the falling bit. After the bit has cut 1 to 2 m through a formation, the string of tools is lifted out, and the hole is cleaned and cleared of the cuttings by means of a *bailer*. The process is known as bailing out the hole.

A bailer essentially consists of a pipe with a valve at the bottom and a ring at the top. When lowered into the well, the valve permits the cuttings to enter the bailer but prevents them from escaping the bailer. After it is filled with cuttings, it is lifted up to the surface and emptied.

In unconsolidated formations, casing should be driven down and maintained near the bottom of the hole to avoid caving. Casing is driven down by means of drive clamps fastened to the drill stem. The up and down motion of the tools, striking the top of the casing, protected by a drive head, sinks the casing. On the bottom of the casing, a drive shoe is fastened to protect the casing, as it is being driven.

(2) **Hydraulic rotary or Direct rotary method of drilling.** This is the fastest method of drilling, and *is especially useful in unconsolidated formations.* The method involves a continuously rotating hollow bit, through which a mixture of clay and water or mud is forced. The bit cuttings are carried up in the hole by the rising mud. *No casing is required during drilling, because the mud itself makes a lining on the walls of the hole, preventing it from caving.*

The drill bit is connected to a hollow steel rod (or drill stem), which, in turn, is connected at the top to a square rod, known as the *Kelley* [Refer Fig. 4.34 (*a*)]. The drill is rotated by a rotating table which fits closely around the Kelley, and allows the drill rod to slide down, as the hole progresses.

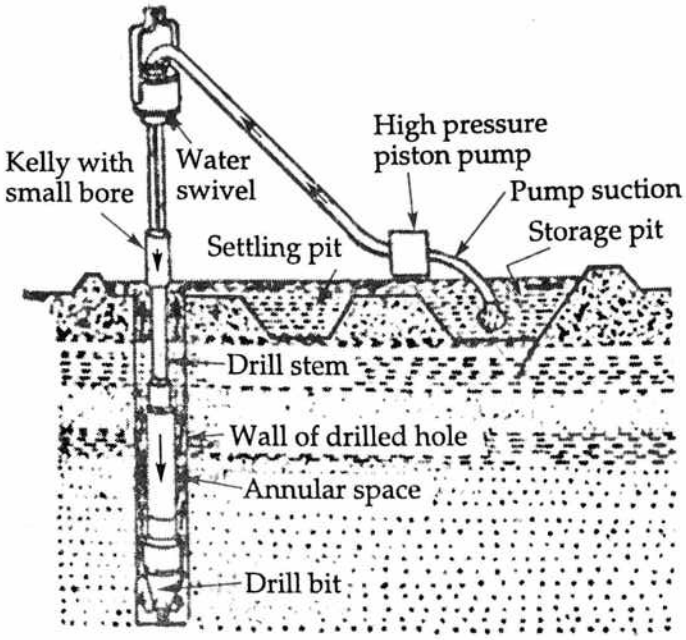


Fig. 4.34 (*a*) Schematic sketch illustrating the basic principles of Direct rotary drilling.

The ***Rotary drilling rig***, such as shown Fig. 4.34 (*b*), consists of a mast, a rotating table, a pump for forcing the mud, a hoist and the engine. The mud, after it emerges out of the hole, is carried to a tank where the cuttings settle out, and the mud can be repumped into the hole.

After the drilling is completed, the casing is lowered into the hole. The clay, deposited on the well-walls during mud pumping, is removed by washing it with water. Water containing some chemicals like sodium hexametaphosphate is forced through the drill rod, and the washings come out through the perforations of the casing. When the washing at one level is completed, the bit is raised and the process repeated.

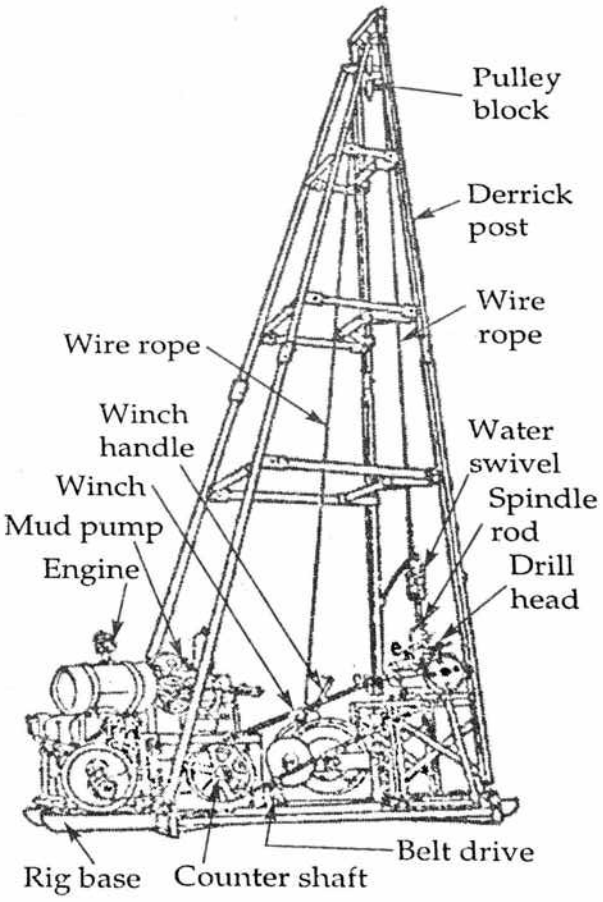


Fig. 4.34 (*b*) Skid mounted light duty Rotary Drilling Rig.

(3) **Reverse rotary method of drilling.** A modification of the hydraulic rotary method is known as the reverse rotary method. This is gaining

popularity day by day. *It is quite useful for making large wells (diameter up to 1.2 m app.) in unconsolidated formations.*

The tool consists of a hollow drill, a drill pipe, and water swivel. In this method, the cuttings are removed by water through a suction pipe called *drill pipe*. The equipment consists of a mast or a derrick, a centrifugal pump, necessary water and power equipments, and the requisite casing pipe.

The hole is driven by pumping water under pressure through the drill bit, while it is churned up and down. The walls of the hole are supported by hydrostatic pressure acting against a film of fine grained material deposited on the walls by the drilling water. Cuttings are removed by water, and after the mixture (water + cuttings) comes out to the surface, it is passed through a settling tank (Refer Fig. 4.35).

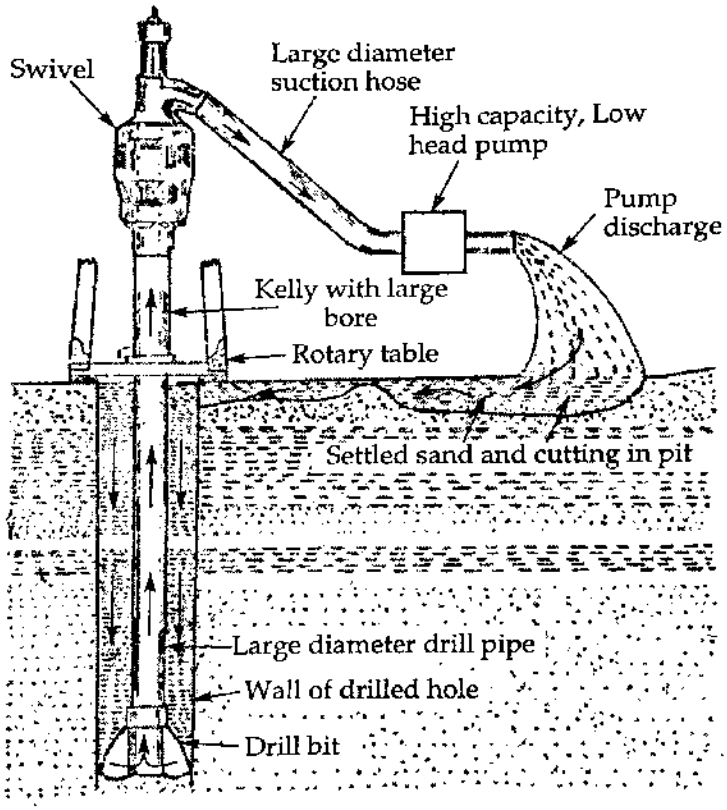


Fig. 4.35. Schematic sketch illustrating the basic principles of Reverse rotary drilling.

The sand settles out here, but the fine grained particles are recirculated, so as to help in stabilising the walls. Casing and cleaning of the walls, etc. is the same as in the hydraulic rotary method.

Comparison of Cable tool and Hydraulic rotary Methods of drilling.

Advantages of Cable tool method are given below :

1. A more accurate sample of the formation can be obtained.
2. Lesser amount of water is required during drilling operations.
3. Cable tool rig is lighter and easy to transport.
4. Very useful for consolidated rocks and less useful for loose formations.
5. For shallow-wells, in unconsolidated materials too, it comes out to be cheaper.

Advantages of Hydraulic rotary method are given below :

1. Can be used for larger holes up to 1.5 m diameter.

2. Can be best used for drilling test holes, because the hole can be abandoned with minimum cost.

3. Rotary drilled hole can be gravel packed, which increases its specific capacity, and keeps the fine particles away, thus causing less sand trouble.

4. Casing is to be driven only after the hole completion, and hence, can be set at any desired depth.

5. It is the fastest method of drilling and especially useful in unconsolidated formations.

6. It can handle alternate hard and soft formations with ease and the danger of accidents is lesser. In quick sands, clays, etc., cable tool method is likely to give troubles, as there is a danger of freezing.

Verticality of the bore hole during drilling must be ensured, irrespective of the method adopted for drilling the bore hole. A common specification allows a deviation of 15 cm from the vertical in a length of 30 m.

4.14.6.4. Well log. During drilling a well hole, the description of the materials encountered in sequence through the drilling, has to be recorded, and this record is called a *well log*. A well log will, therefore, record the different types of formations with their correct depths of occurrences. Samples of drill cuttings at different depths, at regular intervals of 1.5 to 2 m, are also collected, to determine the exact nature of the rock being drilled.

The prepared well log will help in designing and commissioning the tubewell.

4.14.7. Installing Well Screens. Well casings and well screens are installed on the basis of the well log. If the strata conditions, as revealed by the well log, warrant a cavity type tubewell, no screen will be necessary.

In such a cavity well, therefore, the casing pipe used in the drilling, may either be left as it is, or if it is costly, then a smaller dia blind well pipe may be inserted, and the casing pipe withdrawn.

In strainer as well as slotted pipe tubewells, screens or slotted pipes will be required, and the procedures for installing such well screens shall vary with the design of the well and the method employed for drilling. The screens or slotted pipes shall have to be located opposite the water bearing strata, in order to draw water from the strata. This is done by assembling together the whole length of the strainers and the blank pipes in exactly the same order and lengths in which they are to be lowered in the bore.

The strainers and the blind pipes are lowered into the bore one by one, starting from the bottom end. During lowering of these well pipes, the casing pipe may be withdrawn, to allow loosening of the surrounding soil, to grip the strainers and blind pipes, to hold them in position.

In unconsolidated formations, sometimes, the casings are left intact to support the bore hole and to freely admit water into the well. In such a case, the casing pipe itself should either contain *perforations* or its lower part be replaced by a screen or a strainer.

Screens are made from various corrosion-resistant metals. *Plastic screens* are also forming their way in the market. Well-screens are very useful in sandy formations, so that the water containing only a limited quantity of sand (below a given size) may enter the well, and the bigger particles are screened and kept away from entering the well. However, the mesh size of the screen is generally decided on the basis of the grain size distribution curves of the aquifer and the gravel pack, as discussed earlier.

4.14.8. Development of Tubewells. Tubewells are developed to increase their specific capacity, prevent sanding, and obtain maximum economic well life. Development means *the stabilisation of the walls of a well adjacent to the screen, by a process which removes the fine particles from the formation immediately surrounding the well screen, leaving coarser particles to contact and surround the screen.*

The main objectives of well development are :

(*i*) to unclog the water bearing formation

(*ii*) to increase the porosity and permeability of the water bearing formation in the vicinity of the well

(*iii*) to stabilise the sand formation around a screened well, so that the well may yield sand free water.

Development is necessary in all gravel packed wells and other screened wells, except when the screen is made of fine wire mesh or coir or other closely knit filters, located in a highly permeable aquifer.

The basic principle in well development is to cause reversals of flow through the screen openings, that will rearrange the aquifer particles. This is essential to break down *bridging* of groups of particles. Fig. 4.36 shows how small particles can bridge between large particles and across the screen openings, when the flow of water is in one direction only. Reversing the direction of flow by ***surging the well*** overcomes this tendency.

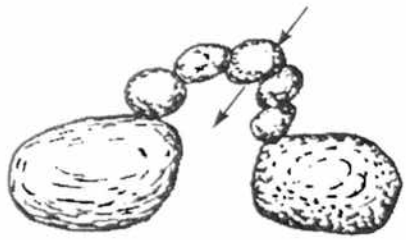


Fig. 4.36. Bridging action caused by one directional flow.

The outflow portion of the ***surge cycle*** breaks down bridging, and the inflow portion moves the fine particles towards the screen and into the well.

The **commonly adopted Tubewell development methods** are briefly discussed below :

(1) **Tubewell development by surging.** In this method, a plunger is worked up and down in the well, so that water is alternately forced out into the surrounding formation and then allowed to flow back into the well. A *surge block* or *surge* plunger is the tool, which is used for this purpose.

The above action loosens the fine sand or gravel particles near the screen, and carries fine particles into the well, from where they are removed.

(2) **Tubewell development by pumping.** In this method, a tubewell may be developed either by *over pumping* or by *rewhiding the well*.

Overpumping involves heavy pumping of the well to cause heavy drawdown. This is not a very effective method for well development, because

the flow of water remains unidirectional, thus not removing bridging of particles.

Rewhiding involves starting and stopping of pumping intermittently to produce relatively rapid changes in the head of the well. While this may be done with any type of pump, it is most effectively done with a turbine type of pump installed with a foot valve.

(3) **Tubewell development by compressed air.** This method is most commonly adopted for developing wells, and may involve either.

(*a*) *backwashing technique*, or

(*b*) *surging technique*.

A combination of both the techniques may sometimes be used for more effective development.

Backwashing technique involves forcing of the well water back into the aquifer by means of compressed air introduced into the well through the top of the casing after it has been closed. When the pressure is released, the water will flow back into the well through the screen to bring fine particles from the area surrounding the well, thus ensuring its removal. The process is continued till no sand is brought in.

Surging technique involves the principles of both the pumping method and the surging method described earlier. The inrush of compressed air creates a powerful surge within the well, and loosens the fine material surrounding the perforations, which may then be brought into the well by continuous air injection. The operation is repeated at intervals along the screened section of the well, until sand arrival is stopped. The principle of pumping is the same as described earlier.

(4) **Tubewell development by jetting.** Jetting with water at high velocity is an effective method of well development. The method involves operating a horizontal water jet inside the well in such a way that the high velocity water streams shoot out through the screen openings. Fine particles are thus washed out of the aquifer, and the turbulence created by the jet brings these fines back into the well through the screen openings above and below the point of operation.

By slowly rotating the jetting tool, and by gradually raising and lowering it, the entire surface of screen can be covered.

Use of dispersing agents in tubewell development. Certain chemicals like Tetra sodium pyrophosphate, sodium tripolyphosphate, sodium hexametaphosphate, and sodium septaphosphate, etc., when added to the well water, and to the water used in *backwashing* or *jetting techniques* of well development described above, considerably helps in mud removal, thereby increasing the effectiveness of the above methods.

4.14.9. Spacing of Tubewells. Tubewells have been generally located without any scientific studies, till recently. It was thought, that for Indo-Gangetic area, tubewells yielding about 45 litres/sec could be located within 2.4 sq. km., which fixed the cultivable commanded area of the well to about 400 hectares. With further research now, tubewells are constructed with a command of 120 hectares. On an average, a tubewell is located in 1.5 sq. km,

and the resultant large scale pumping has not materially lowered the water-table.

4.14.10. Life of a Tubewell and Reasons for its Failure. A normal tubewell lasts for about 15 to 20 years in Northern India. It may fail due to (1) *incrustation*, or due to (2) *corrosion*, as explained below :

(1) **Incrustation.** The incrustation of the well pipe occurs due to the deposition of alkali salts on the inside walls of the pipe. The important alkali salt causing incrustation is *calcium carbonate*, although calcium and magnesium sulphates and silicates may sometimes be the basic binders. The incrustation of the well pipe reduces the effective diameter of the well pipe, and hence reduces the discharge of the tubewell.

The incrustation can be reduced :

(*i*) by reducing the drawdown and hence the pumping rate ;

(*ii*) by using screens having larger area of openings (or larger diameter pipes) so as to keep some allowance for the future incrustation ;

(*iii*) by using such materials for the strainers (screens) that may easily permit the removal of incrustating material by chemical action without effecting that strainer material. In other words, the acids, etc., which are used to remove the incrustation, should not produce any effect on the strainer materials ; and

(*iv*) by properly maintaining and periodically cleaning the well screens.

The incrustation can be delayed or reduced by these four methods, although it cannot be completely eliminated, and thus the life of the tubewell can be increased.

(2) **Corrosion.** The well pipe is gradually destroyed by corrosion due to the action of acidic waters on the pipe material. When chlorides and sulphates or carbon dioxide are present in the water, the well pipe will definitely get corroded. The aquifer sand, surrounding the well pipe, finds a way out into the corroded pipe through the worn out pipe walls, thus bringing sand along with water, Hence, *corrosion results in excessive withdrawal of sand along with well water.*

Thicker pipes may be used to avoid corrosion. Stainless steel strainers will be most suitable, but are very costly. In affluent countries like U.S.A., such stainless steel screens are being progressively used, but are too expensive for developing countries like India. An iron or steel screen can be depended upon only for a limited service life of the tubewell in most of the waters. The life of such a screen can be increased by galvanising (*i.e.*, by zinc coating) the pipe material. Other measures, which can help in reducing corrosion, and thereby increasing the life of the tubewell, are :

(*i*) by reducing the drawdown and the pumping rate ;

(*ii*) by reducing the flow velocity by increasing the percentage of open area or the diameter of the well pipe ;

(*iii*) by using thicker pipes ;

(*iv*) by using corrosion resistant materials for pipes ; and

(*v*) by using corrosion resistant coatings on the pipes.

4.15. Yield of Wells and Tubewells by Thiem's Equilibrium Formulas

4.15.1. Thiem's Formula for Unconfined Aquifers (i.e. for Watertable or Gravity Wells). Let a non-artesian well be driven and water pumped heavily so as to cause a sufficient drawdown. When the water level in the well decreases, the water level in the neighbourhood will also fall down, forming what is called an **inverted cone of depression** all around the well, as shown in Fig. 4.37. The base of this cone is a circle of radius R, known as the **circle of influence** ; and the inclined side is known as the ***drawdown curve.***

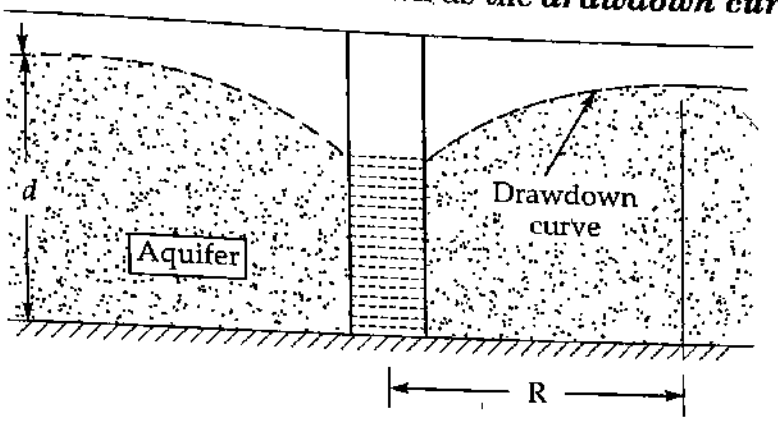


Fig. 4.37. Drawdown curve in an unconfined aquifer.

In the method suggested by Thiem, two observation wells lying within the circle of influence of the main pumped well are to be driven. Let these wells be numbered as 1 and 2 (Fig. 4.38) and let them be at distances of r_1 and r_2 from the main well *i.e.*, centre to centre distance. Let d be the saturated depth of the aquifer *i.e.* the depth below the watertable.

Now, the pumping is to be started from the main well at a sufficient rate, so as to cause heavy drawdown. Then, the pumping is to be adjusted so that the equilibrium conditions are reached. In other words, the rate of pumping becomes equal to the rate of yield and thus causing the water level to attain a constant value. Since the formula involves the use of equilibrium conditions, it is known as *equilibrium formula.* Let s_1 and s_2 be the drawdowns in the two corresponding observation wells ; and h_1 and h_2 be the water depths in these wells.

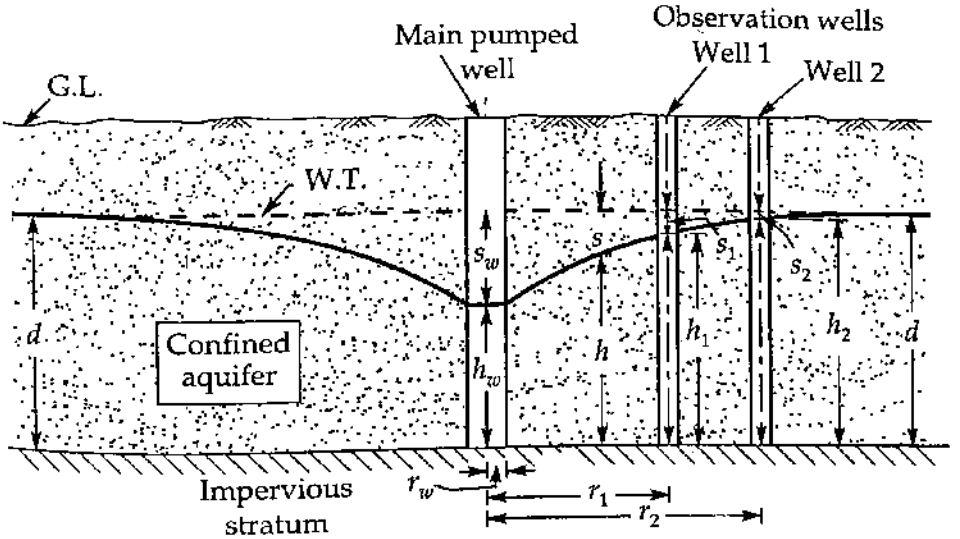


Fig. 4.38. Unconfined aquifer case for derivation of Thiem's formula.

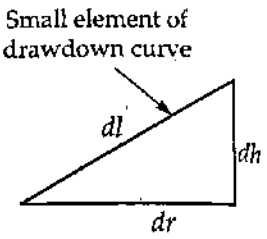
From Darcy's law, the flow through any concentric cylindrical section of the water bearing material is given as :

$$Q = KIA$$

where I = Hydraulic gradient

using cylindrical coordinates, we take r as the radius of any cylinder, and let h be the height of the cone of depression at a distance r from the well.

Also assuming that the inclination of the water surface is small, so that the tangent can be used in place of sine for the hydraulic gradient in Darcy's law, we have



$$I = \frac{dh}{dl} \approx \frac{dh}{dr}$$

$$\therefore \quad I = \frac{dh}{dr}.$$

The area of flow (A) is equal to $2\pi r.h$, assuming that water flows through the full height of the aquifer, and also that the flow is radial and horizontal*.

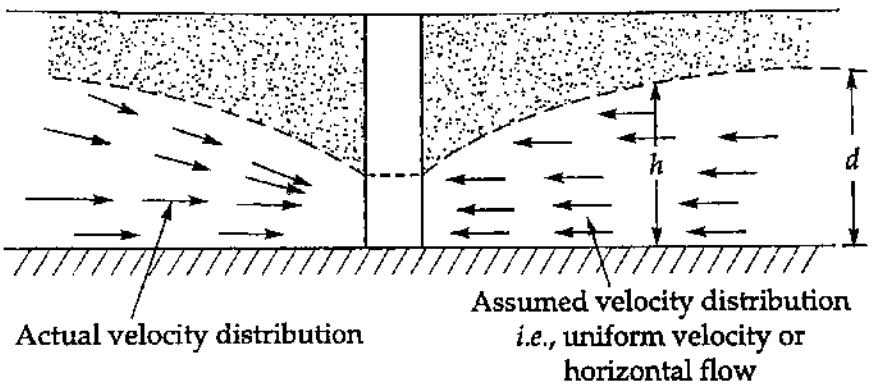


Fig. 4.39. Flow pattern in an unconfined aquifer.

Substituting these values of I and A in Darcy's law, we get

$$Q = KIA = K \cdot \frac{dh}{dr} \cdot 2\pi r.h$$

or $$Q = 2\pi K.h.r.\frac{dh}{dr}$$

or $$\frac{dr}{r} = \frac{2\pi K h dh}{Q}$$

Integrating between the limits r_1 and r_2, and h_1 and h_2, we get

$$\int_{r_1}^{r_2} \frac{dr}{r} = \int_{h_1}^{h_2} \frac{2\pi K}{Q} . h dh$$

where Q = Constant when steady conditions have reached.

K = Soil permeability, assumed to be constant *at all places and at all times, i.e.,* (***homogeneous soil***) and in all directions. *i.e.* (***isotropic soil***) by assuming the soil to be homogeneous and isotropic.

*By horizontal flow, we mean that velocity distribution is assumed to be uniform as shown in Fig. 4.39.

Therefore $$\int_{r_1}^{r_2} \frac{dr}{r} = \frac{2\pi K}{Q} \int_{h_1}^{h_2} h dh$$

or $$\left| \log_e r \right|_{r_1}^{r_2} = \frac{2\pi K}{Q} \left| \frac{h^2}{2} \right|_{h_1}^{h_2}$$

or $$\log_e \frac{r_2}{r_1} = \frac{2\pi K}{Q} \left[\frac{h_2^2 - h_1^2}{2} \right]$$

$$= \frac{\pi K}{Q} \left[h_2^2 - h_1^2 \right]$$

or $$K = \frac{Q \cdot \log_e \frac{r_2}{r_1}}{\pi (h_2^2 - h_1^2)}$$

or $$\boxed{Q = \frac{\pi K (h_2^2 - h_1^2)}{2.3 \log_{10} \left(\frac{r_2}{r_1} \right)}}$$ for unconfined aquifers ...(4.30)

This important Thiem's formula can be further simplified, if required, as follows :

$$(h_2^2 - h_1^2) = (h_2 + h_1) \cdot (h_2 - h_1)$$

But $$h_2 - h_1 = s_1 - s_2$$

and if the amount of drawdown is small compared to the saturated thickness of the water bearing material, then h_2 and h_1 are nearly equal and each is approximately equal to this saturated thickness say d.

Therefore, $$h_2 + h_1 \approx d + d = 2d$$

or $$(h_2^2 - h_1^2) \approx (s_1 - s_2)\, 2d = 2d(s_1 - s_2)$$

Putting the value in equation (4.30), we get

$$Q \approx \frac{2\pi . Kd . (s_1 - s_2)}{2.3 \log_{10} \frac{r_2}{r_1}} \qquad ...(4.31)$$

Introducing the coefficient of transmissibility T in place of K *i.e.* $T = Kd$, we have

$$Q \approx \frac{2\pi T (s_1 - s_2)}{2.3 \log_{10} \frac{r_2}{r_1}} \qquad ...(4.32)$$

This is another form of Thiem's formula for unconfined aquifers. The various assumptions that we have made in its derivations are again summarised below :

1. The aquifer is homogeneous, isotropic, and of infinite and areal extent, so that its coefficient of transmissibility or permeability is constant everywhere.
2. The well has been sunk through the full depth of the aquifer, and it receives water from the entire thickness of the aquifer.

3. Pumping has continued for a sufficient time at a uniform rate, so that equilibrium stage or steady flow conditions have reached.
4. Flow lines are radial and horizontal and flow is laminar.
5. The inclination of the water surface is small so that its tangent can be used in place of sine for the hydraulic gradient in Darcy's equation.

4.15.2. Thiem's Formula for Confined Aquifers (i.e. for Pressure or Artesian wells). The formula derived for an unconfined aquifer has to be slightly modified in the case of an artesian aquifer. The conditions here are as shown in Fig. 4.40.

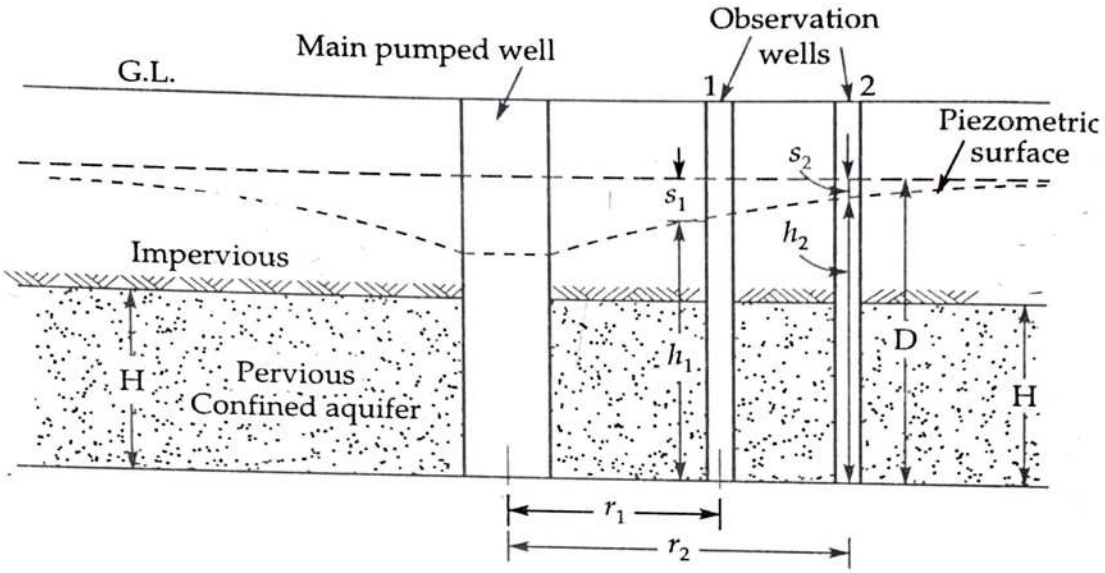


Fig. 4.40. Confined aquifer case for derivation of Thiem's formula.

In a confined aquifer, the flow is actually radial and horizontal (Refer Fig. 4.41) and, therefore, it has not to be assumed as such, as was done in the unconfined case. Rest of the assumptions remain the same and hold good in this case also.

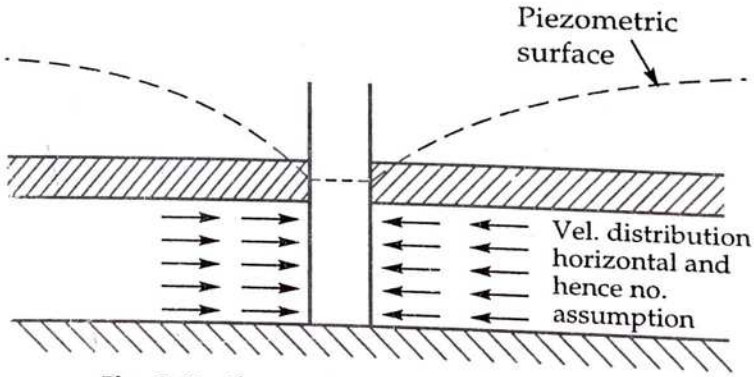


Fig. 4.41. Flow pattern in a confined aquifer.

In this case, the discharge equation is

$$Q = KIA = K \cdot \frac{dh}{dr} 2\pi H$$

as water is drawn from the cylinder of radius r and ht. = H, where H = height of the confined aquifer.

or $$Q = 2\pi K.H.r.\frac{dh}{dr}$$

or $$\frac{dr}{r} = \frac{2\pi KH}{Q}.\,dh$$

Integrating between the limits of r_1 and r_2, we get

$$\int_{r_1}^{r_2} \frac{dr}{r} = \frac{2\pi KH}{Q}\int_{h_1}^{h_2} dh$$

or $$\left|\log_e r\right|_{r=r_1}^{r=r_2} = \frac{2\pi KH}{Q}.\left|h\right|_{h=h_1}^{h=h_2}$$

or $$\log_e \frac{r_2}{r_1} = \frac{2\pi KH}{Q}\left[h_2 - h_1\right]$$

or $$Q = \frac{2\pi KH(h_2 - h_1)}{2.3\log_{10}\frac{r_2}{r_1}} \qquad ...(4.33)$$

But $h_2 - h_1 = s_1 - s_2$

Therefore, $$\boxed{Q = \frac{2\pi KH(s_1 - s_2)}{2.3\log_{10}\frac{r_2}{r_1}}}$$ for confined aquifers ...(4.34)

Since $KH = T$, we get

$$Q = \frac{2\pi.T(s_1 - s_2)}{2.3\log_{10}\frac{r_2}{r_1}} \qquad ...(4.35)$$

where $s_1 - s_2$ is the difference of water levels between the two observation wells after the steady conditions have reached.

4.15.3. Limitations of Thiem's formulae. Various assumptions have been made in the derivation of the above Thiem's formulae. But, however, in actual practice, none of these conditions may get fulfilled ; say for example, an aquifer is not fully homogeneous, or the well might have been dug half way through the aquifer, or permeability may not be uniform, or the ground watertable may be inclined and thus the base of the cone may not be a circle, or the equilibrium conditions might have not fully reached.

However, it is very difficult to assess the effects of these factors and despite the various limiting assumptions, Thiem's formulae are widely used in ground water problems and many of its limitations are removed by appropriate adjustments.

4.16. Dupuit's Equilibrium Formulas for Estimating Yield of Wells and Tubewells

The formulae put forward by Thiem and which have been derived earlier for a gravity well as well as for a pressure well are the refined forms of Dupuit's basic formulas.

We will now discuss the original Dupuit's formulas. In Dupuit formulas, no observation wells (as constructed in Thiem's formulas) are constructed. The main well is pumped out so as to get sufficient drawdown, and then the rate of pumping is so adjusted as to establish equilibrium conditions (*i.e.*, rate of inflow becomes equal to the rate of outflow, and water level in the well becomes constant).

All the assumptions which have been made in the Thiem's formulas hold good in Dupuit's formulas also. The only difference is that the integration which was done between the limits of r_1 and r_2 (radii of the two observation wells) in Thiem's formula is changed and the integration is done between the limits r_w and R, where r_w is the radius of the pumped well and R is the radius of influence. The *radius of influence* being the distance from the centre of the pumped well to the point where the drawdown is zero or is inappreciable. The complete derivations of the Dupuit's formulae for an unconfined aquifer (gravity well) as well as for a confined aquifer (pressure well) are given below:

4.16.1. Dupuit's Formula for Unconfined Aquifers (Gravity Wells). (Refer Fig. 4.42).

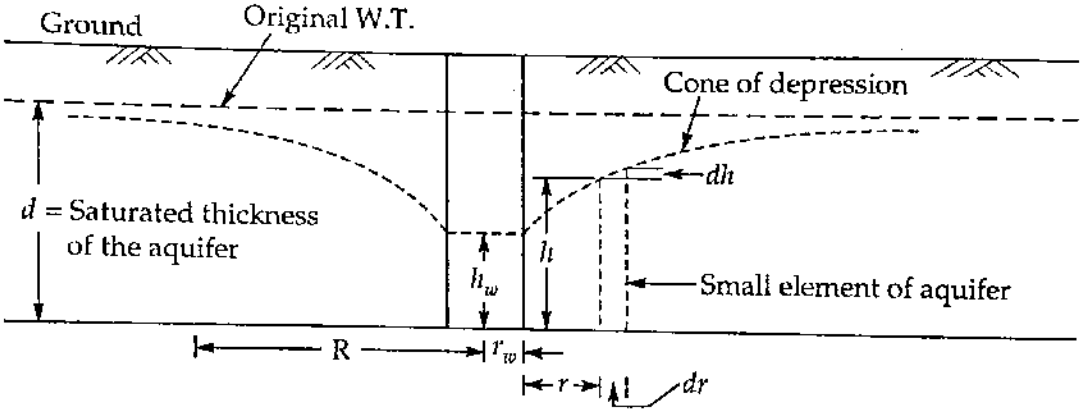


Fig. 4.42. Unconfined aquifer case for derivation of Dupuit's formula.

$$Q = K.I.A$$

or
$$Q = K \cdot \frac{dh}{dr} \cdot 2\pi rh$$

or
$$\frac{dr}{r} = \frac{2\pi K}{Q} \cdot h.dh$$

Integrating between the limits r_w and R, we get

$$\int_{r_w}^{R} \frac{dr}{r} = \frac{2\pi K}{Q} \int_{h_w}^{d} h.dh$$

$$\left| \log_e r \right|_{r_w}^{R} = \frac{2\pi K}{Q} \left| \frac{h^2}{2} \right|_{h_w}^{d}$$

or
$$\log_e \frac{R}{r_w} = \frac{\pi K}{Q} \cdot \left[(d^2 - {h_w}^2) \right]$$

or
$$2.3 \log_{10} \frac{R}{r_w} = \frac{\pi K}{Q} \left[(d^2 - {h_w}^2) \right]$$

or $$\boxed{Q = \frac{\pi K(d^2 - h_w^2)}{2.3 \log_{10} \frac{R}{r_w}}}$$ for unconfined aquifers ...(4.36)

Since the value of R is not easily assessable, various arbitrary values have been assigned to R by various investigators. Slitcher gives it as 150 m, and Tolman calls it as 300 m. But a more realistic picture is obtained, when R is taken as : $R \propto Q$ or $R = CQ$

where C is a constant and Q is the discharge

Putting $R = CQ$ in equation (4.36), we get

$$Q = \frac{\pi K(d^2 - h_w^2)}{2.3 \log_{10} \left(\frac{CQ}{r_w}\right)} \quad ...(4.37)$$

Q can be determined by Hit and Trial method.

4.16.2. Dupuit's Formula for Confined Aquifers (Artesian Wells). (Refer Fig. 4.43).

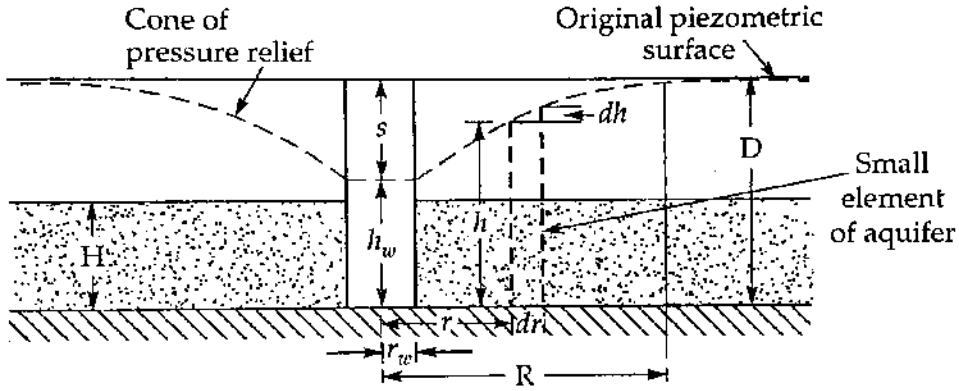


Fig. 4.43. Confined aquifer case for derivation of Dupuit's formula.

$$Q = KIA$$

or $$Q = K \cdot \frac{dh}{dr} \cdot 2\pi \cdot rH$$

or $$\frac{dr}{r} = \frac{2\pi KH}{Q} \cdot dh$$

Integrating between r_w and R, we get

$$\int_{r_w}^{R} \frac{dr}{r} = \frac{2\pi KH}{Q} \int_{h_w}^{D} dh$$

or $$\left| \log_e r \right|_{r=r_w}^{r=R} = \frac{2\pi KH}{Q} \cdot \left| h \right|_{h=h_w}^{h=D}$$

or $$2.3 \log_{10} \frac{R}{r_w} = \frac{2\pi KH}{Q} \left[(D - h_w) \right]$$

or
$$Q = \frac{2\pi KH(D - h_w)}{2.3 \log_{10} \frac{R}{r_w}} \quad ...(4.38)$$

or
$$\boxed{Q = \frac{2\pi KHs}{2.3 \log_{10} \frac{R}{r_w}}} \quad \text{for confined aquifers} \quad ...(4.39)$$

where H = total ht. of confined aquifer.
h_w = artesian pressure in the well.
r_w = radius of the well.
D = initial artesian pressure at the bottom of the aquifer or the initial ht. of the piezometric surface from the bottom of the well.
s = drawdown = $(D - h_w)$.

4.17. Partial Penetration of an Aquifer by a Well

From eqn. (4.36), we have

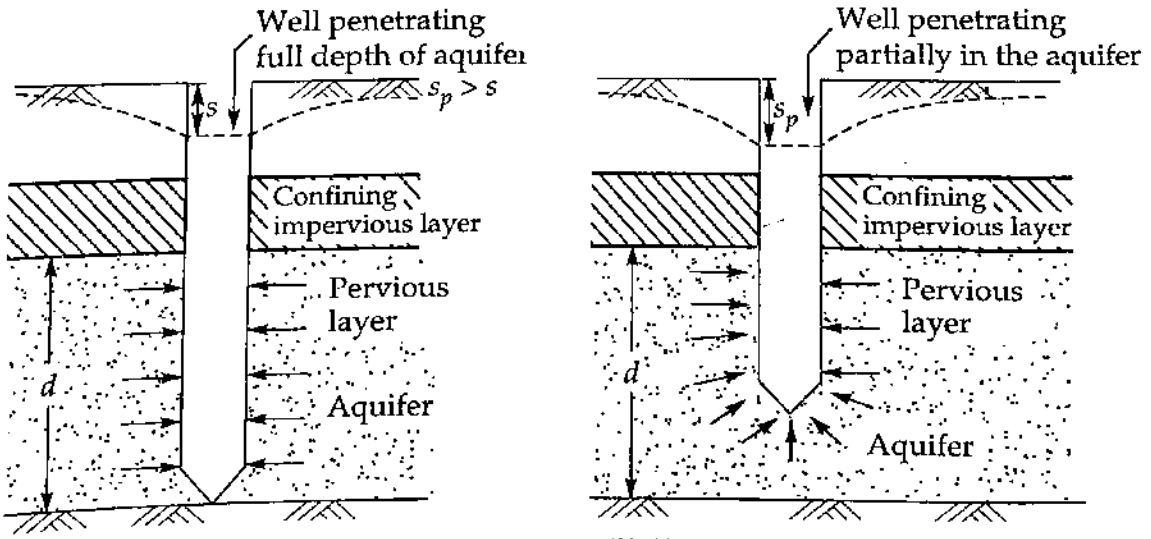
$$Q = \frac{\pi K(d^2 - h_w^2)}{2.3 \log_{10} \left(\frac{R}{r_w}\right)} \quad \text{(for unconfined or gravity wells)}$$

and from eqn. (4.38), we have

$$Q' = \frac{2\pi KH(D - h_w)}{2.3 \log_{10} \left(\frac{R}{r_w}\right)} \quad \text{(for confined wells)}$$

Note. The value R may be assumed at about 300 m for unconfined wells and 3000 m for confined wells.

The above equations have been derived, respectively for a gravity well and for an artesian well, which penetrate throughout the aquifer. But, if a well



(*a*) Showing flow lines in a well fully penetrating the aquifer

$s_p > s$
$Q_p < Q$

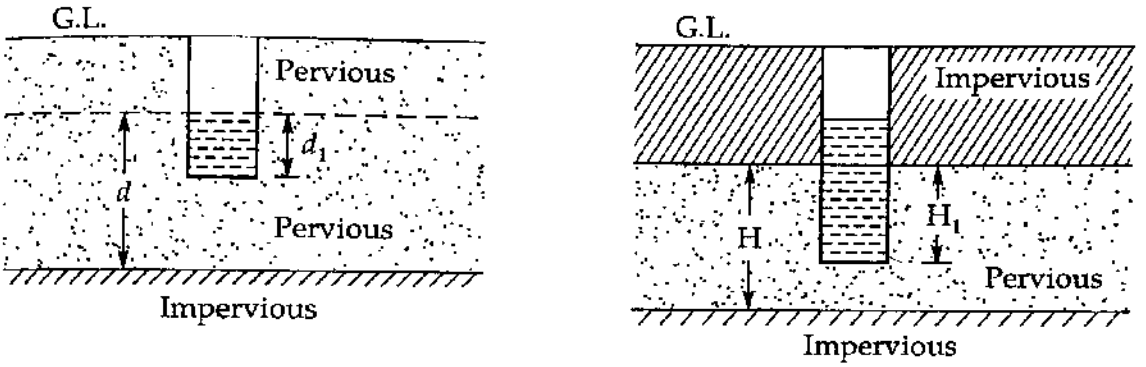
(*b*) Showing flow lines in a well partly penetrating the aquifer

Fig. 4.44

does not penetrate up to the bottom of the aquifer, these formulas will not be applicable, as the nature of the flow will become three dimensional. It will not only be radial but will also have an upward component, as shown in Fig. 4.44. The measured drawdown (s_p) in the well influenced by the upward vertical flow component as in the case of a partially penetrating well will be more than that in a fully penetrating well (s) for the same discharge. *The discharge obtained by the fully penetrating well* (Q) will of course be more than that of a partially penetrating well (Q_p) for the same drawdown.

Kozeny has given a correction factor, and according to him the discharge Q_p through a partially penetrating well is given as follows :

Discharge (Q_p) for a partially penetrating unconfined (gravity) well [Fig. 4.45 (a)], Q_p is given as :



(*a*) Partially penetrating gravity (unconfined) well

(*b*) Partially penetrating artesian (confined) well

Fig. 4.45

$Q_p = Q \times$ Correction factor (F) ; where

$$F = \left[1 + 7 . \sqrt{\frac{r_w}{2d_1}} . \cos\frac{\pi\alpha}{2}\right] \times \alpha \qquad ...(4.40)$$

where Q = Discharge of fully penetrating unconfined (gravity) well given by Eq. (4.36)

d_1 = Actual penetration depth below the water-table

d = Full depth of the aquifer below the water-table

$\frac{d_1}{d} = \alpha$ = Penetration fraction.

Similarly, the discharge (Q_p) for a partially penetrating artesian well (confined aquifer) as per Fig. 4.45 (b) is given as :

$Q_p = Q \times$ Correction factor (F) ; where

$$F = \left[1 + 7 . \sqrt{\frac{r_w}{2H_1}} . \cos\frac{\pi\alpha}{2}\right] \times \alpha \qquad ...(4.41)$$

where Q = Discharge of fully penetrating confined (artesian) well, given by Eq. (4.38).

H = Full depth of the confined aquifer.

H_1 = Depth upto which the well penetrates.

$\frac{H_1}{H} = \alpha$ = Penetration fraction.

D, h_w, r and R have the same meaning as given earlier.

Muskat (1946) has however stated that the above correction factor (F) holds good for specific capacity $\left(i.e. \frac{\text{discharge}}{\text{drawdown}}\right)$ rather than for discharge. In other words, according to Muskat.

$$\left[\text{Specific capacity } C_p \text{ of partially penetrating well } i.e. \frac{Q_p}{s_p}\right]$$

$$= \left[\text{Specific capacity } C \text{ of fully penetrating well } i.e. \frac{Q}{s}\right] \times F.$$

or $$C_p = C \,.\, F \qquad ...(4.42)$$

or $$\frac{Q_p}{s_p} = \frac{Q}{s} \,.\, F$$

or $$Q_p = QF\left(\frac{s_p}{s}\right) \qquad ...(4.43)$$

Example 4.7. *A 40 cm well penetrates an aquifer of 30 m thickness and the length of the strainer (screened portion) is 10 m. The yield is 2000 lpm with a drawdown in the well of 3 m. If the length of the screen is increased to 20 m, what will be the drawdown in the well and the increase in the specific capacity.*

Solution. Correction factor (F) for confined aquifers for obtaining discharge or specific capacity values of partially penetrating well is given by eqn. (4.41) as :

$$F = \left[1 + 7 \,.\, \sqrt{\frac{r_w}{2H_1}} \,.\, \cos\frac{\pi\alpha}{2}\right] \,.\, \alpha$$

where $\alpha = \frac{H_1}{H}$

When partial penetration depth (H_1) equals 10 m and full penetration depth (H) = 30 m, we have $\alpha = \frac{10}{30}$

$$F = \left[1 + 7\sqrt{\frac{0.4}{2 \times 10}} \,.\, \cos\frac{\pi}{2} \times \frac{10}{30}\right]\frac{10}{30} = 0.619$$

Similarly, when partial penetration depth (H_1) equals 20 m, we have

$$F = \left[1 + 7 \,.\, \sqrt{\frac{0.4}{2 \times 20}} \,.\, \cos\frac{\pi}{2} \,.\, \frac{20}{30}\right] \,.\, \frac{20}{30} = 0.9$$

Let C represents the specific capacity of fully penetrating well, and C_p the specific capacity of partially penetrating well. *Then for 1st case of 10 m penetration*, we have

$$C_p = CF \text{ (... Eq. 4.42)}$$

$$\text{where } C_p = \frac{Q_p}{s_p} = \frac{2 \text{ m}^3/\text{min}}{3 \text{ m}} = 0.667 \text{ m}^2/\text{min}$$

$$F = 0.619 \text{ (calculated above)}$$

$$\therefore \quad 0.667 = C \times 0.619$$

$$\therefore \quad \text{or} \quad C = \frac{0.667}{0.619} = 1.08 \text{ m}^2/\text{min}$$

For the 2nd case of 20 m penetration, we have

$$C_p' = C \,.\, F' \text{ (Eq. 4.42)}$$

where $F' = 0.9$ (calculated above)

$$\therefore \quad C_p' = 1.08 \times 0.9 = 0.97 \text{ m}^2/\text{min}$$

$$\text{But} \quad C_p' = \frac{Q_p'}{s_p'}$$

$$\therefore \quad 0.97 \text{ m}^2/\text{min} = \frac{2 \text{ m}^3/\text{min}}{s_p'}$$

$$\text{or} \quad s_p' = \frac{2}{0.97} \text{ m} = 2.06 \text{ m}$$

Drawdown in 2nd case = s_p' = **2.06 m. Ans.**

(Reduced drawdown due to more penetration for the same discharge)

% age increase in sp. capacity in the 2nd case

$$= \frac{C_p' - C_p}{C_p}$$

$$= \frac{0.97 - 0.667}{0.667} = \mathbf{45.4\%.} \quad \textbf{Ans.}$$

4.18. Spherical Flow in a Well

Fig. 4.46 shows a special case of partially penetrating well, where the well just penetrates up to the top surface of the semi-infinite porous medium. In this case, $H_1 = 0$, and the equation (4.41) is not applicable, because the flow towards the well becomes purely spherical. The discharge Q_s from such a well can, however, be calculated from the equation,

$$Q_s = 2\pi K.r_w (D - h_w) \qquad \text{...(4.44)}$$

where $D - h_w$ is the drawdown in the well.

Whereas in the case of a simple radial flow in a fully penetrating artesian well, the discharge is given by the equation (4.38) as :

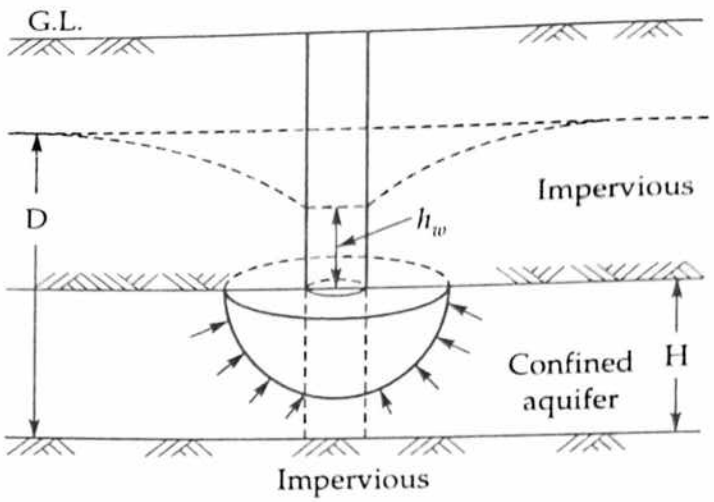


Fig. 4.46. Spherical flow in a well.

$$Q = \frac{2\pi K.H.(D - h_w)}{2.3 \log_{10}\left(\frac{R}{r_w}\right)} \quad \text{...(i.e. Eq. 4.38)}$$

Now
$$\frac{Q_s}{Q} = \frac{2\pi K r_w.(D - h_w)}{\left(\frac{2\pi KH(D - h_w)}{2.3 \log_{10}\left(\frac{R}{r_w}\right)}\right)}$$

or
$$\frac{Q_s}{Q} = \frac{2.3\, r_w \log_{10}\left(\frac{R}{r_w}\right)}{H}$$

or
$$\frac{Q_s}{Q} = 2.3\left(\frac{r_w}{H}\right)\log_{10}\left(\frac{R}{r_w}\right) \quad \text{...(4.45)}$$

For example, if

r_w = 10 cm = 0.1 m ; and
R = 3000 m

Then $\frac{R}{r_w}$ = 30,000 ; and let

H = thickness of the confined aquifer = 20 m (say)

Then
$$\frac{Q_s}{Q} = 2.3\left(\frac{0.1}{20}\right)\log_{10}(30{,}000) = 0.05 \approx \frac{1}{20}$$

This shows that the yield in a spherical flow is much less than that in a radial flow. Hence, *the spherical flow is much less efficient than the radial flow.*

4.19. Interference Among Wells

If two or more wells are constructed in such a way that they are near to each other and their cones of depressions intersect, they are said to interfere.

Such mutual interference of wells decreases the discharges of the interfering wells.

Muskat has proposed the following formulas for computation of discharges from such interfering wells. These formulas have been found to yield reliable results.

(A) For Confined aquifers (*i.e.* **Artesian wells)**

(1) *For two artesian identical* wells at a distance B apart,*

$$Q_1 = Q_2 = \frac{2\pi KH.(D - h_w)}{2.3 \log_{10}\left(\frac{R^2}{r_w \,.\, B}\right)} \qquad ...(4.46)$$

(2) *For three artesian identical wells at distances B apart, in a pattern of equilateral triangle,*

$$Q_1 = Q_2 = Q_3 = \frac{2\pi KH(D - h_w)}{2.3 \log_{10}\left(\frac{R^3}{r_w \,.\, B^2}\right)} \qquad ...(4.47)$$

(3) *For three artesian identical wells, at distances B apart in a straight line.*

$$Q_1 = Q_3 = \left[\frac{\frac{2\pi KH}{2.3}(D - h_w) \,.\, \log_{10}\left(\frac{B}{r_w}\right)}{\log_{10}\left(\frac{R}{B}\right) \,.\, \log_{10}\left(\frac{B}{r_w}\right) + \log_{10}\left(\frac{B}{2r_w}\right) \,.\, \log_{10}\left(\frac{R}{r_w}\right)}\right] \qquad ...(4.48)$$

and

$$Q_2 = \left[\frac{\frac{2\pi KH}{2.3}(D - h_w) \,.\, \log_{10}\left(\frac{B}{2r_w}\right)}{2.\log_{10}\left(\frac{R}{B}\right) \,.\, \log_{10}\left(\frac{B}{r_w}\right) + \log_{10}\left(\frac{B}{2r_w}\right) \,.\, \log_{10}\left(\frac{R}{r_w}\right)}\right] \qquad ...(4.49)$$

where Q_1 and Q_3 are the discharges of the outerwells, and Q_2 is the discharge of the middle well.

Total discharge = $Q_1 + Q_2 + Q_3$.

(B) For Unconfined aquifers (*i.e.* **Gravity wells)**

All the three above formulas can be applied to unconfined aquifers by replacing $H.D$ by $\frac{d^2}{2}$, and $H.h_w$ by $\frac{h_w^{\,2}}{2}$. Thus,

(1) *For two identical gravity wells at distance B part, formula would therefore, become*

$$Q_1 = Q_2 = \frac{2\pi K\left(\frac{d^2}{2} - \frac{h_w^{\,2}}{2}\right)}{2.3 \log_{10}\left(\frac{R^2}{r_w \,.\, B}\right)}$$

*Wells of the same dia., drawdown, and discharge over the same period of time.

or $$Q_1 = Q_2 = \left[\frac{\pi K(d^2 - h_w{}^2)}{2.3 \log_{10}\left(\frac{R^2}{r_w \,.\, B}\right)}\right] \quad ...(4.50)$$

(2) *For three identical gravity wells at distances B apart, in a pattern of equilateral triangle*, formula would be

$$Q_1 = Q_2 = Q_3 = \frac{2\pi K \,.\left(\frac{d^2}{2} - \frac{h_w{}^2}{2}\right)}{2.3 \log_{10}\left(\frac{R^3}{r_w \,.\, B^2}\right)}$$

or $$Q_1 = Q_2 = Q_3 = \frac{\pi K(d^2 - h_w{}^2)}{2.3 \log_{10}\left(\frac{R^3}{r_w \,.\, B^2}\right)} \quad ...(4.51)$$

(3) *For three identical gravity wells at distances B apart in a straight line*, the formulas would become

$$Q_1 = Q_3 = \left[\frac{\frac{\pi K}{2.3}(d^2 - h_w{}^2) \,.\, \log_{10}\frac{B}{r_w}}{\log_{10}\frac{R}{B}\log_{10}\left(\frac{B}{r_w}\right) + \log_{10}\left(\frac{B}{2r_w}\right) \,.\, \log_{10}\left(\frac{R}{r_w}\right)}\right] \quad ...(4.52)$$

and $$Q_2 = \left[\frac{\frac{\pi K}{2.3}(d_2 - h_w{}^2) \,.\, \log_{10}\left(\frac{B}{2r_w}\right)}{\left[2 \log_{10}\left(\frac{R}{B}\right) \,.\, \log_{10}\left(\frac{B}{r_w}\right) + \log_{10}\left(\frac{B}{2r_w}\right) \,.\, \log_{10}\left(\frac{R}{r_w}\right)\right]}\right] \quad ...(4.53)$$

Example 4.8. *Three wells, each having a diameter of 10 cm are installed at the vertices of an equilateral triangle 12 m apart in a confined aquifer. The radius of influence of each well is 400 m, and coeff. of permeability K is 20 m/day. The drawdown in each well is 2 m. The thickness of the confined aquifer is 15 m. Find the discharge of each well, and the percentage decrease in discharge because of well interference.*

Solution.

$$r_w = \frac{10}{2} \text{ cm} = 5 \text{ cm} = 0.05 \text{ m}$$

$B = 12$ m

$R = 400$ m

$K = 20$ m/day.

Drawdown in well

$$= s = (D - h_w) = 2 \text{ m}$$

$H = 15$ m

Using equation (4.47), we have the discharge of each of the three interfering artesian wells, arranged in a pattern of equilateral triangle at distances B apart, as :

$$Q_1 = Q_2 = Q_3 = \frac{2\pi KH(D - h_w)}{2.3 \log_{10}\left(\frac{R^3}{r_w \cdot B^2}\right)}$$

$$= \frac{2\pi \times 20 \times 15 \times 2}{2.3 \log_{10}\left\{\frac{400^3}{0.05 \times 12^2}\right\}} \text{ m}^3/\text{day} = 235.76 \text{ m}^3/\text{day} \quad ...(1)$$

The discharge of each such well without interference is given by equation (4.38), as

$$Q = \frac{2\pi KH \cdot (D - h_w)}{2.3 \log_{10}\left(\frac{R}{r_w}\right)} = \frac{2\pi \times 20 \times 15 \times 2}{2.3 \log_{10}\left(\frac{400}{0.05}\right)} = 419.73 \text{ m}^3/\text{day}.$$

Hence, reduction in discharge due to interference

$$= \left[\frac{419.73 - 235.76}{419.73}\right] \times 100\% = \mathbf{43.83\%. \text{ Ans.}}$$

4.20. Surface of Seepage and Free Surface Curve

The surface throughout which the pressure is atmospheric is known as *free surface.*

Let us consider an unconfined aquifer and let AD be the position of the original free water surface (*i.e.*, the watertable) which is approximately horizontal. Let the gravity well of radius r_w be constructed throughout the depth of this aquifer. Let d be the height from the bottom of the well to the ground watertable.

Let the water be pumped from this well (Fig. 4.47). After the pumping, the water will stand in the rest of the bore holes along the line $AB'C'D$ marked as "Free Surface Curve" provided the bore holes were only just deep enough to reach the free surface. Thus, in the aquifer above the surface $AB'C'D$, there is no ground water except capillary moisture.

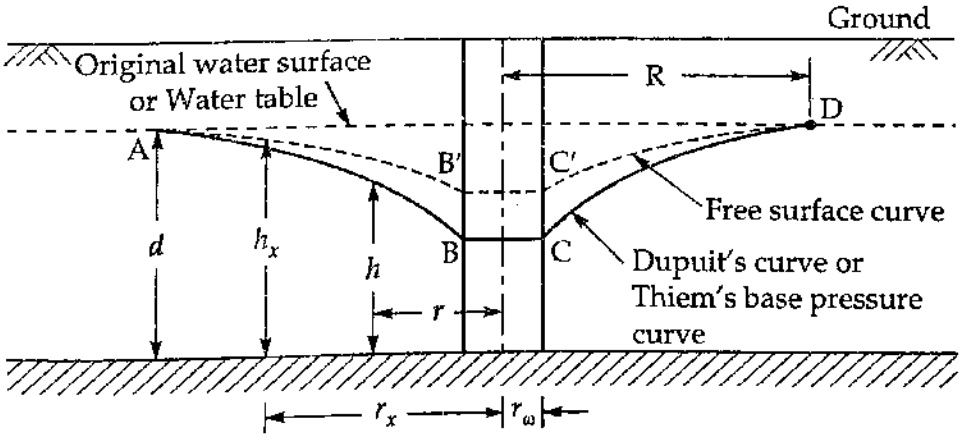


Fig. 4.47. Free surface curve Vs Dupuit's base pressure curve in a gravity well.

But according to Dupuit's formula, the water level in the pumped well will not be at $B'C'$ level but at a slightly lower than this level, *i.e.* at BC. Hence, the drawdown in the pumped well is slightly more than the drawdown of the

ground watertable immediately adjacent to the well [See Fig. 4.48]. This vertical surface of the ground ($C'C$) forming outside of the well hole, which is exposed between the surface of water in the well and the free surface is known as *surface of seepage* ; while *ABCD* will be the cone of depression or Dupuit's base pressure curve.

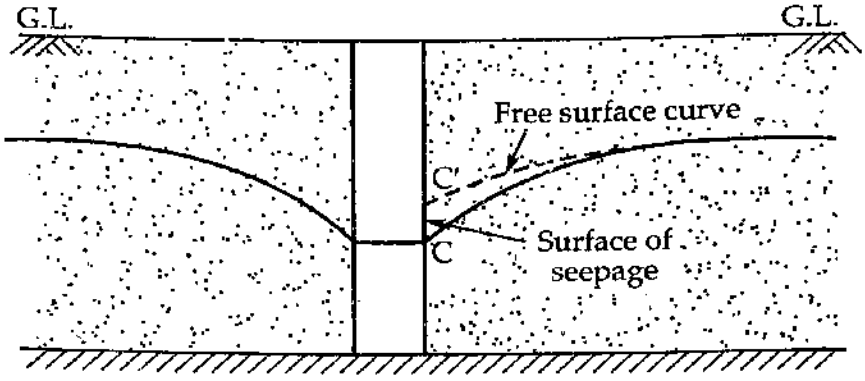
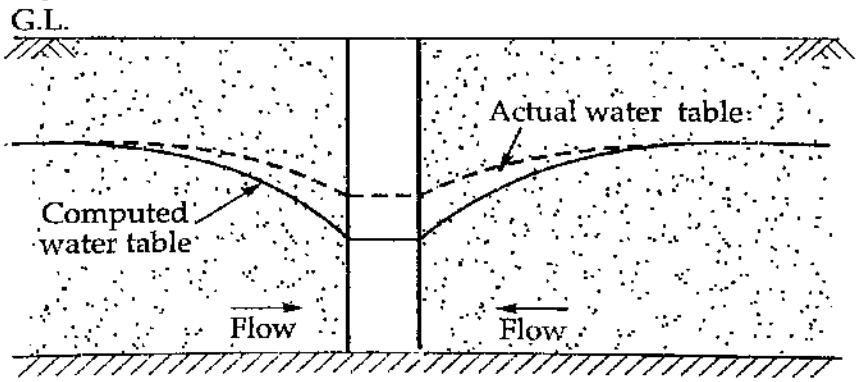


Fig. 4.48: Surface of seepage in a gravity well.

The difference between the actual free surface and the Dupuit's base pressure curve in a gravity well, arises due to the Dupuit's assumption of "horizontal and radial flow". In fact, in unconfined aquifers, the velocity distribution will not be horizontal near the well but Dupuit assumed it to be so.

The actual watertable deviates more and more from the Dupuit's computed watertable in the direction of flow, as shown in Fig. 4.49.



Actual watertable deviating more and more in the direction of flow

Fig. 4.49

The fact that the actual watertable lies above the Dupuit's *computed* watertable can be explained by the fact that the Dupuit's flows are all assumed horizontal, whereas the actual prevailing velocity of the same magnitude have a downward vertical component so that a greater saturated thickness is required for the same discharge.

This discrepancy indicates that the watertable does not follow the parabolic path (as given by Dupuit). Nevertheless, for milder slopes where the sine and tangent are nearly equal, Dupuit's curve closely predicts the watertable position except near the outflow into the well.

Equation of free surface curve. The free surface of water in the core of depression surrounding a pumped well, is the surface of water under atmospheric pressure. We must not forget that the free surface curve does not coincide with the Dupuit's base pressure curve but lies slightly above it. The curve is represented by the following equation.

$$Q = \left[\frac{\pi K}{2C}\right]\left[\frac{(d-h)d}{\log_{10}\left(\frac{R}{0.1d}\right)}\right] \qquad ...(4.54)$$

where (r, h) = any point on the curve

R = Radius of influence

K = Permeability coefficient

C = a constant depending on the value of $\frac{r}{R}$. Its value can be obtained by the curve shown in Fig. 4.50.

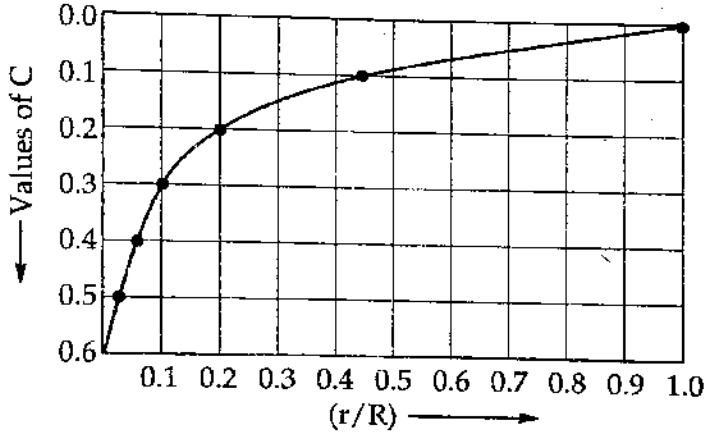


Fig. 4.50. Figure giving value of C in equation (4.54).

4.21. Well Loss and Specific Capacity of Wells

4.21.1. Well loss. When water is being pumped out of an artesian well, the drawdown caused includes not only the drawdown which is given by logarithmic drawdown curve (Equation 4.39), but also includes a certain drawdown caused by the flow of water through the well screen and its axial movement within the well up to the pump intake. The drawdown caused by the flow through the screen and axial movement within the well brings the water level in the well from AB to CD (Fig. 4.51). This vertical drawdown AC or BD is known as well loss. The magnitude of this well loss may be taken as equal to $C_2.Q^2$, since the flow in the vicinity of the well face is turbulent.

The total drawdown can then be obtained by adding the two drawdowns.

The first drawdown is obtained from Equation (4.39) ; according to which

$$Q = \frac{2\pi KH.s}{2.3\log_{10}\frac{R}{r_w}}$$

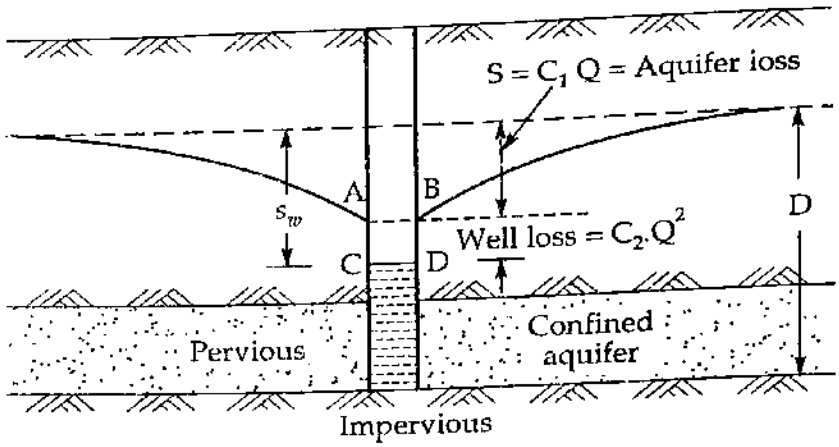


Fig. 4.51. Well loss.

or
$$s = \frac{2.3\,Q \log_{10} \frac{R}{r_w}}{2\pi KH}$$
$$= C_1.Q$$

$$\text{where } C_1 \text{ is a constant} = \frac{2.3 \log_{10} \frac{R}{r_w}}{2\pi KH}$$

The total drawdown is then given as :

$$= C_1Q + C_2.Q^2 \qquad ...(4.55)$$

where C_2Q^2 = **well loss**

and C_1Q is known as **aquifer loss.**

The magnitude of the well loss has an important bearing on the pump efficiency. Abnormally high value of the well loss indicates clogging and encrustation of the well screens, needing immediate remedial action.

The coefficients C_1 and C_2 are determined by pump test data of drawdown at various discharges.

4.21.2. Specific Capacity. The specific capacity of a well is defined as the well yield per unit of drawdown. Hence, the

$$\text{Sp. capacity} = \frac{\text{Discharge of the well}}{\text{Drawdown in the well}}$$
$$= \frac{Q}{C_1Q + C_2 \,.\, Q^2}$$

$$\therefore \quad \text{Sp. capacity} = \left[\frac{1}{C_1 + C_2Q}\right] \qquad ...(4.56)$$

The equation (4.56) clearly shows that the sp. capacity of the well is not constant but decreases as the discharge increases.

4.22. Efficiency of a Well

It has been discussed in the previous article that the discharge from a well is approximately proportional to the drawdown (s) ; since $s = C_1Q$, neglecting well loss. The discharge per unit of drawdown was called Sp. capacity of the

well. This specific capacity will be different for different well designs. For determining the best drawdown discharge conditions for a well, the well may be operated under varying drawdown conditions, and a graph may be plotted between discharge and drawdown (called yield-drawdown curve) as shown in Fig. 4.52.

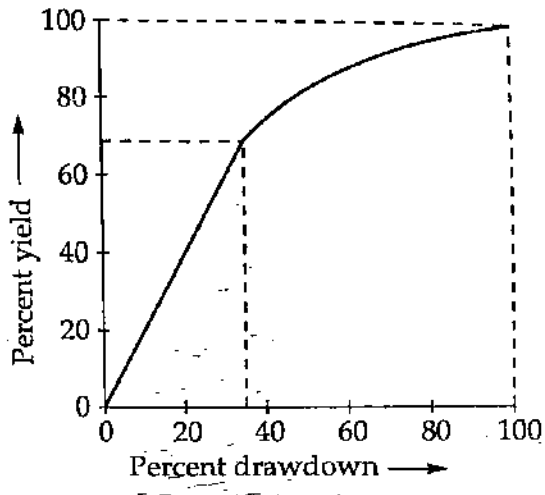


Fig. 4.52. Yield-drawdown curve.

The curve obtained is a straight line up to a certain stage of drawdown, beyond which the drawdown increases disproportionately to yield. This places an optimum and efficient limit to the drawdown which may be allowed to be created in a well. This is generally found to be 70% of the maximum drawdown which can be created in a well.

Example 4.9. *A pumping test was made in a medium sand and gravel to a depth of 15 m where a bed of clay was encountered. The normal ground water level was at surface. Observation holes were located at distances of 3 m and 7.5 m from the pumping well. At a discharge of 3.6 litres/sec. from the pumping well, a steady state was attained in about 24 hrs. The drawdown at 3 m was 1.65 m and at 7.5 m was 0.36 m. Compute the coefficient of permeability of the soil.*

Solution. $Q = 3.6$ litres/sec ; $d = 15$ m

$r_1 = 3$ m ; $s_1 = 1.65$ m ; $h_1 = 15 - 1.65 = 13.35$ m

$r_2 = 7.5$ m, $s_2 = 0.36$ m ; $h_2 = 15 - 0.36 = 14.64$ m

Using equation (4.30), we get

$$Q = \frac{\pi K(h_2^2 - h_1^2)}{2.3 \log_{10}\left(\frac{r_2}{r_1}\right)}$$

$$\therefore \quad \frac{3.6}{1000} \text{ m}^3/\text{sec.} = \frac{\pi K}{2.3}\left[\frac{(14.64)^2 - (13.35)^2}{\log_{10}\left(\frac{7.5}{3}\right)}\right]$$

$$= \frac{\pi K}{2.3}\left[\frac{27.99 \times 1.29}{0.398}\right] \text{m}^2$$

or $$K = \left[\frac{2.3 \times 0.398}{\pi \times 27.99 \times 1.29}\right] \times \frac{3.6}{1000} \times 100 \text{ cm/sec}$$

$$= 0.00289 \text{ cm/sec.} \quad \textbf{Ans.}$$

Example 4.10. *A well penetrates into an unconfined aquifer having a saturated depth of 100 metres. The discharge is 250 litres per minute at 12 metres drawdown. Assuming equilibrium flow conditions and a homogeneous aquifer, estimate the discharge at 18 metres drawdown. The distance from the*

$$h_1 = 90 - 6 = 84 \text{ m}$$
$$h_2 = 90 - 1.5 = 88.5 \text{ m}$$

(*a*) $$1.36 = \frac{\pi K[(88.5)^2 - (84)^2]}{2.3 \log_{10}\left(\frac{15}{6}\right)}$$

or $$\pi K = \frac{1.36 \times 2.3 \times 0.398}{172.5 \times 4.5} \text{ m/min} = 1.603 \times 10^{-3} \text{ m/min}$$

or $$K = \mathbf{0.51 \times 10^{-3} \text{ m/min.}} \quad \textbf{Ans.}$$

(*b*) Now $r_w = 0.3$ m

$r_2 = 15$ m

$h_2 = 88.5$ m

$h_w = ?$

Using $$Q = \frac{\pi K\,(d^2 - h_w^2)}{2.3 \log_{10}\left(\frac{r_2}{r_w}\right)}$$

we get $$Q = \frac{\pi K(88.5^2 - h_w^2)}{2.3 \log_{10}\left(\frac{15}{0.3}\right)}$$

But $Q = 1.36$ m^3/min

and $\pi K = 1.603 \times 10^{-3}$ m/min (calculated above)

$\therefore$ $$1.36 = \frac{1.603 \times 10^{-3}(88.5^2 - h_w^2)}{2.3 \log_{10}(50)}$$

or $$\frac{1.36 \times 2.3 \times 1.69}{1.603 \times 10^{-3}} = (88.5)^2 - h_w^2$$

or $$3{,}290 = 7{,}820 - h_w^2$$

or $$h_w^2 = 7{,}820 - 3{,}290 = 4{,}530$$

or $$h_w = 67.4 \text{ m.}$$

$\therefore$ Drawdown in the pumped well

$$= 90 - 67.4 = \mathbf{22.6 \text{ m.}} \quad \textbf{Ans.}$$

(*c*) *Specific capacity of the well.* It is the discharge for a unit (*i.e.* 1 m) drawdown in the pumped well.

Let us first find out the value of R

$$1.36 = Q = \frac{\pi K\,(d^2 - h_w^2)}{2.31 \log_{10}\left(\frac{R}{r_w}\right)} = \frac{\pi K\,(90^2 - 67.4^2)}{2.3 \log_{10}\left(\frac{R}{0.3}\right)}$$

or $$1.36 = \frac{1.603 \times 10^{-3} \times 157.4 \times 22.6}{2.3 \log_{10}\left(\frac{R}{0.3}\right)}$$

or $$\log_{10}\left(\frac{R}{0.3}\right) = \frac{1.603 \times 157.4 \times 22.6}{2.3 \times 1{,}360} = \mathbf{1.824.}$$

Taking antilog, we get

$$\frac{R}{0.3} = 66.7$$

or $$R = 20.01$$

Say $$R = 20 \text{ m}$$

Now

$$\text{Specific capacity} = Q_{\text{unit drawdown}} = \frac{\pi K(90^2 - 89^2)}{2.3 \log_{10}\left(\frac{20}{0.3}\right)} = \frac{1.603 \times 10^{-3} \times 179 \times 1}{2.3 \times 1.824}$$

$$= 68.3 \times 10^{-3} \text{ m}^3/\text{min.}$$

Hence, specific capacity

$$= \mathbf{68.3 \text{ litres/minute.}} \quad \textbf{Ans.}$$

(*d*) *Maximum discharge* will occur when

$$h_w = 0$$

$$\therefore \quad Q_{\max} = \frac{\pi K(90^2 - 89^2)}{2.3 \log_{10}\left(\frac{20}{0.3}\right)} = \frac{1.603 \times 10^{-3} \times 8100}{2.3 \times 1.824} = 3.09 \text{ m}^3/\text{min.}$$

Hence, maximum rate of discharge

$$= \mathbf{3{,}090 \text{ litres/minute.}} \quad \textbf{Ans.}$$

4.23. Non-Equilibrium Formula for Confined Aquifers (Unsteady Radial Flows)

The main drawback of the equilibrium formulas given by Thiem and Dupuit, was the problem to attain equilibrium conditions, which is not an easy job to achieve. The pumping has to be continued at a uniform rate for a very long time, so as to achieve steady flow conditions. Further research was, therefore, carried out to simplify the process and to calculate the yield in some other way.

A major advancement in this field was made by Thies, when he developed his non-equilibrium formula by introducing the time factor t. This formula was derived by Thies in 1935, by comparing the flow of water with the flow of heat by conduction. The formula evolved by him involved the solution of a complicated integral, the solution of which requires the use of various tables and graphs.

Later, Jacob derived the same formula by directly using the hydraulic concept. He also slightly modified the Thies's formula by making a slight approximation, so as to simplify it further. The final modified formula which was arrived at the end, by Jacob, is given as :

$$\boxed{s = \frac{Q}{4\pi T}\left[\log_e \frac{4Tt}{r^2 . A} - 0.5772\right]} \qquad \ldots(4.57)$$

where s = Drawdown in the observation well after a time t

T = Coefficient of transmissibility

Q = Constant discharge pumped out from the well

A = Coefficient of storage of measured drawdown

r = Radial distance of the observation well from the main pumped well

The derivation of this formula is given in "*Water Resources Engg. Vol. I—Hydrology and Water Resources Engineering*" by the same author, and may be referred to in specific needs.

This is the important final equation and must be remembered. If, in an observation well at a distance r, the drawdowns are respectively s_1 and s_2 at times t_1 and t_2 after the pumping was started in the main well, then

$$s_1 = \frac{Q}{4\pi T}\left[\log_e \frac{4Tt_1}{r^2 . A} - 0.5772\right] \quad ...(A)$$

and

$$s_2 = \frac{Q}{4\pi T}\left[\log_e \frac{4Tt_2}{r^2 A} - 0.5772\right] \quad ...(B)$$

Subtracting (A) from (B), we get

$$s_2 - s_1 = \frac{Q}{4\pi T}\left[\log_e \frac{4Tt_2}{r^2 A} - \log_e \frac{4Tt_1}{r^2 A}\right]$$

$$= \frac{Q}{4\pi T}\left[\log_e \frac{4Tt_2}{r^2 A} \cdot \frac{r^2 A}{4Tt_1}\right]$$

$$= \frac{Q}{4\pi T} \log_e \frac{t_2}{t_1}$$

$$= \frac{2.3Q}{4\pi T} \log_{10} \frac{t_2}{t_1}$$

or

$$\boxed{s_2 - s_1 = \frac{2.3Q}{4\pi T} . \log_{10} \frac{t_2}{t_1}} \quad ...(4.58)$$

The above formula holds good for larger values of t. It is evident from the above equation that if the drawdown is noted for various values of t on the given observation well at a distance r from the main pumped well, and a graph is plotted between log t and s, it will be a straight line (Fig. 4.53), with the limitation that the initial values (*i.e.* when t is small) may not exactly lie on a straight line. From this straight line, two values of t_1 and t_2 and corresponding s_1 and s_2 can be read out, and knowing the value of T, Q can be worked out easily.

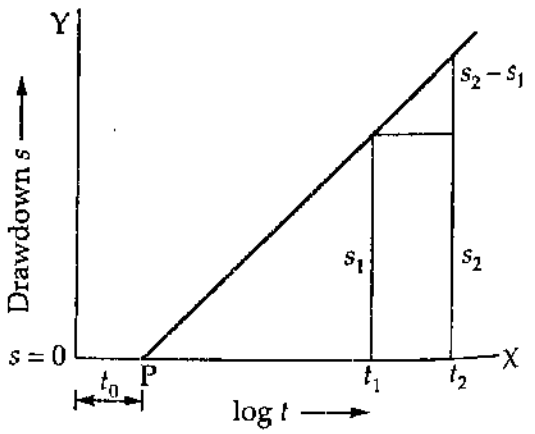


Fig. 4.53

The straight line can be produced so as to cut the X-axis at the point P. Let the value of t at P is represented by t_0.

Now, we know that

$$s = \frac{Q}{4\pi T}\left[\log_e \frac{4Tt}{r^2 . A} - 0.5772\right]$$

at $s = 0, t = t_0$

$$\therefore \quad 0 = \frac{Q}{4\pi T}\left[\log_e \frac{4Tt_0}{r^2 A} - 0.5772\right]$$

Since $\frac{Q}{4\pi T} \neq 0$

$$\log \frac{4Tt_0}{r^2 A} - 0.5772 = 0$$

or $$\log_e \frac{4Tt_0}{r^2 A} = 0.5772$$

or $$\frac{4Tt_0}{r^2 . A} = e^{0.5772}$$

or $$A = \frac{4Tt_0}{r^2 e^{0.5772}} = \frac{2.25Tt_0}{r^2}$$

Hence, $$A = \frac{2.25Tt_0}{r^2} \qquad ...(4.59)$$

The coefficient of storage A, can be worked out in this manner.

Example 4.13. *In an artesian aquifer, the drawdown is 1.2 metres at a radial distance of 10 metres from a pumped well after two hours of pumping. On the basis of Thies' non-equilibrium equation, determine its pumping time for the same drawdown (i.e. 1.2 m) at a radial distance of 30 metres from this well.*

Solution. The Thies' non-equilibrium equation is

$$s = \frac{Q}{4\pi T}\left[\log_e \frac{4Tt}{r^2 . A} - 0.5772\right] \qquad ...(i.e.\ Eq.\ 4.57)$$

In the given equation, the drawdown is the same in both the observation wells, therefore, $s_1 = s_2 = 1.2$ m.

well (1)	well (2)
$r_1 = 10$ m.	$r_2 = 30$ m.
$s_1 = 1.2$ m	$s_2 = 1.2$ m
$t_1 = 2$ hrs.	$t_2 = ?$

Now $$s_1 = \frac{Q}{4\pi T}\left[\log_e \frac{4Tt_1}{r_1^2 A} - 0.5772\right]$$

$$s_2 = \frac{Q}{4\pi T}\left[\log_e \frac{4Tt_2}{r_2^2 A} - 0.5772\right]$$

But $$s_1 = s_2$$

$$\therefore \quad \frac{Q}{4\pi T}\left[\log_e \frac{4Tt_1}{r_1^2 A} - 0.5772\right] = \frac{Q}{4\pi T}\left[\log_e \frac{4Tt_2}{r_2^2 A} - 0.5772\right]$$

or $$\log_e \frac{4Tt_1}{r_1^2 A} = \log_e \frac{4Tt_2}{r_2^2 A}$$

or $$\frac{4Tt_1}{r_1^2 A} = \frac{4Tt_2}{r_2^2 A}$$

or $$\frac{t_1}{r_1^2} = \frac{t_2}{r_2^2}$$

Putting the respective values, we get

or $$\frac{2 \text{ hrs.}}{(10)^2} = \frac{t_2}{(30)^2}$$

or $$t_2 = \frac{(30)^2}{(10)^2} \times 2 \text{ hrs.} = 9 \times 2 \text{ hrs.} = 18 \text{ hrs.}$$

or $$\mathbf{t_2 = 18 \text{ hrs.}} \quad \textbf{Ans.}$$

4.24. Design of a Strainer Tubewell

The design of a strainer type tubewell essentially consists of designing the size of the tube, the size of the bore hole, the length of the strainers, and the type and horse power of the pumping arrangement required to lift the water. The design considerations for these parts of a tubewell are discussed below :

(*i*) **Size of the Tube.** The diameter of the tube pipe is decided from the considerations of permissible flow velocity through the tube. Since the strainers are installed at different levels, the velocity of water in a tube of a fixed size will not be constant but will be increasing towards the top. Hence, it is theoretically possible to reduce the size of the tube from top towards the bottom, such that the velocity is more or less constant throughout the tube length. However, it is generally not economical to install varying size tube and hence, a single sized or at the most, a two sized tube (in case the aquifers are available at too much different depths) may be used in actual practice. The velocity through the tube may be limited between 1.5 m/sec to 4.5 m/sec. Knowing the design discharge of the tubewell and the velocity, the area of the tube, and hence its diameter can be easily calculated. The nearest available size in the market may then be used.

(*iii*) **Size of the Bore Hole.** Normally, the size of the bore hole should be at least 5 cm bigger than the size of the tube, so as to facilitate the lowering of the tube in the hole. The bore size should, however, be increased suitably, if a gravel pack is to be provided around the screen.

(*iv*) **Length of the Strainer.** After deciding the diameter of the tube, the length of the strainer required to obtain the design discharge may be calculated for unconfined or confined aquifer, as the case may be, by using Eq. (4.36) or (4.39) as given below :

(*a*) *For Unconfined Aquifer Case*, using Eq. (4.36), we have

$$Q = \frac{\pi K(d^2 - h_w{}^2)}{2.3 \log_{10} \frac{R}{r_w}} = \frac{\pi K(d - h_w)(d + h_w)}{2.3 \log_{10} \frac{R}{r_w}}$$

If s is the drawdown or the depression head, then w.r.t. Fig. 4.38 we can easily write

$$d - h_w = s$$

and $$d = h_w + s$$

$$\therefore \quad Q = \frac{\pi K s(h_w + s + h_w)}{2.3 \log_{10} \frac{R}{r_w}} = \frac{2\pi K s\left(h_w + \frac{s}{2}\right)}{2.3 \log_{10} \frac{R}{r_w}}$$

or $$\left(h_w + \frac{s}{2}\right) = \frac{2.3\, Q \log_{10} \frac{R}{r_w}}{2\pi K s}$$

or $$h_w = \left[\frac{2.3\, Q \log_{10} \frac{R}{r_w}}{2\pi K.s} - \frac{s}{2}\right] \qquad ...(4.60)$$

h_w in this case, represents nothing but the length of the strainer needed. In the above equation, r_w is the radius of the well and is known by now. The value of radius of influence (R) may be assumed between 250 and 500 m. Such a wide variation in the value of R will, at the most change the discharge up to a maximum of 12%, since the relation between Q and R is logarithmic. The suitable value of permeability coefficient (K) may be assumed. The design discharge and depression head (*i.e.* drawdown) are also known ; hence, the value of h_w *i.e.*, the length of the strainer can be easily calculated.

(*b*) *For Confined Aquifer Case*, using Eq. (4.39), we have

$$Q = \frac{2\pi KH.s}{2.3 \log_{10} \frac{R}{r_w}}$$

or $$H = \frac{2.3\, Q \log_{10} \frac{R}{r_w}}{2\pi K.s}$$

In this equation, H represents the length of the strainer, which can be easily calculated. This designed strainer length is provided in one or more aquifers depending upon the site availability.

(*iv*) **Type of Pumping Arrangement.** Three types of pumps are generally used to lift water from wells. They are :

(1) *Ordinary centrifugal pumps* ;

(2) *Jet centrifugal pumps ;* and

(3) *Bore hole type pumps.*

All these different types of pumps are described in details in chapter 7. However, they have been touched here in order to give some idea of their use in the extraction of ground water and to complete the design of the tubewells.

(1) **Ordinary Centrifugal pumps.** An ordinary centrifugal pump, popularly called a **monoblock pump,** lifts water from the lower level to a higher level by creating the required pressure with the help of centrifugal action.

Such pumps are available in two models ; in one model the pump and the motor are built into one body ; and in the other, the pump and the motor are separate bodies, though coupled together through a visible shaft.

The maximum suction head, under which this pump can practically work effectively, is about 6 to 8 m. Hence, such a pump can be used only at places, where the fluctuation in water-table plus the depression head is limited to a maximum of 8 m or so. For larger values of suction heads, the jet pumps and bore hole type pumps are used.

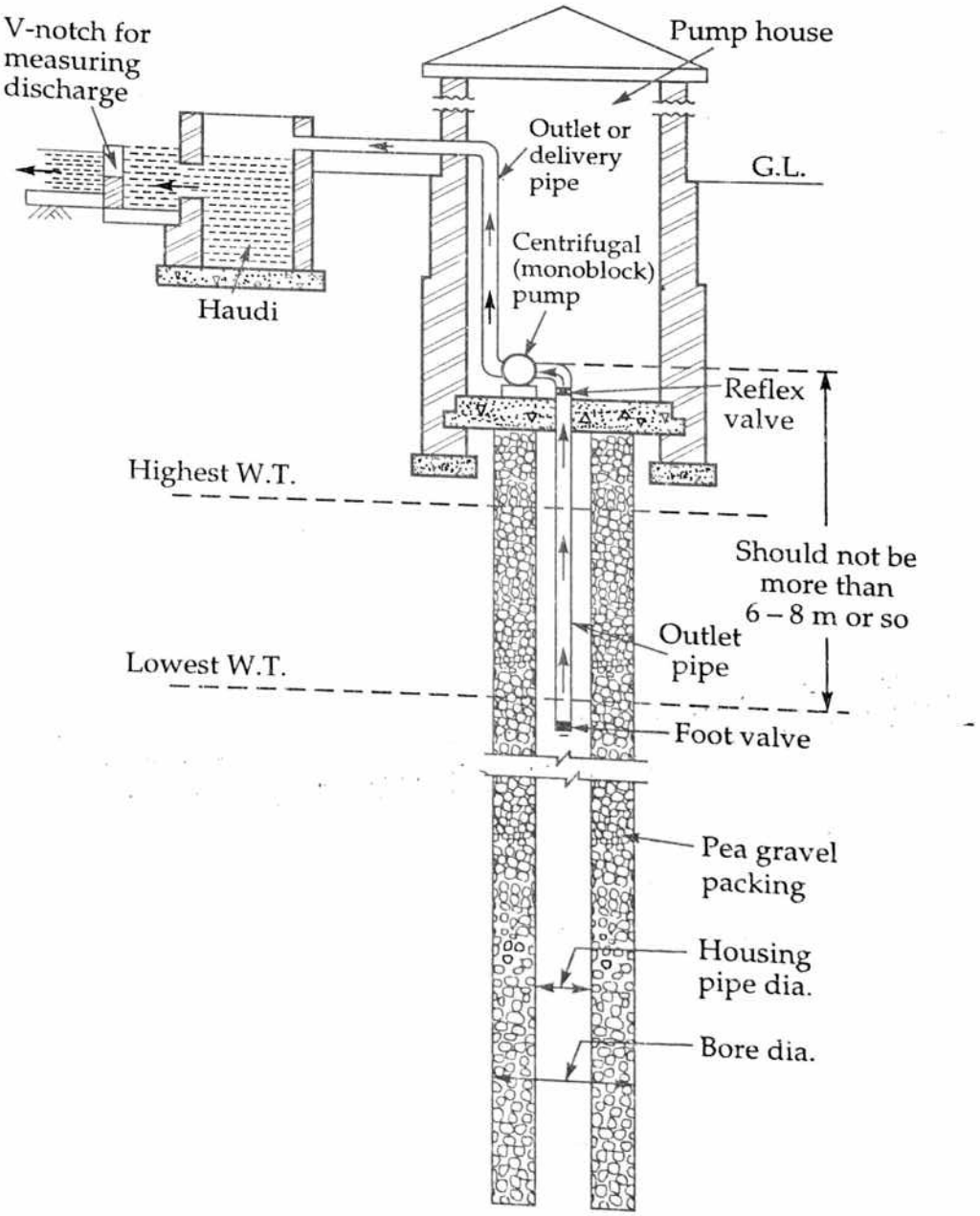


Fig. 4.54. Tubewell assembly using an ordinary monoblock centrifugal pump.

A section of a tubewell using a monoblock pump is shown in Fig. 4.54. In this arrangement, a sump well is usually constructed so as to place the pump at a required lower level. The pump is to be placed slightly above the highest water table level, so as to avoid its submergence. The minimum water table level should not be lower than the pump level by more than 6 to 8 m as shown, otherwise the pump will not work.

Hence, such an arrangement can be used only where water is available at smaller depths from the surface (*i.e.* generally for shallow cavity tubewells) and it cannot be used for deep tubewells, where the water may be available quite deep.

The sump well is also generally of a large size and deep enough, and has to be plugged at the bottom with concrete. This arrangement may sometimes make it costly, though this type of pump is the cheapest. Hence, such type of pumping arrangement is not preferred in modern days, when the bore hole type pumps are increasingly used.

(2) **Jet Pumps.** Jet centrifugal pumps find extensive use in individual domestic tubewells, and are very suitable for lifting water in ordinary three storeyed houses in areas where water table level is within about 30 m or so from the ground surface. **Single stage jet pumps** are designed for lifting water with suction lift of the order of 9 to 36 m ; while the **twin stage jet pumps** are designed for suction lift of the order of 24 to 54 m, as shown in table 4.7.

Such pumps are compact machines, like monoblock centrifugal pumps, and are placed at the ground level. The bore using a jet pump shall, however, accommodate two pipes ; *i.e.* (*i*) the normal suction pipe ; and (*ii*) a pressure pipe connected to the delivery side of the pump at one end, and to the suction pipe through the jet assembly at the other end near the foot valve inside the bore, as shown in Fig. 4.55.

The specifications of different types of jet pumps being manufactured by Crompton Greaves in India are shown in table 4.7.

(3) **Bore hole type pumps.** Such pumps consist of special centrifugal pump impellers connected in series, mounted on a vertical shaft and driven by a motor. They are of small diameter and can be lowered in the casing pipe of 3 to 4″ diameter (4″ dia. bores are more common). Sometimes, when ground water levels are low, the upper 20 to 50 m portion of the bore, to be provided with PVC blind pipe, is kept wider (say of 5″-6″ dia.) than the rest bottom portion of the bore (say of 4″ dia.), where the slotted M.S. pipe is to be laid. This will help in drilling the lower portion of the bore through the rocky lower strata.

The lower portion of the casing pipe is either provided with slots, or is provided with a strainer of suitable length, to be placed in the aquifer depth(s). When fine sand or mica, layers are found to be existing in the aquifer layer(s), which are likely to be withdrawn with water in the bore, then gravel pack layer is provided around the screen or the slotted pipe. The pump is attached to the bottom of the tube G.I. pipe (*i.e.* delivery pipe), which is usually of 32 mm to 50 mm dia. size, depending upon the horse power of the pump used. Two types of bore hole pumps ; *i.e.* (*i*) *Submergible pumps* ; and (*ii*) *Turbine type* pumps are available, though, now-a-days, the submergible pumps are being invariably used.

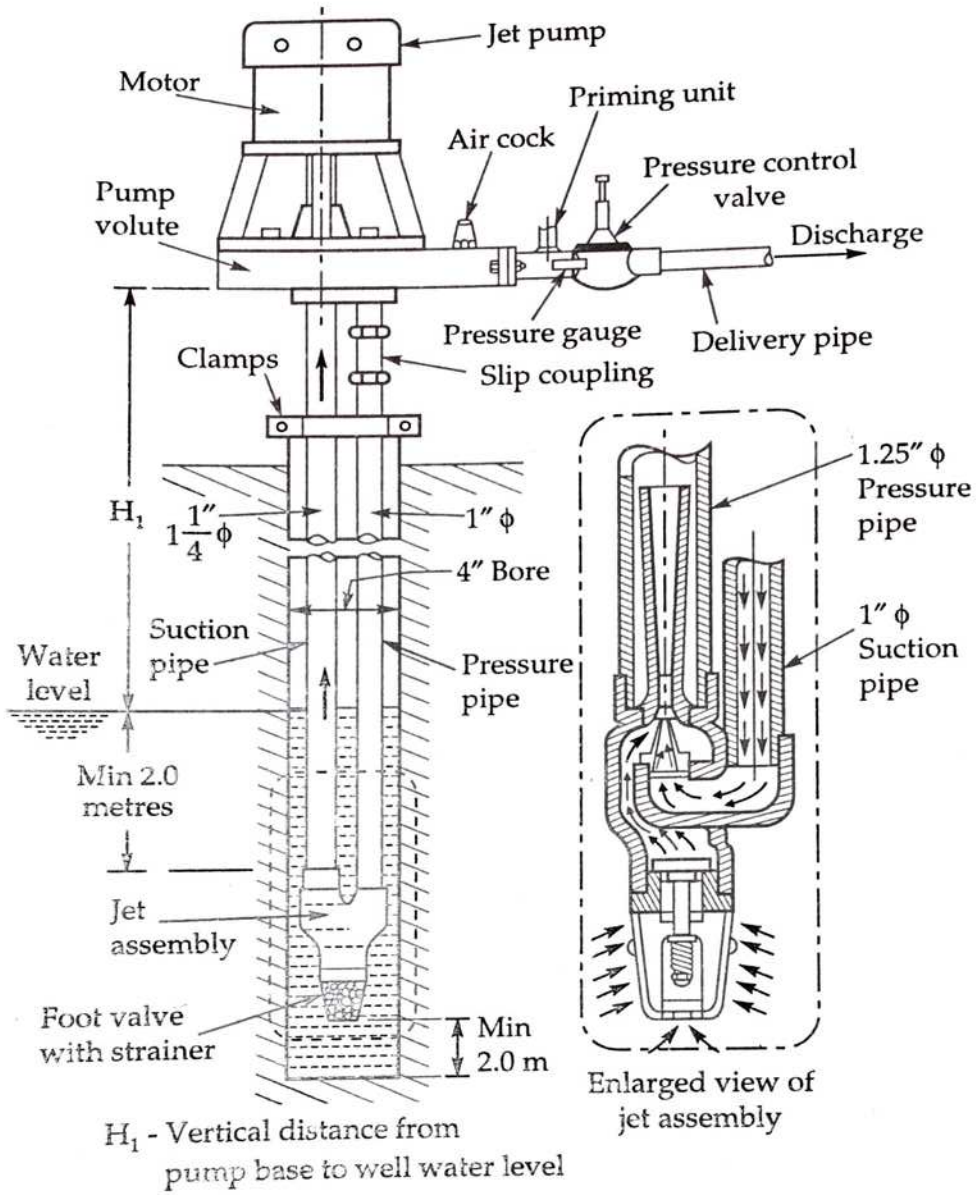


Fig. 4.55. Jet pump installation.

In **submergible pumps**, the motor and the pump are attached together, and lowered inside the bore. The pump is attached above the motor vertically, as shown in Fig. 4.56.

The pump motor assembly will have to be lowered into the bore by means of hoist/chain pulley block and supporting clamps. The use of submergible pumps for lifting tubewell waters become necessary, when ground water level goes down, say below 25 m to 30 m from the ground level, as to make the functioning of jet pumps impracticable, since jet pumps usually work effectively for static suction lifts of such magnitudes only. The submergible pumps may however, be used for total head (suction head + delivery head + Losses in pipes, bends, gate valves, etc.) of the order of 25 m to 120 m, or even more. Usually, 1 to 1.5 HP pumps are quite sufficient for individual homes, while higher horse power pumps (upto 5 HP) are available for community tubewells or for tubewells installed in apartment blocks. The specifications of

Table 4.7. Specifications of a Single Stage and Twin Stage Jet Pumps being manufactured by Crompton Greaves Ltd. in India

Rating Ref.	*KW (HP)*	*Minimum clear bore dia- mm (inch)*	*Pipe connections in mm (inch)*			*Delivery head in metres (Feet)*	*Capacity in LPH (GPH) at various suction lift (i/c static suction lift + friction had loss + depression head) in metres (feet)*									
			Suction	*Pressure*	*Delivery*		*9 (30)*	*12 (40)*	*15 (50)*	*18 (60)*	*21 (70)*	*24 (80)*	*27 (90)*	*30 (100)*	*33 (110)*	*36 (120)*
JCTH052/ JHO52 (1 Ph)	0.37 (0.5)	100 (4)	32 ($1\frac{1}{4}$)	25 (1)	20 (3/4)	14 (46)	1840 (405)	1670 (367)	1490 (328)	1300 (286)	1110 (242)	920 (202)				
JCTM12/ JM12 (1 Ph)	0.75 (1.0)	100 (4)	32 ($1\frac{1}{4}$)	25 (1)	25 (1)	18 (59)	2200 (485)	2150 (474)	1980 (438)	1750 (386)	1600 (352)	1380 (304)	1150 (253)	950 (209)	780 (172)	
JCTH12/ JH12 (1 Ph)	0.75 (1.0)	115 ($4\frac{1}{2}$)	40 ($1\frac{1}{2}$)	32 ($1\frac{1}{4}$)	25 (1)	18 (59)	2400 (528)	2350 (517)	2220 (489)	1990 (438)	1730 (387)	1540 (339)	1320 (290)	1110 (244)	850 (187)	650 (143)
JL12 (1 Ph)	0.75 (1.0)	80 (3)	25 (1)	25 (1)	25 (1)	21 (69)	1640 (361)	1600 (352)	1540 (339)	1400 (308)	1250 (275)	1100 (242)	950 (209)	800 (176)		

Table 4.7 (Contd.)

Rating Ref.	*KW (HP)*	*Minimum clear bore dia- mm (inch)*	*Pipe connections in mm (inch)*			*Delivery head in metres (Feet)*	*Capacity in LPH (GPH) at various suction lift (i/c static suction lift + friction had loss + depression head) in metres (feet)*									
			Suction	*Pressure*	*Delivery*		*24 (80)*	*30 (100)*	*36 (120)*	*42 (140)*	*48 (160)*	*54 (180)*				
JM1.52	1.1	100	32	25	25	14	1780	1600	1380	1200	1000	800				
(1 Ph)	(1.5)	(4)	(1 1/4)	(1)	(1)	(46)	(392)	(352)	(304)	(264)	(220)	(176)				
JH1.52	1.1	115	40	32	25	14	2000	1800	1600	1400	1190	980				
(1 Ph)	(1.5)	(4 1/2)	(1 1/2)	(1 1/4)	(1)	(46)	(440)	(396)	(352)	(308)	(262)	(216)				
JM	1.5	100	32	25	25	18	2120	1980	1780	1580	1350	1120				
(1 Ph)	(2.0)	(4)	(1 1/4)	(1)	(1)	(59)	(467)	(436)	(392)	(348)	(297)	(246)				
JH22	1.5	115	40	32	25	18	2350	2000	2000	1760	1530	1300				
(1 Ph)	(2.0)	(4 1/2)	(1 1/2)	(1 1/4)	(1)	(59)	(517)	(485)	(440)	(388)	(337)	(286)				
JH32	2.2	125	50	40	32	30		3040	2775	2520	2260	2000				
(3 Ph)	(3.0)	(5)	(2)	(1 1/4)	(1 1/4)	(100)		(669)	(610)	(554)	(497)	(440)				

Note 1. The above performance is for jet submergence of min 2 metres from water level.

Note 2. Delivery head pressure indicated above is minimum pressure required for successful operation of jet.

Note 3. Delivery Head = Static Delivery Head + Friction and Other Losses in Delivery System.

Table 4.8. Specifications of 100 mm Submergible Pump Sets being Manufactured by Crompton and Greaves Ltd. in India.

Rating Ref.	*No. of Stage*	*Motor HP*	*Delivery Outlet size in mm*	*Discharge (L.P.M.)*															
				10	*12*	*20*	*30*	*40*	*50*	*60*	*70*	*80*	*90*	*100*	*110*	*120*	*130*	*140*	*150*
				Total Head (Metres)															
E8K0.5	8	0.5	32		45	40	33	26											
E12K0.5	12	0.5	32		60	51	41	28											
E8H0.75	8	0.75	32				40	35	34	30	26	21							
E8C1	8	1.0	40						40	39	36	35							
E13B1	13	1.0	32			53	51	47	43	38	30	22	11						
E9H1.1	9	1.1	32				45	41	38	34	29	24							
E14K1.1	14	1.1	32	68		57	43	25											
E15G1.1	15	1.1	32	69		63	57	51	40	36	27	16							
E15K1.1	15	1.1	32		78	67	53	38											
E10C1.5	10	1.5	40						50	49	45	43							
E12C1.5	12	1.5	40						60	58	54	51	48	45	41	35	30	25	17
E16B1.5	16	1.5	32			65	62	58	53	47	36	26	14						
E20B1.5	20	1.5	32			82	78	72	66	58	45	33	17						
E24G1.5	24	1.5	32	110		101	91	82	70	58	43	26							

Rating Ref.	No. of Stage	Motor HP	Delivery Outlet size in mm	Discharge (L.P.M.)																
				10	12	20	30	40	50	60	70	80	90	100	110	120	130	140	150	
				Total Head (Metres)																
E28G1.5	28	1.5	32	129		118	106	95	81	67	50	31								
A14C2	14	2.0	50						70	68	63	60	56	52	48	41	35	29	20	
A16C2	16	2.0	50						80	78	72	68	64	60	55	55	47	33	23	
				Head (Metres) / Discharge (L.P.M.)																
A18D3	18	3	40	105/50		101/60	94/80		90/90		84/100		75/110	65/120		52/130		35/140		20/150
A15E3	15	3	50	83/50		79/60	65/100		59/120		52/140		44/160	36/180		29/200		22/220		14/240
A9F3	9	3	50	38/150		36/180	33/210		30/240		26/270		21/300	16/330		10/360				
A39D5TP	30	5	40	175/50		168/60	157/80		150/90		140/100		125/110	108/120		87/130		58/140		5/150
A25E5TP	25	5	50	138/50		132/60	120/80		108/100		87/140		73/160	60/180		48/200		37/220		23/240
A14F5TP	14	5	50	59/150		56/180	51/210		47/240		40/270		33/300	25/330		16/360				

the available submergible pump models being manufactured by Crompton and Greaves Ltd. in India are shown in table 4.8. Depending upon ones requirements of head and discharge, one particular model may be chosen.

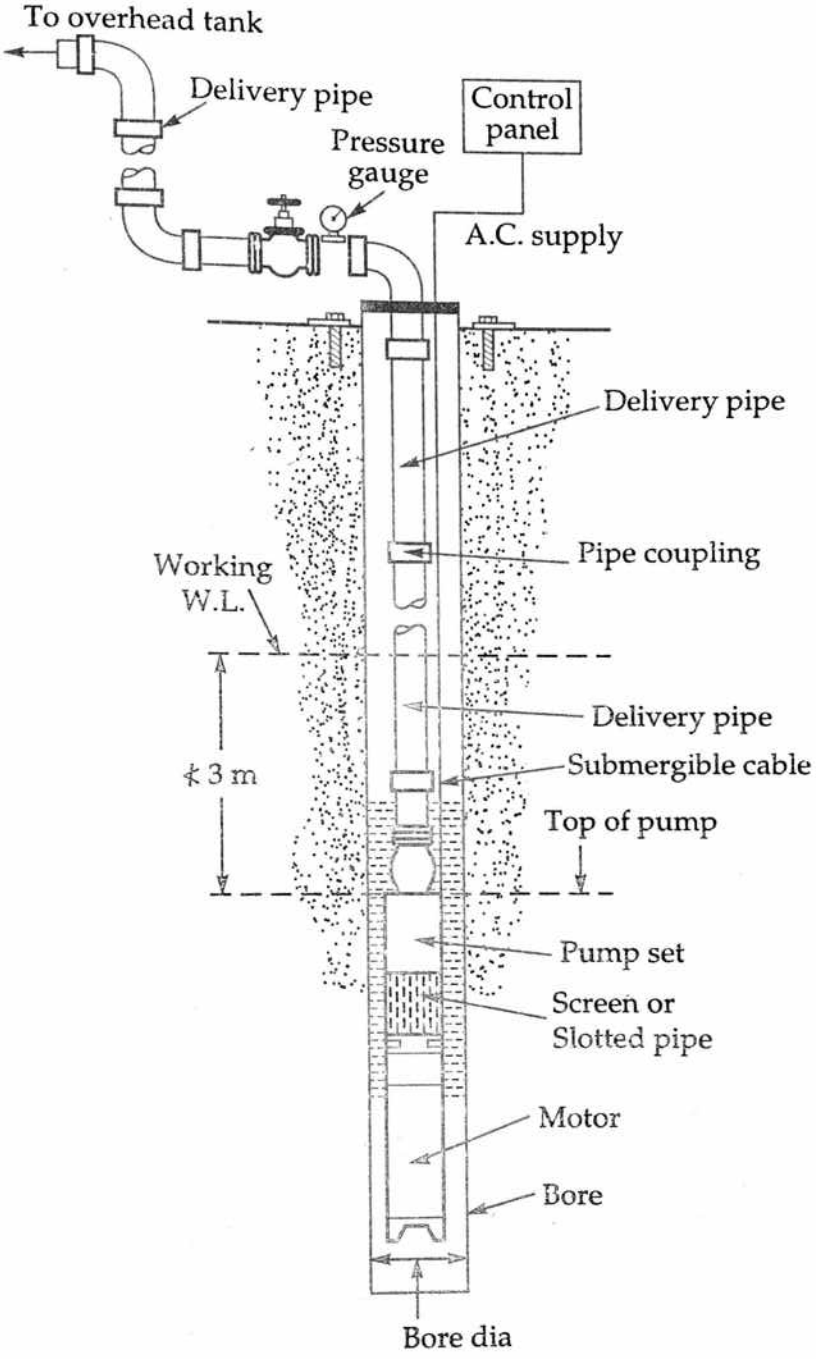


Fig. 4.56. Submergible pump installation in the bore.

In **turbine type pump**, only the pump is attached to the bottom of the delivery pipe and lowered into the bore, while the motor is attached at the top of the delivery pipe and is placed at the ground level, as shown in Fig. 4.57. Such pumps are costlier than the submergible ones, and are rarely used, and that too, in State tube wells.

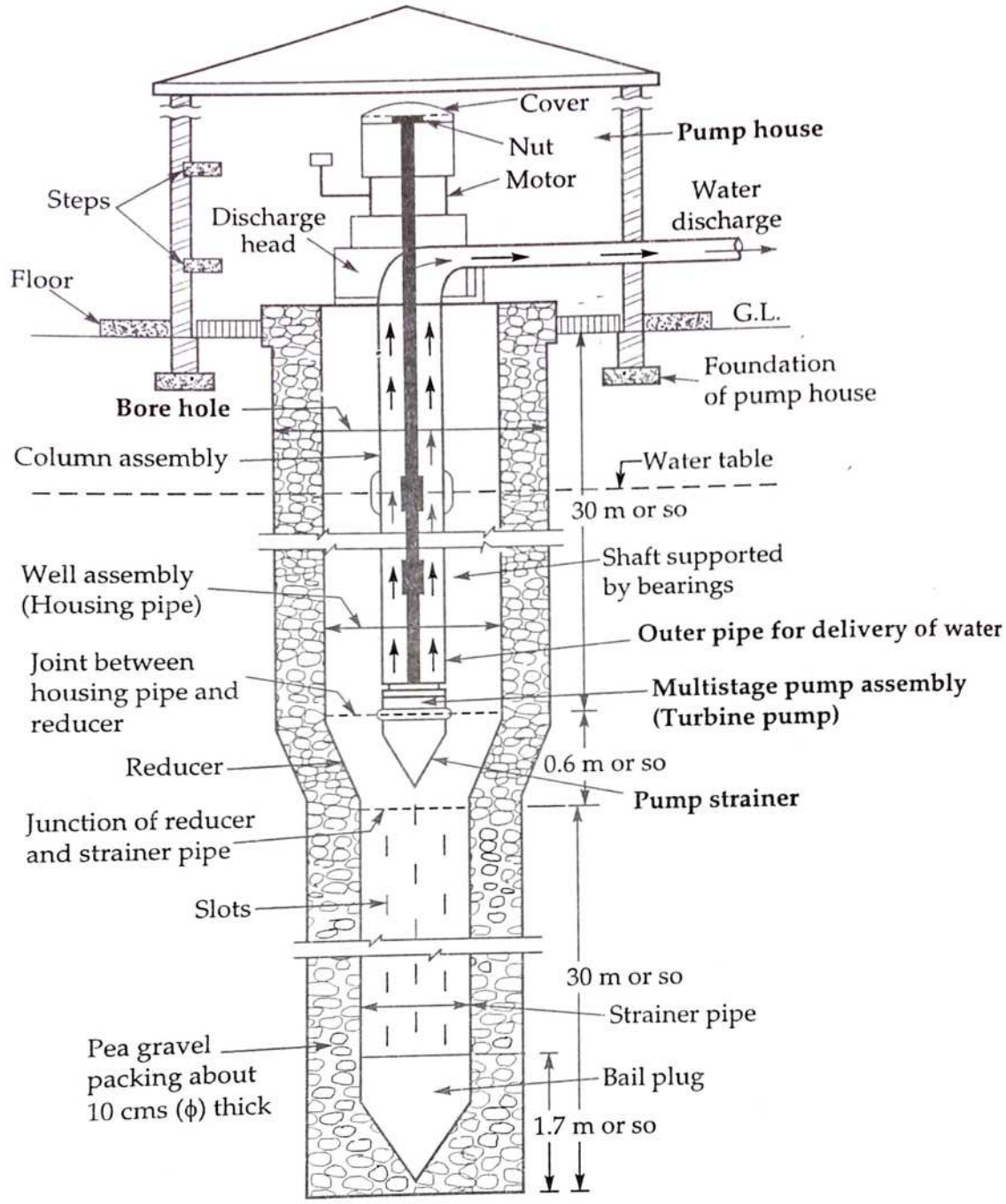


Fig. 4.57. Tubewell assembly using a turbine type of bore hole pump.

Photographic views of all types of pumps are shown in Fig. 4.58.

(*v*) **Horse power of the pump motor.** The power of a motor is given by the equation

$$\text{Power} = \frac{wQ.H.}{\eta} \text{ kW, where 1 HP} = 0.735 \text{ kW} \qquad \ldots(4.61)$$

$$\therefore \qquad \text{HP} = \frac{wQH}{0.735\eta} \qquad \ldots(4.62)$$

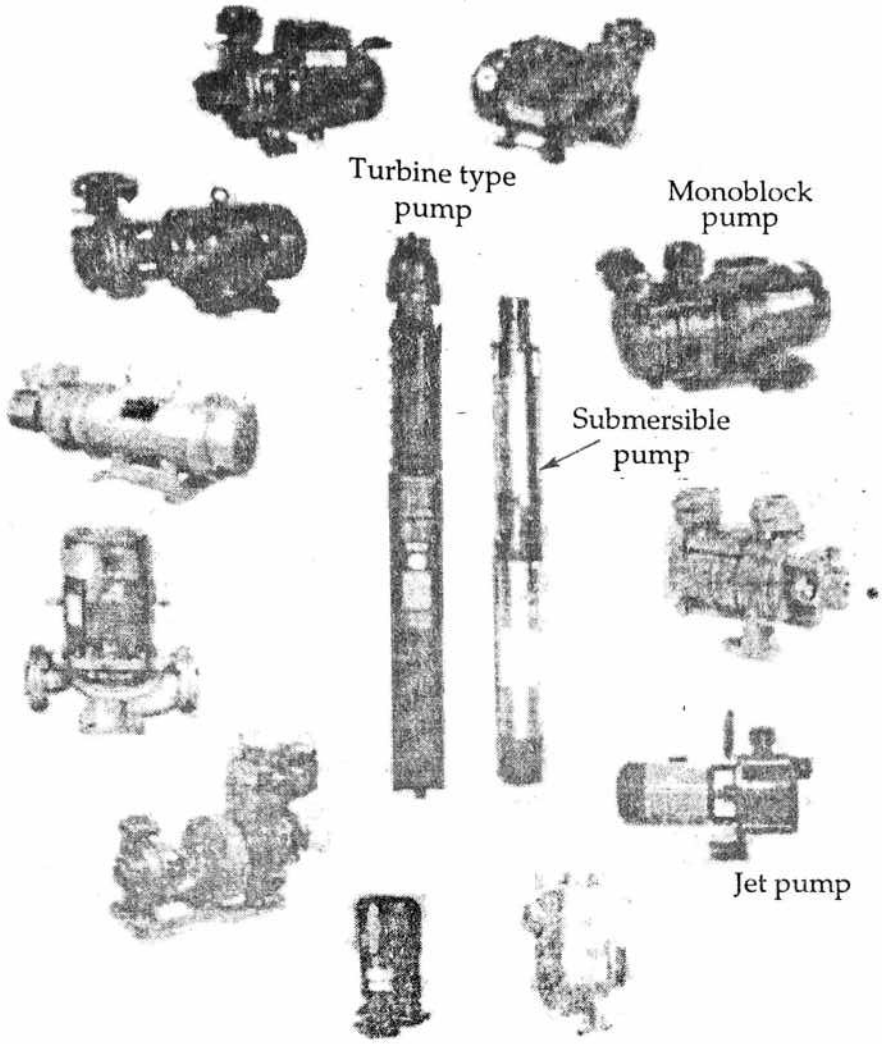


Fig. 4.58. Photo views of various types of pumps.

where Q = Discharge to be delivered in m^3/s

H = Total lift, *i.e.* the head against which the motor has to work, in m

w = Unit weight of water in $kN/m^3 = 9.81\ kN/m^3$

η = Efficiency of the pump set, and is generally taken as 65% (*i.e.* 0.65)

The total head (H) against which the motor has to work, consist of :

(*a*) Maximum depth of general water level below the ground.

(*b*) Maximum depression head.

(*c*) Velocity head.

(*d*) Frictional losses in the tube pipe including the losses at bends and strainer openings.

The frictional loss in the pipe (h_f) may be calculated by using the equation

$$h_f = \frac{f' l . V^2}{2gd} \qquad ...(4.63)$$

where f' = Coefficiency of friction* which is generally taken to be about 0.024

l = Total length of pipe line including the vertical as well as horizontal length

V = Velocity of flow in the pipe

d = Diameter of the pipe line.

The losses at the bends and those at the entry of the strainer openings are generally taken to be equal to 25% to 30% of the frictional losses in the pipe.

Knowing the total value of H *i.e.*, the head against which the motor has to work, the horse power of the motor can be easily calculated.

Example 4.14. *Design a tubewell to deliver 33,000 gallons per hour at a depression head of 5 m. The average water level is 10 m below the ground in October and 15 m in July. The geological investigations have yielded the following results at the site of boring :*

Depth	***Type of Strata***
0 to 5 m	*Surface clay*
5 to 20 m	*Very fine sand*
20 to 30 m	*Clay with Kankar*
30 to 50 m	*Coarse sand*
50 to 60 m	*Clay*
60 to 70 m	*Medium sand*
Below 70 m	*Clay with Sand stone*

Solution.

(*i*) **Design of the tube**

Assume the velocity of flow in the tube = 2.5 m/sec

Discharge required = 33,000 gallons per hour

$= 33{,}000 \times 4.55$ litres per hour

$= \dfrac{33{,}000 \times 4.55}{1{,}000}$ cubic metres per hour

$= \dfrac{33 \times 4.55}{60 \times 60}$ cubic metres per sec $= \mathbf{4.17 \times 10^{-2}\ m^3/s}$

Area of the tube $= \dfrac{\text{Discharge}}{\text{Velocity}} = \dfrac{4.17 \times 10^{-2}}{2.5}\ m^2 = 167\ cm^2$

Diameter of the tube $(d_w) = \sqrt{\dfrac{4}{\pi}A} = \sqrt{\dfrac{4}{\pi} \times 167} = 14.6$ cm.

Use 15 cm dia pipe

The actual velocity of flow in 15 cm dia pipe

$$= \frac{4.17 \times 10^{-2}}{\frac{\pi}{4} \times (0.15)^2} = \mathbf{2.36\ m/sec.}$$

*For detailed description, please refer chapter 6.

(ii) **Size of the bore hole.** The size of the bore hole should be 5 cm more than that of the pipe. Hence, use 20 cm dia bore hole.

(iii) **Length of the strainer.** From the geological investigation report, it is evident that the main aquifers are confined between clay strata. Hence, the discharge formula for confined stratum will be used to work out the strainer length. Using equation (4.39), we have

$$Q = \frac{2\pi KH.s}{2.3 \log_{10} \frac{R}{r_w}}$$

or
$$H = \frac{2.3\, Q \log_{10} \frac{R}{r_w}}{2\pi K.s}$$

In this case $Q = 4.17 \times 10^{-2}$ m³/s

r_w = Radius of tube = $\frac{0.15}{2}$ = 0.075 m

s = Depression head = 5 m (given)

R = 350 m (assumed)

K = Permeability coefficient for sand

= 0.04 cm/sec (assumed—Table 4.2) = 0.0004 m/sec

$$\therefore \quad H = \frac{2.3 \times 4.17 \times 10^{-2} \times \log_{10} \frac{350}{0.075}}{2 \times 3.14 \times 0.0004 \times 5} = 28 \text{ m}$$

Check for the entrance velocity (v_e)

Using eqn. (4.26) as : $h = \frac{Q}{A_0 \cdot v_e}$

where h = length of the screen = H (here) = 28 m

$$\therefore \quad 28 = \frac{4.17 \times 10^{-2}}{(\pi \times 0.15 \times 15\%) \times (v_e)}$$

(assuming 15% opening area in the screen)

= 0.0197 m/s

= 1.97 cm/s < 2 cm/s (O.K.).

Note. If v_e is found to be more than the safe entrance velocity of 2 to 3 cm/s, then either the dia of the screen or the length of the screen shall have to be increased. In case this is not practically feasible, then the percentage of the opening area shall have to be increased from 15% to about 20 to 25% (preferably 20%).

The selected 15 cm dia screen in 19 m length can now be provided in the coarse sand aquifer (lying at depth between 30 to 50 m), and 9 m depth can be provided in the medium sand aquifer (lying at depth between 60 to 70 m), thus providing a total 28 m screen length. In both the cases, 0.5 m aquifer depths shall be left unscreened at either ends.

This arrangement will lead to screening of 95% of each aquifer depth, as against the preferred value of 90%. Even this anomaly can be corrected by using a screen of 20 cm dia, and recalculating the length of the screen.

(*iv*) **Type of Pump Required.** Since the fluctuations of water--table (*i.e.* 15 m − 10 m = 5 m) plus the depression head (5 m) totals to 10 m, which is more than 6 to 8 m, the centrifugal pump is out of question. Bore hole type of pump shall, therefore, be used.

(*v*) **Horse Power of the Motor**

$$\text{H.P.} = \frac{w.QH}{0.735.\eta} \qquad \text{...(Refer Eq. 4.62)}$$

where $Q = 4.17 \times 10^{-2}$ m^3/s

$\eta = 0.65$

$w = 9.81$ kN/m^3

H = Total head against which the motor has to work

= maximum depth of watertable + Depression head + Velocity head + Losses.

Where,

(*a*) Maximum depth of watertable = 15 m (given)

(*b*) Maximum depression head = 5 m (given)

(*c*) Velocity head $= \frac{V^2}{2g} = \frac{(2.36)^2}{2 \times 9.81} = 0.29$ m

(*d*) Loss of head due to friction in pipe (h_f) plus losses at bends in strainer, etc. (say 0.25 h_f) where

$$h_f = \frac{f' l.V^2}{2gd}$$

Assuming $f' = 0.024$

l = Length of pipe line

= Vertical length of pipe including the blind pipe and strainer + Horizontal length of delivery pipe.

= 70 m + 10 m = 80 m

$$\therefore \qquad h_f = \frac{0.024 \times 80 \times (2.36)^2}{2 \times 9.81 \times 0.15} = 3.64 \text{ m.}$$

Losses in strainer and bends, etc.

$= 0.25\ h_f = 0.25 \times 3.64 = 0.91$ m.

Total losses $= 3.64 + 0.91 = 4.55$ m.

$\therefore \qquad H = 15 + 5 + 0.29 + 4.55 = 25.84$ m

Hence, $$\text{H.P.} = \frac{w.Q.H}{75\eta}$$

$$= \frac{9.81 \times 4.17 \times 10^{-2} \times 25.84}{0.735 \times 0.65} = 22.1 \text{ H.P. ; say 23 H.P.}$$

Hence, 23 H.P. motor is required. **Ans.**

4.25. Quality and Quantity of Ground Water and Its Usefulness for Public Water Supplies

Quality. As pointed out in the previous chapter, the ground water supplies are generally purer and safer, because the water before getting stored in the underground reservoir, undergoes natural filtration during percolation through the soil pores. These waters are, therefore, generally free from suspended impurities and organic matter, which in turn, is generally responsible for disease producing bacteria. These waters are, therefore, less likely to be contaminated by bacteria. However, they are likely to contain greater amounts of dissolved salts, minerals, gases, etc. These may impart different tastes, odours and some other properties, such as hardness, etc. to it. Hard rocky soils which do not easily dissolve, therefore, provide better ground water. Although the quality of ground water is quite safe, yet it is advisable to protect these sources from possible dangers of pollution. Since the chances of outside pollution are much less in an artesian aquifer, the quality of tubewell water, drawing water from the 2nd or even lower water bearing strata, is generally very safe. These waters can be supplied to the public after proper testing, with minor treatments or sometimes even without any treatment. Ground waters are generally found colder in hot seasons and hotter in cold seasons, and are thus, very useful for domestic uses.

Quantity. The quantity of ground water available is generally less than that of surface sources. The amount of ground water available at a particular place depends upon the underground storage and the geological formations of the area or substrata. This available quantity of ground water, however, remains fairly constant in different times of the year, except for shallow wells or springs, where large scale fluctuations may take place.

Owing to its inherent advantages in better quality, and due to lesser developments of modern purification plants, ground waters are being largely tapped in India. About 70 to 80 per cent of our population get their supplies from underground sources. However, due to the excessive use of ground water, the water table levels in most parts of the country are falling day by day. Proper conservation, protection, and recharge of this vital source is, therefore, extremely important for us.

4.26. Comparative Study of Surface and Sub-surface Supplies

The salient features of the surface and sub-surface sources as discussed in this as well in the previous chapter, are summarised in table 4.8 below :

Table 4.8. Comparison of Surface Sources and Ground Water Sources

S. No. (1)	*Item* (2)	*Surface Sources* (3)	*Sub-surface Sources or Ground water Sources* (4)
1.	Forms in which available	Ponds, lakes, streams, rivers, artificial storage reservoirs, etc.	Infiltration galleries, infiltration wells including Ranney wells, springs (gravity and artesian), wells (shallow and deep), tubewells, etc.

Contd...

Table 4.8. Continues

S. No. (1)	*Item* (2)	*Surface Sources* (3)	*Sub-surface Sources or Ground water Sources* (4)
2.	Quantity	Huge amounts are available during rainy seasons, but the available quantity is considerably reduced during summers. To ensure constant and continuous supplies throughout the period, storage reservoirs are generally constructed, which store water during rains and utilise the same during dry periods. Since surface sources can provide sufficient amounts of water, they are very useful for public supplies for big cities and industrial towns. They are also useful for irrigation supplies.	The available yields are usually comparatively smaller, and are useful for towns and villages only.
3.	Quality	Surface waters usually contain huge amounts of suspended impurities. Most of the Indian rivers carry huge amounts of inorganic matter, such as silt and mud, especially when they flow through alluvial tracts. Since the surface waters flow through various regions, they pick up a lot of organic impurities in the form of vegetables, dead animals, eatables, and other household wastes, etc. Sometimes industrial wastes and pollutants also get added to them. The river waters are, therefore, highly polluted and unsafe. A lot of treatment is required when direct rivers are used as sources of supplies. However, when water is drawn from storage reservoirs, it contains lesser suspended matter as it settles down in still	Ground water generally contains lesser suspended matter. As the water passes through the porous earth strata before becoming the ground water, it undergoes natural filtration which reduces the suspended impurities. These waters, however, generally contain larger amounts of dissolved salts, minerals and gases which they pick up during their long journeys through the soil strata. The dissolved salts make these waters saltish and hard. Moreover, shallow and unconfined or gravity well waters are liable to be contaminated with higher bacterial load. However, the chances of contamination are less in artesian well waters.

Contd...

(1)	(2)	(3)	(4)
		reservoir water. This artificial storage, on the other hand, introduces organic impurities by encouraging plant and weed growth. However, controlled plant growth sometimes produces useful results by producing oxygen and thus removing the colour of the water and making it purer. These waters, therefore, require thorough checkup and a lot of treatment before they can be supplied to the public.	These waters generally require lesser treatment and can generally be supplied to the public with no treatment or minor treatment alone.

PROBLEMS

1. (*a*) What is meant by 'Porosity' and 'Permeability', and how do they affect the ground water storage ?

   (*b*) Briefly mention the various zones of ground water, and point out the importance of the 'zone of saturation' in this connection.

2. (*a*) Write a brief note on the movement of ground water, and define and explain the importance of coefficient of permeability in this connection.

   (*b*) Define and briefly explain any four of the following terms :

   (*i*) Capillary fringe ;

   (*ii*) Specific yield ;

   (*iii*) Pellicular water ;

   (*iv*) Specific retention or Field capacity ;

   (*v*) Aquifers and their types ;

   (*vi*) Coefficient of storage ; and

   (*vii*) Perched aquifers.

3. Enumerate the various forms in which ground water occurs in nature. Discuss briefly the exploitation of its most important source.

4. (*a*) What are 'Infiltration galleries' and 'Infiltration wells'. Explain both with neat sketches. Also define a 'Ranney well'.

   (*b*) What are springs, and how are they formed. Discuss briefly the various types of springs.

5. (*a*) What is a dug well, and what are their types ? How are dug wells constructed ?

   (*b*) Enumerate the methods which are used for determining the yield of dug wells. Discuss briefly any one of these methods.

6. (*a*) Differentiate between 'shallow' and 'deep' dug wells. Define and explain the importance of 'cavity formation' in such wells, and compare the possible yields from both these types of dug wells.

   (*b*) Design dug well in fine sand for a yield of 0.003 cumec when operated under a depression head of 3.5 metres. Assume the value of 'specific capacity' of the aquifer as equal to 0.50 m^3/hr./sq. m of area/m of depression head.

7. (*a*) Describe briefly the different methods of sinking tubewells. Draw a neat sketch of a tubewell and its various parts. (*Engineering Services 1971*)

(*b*) Calculate the discharge from a tubewell 50 centimetres in diameter with a drawdown of 2.5 metres in an unconfined water-bearing stratum 15 metres thick. Permeability may be taken as 1,600 litres per day, and radius of influence as 250 metres. (*Engineering Services 1971*)

8. (*a*) Give an equation defining Dacy's law. What is its limitation ?

(*b*) What is an artesian well ?

(*c*) The cost of pumping increases as pipe diameter is reduced but the cost of pipe decreases. How will you make a choice ? (*Engineering Services 1974*)

9. (*a*) Describe an artesian well, and how is it formed ?

(*b*) Design an infiltration gallery, given the following :

Population	= 75,000
Per capita supply	= 150 litres/day
Length of test gallery constructed	= 6 metres
Depth of aquifer	= 5 metres
Pumping rate	= 600 l.p.m.
Draw-down (corresponding to the above pumping rate)	= 1.5 metres.

It is desired to maintain a depth of atleast 1.0 metre above the sole of the aquifer. Draw a typical cross-section of gallery indicating different layers.

10. (*a*) Differentiate between confined and unconfined aquifers.

(*b*) Derive an expression for determining the discharge from an unconfined aquifer.

(*c*) The diameter of a tubewell is 30 cm and the depth of the aquifer is 15 m. The radius of circle of influence of the well is 135 m. The coefficient of permeability of the soil is 30 m/day. Calculate the drawdown of the well when the yield of the well is 2,000 m^3/day.

11. What is meant by Tubewells ? What are their types ? Describe the most widely used type of tubewell with a neat sketch. What are the approximate values of the average yield and depth of such a tubewell ?

12. (*a*) Enumerate the different methods which are used for drilling tube wells. Discuss any one of these methods in detail.

(*b*) Discuss briefly the design principles involved in the design of a strainer type of a tubewell.

(*c*) What is the average life of a tubewell and what are the reasons for their failures ? What remedies will you suggest for increasing their life ?

13. (*a*) Distinguish between non-equilibrium and equilibrium conditions in an aquifer from which water is withdrawn through a well. Explain when the above conditions can be expected in an aquifer.

(*b*) Derive a formula for discharge of well in a homogeneous artesian aquifer, assuming equilibrium flow conditions.

(*c*) In an artesian aquifer, the drawdown is 1.5 m at a radial distance of 8 metres from a well after two hours of a pumping. On the basis of Thies' non-equilibrium equation, determine the pumping time for the same drawdown (1.5 metres) at a radial distance of 20 metres from the well.

[Ans. 12.5 hours]

14. Distinguish with sketches, if necessary, the difference between an unconfined and a confined aquifer. Derive a formula for discharge of well in a homogeneous unconfined aquifer assuming equilibrium flow conditions, state the assumptions on which the formula is based.

15. (*a*) What is a Dupuit's equation ? State the assumptions that enter in its development. Explain the Thies' formula. What is well function ?

(*b*) A 30 cm dia well penetrates 20 m below the static watertable. After 24 hours of pumping at 5,000 litres per minute, the water level in a test well at 100 m away is lowered by 0.5 m, and in a well at 30 m away, the drawdown is 1 m. What is the transmissibility of the aquifer ?

16. (*a*) Explain :

(*i*) Specific retention of a soil

(*ii*) Specific yield of an aquifer

(*iii*) Storage coefficient of an aquifer

(*iv*) Specific capacity of a well.

(*b*) Step drawdown test was carried out in a well constructed to give a yield of 2300 litres per minute. The data obtained is given below. Determine the well loss and the efficiency of the well.

Yield in l.p.m.	*Drawdown in metres*
1230	3.72
1840	5.65
2460	7.62
3070	9.62
4080	13.07
6130	20.21

17. (*a*) State Dupuit's assumptions for obtaining general equations governing ground-water flow. Derive an expression for the confined aquifer. How can the expression be used to evaluate the aquifer permeability ?

(*b*) A well penetrating an aquifer which is underlain and overlain by impermeable layers was tested with a uniform discharge of 100 litres/min. The steady state drawdowns measured in two observation wells which were at 1 m and 10 m radial distances from the centre of the pumped well were 13.4 m and 4.2 m respectively. Determine the hydraulic properties of the aquifer, if its saturated thickness is 10 m.

[**Ans.** Transmissibility = 0.039 m^2/min.
Hydraulic conductivity = 0.00397 m/min]

18. A 10 cm diameter well was pumped at a uniform rate of 500 litres/min, while observations of drawdown were made in an observation well located at a distance of 50 m from the well. The original head of water, measured from the top of the impervious layer was 25 m. The hydraulic conductivity of the aquifer was 1.83×10^{-3} m/min. Determine the drawdown at the face of the well using Dupuit-Thiem equation and assuming that the flow to the unconfined aquifer is under steady state. [**Ans.** 20 m]

19. Write short notes on any four :

(*i*) Infiltration wells and infiltration galleries

(*ii*) Aquifers and aquicludes

(*iii*) Darcy's law for measuring velocity of ground water flow

(*iv*) Permeability and transmissibility and their relationship

(*v*) Measuring the yield of underground water sources

(*vi*) Surface of seepage and free surface curve

(*vii*) Spherical flow in a well

(*viii*) Cavity formation in wells

(*ix*) Tubewells.

20. Write detailed notes on any three of the following :

(*i*) Deep and shallow tubewells

(*ii*) Design of gravel packs for tubewells

(*iii*) Tubewell screens

(*iv*) Development of tubewells

(*v*) Drilling of tubewells.

—5

Intakes for Collecting Surface Water

5.1. Definition and General Introduction

Whenever the water is withdrawn from a surface source such as a lake or a river or a reservoir, and the entrance of the withdrawal conduit is not an integral part of a dam or any other related structure, then an intake structure must be constructed at the entrance of the conduit. (*The basic function of the intake structure is to help in safely withdrawing water from the source over a predetermined range of pool levels and then to discharge this water into the withdrawal conduit* (normally called ***intake conduit***), through which it flows up to the water treatment plant.) In case of a reservoir where gravity flow is possible, the water may be directly taken through the conduit up to the treatment plant, whereas, in case of direct river supplies, the water after entering the intake well may have to be lifted by low lift pumps and then taken to the treatment plant through conduits.

An intake structure constructed at the entrance of the conduit and thereby helping in protecting the conduit from being damaged or clogged by ice, trash, debris, etc., can vary from a *simple concrete block supporting the end of the conduit pipe* to huge *concrete towers* housing intake gates, screens, pumps, etc. and even sometimes, living quarters and shops for operating personnel.

5.2. Factors Governing the Location of an Intake

The site for locating the intake should be selected carefully, keeping the following points in mind :

(*i*) As far as possible, the site should be near the treatment plant so that the cost of conveying water to the city is less.

(*ii*) The intake must be located in the purer zone of the source so that the best possible quality of water is withdrawn from the source, thereby reducing the load on the treatment plant.

(*iii*) The intake must never be located at the downstream or in the vicinity of the point of disposal of waste water. When it becomes necessary to locate the intake in the close proximity of the disposal of a drain etc., it is advisable to construct a weir or a barrage upstream of the disposal point and install the intake upstream of the barrage. In Delhi, Wazirabad barrage was constructed upstream of Najafgarh drain in 1958, only after the outbreak of Jaundice in 1956, caused by the mixing of contaminated water of Najafgarh drain with Yamuna water at intake point. Had it been foresighted while locating the intake well itself, Delhi perhaps would have been saved of that water borne disease, which affected many people.

(*iv*) The intake should never be located near the navigation channels, as otherwise, there are chances of intake water getting polluted due to the discharge of refuse and waste from ships and boats.

(*v*) The site should be such as to permit greater withdrawal of water, if required at a future date. Thus, there should be sufficient scope for future additions and expansions.

(*vi*) The intake must be located at a place from where it can draw water even during the driest periods of the year. Thus, the intake must be located in deep waters, sufficiently away from the *shore line* (*i.e.*, khadir banks*). As otherwise, during dry periods, when the water level goes down and the water recedes inside, the intake may be left dry without any water to be drawn around it. *In case of rivers which meander considerably and also where there is scarcity of water during summer seasons,* a diversion weir or a barrage must be constructed. This will ensure sufficient quantity of water in the "undersluices"**, from where the water can enter the intake well or intake tower standing within the under sluice pocket of the river ; or from where an "offtake channel"** may lead the water directly to the treatment plant (when treatment plant of the city is situated lower than the "pond level" of the undersluices) or to the sump well situated near the shore for lift. The construction of a diversion weir will ensure constant supplies. For example in Delhi, before the construction of Wazirabad barrage, water was not reaching the intake well during summer seasons, thus creating troubles. These troubles have, however, been eliminated after the construction of this barrage.

(*vii*) The intake site should remain easily accessible during floods and should not get flooded. Moreover, the flood waters should not be concentrated in the vicinity of the intake.

(*viii*) In meandering rivers, the intakes should not be located on curves or at least on sharp curves. If they have to be located on curves, it will be better to locate them on *concave banks** rather than locating them on convex banks*. Although scouring tendencies will be more on the concave side ; yet atleast, the water will remain available on this side ; whereas, on a convex bank, water may not remain available due to silting and consequent blockage.

TYPES OF INTAKES

Various types of intakes that are commonly used are described below :

5.3. Simple Submerged Intakes

A simple submerged intake consists of a simple concrete block or a rock filled timber crib supporting the starting end of the withdrawal pipe as shown in Figs. 5.1 and 5.2. The withdrawal pipe is generally taken up to the sump well at shore, from where, the water is lifted by pumps.

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*Pl. refer chapter on "Rivers, their Behaviour, Control and Training" in "Irrigation Engineering and Hydraulic Structures" by the same author.

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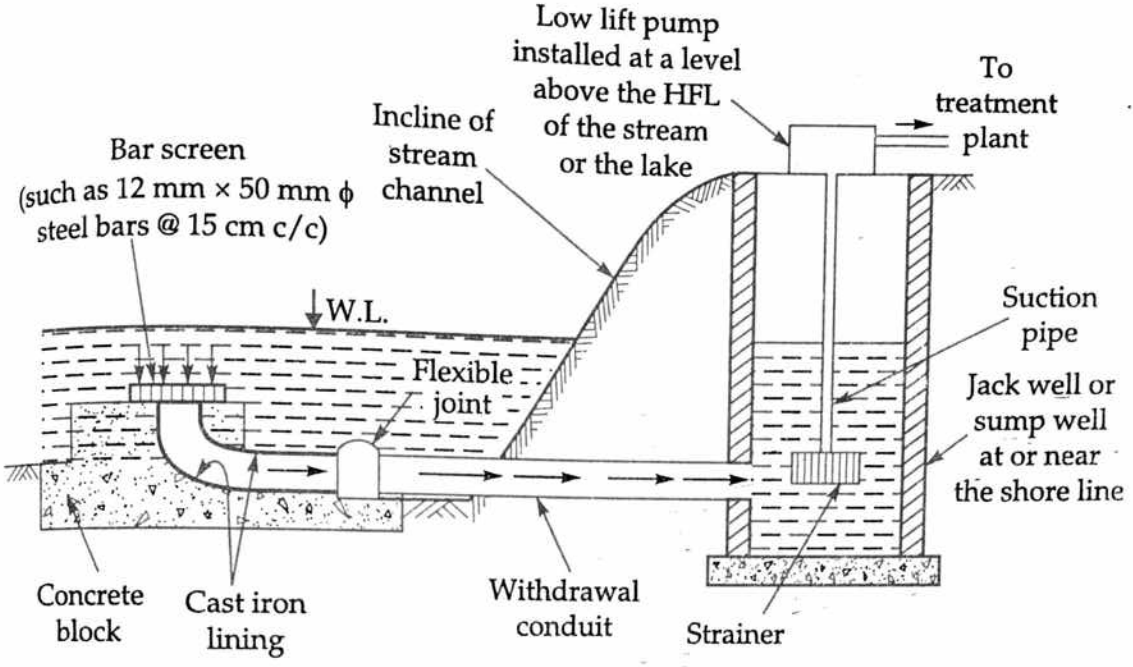

Fig. 5.1. Simple concrete block—submerged intake.

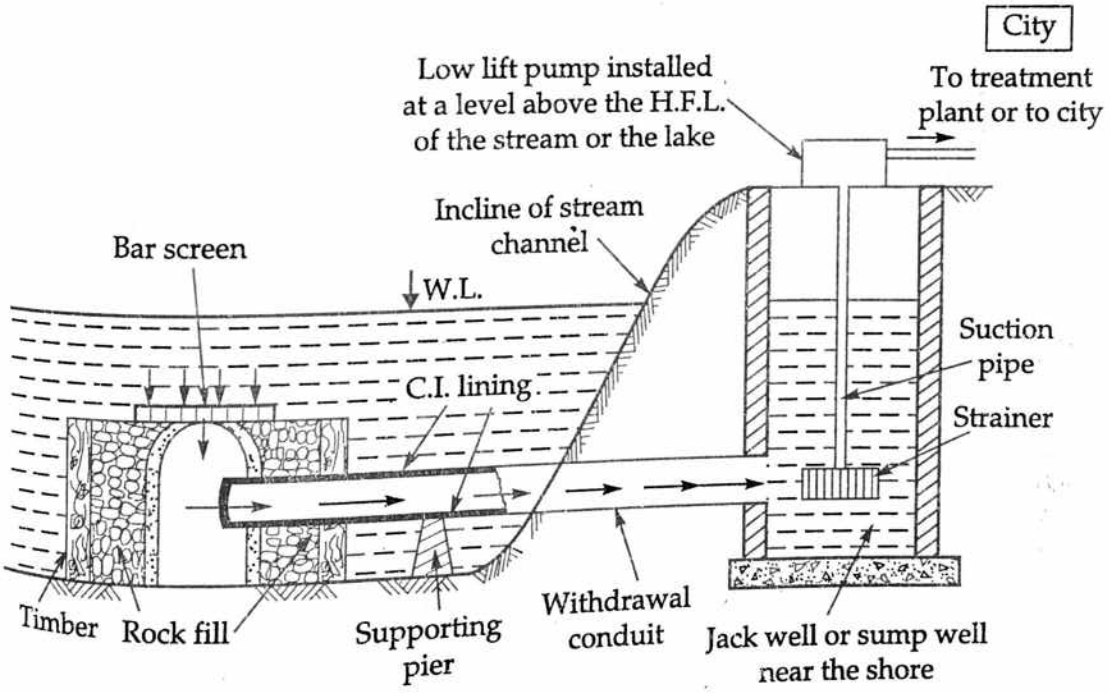

Fig. 5.2. Rock filled timber crib—submerged intake.

The intake opening is generally covered by ***screen*** so as to prevent the entry of debris, ice, etc. into the withdrawal conduit. In case of lakes, where silt tends to settle down, the intake opening is generally kept at about 2 to 2.5 m above the bottom of the lake and thus to avoid the entry of large amounts of silt and sediment.

Such intake structures should be placed in streams or in lakes at a place where they may not get buried under sediment and where there are deep waters. (Note : In lakes, deep waters are generally available in the middle).

(*iv*) The intake should never be located near the navigation channels, as otherwise, there are chances of intake water getting polluted due to the discharge of refuse and waste from ships and boats.

(*v*) The site should be such as to permit greater withdrawal of water, if required at a future date. Thus, there should be sufficient scope for future additions and expansions.

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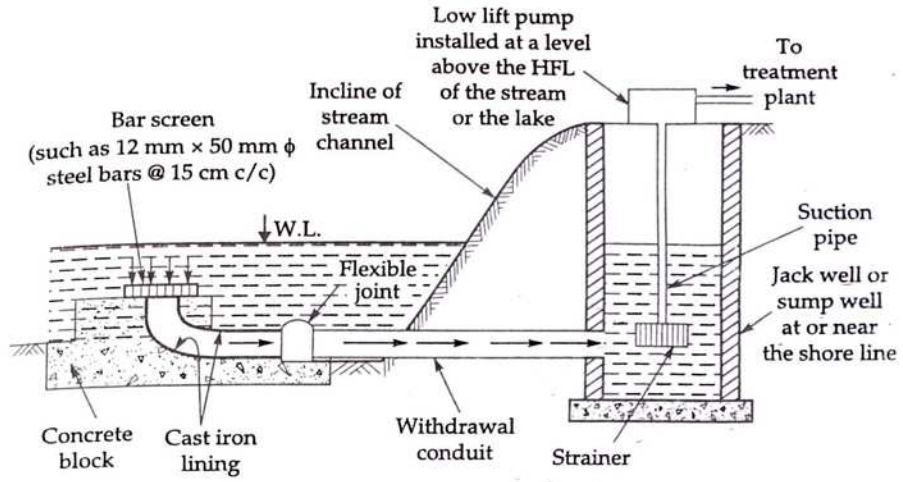

Fig. 5.1. Simple concrete block—submerged intake.

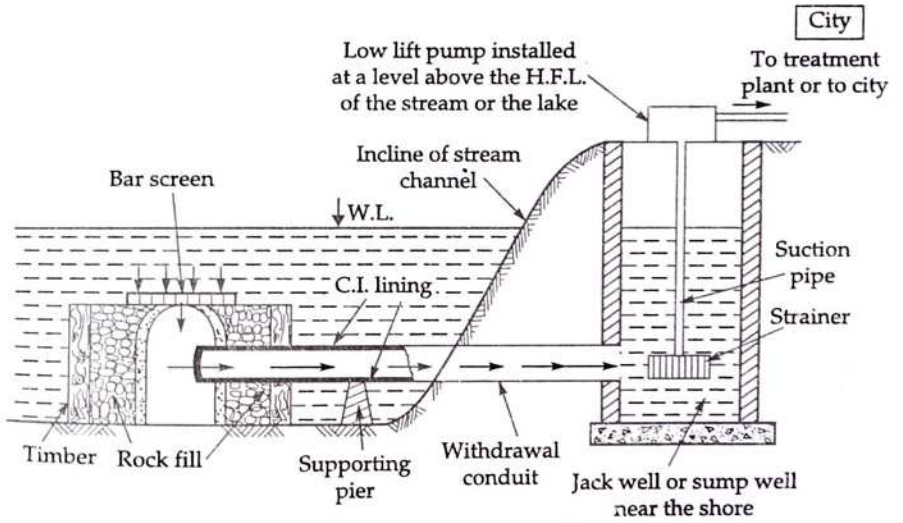

Fig. 5.2. Rock filled timber crib—submerged intake.

The intake opening is generally covered by ***screen*** so as to prevent the entry of debris, ice, etc. into the withdrawal conduit. In case of lakes, where silt tends to settle down, the intake opening is generally kept at about 2 to 2.5 m above the bottom of the lake and thus to avoid the entry of large amounts of silt and sediment.

Such intake structures should be placed in streams or in lakes at a place where they may not get buried under sediment and where there are deep waters. (Note : In lakes, deep waters are generally available in the middle).

(*iv*) The intake should never be located near the navigation channels, as otherwise, there are chances of intake water getting polluted due to the discharge of refuse and waste from ships and boats.

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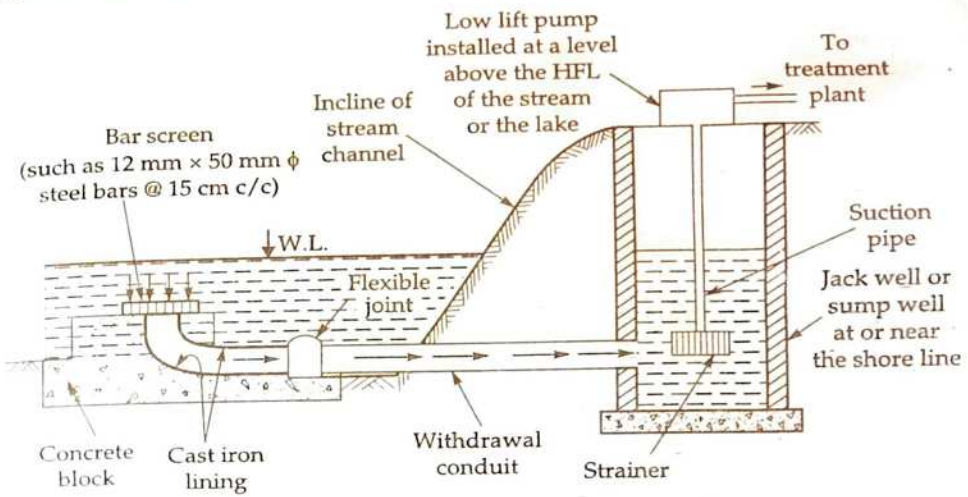

Fig. 5.1. Simple concrete block—submerged intake.

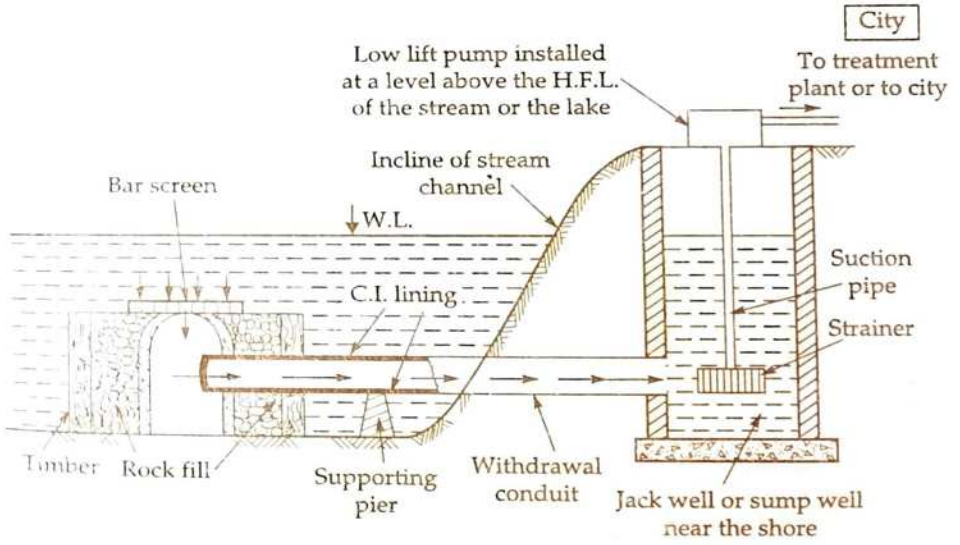

Fig. 5.2. Rock filled timber crib—submerged intake.

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Such intake structures should be placed in streams or in lakes at a place where they may not get buried under sediment and where there are deep waters. (Note : In lakes, deep waters are generally available in the middle).

These submerged intakes are cheap and generally do not obstruct navigation. They are, therefore, widely used for small water supply projects drawing water from streams or lakes having relatively little change in water surface elevation throughout the year. These intakes are not used on bigger projects on rivers and reservoirs, as their main disadvantage is the fact that they are not easily accessible for cleaning, repairing, etc.

5.4. Intake Towers

Intake towers are generally used on large projects and on rivers or reservoirs where there are large fluctuations of water level. Gate controlled openings at various levels, called *ports*, are generally provided in these concrete towers, which may help in regulating the flow through the towers and *permit some selection of the quality of water to be withdrawn.* Access to these towers is generally provided for operating the gates, etc., by means of a foot bridge from the tower up to the dam or upto the shore.

If the entry ports are submerged at all levels, then there is no problem of any clogging or damage by ice or debris, etc. However, the level of the lowest port should be high enough above the reservoir bed, so that the sediment is not drawn into them. There are two major types of intake towers :

(*i*) *Wet intake towers* ; and

(*ii*) *Dry intake towers.*

5.4.1. Wet Intake Towers. A typical section of a wet intake tower is shown in Fig. 5.3. It may consist of a concrete circular shell filled with water up to the

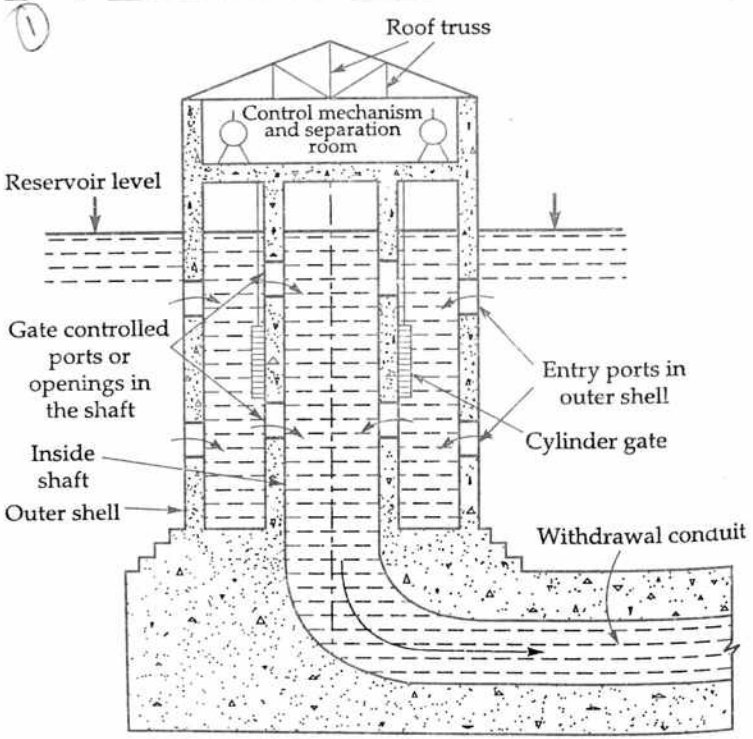


Fig. 5.3. Wet intake tower standing in the river or reservoir.

reservoir level and has a vertical inside shaft which is connected to the withdrawal pipe. The withdrawal may be taken directly to the treatment plant in case no lift is required (such as in a reservoir) or to the sump well in case a low lift is required (such as in case of a river). The withdrawal conduits may lie over the bed of the rivers or may be in the form of tunnels below the river bed. Openings are made into the outer concrete shell, as well as, into the inside shaft, as shown. Gates are usually placed on the shaft, so as to control the flow of water into the shaft and the withdrawal conduit. The water coming out of the withdrawal conduit may be taken to pump house for lift, if the city's water treatment plant is located at high elevation ; or may be taken directly to the treatment plant, if it is situated at lower elevation.

5.4.2. Dry Intake Towers. The essential difference between a dry intake tower and a wet intake tower is that, whereas in a wet intake tower, the water enters from the entry ports into the tower and then it enters into the conduit pipe through separate gate controlled openings ; in a dry intake tower, the water is directly drawn into the withdrawal conduit through the gated entry ports, as shown in Fig. 5.4.

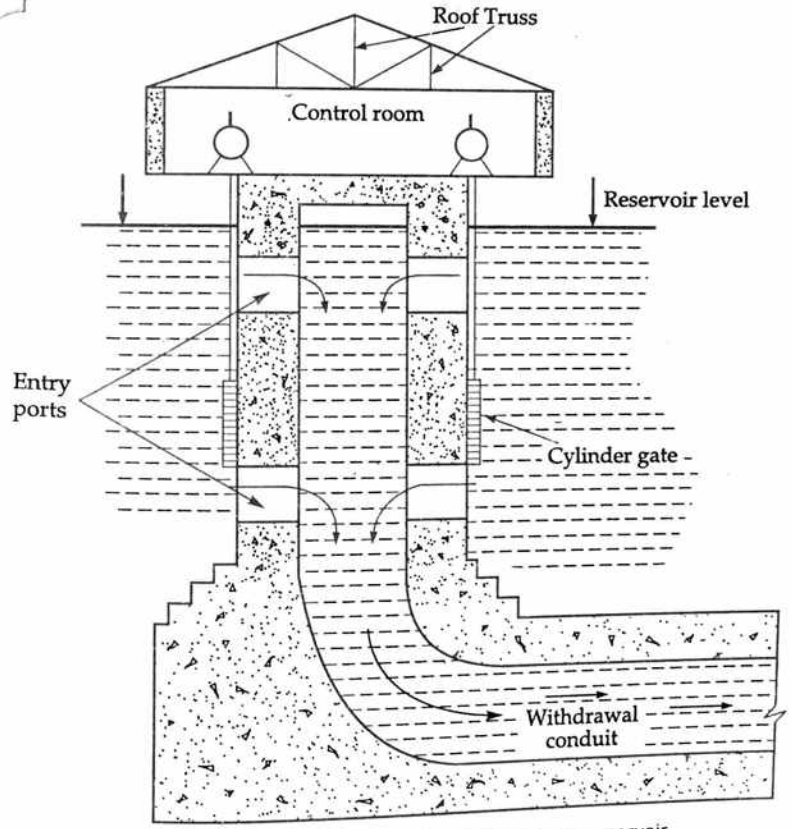


Fig. 5.4. Dry intake tower standing in the river or reservoir.

A dry intake tower will, therefore, have no water inside the tower if its gates are closed, whereas the wet intake tower will be full of water even if its gates are closed. When the entry ports are closed, a dry intake tower will be subjected to additional buoyant forces and hence, must be of heavier construction than the wet intake towers. However, the dry intake towers are useful and beneficial in the sense that water can be withdrawn from any selected level of the reservoir by opening the port at that level.

Intake towers are huge structures of various designs standing in the reservoir or the river, and hence, should be located, as not to interfere with navigation. They must be properly designed so as to withstand the worst possible combinations of various forces, such as hydrostatic pressure, wind and earthquake forces, and forces caused by waves, ice, debris, etc.

5.5. Medium Sized River Intake Structures

Medium sized river intake structures are generally constructed for withdrawing water from almost all rivers, and are a *via media* between the submerged intakes (usually adopted for small streams) and the intake towers (usually adopted for reservoirs). River intakes are discussed in this article; while the canal intakes are discussed in the next article.

River intake structures can be broadly classified into the following two types:

(*i*) *Twin well type* of intake structure ; and

(*ii*) *Single well type* of intake structure.

Both these types of river intake structures are discussed below :

5.5.1. Typical Twin Well Type of River Intake Structure. This is the most commonly used type of river intake, which is generally constructed on almost all types of rivers, where the river water hugs the river bank. Such a condition is usually available on non-alluvial rivers ; while in meandering alluvial rivers, such a condition can be obtained by constructing a weir across the river as to store water up to pond level, thus making it available near the river bank on the under-sluice side.

Typical details of such an intake structure are shown in Fig. 5.5. More detailed section and plan of such an intake structure constructed on Krishna river in Belgaum district of Karnataka State of India, are shown in Fig. 5.6. **Colour Photo Fig. 5.1** shows the Water Supply Intake Structure provided for Wazirabad Water Treatment Plant at New Delhi.

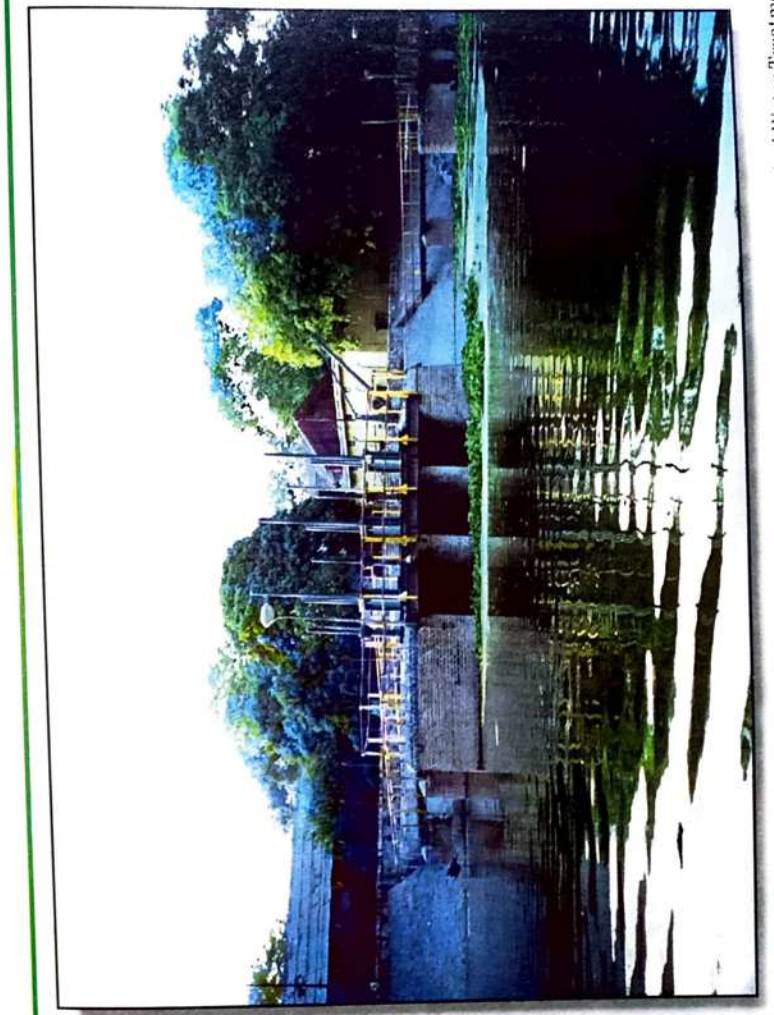
As can be seen from these figures, such a typical river intake structure consists of :

(*i*) *an inlet well* ;

(*ii*) *an inlet pipe* ; and

(*iii*) *a jack well.*

The **inlet well** or the ***intake well*** or the *collector well* is a *circular* or more preferably an *oblong well* (rectangular well with circular ends), located in the river bed, somewhat away from the river bank, amidst water, so that it always remains surrounded with water, even during low flow stage. The well is built in masonry or concrete, and is raised above the river HFL and covered at the top by wooden sleepers, etc. ; as to make it approachable from the river bank through a foot bridge arrangement.



Coloured Fig. 5.1. Photographic view of the "Inlet Well" at the Water Supply Intake of Wazirabad Water Treatment Plant at New Delhi. The Water Treatment Plant is in the background.

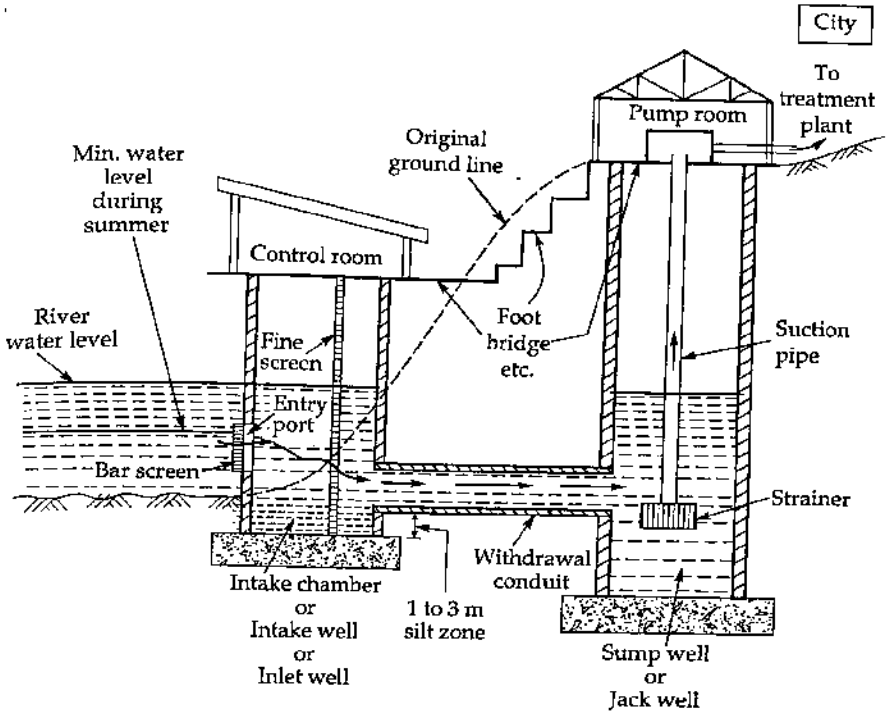


Fig. 5.5. Section of a typical twin well type of a river intake.

River water enters into this well through the openings or ports, which are left in the well steining, and fitted with vertical bar screens. These screens may be made of vertical iron bars of 20 mm dia, placed vertically @ 30 to 50 mm horizontal clear spacing and fitted to an angle iron frame, which may be fixed properly in the opening. Depending upon the discharge to be entered into the inlet well, the total area of such openings can be worked out by restricting the flow velocity through the screens to a value of about 15 to 20 cm/s, as to prevent entry of debris, etc. The total required area of opening(s) can be provided by providing one or more screened openings at one level. Such openings or ports are infact usually provided at 2 to 3 levels in the well, thus providing one or more ports at each level, depending on the pumping rate. The lower layer of ports permits the direct entry of water at the low flow stage of the river ; while the upper layer of ports meet the requirement of the high flood stage. The middle level ports are usually provided at the normal river level in rivers with fluctuating levels. *Sometimes, screened openings are provided in the full height of the inlet well and are controlled by gates, as to enable the water to enter the inlet well from any desired level, depending upon the river water level.* This is more helpful when river water is likely to fluctuate considerably. These ports can be closed or opened by gates to be operated from the top platform, to adjust the opening at any level.

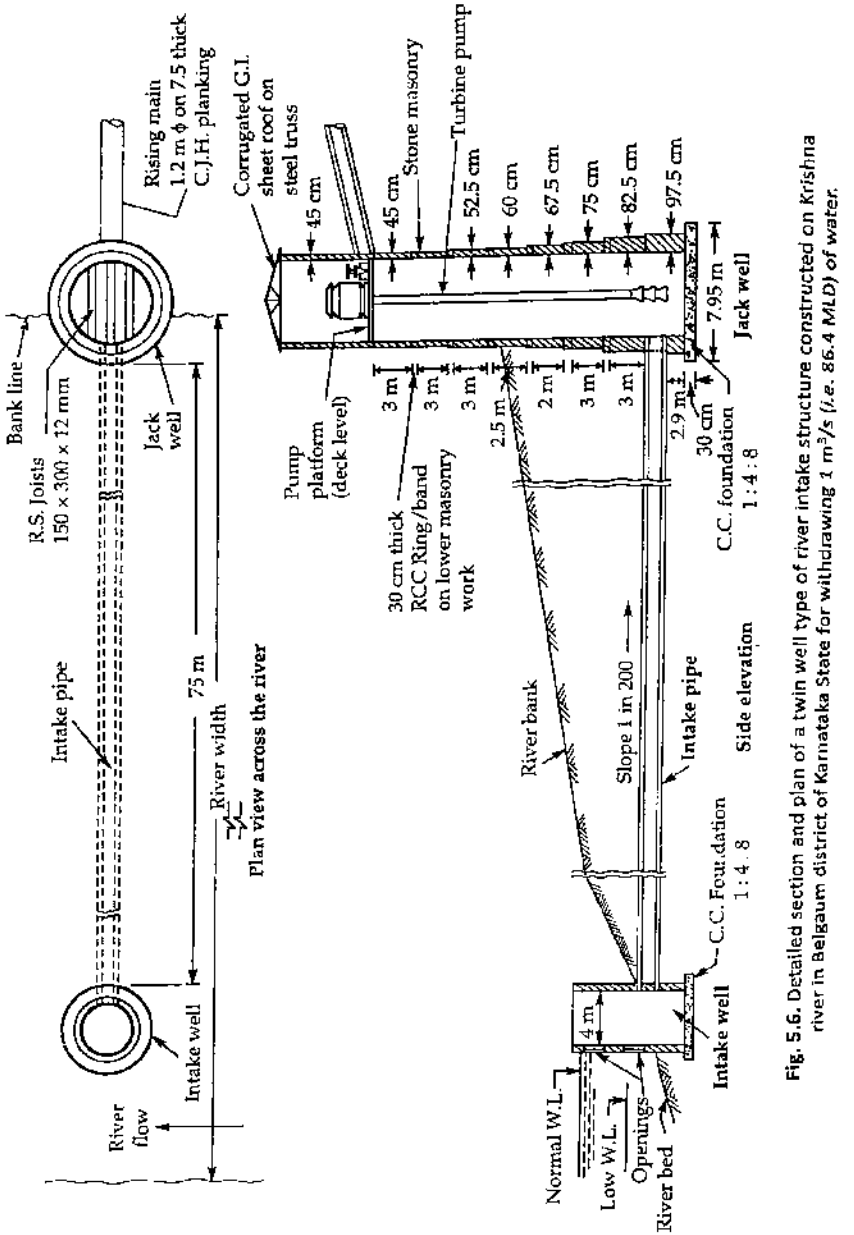


Fig. 5.6. Detailed section and plan of a twin well type of river intake structure constructed on Krishna river in Belgaum district of Karnataka State for withdrawing 1 m^3/s (*i.e.* 86.4 MLD) of water.

The well steining (wall thickness) between the screens is made of R.C.C. and is usually shaped as a hydrofoil to allow smooth entry of water into the intake well.

The **intake well** is connected to the *jack well* (sump well), which is constructed on the river bank, by a R.C.C. *intake pipe*. The section of the intake well below the level of the intake pipe helps to provide space for accumulation of silt and sediment. This depth may be kept at about 1 to 3 m.

or so. This deposited silt will have to be removed mechanically or manually to keep the intake pipe free of silt. The intake well thus helps to keep away silt and debris from the intake pipe and jack well.

The **intake pipe** connecting the intake well with the jack well is usually of a non-pressure type, and is laid with a gentle slope of 1 in 200 or so, towards the jack well. The diameter of this intake pipe depends upon its discharge capacity, which should be equal to the pumped discharge from the jack well. The pipe size should be such that the flow velocity does not exceed about 1.2 m/s. The dia of this pipe should, however, be not less than 45 cm, since smaller pipes may get chocked and become difficult to clean, thereby necessitating their removal and replacement with new pipes. This pipe shall be designed as a gravity main, since the water on both ends of this pipe will be under atmospheric pressure.

Water entering the **jack well** from the intake pipe is lifted by pumps and is fed into the rising main through the delivery pipe of the pump. The jack well is constructed on high ground (river bank) with a suitable length of the inlet pipe (intake pipe). It should be located as close to river as possible. Although, it would be advantageous to locate the jack well on high ground above the river HFL, but this becomes rarely possible without increasing the inlet length, and consequently the cost. It may therefore become economical to locate it near the water body at low-flow stage, even if a good portion of its height remains above the ground level. In such a case, access should be provided by a rigid steep ladder.

The jack well should be founded on hard strata with a bearing capacity of not less than 450 kN/m^2. In rare cases, where such a strata is not met with, an R.C.C. raft may be laid at the bottom. Even when the well is dug through rocky strata, it may be desirable to smoothen the sides or provide a masonry lining in the well. This will facilitate access, down to the bottom of the well, for maintenance and desilting operations.

Suitable high-head turbine pumps or centrifugal pumps may be used to lift water from the jack well. The vertical centrifugal pumps or motors of turbine pumps are installed over the roof of the jack well in the constructed control room, which may preferably be made in square shape, over the circular roof by projecting cantilevers from the circular steining of the well. The diameter of the jack well may be kept at about 4 to 5 metres, and should be enough to accommodate the suction pipes and strainers of the pumps installed to lift the designed discharge, and to provide easy access for cleaning, etc. Usually, a minimum of 2 pumps may be installed in parallel to feed to a common outlet (rising main). For installing turbine pumps, the bottom of the jack well should be kept at about 3.5 m below the lowest water level in the well to accommodate the pump bowl assembly, suction pipe and strainer, and to allow for silt pocket. For centrifugal pumps, this depth may be kept at only about 2 m or so. The pump house constructed over the roof of the jack well will house the vertical centrifugal pumps or the vertical motors of the turbine pumps, the starters and the switch boards, etc. When centrifugal pumps are used, then the static suction head in centrifugal pumps should usually be limited to 4 m to permit pumping at high efficiency, and should in no case exceed 6 m. In

most of the cases, however, it may not be possible to do so due to larger fluctuations of water level, since the difference in HFL and lowest river level itself may be higher than this value. Under such circumstances, naturally, vertical turbine pumps will have to be used. Alternatively, horizontal centrifugal pumps may be installed in an adjoining building (under ground pump house), withdrawing water from the jack well through a network of cast iron pipes and valves, etc. as shown in Fig. 7.17.

To prevent vortex formation in the sump well (jack well), the intake pipe should approach the side walls of the sump readily at right angles. To prevent free cascading flow into the water surface, the intake pipe should preferably be extended by a long sweep bend to discharge below the water level. In case of round sump wells, vertical baffle boards can be installed opposite the pump entrance to prevent vortex formation around the pump suction.

5.5.2. Single Well Type of a River Intake. In alluvial rivers, water is usually ponded up by constructing a weir across the river. From the upstream side of such a weir, a channel may sometimes be taken off, as in a usual diversion head works scheme*. The water entering this offtake canal may then be collected and lifted for supply to the City Treatment Plant located at higher elevation. The intake structure for collecting water in such a case shall then be located in this offtake channel. Similarly, sometimes, an approach channel may be excavated from the deep river towards the river bank up to the site of intake. In such situations, the intake structure may be constructed across the off-take or approach channel in the form of a single jack well (sump well) provided with direct entry ports, as shown in Fig. 5.7.

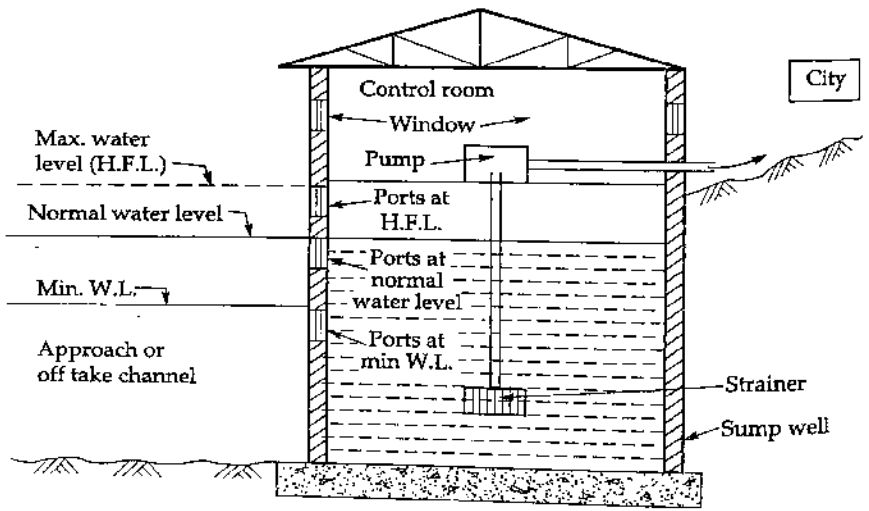


Fig. 5.7. Section of a Single Well Type of a river intake to be located across an off-take channel or an approach channel.

*Please refer chapter on Canal Head Works in "Irrigation Engg. and Hydraulic Structures" by the same author.

This arrangement will eliminate the construction of a separate inlet well and the inlet pipe, as are required in the twin well type of river intake. Openings or ports fitted with bar screens are provided in the jack well itself, rather than in the inlet well, as explained in the twin well structure. The sediment entering this intake will usually be less, since clearer water will enter the off-take channel. This sediment will partly settle down in the bottom silt zone of the jack well or may be lifted up with the pumped water, since the modern day pumps can easily lift sedimented waters. The jack well can be periodically cleaned manually, by stopping the water entry into the well.

Example 5.1. *Design a river intake with respect to (i) number and size of the openings in the intake well ; (ii) the size, shape and the height of the intake well ; and (iii) the gravity pipe for raw water connecting the intake well and jack well. The data supplied is :*

R.L. of river bed	*= 100 m*
R.L. of lowest water level	*= 102 m*
R.L. of normal water level	*= 115 m*
R.L. of high flood level	*= 120 m*
Population to be served	*= 50,000*
Average water demand	*= 200 l.p.h.*

Assume other suitable data required.

Solution. Average water demand to be pumped

$$= 50{,}000 \times 200 \text{ l/d} = 10 \text{ MLD}$$

Max. water demand = 1.8 × 10 MLD = 18 MLD

Assuming the pumping to take place for 16 hrs a day, the discharge to be pumped

$$= \frac{18 \times 10^6}{10^3 \times 16 \times 60 \times 60} \text{ m}^3/\text{s} = 0.3125 \text{ m}^3/\text{s}.$$

(*i*) and (*ii*) **Design of Inlet Well**

Let us provide an **oblong** shaped inlet well, with openings provided at 3 levels ; one layer of opening(s) shall be kept below RL 102.0 m (lowest river level) ; and the other may be kept below HFL stage *i.e.* below RL 120 m. A middle layer of ports can be provided just below the normal river water level of RL 115 m.

These openings shall be fitted with bar screens made of 20 mm dia steel bars of say 50 mm openings (clear). Let the velocity through the bar screens be limited to 0.16 m/s.

∴ The area of openings required at each level

$$= \frac{Q}{v}$$

$$= \frac{0.3125}{0.16} \text{ m}^2 = 1.95 \text{ m}^2.$$

Let us provide 1 m height of screen openings ; then the clear length of the openings required = 1.95 m.

No. of openings required = $\frac{1.95}{0.05} = 39$

No. of bars = 38

Length occupied by 20 mm ϕ bars

$= 38 \times 0.02 = 0.76$ m

$\therefore$ Total length of screen

$= 1.95 + 0.76 = 2.71$ m ; say **2.8 m.**

Let us provide 2 ports at each level. The size of each port will then be 1 m ht. × 1.4 m length. In all, there will be 6 screened ports ; 2 at each of the three levels. 2 screened ports will thus be provided within the well steining between RL 102 m to 101 m, each having 1.4 m length. 2 other screened ports shall similarly be provided between RL 115 m to 114 m ; and 2 others between RL 120 m to 119 m, as shown in Fig. 5.8.

These ports can be fitted in an oblong well, consisting of rectangular length of 3 m (sufficient to fix 2 bar screens each of length 1.4 m) and provided with circular ends. The well can have a width of say **2 m. Ans.**

This inlet well can be sunk into the river bed by say 3 m below the river bed, as to provide space for accumulation of sand and silt. So let us keep the bottom of inlet well at RL 97 m.

Also, let us provide a free-board of 2 m over the river HFL to fix the bottom level of the roof of the well. Hence, provide invert level of roof at RL 122 m. The height of inlet well will then be 122 m – 97 = **25 m. Ans.**

***(iii)* Design of gravity pipe connecting intake well and jack well**

The intake pipe shall be designed to flow by gravity at a max. velocity of say 1.2 m/s.

$\therefore$ $Q = 0.3125$ m^3/s

$v = 1.2$ m/s

Area of pipe required $= \frac{\pi}{4} \cdot d^2 = \frac{Q}{v} = \frac{0.3125}{1.2}$

$\therefore$ $d = 0.58$ m ; Hence use 60 cm dia R.C.C. intake pipe, giving a velocity of

$\frac{0.3125}{\frac{\pi}{4}(0.6)^2} = 1.1$ m/s.

Using Manning's formula, we have

$$v = \frac{1}{n} \cdot R^{2/3} \cdot \sqrt{S}$$

$$1.1 = \frac{1}{0.017}\left(\frac{0.6}{4}\right)^{2/3} \cdot \sqrt{S} \qquad \left(\because R = \frac{A}{P} = \frac{\frac{\pi}{4}d^2}{\pi d} = \frac{d}{4}\right)$$

or $$S = \frac{1}{228} \text{ ; say 1 in 200}$$

Hence, lay 0.6 m dia intake pipe at a gradient of 1 in 200. **Ans.**

The details of the designed intake well are shown in Fig. 5.8.

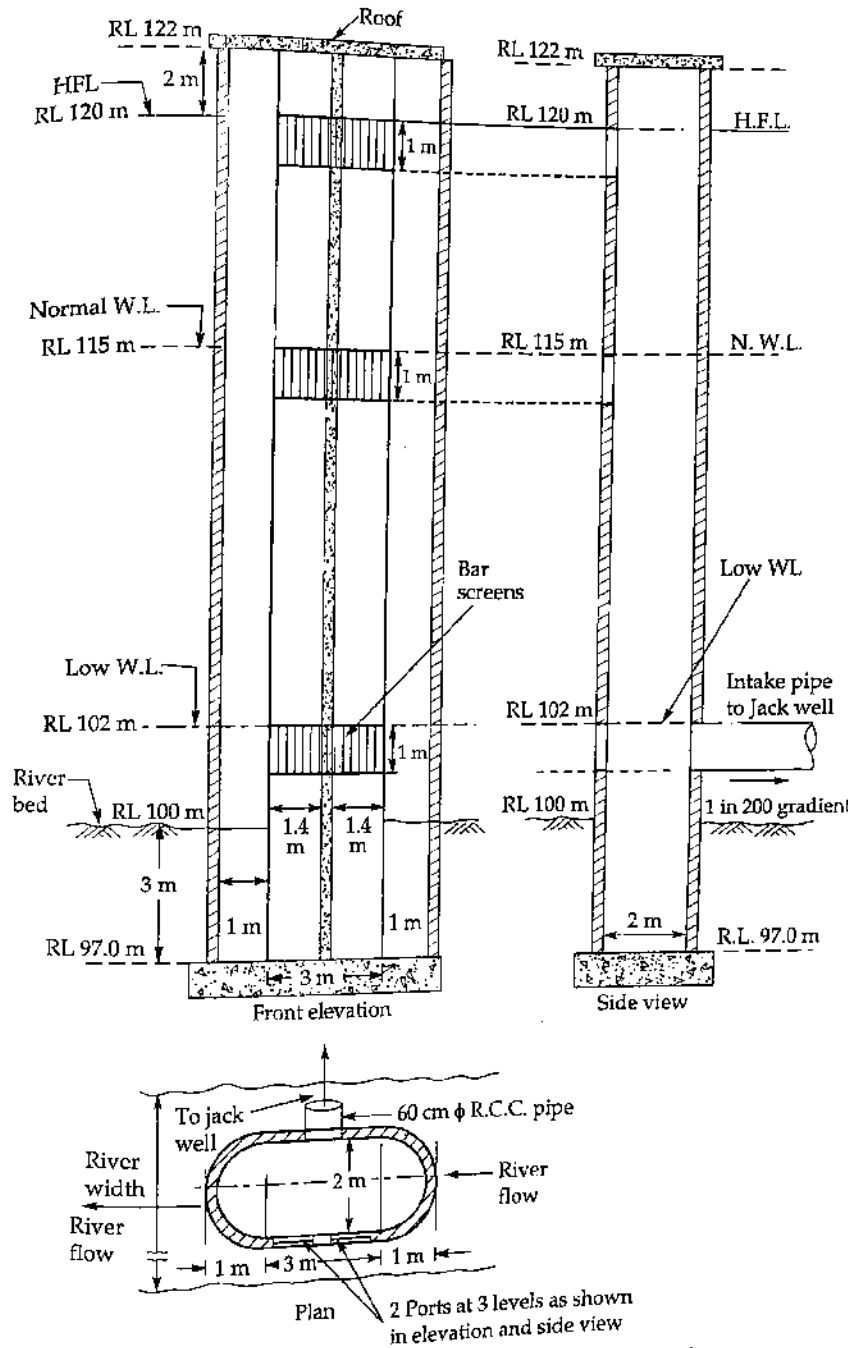


Fig. 5.8. Designed details of the Intake Structure for Example 5.1.

5.6. Canal Intakes

In case of canals, the intake well is generally located in the bank of the canal, and water enters the chamber through an inlet pipe (Fig. 5.9), covered with a fine screen. The water coming out of the chamber through the outlet conduit may be taken to the sump well or city, as desired.

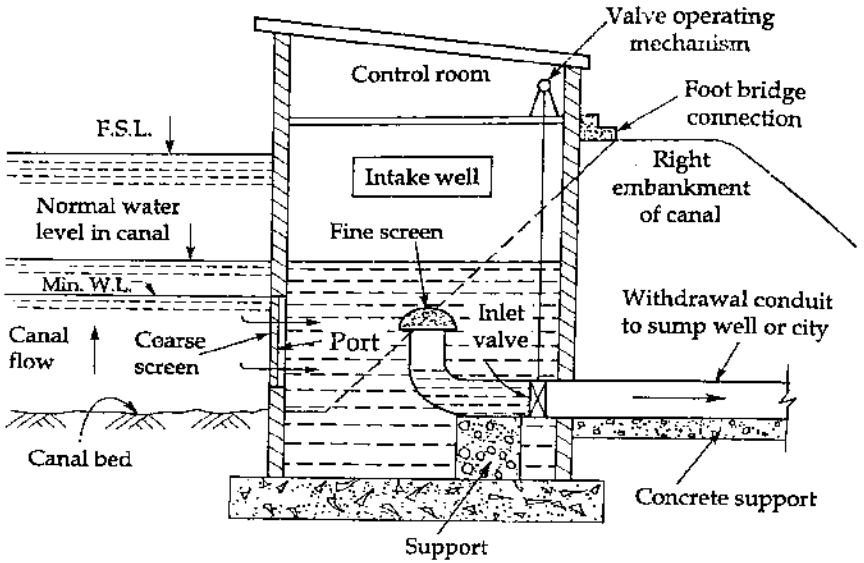


Fig. 5.9. Canal Intake Well.

Since the flow area in the canal is obstructed by the construction of intake well, the flow velocity in the canal increases due to reduction in water way, and hence pitching is generally provided on the upstream as well as downstream portions of the canal near the intake.

The entry of water in the intake well takes through a coarse screen, the top of which is generally provided at minimum water level in the canal, and bottom is about 0.15 m above the canal bed to avoid entry of bed load. An additional fine screen is provided at the inlet end of the withdrawal conduit. This inlet end is of bell mouth shape with perforations of fine screen on its surface. An outlet valve, operating from the top, is provided to control the entry of water into the outlet pipe.

The flow velocity through the outlet conduit is generally kept at about 1.5 m/sec, and this helps in determining the area and dia of the withdrawal conduit.

The area of coarse screen is designed by limiting the flow velocity to as low as 0.15 m/sec or so. The flow velocity through the bell mouth inlet is limited to about 0.3 m/sec or so. The head loss in the intake conduit upto treatment works, can be determined by using Hazen William formula, equation (6.12), as given in the next chapter.

Example 5.2. *Design a bell mouth canal intake for a city of 75,000 persons, drawing water from a canal which runs only for 10 hours a day with a depth of 1.5 m. Also calculate the head loss in the intake conduit if the treatment works are $\frac{1}{4}$ km away. Draw a neat sketch of the canal intake. Given average consumption per person = 150 l/d. Assume the velocity through the screens and bell mouth to be less than 16 cm/s and 32 cm/s, respectively.*

(Civil Services, 1984)

Solution.

Discharge required by the city

$$= 1500 \times 75000 \text{ l/d} = 11250000 \text{ l/d} = 11.25 \text{ Ml/d}.$$

Since the canal runs only for 10 hrs a day, this whole daily flow is required to be drawn in 10 hrs.

∴ Intake load per hr

$$= \frac{11.25}{10} \text{ Ml/hr} = 1.125 \text{ Ml/hr}$$

$$= \frac{1.125 \times 10^6}{10^3} \text{ m}^3\text{/hr}$$

$$= \frac{1.125 \times 10^3}{60 \times 60} \text{ m}^3\text{/sec} = 0.3125 \text{ m}^3\text{/sec}.$$

Design of Coarse Screen

Area of coarse screen (which may be made as vertical iron bars of 20 mm ϕ @ 3 to 5 cm centre to centre),

$$= \frac{\text{Discharge entering the well through this coarse screen}}{\text{Velocity through the screen}}$$

Max. velocity through this screen is 0.16 m/sec.

∴ Min. area of screen required

$$= \frac{0.3125}{0.16} \text{ m}^2 = 1.95 \text{ m}^2$$

Ht. of screen provided

$$= 1.5 - 0.15 - 0.3^* = 1.05 \text{ m}$$

∴ Min. length of screen (openings) required

$$= \frac{1.95 \text{ m}^2}{1.05 \text{ m}} = 1.86 \text{ m}$$

Now, assuming the clear opening widths between vertical bars to be 30 mm (0.03 m) each, we have

the No. of openings $= \frac{1.86}{0.03} = 62$

∴ No. of bars $= 61$

Length occupied by bars of 20 mm ϕ

$$= 61 \times 0.02 \text{ m} = 1.22 \text{ m}$$

*Assuming the min. water level in canal to be 0.3 m below FSL, as shown in Fig. 5.10, thereby keeping the top of screen at 1.5—0.3 m above bed level.

$\therefore$ Total length of screen $= 1.86 + 1.22$

$= 3.08$ m ; say 3.1 m.

Hence, provide 3.1 m length of coarse screen of height 1.05 m in the rectangular intake well, as shown in Fig. 5.10. **Ans.**

Design of Bell mouth entry

Area of bell mouth entry

$$= \frac{\text{Discharge}}{\text{Velocity through the bell mouth}} = \frac{0.3125 \text{ m}^3/\text{sec}}{0.32 \text{ m/sec (given)}} = 0.98 \text{ m}^2.$$

If d is the dia. of the bell mouth entry, then

$$\frac{\pi}{4} \cdot d^2 = 0.98$$

$$\therefore \quad d = \sqrt{\frac{0.98 \times 4}{\pi}} = 1.12 \text{ m ; say } 1.2 \text{ m.}$$

Hence, use 1.2 m ϕ bell mouth provided with perforations for fine screen. **Ans.**

Design of Intake conduit

Assuming the flow velocity in conduit as 1.5 m/sec, we have

The area of conduit required

$$= \frac{\text{Discharge}}{\text{Velocity}} = \frac{0.3125}{1.5} \text{ m}^2 = 0.208 \text{ m}^2.$$

Dia. of pipe (D) is given by

$$\frac{\pi}{4} D^2 = 0.208$$

$$\therefore \quad D = \sqrt{\frac{0.208 \times 4}{\pi}} = 0.515 \text{ m ; say } 0.5 \text{ m}$$

We may hence use 0.5 m dia. conduit. **Ans.**

Flow velocity through this 0.5 m dia. conduit will then be

$$= \frac{0.3125}{\frac{\pi}{4} \times (0.5)^2} = 1.59 \text{ m/sec.}$$

Head loss through the conduit upto treatment works is calculated by equation (6.12), which states that

$$V = 0.85 \, C_H \cdot R^{0.63} \cdot S^{0.54}$$

where C_H = Coeff. of the pipe material

= 130 for cast iron pipe (from table 6.2)

R = Hydraulic mean depth of pipe

$= \frac{d}{4}$ (for pipes running full)

$= \frac{0.5}{4} = 0.125$ m

V = Flow velocity through the pipe

= 1.59 m/sec

S = slope of the energy line

$$\therefore \quad 1.59 = 0.85 \times 130 \times (0.125)^{0.63} \cdot S^{0.54}$$

or
$$S^{0.54} = \frac{1.59}{0.85 \times 130 \times (0.125)^{0.63}} = 0.0533$$

$$\therefore \quad S = (0.0533)^{\frac{1}{0.54}} = (0.0533)^{1.85} = 4.41 \times 10^{-3}$$

Now
$$S = \frac{H_L}{L} = \frac{\text{Head loss}}{\text{Length of pipe}}$$

$$\therefore \quad H_L = L \cdot S = \left(\frac{1}{4} \times 1000 \text{ m}\right) \times 4.41 \times 10^{-3}$$

$$\left[\because \quad L = \frac{1}{4} \text{ km} = \frac{1}{4} \times 1000 \text{ m}\right]$$

= 1.1025 m. Ans.

Fig. 5.10 shows the detailed sketch of this designed canal intake. **Ans.**

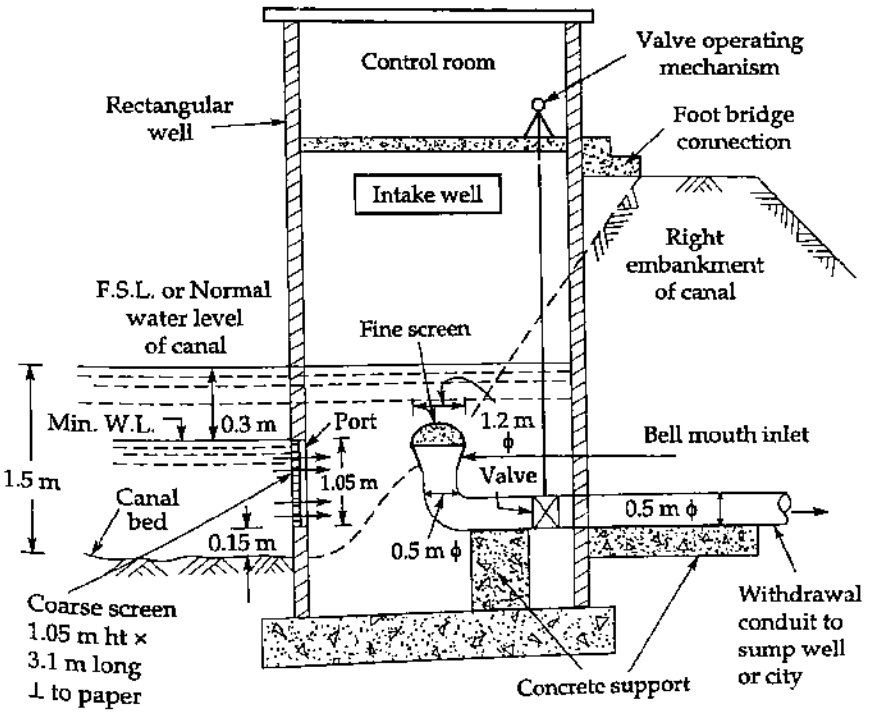


Fig. 5.10. Canal Intake Well for Example 5.2.

5.6. Intakes for Sluice-ways of Dams

In case a reservoir is developed by constructing a dam, it is a common practice to fetch water through the sluiceways* from inside the dam. This water may be used for water supplies if the reservoir is a single purpose water supply reservoir, or for multipurposes if the reservoir is a multipurpose reservoir.

In case of earthen dams, the intake for withdrawing water into the sluiceway of the dam, is generally located near the upstream toe of the dam, as shown in Fig. 5.11. Whereas for masonry dams, the intake well for withdrawing water is generally located inside the body of the dam, as shown in Fig. 5.12. However, sometimes, even for earthen dams provided with central impervious cores, the intake wells or **valve towers** (as they are called) have also been located within the body of the dam, as shown in Fig. 5.13.

In all these arrangements, the intake conduits installed at different levels, let the water enter to a common conduit (or a well or a tower) which conveys the water to the sluiceway tunnel passing through the body of the dam. The intake conduits are provided with valves which may be closed or opened by operating them from the control room, as and when desired, so as to withdraw water from any permissible desired level. The arrangement is similar to that of a dry intake tower described earlier, with the only difference that it was surrounded by water on all sides and was standing in the river, whereas this valve tower is fed through conduits and is standing in the dam or very near the dam.

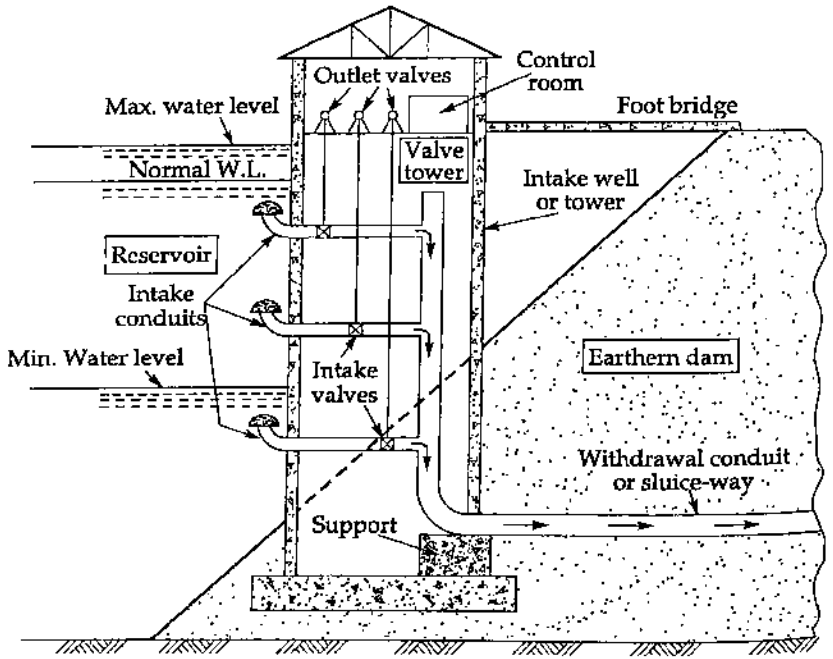


Fig. 5.11. Valve tower situated at the upstream toe of an earthen dam.

*Openings in the dam body.

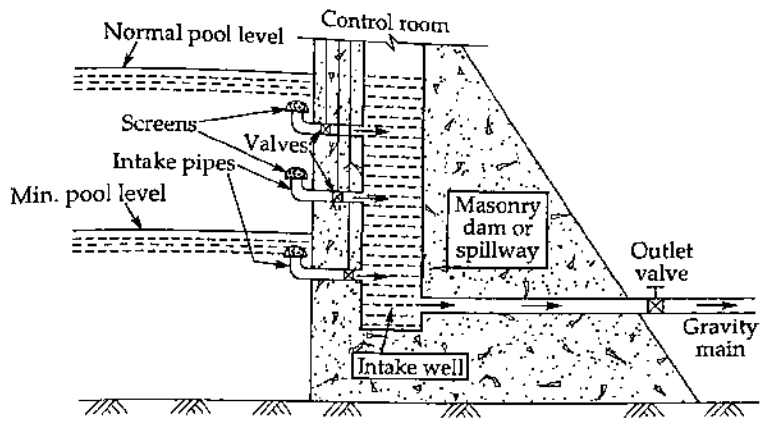


Fig. 5.12. Intake well or Valve tower for concrete gravity or masonry dams.

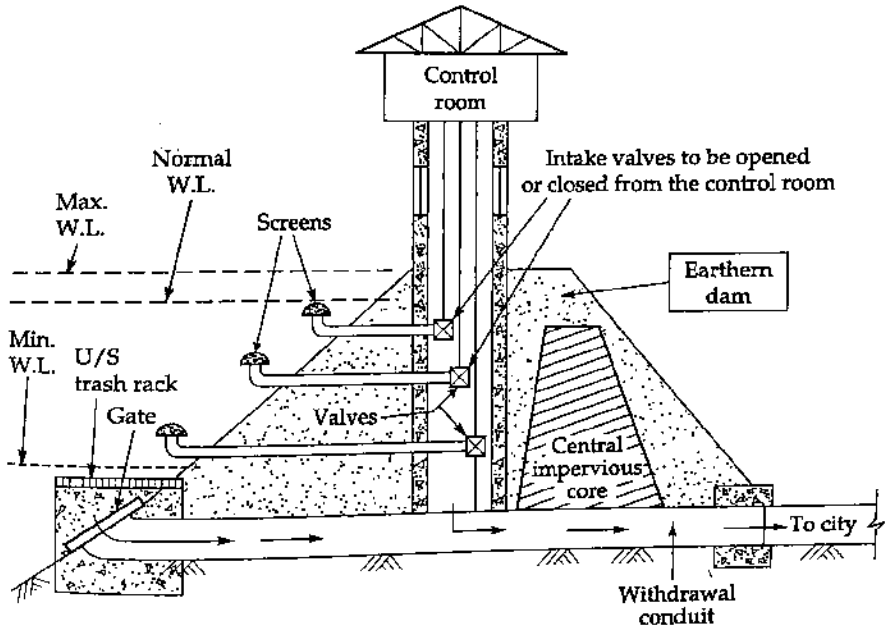


Fig. 5.13. Valve Tower situated within an earthen dam.

PROBLEMS

1. (*a*) What is a 'river intake' ? What are the factors which govern the location of an intake structure on a meandering river ?
   (*b*) Draw a neat sketch showing the intake arrangements which you suggest for drawing water for a city water supply plant being fed direct from a 'river under-sluices'.

   [**Note.** Suggest Twin Well Type—Medium Sized River Intake, See Figs. 5.5, 5.6 and 5.8.]

2. What is an 'intake structure' ? Enumerate the various types of intakes, and discuss in details any two of them.
3. (*a*) What are intake towers ? Differentiate between 'dry' and 'wet' intake towers.
   (*b*) Draw neat sketches of valve towers constructed in gravity and earthen dams.
4. Write brief notes with neat sketches on the following :
   (*i*) Simple submerged intake ;
   (*ii*) Dry intake tower ;
   (*iii*) An intake well for a canal ;
   (*iv*) Valve tower standing within an earthen dam ;
   (*v*) A typical river intake.

—6

Conduits for Transporting Water

6.1. Definition and General Introduction

As pointed out earlier, the entire water supply scheme consists of collecting water from the source or sources, carrying it or transporting it to the treatment plant in the city, and finally distributing the treated water among the consumers. In case of surface sources, the water is collected by means of an intake structure described in the previous chapter, and then transported to the city either raw in case the treatment plant is located within the city ; or transported after treatment in case the treatment plant is located away from the city near the source. Generally, the treatment plant is located within the city limits and, therefore, raw water after being collected, is conveyed to the city by means of conduits.

When the source of supply is near to the city limits, such as in Delhi, where Yamuna is in close proximity of city limits, the problem of transportation or transmission is less, as the length of the conduits required for carrying water from the intake to the treatment plant or city is less. However, when water is brought from far off sources, the length of conduits required and the resultant expenditure on transporting water may be enormous. Say for example, in Mumbai, the water is being brought from *Tanna, Vihar and Tulsi* lakes which are far away from the city. Similarly, in Delhi, as Yamuna supplies are insufficient, "the Delhi Water Supply and Sewage Disposal Undertaking" (now renamed as "Delhi Jal Board") has laid conduits in 14 km length to bring Ramganga reservoir waters from Moradnagar in U.P. to its Bhagirathi Water Treatment Plant in Trans Yamuna area.

In case of underground sources, such as, wells and tube wells (where no separate intake is required but lift is reqd.), the problem of conveying water from the source to the city does not normally arise as these sources are generally located within the cities. However, in case of infiltration wells and Rainy wells situated near the river banks, the water has to be transported as in the case of surface sources.

Further, if the city is situated at an elevation higher than the elevation of the source (strictly speaking the elevation of the lowest water level of the source), the water will have to be pumped and carried through pressure pipes. A low lift is generally required when the water is withdrawn from the rivers. On the other hand, if the source is available at higher elevation than the city, the water can be conveyed without lifting, through open or closed conduits either by simple gravity or under pressure. This is generally possible in case the water is withdrawn from storage reservoirs or lakes. For example, in Mumbai, the water is withdrawn from the lakes without any pumping.

6.2. Various Types of Conduits

Depending upon the conditions and characteristics of flow, the conduits may be divided into :

(1) *Gravity conduits* ; and

(2) *Pressure conduits.*

These are described below :

6.2.1. Gravity conduits. Gravity conduits are those in which the water flows under the mere action of gravity. In such a conduit, the *hydraulic gradient line* will coincide with the water surface and will be parallel to the bed of the conduit, as shown in Fig. 6.1. This is so, because in such a flow, the water is all along at atmospheric pressure, and thus there is no pressure term in Bernoulli's equation.

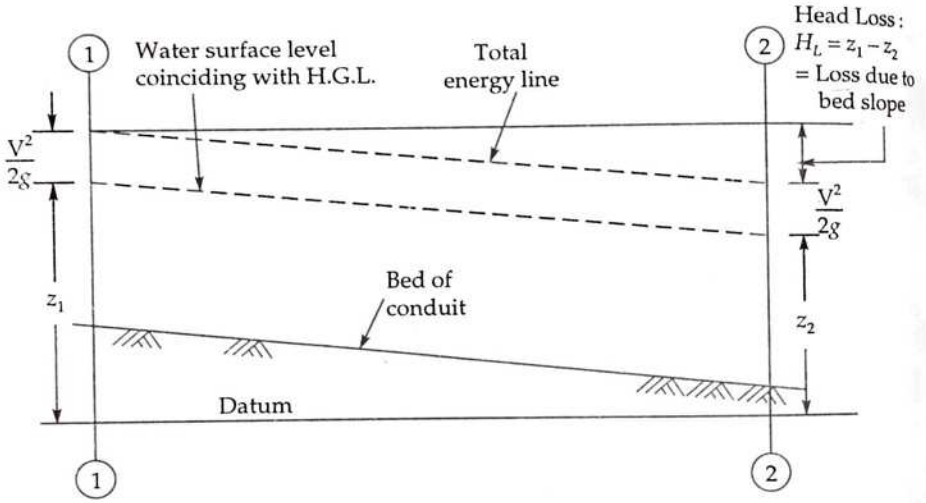


Fig. 6.1. Flow illustration in a Gravity Conduit.

Since the bed of such a conduit has to follow the gradual slope of the hydraulic gradient line (which governs the flow velocities, etc.), the *gravity* or *grade* conduits will, therefore, have to follow this small gradual slope and cannot follow the available natural slope. *These conduits cannot, therefore, go up and down hills and valleys as desired by the existing topography of the area. Hence, the gravity conduits will have to be taken above the NSL on trestles, or pillars in valleys and depressions ; or will have to be taken below the NSL through tunnels or through canals in deep cuttings.* Alternatively, they may be carried along zig-zag paths like roads, highways, etc. thus requiring enormous length of conduit and increased cost.

Gravity conduits can be in the form of *canals, flumes*, or *aqueducts*, as given below :

(*i*) **Canals.** Canals are the open channels which are constructed by cutting high grounds and constructing banks on low grounds. They are generally constructed in balanced cut and fill, and are cheap to build in suitable soils. They may be either lined or unlined depending upon the nature of the ground,

available slopes, design velocities, quality of water required, losses of water, etc. However, they are generally not used for water supplies but are often used for irrigation. When used for water supplies, they have to be lined so as to obtain better quality water. The design and other details of lined as well as unlined canals are available in "*Irrigation Engg. and Hydraulic Structures*" written by the same author, and may be referred to in specific needs.

(*ii*) **Flumes.** Open channels supported above the ground over trestles, etc., are called *flumes*. They are used to convey water across valleys and minor depressions or over drains and other obstructions in their path. They may be made of masonry, R.C.C., metal or wood, and are usually circular or rectangular in cross-section.

(*iii*) **Aqueducts.** Aqueducts or strictly speaking *grade aqueducts* are closed, rectangular or circular or horse shoe sections, built of masonry or R.C.C. They wind their way through landscape, with gradual slopes, like roads, railways and highways. *Although they are covered or closed, yet water does not flow under pressure as they are not allowed to run full.* They are generally designed $\frac{1}{2}$ or $\frac{3}{4}$th full. The aqueducts which have been designed as grade aqueducts should not run full under pressure, because the tension developed under such circumstances may open out the joints of masonry work endangering the structural stability, thus causing serious leakage of water. Inspite of being closed, their waters are susceptible to contamination, as the pollution can seep into their waters through cracks and leaky joints. The shape of the section to be chosen is discussed below.

From hydraulic point of view, a ***circular section* provides the maximum hydraulic mean depth** or maximum area per unit of wetted perimeter $\left(i.e.\ R = \frac{A}{P}\right)$ ***and is, therefore, the most efficient.***

Moreover, since the perimeter per unit of cross-section is the least, the construction cost or material required is also the least. But circular sections cannot be easily supported on ground and, therefore, *seldom used.* A *rectangular shape* is hydraulically inefficient and requires more material for construction, but is more stable to support on the ground. Rectangular sections are, therefore, widely used. *Horse shoe* shape, as shown in Fig. 6.2, is a compromise between the circular and rectangular shapes, and may be used with great advantage. However, their construction is a little more difficult.

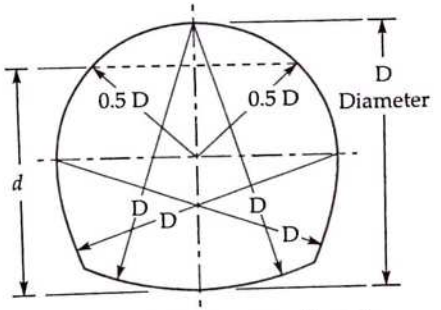


Fig. 6.2. Common proportions of Horse shoe Section.

6.2.2. Pressure Conduits. In pressure conduits, which are closed conduits and as such no air can enter into them, the water flows under pressure above the atmospheric pressure. The hydraulic gradient line for such a conduit can

be obtained by joining the water surface elevations in the piezometers installed in the conduit at various places, as shown in Fig. 6.3. The bed or the invert of the conduit in pressure flows is thus independent of the grade of the hydraulic gradient line which really governs the flow velocities.

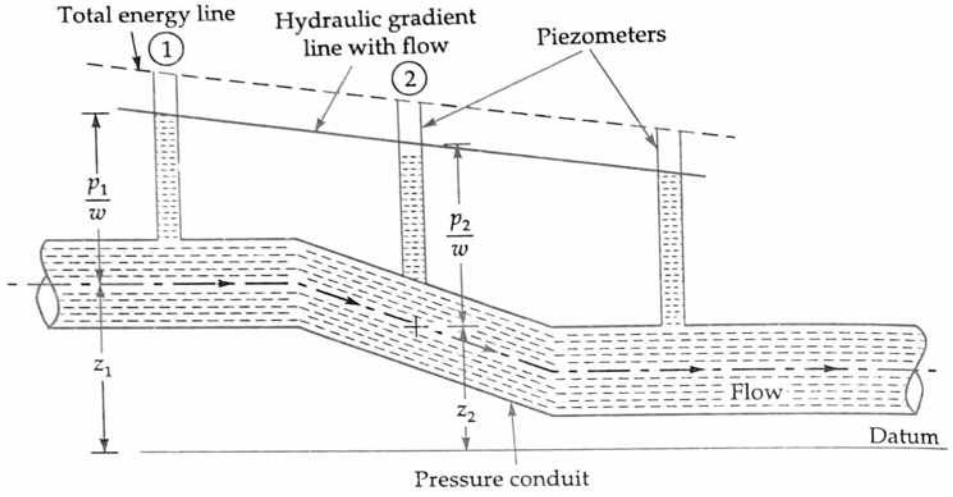


Fig. 6.3. Flow illustration in a Pressure Conduit.

The pressure pipes can, therefore, follow the natural available ground surface and can freely go up and down hills or can dip beneath valleys or mountains, sometimes even rising above the hydraulic gradient lines and thus requiring lesser length of conduit.*

The pressure aqueducts may be in the form of closed pipes or closed aqueducts and tunnels called *pressure aqueducts* or pressure tunnels designed for the pressure likely to come on them and for various other forces, as discussed a little later. *The aqueduct as well as tunnel sections are generally kept circular for hydraulic as well as structural reasons.* Due to their circular shapes, every pressure conduit is generally termed as a *pressure pipe*. When a pressure pipe drops beneath a valley, stream, or some other depression, it is called a *sag* or a *depressed pipe* or an *inverted siphon*.

As pointed out earlier, the pressure pipes prove economical than canals or flumes because they can generally follow shorter routes. However, their biggest advantage is : that the water moving through such a conduit is not exposed anywhere, and hence, there are no chances or very less chances of its getting polluted. *The pressure pipes are, therefore, invariably and universally used for water supplies, whereas gravity conduits are generally adopted for carrying sewage and drainage.* Since the water wasted in percolation, evaporation, etc. in the canals, also gets saved in the pressure conduits, they are preferably used when the water is scarce.

*The **negative pressure** will be created in such a case, and since there is a limit to allowable negative pressure due to cavitation, etc., encroachment upto a maximum of 8.3 m is generally permitted.

6.3. Hydraulics of Flow and Design of Pressure Pipes as Gravity Mains

Although the pressure pipes can be laid at any depth below the hydraulic gradient line, yet for economic considerations they should generally hug the hydraulic gradient line in profile and the straight line in plan. If the pipe goes much below the hydraulic gradient line, naturally the pressure developed* in the pipe may be too high to be borne by an ordinary pipe material and will necessitate the construction of thicker and stronger pipes, which may create enormous constructional difficulties and increased costs. It, therefore, follows that the shortest route, which may create too high pressures in the pipes, may not always prove to be the cheapest. Hence, the size and shape of the supply pipes must be determined by hydraulic, structural, and economical considerations.

Further, the hydraulic gradient should be such as to generate velocities which are neither so small as to require large size diameter pipe, nor so large as to cause excessive loss of pressure head. The velocities should also be non-silting and non-scouring. The flow velocities are normally kept between 0.9 m/sec to 1.5 m/sec, though velocities upto 3 m/sec to 6 m/sec can be resisted by the commonly available pipes or pipe materials. *It is a normal practice to design these pipes in such a way that the available pressure head between the source and the city is just lost in overcoming the frictional resistance offered to the flow by the pipe interior.* Although, in addition to the head lost due to pipe friction (represented by H_L), there are minor losses caused by the abrupt changes in the flow geometry as a result of changes in pipe size, bends, valves and fittings of all types ; however, in long pipes such as used for conveying water from the source to the city, these minor losses (represented by H_{Lm}) can often be neglected. The **head loss** caused by pipe friction can be found by using either of the following formulae :

(1) *Darcy-Weisbach formula* ;

(2) *Manning's formula* ;

(3) *Hazen-William's formula* ; and

(4) *Modified Hazen-William's formula.*

Out of the above formulas, the Hazen-William's formula is most widely used. The use of these formulas is explained below :

6.3.1. Darcy-Weisbach formula. It is more than 100 years old formula, and states that

$$H_L = \frac{f'.L}{d} \cdot \frac{V^2}{2g} \qquad ...(6.1)$$

where H_L = Head loss in metres

L = Length of pipe in metres

d = Diameter of the pipe in metres

V = Mean velocity of flow through the pipe in m/sec.

*The pressure developed in the pipe when flow is taking place is given by the ordinate of the hydraulic gradient line from the centre of the pipe.

g = Acceleration due to gravity

f' = Dimensionless **friction factor*** generally varying between 0.02** (for new smooth pipes) to 0.075 (for old rough pipes), and depends upon Reynold number $\left(i.e.\ R_e = \frac{Vd}{\nu}\right)$ and relative roughness of the pipe. The relative roughness $\left(i.e.\ \delta = \frac{2e}{d}\right)$ of a pipe depends upon the absolute roughness (e) of the inside surface and the diameter of the pipe d. The approximate values of f' are given by the following empirical relations.

$$f' = 0.04\left[1+\frac{1}{35d}\right] \text{ for old pipes} \qquad ...(6.2)$$

$$= 0.02\left[1+\frac{1}{35d}\right] \text{ for new pipes} \qquad ...(6.3)$$

The accurate value of f' depends upon R_e and δ, and may be given by a formula, such as

$$f' = a + \frac{b}{R_e^m} \qquad ...(6.4)$$

where a, b and m are constants depending upon δ.

(*i*) ***For laminar flows, i.e. upto R_e = 2,000;*** the relation is simple

i.e.
$$f' = \frac{64}{R_e} \qquad ...(6.5)$$

(*ii*) ***Above R_e = 4,000, turbulent flow is fully established*** and the relation suggested by Nikuradse and others are :

(*a*) $$\left[\frac{1}{\sqrt{f'}} = 2\log_{10} R_e\sqrt{f'} - 0.8\right] \text{ for smooth pipes} \qquad ...(6.6)$$

(*b*) $$\left[\frac{1}{\sqrt{f'}} = 2\log_{10}\frac{d}{2e} + 1.74\right] \text{ for rough pipes} \qquad ...(6.7)$$

For smooth pipes and R_e varying between 20,000 to 20,00,000, Slitcher has given the formula

$$f' = 0.005 + \frac{0.396}{R_e^{0.3}} \qquad ...(6.8)$$

(*d*) Nickuradse has given for smooth pipes and for R_e varying between 20,000 to 32,40,000 as

*Certain authors represent this constant by f and certain others by $4f$. So to avoid confusion, we have used a symbol f'.

**Generally, an average value of 0.024 is considered for designs, which all uses new pipes.

$$f' = 0.0032 + \frac{0.221}{R_e^{0.237}} \qquad ...(6.9)$$

The design of pressure conduits is generally governed under these equations as the flow of water through pressure conduits is usually turbulent.

(*iii*) In the critical zone between R_e = 2000 and R_e = 4000, both R_e and $\frac{2e}{d}$ make their appearance, and the relation suggested by Colebrook and White is :

$$\frac{1}{\sqrt{f'}} = 1.74 - 2\log_{10}\left[\frac{2e}{e} + \frac{18.7}{R_e\sqrt{f'}}\right] \qquad ...(6.10)$$

The values of $2e$ for new pipes may be taken as given in Table 6.1.

Table 6.1

S. No.	*Material of pipe*	*Value of 2e in mm × 10^{-3}*
1.	Glass, copper, lead, or asbestos	1.524
2.	Steel or wrought iron	45.75
3.	Asphalted cast iron	122
4.	Galvanised iron	152
5.	Cast iron	300
6.	Concrete	300—3,000
7.	Riveted steel	900—9,000

Darcy-Weisbach formula, though very old, is *not generally preferred* by water engineers in solving the routine fluid transport problems encountered in water transmission lines and pipe networks, mainly because of uncertain values of $\frac{2e}{d}$ and f'. Even if e is known, d is not pre-known. A trial and error solution will, therefore, be required, making it complicated.

6.3.2. Manning's formula. Manning's formula, though generally used for gravity conduits, is also applicable to turbulent flow in pressure conduits and yields good results, provided the roughness coefficient n is accurately estimated*. Head loss, according to Manning's equation is given as :

$$H_L = \frac{n^2.V^2.L}{R^{4/3}} \qquad ..(6.11)$$

where n = Manning's rugosity coefficient

L = Length of pipe in metres

V = Flow velocity through the pipe in m/sec

R = Hydraulic mean depth of the pipe.

$$= \left[\frac{A}{P} = \frac{\frac{\pi d^2}{4}}{\pi d} = \frac{d}{4}\right] \text{ in metres.}$$

*n increases as R increases.

This formula has been discussed in details in the design of sewers in the author's another volume on "Sewage Disposal and Air Pollution Engg.", because it is generally used for sewer designs.

6.3.3. Hazen-William's formula. This formula is widely used for pipe flows and states that

$$V = 0.85 C_H \cdot R^{0.63} \cdot S^{0.54} \qquad ...(6.12)$$

where C_H = Coefficient of hydraulic capacity of the pipe material, given in Table 6.2. The smoother the pipe, the greater is the value of C_H. C_H is termed as **Hazen-Williams' Coefficient.**

R = Hydraulic mean depth of the pipe in metres

$= \left(\frac{d}{4}\right)$ for circular pipes flowing full.

S = Slope of the energy line

V = Flow velocity through the pipe in m/sec.

Table 6.2. Values of C_H for Hazen Williams' formula (Eq. 6.12)

Pipe material	*Value of C_H (in MKS units) Depending upon the smoothness of the pipe material*
Concrete (regardless of age)	130
Cast Iron	
New	130
5 years old	120
20 years old	100
Welded steel (New)	120
Riveted steel (New)	110
Vitrified clay	110
Brick Sewers	100
Asbestos-Cement	140

The carrying capacities of pipes decrease with time, because with the passage of time their surfaces go on becoming more and more rough. Hence, it is customary to select higher values of e and n and lower values of C_H for old pipes than are generally taken for new pipes.

6.3.4. Modified Hazen-William's Formula. The most widely used Hazen-William's formula is sometimes not preferred by certain designers, because it suffers from the following limitations :

(*i*) The Hazen-William's coefficient C_H is not dimensionless but has the units of $L^{-0.37}\ T^{-1}$. Hence, its value changes with change in the employed units.

(*ii*) The numerical constant of 0.85 (in MKS units) in Hazen-William's formula has been calculated for an assumed hydraulic mean depth (R) of 0.3 m (1 ft) and friction slope of 1/1000. However, the formula is used for all ranges of pipe dia and friction slopes. This practice may lead to an error up to ± 30% in the evaluation of velocity ; and ± 55% in head loss (H_L) due to pipe friction.

(*iii*) The Hazen-William's coefficient, which should as a matter of fact, be a representative of friction conditions of a pipe and hence depend on pipe roughness and Reynold's number, does infact considered as independent of pipe dia, velocity of flow and viscosity. When compared with Darcy-Weisbach formula (which is dimensionally consistent), ± 45% error is found between estimates of Darcy-Weisbach friction factor f', and its equivalent value computed from Hazen-William's C_H for different pipe materials. It has been observed that for higher C_H values (new and smooth pipes) and larger diameters, the error is less ; whereas it is appreciable for lower values of C_H (old and rough pipes), and lower diameters at higher velocities.

Considering these limitations of Hazen-William's formula, a modified Hazen-William's formula has been derived for use from Darcy-Weisbach and Cole-brook-White equations, which obviates those limitations. ***The Modified Hazen-William's formula states that :***

$$V = \frac{3.83\, C_R \left[d^{0.6575} (g.S)^{0.5525}\right]}{\nu^{0.105}} \qquad ...(6.13)$$

where C_R = dimensionless coefficient of roughness (values given in Table 6.3)

d = pipe diameter

g = acceleration due to gravity

S = friction slope = $\frac{H_L}{L}$

ν = viscosity of liquid.

Using ν for water at 20°C = 10^{-6} m^2/s

and $g = 9.81$ m/s^2

Eq. (6.13) reduces to :

$$\therefore \quad \boxed{V = 143.534\, C_R \cdot R^{0.6575} \cdot S^{0.5525}} \qquad ...(6.14)$$

where R = Hydraulic radius or Hydraulic mean depth

$= \frac{A}{P} = \frac{d}{4}$ (for circular pipes)

S = Friction slope = $\frac{H_L}{L}$.

When S is substituted by $\frac{H_L}{L}$, $R = \frac{d}{4}$ and $V = \frac{Q}{\frac{\pi}{4} d^2}$ for circular conduits, the above eqn. yields

$$H_L = \text{Friction head loss} = \frac{L \cdot \left(\frac{Q}{C_R}\right)^{1.81}}{994.62 \cdot d^{4.81}} \qquad ...(6.14a)$$

where d = Internal dia of pipe in m

Q = Flow in pipe.

Table 6.3. Recommended C_R Values in Modified Hazen-William's Formula at 20°C

S. No.	Pipe Material	Diameter (mm)		Velocity m/s		C_R value when new	C_R value for design period of 30 yrs.
		From	To	From	To		
1.	R.C.C	100	2000	0.3	1.8	1.0	1.0
2.	A.C.	100	600	0.3	6.0	1.0	1.0
3.	HDPE/PVC	20	100	0.3	1.8	1.0	1.0
4.	C.I. (for water with positive Langelier's index)	100	1000	0.3	1.8	1.0	0.85*
5.	C.I. (for water with negative Langelier's index)	100	1000	0.3	1.8	1.0	0.53*
6.	Steel (for waters with negative Langelier's index)	100	2000	0.3	2.1	1.0	0.73*
7.	SGSW	100	600	0.3	2.1	1.0	1.0
8.	G.I. (for waters with positive Langelier's index)	15	100	0.3	1.5	0.87*	0.74*

*These are average C_R values which result in a maximum error of ± 5% in estimation of surface resistance.

A few examples have been solved below to explain the design of pressure pipes when used for conveying water from the source to the city.

Example 6.1. *Estimate roughly the sizes of supply conduits leading to an adequate service reservoir, serving*

(a) a relatively small town of 25,000 population.

(b) a relatively large city with industrial establishments having a population of 5 lakh people.

Also find the hydraulic gradients at which the pipe lines are proposed to be laid. Assume any suitable data according to Indian conditions, where required.

Solution. Assume average daily water consumptions as follows :

(a) for town of 25,000 population = 120 litres/capita/day

(b) for city with industrial establishments having a population of 5 lakhs = 270 litres/capita/day.

Average quantity of water required.

(a) for town = 120 × 25000 litres/day
= 3 million litres/day = 3 M.l.d.

(b) for city = 270 × 5,00,000 litres/day = **135 M.l.d.**

Assuming maximum daily demand at 1.8 times the average, the maximum quantity of water required :

(a) for town = 3 × 1.8 = 5.4 M.l.d.

$$= \frac{5.4 \times 10^6}{10^3 \times 24 \times 60 \times 60} \text{ cumecs} = 0.063 \text{ cumecs}$$

(b) for city = 135 × 1.8 = 243 M.l.d.

$$= \frac{243 \times 10^6}{10^3 \times 24 \times 60 \times 60} \text{ cumecs} = 2.81 \text{ cumecs.}$$

Assuming a flow velocity of 1.2 m/sec through the circular conduit, we get

$$Q = A.V$$

(a) for town, $Q = 0.063 = A \times 1.2$

$$\therefore \quad A = \frac{0.063}{1.2} \text{ m}^2 = 0.0525 \text{ m}^2$$

or Area of pipe required = 525 cm²

∴ Diameter of pipe required = 24.9 cm.

use 25 cm dia pipe. Ans.

(b) for city $Q = 2.81 = A \times 1.2$

$$\therefore \quad A = \frac{2.81}{1.2} = 2.36 \text{ m}^2$$

∴ Diameter of pipe required = 1.74 m.

use 1.8 m dia pipe. Ans.

Hydraulic gradient by the Hazen-William's formula

Using equation (6.12), and $C_H = 110$, we have

$$V = 0.85\, C_H . R^{0.63} . S^{0.54}$$

$$= 0.85 \times 110 \times \left(\frac{d}{4}\right)^{0.63} S^{0.54}$$

$$= 93.5 \left(\frac{d}{4}\right)^{0.63} S^{0.54}$$

(a) *for town*

$$V = 1.2 = 93.5 \left(\frac{0.25}{4}\right)^{0.63} S^{0.54}$$

or $$1.2 = 93.5\, (0.0625)^{0.63} . S^{0.54}$$

or $$1.2 = 93.5 \times 0.174\, S^{0.54}$$

or $$\frac{12}{93.5 \times 0.174} = S^{0.54}$$

or $$\left(\frac{1}{13.54}\right)^{\frac{1}{0.54}} = S$$

or $$S = \left(\frac{1}{13.54}\right)^{1.852} = \frac{1}{(13.54)^{1.852}} = \frac{1}{125}$$

Thus, the hydraulic gradient is $\frac{1}{125}$ *i.e.* 1 m fall in 125 m length. **Ans.**

(b) for city

$$V = 1.2 = 93.5 \left(\frac{1.8}{4}\right)^{0.63} S^{0.54}$$

or $1.2 = 93.5 \times (0.45)^{0.63} . S^{0.54}$

or $1.2 = 93.5 \times 0.605\, S^{0.54}$

or $S^{0.54} = \frac{12}{93.5 \times 0.605} = \frac{1}{47.2}$

or $S = \frac{1}{(47.2)^{1.852}} = \frac{1}{1260}$

Hence, the hydraulic gradient is $\frac{1}{1260}$ *i.e.* 1 m fall in 1260 m length. **Ans.**

Note. It proves that in smaller size pipes at equal velocities, the head loss is more. Hence, the cost of pumping will be increased by using a smaller size pipe, although the cost of the pipe itself will be reduced.

• **Example 6.2.** *Estimate the hydraulic gradient in a 2 m diameter smooth concrete pipe carrying a discharge of 3 cumecs at 10°C temperature by using (a) Darcy-Weisbach formula ; (b) by Manning's formula, (c) by Hazen-William's formula ; (d) by Modified Hazen-William's formula. Assume suitable data not given.*

Solution. $Q = 3$ cumecs.

Dia of pipe = 2 m

Area of pipe $= A = \frac{\pi}{4} d^2 = \frac{\pi}{4} \times 4 = 3.14$ m^2

$\therefore$ Velocity of flow $= V = \frac{Q}{A} = \frac{3}{3.14} = 0.955$ m/sec.

(a) Darcy-Weisbach formula (Eq. 6.1) states that

$$H_L = \frac{f' L}{d} . \frac{V^2}{2g} \quad ...(i.e.\ \text{Eq. } 6.1)$$

Let us evaluate f', first

Now, R_e = Reynold number $= \frac{Vd}{\nu}$

where ν = kinematic viscosity of water at 10°C from table 9.1

$= 1.31 \times 10^{-6}$ m^2/sec

$\therefore$ $R_e = \frac{0.955 \times 2}{1.31 \times 10^{-6}} = 1.46 \times 10^6 = 14{,}60{,}000$

Using Eq. (6.8), we have

$$f' = 0.005 + \frac{0.396}{R_e^{0.3}}$$

$\therefore$ $f' = 0.005 + \frac{0.396}{(14{,}60{,}000)^{0.3}} = 0.005 + \frac{0.396}{69.5}$

$= 0.005 + 0.0057 = 0.0107$; say 0.011.

When the value of f' is obtained by Eq. (6.6), we get

$$\left[\frac{1}{\sqrt{f'}} = 2 \log_{10} R_e \sqrt{f'} - 0.8\right] \quad ...(i.e.\ \text{Eq. } 6.6)$$

or $\frac{1}{\sqrt{f'}} = 2 \log_{10} (1.46 \times 10^6) \sqrt{f'} - 0.8$

or $\frac{1}{\sqrt{f'}} = 2 \times 6.14 \sqrt{f'} - 0.8$

or $1 = 12.28 f' - 0.8 \sqrt{f'}$

or $f' - 0.0652 \sqrt{f'} - 0.0815 = 0$

or $\sqrt{f'} = \frac{0.0652 \pm \sqrt{0.00425 + 0.326}}{2} = \frac{0.0652 \pm \sqrt{0.3302}}{2}$

$= \frac{0.0652 \pm 0.514}{2}$ (Ignoring unfeasible negative sign)

$= \frac{0.639}{2} = 0.32$

$\therefore$ $f' = 0.01 + 0.011$, as given by equation 6.8

Hence use $f' = 0.011$.

Now using Eq. (6.1), we have

$$H_L = \frac{f' L}{d} \frac{v^2}{2g}$$

or $H_L = \frac{0.011 \times L}{2} \frac{(0.955)^2}{2 \times 9.81}$

or $\frac{H_L}{L} = \frac{0.011}{2} \times \frac{0.911}{19.62} = \frac{0.011 \times 0.911}{39.24} = \frac{1}{3920}$

Thus, the hydraulic gradient is $\frac{1}{3920}$, *i.e.*, 1 m fall in 3920 m length. **Ans.**

(b) Using Manning's formula i.e. Eq. (6.11)

we have $H_L = \frac{n^2 V^2}{R^{4/3}} . L$

Using $n = 0.013$, $R = \frac{d}{4} = 2/4 = 0.5$ m, we get

$$\frac{H_L}{L} = \frac{(0.013)^2 \times (0.955)^2}{(0.5)^{1.33}}$$

$$= \frac{0.000169 \times 0.911}{0.397} = \frac{1}{2480}$$

Thus, the hydraulic gradient is $\frac{1}{2480}$, *i.e.* 1 m fall in 2480 m length. **Ans.**

(c) Using Hazen-William's formula i.e. Eq. (6.12)

we have $V = 0.85\, C_H . R^{0.63} . S^{0.54}$

Using $C_H = 130$ (from Table 6.2)

we have $0.955 = 0.85 \times 130 \times \left(\frac{2}{4}\right)^{0.63} . S^{0.54}$

or $0.955 = 0.85 \times 130(0.5)^{0.63} . S^{0.54}$

$= 0.85 \times 130 \times 0.646 . S^{0.54}$

or $S^{0.54} = \frac{0.955}{0.85 \times 130 \times 0.646} = \frac{1}{74.8}$

or $S = \frac{1}{(74.8)^{\frac{1}{0.54}}} = \frac{1}{(74.8)^{1.852}} = \frac{1}{2950}$

Thus, the hydraulic gradient is $\frac{1}{2950}$, *i.e.* 1 m fall in 2950 m length. **Ans.**

(*d*) *Using Modified Hazen-William's formula* ; *i.e.* Eq. (6.14),

we have $V = 143.534\, C_R . R^{0.6575} . S^{0.5525}$

where $V = 0.955$ m/s (computed earlier)

$C_R = 1.0$ (from Table 6.3 for concrete pipe)

$R = \frac{d}{4} = \frac{2\text{m}}{4} = 0.5$

$S = ?$

Putting the values, we get

$0.955 = 143.534 \times 1 \times (0.5)^{0.6575} . (S)^{0.5525}$

or $0.955 = 143.534 \times 0.634\, S^{0.5525}$

or $S^{0.5525} = \frac{0.955}{143.534 \times 0.634} = 0.01049$

or $S = (0.01049)^{\frac{1}{0.5525}} = 2.6185 \times 10^{-4} = \frac{1}{3819}$

Thus, the hydraulic gradient by Modified Hazen-William's formula will be $\frac{1}{3819}$ *i.e.* 1 m fall in 3819 m length ; which value will be close to the value obtained from Darcy-Weisbach formula. **Ans.**

Example 6.3. *In a water supply scheme to be designed for serving a population of 4 lakhs, the storage reservoir is situated at 8 km away from the city and the loss of head from source to city is 16 metres. Calculate the size of the supply main by using Weisbach formula as well as by using Hazen's formula assuming a maximum daily demand of 200 litres per day per person and half of the daily supply to be pumped in 8 hours. Assume coefficient of friction for the pipe material as 0.012 in Weisbach formula, and $C_H = 130$ in Hazen's formula.*

Solution. Maximum daily per capita demand

= 200 litres/person/day

Population = 4,00,000.

Maximum water demand per day

= 4,00,000 × 200 litres/day

= 80 million litres/day = **80 Mld.**

Maximum water demand for which the supply main is to be designed

$= \frac{12}{8} \times 80$ Mld

(∵ half the daily supply is to be pumped in 8 hours)

= **120 Mld.**

$= \frac{120 \times 10^6}{10^3}$ cubic metres/day

$= \frac{120 \times 10^3}{24 \times 60 \times 60}$ cumecs = 1.39 cumecs

Now $Q = 1.39$ cumecs

$L = 8$ km = 8000 metres

$H_L = 16$ metres

$d = ?$

(*a*) *Using Darcy-Weisbach formula**

$H_L = \frac{f' . L}{d} . \frac{V^2}{2g}$

But $V = \frac{Q}{A} = \frac{Q}{\frac{\pi}{4} d^2}$

$\therefore \quad H_L = \frac{f' . L}{d} . \frac{\left[\frac{Q}{\frac{\pi}{4} d^2}\right]^2}{2g} = \frac{f' . L}{2gd} . \frac{16 . Q^2}{\pi^2 d^2}$

$= \frac{8}{g\pi^2} . \frac{f' . LQ^2}{d^5}$. Using $f = 0.012$ (given), we get

$16 = \frac{8}{9.81 \times (3.14)^2} . \frac{(0.012)(8000)(1.39)^2}{d^5}$

Solving, we get, $d^5 = 0.957$

or $d = (0.957)^{1/5} = 1.092$ m.

So, use the nearest standard available dia pipe (say 1.25 m). **Ans.**

(*b*) *Using Hazen-William's formula as :*

$V = 0.85\, C_H . R^{0.63} . S^{0.54}$

where $C_H = 130$ (given)

$R = \frac{d}{4}$ for circular pipe running full

$V = \frac{Q}{A} = \frac{1.39}{\frac{\pi}{4} d^2}$

*This is generally not adopted, as the value of f' cannot be correctly determined unless the dia of the pipe is pre-known and, thus, involves trial.

$$\therefore \quad \frac{1.39}{\frac{\pi}{4}d^2} = 0.85 \times 130 \left(\frac{d}{4}\right)^{0.63} \cdot \left(\frac{H_L}{L}\right)^{0.54}$$

$$= 0.85 \times 130 \times \frac{(d)^{0.63}}{2.4} \left(\frac{16}{8000}\right)^{0.54}$$

or $$\frac{1.77}{d^2} = 46\, d^{0.63} \frac{1}{28.7}$$

or $$\frac{1.77 \times 28.7}{46} = d^{2.63}$$

or $$d^{2.63} = 1.104$$

or $$d = (1.104)^{\frac{1}{2.63}} = 1.038 \text{ m}$$

So, use the nearest standard available dia pipe (say 1.25) m dia. **Ans.**

Example 6.4. *Water has to be supplied to a town with one lakh population at the rate of 150 litres per capita per day from a river, 1.8 km away. The difference in elevation between the lowest water level in the sump and service reservoir is 36 metres. Determine the size of the main and the horse power of the pump required. Assume suitable data where necessary.*

Solution. Per capita demand

= 150 litres per day.

Population = 1 lakh

Average quantity of water required

= 150 × 1,00,000 litres per day

= 15 million litres per day = 15 Mld.

Assuming the maximum demand to be 1.8 times the average annual demand, we have

Maximum demand = 15 × 1.8 Mld = 27 Mld

$$= \frac{27 \times 10^6}{10^3} \text{ cubic metres per day}$$

$$= \frac{27 \times 10^3}{24 \times 60 \times 60} \text{ cubic metres per sec.} = 0.312 \text{ cumecs.}$$

Further, assuming that the pumps are working for 12 hours a day to supply full days demand, we have

Maximum draft required (Q)

$$= 0.312 \times \frac{24}{12} = 0.624 \text{ cumecs.}$$

Now, assuming the flow velocity through the pressure pipe to be 1.5 m/sec, we have

Area of the pipe required

$$= A = \frac{Q}{V} = \frac{0.624}{15} \text{ m}^2 = 0.416 \text{ m}^2$$

$\therefore$ Dia of the pipe required = 0.728 m.

So use 0.75 m dia pipe.

$\therefore$ Actual area provided

$$= \frac{\pi}{4} \times (0.75)^2 = 0.441 \text{ m}^2$$

The actual velocity

$$V = \frac{Q}{A} = \frac{0.624}{0.441} = 1.42 \text{ m/sec.}$$

Head loss can now be calculated by using

$$V = 0.85\, C_H\, R^{0.63}\, S^{0.54}$$

Assuming $C_H = 120$,

$R = \frac{d}{4}$, we have

$$1.42 = 0.85 \times 120 \times \left(\frac{0.75}{4}\right)^{0.63} S^{0.54}$$

$$= 0.85 \times 120 \times 0.348\, S^{0.54}$$

or $$1.42 = 35.5\, S^{0.54}$$

or $$S^{0.54} = \frac{1.42}{35.5} = 0.04$$

or $$S = (0.04)^{\frac{1}{0.54} = 1.85} = 0.0026$$

or $$\frac{H_L}{L} = 0.0026$$

or $$H_L = 0.0026L$$

$\therefore$ Head loss from source to service reservoir in a length of 1.8 km (or 1800 m)

$$= 0.0026 \times 1800 = 4.68 \text{ m.}$$

The head difference between the elevation of sump well and service reservoir = 36 m (given).

The head loss in supply main = 4.68 m (calculated above)

Total head lift required

$$H = 36 + 4.68 = 40.68 \text{ m.}$$

The power of the motor reqd.

$$= \frac{\gamma_w . Q . H}{\eta} \text{ k.Nm/sec } (i.e. \text{ kW})$$

where γ_w = unit wt. of water in kN/m^3 = 9.81 kN/m^3

Q = discharge in m^3/s

H = Total head lift reqd. in m

η = Efficiency of pump set.

Brake horse power of pumps required (metric) is, hence, given as

$$\text{H.P.} = \frac{\gamma_w . Q . H}{0.735\eta}$$

$\therefore$ 1 Metric H.P. = 0.735 kW

the dia. of the pipes and the length of second pipe required to be laid. Make use of William Hazen's equation. Also derive an expression expressing H_L in terms of L and d from the original William Hazen's formula. Assume $C_H = 120$.

Solution. Initial discharge

$$= 24 \text{ M.l.d.}$$

$$= \frac{24 \times 10^6}{10^3} \text{ cubic metres per day}$$

$$= \frac{24 \times 10^3}{24 \times 60 \times 60} \text{ cumecs} = 0.278 \text{ cumecs.}$$

Now, the total available head between storage reservoir and service reservoir (*i.e.*, 150 metres) is assumed to have been dissipated in frictional resistance offered by the pipe to the flow ; so we have by using Hazen William's equation

$$V = 0.85\, C_H . R^{0.63}\, S^{0.54}$$

$$\text{where } S = \frac{H_L}{L} \frac{150}{50{,}000} = 0.003$$

$$V = \frac{Q}{A} = \frac{0.278}{\frac{\pi}{4} d^2}$$

$$C_H = 120$$

$$\therefore \quad V = \frac{0.278}{\frac{\pi}{4} d^2} = 0.85 \times 120 \left(\frac{d}{4}\right)^{0.63} \times (0.003)^{0.54}$$

or $$0.278 = \frac{\pi}{4} \times \frac{0.85 \times 120}{2.4} \times 0.044\, d^{2.63} = 1.47\, d^{2.63}$$

or $$d^{2.63} = \frac{0.278}{1.47} = 0.189$$

or $$d = (0.189)^{\frac{1}{2.63} = 0.38} = 0.53$$

$\therefore$ Dia of pipe used = 0.53 m. **Ans.**

Subsequently the discharge is increased to a value

$$= \frac{35 \times 10^6}{10^3 \times 24 \times 60 \times 60} \text{ cumecs} = 0.406 \text{ cumecs.}$$

Now, let a pipe of length equal to l km be added and laid alongside the already existing pipe of total length 50 km, as shown in Fig. 6.5.

The total discharge (*i.e.*, 0.406 cumecs) now flows through the original pipe for a length equal to $(50 - l)$ km, and then half of the discharge (*i.e.* 0.203 cumecs) will flow through each, as both the pipes are of equal diameter.

Equating the total head loss in this pipe system to the available head, we get

150 = Head loss in length $(50 - l)$ km with 0.406 cumecs
+ Head loss in length l km with 0.203 cumecs.

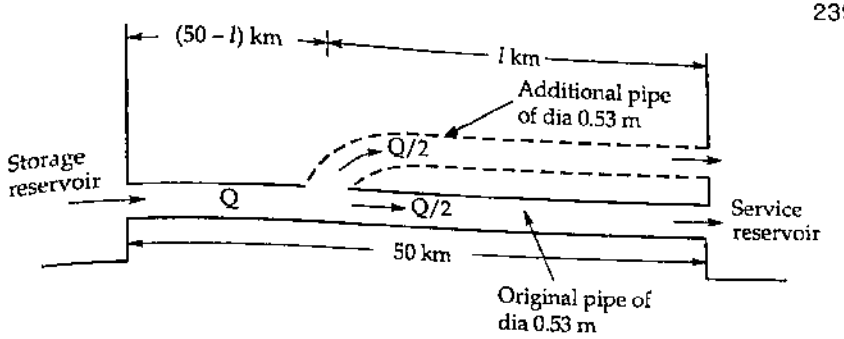


Fig. 6.5

To evaluate, it will be easy, if we first of all, deduce Hazen's formula in terms of $\frac{H_L}{L}$ and Q.

Now $$V = 0.85\, C_H R^{0.63} S^{0.54} \text{ ...(i.e. Eq. 6.12)}$$

or $$\frac{Q}{\frac{\pi}{4} d^2} = 0.85 \times C_H \times \left(\frac{d}{4}\right)^{0.63} . S^{0.54}$$

or $$Q = \frac{0.85 \times C_H}{2.4} \frac{\pi}{4} d^{2.63} . S^{0.54}$$

or $$Q = 0.278\, C_H\, d^{2.63} . S^{0.54}$$

or $$Q = 0.278\, C_H\, d^{2.63} . \frac{H_L^{0.54}}{L^{0.54}}$$

or $$H_L^{0.54} = \frac{Q . L^{0.54}}{0.278\, C_H . d^{2.63}}$$

or $$H_L = \frac{Q^{\frac{1}{0.54}} . L}{(0.278)^{\frac{1}{0.54}} (C_H)^{\frac{1}{0.54}} d^{\frac{2.63}{0.54}}}$$

or $$H_L = \frac{Q^{1.85} L}{(0.278)^{1.85} C_H^{1.85} d^{4.87}}$$

or $$H_L = \frac{Q^{1.85} . L}{0.094 . C_H^{1.85} . d^{4.87}}$$

or $$\boxed{H_L = \frac{1}{0.094} \left(\frac{Q}{C_H}\right)^{1.85} \frac{L}{d^{4.87}}} \quad \text{...[6.12 (a)]}$$

This is the required William Hazen's equation expressing H_L in terms of Q, L, d and constant C_H ; and may be remembered by students.

In this equation, we have

$$C_H = 120$$

$$\therefore \quad H_L = \frac{1}{0.094} \left[\frac{Q^{1.85}}{(120)^{1.85}}\right] \frac{L}{d^{4.87}}$$

or $$H_L = \frac{1}{670}\frac{Q^{1.85}.L}{d^{4.87}}$$

Substituting the values, we get

$$150 = \frac{(0.406)^{1.85}(50-l)(1000)}{670(0.53)^{4.87}} + \frac{(0.203)^{1.85}(l)(1000)}{670\times(0.53)^{4.87}}$$

or $$150 = \frac{1000}{670\times 0.046}\left[0.188\,(50-l) + 0.053\,l\right]$$

or $$\frac{150\times 670\times 0.046}{1000} = 9.4 - 0.188\,l + 0.053\,l$$

or $$4.62 = 9.4 - 0.135\,l$$

or $$0.135\,l = 9.40 - 4.62 = 4.78$$

or $$l = \frac{4.78}{0.135} = 35.4 \text{ km.}$$ **Ans.**

Hence, an addition of the same dia pipe is required in a length of 35.4 km. **Ans.**

Example 6.6. *A pipe line 0.6 m diameter is 1.5 km long. To augment the discharge, another pipe line of the same diameter is introduced parallel to the first in the second half of its length. Find the increase in discharge if $f = 0.04$ and head at the inlet is 30 m.* *(Engg. Services, 1987)*

Solution. The question is to be solved in the same manner as the previous one, with the only difference that here Darcy-Weisbach equation is to be used instead of Hazen-William's equation for discharge.

The Darcy Weisbach equation is

$$H_L = \frac{f'.L.V^2}{2gd} = \frac{f'.L.\left(\frac{Q}{\frac{\pi}{4}d^2}\right)^2}{2gd}$$

or $$\boxed{H_L = \frac{8}{g\pi^2}.\frac{f'L.Q^2}{d^5}} \qquad ...(6.1a)$$

In the first case, when a single pipe line is laid, if the discharge is Q_1, then

$H_L = 30$ m (given)

$d = 0.6$ m (given)

$f' = 0.04$ (given)

$L = 1.5$ km (given) $= 1500$ m

$$\therefore \quad 30 = \frac{8}{9.81\times(3.14)^2}\frac{0.04\times 1500\times Q_1^2}{(0.6)^5}$$

or $$Q_1 = 0.686 \text{ cumecs.}$$

In the second case, when additional pipe is laid in the second half length of the pipe, let us assume that the discharge is Q_2, then this discharge Q_2 flows through the original pipe for a length equal to $\frac{1}{2}\times 1500 = 750$ m, and then half

of the discharge $\left(i.e.\ \frac{Q_2}{2}\right)$ flows through each, as both pipes are of equal dia.

The total head loss (30 m) will be

30 = Head loss in a length of 750 m with Q_2 discharge

+ Head loss in a length of 750 m with $\frac{Q_2}{2}$ discharge

or $$30 = \left[\frac{8}{g\pi^2}.\frac{0.04\times 750(Q_2)^2}{(0.6)^5}\right] + \left[\frac{8}{g\pi^2}\frac{0.04\times 750\times\left(\frac{Q_2}{2}\right)^2}{(0.6)^5}\right]$$

or $$30 = \frac{8}{g\pi^2}.\frac{0.04\times 750}{(0.6)^5}\left[Q_2^2 + \frac{Q_2^2}{4}\right]$$

or $$\frac{5}{4}Q_2^2 = \frac{30\times 9.81\times(3.14)^2\times(0.6)^5}{8\times 0.04\times 750} = 0.940$$

or $$Q_2 = 0.857 \text{ cumecs.}$$

Hence, increase in discharge

$$= Q_2 - Q_1 = 0.867 - 0.686 = 0.181 \text{ cumecs.}$$ **Ans.**

Example 6.7. *For supplying water to a colony, it is intended to store water in a large tank, whose water surface elevation is presently 85.37 m RL. This tank is being supplied by a pipe line from a junction J, the friction loss in metres of water in this pipe being $0.6\,Q^{1.95}$ where Q is the discharge in m^3/sec. Junction J receives water under pressure and without leakage from (i) a large reservoir I whose water surface elevation is constant at 100.00 m RL through a pipe characterised by $h_f = 0.35\,Q^2$, and (ii) a second large reservoir II with its constant water level at 96 m RL, through a pipe characterised by $h_f = 0.46\,Q^{1.85}$. Neglecting losses other than due to pipe friction, determine the incoming flow into the tank in the colony.* *(Engg. Services, 1990)*

Solution. The given conditions are shown in Fig. 6.6.

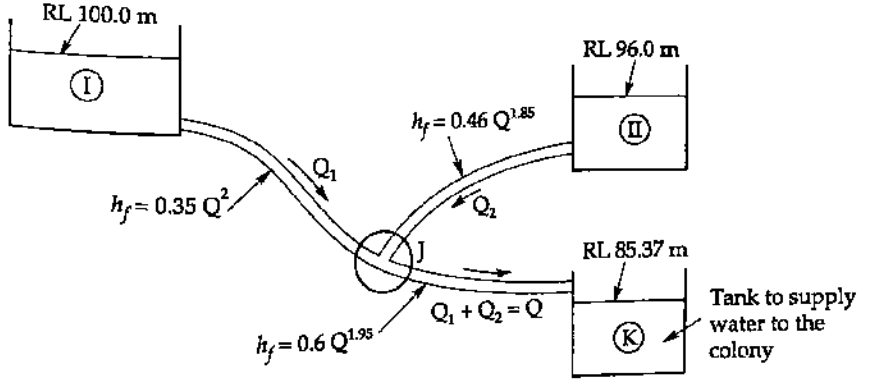


Fig. 6.6

By applying the Bernoulli's theorem, balancing pressures between I JK, and II JK, we have :

$$(i) \quad 100.00 - 0.35\,Q_1^{\,2} - 0.6\,(Q_1 + Q_2)^{1.95} = 85.37 \qquad ...(i)$$

$$(ii) \quad 96.00 - 0.46\,Q_2^{\,1.85} - 0.6\,(Q_1 + Q_2)^{1.95} = 85.37 \qquad ...(ii)$$

$$- \quad + \quad +$$

Subtracting (ii) from (i), we get

$$4.00 - 0.35\,Q_1^{\,2} + 0.46\,Q_2^{\,1.85} = 0$$

or $$Q_1^{\,2} = \frac{4.0 + 0.46\,Q_2^{\,1.85}}{0.35} = 11.43 + 1.31\,Q_2^{\,1.85}$$

or $$Q_1 = \sqrt{11.43 + 1.31\,Q_2^{\,1.85}}$$

Substituting in (i), we get

$$100 - 0.35\,[11.43 + 1.31\,Q_2^{\,1.85}] - 0.6\,(Q_1 + Q_2)^{1.95} = 85.37$$

or $$100 - (4 + 0.46\,Q_2^{\,1.85}) - 0.6\,(Q_1 + Q_2)^{1.95} = 85.37$$

or $$96 - 0.46\,Q_2^{\,1.85} - 0.6\,(Q_1 + Q_2)^{1.95} = 85.37$$

or $$10.63 = 0.46\,Q_2^{\,1.85} + 0.6\left[\sqrt{11.43 + 1.31\,Q_2^{\,1.85}} + Q_2\right]^{1.95}$$

or $$0.46\,Q_2^{\,1.85} + 0.6\left[\sqrt{11.43 + 1.31\,Q_2^{\,1.85}} + Q_2\right]^{1.95} = 10.63$$

Solving by hit and trial, we get

$$Q_2 = 0.797$$

Then, $$Q_1 = \sqrt{11.43 + 1.31 \times (0.797)^{1.85}} = 3.506$$

$\therefore$ $$Q = Q_1 + Q_2 = 4.303 \text{ cumecs.}$$ **Ans.**

Example 6.8. *A water supply main trifurcates at a junction point J into three branches each feeding a separate reservoir. The details of the pipes and reservoirs are as follows :*

Pipe	*Diameter*	*Length*	*Darcy Weisbach friction factor (f)*	*Terminal Reservoir*	*Reservoir elevation (m)*
JA	*20 cm*	*2.0 km*	*0.02*	*A*	*80.00*
JB	*20 cm*	*2.5 km*	*0.02*	*B*	*70.00*
JC	*20 cm*	*3.0 km*	*0.02*	*C*	*60.00*

If the inflow from the main at the junction is 0.250 m^3/s, determine the delivery into each reservoir. (*Civil Services, 1990*)

Solution. The given data is marked in Fig. 6.7 and then analysed using Bernoulli's equation.

Pressure at J $$= 80 + \frac{p_a}{w} + H_{L_1} = 70 + \frac{p_a}{w} + H_{L_2}$$

$$= 60 + \frac{p_a}{w} + H_{L_3}$$

$\therefore$ $$H_{L_1} + 10 = H_{L_2} \quad \text{and} \quad H_{L_2} + 10 = H_{L_3}$$

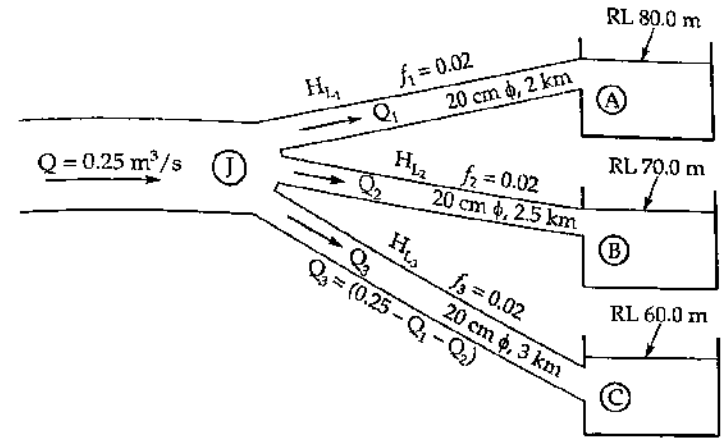


Fig. 6.7

Use Weisb

where $$H_{L_1} = \frac{f_1 LQ^2}{2gd \cdot \left(\frac{\pi}{4}\right)^2 \cdot d^4}$$

$$= \frac{0.02 \times 2000 \times Q_1^2}{2 \times 9.81 \times \left(\frac{\pi}{4}\right)^2 (0.2)^5} = 10328\,Q_1^{\,2}$$

Similarly, $$H_{L_2} = \frac{0.02 \times 2500 \times Q_2^2}{2 \times 9.81 \left(\frac{\pi}{4}\right)^2 \times (0.2)^5} = 12910\,Q_2^{\,2}$$

But $$H_{L_1} + 10 = H_{L_2}$$

$\therefore$ $$10328\,Q_1^{\,2} + 10 = 12910\,Q_2^{\,2}$$

or $$\boxed{Q_1 = \sqrt{\frac{12910\,Q_2^2 - 10}{10328}}} \qquad ...(i)$$

Also, $$H_{L_3} = \frac{0.02 \times 3000 \times (0.25 - Q_1 - Q_2)^2}{2 \times 9.81 \times \left(\frac{\pi}{4}\right)^2 (0.2)^5}$$

$$= 15492\left[0.25 - Q_1 - Q_2\right]^2$$

But $$H_{L_2} + 10 = H_{L_3}$$

$\therefore$ $$12910\,Q_2^{\,2} + 10 = 15492\left[0.25 - \sqrt{\frac{12910\,Q_2^2 - 10}{10328}} - Q_2\right]^2$$

Solving by trial, $$\boxed{Q_2 = 0.083 \text{ m}^3/\text{s}}$$

Substituting $Q_2 = 0.083$ in (i), we have

$Q_1 = 0.087$ m³/s.

$\therefore$ $Q_3 = 0.25 - 0.083 - 0.087 = 0.08$

Hence
$Q_1 = 0.087$ m³/s
$Q_2 = 0.083$ m³/s
$Q_3 = 0.080$ m³/s
} **Ans.**

Example 6.9. *Water flows in a 80 mm pipe at Reynolds number 80,000. The pipe is estimated to have an equivalent sand grain roughness of size 0.16 mm. Determine the head loss expected in 500 m length of the pipe. How much head loss would be expected if this pipe was smooth ? Take kinematic viscosity of water as 10^{-6} m²/s. The following explicit equation for friction factor may be used*

$$\frac{1}{\sqrt{f}} = 1.14 - 2 \log \left(\frac{k}{d} + \frac{21.25}{R^{0.9}} \right)$$

in which k is equivalent sand grain roughness, d the pipe diameter, and R the pipe Reynolds number. (*Engg. Services, 1992*)

Solution.

$$\frac{1}{\sqrt{f}} = 1.14 - 2 \log \left(\frac{k}{d} + \frac{21.25}{R^{0.9}} \right)$$

$$\therefore \quad \frac{1}{\sqrt{f}} = 1.14 - 2 \log \left[\frac{0.16 \text{ mm}}{80 \text{ mm}} + \frac{2125}{(80{,}000)^{0.9}} \right] = 6.239$$

$\therefore$ $f = 0.02569$

$$R = \text{Reynold No.} = \frac{V \cdot d}{\nu}$$

$$\therefore \quad 80{,}000 = \frac{V \times 0.08}{10^{-6}}$$

or
$$V = \frac{80{,}000 \times 10^{-6}}{0.08} = 1 \text{ m/s}$$

Head loss through 500 m length of pipe

$$= \frac{f L V^2}{2gd} = \frac{0.02569 \times 500 \times (1)^2}{2 \times 9.81 \times 0.08}$$

$$= \mathbf{8.18 \text{ m.}} \quad \textbf{Ans.}$$

When the pipe is smooth, naturally $f = 0$,

and head loss H_L will also be zero. **Ans.**

6.5. Forces Acting on Pressure Conduits

The structural design of the pressure pipes should be carried out, so as to enable them to withstand the various forces likely to come on them. The following forces generally come into play in the pressure pipes :

(1) Internal pressure of water including water hammer pressure*—*to be resisted by using materials strong in tension.*

(2) Pressure due to external loads in the form of backfill, traffic loads, etc.—*to be resisted by using materials strong in compression.*

*Explained a little later.

(3) Longitudinal temperature stresses created when pipes are laid above the ground—*to be resisted by providing expansion joints.*

(4) Longitudinal stresses created due to unbalanced pressures at bends, or at points of changes of cross-section—*to be resisted by holding the pipe firmly by anchoring it in massive blocks of concrete or stone masonry.*

(5) Flexural stresses produced when pipes are supported over trestles, etc.

The various forces enumerated above are explained below in details :

6.5.1. Internal Pressure of Water. The internal pressure within a pipe is caused by the water head to which the conduit can be subjected, and by the additional transient pressures. When a pipe is running full, the pressure in the pipe (p/w) is equal to the vertical ordinate between the H.G. line and the centre line of the pipe. But when the water is at rest, the pressure in the pipe is equal to the full static head of water $\left(\frac{p_s}{w}\right)$ (*i.e.,* the ordinate between the reservoir water level and the centre of the pipe) as shown in Fig. 6.8.

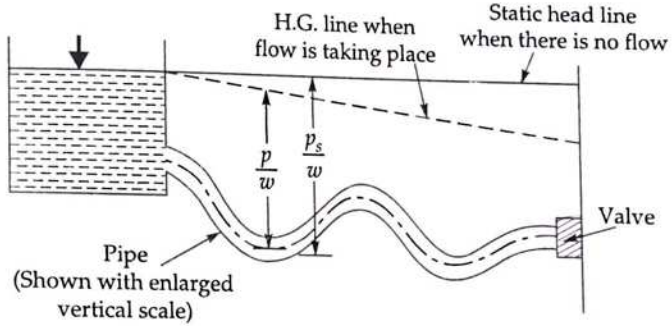


Fig. 6.8

In addition to this static water head, increased transient pressure is caused momentarily, when gates, valves, pumps, etc., are closed or opened. The additional pressure created by such a transient phenomenon is known as *water hammer pressure* (p_h).

The max. internal pressure likely to come under worst circumstances is usually taken equal to the sum of full static pressure and the water hammer pressure. Hence, if the static pressure is represented by p_s and the water hammer pressure is represented by p_h, then the total internal max. pressure that can occur in the pipe $p = p_s + p_h$. The internal pressure creates transverse stresses or circumferential tension called ***Hoop tension***, in the pipe walls. The magnitude of Hoop tension developed in this manner is approximately given by the formula :

$$\sigma = \frac{pd}{2t} \qquad \ldots(6.15)$$

where σ = Circumferential tensile stress produced. It has to be counteracted by providing Hoop's reinforcement in R.C.C. pipes and should not

exceed the safe allowable tensile stress of the pipe material, otherwise bursting of pipe may occur.

p = Total internal pressure including the full static water pressure and water hammer pressure.

d = Diameter of the pipe in metres.

t = Thickness of the pipe in metres.

6.5.1.1. Water hammer pressure. When a liquid flowing in a pipe is abruptly stopped by closing of a valve, etc., the velocity of the water column behind is retarded, and its momentum gets dissipated due to the conversion of kinetic energy into elastic energy. A series of positive and negative pressure waves are produced, which travel back and forth in the pipe, till they are damped out by friction. This phenomenon is known as ***water hammer***. In fact, in all hydraulic problems, we generally consider water as incompressible or non-elastic, but here is an example which brings the elastic properties of water and that of the pipe into action. What actually happens when a valve, etc., is suddenly closed, is explained below with reference to Fig. 6.9.

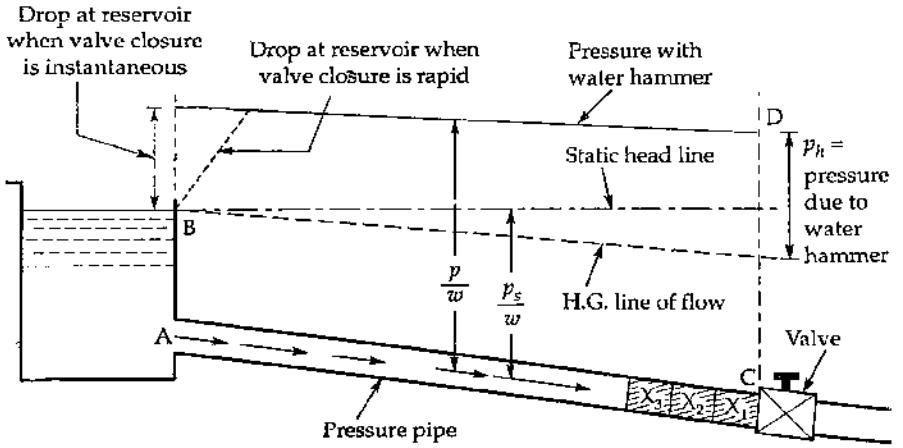


Fig. 6.9

When a valve is suddenly *closed*, an element of water (say X_1) just upstream of the valve, comes to rest ; whereas, the water upstream of this element continues to move with full velocity (V) and, therefore, strikes the element X_1 and thereby compressing the same. This results in an increased pressure in the element X_1. A part of this compression is resisted by compression of water and the remaining leads to *stretching* or extension of the pipe walls surrounding the element X_1. In the next step, the forward motion of another element X_2 (just upstream of X_1) is stopped, and it too, is compressed by the water upstream of it which still possesses forward motion. The process is repeated on successive elements, until in a relatively short time, the pressure wave has travelled back to the reservoir and all the water in the pipe is brought to rest. This leads to an excess pressure at the reservoir end A, which

is not possible because the pressure at B cannot remain more than the hydrostatic pressure (depth AB). Hence, the pressure at A instantly drops down to the normal value (*i.e.*, depth AB).

But since the pressure in almost whole of the pipe is more (*i.e.*, represented by CD in Fig. 6.9) than the pressure at A, the water of the pipe starts flowing back into the reservoir, and a wave of pressure unloading (or reflected wave) travels along the pipe from A to C. At the instant this pressure unloading wave reaches C, the entire water is under the static head (equal to depth AB). The water is still flowing back into the reservoir and this reverse velocity must now be stopped instantaneously. This will produce a drop in pressure at C, which will be below the static head and its magnitude will be the same as it was an instant before, *i.e.*, $(-p_h)$. A *wave of negative pressure* (called *wave of rarefaction*), therefore, gets set up, which travels towards the reservoir from C to A. As soon as this negative wave reaches the point A, the velocity of the water in the entire pipe is zero, but the pressure everywhere in the pipe is negative $(-p_h)$ except at A, where it has to be equal to the depth AB. This pressure difference starts the flow from the reservoir into the pipe, and a wave of *reflected negative pressure* (*i.e.* the *wave of pressure unloading*) travels from A to C. As this wave reaches C, the entire water in the pipe is moving with its original velocity towards the valve located at C, and the pressure in the pipe is brought to its initial value. *This completes one full cycle of waves of positive and negative pressures generated due to the instantaneous valve closure.* The process goes on repeating until damped out by friction, as shown in Fig. 6.10. The time required for a round trip of the pressure wave from C to A, and back from A to C, is given by $\frac{2S}{Up}$, where S is the distance of the reservoir from the valve, and U_p is the velocity or *celerity* of the pressure wave.

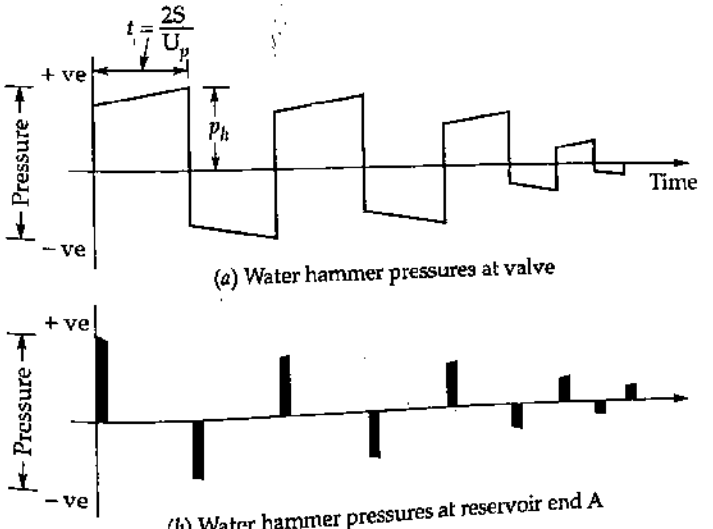


Fig. 6.10. Water hammer pressures developed due to closure of a valve.

In case a valve is suddenly *opened*, expansion of the water upstream of the valve occurs, and a wave of negative pressure travels up to the reservoir. The wave is reflected back upto the valve, again producing a wave of positive pressure, which finally gets reflected.

Estimating water hammer. From the above discussion, it becomes very clear that the process of closure or opening of a valve, etc., causes additional pressures. The more rapid the closure, the more rapid is the change in momentum, and hence greater is the additional pressure developed. This additional pressure called water hammer pressure may be so high as to cause bursting of the pipe shell (due to increased circumferential tension) if not accounted for in the design.

The maximum water hammer pressure is developed when the valve is quickly or instantaneously closed. The maximum pressure developed in this case is given by

$$p_{h(\max)} = \rho_w \cdot U_P \cdot V \qquad ...(6.16)$$

where $p_{h(\max)}$ = Maximum water hammer pressure developed in N/m²

U_P = Velocity of the pressure wave generated

ρ_w = Density or water $\left(i.e. \frac{\gamma_w}{g}\right)$

where γ_w is unit wt. of water = 9.81 kN/m³

and g is acceleration due to gravity = 9.81 m/sec²

V = Velocity of water in the pipe (which is reduced from V to 0)

The term U_P (*i.e.* the velocity of pressure wave generated) is given by the equation

$$U_P = \sqrt{\frac{E_w}{\rho_w}}^{*} \cdot \frac{1}{\sqrt{1+\frac{E_w}{Ep}\cdot\frac{d}{t}}} \qquad ...(6.17)$$

*The term $\sqrt{E/\rho}$ represents nothing but the velocity of a pressure wave in any medium which is the same as the velocity of sound in that medium and is given by $\sqrt{E/\rho}$. The velocity of the pressure wave created by water hammer is, however, reduced and is less then $\sqrt{E_w/\rho_w}$, because of the elasticity of the pipe walls. When only circumferential stretching is taking place and the longitudinal extension of the pipe is prevented, the velocity of the pressure wave generated (U_P) is given by $\sqrt{\frac{E_w}{\rho_w}} \cdot \frac{1}{\sqrt{1+\frac{E_w}{Ep}\cdot\frac{d}{t}}}$ (as explained in "Engineering Hydraulics" by Hunter Rouse, which is beyond the scope of this book).

where E_w = Modulus of elasticity of water or Bulk modulus of compression of water (as it is called)

E_P = Modulus of elasticity of pipe material

d = Diameter of the pipe

t = Thickness of the pipe shell

ρ_w = Density of water $\left(i.e. \frac{\gamma_w}{g}\right)$

Combining equations (6.16) and (6.17), we get

$$p_{h(\max)} = \rho_w \cdot \sqrt{\frac{E_w}{\rho_w}} \cdot \frac{1}{\sqrt{1+\frac{E_w}{Ep}\cdot\frac{d}{t}}} \cdot V$$

where $\sqrt{\frac{E_w}{\rho_w}}$ = **velocity of sound in water ≈ 1433 m/sec** ...(6.18)

Using S.I. units, we have

$$\therefore \quad p_{h(\max)} = \left(\frac{9.81}{9.81}\right)(1433)\frac{V}{\sqrt{1+\frac{E_w}{Ep}\cdot\frac{d}{t}}} \text{ kN/m}^2$$

or $$p_{h(\max)} = \frac{1433\,V}{\sqrt{1+\frac{E_w}{E_P}\cdot\frac{d}{t}}} \text{ kN/m}^2$$

or $$p_{h(\max)} = \frac{1433\,V}{\sqrt{1+\frac{k.d}{t}}} \text{ kN/m}^2 \qquad ...(6.19)$$

where $k = \frac{E_w}{E_P} = \frac{\text{Module of elasticity of water *}}{\text{Modulus of elasticity of the pipe material *}}$

The values of k for steel comes out to be 0.01 ; for cast iron = 0.02 ; and for cement concrete = 0.1.

If the valve is ***closed gradually***, a series of small pressure waves are transmitted up the pipe. The wave goes up to the reservoir and comes back to the point of the valve. Thus, if S is the distance of the valve from the reservoir, then the time taken by the wave in going to the reservoir and coming back to the valve, called *critical time*, is given by

$$T_c = \frac{2S}{U_P} \qquad ...(6.20)$$

*Value of E for water (*i.e.* E_w) = 21×10^5 kN/m²

E_P for steel = 21×10^7 kN/m²

E_P for cast iron = 10.5×10^7 kN/m²

E_P for cement concrete = 21×10^6 kN/m²

If the actual time of closure T is less than or equal to the critical time T_c, naturally full maximum water hammer will be developed. However, when the actual closure time T is more than the critical time T_c, negative pressure waves are superimposed on the positive waves and *full pressure is not developed.* The pressure developed in such a case is given by

$$p_h = p_{h(\max)} \left(\frac{T_c}{T} \right) \qquad ...(6.21)$$

It can, therefore, be concluded that the water hammer pressures can be considerably reduced by using slow closing valves.

In smaller pipe lines, they are kept moderate by maintaining moderate velocities. But for larger pipe lines, which when operated at relatively high velocities prove more economical, the water hammer pressures are kept under control by providing surge tanks, pressure relief valves, etc. If such arrangements are not provided, the cost of making the pipes strong enough to withstand water hammer may become too high.

The water hammer pressure can be calculated by using the Eqs. from (6.16) to (6.21). However, for ordinary purposes, under a normal range of pressures encountered in municipal water works systems, the following values of water hammer pressure may be adopted for cast iron and for steel pipes. For centrifugally cast iron pipes, somewhat higher allowances may be made for larger sizes.

Table 6.5. Normal Values of water hammer pressures to be adopted for steel and cast iron pipes

Pipe diameter in cm	*Water hammer pressure (kN/m^2)*
7.5 to 25	840
30	770
40	700
50	630
60	600
75	560
90	530
105 to 150	490

These allowances are applicable for normal pressures ; and for very high pressures, the calculation should be carried out to evaluate the water hammer pressure, and provision of surge tank, etc. may be thought of.

Example 6.10. *The velocity of water flowing from a reservoir into a 1 metre dia steel pipe is 2 m/sec. If a valve is situated in the pipe line at a point 2 km from the reservoir, evaluate the water hammer pressure developed by the closure of this valve, if*

(a) the closure time is 2 sec,

(b) the closure time is 6 sec.

The thickness of the pipe-shell may be taken to be 2.5 cm.

Solution. Using Eq. (6.17), we have

$$U_P = \sqrt{\frac{E_w}{\rho_w}} \cdot \frac{1}{\sqrt{1 + \frac{E_w}{E_P} \cdot \frac{d}{t}}}$$

But from Eq. (6.18), $\sqrt{\frac{E_w}{\rho_w}} = 1433$ m/sec.

Also, $$\frac{E_w}{E_P} = \frac{\text{Modulus of elaticity of water}}{\text{Modulus of elasticity of steel pipe}}$$

$$= \frac{21 \times 10^5 \text{ kN/m}^2}{21 \times 10^7 \text{ kN/m}^2} = 0.01$$

$d = 1$ metre (given)

$t = 2.5$ cm $= 0.025$ m (given)

$\therefore$ $$U_P = 1433 \cdot \frac{1}{\sqrt{1 + \frac{0.01 \times 1}{0.025}}} \text{ m/s}$$

$$= 1433 \cdot \frac{1}{\sqrt{1 + 0.4}} \text{ m/s}$$

$$= 1433 \cdot \frac{1}{\sqrt{1.4}} = \mathbf{1210 \text{ m/sec.}}$$

Critical time $T_c = \frac{2.S}{U_P}$ (from Eq. 6.20)

$$= \frac{2 \times (2 \text{ km})}{1210 \text{ (m/sec)}} = \frac{2 \times 2000}{1210} \text{ sec.} = \mathbf{3.3 \text{ sec.}}$$

Now $p_{h(\max)}$ = Maximum water hammer pressure

$= \rho_w \cdot U_P \cdot V$ [Refer eq. (6.16)]

$$= \left[\left(\frac{9.81}{9.81} \right) \times 1210 \times 2 \right] \text{kN/m}^2 ... \left[\because \rho_w = \frac{\gamma_w}{g} = \frac{9.81 \text{ kN/m}^3}{9.81 \text{ m/s}^2} \right.$$

$\left. V = 2 \text{ m/s} \quad \text{(given)} \right]$

$= \mathbf{2420 \text{ kN/m}^2}$.

Case (*a*) When the time of closure

$T = 2$ sec.

and since critical time

$T_c = 3.3$ sec (calculated above)

we have $T < T_c$

Therefore, full water hammer pressure = 2420 kN/m² will be developed. **Ans.**

Case (*b*) When $T = 6$ sec, $T_c = 3.3$ sec, the water hammer pressure developed is given by eqn. (6.21) as

$$= \left(\frac{T_c}{T}\right) p_{h(\max)}$$

$$= \frac{3.3}{6} \times 2420 \text{ kN/m}^2$$

$$= 1325.5 \text{ kN/m}^2. \quad \textbf{Ans.}$$

Example 6.11. *Derive the expression for the pressure rise due to water hammer caused by a sudden closure of the discharge through the pipe if (i) the pipe elasticity is not considered ; and (ii) the pipe elasticity is fully considered.* (*Engg. Services, 1990*)

Solution. Water hammer, as discussed in article 6.5.5.1, is the pulsation of pressures above and below the operating pressure, resulting from the pressure wave, caused by the sudden closure of a discharge valve. The sudden acceleration/deceleration of the velocity of water caused by such a pressure wave, exerts a force, which is absorbed largely by the elastic properties of water, and partly by the elastic properties of the pipe material. When the elasticity of the pipe is ignored, then the velocity of the pressure wave is given by the eqn.

$$U_p' = \sqrt{\frac{E_w}{\rho_w}}^{*} \qquad \ldots(6.17\ a)$$

where $\sqrt{\frac{E_w}{\rho_w}}$ equals the velocity of sound in water $\simeq 1433$ m/sec

Max. water hammer pressure $p_{h(\max)}$ is then given by eqn.

$$p_{h(\max)} = \rho_w \cdot U_p' \cdot V$$

where V = Velocity in pipe

Hence $p_{h(\max)} = \rho_w \cdot \sqrt{\frac{E_w}{\rho_w}} \cdot V = 1433\ \rho_w \cdot V$

is the eqn. for water hammer when elasticity of pipe material is ignored. **Ans.**

(*i*) When the elasticity of pipe material is also considered, then the velocity of pressure wave is given by

$$U_p = \sqrt{\frac{K}{\rho_w}}^{*}$$

where $K = \dfrac{E_w}{1 + \dfrac{E_w}{E_p} \cdot \dfrac{d}{t}}$

where E_w = Bulk modulus of elasticity of water

E_p = Modulus of elasticity of pipe material

*Derivation is available in books on "Fluid Mechanics".

d = dia of pipe

t = thickness of pipe shell.

$\therefore$ $p_{h(\max)}$ in this case is worked out as :

$$p_{h(\max)} = \rho_w \cdot U_p \cdot V = \rho_w . V . U_p$$

$$= \rho_w \cdot V \cdot \sqrt{\frac{E_w}{\rho_w}} \cdot \frac{1}{\sqrt{1 + \dfrac{E_w}{E_p} \cdot \dfrac{d}{t}}}$$

$$= \frac{\rho_w \cdot V \cdot (1433)}{\sqrt{1 + \dfrac{E_w}{E_p} \cdot \dfrac{d}{t}}} = \frac{1433\ \rho_w \cdot V}{\sqrt{1 + \dfrac{E_w}{E_p} \cdot \dfrac{d}{t}}}$$

is the required eqn. for water hammer when elasticity of the pipe material is also considered. **Ans.**

Example 6.12. *A steel penstock 60 cm in diameter has a cell thickness of 1.2 cm. The modulus of elasticity of the pipe shell material is 2.1×10^6 kgf/cm^2 (2.1×10^5 N/mm^2) with Poisson's ratio of $\frac{1}{4}$, and the volume modulus of elasticity of water is 2.1×10^4 kgf/cm^2 (2.1×10^3 N/mm^2). The pipe is designed to discharge water at mean velocity of 2.1 m/sec. Determine the water hammer pressure rise caused by sudden closure of the valve at the downstream end ; (1) neglecting the elasticity of the pipe material ; and (2) considering also the elasticity of the pipe material.* (*Engg. Services, 1990*)

Solution. Case 1. *When elasticity of pipe is ignored* and MKS Units are used, then

$$p_{h(\max)} = 1433 \cdot \rho_w \cdot V$$

$$= 1433 \cdot \left(\frac{1000}{9.81}\right) \times 2.1 \text{ kgf/m}^2$$

$$\left[\because \quad \rho_w = \frac{\gamma_w}{g} = \frac{\text{Unit wt. of water (1000 kgf/m}^3\text{)}}{g} \right.$$

$$\left. V = 2.1 \text{ m/sec.} \right]$$

$$= 30.68 \times 10^4 \text{ kgf/m}^2$$

$$= \textbf{30.68 kgf/cm}^2. \quad \textbf{Ans.}$$

Case 2. *When elasticity of pipe shell is considered*

$$p_{h(\max)} = 1433 \cdot \rho_w \cdot V \left[\frac{1}{\sqrt{1 + \dfrac{E_w}{E_p} \cdot \dfrac{d}{t}}}\right]$$

$$= 30.68 \left[\frac{1}{\sqrt{1 + \dfrac{2.1 \times 10^4}{2.1 \times 10^6} \times \dfrac{0.60}{0.012}}}\right]$$

[$\because$ $E_w = 2.1 \times 10^4$ kgf/cm^2 (given)
$E_P = 2.1 \times 10^6$ kgf/cm^2 (given)
$d = 60$ cm $= 0.60$ m (given)
$t = 1.2$ cm $= 0.012$ m (given)]

$$= 30.68\left[\frac{1}{\sqrt{1+0.5}}\right]$$

$$= \frac{30.68}{\sqrt{1.5}} = \mathbf{25.05\ kgf/cm^2.}\ \textbf{Ans.}$$

Example 6.13. *A 0.5 m diameter and 100 m long pipeline carrying 0.5 m^3/sec discharge is fitted with a valve at downstream end. Calculate the rise of pressure caused within the pipe due to the valve closure in (i) 1 second, (ii) instantaneously.*

Take the Sonic velocity as 1430 m/s. *(Engineering Services, 1994)*

Solution. The velocity of pressure wave generated by water hammer is given by eqn. (6.17) as

$$U_P = \sqrt{\frac{E_w}{\rho_w}}\,\frac{1}{\sqrt{1+\frac{E_w}{E_p}\cdot\frac{d}{t}}}$$

where the term $\dfrac{1}{\sqrt{1+\frac{E_w}{E_P}\cdot\frac{d}{t}}}$ represents the reduction caused in the velocity of the pressure wave by the elasticity of the pipe material. This term can be ignored, since the results will be on conservative side, and because the thickness of the pipe (t) and the material of pipe to reflect its E_P are not given.

Hence, $U_P = \sqrt{\dfrac{E_w}{\rho_w}}$ = Velocity of sound in water, *i.e.* Sonic velocity = 1430 m/s.

Now, *Max. water hammer pressure* $p_{h(max)}$

$= \rho_w \,.\, U_P \,.\, V$ ...(*i.e.* Eq. 6.16)

where $V = \dfrac{Q}{A} = \dfrac{0.5}{\frac{\pi}{4}\times(0.5)^2} = \mathbf{2.548\ m/s.}$

$$\rho_w = \frac{\gamma_w}{g} = \frac{9.81\,\text{kN/m}^3}{9.81\,\text{m/sec}^2}$$

$\therefore$ $p_{h(\max)} = \left[\dfrac{9.81}{9.81}\times 1430\times 2.548\right]$ kN/m^2 = **3643.64 kN/m^2.**

This maximum water hammer pressure will be caused when the valve is closed instantaneously. However, when the valve is closed in $T = 1$ sec, the pressure is given by eqn. (6.21), as

$$p_h = \left(\frac{T_c}{T}\right) p_{h(\max)}$$

where T_c = Critical Time of closure, as given by eqn. (6.20) as $T_c = \dfrac{2S}{U_P}$

where S = The distance in the pipe, which the wave has to travel
= 100 m (given)

$\therefore$ $T_c = \dfrac{2\times 100}{1430}$ s $= 0.14$ s

$\therefore$ $p_h = \left(\dfrac{0.14}{1}\right) \times 3643.64$ kN/m^2 = **510.1 kN/m^2.**

Hence (*i*) Pressure developed by the closure of the valve in 1 sec = **510.1 kN/m^2.** **Ans.**

(*ii*) Pressure developed by the closure of the valve instantaneously = **3643.64 kN/m^2. Ans.**

6.5.1.2. Thickness of metal pipe for safe design against internal pressure of water. The thickness of the steel or cast iron pipe which will be safe for bearing internal pressure, may be calculated easily by using

$$t = \frac{1}{\eta}\cdot\frac{pd}{2f} \qquad \text{...(6.22)}$$

where t = Thickness of pipe shell
d = Diameter of the pipe
f = Permissible tensile stress in the pipe material (generally taken as 770 kg/cm^2 (77,000 kN/m^2) for cast iron pies and 1260 kg/cm^2 (1,22,600 kN/m^2) for steel pipes)
p = Internal pressure of water including static head and water hammer
η = Efficiency of the joint. For steel pipes it is generally taken as 0.9 for welded, 0.75 for double riveted, 0.63 foı single riveted joints. The least thickness for a steel pipe is 5 mm.

The thickness obtained by the above formula is generally increased by about 2 to 3 mm, so as to make allowance for corrosion.

Example 6.14. *Water is being brought from an impounding reservoir to the city by means of a 1 metre diameter steel pipe across an undulating ground. The water level in the reservoir is 390 m whereas the lowest level of the pipe in its L-section is 280 m. Determine the thickness of the pipe required with respect to the max. pressure, if the allowance for water hammer is made for 863 kN/m^2 over static head. The steel pipe may be taken to be riveted with*

efficiency of joint as 70%. The allowable working stress in steel may be taken as 122600 kN/m².

Solution. Static head of water

$$p_s = (390 - 280) \text{ m}$$
$$= 110 \text{ m of water head}$$
$$= 110 \times 9.81 \text{ kN/m}^2$$

[10.2 m of water head = 100 kN/m² pressure,

$\therefore$ **1 m of water head = 9.81 kN/m²**]

or $= 1079 \text{ kN/m}^2$.

Water hammer

$$p_h = 863 \text{ kN/m}^2 \text{ (given)}$$

Total internal pressure

$$p = 1079 + 863 = 1942 \text{ kN/m}^2$$

Now, using Eq. (6.22), we have

$$t = \frac{1}{\eta} \frac{p \cdot d}{2f}$$

Here $p = 1942 \text{ kN/m}^2$

$f = 122600 \text{ kN/m}^2$

$d = 1$ m (given)

$\eta = 0.7$ (given)

$$\therefore \quad t = \frac{1}{0.7} \times \frac{1942 \times 1}{2 \times 122600} \text{ metres}$$

$$= 0.0113 \text{ m} = 11.3 \text{ mm}$$

Add 3 mm extra for corrosion, etc.

$\therefore$ Thickness required = 14.3 mm

Use 15 mm thickness. **Ans.**

Example 6.15. *Water is supplied to a town through a cast iron gravity main, 0.5 m diameter and 30 km long, across an undulating ground from the headworks to the service reservoir in the town. The pipe off-take at the headworks is at RL + 480 m and the maximum water level of the reservoir is at RL + 370 m. The lowest ground level in the L-section of the main is at RL + 280 m at a point 15 km from the off-take. If it is desired to increase the supply to the town by 30%.*

(a) For what distance would you lay a duplicate pipe of the same size ?

(b) What is the expected thickness of this pipe at a point 15 km away (where the RL of the centre of the pipe is 280 m) ?

The water hammer pressure may be neglected as additional arrangements have been made for dissipating the same.

Solution. The maximum head difference between the source and the service reservoir

$$= 480 - 370 = 110 \text{ m.}$$

The pipe is designed as a gravity main, which means that the entire available head is lost in overcoming losses. Further, neglecting the minor losses, the available head is lost in overcoming friction. Using William Hazen's equation, we have

$$V = 0.85 \cdot C_H \cdot R^{0.63} \cdot S^{0.54}$$

where $C_H = 120$ from Table 6.2 for cast iron pipe

and $R = \frac{d}{4} = \frac{0.5}{4} = 0.125$ m

$H_L = 110$ m

$L = 30$ km $= 30{,}000$ m

$$\therefore \quad V = 0.85 \times 120 \times (0.125)^{0.63} \cdot \left[\frac{110}{30{,}000}\right]^{0.54}$$

$$= 0.85 \times 120 \times (0.125)^{0.63} \times (0.00367)^{0.54}$$

$$= 0.85 \times 120 \times 0.27 \times 0.065 = 1.79 \text{ m/sec.}$$

Now, $\quad Q = A.V = \left[\frac{\pi}{4} \times (0.5)^2\right] \times 1.79$

$$= 0.196 \times 1.79 = \mathbf{0.35 \text{ cumec.}}$$

New Discharge $Q' = 1.3\,Q$

$$= 1.3 \times 0.35 = \mathbf{0.46 \text{ cumec.}}$$

Let the duplicate pipe be laid in a length of l km, as shown in Fig. 6.11.

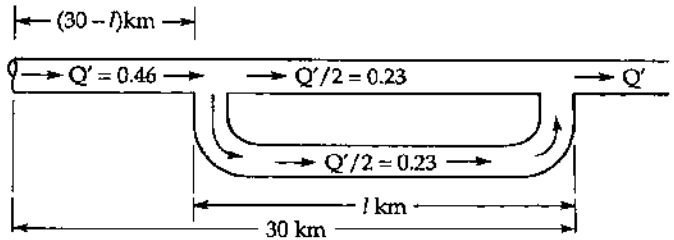


Fig. 6.11

Now, equating the available head as equal to the head loss, by using the Eqn. [6.12 (*a*)], we have

$$H_L = \frac{1}{0.094}\left(\frac{Q}{C_H}\right)^{1.85} \frac{L}{d^{4.87}}$$

where $C_H = 120$

$$\therefore \quad H_L = \frac{1}{0.094} \frac{Q^{1.85}}{(120)^{1.85}} \cdot \frac{L}{d^{4.87}}$$

or $$H_L = \frac{1}{670} \frac{Q^{1.85} L}{d^{4.87}}$$

Substituting other values, we get

$$110 = \frac{(0.46)^{1.85}(30-l)(1000)}{670(0.5)^{4.87}} + \frac{(0.23)^{1.85}(l)(1000)}{670(0.5)^{4.87}}$$

or $$110 = \frac{1000}{670 \times 0.034}[0.238(30-l) + 0.066l]$$

or $$110 = \frac{1000}{670 \times 0.034}[7.13 - 0.238\,l + 0.066l]$$

or $$\frac{110 \times 670 \times 0.034}{1000} = 7.13 - 0.172\,l$$

or $$2.51 = 7.13 - 0.172\,l$$

or $$0.172\,l = 4.62$$

or $$l = \frac{4.62}{0.172} = 26.9 \text{ km}$$

Hence, the duplicate pipe should be laid in a length of 26.9 km. **Ans.**

(*b*) **Thickness of pipe** (Refer Fig. 6.12).

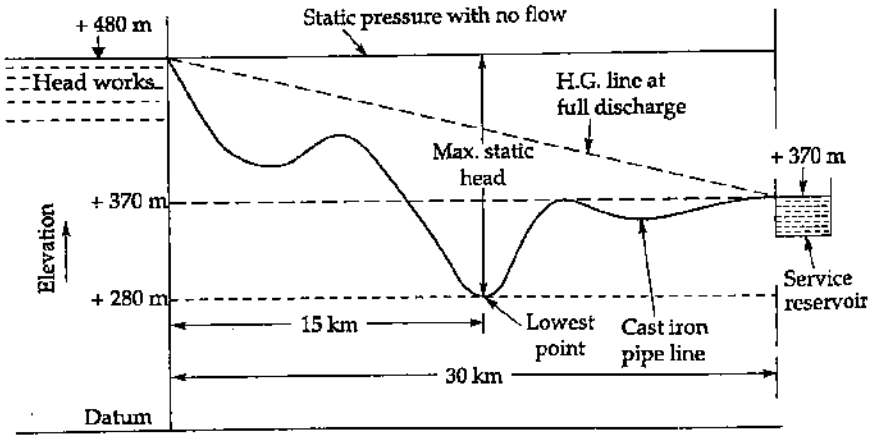


Fig. 6.12. *l*-section of the pipe in Example 6.15. (Vertical scale shown enlarged)

The max. static head = 480 – 280 = 200 m of water head

= 200 × 9.81 kN/m² = 1962 kN/m²

The total internal pressure

= p = 1962 kN/m²

Using $t = \frac{1}{\eta}\frac{p.d}{2f}$ ...(*i.e.*, Eq. 6.22)

where f = allowable tensile stress in cast iron

≈ 75000 kN/m² (assumed)

η = 1.0 for C.I. pipe joints (assumed)

We have, the thickness of the pipe required at the point of maximum pressure

$$t = \frac{1962 \times 0.5}{2 \times 75000_1} \text{ m} = 0.65 \text{ cm}$$

Use 0.7 cm thick pipe.

Thus, the expected thickness of the pipe carrying this water is 7 mm. **Ans.**

6.5.2. Pressures due to External Loads. Pipes are often buried under the ground and placed in trenches which are backfilled. The weight of the pipe, the weight of the backfill and the superimposed traffic loads if any, will then be transferred to the pipe. This will produce compressive stresses in the pipe material (when the pipe is empty and there is no internal pressure to counteract these loads) and the material may fail in compression if it exceeds the allowable compressive stress of the pipe material. The pipes should, therefore, be checked for this possible failure. However, in water pipe lines, it is not a big problem, because such pipes are generally not laid very deep. Moreover, the steel pipes, the reinforced concrete pipes, and the cast iron pipes, which are commonly used for transporting water, are quite good in compression.

The stresses produced due to external loadings, at present, can be evaluated by using certain empirical formulas, as given below.

These formulas are based on the experiments carried out by Marston, etc. at Iowa State College in U.S.A.

(*a*) For pipes resting on or projecting above the undisturbed ground in cohesionless soil and covered with such cohesionless fills, such as in a highway culvert, (Fig. 6.13), the external load from the backfill likely to come per unit length of pipe (W) is given as :

$$W = C_p \cdot \gamma \cdot D^2 \qquad ...(6.23)$$

where C_p = a coefficient, whose value depends upon the type of the pipe and character of foundation and backfill. Typical values of C_p are given in table 6.6

γ = Unit wt. of the fill material

D = The external diameter of the pipe

= (Internal diameter + 2 × thickness)

Table 6.6. Values of C_p in Eqn. (6.23)

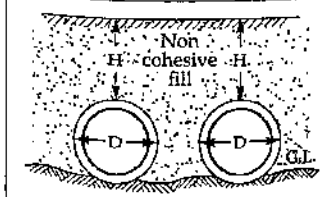
Conditions (1)	*Cover depth* $\frac{H}{D}$ = *External pipe dia* (2)	*Values of* C_p *For rigid pipes* (3)	*Values of* C_p *For flexible pipes* (4)
	1.0	1.2	1.1
	2.0	2.8	2.6
	3.0	4.7	4.0
	4.0	6.7	5.4
	6.0	11.0	8.2
	8.0	16.0	11.0

Fig. 6.13

(*b*) For *flexible* pipes (such as *steel* pipes) buried in narrow trenches and thoroughly compacted side fills, such as shown in Fig. 6.14, the external load per unit length of pipe from the backfill is given as :

$$W = C \,.\, \gamma \, B \,.\, D \qquad ...(6.24)$$

where C = a coefficient characteristic of the fill material and the ratio H/B, where H is the cover depth and B is the trench width. Typical values are given in table 6.7.

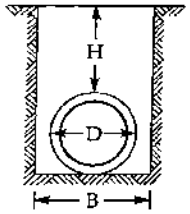
D = External dia of the pipe.

(*c*) For *rigid* pipes (such as concrete, cast iron, vitrified clay, etc.) buried in narrow trenches and thoroughly compacted with cohesionless fills, the external load per unit length of the pipe from the backfill is given as :

$$W = C \,.\, \gamma \,.\, B^2 \qquad ...(6.25)$$

where C, γ and B have the same meaning as given above.

Table 6.7. Values of C in Eqns. (6.24) and (6.25)

Conditions	*Fill Material*	*Dry Sand and Gravel*	*Saturated top soil*	*Clay*	*Saturated clay*
	Unit wt. in kN/m³	*16*	*16*	*19.2*	*20.8*
	Values of $\frac{H}{B}$	*Value of C*	*Value of C*	*Value of C*	*Value of C*
	1.0	0.84	0.86	0.88	0.90
	2.0	1.45	1.50	1.55	1.62
	3.0	1.90	2.00	2.10	2.26
	4.0	2.22	2.33	2.49	2.65
	5.0	2.45	2.60	2.80	3.03
	6.0	2.60	2.78	3.04	3.33
	7.0	2.75	2.95	3.23	3.57
	8.0	2.80	3.03	3.37	3.76
Narrow trench	9.0	2.88	3.11	3.48	3.92
such that	10.0	2.92	3.17	3.56	4.04
$B \leq 1.5\,D$.	12.0	2.97	3.24	3.68	4.22
Fig. 6.14	14.0	3.00	3.28	3.75	4.34

(*d*) The amount of superimposed load (such as traffic load, etc.), which is transmitted to the pipe can be evaluated by using Boussinesq's equation. Assuming the fill surface to be horizontal, the equation is

$$p_t = \frac{3 \,.\, H^3 P}{2\pi \,.\, Z^5} \qquad ...(6.26)$$

where p_t = unit pressure developed at any point in the fill at a depth H below the surface due to traffic load.

P = Load superimposed.

Z = The slant height of the considered point from the load P.

H = Distance of the top of pipe below the surface of the fill.

The *total traffic load developed on a unit length of conduit* (W') can be found by integrating Eqn. (6.26) over the projected area of the pipe. This integration can be done by subdividing the projected area of the pipe into small squares and then computing the load on each sub-division and finally adding these values to obtain a total.

In case of unsurfaced roads, the impact from moving loads may nearly double the computed loads. Paving considerably reduces the effect of impact. Moreover, the effect of superimposed load decreases rapidly as the depth of cover increases (because Z^5 increases much more than H^3).

The total load per unit length of the pipe from the backfill and from traffic can be found by adding W with W'. The compressive stress produced, which should be checked when the pipe is empty, will then be given as :

$$\frac{W + W'}{t} \text{ kN/m}^2 \qquad ...(6.27)$$

(where t is in metres).

Note. There is sometimes the possibility of creation of vacuum in the pipe due to water hammer, which will increase the compressive stresses. The pipe thickness should be checked for that possibility also, so as to avoid collapse in such conditions.

Example 6.16. *A pipe 1 metre in diameter, buried in a trench 1.4 m wide, is backfilled with dry sand. The top of the pipe is 2.5 m below the surface of the fill. The pipe passes at right angles under a one lane road which carries a vehicle whose loading (including impact) consists of two concentrated 8 kN loads located at 1.8 m apart transverse to the roadway. Find the maximum vertical force exerted on a unit length of the pipe, if*

(*a*) *the pipe is made of steel.*

(*b*) *the pipe is a cast iron pipe.*

The thickness of the pipe may be neglected for calculating the external dia. of the pipe.

(c) *what is the stress produced in steel pipe if its thickness is 1 cm ?*

Solution. We will first of all evaluate the forces caused by the *backfill.*

In this question, we have

$D \approx d = 1$ m (neglecting thickness, as given)

$B = 1.4$ m

γ for dry sand = 16 kN/m³

$H = 2.5$ m.

Case (*a*) The pipe is of steel—which is a flexible pipe

∴ Using eqn. (6.24), we have

$$W = C.\, \gamma B.D$$

The value of C from table 6.7 for $\frac{H}{B} = \frac{2.5}{1.4} = 1.8$ and dry sand is given

$$= 0.84 + \frac{1.45 - 0.84}{1} \times 0.8$$

$$= 0.84 + 0.61 \times 0.8 = 084 + 0.49 = \mathbf{1.33.}$$

Substituting in Eq. (6.24), we get

$$W = [1.33 \times 16 \times 1.4 \times 1] \text{ kN/m}$$

$$= \mathbf{29.8\ kN/m.} \qquad ...(i)$$

Case (*b*) The pipe is of cast iron, which is a rigid pipe. Using Eqn. (6.25), we have

$$W = C.\ \gamma.\ B^2 = 1.33 \times 16 \times (1.4)^2$$

$$= \mathbf{41.7\ kN/m.} \qquad ...(ii)$$

Now, we will evaluate the forces caused by the *traffic load*.

Using Eqn. (6.26), we have

$$p_t = \text{pressure developed due to traffic load} = \frac{3H^3.P}{2\pi Z^5}$$

Here in this question, we have

Z = The slant height from one of the wheel load to a point on the pipe midway between the load.

$$= \sqrt{\left(\frac{\text{space between the wheels}}{2}\right)^2 + H^2}$$

$$= \sqrt{\left(\frac{1.8}{2}\right)^2 + (2.5)^2} = \sqrt{0.81 + 6.25} = \sqrt{7.06}$$

or $Z = \mathbf{2.66\ m.}$

But $p_t = \dfrac{3.H^3.P}{2\pi Z^5}$

Hence, the pressure developed due to single wheel load

$$\frac{3 \times (2.5)^3 \times 8}{2 \times 3.14 \times (2.66)^5} = \frac{3 \times 15.6 \times 8}{2 \times 3.14 \times 134} \text{ kN/m}^2$$

$$= \mathbf{0.444\ kN/m^2.}$$

Total force due to traffic load per metre length of pipe

$W' = p_t \times$ No. of wheels $\times$ projected area of the pipe

$= (0.444) \times 2(1)$

[$\because$ projected area* $\approx$ Dia $\times$ 1 metre length = 1 m^2]

or $W' = 0.888$ kN/m length.

Total external force inclusive of backfill and traffic per metre length of pipe in case (*a*)

$$W + W' = 29.8 + 0.888 = 30.688 \text{ kN/m}$$

Say 30.69 kN/m. **Ans.**

*The more accurate estimate of superimposed load could have been made by analysing smaller unit areas and then adding the load on each subdivision (Z will then be different in all considered squares). But this labour seems hardly justified in this case.

Total external force inclusive of backfill and traffic per metre length of pipe in case (*b*)

$$= 41.7 + 0.888 = 42.588 \text{ kN/m ; say } 42.59 \text{ kN/m.} \quad \textbf{Ans.}$$

Using Eqn. (6.27),

$$\text{Stress produced} = \frac{W + W'}{t} = \frac{42.59}{0.01} = 4259 \text{ kN/m}^2. \quad \textbf{Ans.}$$

6.5.3. Temperature Stresses. When pipes are laid above the ground, they are exposed to atmosphere and are, therefore, subjected to temperature changes. They expand during day time and contract at night. If this expansion or contraction is prevented due to fixation or friction over the supports, *longitudinal stresses* are produced in the pipe materials. The amount of these stresses may be calculated as below :

Elongation = $L\alpha.T$.

But, Strain = Elongation per unit length

$$= \frac{L.\alpha T}{L} = \alpha\ .\ T$$

Stress = E. Strain = $E_p.\alpha T$ ...(6.28)

or $f = E_p\ \alpha.T$

where E_p = Modulus of elasticity of the pipe material (values given in table 6.8)

α = Co-efficient of expansion of the pipe material (values given in Table 6.9)

T = Change in temperature in °C.

Table 6.8. Values of E_p for various pipe materials

S. No.	*Material*	*Value of Ep in kN/m²*
1.	Steel	21×10^7
2.	Cast iron	10.5×10^7
3.	Cement Concrete	21×10^6

Table 6.9. Values of α for various pipe materials

S. No.	*Material*	*Coefficient of linear expansion (α) in m/m/°C (20°—100°)*
1.	Steel	12.6×10^{-6}
2.	Cast iron	11.0×10^{-6}
3.	Concrete	9.8×10^{-6}
4.	Copper	16.5×10^{-6}

Special types of expansion joints at suitable intervals (say 20 m to 30 m or so) must be provided to counteract these stresses.

6.5.4. Stresses due of Flow around Bends and Changes in Cross-section. Whenever the velocity of a flow (either magnitude or direction) changes, there is a change in the momentum and, therefore, by Newton's Second Law of motion, a force is exerted which is proportional to the rate of change of momentum. The force required to bring this change in momentum

comes from the pressure variation within the fluid and from forces transmitted to the fluid from the pipe walls.

The free body diagram of various forces acting on the water contained in a *horizontal pipe bend* is shown in Fig. 6.15. Applying momentum equation, and resolving the forces in x and y direction, we get

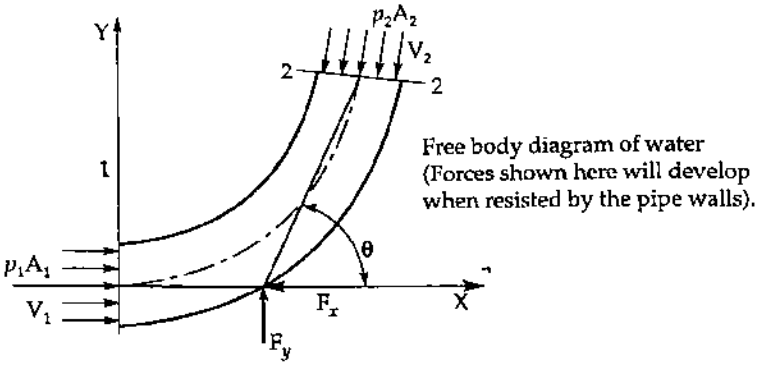


Fig. 6.15. Plan of a bend.

$$p_1A_1 - F_x - p_2A_2 \cos\theta = \frac{wQ}{g}[V_2 \cos\theta - V_1] \qquad ...(6.29)$$

and
$$F_y - p_2A_2 \sin\theta = \frac{wQ}{g}[V_2 \sin\theta] \qquad ...(6.30)$$

where p_1 and p_2 are the pressures, V_1 and V_2 are the velocities at sections 1-1 and 2-2, respectively.

Q = discharge flowing

For a pipe of a uniform cross-section, $p_1 = p_2 = p$, $V_1 = V_2 = V$, $A_1 = A_2 = A$; and for a bend of 90°, Eqn. (6.29) becomes

$$pA - F_x = \frac{wQ}{g}(-V)$$

or
$$F_x = pA + \frac{wQ}{g} \cdot V \qquad [\because \cos 90° = 0]$$

or
$$F_x = \left[pA + \frac{w}{g} \cdot A \cdot V^2\right] \qquad ...(6.31)$$

Similarly Eqn. (6.30) becomes

$$F_y - pA = \frac{wQ}{g} \cdot V$$

or
$$F_y = pA + \frac{wQ}{g} \cdot V \qquad [\because \sin 90° = 1]$$

or
$$F_y = pA + \frac{w}{g} \cdot AV^2 \qquad ...(6.32)$$

The resultant force $= \sqrt{F_x^2 + F_y^2}$

$$= \sqrt{2}\left[pA + \frac{w}{g} \cdot AV^2\right] \qquad ...(6.33)$$

This is the total force in this specific case.

These forces F_x and F_y and their resultant are the forces which are transmitted from the pipe to the water. An equal and opposite force must, therefore, be developed in the form of stresses in the pipe wall.

These impressed external forces try to throw away the pipe in the direction of the resultant force (opposite to what is shown as stresses in Fig. 6.15, and as shown in Fig. 6.16). An equal and opposite force must, therefore, be developed in the form of stresses in the pipe wall as shown in Fig. 6.15. These stresses may be eliminated or reduced by holding the pipe firmly by anchoring it in massive blocks of concrete.

Similar problem will occur when the cross-sectional area of the pipe is suddenly changed, as shown in Fig. 6.17.

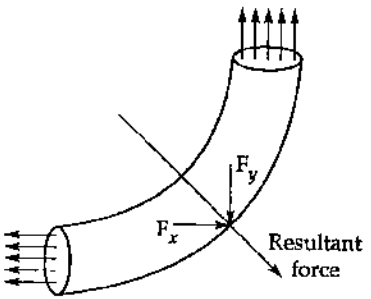


Fig. 6.16

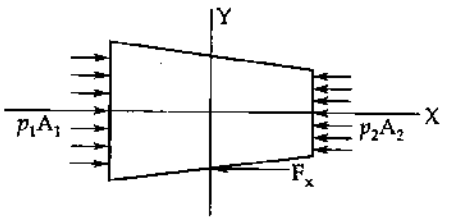


Fig. 6.17

6.5.5. Flexural stresses. Many a times, pressure pipes are carried supported between trestles, or piers, or bridge hangers like beams. Similarly, sometimes the rain water, etc. may wash off the ground from below the pipes resting on the ground, thus exposing them like beams supported between two supports. Under all such circumstances, bending stresses get produced in the pipe, since the pipe acts like a beam with loads resulting from the weight of the pipe, weight of the water in the pipe, and any other superimposed loads. The stresses caused by this beam action may be determined by the usual methods of structural analysis applied to the beams. However, in normal circumstances, the stresses produced are small for smaller spans, and hence often neglected, except for long spans where there are huge superimposed loads.

6.6. Certain Important Definitions

Here we shall define certain important terms which are frequently used in India for defining various pressures in pressure pipes by the manufacturers of these pipes.

Working Pressure. Working pressure may be defined as the actual maximum pressure (including abnormal conditions such as water hammer) to which the pipe will be subjected during its operation.

Design Pressure. Design pressure may be defined as the maximum pressure for which the pipe has been designed. This is equal to the product of the working pressure and suitable factor of safety to cover abnormal increase in pressure due to unforeseen circumstances.

Test Pressure. Test pressure may be defined as the maximum pressure which the pipe can withstand without any leakage when tested for hydrostatic pressure in accordance with the standard methods of testing.

6.7. Various Types of Pressure Pipes

Depending upon the construction material, the pressure pipes are of the following types :

(1) Cast iron pipes (2) Steel pipes
(3) Reinforced cement concrete pipes (4) Hume steel pipes
(5) Vitrified clay pipes (6) Asbestos cement pipes
(7) Miscellaneous types of pipes.

The selection of a particular type of material for a pipe depends mainly upon the relative economy, the pressures likely to come and the working pressures, maximum permissible sizes and capacities, availability of materials and labour for their construction, etc. The various types of pipes and their relative utilities are discussed below :

6.7.1. Cast Iron Pipes. Cast iron pipes are widely used for city water supplies. They are sufficiently resistant to corrosion* and may last as long as 100 years or so. They are generally manufactured in lengths of about 3.5 metres, but may be manufactured in lengths up to 6 m or so, on special orders.

Cast iron pipes can be manufactured by two methods. One is the ordinary ***sand moulding*** and the other is the ***centrifugal process.*** Pipes cast in horizontal position by ordinary sand moulding are called "*McWane*" pipes, while those cast vertically are called "*Pit cast pipes*". Horizontal cast pipes are 100 percent stronger in tension and 50 percent stronger in rupture than vertically cast iron pipes.

Centrifugally cast iron pipes are made either in sand or metallic moulds. The former are called *Sand spun pipes* and the latter are called *Delavaud* pipes. The spun iron pipes are denser and tougher than the pipes moulded by ordinary methods. In spun iron casting in metallic moulds, the molten metal is applied to the interior of a water cooled cylindrical metal mould which is rotated rapidly. Centrifugal force holds the metal against the mould and forms a homogeneous pipe with a cylindrical bore. After the pipe cools down to a temperature of about 800°C, it is taken from the machine to a furnace where it is raised to about 925°C, and then slowly cooled to about 650°C so as to anneal it. After hydrostatic testing, it is cooled.

Indian Standards. **Centrifugally cast iron pipes (called spun iron pipes)** are generally available in India in three types (*i.e.* Class LA, Class A, and Class B) as per the provisions of I.S. 1536—1976 as given in Table 6.10.

*Explained in article 6.7.2.1.

Table 6.10. Types of Cast iron (spun) pipes as per I.S. 1536—1976

S. No.	Category of pipe in order of increasing thickness for the same diameter	Test Pressure in kN/m^2	Working Pressure in kN/m^2
1.	Class LA	3500	1200
2.	Class A	3500	1800
3.	Class B	3500	2000

The nominal internal diameters available are 80, 100, 125, 150, 200, 250, 300, 350, 400, 450, 500, 600, 700 and 750 mm. Their respective thicknesses for class LA are 7.2, 7.5, 7.9, 8.3, 9.2, 10.0, 10.8, 11.7, 12.5, 13.3, 14.2, 15.8, 17.5, and 18.3 mm, respectively. For class A, the respective thicknesses are 7.9, 8.3, 8.7, 9.2, 10.1, 11.0, 11.9, 12.8, 13.8, 14.7, 15.6, 17.4, 19.3 and 20.2 mm. And for Class B, the respective thicknesses are 8.6, 9.0, 9.5, 10.0, 11.0, 13.0, 14.0, 15.0, 16.0, 17.0, 19.0, 21.0 and 22.0 mm.

Vertically cast iron pipes are generally available in India in two types (Class A and Class B) as per the provisions of IS 1537— 1967. The thicknesses of Class B pipes are more than those of Class A for the same diameters. These pipes are available in internal nominal diameters of 80, 100, 125, 150, 200, 250, 300, 350, 400, 450, 500, 600, 700, 750, 800, 900, 1000, 1100, 1200, and 1500 mm. Their respective thicknesses for Class A pipes are 7.9, 8.3, 8.7, 9.2, 10.1, 11.0, 11.9, 12.8, 13.8, 14.7, 15.6, 17.4, 19.3, 20.2, 21.1, 22.9, 24.8, 26.6, 28.4 and 33.9 mm. For Class B, the respective thicknesses are : 8.6, 9.0, 9.5, 10.0, 11.0, 12.0, 13.0, 14.0, 15.0, 16.0, 17.0, 19.0, 21.0, 22.0, 23.0, 25.0, 27.0, 29.0, 31.0 and 37.0 mm. The test pressures are as given in table 6.11.

Table 6.11. Test Pressures in Vertically Cast Iron Pipes as per IS 1537—1976

S. No.	Diameter	Test pressure kN/m^2 using socket and spigot joint	
		Class A	Class B
1.	Upto and including 300 mm	2000	2500
2.	Over 300 mm and upto and including 600 mm	2000	2500
3.	Over 600 mm and upto and including 1000 mm	1500	2000
4.	Over 1000 mm and upto 1500 mm	1000	1500

As is evident from the above data, the spun iron pipes are, thinner than the vertically cast iron pipes, and hence are lighter and can be manufactured in longer lengths. In order to protect these pipes from corrosion, they are generally coated with bituminous linings. Larger sizes may be provided with cement mortar (1 : 2) lining, whereas smaller sizes, such as those to be used in small service lanes for water supply distribution system, are generally coated with zinc and are known as ***Galvanised iron pipes***.

The *advantages* of cast iron pipes are :

(*i*) moderate in cost

(*ii*) easy to join

(*iii*) strong and durable

(*iv*) corrosion resistant

(*v*) long life upto 100 years or so

(*vi*) service connections can be easily made.

The *disadvantages* of cast iron pipes are :

(*i*) water carrying capacity decreases with time, as the value of friction factor increases due to tuberculation in certain waters.

(*ii*) they cannot be used for high pressures. Generally, not used for pressures above 700 kN/m^2.

(*iii*) when large, they are very heavy and uneconomical.

(*iv*) they are likely to break during transportation or while making connections.

Joints. The following types of joints are generally used in cast iron pipes :

(*i*) Socket and spigot joint

(*ii*) Flanged joint

(*iii*) Mechanical joint called dresser coupling

(*iv*) Flexible joint

(*v*) Expansion joint.

These joints are briefly described below :

(*i*) **Socket and Spigot joint.** Cast iron pipes are being joined by a "socket and spigot joint" (sometimes called ***bell and spigot joint***) since olden days, perhaps a century or so. This joint is till today used on a large scale.

The cast iron pipes which are to be joined by the "socket and spigot joint", are made in such a way that one of their end is enlarged, whereas the other end is normal. The enlarged end is called "socket" or "bell", while the normal end is called "spigot". The spigot is fitted into the socket. A few strands of jute are wrapped around the spigot before inserting it into the socket (or bell) and then, more jute is packed into the joint. The remaining space between the socket and the spigot is finally filled with *molten lead*, which gets solidified and tightly caulked into the joint after cooling, and thus making a water tight joint (see Fig. 6.18). The quantity of lead required per joint varies from 3.5 to 4 kg for 15 cm dia pipe, to about 45 to 50 kg for 1.2 metre dia pipe.

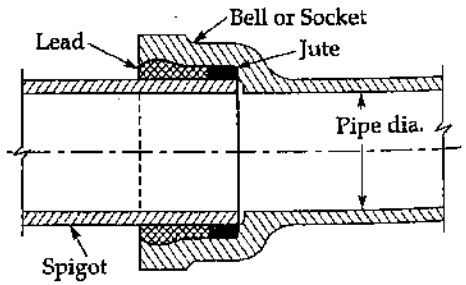


Fig. 6.18. Socket and spigot joint.

This type of joint is somewhat flexible and allows the pipes to be laid on flat curves without the use of specials. However, skilled labour is required for this type of joints. For economy, sometimes certain patented compounds of sulphur and other materials and neat cement mortar are also used in place of lead. These materials, though cheaper than lead, do not provide as much flexible joint as is provided by lead.

(*ii*) **Flanged joint.** Flanged joints are used for pumping stations, filter plants, and at other locations where it may be necessary to occasionally disjoint the pipe. Cast iron pipe lengths to be joined by this joint are cast in such a way as to have flanges at both ends. In case of steel pipes to be joined by such joints, flanges are separately cast and then screwed down or welded at both the ends of the respective pipe lengths.

Two flanges are brought together, keeping a rubber washer (called gasket) in between them, so as to make them water tight. They are then fixed by means of nuts and bolts (See Fig. 6.19).

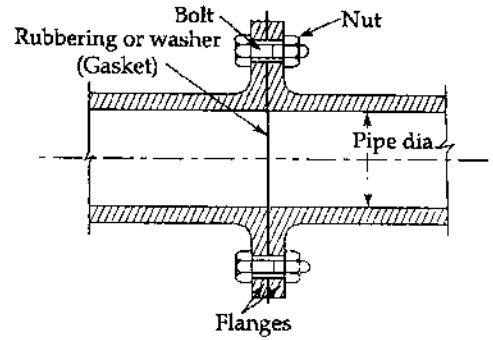


Fig. 6.19. Flanged joint.

These joints are strong but rigid, and hence cannot be used where deflections or vibrations are expected. They are expensive and mostly used for indoor works (such as in pumping stations, filter plants, etc. as pointed out earlier.

(*iii*) **Mechanical joint or Dresser coupling.** This type of a joint is used when it is required to join the plain ends of cast iron pipes. A special type of metallic collar is then fitted and tightened over the abutting ends, thus forming a mechanical joint. One of the most commonly used type of a mechanical joint is a ***dresser coupling***. In this joint, a gasket and an iron ring are slipped over each of the abutting ends of the pipes, and an iron sleeve is inserted between the gaskets. The iron rings are then tightened by means of bolts. (See Fig. 6.20).

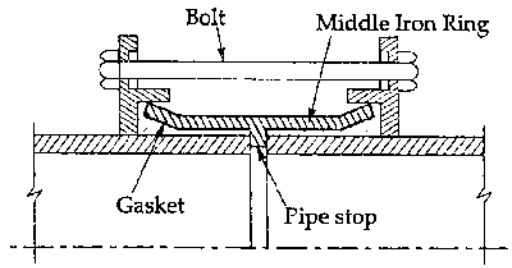


Fig. 6.20. Dresser Coupling Joint (Mechanical joint).

These joints are strong and rigid. They can withstand vibrations and are, therefore, useful for pipes to be carried over bridges or below bridges in hangers.

(*iv*) **Flexible joint.** Flexible joints are used where large scale flexibilities are required. Say for example, when pipes are laid in rivers with uneven beds, large scale settlements may break the ordinary type of joint. Similarly, while laying pipes on curves, the provided joints must be flexible, otherwise they may break. In all such circumstances, flexible joints are used.

The pipes to be provided with such a joint are cast with special types of ends. The socket is spherical ; and the spigot, though plain, is having a bead at

the end. *A retainer ring* is placed over the bead which keeps the special rubber gasket (rubber gasket lined with a special kind of fabric called "*duck*") in position. A split cast iron "gland" ring is then placed over it. They are then tightened by means of nuts and bolts. The assembly is shown in Fig. 6.21. The spigot-end can be moved to give away the desired deflection and nuts are tightened over the gland ring.

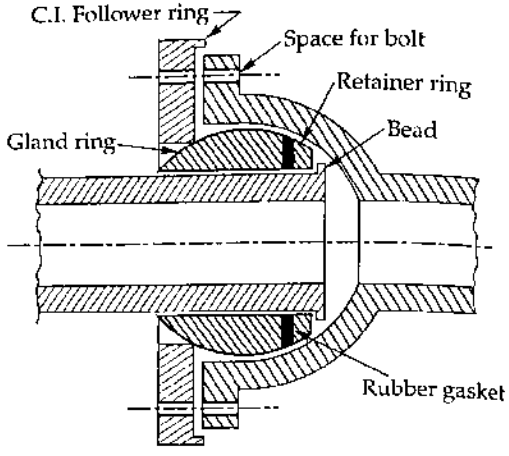


Fig. 6.21. Flexible joint.

(*v*) **Expansion joint.** As pointed out earlier, expansion joints are provided at suitable intervals in the pipe lines, so as to counteract the thermal stresses produced due to temperature variations. For providing expansion joints in cast iron pipes, the socket end is cast flanged and the spigot end is plain. The socket end is connected rigidly to an annular ring which can slide freely over the spigot end. While making this joint, a small space (equal to L. α T.) is kept between the face of the spigot and the inner face of the socket, and the spigot is filled up by means of a rubber gasket. The flanges are then tightened by means of nuts and bolts, as shown in Fig. 6.22.

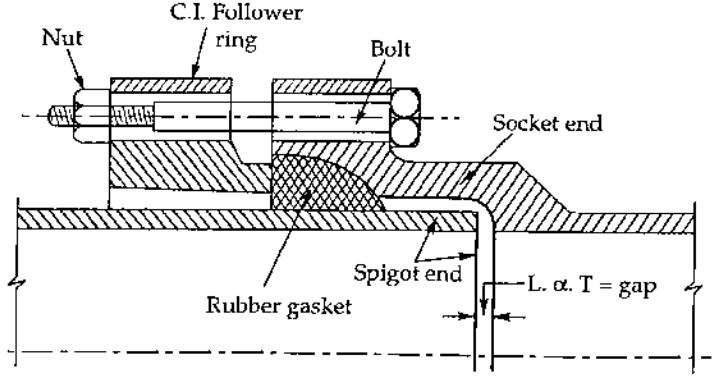


Fig. 6.22. Expansion joint (for cast iron pipes).

When the pipes expand, the socket-end moves forward and the gap left (equal to $L \cdot \alpha L$.) just gets closed. Similarly, when the pipes contract the socket moves backward creating the gap. All the time, the annular ring follows the movements of the socket and maintains the gasket in position, thus keeping the joint watertight.

6.7.2. Steel Pipes. Steel plates of varying thickness for withstanding different pressures are generally bent and welded (or sometimes riveted) so as to manufacture *steel pipes*. Welded pipes are smoother and stronger than riveted pipes and are, therefore, generally used these days. Since steel is

strong in tension, even large size diameter pipes (say upto 6 m or so) can be made of thin shells, and these pipes can be designed for withstanding high internal pressures.

Steel pipes are hence, frequently used for *raw water trunk mains, inverted siphons*, or *pumping mains*, where pressures are high and sizes larger. Owning to the difficulty in making connections, steel pipes are *seldom used for distribution mains*, where cast iron pipes are frequently used. Steel being much stronger than cost iron, large steel pipes constructed for high pressures are much lighter in weight, besides being cheaper, easier to construct and more easily transported than cast iron pipes. There are, however, important disadvantages. *Steel pipes are not adopted to withstanding external loads of backfill, traffic, etc. ; while a partial vacuum caused by emptying of the pipe may cause collapse or distortions, if not designed properly*. Riveted steel pipes suffer loss of capacity owing to the effect of joints and rivets in reducing the net diameter of the pipe. The thinner walls and *greater susceptibility to corrosion* are likely to cause high maintenance charges and shortened life. Steel pipes, under average conditions, should have a life of 25 to 50 years. Unfavourable conditions are : highly corrosive water carried by the pipes and corrosive soil under which the pipes are buried. Sand, clay, shale and ashes are especially likely to cause pitting from the outside, when steel pipes are buried underground. They are, hence, usually preferred for carrying pumped in raw water supplies to the treatment plants on trestles.

Galvanised steel pipes with circumferential corrugations are much more stronger than ordinary steel pipes. They are, therefore, used where large dia pipes of smaller thicknesses are required. Such pipes are usually manufactured in various sizes varying from 20 cm to 2 m in diameter and are lighter and can be more easily transported over distances.

Since steel pipes are not cast but made by riveting or welding steel plates, they may, therefore, vary in length and diameter both. The various pipe lengths are connected by riveted or welded joints. Expansion joints are generally not required in steel pipes which are buried under the ground, because they are not subjected to large temperature changes. However, pipes which are exposed to the atmosphere may require expansion joints, so as to minimise temperature stresses. A typical expansion joint used in steel pipes is shown in Fig. 6.23.

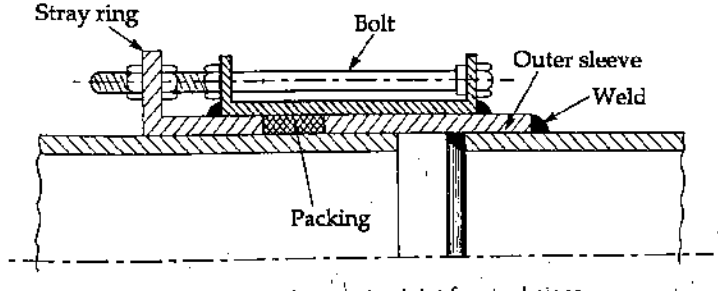


Fig. 6.23. Typical expansion joint for steel pipes.

Steel pipes get rusted quickly, which reduces their life as well as carrying capacities. Since they get easily affected by acidic or alkaline waters, and even atmospheric agencies may produce adverse effects on them, they usually

necessitate protective coatings from inside as well as outside. Plain steel pipes are, therefore, normally not used in India, and in their place, steel shells, coated from inside as well outside with 1 : 2 cement mortar (called **hume steel pipes***) are generally used. Such pipes combine the advantages of steel (being strong in tension) as well as those of concrete (being strong in compression) and are, therefore, widely adopted in India, Steel pipes with other types of coatings are frequently used in U.S.A.

6.7.2.1. Corrosion of metal pipes. When water flows through a metal pipe (such as a cast iron or a steel pipe), it attacks and disintegrates the surface of the pipe. The material of the pipe thus gets dissolved and rusted, thereby reducing the life and carrying capacity of the pipe. This phenomenon which leads to the progressive disintegration of the pipe is known as *corrosion* or strictly speaking chemical corrosion. The corrosion of pipes not only reduces their lives and carrying capacities, but may also impart colour and odour to the flowing water and may make it dangerous for drinking. Various theories have been advanced to explain the reasons of corrosion, but the most accepted theory is the theory of electrolysis which is often caused by galvanic action.

In general, the corrosion of metal pipes may occur if iron enters solutions as positive ions (*i.e.*, Fe^{++} ions) and combines with the negative ions of water (*i.e.* OH^- ions), thus forming ferrous hydroxide [$Fe(OH)_2$]

$$Fe^{++} + 2OH^- \longrightarrow Fe(OH)_2$$

This ferrous hydroxide reacts with the dissolved oxygen to form ferric hydroxide [$Fe(OH)_3$]. This ferric hydroxide is in the form of insoluble red precipitate and gets deposited on the pipe surface. The process, therefore, leads to the formation of ***tubercules*** of ferric hydroxide on the inside surface of the pipe. The initial film of rust coating adheres to the pipe interior and tries to protect the pipe from further corrosion. But this coating is not so impermeable, and some corrosion continues. *This deposit of ferric hydroxide formed on the interior surface of the pipe is known as* **tuberculation**, *and it increases the roughness of the pipe and thereby reducing its carrying capacity.* The decrease in the carrying capacity is generally large. So much so, that cast iron pipe of a small diameter can have its capacity reduced to 50 per cent in 5 years or so. Such pipes will have to be reconditioned by cleaning and lining their interiors by centrifugally cast cement linings.

Corrosion of metal pipes may often occur due to electrolysis. Stray electric currents** from any grounded device may enter the pipe, and flow through it to some point of departure. Also, when two dissimilar metals are immersed in an electrolyte such as water, an electric potential is developed and an electric current starts flowing between the metals. The metal which lies high in the "electrochemical series" will get dissolved, and deposition will start, on the metal which lies low in the "electrochemical series". Thus, the metal which is high in the series becomes the anode, and the metal which is lower in the series becomes the cathode. This type of potential may develop between the pipe metal and the pipe fittings (of different metals) ; and also between the pipe metal and other metals present as impurities in the pipe metal.

*Explained in article 6.7.3.

**Only D.C. currents may cause electrolysis and consequent corrosion. A.C. currents, due to constant change of polarities do not yield to any appreciable corrosion.

Thus, if the material of the pipe fittings or the impurities present in the pipe material are such as to fall in the electrochemical series at a place lower than that of the pipe metal itself*, naturally electrolysis will set up and the pipe metal will go on dissolving. The dissolved metal being washed down by the flowing water, and thus making it impure. The corrosion continues indefinitely, if not checked, thus leading to the complete destruction of the pipe.

Both the pH value** and the oxygen content of the water affect the above reaction. However, under normal ranges of pH values generally encountered in potable waters, the rate of corrosion is nearly independent of pH and is proportional to the dissolved oxygen concentration. The basic chemical equations are :

At the anode $$Fe \longrightarrow \underset{\text{(Electrons)}}{Fe^{++} + 2e}$$

At the cathode $$\underset{\text{Oxygen in solution}}{\tfrac{1}{2}O_2} + \underset{\text{Water}}{H_2O} + \underset{\text{Electron}}{2e} \longrightarrow \underset{\text{Hydroxyl ions}}{2OH^-}$$

$$Fe^{++} + 2OH^- \longrightarrow \underset{\text{Ferrous hydroxide}}{Fe(OH)_2}$$

When water is alkaline and free from carbon dioxide

$$2Fe(OH)_2 + \tfrac{1}{2}O_2 + H_2O \longrightarrow \underset{\text{Ferric hydroxide}}{2Fe(OH)_3} \text{ (sticks to anode)}$$

When water is acidic and contains free CO_2

$$Fe(OH)_2 + 2CO_2 \longrightarrow \underset{\text{Ferrous bicarbonate}}{Fe(HCOC_3)_2}$$

$$2Fe(HCO_3)_2 + O_2 + H_2 \longrightarrow \underset{\text{Sticks to anode}}{2Fe(OH)_3} + 4CO_2 \uparrow$$

Just as the interior of a pipe surrounded by corrosive water gets corroded (as explained above), in the same way, when the exterior of the pipe is surrounded by soils of high electrical conductivity, electrolysis is likely to set up, causing corrosion of the pipe metal from outside also.

Corrosion Control. Corrosion of metal pipes may be reduced in various ways as described below :

(*i*) ***Protective coatings.*** The pipe surfaces are coated with coatings of paint, galvanizing, bituminous compounds, cement linings, etc. so as to protect the pipe metal from corrosion. Red lead paint and zinc pigments offer some degree of protection and are often used for painting the exteriors of the pipes. Small sized pipes are often galvanized by dipping them in molten zinc. This is quite effective but is a costly process. Large size pipes are often protected by bituminous coatings or cement linings. Sometimes, 1 : 2 cement mortar is used as a protective coating material.

*The common metals such as copper, tin, lead, etc. are having somewhat lower electrical potential than iron.

**Described in chapter 8.

(*ii*) ***Selecting proper pipe materials.*** The pipe metal may be so chosen as to be more resistant to corrosion. Certain alloys of iron or steel with chromium, copper or nickel have been found to be better than the pure iron or steel. Moreover, if steel or iron are to be used, then they should be as pure as possible.

(*iii*) ***Quality of water.*** The water passing through the pipe should be made as less corrosive as possible. This can be accomplished by raising the pH of water (by adding alkalinity in the form of lime or powdered chalk), by reducing the dissolved oxygen and carbon dioxide, by adding certain chemical compounds which reduces tuberculation (such as sodium hexa-metaphosphate in small doses of the order of 0.5 to 2 ppm).

(*iv*) ***Cathodic protection.*** Electrolytic corrosion can be prevented by connecting the pipe with negative terminal of a D.C. generator and connecting its positive terminal with blocks of zinc or magnesium buried in the ground near the pipe. This ensures the flow of a current from the buried block (anode) towards the pipe (cathode), which is in opposition to the corrosion process of electrolysis. This, therefore, retards the migration of metallic ions from the pipe (as the current now flows towards the pipe and not away from the pipe).

These are the various ways in which corrosion may be reduced or controlled. However, corrosion control is a specialised field and experts must be consulted whenever problems are encountered.

6.7.3. Cement Concrete Pipes. Plain cement concrete pipes are manufactured in small sizes (*i.e.* upto a maximum of about 0.6 m diameter) ; while they are reinforced with steel for large diameter pipes. They are easily available in sizes upto diameters say about 1.8 metres and may be got manufactured for larger diameters say upto about 4.5 metres, on special orders. These pipes may either be prepared at site by transporting various ingredients (*i.e.* cement, steel, aggregates, water etc.) or can be manufactured in factories and then transported to site. They are known as ***cast in situ*** pipes in the former case, and ***precast pipes*** in the latter case. Cast in situ pipes are useful when the site conditions are difficult and where it may be difficult to carry the pipes. But when pipes are cast at site, lesser supervision and check is possible as compared to the case of precast pipes which are cast in factories and thus subjected to greater quality control.

Reinforced Cement Concrete (R.C.C.) pipes can be manufactured in three different ways, *viz*.

(*a*) pipes having bar and mesh reinforcement, and concrete poured by usual ordinary methods of concrete-pouring and tamped.

(*b*) pipes made by rotating the mould (or the form) rapidly about the pipe axis. The mould contains concrete and fabricated reinforcement. The centrifugal force throws off the concrete which then spreads in a uniform layer over the internal surface of the mould and embed the reinforcement, thus providing a high density watertight concrete surface. This type is known as ***centrifugal type***.

(*c*) the third type of pipes are made by lining thin cylindrical steel shells both internally and externally with rich cement concrete. These are stronger

and more watertight than the first two and are used for larger pressures. They are known as *cylinder type.*

However, these cylinder type R.C.C. pipes cast by centrifugal action are manufactured under the patented name of ***hume steel pipes*** and are widely used in India. They are, therefore, classified separately.

R.C.C. pipes are generally made from 1 : 2 : 2 cement concrete with maximum size of aggregates as 6 mm. They are provided with circumferential reinforcement to carry hoop tension and a nominal longitudinal reinforcement equal to 0.25% of the gross cross-sectional area of concrete. Their thickness may vary from 2.5 cm to 6.5 cm for pipes of diameters varying from 0.1 m to 1.2 m. Such R.C.C. pipe lengths are joined by placing the protruding end bars of different lengths butting against one another and welding them, and finally filling the gap with rich cement concrete, so as to provide a watertight joint.

With the advancement in prestressed concrete, the **pre-stressed concrete pipes** have also come into the market. In these pipes, the permanent internal stresses upto the desired extent are deliberately introduced in the steel reinforcement, so as to counteract the stresses that are likely to come by external loads when such a pipe is subjected to loading.

The plain cement concrete pipes are used for low heads (of the order of 150 kN/m^2) ; the reinforced cement concrete pipes are used for medium heads upto 750 kN/m^2 ; and prestressed concrete pipes are used for large heads.

Indian Standards. The ordinary reinforced cement concrete pipes are manufactured in India as per the provisions of I.S. 458—1988. They are widely used as distribution mains and pumping mains and are available in diameters ranging from 80 to 1200 mm. The various available standard diameters (internal) are : 80, 100, 150, 250, 300, 350, 400, 450, 500, 600, 700, 800, 900, 1000, 1100 and 1200 mm. They are generally available in three classes (*i.e.* P_1, P_2 and P_3) as shown in Table 6.12.

Table 6.12. Ordinary R.C.C. pipes as per I.S. 458—1988

Category	*Dia available*	*Test pressure in kN/m^2*	*Places where used*
Class P_1	80—1200 mm	200	Used on gravity mains, design pressure not exceeding 2/3 of test pressure.
Class P_2	80—600 mm	400	Used on pumping mains, the design pressure not exceeding 1/2 of the test pressure.
Class P_3	80—400 mm	600	—do—

The steel cylinder reinforced cement concrete pipes are manufactured in India as per the provisions of I.S. 1916—1989, and are available in diameters ranging from 200 mm to 1800 mm. They are used in water mains, sewers, irrigation works, etc. The various classes and their test pressures are given in table 6.13.

Table 6.13. Steel cylinder R.C.C. pipes as per I.S. 1916—1989

Category	*Test pressure in kN/m²*	*Working pressure*
Class 1	500	In general, working pressure for pumping mains is taken as 50% of the test pressure ; and for gravity mains is taken as 2/3rd of the test pressure.
Class 2	1000	
Class 3	1500	
Class 4	2000	
Class 5	2500	
Special class	Above 2500 ; to be specified by purchasers	

The prestressed concrete pipes can be designed for high heads and are manufactured in India as per the provisions of I.S. 784—1978. They are available in diameters ranging from 80 mm to 1800 mm. These pipes are subjected to hydrostatic pressure test as specified in I.S. 3597—1985, and the test pressure must be twice the designed pressure.

Advantages of R.C.C. Pipes are given below :

(*i*) They can resist external compressive loads and do not collapse under nominal vacuums and traffic loads.

(*ii*) They are not corroded from inside by normal potable waters and from outside by ordinary soils.

(*iii*) They are quite strong and their useful life is of the order of 75 years or so.

(*iv*) They are easy to construct either at site or at factories and with local ingredients.

(*v*) The coefficient of expansion being low, expansion joints may not be needed when laid above the ground.

(*vi*) If laid under water, the empty pipes do not float because of their heavy weights.

Disadvantages of R.C.C. Pipes are given below :

(*i*) They are likely to corrode by ground waters due to the presence of acids, alkalis or sulphur compounds.

(*ii*) They are difficult to be repaired.

(*iii*) They cannot withstand very high pressures.

(*iv*) They are heavy and bulky, and hence difficult to transport.

(*v*) Making of connections in them is a little difficult job.

(*vi*) They tend to leak due to shrinkage cracks and porosity.

6.7.4. Hume Steel Pipes. As pointed out earlier, hume steel pipes are R.C.C. spun pipes patented under this name. They consist of thin steel shell coated from inside with cement mortar by centrifugal process. The thickness of the inside coating varies from 1.2 to 3.75 cm depending upon the size of the pipe. They are also coated from outside, so as to protect the steel from external weather or soil action. The thickness of external coating is 2.5 cm for pipes upto 1 metre in diameter, and is 3.75 cm for pipes of larger diameters. The thickness of steel shell depends upon the size of the pipe and also upon the pressure to be borne by the pipe. Like all R.C.C. pipes, they are heavy and difficult to handle.

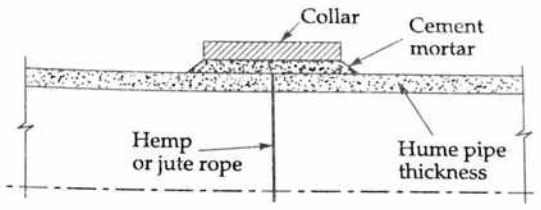
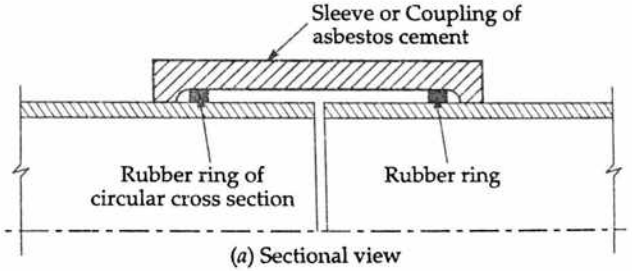


Fig. 6.24. Typical joint for connecting hume pipes.

The most commonly used type of expansion joint in hume pipes is shown in Fig. 6.24. A hemp or jute rope is inserted between the pipes and collared to maintain the alignment of pipes. The remaining gap is then filled up by rich cement mortar as shown.

6.7.5. Vitrified Clay Pipes. They are generally not used as pressure pipes for carrying waters, but are extensively used for carrying sewage and drainage at partial depths. These pipes are free from corrosion and provide a smooth hydraulically efficient surface. They are not used as pressure pipes because clay is very weak in tension, and formation of watertight joints becomes difficult in them. Clay pipes are commonly made in lengths of about 0.6 to 1.2 m or so.

6.7.6. Asbestos Pipes. Asbestos, silica and cement are converted under pressure to a dense homogeneous material possessing high strength, called *asbestos cement*. This material is used for casting these pipes. The asbestos fibre which is thoroughly mixed with cement, serves as reinforcement. These pipes are generally available in different sizes, say from 10 to 90 cm in diameters in about 4 metres lengths. These pipes are generally made in 4 different grades to withstand pressures of 350 kN/m² to 1400 kN/m², as per provisions of IS 1592—1989. These pipes are joined by means of special type of coupling called ***simplex joint***, which consists of a pipe sleeve and two rubber rings which are compressed between the pipe and the interior of the sleeve as shown in Fig. 6.25.



(*a*) Sectional view

(*b*) Photographic view

Fig. 6.25. Simplex joint for A.C. pipes.

Advantages of asbestos pipes are given below :

(*i*) They are light and hence easy to transport.

(*ii*) They can be easily assembled without skilled labour.

(*iii*) They are highly resistant to corrosion.

(*iv*) They are highly flexible and may permit as much as 12° deflection in laying them around curves.

(*v*) Expansion joints are not required as the coefficient of expansion is low and the joints are also flexible.

(*vi*) They are very smooth and thus provide a hydraulically efficient pipe. Their carrying capacities do not reduce with time.

(*vii*) They are very suitable to be used as small size distribution pipes.

Disadvantages of asbestos pipes are given below :

(*i*) They are costly.

(*ii*) These pipes do not have much strength and are brittle and soft. They are liable to get damaged by excavating tools or during transportation transits.

(*iii*) The rubber joint seals may deteriorate if exposed to gasoline or other petroleum products, and hence cannot be used for transporting petroleum products.

6.7.7. Miscellaneous Types of Conduits. Various other materials which may be used for manufacturing pipe conduits are : copper, wrought iron, plastics, etc. **Copper pipes** are very costly although they are highly resistant to acidic as well as alkaline waters. They can be bent easily and do not sag due to heat. They are, therefore, very useful for carrying hot water in the interior of the buildings. **Wrought iron pipes** are lighter than cast iron pipes and can be more easily cut, threaded and worked. They are more costly but neat in appearance. They are generally manufactured in small sizes and are very useful for indoor works. However, they corrode quickly and are less durable. They are, therefore, generally protected by galvanising them with zinc coatings, and they are then known as **Galvanised iron pipes.**

Plastic pipes are lighter and free from corrosion. But they are of low strengths and less durable. Moreover, they cannot withstand high temperatures exceeding 60°C or so. They may, however, be used for minor works in house connections, etc. ; in which they are finding extensive use these days. The manufacture of low and high density Polyethylene pipes for potable water supplies, in India is governed by the provisions of I.S. 3076—1985 and I.S. 4984—1995 respectively. The unplasticised PVC pipes for potable waters are manufactured according to I.S. 4985—1988.

They can withstand pressures upto about 1000 kN/m^2 (100 m head of water).

6.8. Laying of Water Supply Pipes

Pipes are used for conveying water from the source to the city, and also for distributing the same within the city among the consumers. When pipes are used for bringing water from the source to the city, only one or two lines of the same size are generally laid, whereas when they are used for distributing water, they are generally of varying sizes, having many connections and branches. In the former case, pipes used are generally made of steel or R.C.C., whereas in the latter case, cast iron pipes (with bituminous coatings) are generally used. Galvanised iron pipes are used for service connections to the various houses.

Pipes are laid either above the ground or below the ground. Generally, the pipes bringing water from the source to the city, are laid on the ground, whereas the distributing mains taking the water within the localities, are laid below the roads and streets. The pipe lines, in general, should follow the profile of the ground, and that location is chosen which is most favourable with respect to the resulting construction costs and pressures.

When pipes are laid on the ground (or above the ground), they must be laid on a well compacted formation of suitable width so as to avoid future settlements. They may be laid directly over the compacted soil formation or may be laid over small masonry or cement concrete supports at 6 to 12 metres apart. This arrangement, though costly, facilitates inspection, maintenance, repairs, etc., and is generally adopted these days.

When pipes are buried under the ground, they are laid in trenches excavated up to the required depths. The top of the pipe is generally kept about 1 metre below the road surface, so as to minimise the impact and traffic loads transmitted to the pipes. The width of the trench is generally kept 30 to 50 cm more than the outside diameter of the pipe, subjected to a minimum of about 75 cm, which is required for conveniently laying the pipe.

6.9. Pipe Appurtenances

In order to isolate and drain the pipe line sections for tests, inspections, cleaning and repairs ; a number of appurtenances such as gates, valves, manholes, insulation joints, expansion joints, anchorages, etc., are provided at various suitable places along the pipe lines, as described below.

6.9.1. Gates and Valves in Pipe Lines. A large number of different types of valves are required for the proper functioning of the pipe line, as described below and shown in Fig. 6.26 :

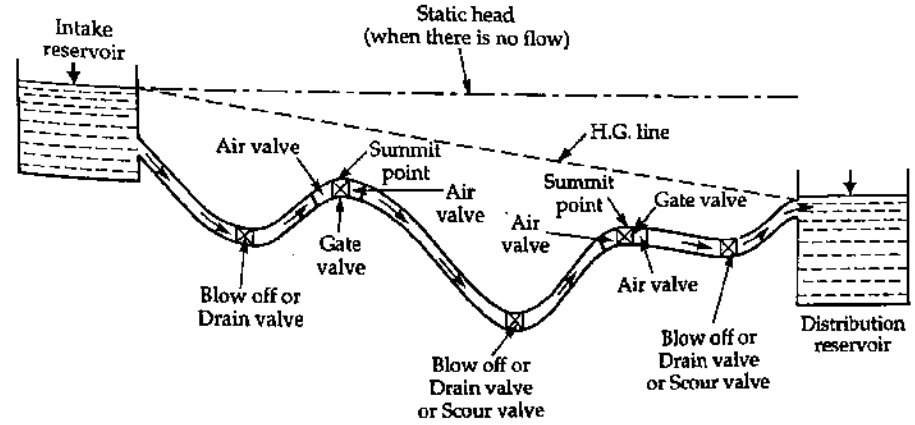


Fig. 6.26. Profile of the pressure pipe showing the locations of gates and valves.

(*a*) **Gate valves or Sluice valves.** Gates or sluice valves are used to regulate the flow of water through the pipes. They are similar to gate valves used in dams but are not so large. In large pipe lines, bringing water from the source to the city, they are generally located along the pipe line at intervals of about 3 to 5 kilometres, so as to divide the pipe line into different sections. Thus, during repairs, only one section can be cut off at a time, by closing the gate at both ends of the section.

The gate valves are usually placed at the *summits** (Fig. 6.26) of the pressure conduits, because when so placed at these points of low pressures, they can be of cheaper and less stronger materials, and also, they can be operated easily with less force. For economy, in large diameter pipes, valves of smaller diameter than the pipe itself are generally used. However, the saving in the valves cost must be balanced against the increased head loss through the valve (because head loss through the valve increases with the reduction in the size of its opening because of higher velocity of flow) including the extra loss in contraction and re-expansion.

The most commonly used type of a gate valve or a sluice valve is shown in Fig. 6.27. The valve is made of cast iron with brass, bronze or stainless steel mountings. The ends of the valve are properly jointed on both sides of the pipe lengths, by suitable standard joints (described earlier). In its simplest form, the valve consists of a wedge shaped circular disc fitted closely in a recess against the opening in the valve. This is connected to a wheel (or handle) by means of a threaded spindle, as shown. The valve can be raised or lowered by rotating the wheel from the top either manually or mechanically.

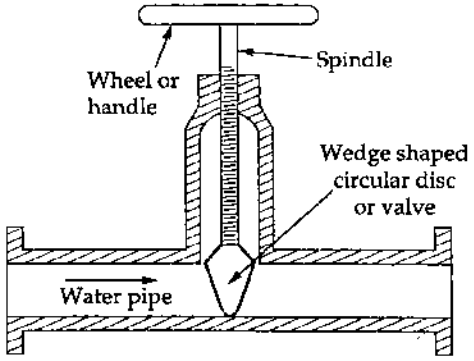


Fig. 6.27. Vertical section of a typical sluice valve.

Double disc parallel seat cast iron valves, as shown in Fig. 6.28, are most commonly used in the distribution mains.

*Summits are the points where the pipe is nearer to the hydraulic gradient line, and thus "summits" represent the points of low pressures.

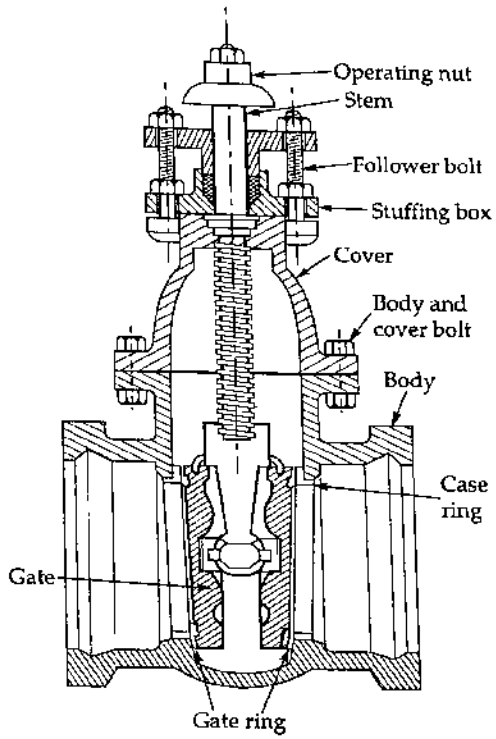


Fig. 6.28. Double disc type or Gate type sluice valve.

(*b*) **Air valves.** Air valves are the special kind of valves which are generally placed along the pipe line at "summits" on both sides of the sluice valves (as shown in Fig. 6.26) and also on the downstream side of all other sluice valves. When placed on summits which are very near the H.G. line or sometimes above the H.G. line (especially during –ve water hammers), they ensure the safety of the pipe against *collapse**. Similarly, when placed below the ordinary sluice valves, such as shown in Fig. 6.29, they will protect the pipe against the negative pressures which may be developed, when during steady flow the valve *A* is suddenly closed. (∵ as the valve *A* is closed, water continues to flow and vacuum gets created downstream of *A*).

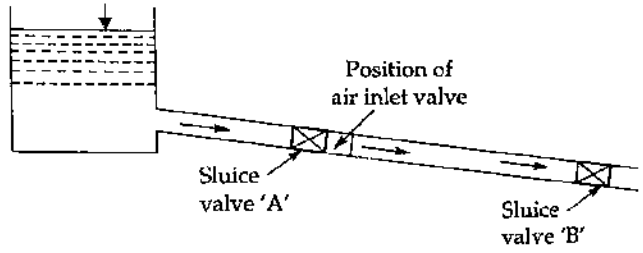


Fig. 6.29. Position of air inlet valve below a sluice valve in a pipe line.

*Failure by compression due to vacuum.

Such a valve will open out automatically as soon as the pressure in the pipe falls below a certain fixed predetermined value, and thus allowing air to enter the pipe. Such a valve is, therefore, known as an ***air inlet valve.***

Just as outside air is required to be admitted to a pipe, similarly sometimes air gets accumulated at high points when the supply is restored and the pipe is refilled after repairs. The accumulated air obstruct the free flow of water and the pipe may get air locked. ***Air relief valves*** are, therefore, required to be provided at all summits to remove the accumulated air under such circumstances.

Specially designed air valves which can function both ways (*i.e.*, admit air when negative pressures get developed in the pipe and remove the displaced air when the line is filled up with water after completion of repairs, etc.) are, therefore, generally provided on both sides of summits.

The most commonly used type of a **single orifice air valve** acting as *air-inlet* as well as an *air-relief* valve is shown in Fig. 6.30. It consists of a cast iron chamber (circular or rectangular), a lever and a poppet floating valve as shown. The chamber is fitted to the pipe top, and the weighted float and the lever in it are so adjusted that when the chamber is filled with water under pressure from the pipeline below, the float and the lever remain in a raised position and the valve closes. But when air goes on accumulating at the top and builds up some pressure, the water level gets depressed, the float sinks down along with water level, and thus opens the valve. The air then escapes out but only to the extent that no water escapes along with air. The water level in the chamber, therefore, rises again ; the float is lifted, and the valve is closed.

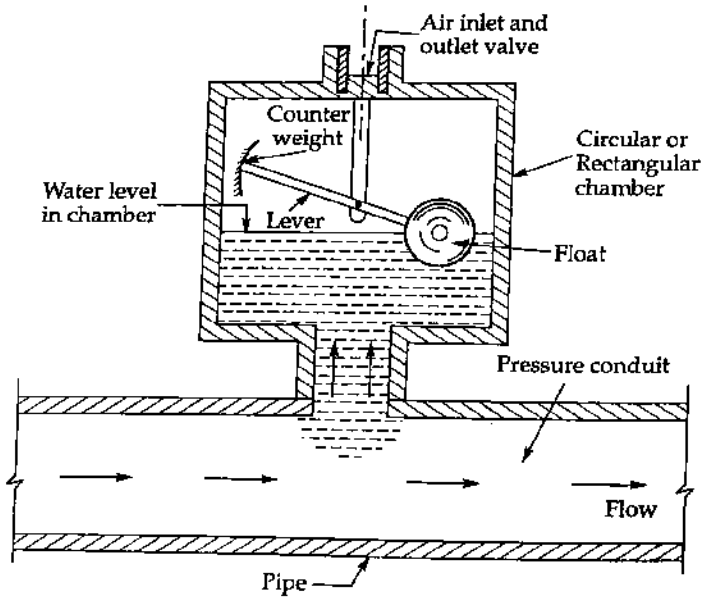


Fig. 6.30. Vertical section of a poppet type air-valve.

In a similar manner, these automatic valves help in admitting outside atmospheric air into the pipe when the negative pressures get created in the pipe. As the pressure decreases, the float drops and uncovers the opening and thereby admitting air in the pipe so as to counterbalance the negative pressures.

These ***poppet type air valves*** are automatic, and are very useful as they can function automatically *both ways.*

A *double acting orifice air valve* is also sometimes provided. It is made up of two cast iron chambers interconnected and provided with separate balls of different composition and orifices designed to operate for different conditions of high pressure and low pressure of flow in pipe line.

(*c*) **Blow off valves or Drain valves or Scour valves.** In order to remove the entire water from within a pipe (after closing the supply), small gated off-takes are provided at low points, as shown in Fig. 6.26. These valves are known as *blow off valves* or *drain valves* or *scour valves*. These valves, are necessary at low level points for completely emptying the pipe for inspection, repairs, etc. When opened, water comes out of these valves quickly under gravity and they are made to discharge water into some natural drainage channel or into a sump from which the water can be pumped out. It may, however, be stressed that there should be no direct connection between the valve and the sewer or the drain, so as to avoid the possibility of pollution travelling into the water pipe. For safety, two drain valves are generally placed in series, so as to reduce the chances of such pollution reaching the water in the conduit.

(*d*) **Pressure-relief valves.** Water hammer pressures in pressure pipes can be reduced by using *pressure relief valves*. Such a valve is adjusted to open out automatically as soon as the pressure in the pipe exceeds a certain fixed-predetermined value. Due to the opening of this valve, certain water will get out of the pipe, and thus, reducing the pressure in the pipe. As soon as the water hammer pressure reduces, and the pressure in the pipe falls up to the fixed value, the valve will close automatically. Such type of valves are useful on small pipelines, where the escape of a relatively smaller amount of water will alleviate water hammer pressures. A simple sketch of a pressure relief valve is shown in Fig. 6.31. Since the positive water hammer pressure is

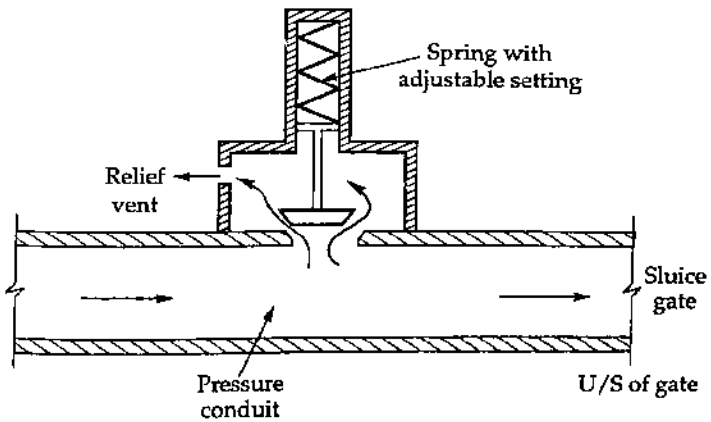


Fig. 6.31. Vertical section of a pressure relief valve in open position.

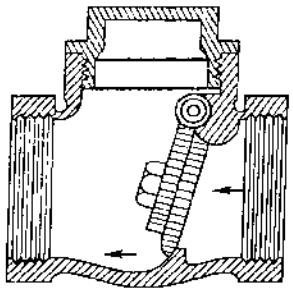
developed due to the sudden closure of a sluice valve on its upstream side, such relief valves may be provided on the upstream side of the sluice valves.

Even if not provided specifically for water hammer, such valves are often placed along the pipe line at suitable intervals (especially at low points where the pressures are high), so as to function during emergencies, when pressure rises in the pipe above the design value, and thus to help in protecting the pipe joints from getting loosened or the pipes from getting burst.

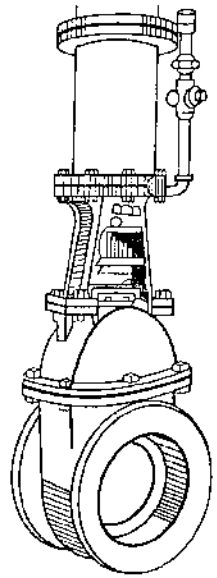
(*e*) **Check valves or Reflux valves.** Check valves are also sometimes called ***non-return valves*** because they prevent water to flow back in the opposite direction. They may be installed on the delivery side of the pumping set, so as to prevent the back flow of stored or pumped water, when the pump is stopped.

Check valves are also installed on pump discharges to reduce water hammer forces on the pump.

Such a valve may be a simple *swing check* or *ball devices* (Fig. 6.32) in small lines ; but in large installations, they should be designed to close slowly, usually with discharge of some water through a bypass.



(*a*) Swing type of check valve (flap type)



(*b*) Hydraulic cylinder valve with piping and four-way control valve.

Fig. 6.32

A *swing type* of check valve consists of a disc or a flap covering the inside area of the pipe, which opens only in one direction, as shown in Fig. 6.32 (*a*). So when water tends to flow in the opposite direction, the flap or the disc closes the opening, and the flow is prevented.

The simplest type of a check valve is a ***flap shutter*** hinged at the outlet end of a pipe, which opens out in the direction of flow by the pressure of the flowing water, but closes of its own weight when the flow in the permissible direction ceases, and it cannot open out in the opposite direction.

Check valves are also required at inter-connections between a polluted water system and a potable water system, so as to prevent the entry of pollution into the pure water. A check valve installed at the end of a suction line is called a ***foot valve***, and prevents draining of the suction when the pump stops.

6.9.2. Manholes. Manholes are provided at suitable intervals along the pipe line, so as to help its laying, and to serve for inspections and repairs. They are generally provided on large pipe lines bringing water from the source to the city at intervals of about 300 to 600 metres or so. They are usually provided in case of steel, hume steel, or R.C.C. pipes (which are commonly used for conveyance of water from the source to the city) and are less common on cast iron pipes.

6.9.3. Insulation Joints. Insulation joints are provided along the pipe lines at suitable intervals, so as to insulate the pipe against the flow of stray electric currents, and thus, to check electrolysis. Rubber gaskets or rings can be provided as insulators, in between the pipe lengths, so as to prevent the flow of electric current between them. Similarly, sometimes, sufficient length of the pipe is covered with rubber covering, so as to provide appreciable resistance to the flow of current.

6.9.4. Anchorages. As pointed out earlier, the pipes try to pull apart and get out of the alignment at bends and other points of unbalanced pressures. At such places, the forces exerted on the joints due to longitudinal shearing stresses caused by these unbalanced pressures are enormous, and the joints may get loosened, ultimately leading to excessive leakage or failure of the pipe. In all such circumstances, in order to prevent the pipes from pulling apart, pipes are *anchored* by firmly embedding these portions in massive blocks of concrete or masonry, which absorbs the side thrusts.

Similarly, when pipes are laid on steep slopes, they try to slip and thereby pull apart, and the resistance of their joints may be insufficient to balance the longitudinal shearing stresses. Pipes are, therefore, anchored under such circumstances also. In the same way, when pipes are rigidly jointed without or with inadequate provisions of expansion joints, they are anchored, so as to control the loosening of the joints because of the increased longitudinal stresses caused by expansions and contractions due to temperature changes.

The anchors consisting of cement concrete or masonry blocks are generally used. The pipes must be well bonded to these anchors by angle-irons welded on to the pipe. In cast iron pipes, the anchors may take the form of lugs, cast on such pipes and their fittings, to hold tie rods that prevent movement of the pipe line.

6.10. Testing of the Pipe Lines

After a pipe line has been laid, fitted with all appurtenances and accessories, painted both from inside as well as outside by means of protective paints, etc., the pipe line will be tested for the soundness in its construction. The soundness of the construction is examined by performing the **pressure test** on

the pipe line. The step by step procedure adopted for performing this test is described below :

(*i*) The pipe line is tested from section to section. Thus, at a time, only one particular section lying between two sluice valves is taken up for testing.

(*ii*) The downstream sluice valve is closed, and water is admitted into the pipe through the upstream sluice valve. The air valves will be properly operated during filling up of the pipe.

(*iii*) The upstream valve, through which water was admitted, is closed, so as to completely isolate the pipe section from the rest of the pipe.

(*iv*) Pressure gauges are then fitted along the length of the pipe section at suitable intervals (say 1 km or so) on the crown, through holes left for this purpose.

(*v*) The pressure in the pipe line is now raised by means of a small hand force pump or a hydraulic pressure pump, till the test pressure (to be measured on the pressure gauge fixed on the pipe) is nearly 25-50% above the highest working pressure.

(*vi*) The pipe and the joints are then visualised for water tightness. The applied test pressure should also maintain itself without any appreciable loss during the observation period, which may be at least 4 hours.

When the field test pressure is less than $\frac{2}{3}$rd the works test pressure, then the observation period should be increased to atleast 24 hours.

(*vii*) The pipe is finally emptied through drain valves, and the observed defects (in the test) are rectified, so as to make the line fit for use. The pipe is again tested by repeating the test, so as to ensure proper rectification of defects already done.

After the satisfactory completion of the pressure test, a **leakage test** at a pressure to be specified by the authority for a duration of 2 hours may also be performed. Leakage is defined as the quantity of water that is required to be supplied for maintaining the specified leakage test pressure after the pipe has been filled with water and the air is expelled.

In a newly laid pipe line, there should generally be no leakage. Moreover, the *allowable leakage* during the maintenance stage of pipes carefully laid and well tested during construction, should also not exceed the value given by Eq. (6.34) below :

$$q_L = \frac{N.D\sqrt{P}}{115} \qquad ...(6.34)$$

where q_L = Allowable leakage in cm^3/hr

N = No. of joints in the length of the pipe line

D = Diameter of pipe in mm

P = The average test pressure during the leakage test in kg/cm^2 (*i.e.* 10 m of water head)

Disinfection of Pipe Lines before Use. After the pipe line has been tested and corrected for defects, it is ready for transporting untreated water to the city from the source. However, when the pipe lines are carrying treated water, they must be disinfected before use. The pipes are disinfected by keeping them full with water and adding chlorine in amounts, as to maintain a residue of 50 mg/l (*i.e.* 50 ppm). This residue is maintained for 12 hours and the pipe is emptied and flushed with fresh treated water, thus making the pipe ready for carrying potable water to the consumers or to the storage tanks.

PROBLEMS

1. Why are the pressure pipes most commonly used for conveying water from distant sources to the towns of supply ? What are the drawbacks of open channels and masonry aqueducts in this respect ?
2. (*a*) Discuss as to why the raw water is carried-from its source to the city treatment plant, through *pressure conduits* as against the gravity canals ?
   (*b*) Discuss critically the statement "the pressure pipes can although go up and down the hydraulic gradient line, yet for economical considerations, they should hug the hydraulic gradient line in profile and the straight line in plan".
3. (*a*) What design criterion is commonly employed in finding out the sizes of the gravity mains used for conveying water from high level impounded reservoirs to towns situated miles away ?
   (*b*) A town having a population of 1.2 lakhs is to be supplied with water from a reservoir 5 km away. It is stipulated that half the daily supply of 200 lphd will have to be delivered in 8 hours ; what should be the size of the supply main if the head available is 16 metres. Take the value of C for the main as 100 and use Hazen-William formula. **[Hint.** Follow example 6.3]
4. Discuss briefly the common stresses produced in pipe lines used for conveying water. What arrangements are made in their design and construction to resist them ?
5. (*a*) What is meant by 'water hammer', and how is it produced in pipes conveying water under pressure ? What precautions should be taken and arrangements made to reduce its effects ? What provisions are normally made in the design of pipe lines on this account ?
   (*b*) A reflux valve on a 75 cm dia steel main, 3 km long, is closed in 1 minute when the rate of flow was 60 million litres per day. What extra pressure will get developed due to the closing of the valve, assuming E for steel as 2.1×10^6 kg/cm^2, and thickness of the pipe shell as 10 mm.
   [Hint. Follow example 6.10]
6. (*a*) How are temperature stresses produced in pipe lines laid above the ground ? Describe by drawing suitable sketches, arrangements made to avert them.
   (*b*) Estimate the temperature stress produced in 0.9 m dia steel main laid above the ground with the following data :
   (*i*) Distance between anchor supports = 30 metres ;
   (*ii*) Variation in temperature = 16°C.
   (*iii*) Thickness of pipe shell = 10 mm.
   (*iv*) Co-efficient of expansion of steel = 12.6×10^{-6} per °C.
   (*v*) E for steel = 21×10^7 kN/m^2.
   (*c*) If an expansion joint is provided, what minimum sliding gap will be required to avert the above stresses ?
   (*d*) Find out the thrust imposed on the anchor supports, if the expansion joint fails to operate due to poor maintenance, assuming that 50% of it is lost in overcoming the skin friction with the ground.

7. (*a*) Enumerate the various forces which may act on pressure conduits carrying water supplies. Also discuss briefly, any two of these forces.

(*b*) A 0.6 m dia cast iron main is subjected to an internal pressure of 420 kN/m^2. Assuming a safe working stress of 56,000 kN/m^2 in tension, calculate

(*i*) the thickness of the shell necessary to resist the internal pressure and

(*ii*) if the above pipe is buried in sandy soil weighing 16 kN/m^3, with a cover of 1.5 m, what will be compressive force produced on this account ?

[**Hint.** (*a*) use eq. 6.13, and work out t = 2.25 min. **Ans.**

(*b*) use eq. 6.21, and work out W = 21.92 kN/m length of pipe. **Ans.**]

8. What are the different materials, which are commonly used for water supply pipes ? Discuss their comparative merits and demerits.

9. Briefly and critically discuss the use of cast iron, steel and R.C.C. as materials for water supply pipes. How are the first two types of pipes get corroded ? Explain this corrosion phenomenon and also suggest remedial measures.

10. (*a*) Discuss briefly the procedure followed in laying and testing the water supply mains.

(*b*) Enumerate the various appurtenances which are generally provided while laying water supply mains bringing water from a source to the city. Sketch and explain importance and functioning of gate valves, reflux valves, and manholes in this context.

11. (*a*) Distinguish between the following :

(*i*) Expansion joint and flanged joint ;

(*ii*) Cathodic protection and protective linings ;

(*iii*) Gravity conduits and pressure conduits ;

(*b*) Give reasons for the following :

(*i*) A hard rubber gasket is used in a flanged joint ;

(*ii*) The repairs of cement concrete pipes are difficult ;

(*iii*) Copper pipes are used to convey hot water ;

(*iv*) The corrosion of pipes affects their carrying capacity.

12. Write short notes on any four of the following :

(*i*) Cement concrete and R.C.C. water mains ;

(*ii*) Corrosion of metal pipes ;

(*iii*) Testing and disinfection of water mains ;

(*iv*) Pipe appurtenances in laying water supply mains ;

(*v*) Jointing of cast iron water supply mains.

—7

Pumps for Lifting Water

7.1. General Introduction

In a water supply scheme, pumps are required at one or more of the following stages :

(*i*) To lift the water at the source, when the water cannot flow by gravity into the mains (such as in the case of low levelled surface sources and ground water sources).

(*ii*) To lift the water at the treatment plant, if sufficient natural slope is not available, as to cause gravitational flow between different units of the treatment plant.

(*iii*) To lift the water after the treatment, so as to force the water into the distributing mains, either directly or through a service reservoir.

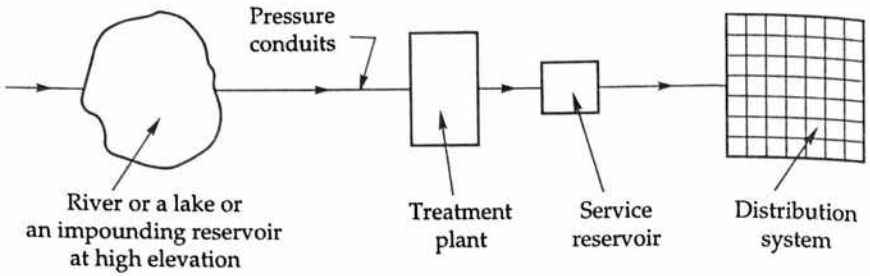
(*iv*) When the pressure in the distributing mains has to be increased or boosted at some intermediate points within the distribution system, so as to enable the water reach up to the reqd. height of three storeys.

In case of surface sources situated at high elevations in hills or in mountainous areas, it may be possible to bring the water from the source to the centre of supply, and then to distribute it among the consumers by mere gravity alone. But such cases are rare and usually, however, the water has to be lifted by pumps at one or more points. Sometimes, the *first lift* is required just at the source, when the water drawn into the sump well through the intake conduit, is pumped into the pressure pipes, leading the water to the city or the treatment plant. In such cases, the second and a bigger lift is then generally required after the "treatment" so as to force the water into the distribution system or to store it in the elevated tanks. Sometimes, no first lift is required at the source, as the water is transported from the source to the city (or the treatment plant) by mere gravity. However, in certain cases, the first lift may be required at the treatment plant itself.

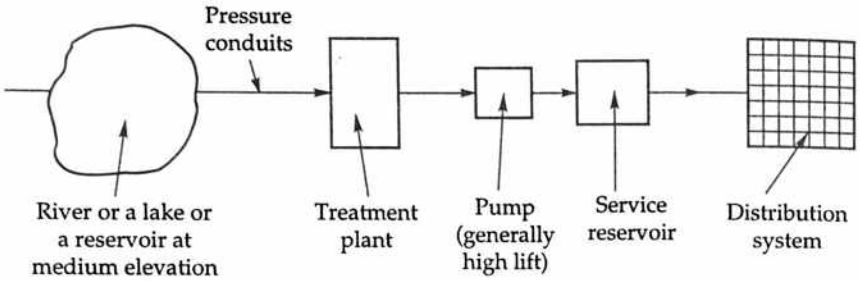
In case of ground water sources, the water is lifted (first lift) from wells or tubewells (unless the source is a spring or an artesian well) and collected into a collecting basin, from where the water enters the treatment plant. And after the treatment, the water is again lifted (second lift) so as to force it into the distribution system. In such sources, it is, however, rare when water is forced into the distribution system in one single lift.

Even in the distribution system, the water may be lifted and boosted at various places, depending upon the economy and ease in construction.

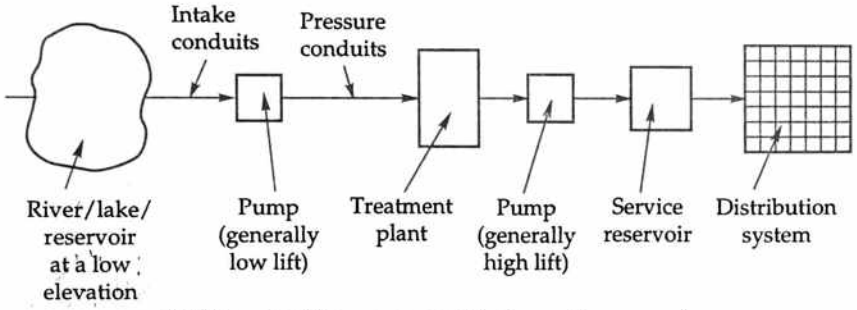
The lifts and other operations which are commonly involved in different cases are sketched in Fig. 7.1. Case (*a*) where absolutely no lift is required is a



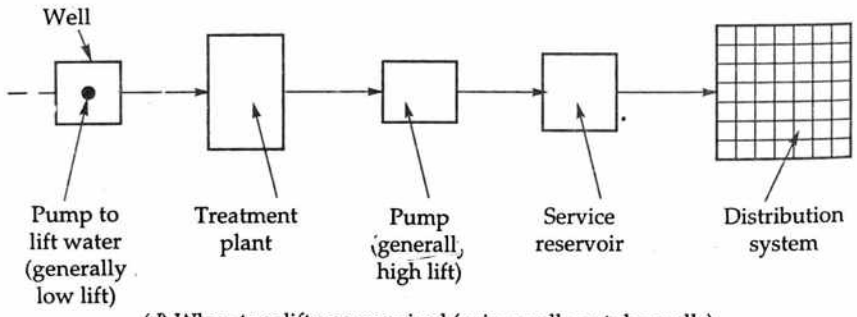
(*a*) When no lift is required (using surface source).



(*b*) When only one lift is required (using surface source).



(*c*) When two lifts are required (using surface source).



(*d*) When two lifts are required (using wells or tube-wells).

Fig. 7.1

gravity flow system ; case (*b*) where partial lift is required is a *combined gravity* and *pumping system* ; and cases (*c*) and (*d*) where lift is a must, is a purely *pumping system*.

7.2. Types of Pumps

There are various types of pumps, but the two types which the hydraulic engineers generally encounter, are :

(1) Roto-dynamic pumps ; and

(2) Displacement pumps.

A ***rotodynamic pump*** has a wheel or a rotating element which rotates the water in a casing, and thus, imparting energy to the water. Such a pump may be of the following two types :

(*a*) *Centrifugal pump ;* and

(*b*) *Axial-flow pump.*

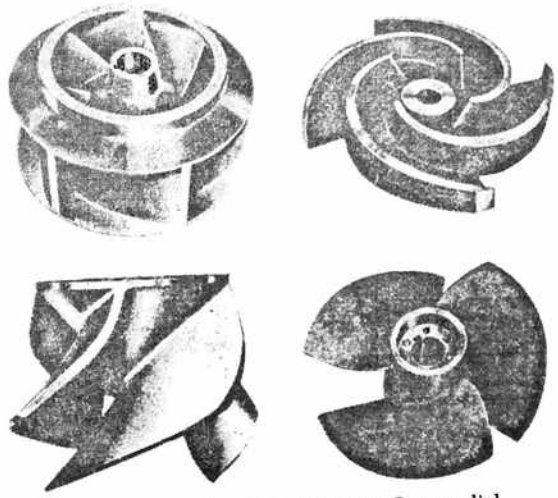
A ***displacement pump*** works on the principle of mechanically inducing vacuum in a chamber, thereby drawing in a volume of water which is then mechanically displaced and forced out of the chamber. Such a pump may be of the following two types :

(*a*) *the reciprocating pump ;* and

(*b*) *the rotary type pump.*

In addition to these two major types of pumps, other types, such as *air lift pumps, jet pumps, hydraulic rams*, etc. are also used under special conditions. All these different types of pumps are described below.

7.2.1. Rotodynamic Pumps. The rotodynamic pumps do have a wheel type rotating element called ***impeller***. The shape of the impeller may be such, as to force the water outward in a direction at right angles to its axis (*radial flow*), or to give water, an axial as well as a radial velocity (*mixed flow*), or to force the water in the axial direction alone (*axial flow*) [Refer Fig. 7.2].

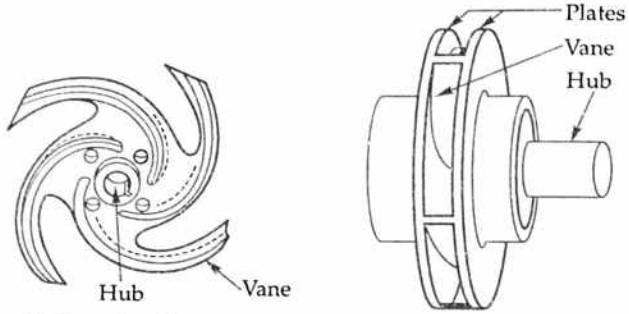


Top left : Closed radial ; Top right : Open radial
Bottom left : Mixed flow ; Bottom right : Propeller

Fig. 7.2. Types of pump impellers of rotodynamic pumps.

Radial flow and mixed flow machines are commonly called **centrifugal pumps,** whereas the axial flow machines (*Vertical propeller*) are called **axial flow pumps**. *Centrifugal pumps are the most common and widely used.*

Types of centrifugal pumps. A radial or a mixed flow impeller which constitutes a centrifugal pump may either be open or closed. The open impeller consists of a hub to which the vanes are attached [Fig. 7.3 (*a*)], while closed impeller is having plates on each side of the vanes [Fig. 7.3 (*b*)].

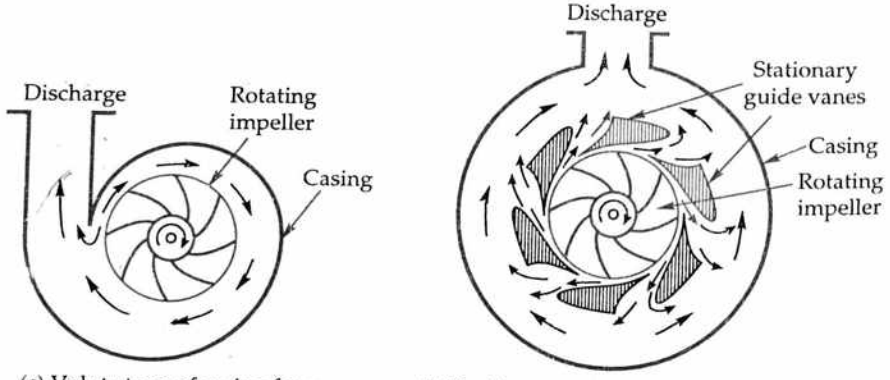


(*a*) Open impellers (*b*) Closed impellers

Fig. 7.3. Centrifugal pump impellers.

The efficiency of the "open impeller centrifugal pump" is generally much less than that of a "closed impeller centrifugal pump". But, however, since the "open impeller" is less likely to be clogged by debris, etc., it is usually adopted for pumping raw waters or sewage containing solids and other impurities.

The casing of a "radial flow centrifugal pump" may either be of a *volute type* or of a *turbine type* [See Figs. 7.4 (*a*) and (*b*)].



(*a*) Volute type of casing for a radial flow centrifugal pump (*b*) Turbine type (Diffuser type) of casing for a radial flow centrifugal pump

Fig. 7.4. Casings for a radial flow centrifugal pump.

In a ***volute type*** of pump, the impeller discharges into a gradually expanding spiral casing, which is designed to produce an equal velocity of flow around the circumference of the impeller and to reduce the velocity of water as it enters the discharge pipe, thus creating the required pressure head. In a

turbine type or ***diffuser type*** of pump, the impeller is surrounded by stationary guide vanes which reduce the velocity of water before the water enters the casing, thus converting the velocity head into pressure head in the casing itself. The casing surrounding the guide vanes is generally circular and concentric with the impeller. The velocity of the water is thus more completely converted into pressure head, resulting in higher efficiency than what is possible in a volute pump.

When only one impeller is used in a pump, it is known as a *single stage pump* ; but when two or more impellers are arranged and placed in such a way that the discharge from one impeller enters the eye of the next impeller, it is called a *double stage* or a *multistage pump*. Such pumps are useful for high lifts.

In the case of a *bore hole pump*, such as a *deep well* ***vertical turbine pump***, used for pumping ground water, several impellers are installed on a vertical shaft, which is suspended and rotated from the prime mover motor placed at the ground surface. The pump bowls are thus kept below the well water level. This type of a pump is installed in a well casing of limited size, and the entire assembly has to be of small diameter (See Figs. 4.57 and 7.5).

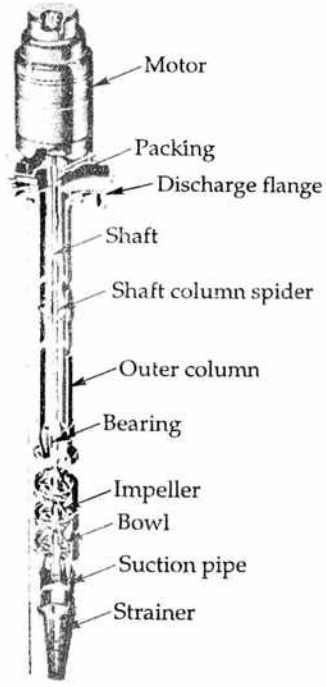


Fig. 7.5. Prospective view of a deep well vertical turbine pump.

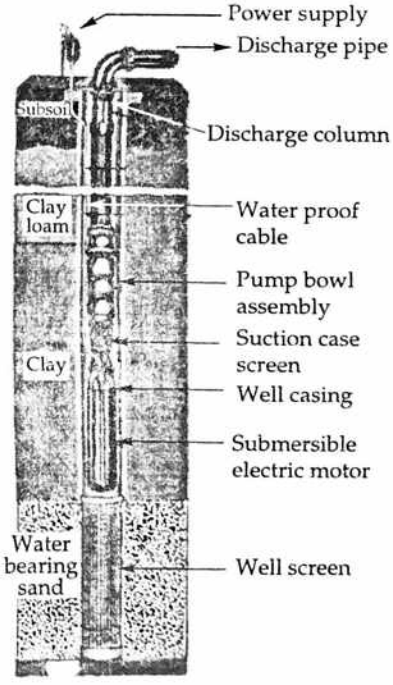


Fig. 7.6. View of a submersible pump, installed in a tubewell bore.

Another type of a deep well pump, called a ***submersible pump*** (Fig. 7.6 and 4.56), differs from the usual deep well turbine pump in the sense that the driving motor here is placed below the pump bowels within the well water level. Thus, the driving shaft is completely eliminated and the water rises t

the surface through a riser pipe on which the assembly is suspended. This elimination of the driving shaft from the ground surface to the pump, reduces the bearing friction, and provides an unobstructed pipe for the delivery of water to the surface. The well water can be prevented from coming in contact with the electrical parts or the motor bearings, by enclosing it in oil-filled case with a mercury seal where the shaft passes through at the top, although in modern days, the windings are allowed to remain surrounded by water which acts as coolant and bearing lubricant.

Priming and operation of centrifugal pumps. A centrifugal pump may have to be *primed* before it is started. The priming consists in filling the pump casing with water, so that the air trapped in the pump does not hinder its operation to reduce its efficiency. A pump installed below the water level does not normally require priming, although some air may be trapped, even in such cases, in the casings of pumps mounted on horizontal shafts. The simplest method of priming is to fill the pump with water from an outside source, while permitting the displaced air to escape through an exhaust valve. Certain pumps are provided with vacuum pumps to remove the air from the casing.

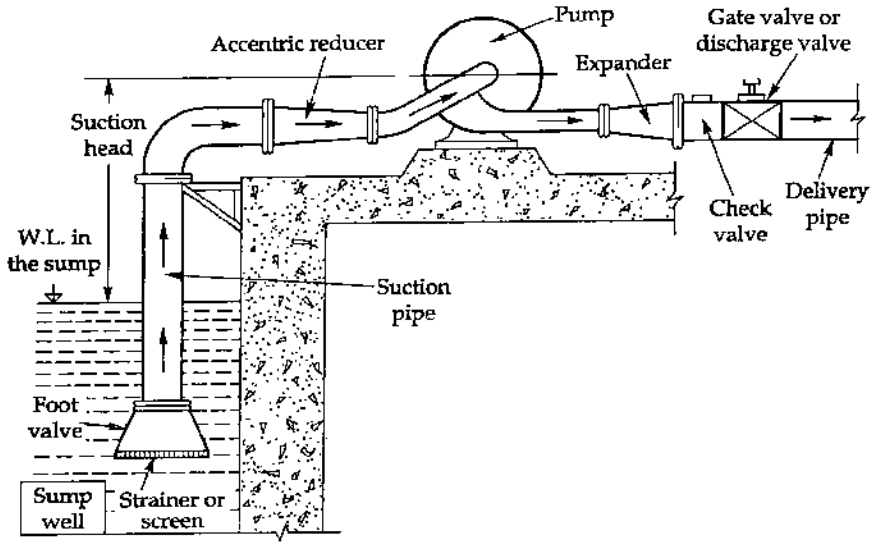


Fig. 7.7. Typical Centrifugal pump installation.

A typical centrifugal pump installation using a horizontal shaft is shown in Fig. 7.7 (Please also see Fig. 4.54). For better efficiency, the proper arrangement of the suction and the delivery pipe is necessary, and the pump should be started with full care after priming. For economy, the diameter of the pump casing at the inlet end as also at the discharge end are often kept smaller than that of the pipe to which the pump is connected. When a horizontal reducer is used at the inlet end, an *eccentric reducer* must be used, so as to prevent air accumulation (Fig. 7.7). A foot valve should also be fitted at the end of the suction pipe, so as to prevent water from leaving the pump, when it is stopped. This, therefore, avoids the necessity of outside priming every time, and serves as a self-priming device. The delivery pipe is often installed with a check valve

and a gate valve. The check valve prevents back flow through the pump if there is a power failure. The gate valve is opened slowly only after the pump has picked up its full speed and the discharge pipe is filled with water and full back pressure exists on the pump. The motor will then pick up the load gradually. If the pump is started with the discharge valve open and with the discharge pipe empty, the conditions could develop in overloading and burning of the motor.

A strainer or a screen is usually provided at the entrance of the suction pipe (Fig. 7.7) while pumping raw water, so as to prevent the entry of debris, etc. which might clog the pump.

Characteristics of centrifugal pumps. Every centrifugal pump and axial flow pump has **operating characteristics** *which depend on its design and speed of operation*. A typical characteristic curves chart is shown in Fig. 7.8.

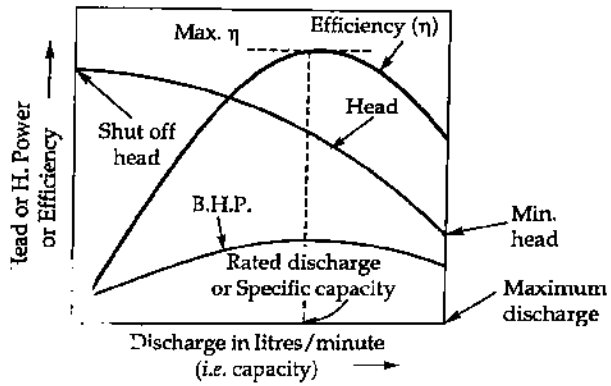


Fig. 7.8. Typical characteristic curves of a centrifugal pump for a given speed.

These are the curves obtained by plotting at constant speed, the variations of head, efficiency, and brake horse power, against the corresponding rate of discharge. These curves provide the following important conclusions and considerations which are helpful while selecting a particular pump for a particular use.

It can be observed from these curves, that for a particular speed of operation :

(*i*) as the discharge *increases*, the head produced *decreases*. The maximum discharge being limited by a certain minimum head, below which, the pump will not work. At zero discharge (*i.e.* when the discharge valve is closed), the head developed is maximum and is known as the ***shut off head***. This should not be permitted to last long, as otherwise, the pressure will abnormally rise above the design head. The efficiency of the pump is zero at this point, as there is no discharge, but the pump requires energy to drive.

(*ii*) The maximum efficiency is obtained at a particular discharge or at a particular use). This discharge at which the efficiency of the pump is maximum is known as the *normal discharge* or the ***rated discharge*** or the *rated capacity* or ***the specific capacity*** of the pump at a given speed. This fact is thus utilised in rating the pumps. It, therefore, follows that when variable discharge is required from a pump working at a particular speed, it cannot

function at its maximum efficiency and will result in reduced efficiencies. Hence, in order to obtain variable quantities of discharge at maximum efficiency, the alternative is to vary the speed with belts and pulleys or with a variable speed motor. This is neither practical nor economical. The preferred way to regulate flow, however, is to provide several pumps of smaller capacities, in parallel, so that their variable numbers may be operated at specific capacity, depending on the flow requirements. With this arrangement, it is possible to operate all the pumps near the maximum efficiency. *In water supply schemes, where water demand is variable throughout the day and year, either this method may be adopted; or sometimes all the pumps are operated at full capacity, and the excess water during the period of less demand is stored in the "service reservoir" and utilised during high demand periods (when demand rate exceeds the rate of pumping).*

(*iii*) The input power, referred to as ***brake horse power (BHP)***, is required from the motor (*i.e.* prime mover) to drive the pump. The BHP increases with the increase in discharge, reaching a peak at somewhat higher rate of discharge than that which produces the maximum efficiency. The nature of the curve varies with the speed of the pump. Hence, the optimum speed must be considered while selecting a pump to obtain higher efficiency.

Example 7.1. *A centrifugal pump running at 1450 rpm has the characteristics given below :*

Discharge, lps	*11.3*	*16.9*	*22.6*	*28.3*	*34.0*	*39.6*	*45.2*
Head, m	*25.8*	*25.0*	*24.1*	*23.2*	*21.4*	*18.9*	*15.8*
Efficiency, %	*65*	*70*	*73*	*74*	*72*	*69*	*62*

Draw the operating characteristics of the pump and determine its specific speed. (*Civil Services, 1987*)

Solution. The operating characteristic curves are plotted in Fig. 7.9.

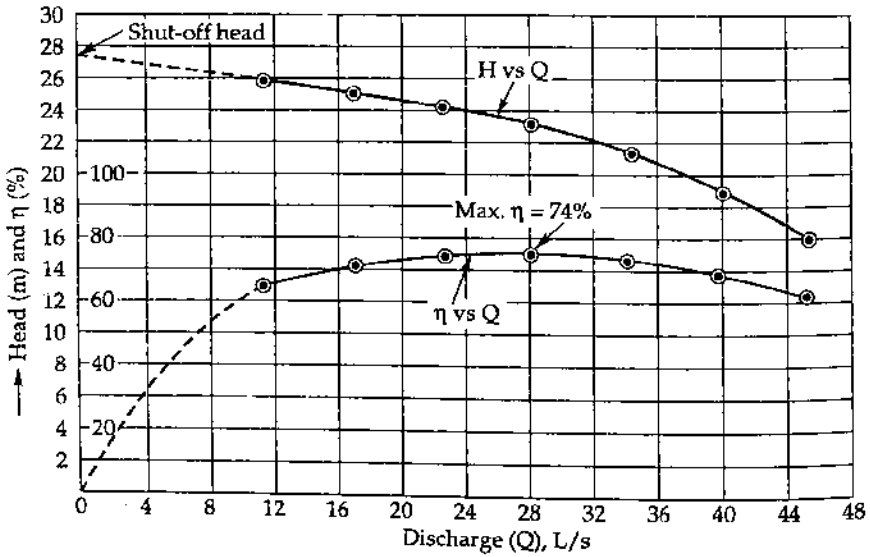


Fig. 7.9

Specific speed (N_s). Specific speed of a centrifugal pump is defined as the speed at which a pump will discharge a unit flow under a unit head at maximum efficiency. It is given by the equation

$$N_s = 51.66 \left[N . \frac{Q^{1/2}}{H^{3/4}} \right] \quad ...(7.1)$$

where Q and H are at maximum efficiency for the given speed N. Q is in cumecs and H is in metres. N and N_s may be in rpm.

By operating a pump at a particular speed N, we can determine that particular set of values of Q and H, which generate maximum efficiency at that speed. And by using those values of N, Q and H, we can determine N_s from eqn. (7.1).

In the given question, the maximum efficiency (74%) is obtained at Q = 28.3 lps, and H = 23.2 m, when the pump is run at a speed (N) of 1450 rpm.

Hence N = 1450 rpm

$$Q = \frac{28.3}{1000} \text{ m}^3/\text{sec} = 0.0283 \text{ m}^3/\text{s}$$

$$H = 23.2 \text{ m}$$

$$\therefore \quad N_s = 51.66 \left[1450 \times \frac{(0.0283)^{1/2}}{(23.2)^{3/4}} \right]$$

$$= \frac{51.66 \times 1450 \times \sqrt{0.0283}}{10.571} = \mathbf{1192 \text{ rpm.}} \quad \textbf{Ans.}$$

Note. Between sp. speeds of 1000 to 4000, *radial flow centrifugal pumps* perform well, from 4000 to 7000—*mixed flow centrifugal pumps* are most efficient; while above 7000, *axial flow pumps* have higher efficiencies. High specific speed is associated with high discharge at low heads, and hence axial flow pumps are particularly suitable to storm water pumping.

Advantages and disadvantages of centrifugal pumps. The rotodynamic pumps, especially those of centrifugal type, are widely used for pumping these days, and have almost replaced the displacement or reciprocating types of pumps (described in the next article).

The various *advantages* possessed by these pumps are :

(*i*) Their initial cost as well as the maintenance cost are comparatively low.

(*ii*) Their size is compact and can be installed in limited space.

(*iii*) Their mechanism is simple, and thus, less skilled labour is required for their operations and repairs.

(*iv*) They can be operated with high speed electric motors, or gas engines and steam turbines.

(*v*) The discharge obtained is steady and is non-pulsating.

(*vi*) They can be used for pumping water containing silt, sand, etc.

(*vii*) They are quite durable and safe against high pressures.

The *limitations* or the various drawbacks of such pumps are :

(*i*) Such pumps require priming.

(*ii*) Such a pump should neither be started with discharge valve open, so as to avoid overloading, nor should the discharge valve be kept closed for longer time after starting the pump, so as to avoid too high pressures.

(*iii*) The discharge pipe has to be provided with check valve, so as to avoid back flow when the pump suddenly stops due to power failure, etc. with the discharge valve open.

(*iv*) The discharge from such a pump varies with the head of water. Hence, when uniform discharge is required at varying heads the pump has to be rotated at variable speeds which is neither, practical nor economical.

(*v*) For high heads, the efficiency is low (say about 50 to 80 per cent).

(*vi*) Their ordinary suction lift is limited (say upto about 6 m or so) and the pump has to be placed under water where higher suction lifts are involved, such as in turbine pumps or in submergible pumps ; thereby, necessitating special types of centrifugal pumps with their increased costs.

7.2.2. Displacement pumps. The displacement pumps or more strictly speaking *positive displacement pumps*, work on the principle of mechanically inducing vacuum in a chamber, and thus, sucking in a certain volume of water, which is then mechanically displaced and forced out of the chamber. The two types of displacement pumps are :

1. Reciprocating pumps ; and
2. Rotary pumps.

They are described below :

7.2.2.1. Reciprocating pumps. The simplest type of a reciprocating pump, which is still widely used in Indian villages and towns and sometimes in cities too, is a *hand operated well pump* (commonly called ***hand pump***). In such a pump (Fig. 7.10), a piston or a plunger reciprocates in a closed vertical cylinder. This is moved up and down by hand. On the *upstroke*, a vacuum gets created in the cylinder, below the piston, thereby opening the check valve V_1 at the base of the pump, thus withdrawing water into the space below the piston. Simultaneously, the water above the piston is forced out of the spout. On the down stroke, the check valve

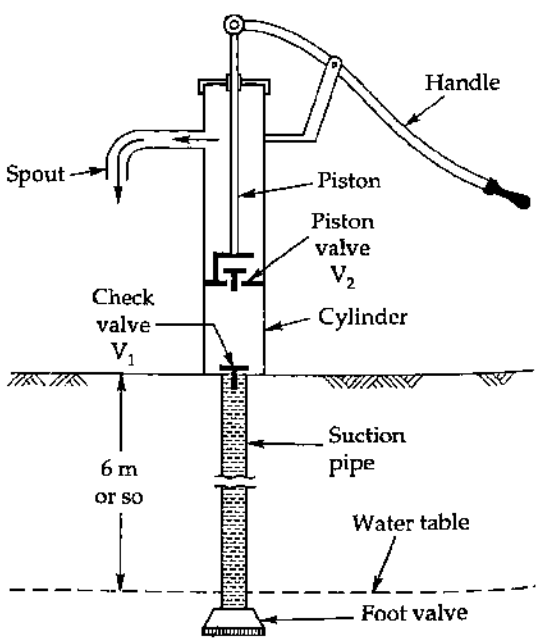


Fig. 7.10. Hand operated reciprocating pump.

V_1 gets closed and the piston valve V_2 gets opened, thus permitting water to enter the cylinder above the piston. On the next upstroke, the piston valve V_2 closes, and the water above the piston comes out of the spout, and at the same time, the check valve V_1 opens, permitting water to enter the space above the piston. The cycle is thus continued, and continuous supplies of water can be obtained.

These pumps can be used in any depth. For wells, where watertable is within 6 m or so, the cylinder is placed above the ground, as shown in Fig. 7.10. The pump, however, requires a foot valve at the lower end of the suction pipe, so as to avoid priming. Where the watertable (or the static water level in the dug well, from which the water is lifted by such a hand pump) is more than 6 m below the ground surface, the cylinder is attached to a drop pipe and placed in the well, as shown in Fig. 7.11.

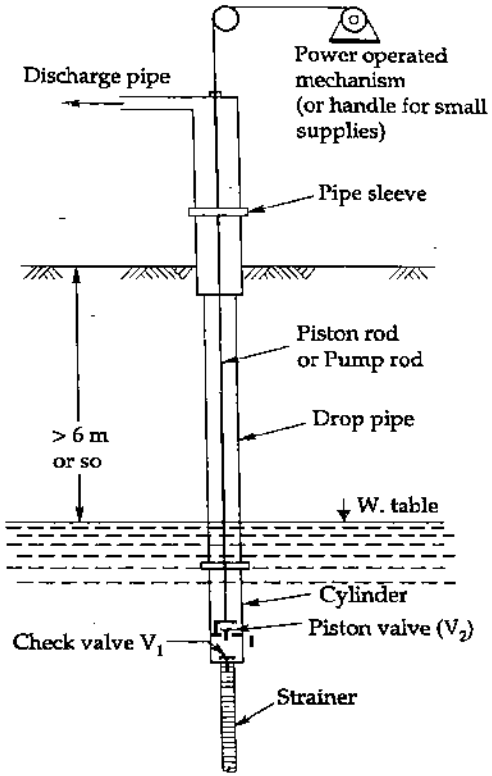


Fig. 7.11. Deep well reciprocating pump.

A simple hand pump type of a reciprocating pump described above, is a *single acting pump*, in which the discharge occurs only on alternate piston strokes. However, *double acting pumps*, which deliver water with each stroke, are also sometimes used, and can provide more uniform flow.

Advantages and disadvantages of reciprocating pumps. Large reciprocating pumps operated by power were widely used for pumping water during the nineteenth century even in the developed countries like U.S.A. However, now a days, except that of small hand pumps, they are almost outdated, and rotodynamic pumps especially of centrifugal type are replacing them. Their *advantages* are, however, given below :

(*i*) Since a definite volume of water is discharged with each stroke, the discharge rate of such a pump depends only on the speed with which the piston is moved and is independent of the head. It, therefore, gives a constant discharge under variable heads. Such pumps are very useful for *very high and variable heads with low discharges*.

(*ii*) They are durable and flexible.

(*iii*) High efficiency is possible, provided the valves and the packings are in good condition. In fact, the efficiency depends upon the ***slippage*** (*i.e.* the amount of water leaking between the piston and the cylinder walls during the stroke). Hence, if the walls and packings are in good condition, slippage may be less than 10 percent of the total output.

The *disadvantages* of the reciprocating pumps are :

(*i*) the initial cost of installing such a pump is as high as four times that of a centrifugal pump.

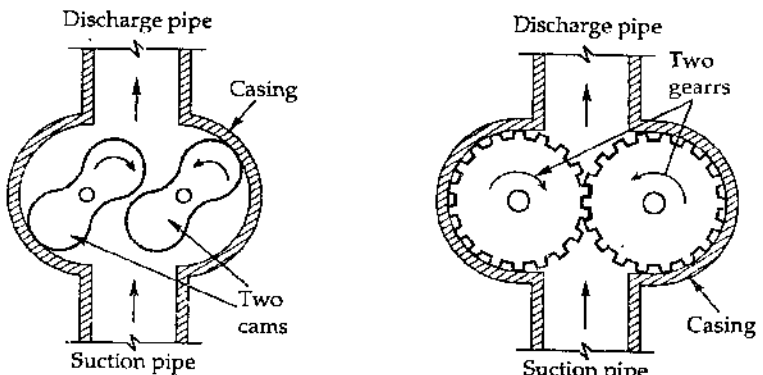
(*ii*) Reciprocating pumps occupy large space (6 to 8 times the space required by the horizontal centrifugal pumps) and needs skilled supervision during operations.

(*iii*) They are unsuitable for pumping waters containing sediments, as in such a case, there may be a sudden stoppage of the pump leading to serious damage to the pump.

(*iv*) The single acting reciprocating pumps produce pulsating flow (*i.e.* the flow with jerks).

(*v*) Maintenance charges are high, because parts like valves require constant attention.

7.2.2.2. Rotary pumps. In rotary type of displacement pumps, the rotary motion is used in place of a reciprocating motion as is used in reciprocating type of displacement pumps. The rotary motion is obtained by using two cams [Fig. 7.12 (*a*)] or two gears [Fig. 7.12 (*b*)] which mesh together and rotate in opposite directions. The rotating elements fit the casing closely. The water enters through the suction pipe and is trapped between the cams (or the teeth of gears) and the casing. It is then forced out though the *discharge pipe*, as the cams or the gears rotate. A definite quantity of water depending on the size and the shape of the gears (or the cams) is thus raised with each revolution.



(*a*) Rotary pump with cams (*b*) Rotary pump with gears

Fig. 7.12

Advantages of rotary pumps

(*i*) Their biggest advantage is the fact that they do not require any priming and are self-primed. They are, therefore, often used to prime large centrifugal or reciprocating pumps.

(*ii*) The flow from such a pump is nearly free from pulsations.

(*iii*) Since they have no valves, they are simple to construct and easy to repair as compared to the reciprocating pumps.

(*iv*) They are often used for fire protection systems for buildings and for small domestic water systems.

(*v*) Their efficiency is high at low to moderate heads and for smaller discharges of the order of 2000 litres per minute, although they have been used for pressures up to 70 kg/cm^2 (700 m head of water) and discharges over 1,35,000 litres per minute.

Disadvantages of rotary pumps

(*i*) The initial cost is high.

(*ii*) They are not durable, as they require frequent replacement of their cams or gears.

(*iii*) Water containing sediments are very injurious to such pumps, as the wear and abrasion will destroy the seal between the cams and the casing.

7.2.3. Miscellaneous Types of Pumps. As pointed out earlier, various other types of pumps, such as airlift pump, jet pump, the hydraulic ram, etc. are also sometimes used, and are briefly described below :

7.2.3.1. Air lift pumps. Air lift pumps are generally used for pumping water from deep wells for lifts of about 60—80 metres. A typical arrangement of an air lift pump is shown in Fig. 7.13. In such a pump, compressed air is forced into the well through a small air pipe and is released through a diffuser into the *eduction pipe* at the bottom of the well, as shown. The air rises in small bubbles in the eduction pipe, thus forming a mixture of air and water. This resulting mixture of air and water inside the eduction pipe is lighter than the water outside the pipe, and hence is forced upward by hydrostatic pressure ; finally coming out through the outlet at the top. A separator is often placed at the outlet, so as to remove air, from water. The reclaimed air is generally cooler than the atmospheric air and can be compressed more cheaply for recirculation.

Air lift pumps can be used for high lifts upto about 150 metres but their efficiency is generally low (lying between 25 to 50 percent or so).

The effectiveness of an air lift pump is generally measured by a factor called ***percentage submergence***, which is given as :

$$\text{Percentage submergence} = \frac{\text{Depth of submergence } (D_s) \times 100}{\text{Depth of submergence } (D_s) + \text{Effective life of pump } (H_e)}$$

$$= \frac{D_s \times 100}{D_s + H_e} \qquad ...(7.2)$$

(**Note.** $D_s + H_e$ represents the effective length of eduction pipe).

The air lift pump will operate at best efficiency if the value of percentage submergence is about 70%. However, its value should be atleast 25% in order to operate the pump even at low efficiency.

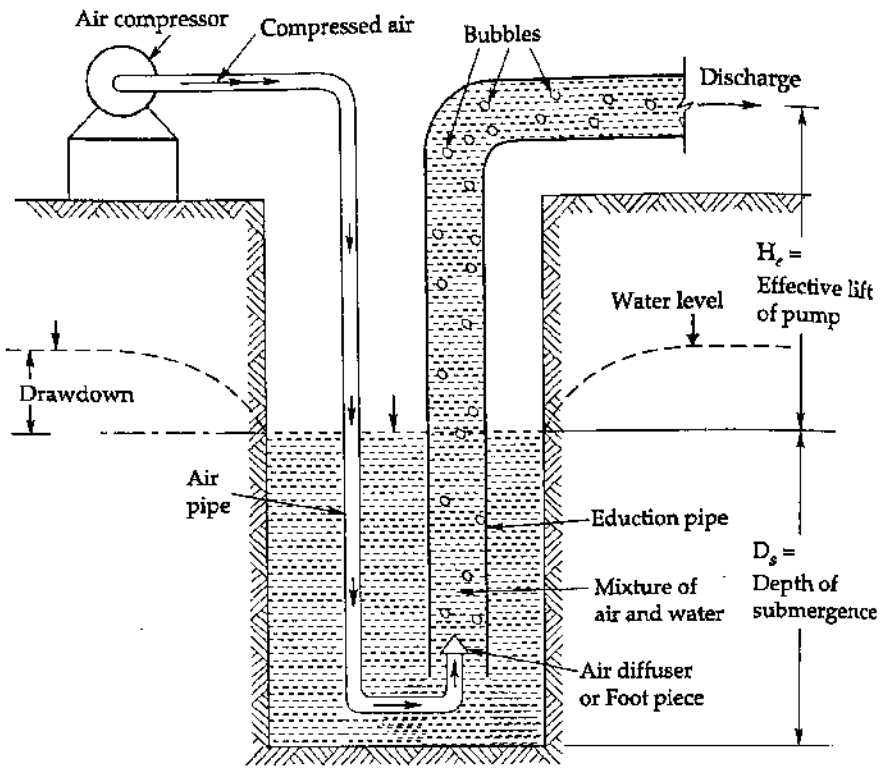


Fig. 7.13. Air lift pump.

Advantages of air pumps

(*i*) Inspite of low efficiency, an air lift pump can deliver large amounts of water from small diameter wells.

(*ii*) The air lift is not harmed by sedimented water and, therefore, such pumps are particularly suitable for use in crooked or damaged wells.

(*iii*) Air lift pumps are generally cheaper, reliable and simple in operation.

(*iv*) There are no moving parts of the pump, which may be in contact with the water. Hence, such pumps can be easily used for highly acidic or alkaline waters.

(*v*) The pumping from a number of wells can be done by installing a common compressor unit.

(*vi*) The yield of a well, using such a pump, can be increased by using more amount of compressed air.

Disadvantages of air lift pumps

(*i*) In order to obtain sufficient value of "percentage submergence", the depth of submergence has to be increased, and thus the well has sometimes to be made deeper than required. This increases the cost.

(*ii*) An air lift pump is generally not adopted for raising water much above the ground level, and if this is necessary, a second pump has to be installed.

(*iii*) Their efficiency is generally low.

(*iv*) The flow obtained is not continuous but is intermittent. This is due to the fact that small air bubbles have a tendency to unite, and large bubbles thus formed, rise more rapidly than water.

(*v*) Air lift pumps are less flexible in fulfilling variable demands.

7.2.3.2. Jet pumps. Jet pumps are often used for pumping water from small wells. They are portable and sometimes used in construction works for dewatering trenches, etc. A typical arrangement of a jet pump is shown in Fig. 7.14. In this pump, compressed air or steam or water is made to enter the pipe *A*, which is nozzled at its discharging end, thus forcing out through the nozzle. The jet of compressed air (or water or steam) thus comes out of the nozzle at a high velocity, and discharges into the throat of another pipe (*B*) as shown. This high velocity jet creates a suction which draws the water up the pipe *B* and finally discharges through the discharge end.

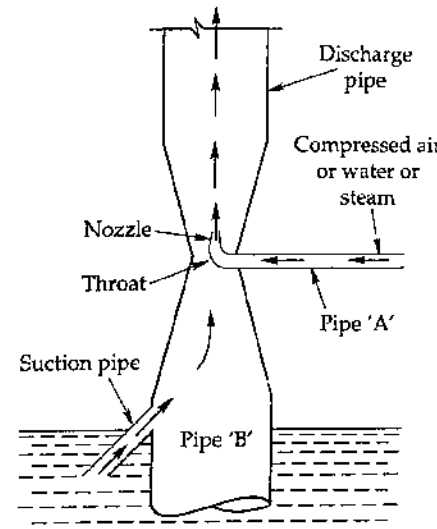


Fig. 7.14. Jet pump.

The efficiency of such a pump is generally low, of the order of 25 per cent or so. However, they are compact and light in weight, thus making them portable and easy to handle. They can also handle sedimented waters without much troubles.

7.2.3.3. The hydraulic ram. A hydraulic ram is a kind of a pumping arrangement which does not utilise any outside power and uses the principle of *water hammer Pressures** developed when a moving mass of water is

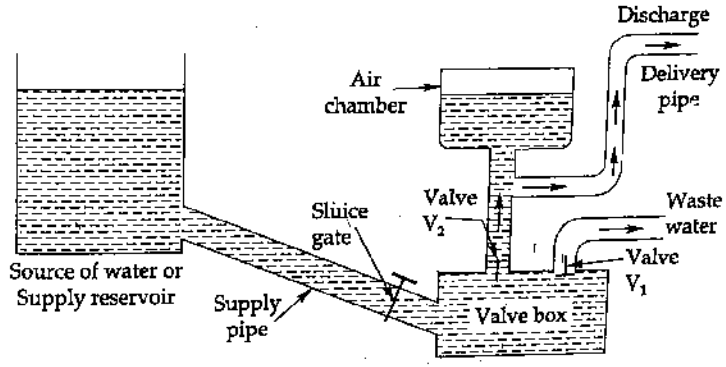


Fig. 7.15. Layout of a hydraulic ram.

*Explained in details in the previous chapter.

suddenly stopped. In a hydraulic ram, a relatively large amount of water must be available at moderate head, so as to lift a small amount of water to higher head. A simple schematic layout for a hydraulic ram is shown in Fig. 7.15.

It consists of a supply pipe bringing water from the source to the *valve box* of the ram (Fig. 7.15). The valve box contains two *automatic* valves, V_1 and V_2. V_1 is the *waste valve* opening *downward*, and V_2 is the delivery valve opening upward. Above the delivery valve V_2, there is an air chamber, to the base of which, a delivery pipe is attached.

To start with, the waste valve remains open due to its self weight. When water flows from the source to the valve box, it goes out of the waste valve. As it happens, the water attains more and more velocity, and thus finally the net force exerted upward on the valve becomes high enough to easily overcome the weight of the valve V_1 and, therefore, forcing it upward and closing it automatically. As soon as this valve V_1 closes, a wave of positive water hammer pressure is developed. The high positive water hammer pressure so developed will open the delivery valve V_2 (upward) and thus allowing some boosted water to pass up the delivery pipe. When the wave of negative or suction pressure returns from the supply reservoir, the delivery valve automatically closes (due to removal of pressure) and the waste valve opens (due to self-weight). The flow again starts from the waste valve which accelerates gradually and thus closing the waste valve suddenly and automatically, so as to begin a new cycle. There may be 50 to 200 cycles per minute.

Depending upon the design of the ram, the air chamber helps in reducing the fluctuations of flow through the pipe. The hydraulic ram is quite suitable for small water supply projects, and is installed at lower levels than the source. It can be used for lifts of the order of 30 m or so.

Advantages of hydraulic ram

(*i*) Its working is simple and when once it starts functioning, practically no attention is required.

(*ii*) The ram is durable.

(*iii*) It is cheap as it does not require any fuel. It can, therefore, also be used advantageously, in situations where no outside source of power is available.

Disadvantages of hydraulic ram

(*i*) Considerable amount of water is wasted through the waste valve, and hence cannot be installed at places where water is scarce. The ratio of the wasted water to the pumped water may vary from 6 : 1 to 2 : 1, depending upon the supply head, the lift and other factors.

(*ii*) It produces considerable noise while working.

7.3. Factors Affecting the Selection of a Particular Type of Pump

The various factors which must be thoroughly considered while selecting a particular type of a pump for a particular project are :

(*i*) capacity of pumps

(*ii*) importance of water supply scheme

(*iii*) initial cost of pumping arrangement

(*iv*) maintenance cost

(*v*) space requirements for locating the pump

(*vi*) number of units required

(*vii*) total lift of water required

(*viii*) quantity of water to be pumped.

Truly speaking, reciprocating pumps are also outdated these days, and for all the ordinary conditions of pumping, the centrifugal types of rotodynamic pumps are frequently used these days, as they provide satisfactory and economic service. However, pumps other than those of centrifugal types may be used under extreme conditions. The choice between the various types of pumps is guided by the following considerations :

For very small discharges, the rotary pumps may prove to be equally satisfactory as the centrifugal pumps, and less costly if the water to be pumped is free of sediment.

A centrifugal pump may pose operational problems, when constant discharge is needed at variable heads (because it requires a variable speed driving in that case) whereas, the discharge through a reciprocating pump depends only on the speed of the pump. Hence, under such circumstances, where water is to be pumped against *very high* but *variable heads* with a higher suction lift, reciprocating pumps may be useful. However, they can be used only when the water to be pumped is free of sediment, and ample finances are available for installing the costly reciprocating pumps.

Centrifugal pumps are especially useful for pumping waste water and water containing solids, although they are equally suitable for pumping treated waters. Even among the centrifugal pumps, a choice is sometimes made out of the horizontal shaft centrifugal pump and the vertical spindle turbine pump and the submergible pump. The horizontal pump is the cheapest and is used under a wide range of pumping conditions, but vertical spindle and submergible pumps may be preferred for handling large quantities of water under low heads, and for wells and tubewells involving higher suction lifts.

Air lift pumps may prove to be cheaper and, therefore, selected when water is required to be pumped simultaneously from a number of wells, because in that case, a common compressor unit can be used to feed all the pumps. However, their efficiency is generally low.

The hydraulic ram and jet pumps may also be used under special circumstances, as pointed out earlier.

7.4. Head, Power and Efficiency of Pumps*

The total head (H) against which a pump has to work consists of

(*i*) the suction lift (H_s) ;

(*ii*) the delivery head (H_d) ; and

(*iii*) the head lost due to friction, entrance and exit in the rising main (H_L).

$$\therefore \qquad H = H_s + H_d + H_L \qquad ...(7.3)$$

*Please see chapter 4 also for horse power of pumps required for pumping ground waters.

The work done by the pump in lifting Q cumecs of water by a head H

$$= \gamma_w . Q.H. \text{ kN.m/s (i.e. J/s or Watts)} \quad ...(7.4)$$

where γ_w = unit wt. of water in kN/m^3 ; *i.e.* 9.81 kN/m^3

Q = discharge to be pumped in m^3/ sec.

The **water horse power** of the pump is then given as :

$$\text{W.H.P.} = \frac{\gamma_w . Q.H}{0.735} \qquad (\because \quad 1 \text{ HP} = 735 \text{ W} = 0.735 \text{ kW}) \quad ...(7.5)$$

If η is the efficiency of the pump set, then the **brake horse power** of the pump (B.H.P.) is given by

$$\text{B.H.P.} = \frac{\text{W.H.P.}}{\eta} = \frac{\gamma_w . Q.H.}{\eta \times 0.735} \quad ...(7.6)$$

7.5. Economical Diameter of the Pumping Mains

As pointed out in the previous chapter, the diameter of a pipe can be reduced (for passing a certain fixed discharge) by increasing the flow velocity through the pipe. But, however, the increased velocity will lead to higher frictional head loss, and thus increased cost of pumping. Hence, although the dia and the cost of pipe can be reduced by choosing a higher flow velocity, the horse power of the pump required will increase, thus increasing the cost of pumping. For optimum conditions, we must choose such a diameter, which together with the pumping cost, will make the total annual expenses, the *minimum*. The diameter which provides such optimum conditions is known as the **economical diameter** of the pipe.

Hence, if the diameter chosen is less than the economical diameter, the cost of pipe will be less, but the head loss will be high and the cost of pumping shall be much more than the resultant saving in the pipe cost. Similarly, if the diameter chosen is more than the economical dia, the cost of pumping will be less but the increase in the cost of pipe will be much more than the resultant saving obtained in the cost of pumping.

An empirical formula given by Lea, connecting the dia and the discharge, which is commonly used in practice is given as :

$$D = 0.97 \text{ to } 1.22\sqrt{Q} \quad ...(7.7)$$

where D = economical dia in metres

Q = discharge to be pumped in cumecs

This relationship gives optimum flow velocity varying between 1.35 to 0.8 m/sec.

For a rigorous analysis, the total cost of pipe and pumping should be worked out at different assumed flow velocities (between 0.8 to 1.8 m/sec) and a graph plotted between the yearly cost and the size of the pipe (Fig. 7.16).

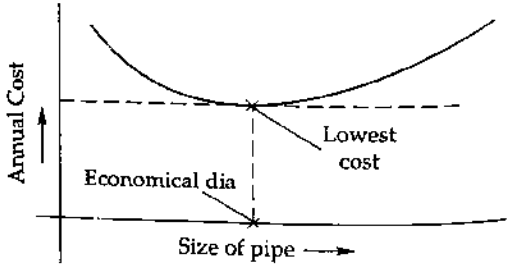


Fig. 7.16

The economical size is the one which gives the least cost, and thus selected by inspection, as shown in Fig. 7.16.

Example 7.2. *A centrifugal pump driven by an electric motor lifts water through a total height of 50 metres from the reservoir to the discharge end. The pump efficiency is 77 per cent and the motor efficiency is 85 per cent. The lift is through 300 metres length of 10 cm diameter pipe and the pumping rate is 1500 litres per minute. If 4f = 0.025, and power costs ₹ 2.25 p per kilowatt hour, what is the cost of power for pumping four million litres of water ?*

Solution. $H_s + H_d$ = Total lift including suction and delivery

= 50 metres

Head loss in pipe friction

$$= H_f = \frac{f' LV^2}{2gd} = \frac{f' . L . Q^2}{2 . g\left(\frac{\pi}{4}\right)^2 d^5}$$

where $f' = 4f = 0.025$

$L = 300$ m

$d = 0.1$ m

$Q = 1500$ litres/minutes $= \frac{1.5}{60}$ cumecs

$= 0.025$ cumecs

$$\therefore \quad H_f = \frac{0.025 \times 300 \times (0.025)^2}{2 \times 9.81 \times (0.785)^2 (0.1)^5} = 38.77 \text{ metres.}$$

Total head against which pump has to work

$= 50 + 38.77 = 88.77$ m.

Water power required

$= \gamma_w . Q.H = (9.81 \times 0.025 \times 88.7)$ kN.m/s or kW

$= 21.77$ kW

$= \frac{21.77}{0.735}$ HP $= 29.62$ HP

Horse power of motor required

$$= \frac{29.62}{\eta_p \eta_m} = \frac{29.62}{0.77 \times 0.85} = \mathbf{45.26 \text{ H.P.}}$$

Power required = 45.26 × 0.735 kW = **33.26 K. watts.**

Time during which 4 million litres of water will be lifted

$$= \frac{\text{Vol. of water to be pumped}}{\text{Pumping rate}}$$

$$= \frac{4 \times 10^6}{1500} \text{ minutes} = \frac{4 \times 10^6}{1500 \times 60} \text{hr} = 44.4 \text{ hr.}$$

Energy required = (33.26 × 44.4) kilo watt hr. = 1476.9 K. watt hr.

Cost of the electrical energy @ ₹ 2.25 per unit

= ₹ 1476.9 × 2.25 = ₹ 3323 **Ans.**

Example 7.3. *Water has to be supplied to a town with one lakh population at the rate of 150 litres per capita per day from a river 2000 m away. The difference in elevation between the lowest water level in the sump and the reservoir is 36 m. If the demand has to be supplied in 8 hours, determine the size of the main and the brake horse power of the pumps required. Assume maximum demand as 1.5 times the average demand. Assume $f = 0.0075$, velocity in the pipe 2.4 m/sec, and efficiency of pump 80 per cent.*

(Engg. Services, 1969)

Solution. Population = 1,00,000

Per capita demand = 150 litres/day/person

∴ Average daily demand

= 1,00,000 × 150 litres/day

= 15 million litres/day = 15 M.l.d.

Maximum daily demand

= 1.5 times the average daily demand

= 1.5 × 15 = 22.5 Mld.

Since the pumps have to work for 8 hours a day, the capacity of pumps should be designed for a daily demand

$$= 22.5 \times \frac{24}{8} = \mathbf{67.5\ Mld.}$$

Discharge required per second

$$= \frac{67.5 \times 10^6}{10^3 \times 24 \times 60 \times 60}\ \text{m}^3/\text{sec} = \mathbf{0.781\ cumec.}$$

Now, Velocity through the pipe = 2.4 m/sec (given)

∴ Area required $= \frac{Q}{V} = \frac{0.781}{2.4} = 0.325\ \text{m}^2$

∴ Diameter of main required

$$= \sqrt{0.325 \times \frac{4}{\pi}}$$

= 0.643 m ; say **0.65 m. Ans.**

Total lift including suction and delivery = 36 m (given)

Head loss due to friction in pipe = H_L or H_f

$$= \frac{f' LV^2}{2gd}$$

where $f' = 4f = 4 \times 0.0075 = 0.03$

$L = 2000$ m

$V = 2.4$ m/sec

$d = 0.65$ m

$$H_L = \frac{0.03 \times 2000 \times (2.4)^2}{2 \times 9.81 \times 0.65} = 27.1\ \text{m.}$$

Total lift against which pump has to work

= 36 + 27.1 = **63.1 m.**

Brake horse power of pump

$$= \frac{\gamma_w . QH}{\eta \times 0.735}$$

$$= \frac{9.81 \times 0.781 \times 63.1}{0.8 \times 0.735} = 822\ \text{H.P.}\quad \textbf{Ans.}$$

Example 7.4. *From a clear water reservoir 3 m deep and maximum water level at 30.00, water is to be pumped to an elevated reservoir at 75.00 at the constant rate of 9,00,000 litres per hour. The distance is 1500 m. Give the economical diameter of the rising main and the water horse power of the pump. Neglect minor losses and take $f = 0.01$.* *(Engg. Services, 1975)*

Solution. The discharge

Q = 9,00,000 litres per hour

$$= \frac{9,00,000}{1000 \times 60 \times 60}\ \text{m}^3/\text{sec.} = \mathbf{0.25\ cumec.}$$

For *economical diameter of the rising main*, using **Lea's formula**, we have

$$D = 1.22\sqrt{Q} \qquad \text{(Refer Eq. 7.7)}$$

$$= 1.22\sqrt{0.25} = 1.22 \times 0.05 = \mathbf{0.61\ m.}$$

Hence, the economical diameter of the rising main = 0.61 m. **Ans.**

Head including maximum suction and delivery head, against which the pump has to work

= 3 + (75.00 − 30.00) = 48 m

The head loss in the pipe

$$H_f = \frac{f' LV^2}{2gd}$$

where $f' = 4f = 4 \times 0.01 = 0.04$

$L = 1500$ m.

V = Velocity of flow $= \frac{Q}{A}$

$$= \frac{0.25}{\frac{\pi}{4}(0.61)^2} = \frac{0.25}{0.291} = 0.855\ \text{m/sec}$$

$d = 0.61$ m

Substituting the values, we get

$$H_f = \frac{0.04 \times 1500 \times (0.855)^2}{2 \times 9.81 \times 0.61} = 3.66\ \text{m.}$$

Neglecting entrance and exist losses as well as the losses in bends, we have

The total losses = $H_L = H_f = 3.66$ m

The total head $H = 48 + 3.66$ m = 51.66 m

Water horse power of pump

$$= \frac{\gamma_w . Q.H}{0.735} = \frac{9.81 \times 0.25 \times 51.66}{0.735} = 172.2\ \text{H.P.}\quad \textbf{Ans.}$$

7.6. Pumping Stations

The location of a pumping station is primarily governed by the location of the place from where it is to receive water, and also by the location of the place where it is to supply that water. The various points which are to be kept in mind while selecting a suitable site are enumerated below :

(*i*) The site should be away from all the sources of contamination or pollution.

(*ii*) The site should be above the highest flood level of the river.

(*iii*) It should be so selected that its future growth and expansion is easily possible.

(*iv*) In case of rivers, the site for the pumping stations should be such that the pumping machinery is able to draw sufficient water so as to meet the peak demand during the busiest hour of the day. In order to achieve this, the dry weather flow in the river, the likely changes in the river course, etc. must be properly studied and considered.

(*v*) Possibilities of fire hazard should also be considered while selecting the site for the pumping stations.

(*vi*) The proximity of the site to the railways, from where the coal can be quickly made available for producing power, may also have to be considered.

The pumping stations should be made attractive in their exterior and interior, which may arouse public faith in the neatness and cleanliness of the water supplied to them. Hence, they should be properly lighted, ventilated and kept neat and clean. Different pumping units and driving engines should be spaced wide apart so as to provide sufficient space for their operation, repairs, maintenance and cleaning (Refer Fig. 7.17).

7.6.1. Setting of Pumps. The pumps can be *placed* either *below* the level of water in the sump well (suction pit), or *above* it. In the former case, there will be no *priming* difficulty. In such a case, however, the pumps should preferably be placed in an adjoining dry pit, and connected to a common *suction pipe* through *gate valves*. (Refer Fig. 7.17). The pumps can then be easily repaired by closing down the in-between gate valve. This type of setting will give a positive suction head without any priming difficulty, and hence is more useful in case of centrifugal pumps which are generally not self-priming. The driving machines (usually electric motors) should of course be placed at a higher elevation, which is dry and safe. They will then be connected to the pumps by means of vertical shafting. (Refer Fig. 7.17, the motors being at top, not visible in bottom plan).

When the pumps are placed *above* the level of the water in the sump well, they can be placed at an elevation which has to be decided by the possible *suction lift* of the pump, which depends upon the atmospheric pressure, the temperature of water, the velocity in the suction pipe, the frictional losses, etc., and is usually limited to 5 metres or so. To keep the losses to minimum, in such a case therefore, suction lines should be made as short as possible and with a continuous upward slope from the foot valve to the pump, in order to avoid air pockets. Such placing must be provided with foot valves at the ends of the suction pipes, which should remain sufficiently submerged in water, so

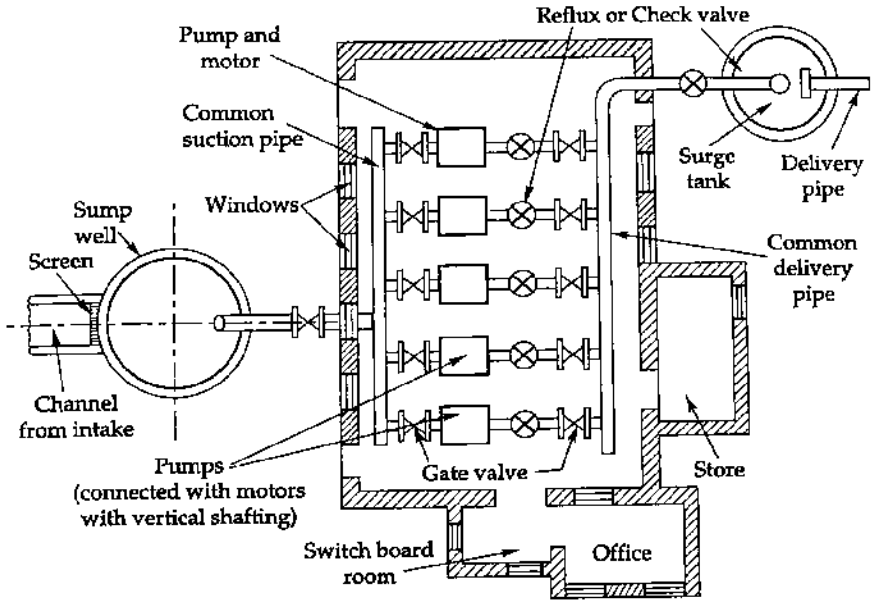


Fig. 7.17. Typical Internal Layout Plan of a Pumping Station (Bottom Plan).

as to prevent entry of air into the suction pipe, and thus to avoid priming difficulties. The driving engines, in such a setting, can be placed at the same elevation as that of the pumps, and thus horizontally coupled to them, so as to avoid difficulties in alignment of vertical shafting.

7.6.2. Piping and Valves at the Pumping Station. A typical layout plan of a raw water pumping station is shown in Fig. 7.17. Cast iron pipes using flanged joints have been provided with few bends, etc., on them. The velocity of water flowing through, generally varies between 0.6 to 1.2 m/sec at average flows. The various valves which are used here, with their common positions are enumerated below :

(*i*) *Sluice or Gate valves*—one on suction side, and the other on the delivery side ;

(*ii*) *Check valves* or *Reflux valves* on the delivery side in between the pump and the gate valve, and

(*iii*) *Pressure relief* and *Air valves* on the delivery side (not shown).

7.6.3. Sizing of the Pump Units and the Standby Capacity. A waterworks pumping station is generally not operated at full capacity, all of the times. Since the efficiency of a pumping unit varies with the load, it is a usual practice to so design such a station, that some of the pump units can be operated at full capacity, all of the time. Hence, two, three, or four pumps may have to be installed. Local conditions, such as the demand, the available storage, and the desirability of the off-peak pumping will fix the sizes of the various pumps. Say for example, if there are four pumps, the two largest may supply the maximum demand ; one of these and the smallest may handle the normal demand, and the other pump may care for the minimum requirement.

Thus, there will always exist some standby capacity to take care of the repairs, breakdowns, etc. Generally, 100% standby capacity against average demand, and $33\frac{1}{2}$% to 50% standby capacity against the peak demand, is considered sufficient and may, therefore, be provided at the pumping stations.

Example 7.5. *Design a raw water pumping station for obtaining water from a perennial river with the following data :*

(a) Population to be served = *5 lakh.*
(b) Daily demand for water = *150 litres per person per day. (average)*
(c) Summer requirements = *150% of average.*
(d) Normal water level in river = *R.L. 202.00 m.*
(e) Low water level in river (during summer) = *R.L. 199.50 m.*
(f) The water is to be pumped in two shifts of 10 hrs each daily, and lifted to the inlet of water treatment plant situated nearby at R.L. 217.00 m.
(g) The losses due to friction in the C.I. rising main of 0.9 m dia. are low due to its short length, and can be assumed to be 2 metres in summer season, and 1.2 metres on an average.
(h) The efficiency of the pumps and the driving motors can be taken as 75% and 90% respectively.

Solution. Average daily water demand
$= (5 \times 10^5)\ (150)$ litres/day.
= 75 million litres/day.
$= \dfrac{75{,}000}{24 \times 60 \times 60}$ cumecs. = 0.868 cumecs.

Maximum daily demand in summer
= 1.5 times the average demand
= 1.5 × 0.868 = 1.3 cumecs.

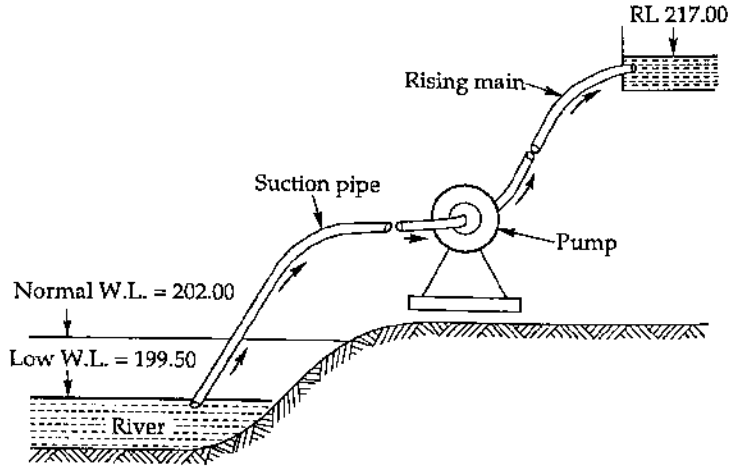


Fig. 7.18. Design flow diagram of a raw water pumping station for example 7.5.

Since the pumps have to work for 20 hrs a day, **the capacity of the pumps** should be designed for the following daily demand :

For a summer day $= 1.3 \times \dfrac{24}{20}$ ...(*i*)
= 1.56 cumecs.

For an average day $= 0.868 \times \dfrac{24}{20}$
= 1.04 cumecs. ...(*ii*)

Height through which water is to be lifted. (Refer Fig. 7.18) is given as :
For an average day = 217.00 – 202.00
= 15.00 m ...(*iii*)
For a summer day = 21700 – 199.50
= 17.50 m. ...(*iv*)

∴ **Total lift against which water is to be lifted** (including losses) is given as :
For an *average day* = 15.00 + 1.2 = 16.2 m. ...(*v*)
For a *summer day* = 17.5 + 2.0 = 19.5 m. ...(*vi*)

∴ *Water horse power required* is given as :

For *an average day* $= \dfrac{\gamma_w . Q . H}{0.735}$
$= \dfrac{9.81 \times 1.04 \times 16.2}{0.735} = 224.63$ H.P.

For a summer day $= \dfrac{9.81 \times 1.56 \times 19.5}{0.735} = 405.60$ H.P.

∴ *Brake horse power* required is given as :

For an average day $= \dfrac{405.60}{0.75 \times 0.90} = 332.8$ H.P. Say **333 H.P.**

For a summer day $= \dfrac{405.60}{0.75 \times 0.90} = 600.9$ H.P. Say **601 H.P.**

(*i*) Hence provide 3 centrifugal pumps, each of 0.52 cumecs capacity, driven by say 172 BHP motors ; and

(*ii*) 3 centrifugal pumps each of 0.26 cumecs capacity, driven by say 86 BHP motors.

Hence, during average days, 2 units of 172 BHP each, with a total of 344 HP will work giving a standby capacity of (172 + 3 × 86 = 430 BHP), which is more than 100%; and with increasing demand, the third 172 BHP unit and one 86 BHP unit can be switched on, with a total of 3 × 172 + 86 = 602 HP, leaving about 25% Standby capacity at maximum demand.

Example 7.6. *Water is pumped from a low level reservoir to a high level reservoir through a main pipeline of 0.45 m dia and 1400 m length. The pump is located at the low level reservoir. At a point along the main line at a distance of 450 m from the high level reservoir, a branch line of 0.3 m dia and 360 m length takes off to discharge 180 l/sec in the atmosphere.*

Level of water surface in high level reservoir = + 30.00 *m*
Level of water in the open end of the 0.3 m dia branch line = 25.50 *m*
Level of water in the low level reservoir = 18.00 *m*
Darcy's frictional coeff. for both pipes = 0.032

Determine the flow rate into the high level reservoir and the theoretical H.P. of the pump, assuming the delivery valve of the pump to be at + 20.0 m.

(*Engg. Services, 1988*)

Solution. The position of service reservoir, pipe line, etc. is roughly sketched in Fig. 7.19.

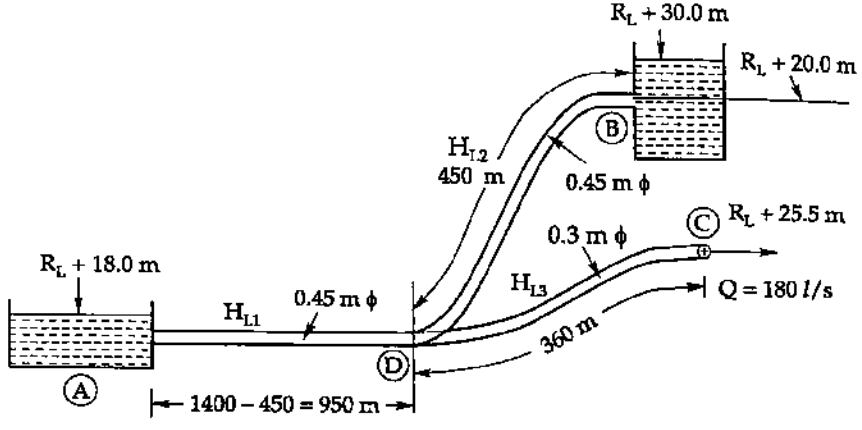


Fig. 7.19

By applying Bernoulli's theorem between D and B, and then between D and C, we can say that pressure at D, *i.e.* P_D would be given by

$$Z_D + P_D = Z_B + P_B + H_{L_2}$$

$$= 20.0 + \left[\frac{p_a}{w} + 10.0\right] + H_{L_2} \qquad ...(i)$$

Also $$Z_D + P_D = Z_C + P_C + H_{L_3}$$

$$= 25.5 + \frac{p_a}{w} + H_{L_3} \qquad ...(ii)$$

Equating (*i*) and (*ii*), we get

$$20.0 + \left[\frac{p_a}{w} + 10.0\right] + H_{L_2} = 25.5 + \frac{p_a}{w} + H_{L_3}$$

or $$\frac{p_a}{w} + H_{L_2} + 30.0 = \frac{p_a}{w} + H_{L_3} + 25.5$$

or $$H_{L_3} = H_{L_2} + 4.5$$

or $$H_{L_2} = H_{L_3} - 4.5 \qquad ...(iii)$$

But H_{L_3} *i.e.* head loss between D and C is given as :

$$H_{L_3} = \frac{f.L.V^2}{2gd} = \frac{f.L\left[\frac{Q}{A}\right]^2}{2gd}$$

$$= \frac{0.032 \times (360)\left[\frac{0.18 \text{ cumec}}{\frac{\pi}{4} \times (0.3)^2}\right]}{2 \times 9.81 \times 0.3}$$

$$= 12.7 \text{ m.} \qquad ...(iv)$$

Using equation (*iii*)

$$H_{L_2} = H_{L_3} - 4.5$$

$$= 12.7 - 4.5$$

or $$H_{L_2} = \mathbf{8.2\ m.} \qquad ...(v)$$

But head loss between D and B (H_{L_2}) is given as

$$= \frac{f\,LV^2}{2gd}$$

$$= \frac{0.032 \times 450 \times V^2}{2 \times 9.81 \times 0.45} = 8.2 \qquad \text{[from eq. } (v)\text{]}$$

or $$V^2 = 5.027$$

or $$V = 2.242 \text{ m/sec.}$$

$\therefore$ $$Q = AV = \frac{\pi}{4} \times (0.45)^2 \times 2.242 = 0.356 \text{ cumecs} = \mathbf{356\ l/sec.}$$

Hence, the flow rate into high level reservoir = 356 *l*/sec. **Ans.**

H.P. of the pump. The pump has to work against the head loss of $H_{L_1} + H_{L_2} + (RL\ 30.0 - RL\ 18.0)$

$$= H_{L_1} + H_{L_2} + 12.0 \qquad ...(vi)$$

where H_{L_1} = Head loss through AD

$$= \frac{0.032 \times 950 \times \left[\frac{0.536}{\frac{\pi}{4} \times (0.45)^2}\right]}{2 \times 9.81 \times 0.45}$$

$$= \mathbf{39.15\ m.}$$

$\left[\because \begin{array}{l} Q \text{ thorugh } AD \\ = Q \text{ thorugh } DC + Q \text{ thorugh } DB \\ = (0.180 + 0.356) \text{ cumecs} \\ = 0.536 \text{ cumecs} \end{array}\right]$

$\therefore$ Head loss against which pump has to work

$$= H_{L_1} + H_{L_2} + 12.0$$

$$= 39.15 + 8.2 + 12.0 = 59.35 \text{ m}$$

Theoretical horse power (H.P.) of the pump

$$= \frac{\gamma_w . Q . H}{0.735} = \frac{9.81 \times 0.536 \times 59.35}{0.735} = \mathbf{424\ H.P.}\ \textbf{Ans.}$$

Example 7.7. *A pump is to deliver water from an underground tank against a static head of 40 m. The suction pipe is 50 m long and is of 25 cm diameter with Darcy-weisbach friction factor $f = 0.02$. The delivery pipe is of 20 cm diameter, 1600 m long and has $f = 0.022$. The pump characteristics can be expressed as :*

$$H_p = 100 - 6000\ Q^2$$

where H_p = pump head in metres ; and

Q = discharge in m^3/s

Calculate the head and discharge of the pump. (*Civil Services, 1991*)

Solution.

(*i*) *Static head* $(H_1) = 40$ m. ...(*i*)

(*ii*) *Head loss in suction pipe* (H_2)

$$= \frac{f\,LQ^2}{2g \cdot \frac{\pi^2}{16} \cdot d^5}$$

$$= \frac{0.02 \times 50 \times Q^2 \times 16}{2 \times 9.81 \times \pi^2 \times (0.25)^5}$$

$$= 84.68\ Q^2 \quad \text{...(ii)}$$

(*iii*) *Head loss in delivery pipe* (H_3)

$$= \frac{0.022 \times 1600 \times Q^2 \times 16}{2 \times 9.81 \times \pi^2 \times (0.25)^5}$$

$$= 90.98\ Q^2 \quad \text{...(iii)}$$

Total head loss $H_p = H_1 + H_2 + H_3$

$$= 40 + 84.68\ Q^2 + 90.98\ Q^2$$

or $$H_p = 40 + 175.66\ Q^2 \quad \text{...(1)}$$

Using given Eqn. of pump

$H_p = 100 - 6000\ Q^2$, we have

or $40 + 175.66\ Q^2 = 100 - 6000\ Q^2$

or $6175.66\ Q^2 = 60$

or $$Q = \sqrt{\frac{60}{6175.66}} = \mathbf{0.0986\ cumecs.}\quad \mathbf{Ans.}$$

$\therefore$ $$H_P = 100 - 6000 \times (0.0986)^2 = \mathbf{41.7.}\quad \mathbf{Ans.}$$

Example 7.8. *For water supply of a town, water is pumped from a river 3 km away into a reservoir. The maximum difference of levels of water in river and the reservoir is 20 m. The population of the town is 50,000 and per capita water demand is 120 litre per day. If the pumps are to operate for a total of 8 hours and the efficiency of pumps is 80%, determine the horsepower of the pumps. Assume friction factor as 0.03, the velocity of flow as 2 m/s, and maximum daily demand as 1.5 times the average daily demand.* (*Engg. Services, 1992*)

Solution. Population of town = 50,000

Per capita demand = 120 l/day

$\therefore$ Average water required

$$= \frac{50{,}000 \times 120}{1000}\ \text{m}^3/\text{day} = 6000\ \text{m}^3/\text{day}$$

Maximum daily water required = $1.5 \times 6000\ \text{m}^3/\text{day} = 9000\ \text{m}^3/\text{day}$

The above quantity has to be pumped from the river in 8 hours to the reservoir.

$\therefore$ Water lifted per second

$$= \frac{9000}{8 \times 60 \times 60}\ \text{m}^3/\text{s} = \mathbf{0.3125\ m^3/s.}$$

Velocity through the pipe lifting the above discharge

= 2 m/s (given)

$\therefore$ dia. of pipe (d) is obtained as :

velocity × area = discharge

or $$2\left(\frac{\pi}{4}d^2\right) = 0.3125 \quad [\because \text{ Vel. = 2 m/s (given)}]$$

$\therefore$ $$d = \sqrt{\frac{0.3125 \times 4}{2\pi}} = 0.446\ \text{m ; say } 0.45\ \text{m.}$$

H_L = Head loss through the pipe of 3000 m long and of dia 0.45 m

$$= \frac{f\,lV^2}{2gd} = \frac{0.03 \times 3000 \times 2^2}{2 \times 9.81 \times 0.45} = 40.77\ \text{m.}$$

Total lift (H) = actual diff. of W.L. in reservoir and river + H_L

$$= 20\ \text{m} + 40.77\ \text{m} = \mathbf{60.77\ m}$$

$$\text{H.P. of pump} = \frac{\gamma_w \cdot QH}{0.735\eta}$$

$$= \frac{9.81 \times 0.3125 \times 60.77}{0.735 \times 0.8} \qquad (\eta = 80\%\ \text{given})$$

$$= \mathbf{316.5\ H.P.}\quad \mathbf{Ans.}$$

PROBLEMS

1. Pubic water supply is conveyed from the source to the centre of supply by gravitational system or pumping system or combined system. Draw three separate longitudinal sections indicating these systems, and show the positions of various details in each. (*Engg. Services, 1968*)

2. (*a*) Name the different types of pumps used generally in water supply scheme. What are the factors on which their selection depends. (*Engg. Services, 1969*)

   (*b*) (*i*) Calculate the head loss in friction in discharging 150 cu m/minute of water through a 1.2 m diameter steel main 900 metres long. Coefficient of friction (f) for steel pipe may be considered as 0.007. [**Ans.** 5.23 metres]

   (*ii*) If the main is duplicated and the same discharge is passed through both the mains over the same length, calculate the gain in head, and *daily* saving effected in pumping charges, assume that the pump works for two shifts of 8 hours and has an overall efficiency of 80% and costs 10 paise per pump horse power per hour.

[**Hint.** Due to duplication, half of the discharge will pass through each pipe, reducing the velocity to half in each pipe, and thereby causing $\frac{1}{4}$th of original head loss (H_L). In each pipe ; and the effective head loss will also be $\frac{1}{4}H_L$, since pipes are in parallel ; thereby giving a saving of $\frac{3}{4}$ of original head loss *i.e.*, $\frac{3}{4} \times 5.23 = 3.9225$ metres. **Ans.**

Daily saving in pumping cost

$$= \frac{\gamma_w . QH_{saving}}{0.735\eta} \times ₹\ 0.1 \times 16$$

$$= ₹\ \frac{9.81 \times 2.5 \times 3.9225}{0.735 \times 0.8} \times 0.1 \times 16$$

$$= ₹\ 261.36\ \text{P.}\quad \text{Ans.}]$$

3. How do you find out the most economical size of a rising main ? List the appurtenances necessary to be installed on rising main between the pumps and an overhead tank. Indicate their relative positions on a line diagram and mention the function of each.

4. Discuss the statement "Although the gravitational system is always to be preferred to a pumping scheme, still the latter is often the only economical way of obtaining the supply". Give the full data you would collect in the design of a pumping plant, pointing out clearly how these help in the design.

5. What are the different types of driving units commonly employed for running the pumping stations in public water supply schemes ? Explain their merits and demerits and state the conditions under which each of them is most suitable.

6. (*a*) When is water required to be lifted up by means of pumping while arranging for water supply schemes serving towns and cities ?

(*b*) What points should be considered in deciding the location of the pumping stations ?

7. What type of pumps will you recommend for the following ?

(*a*) Raw water pumping station lifting water from a turbid surface stream in plains through moderate height.

(*b*) Raw water pumping station lifting water from a surface source in hilly areas against high heads.

(*c*) Lifting filtered water from the clear water reservoir to the balancing distribution reservoir.

(*d*) Forcing water directly into distributions.

(*e*) Lifting water from a deep tubewell.

(*f*) Lifting water from a crooked tubewell.

8. (*a*) Explain the term 'boosting of water pressure' in an existing water distribution system. Draw a neat diagram showing suitable layout for a booster station. (*Engg. Services, 1970*)

(*b*) It is required to increase in a section of an existing water supply scheme, the effective pressure by 6 metres and the discharge by 40 percent. The distribution main is 0.6 metre in diameter, and the length leading to the section is 600 metres. The present discharge is 13,500 litres per minute. Design a suitable booster.

Coefficient of friction of pipe = 0.0075

Efficiency of motor = 0.8. (*Engg. Services, 1970*)

9. (*a*) Mention the method to calculate the most economical size of a rising main. List the appurtenances necessary to be installed on a rising main between pumps and over head tank. Indicate their relative positions on a line diagram.

(*b*) Find the b.h.p. of a pump which lifts water from a well to the treatment plant. Following is the data available :

Quantity of water to be lifted daily	= 40,000 m^3
Length of suction pipe	= 30 m
Length of rising main	= 170 m
Coefficient of friction	= 0.01
Pipe diameter	= 60 cm
Static head through which water is to be raised	= 25 m
Efficiency of pump	= 75%
Efficiency of motor	= 90%

Pumps work for two shifts daily, each shift being of 8 hours duration.

(*A.M.I.E., May 1974*)

[**Ans.** bhp of pump motor = 398.6 HP]

—8

Quality Control of Municipal and Industrial Water Supplies

8.1. General Introduction

Pure water is a chemical compound with each of its molecule (the smallest unit of a compound) containing two hydrogen atoms and one oxygen atom, and nothing else (H_2O being the chemical formula for water). However, pure water can never be available in nature. Even a man made drop of water, prepared in a laboratory by lighting the hydrogen and oxygen gases in a test tube, may not be perfectly pure, because the water drop so formed will dissolve the glass of the test tube (although a very very little bit). The precipitation, in the same manner, at the instant of its formation, contains no impurities, but during the process of formation and fall through the earth's atmosphere, it may dissolve certain gases, traces of minerals, and other substances. When once the precipitation reaches the earth's surface, many more opportunities are presented for the introduction of various physical, chemical, or bacterial impurities in it.

As the rain water flows over the surface of the earth, it picks up or dissolves particles of soil, garbage, sewage, pesticides and other human, animal or chemical wastes. It may also pick up and dissolve certain decayed organic materials, such as plants or dead animals. As the surface water seeps into the "Ground-Water Reservoir," most of the suspended particles are filtered out, but on the other hand, the water dissolves the minerals and salts present in the earth's layers, through which it travels before joining the watertable.

The impurities which water dissolves or picks up as suspended matter, may sometimes make it *more useful and potable* for public uses and especially for drinking, and sometimes it may render it *harmful and unfit*. For example, certain minerals such as iron, calcium, magnesium, fluorine, etc. in small quantities may be useful and good for the health of the people, because human beings need a certain amount of these elements in their bodies. But when these materials and others are dissolved in large amounts or in certain combinations, the water may become unfit or less useful for municipal, industrial and other uses. For example, sometimes the water may contain toxic or poisonous substances such as arsenic, barium, cadmium, chromium, cyanides, lead, selenium silver, copper, etc., which may be very harmful to the public health, even if present in low quantities. Sometimes, the water may contain too much of common salt, thereby rendering it *brackish* and making it undrinkable and less useful for cloth washing or for irrigation and farming. Similarly, sometimes the water may contain harmful bacteria, the presence of which may cause diseases such as cholera, typhoid, dysentery, gastro-enteritis, infectious hepatitis (*i.e.* jaundice), etc.

To ensure safety to public health, economy and utility in industries and other uses, it, therefore, becomes imperative upon the planners and designers of the public water supply schemes, to thoroughly *check, analyse,* and treat the raw available water to safe and permissible limits, before supplying to the public. This is more true and must be strictly adhered to, when water is supplied for municipal uses, such as drinking, bathing, washing, etc. When water is required solely for industries or for irrigation*, the quality standards may vary depending upon the requirement of each industry or farming and irrigation. The quality requirement for certain selected industries are so great that these industries have to develop their own supplies.

8.2. Characteristics of Water

The raw or treated waters can be checked and analysed by studying and testing their physical, chemical, and microscopical characteristics, as explained below :

8.2.1. Physical Characteristics. Physical analysis of water is carried out in order to determine the physical characteristics of water. This includes tests for determining turbidity, colour, taste or odour, temperature, specific conductivity, etc. These tests are described below :

(*i*) **Turbidity.** If a large amount of suspended matter such as clay, silt or some other finely divided organic materials are present in water, it will appear to be *muddy* or *cloudy* or *turbid* in appearance. The turbidity depends upon the finesse and concentration of particles present in water. Although, the clay or other inert suspended particles may not be harmful to health, yet they are to be removed or reduced for aesthetic and psychological reasons.

Since people do not like turbid water, the turbidity of raw water must be measured, and then reduced by treatment to permissible values, so as to make it almost invisible under naked eye. The turbidity is measured by a *turbidity rod* or by a *turbidimeter* with optical observations, and is expressed as the amount of suspended matter in mg/l or ppm.** *The standard unit is that which is produced by 1 milligram of finely divided silica in 1 litre of distilled water*. A turbidity in excess of 5 units is easily detectable in a glass of water, and is thus, usually objectionable. Turbidity in a clear lake water is about 25 units ; and for muddy waters, it exceeds about 100 units. Although certain health standards lay down a permissible limit of 5 to 10 units of turbidity for drinking waters, yet the modern approach is to put no safe limit to turbidity, as in certain cases, even a slight trace in a normally clear water, such as in chalk-well water, might be a sign of serious pollution.***

*Please refer chapter on "Irrigation Techniques and Quality of Irrigation Water" in "Irrigation Engineering and Hydraulic Structures" by the same author.

**The term "parts per million" (ppm) is currently being replaced by the term milligrams per litre (mg/l). Milligrams per litre is defined as the quantity of matter in milligrams (10^{-3} gm) present in one litre of water. For water, ppm and mg/l are approximately equal.

***The term "Pollution" here means the presence in water of any foreign substances (organic, inorganic, radiological, or biological) which tend to lower its quality to a point that it becomes a health hazard or impairs the usefulness of water.

The insoluble particles of soil, organics, microorganisms and other material, impeding the passage of light through water by scattering and absorbing rays, thus causes turbidity, *which can be measured as the interference to the passage of light through the given water.* Turbidity can be measured in the field by using *turbidity rods* and *turbidimeters*, which are discussed below.

Turbidity rod

The turbidity can be easily measured in the field with the help of a **turbidity rod** (Fig. 8.1). It consists of an aluminium rod which is graduated, as to give the turbidity directly in silica units (mg/l). To the upper end of this rod is attached a graduated non-stretchable tape, so as to help in lowering of the rod. A screw containing a platinum needle and a nickel ring is inserted at the lower end of the rod, as shown. The standard platinum needle is 1 mm in diameter and 25 mm long. A vertical rod or a stick is inserted in the nickel ring, so as to support and keep the rod in a vertical position. There is a mark for eye position on the graduated tape, as shown.

In order to measure the turbidity, the graduated aluminium rod is lowered in water, and keeping the eye at its upper end, the submerged needle is viewed. The rod is then slowly moved downwards (and the eye along with it) till the needle just disappears due to the turbidity of water. The length of the rod under water is a measures of the turbidity. The lesser the length, the greater is the turbidity and vice versa. The rod is graduated specially and the reading on it near the surface of water directly gives the turbidity in ppm.

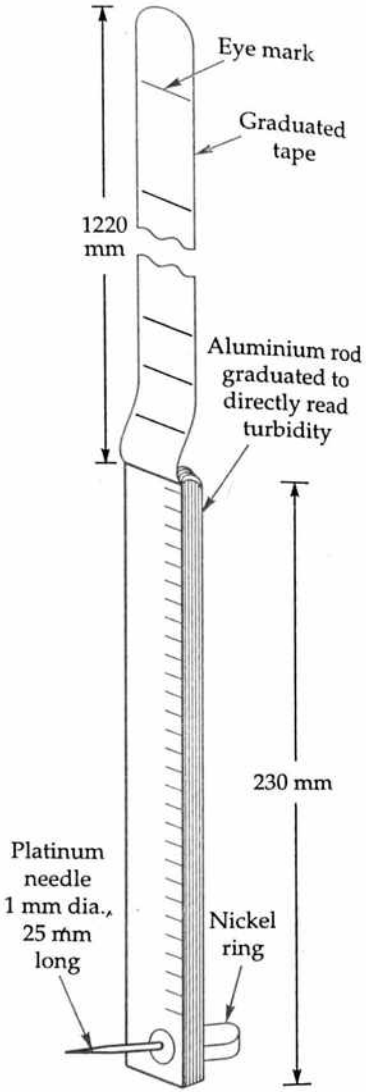
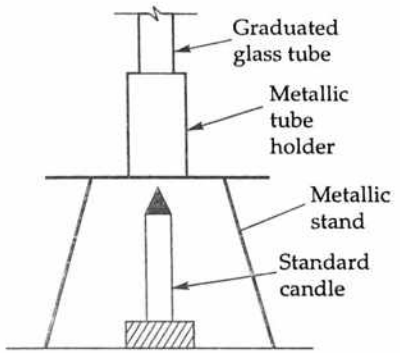


Fig. 8.1. Turbidity Rod.

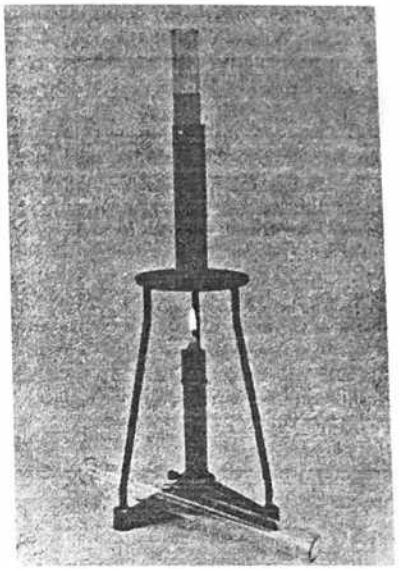
Turbidimeters

The turbidity can be measured in the laboratory with the help of instruments called **turbidimeters.** In general, a turbidimeter works on the principle of measuring the interference caused by the water sample to the passage of light rays. Jackson and Baylis types of turbidimeters used from the past, as well as the modern commercial turbidimeters, are briefly described below :

Jackson's turbidimeter. A **Jackson's turbidimeter**, also called a ***Jackson's candle turbidimeter*** can be used to measure turbidities in the *range between 25-1000 mg/l.* It consists of a calibrated glass tube, which can be placed on a



(*a*) Section through a Jackson candle turbidimeter



(*b*) Photoview of a Jackson candle turbidimeter

Fig. 8.2

metallic cylindrical tube holder fixed over a metallic stand provided with a fixed standard candle at its bottom, as shown in Fig. 8.2 (*a*) and (*b*).

With the glass tube in place over the lighted candle, the water sample is gradually added to the glass tube to increase the height of the water column in the tube, and the candle flame is observed from the top of the tube. The addition of water is stopped as soon as the image of the candle flame ceases to be seen. At this stage, the height of the turbid water will provide just enough turbidity for preventing the candle light to pass through it. This height of water column will, therefore, be more for less turbid water, and *vice versa*.

The height of the water column, measured in the calibrated glass tube, will thus, provide a measure of the turbidity of the water. The longer the light path, the lower the turbidity ; say for example, a light path of 10.8 cm corresponds to the turbidity of 200 JTU*, of 21.5 cm to 100 JTU, and of 72.9 cm to 25 JTU.

Such a turbidimeter cannot measure turbidities lower than 25 JTU or 25 mg/l, which is the typical value for a clear lake water, as against a value exceeding 100 JTU for muddy water.

Since the lowest turbidity value that can be measured on this instrument is 25 JTU or 25 mg/l or so, such a turbidimeter is limited in application to turbid raw waters of natural sources only, and it can not be used to measure the turbidities of treated supplies, for which *Baylis turbidimeters* or modern *nephelometers* are used.

*Jackson Turbidity Unit (turbidity unit on silica scale equalling mg/l).

Baylis turbidimeter. A **Baylis turbidimeter,** shown in Fig. 8.3, *is generally used to measure turbidities in the range of 0-10 mg/l.* It consists of a closed galvanised iron box, on one side of which, two glass tubes can be held vertically side by side. On the other side, in front of the tubes, a 250 watts electric bulb is located with a reflector, so as to throw light on the tubes. The glass tubes are supported at bottom by a white opal glass plate and are surrounded by blue cobalt plates, as shown.

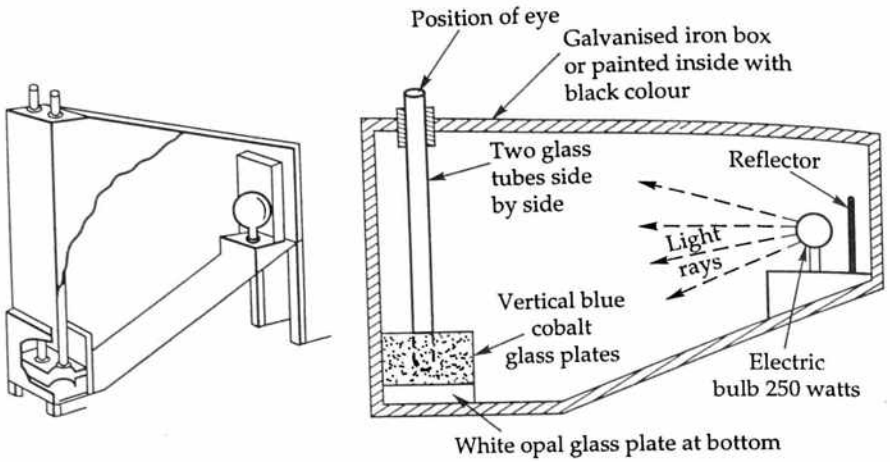


Fig. 8.3. Baylis turbidimeter.

One of the two glass tubes is filled with water sample (whose turbidity is to be measured) and the other is filled with standard water solution of known turbidity. The standard electric bulb is lighted, and its light thus, falls on the water samples in the glass tubes through the cobalt glass plates. The blue colour in both the tubes is observed from the top of the instrument. If the colour of both the tubes differ, the standard solution tube is replaced by anther standard tube of different known turbidity. The process is continued till a matching is obtained in the colours of both the tubes. The known turbidity of the standard solution will then correspond to the turbidity of the sample of water. *Baylis turbidimeter is generally used for measuring turbidities in the range of 0-10 ppm, although it can measure higher turbidities up to about 100 mg/l also.*

Modern commercial turbidimeters, called **Nephelometers** have since been designed, and are being used these days on a large scale, particularly for measuring very low turbidities of drinking waters (up to less than one unit).

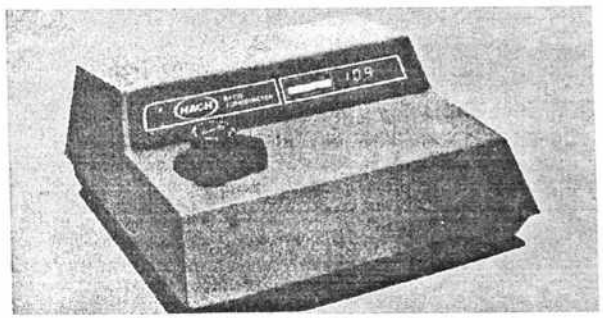
The basic difference between the old and the new turbidimeters is that : in the old instruments, the light was seen or measured after the same passed straight through the turbid water ; while in a modern turbidimeter, the light intensity is measured at right angles to the incident light. Such an instrument therefore uses a photometer to measure the intensity of light passing through the turbid water, after the same is scattered at right angles to the incident light.

Since no direct relationship exists between the intensity of light scattered at 90° and Jackson candle turbidity, calibration of a nephelometer in terms of candle or silica units is not valid. To distinguish between the turbidities measured from instrumental and visual methods, nephelometric turbidity units are abbreviated as **NTU (Naphelometric Turbidity Units).** Occasionally, the nephelometric turbidity units are referred to as **FTU (Formazin Turbidity Units),** since formazin polymer is used as a reference turbidity standard suspension in place of silica suspension.

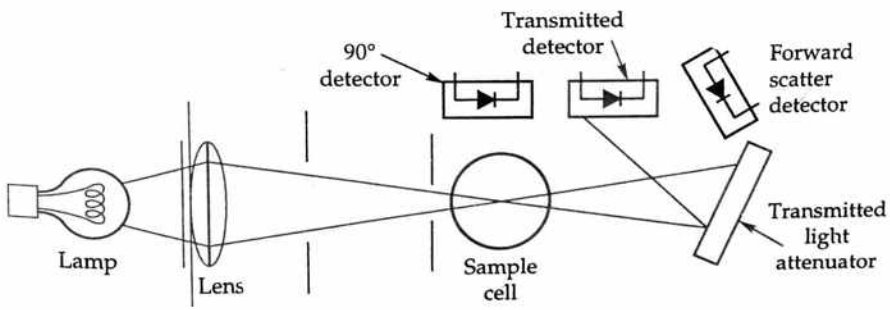
A commercial **Ratio turbidimeter** which can measure the ratio of the 90° scattered light to the transmitted light is illustrated in Fig. 8.4.

In such an instrument, light is focussed by a lens before passing horizontally through the water sample. In an ordinary turbidimeter, this measurement alone would be used to determine the turbidity. In addition, the ratio turbidimeter detects the transmitted light.

At low or moderate turbidity levels, the forward scatter signal is negligible compared to the transmitted signal, *and the output is simply a ratio of the 90° scattered light to the transmitted light.* This ratioing stabilises the instrument and negates the effect of colour, if present in water. At high turbidity levels, the instrument records the ratio of the 90° scattered light to the sum of the forward scatter and the transmitted light, thus allowing linearity over the entire range.



(a) Photoview of a Ratio turbidimeter



(b)

Fig. 8.4. Light path diagram for a ratio turbidimeter, to accurately measure the turbidities for natural cloudy as well as treated waters with turbidities less than 1 NTU.

(*i*) **Colour.** Dissolved organic matter from decaying vegetation or some inorganic materials, such as coloured soils, etc., may impart colour to the water. The excessive growth of algae and aquatic micro-organisms may also sometimes impart colour to the water. The presence of colour in water is not objectionable from health point of view, but may spoil the colour of the clothes being washed in such waters, and is also objectionable from aesthetic and psychological point of view, as people may not like to drink coloured waters.

The colour in water can be easily detected by the naked eye. It can be measured by comparing the colour of water sample with other standard glass tubes (called ***Nessler tubes***) containing solutions of different standard colour intensities. *The standard unit of colour is that which is produced by one milligram of platinum cobalt dissolved is one litre of distilled water.* Different standard colour intensities, representing as 1 ppm, 2 ppm, 3 ppm, ... can thus be easily prepared by dissolving 1 mg, 2 mg, 3 mg ..... respectively of platinum cobalt in 1 litre of water. The maximum permissible colour for domestic supplies is 20 ppm (based on cobalt scale, given above).

For precise determination of small colour intensities, compact instrument properly lighted from inside, called a **tintometer**, is generally used. The instrument contains an eye piece with two holes. A slide of standard coloured water is seen through one hole, and the slide of water to be tested is seen from the other hole. The standard coloured slide is replaced by another, till a matching is obtained, as is done in a turbidimeter. The standard colour intensity corresponding to this matching will represent the intensity of colour of the water sample.

For public supplies, the colour number on cobalt scale should not exceed 20, and should preferably be less than 10.

(*iii*) **Tastes and Odours.** The dissolved organic materials or the inorganic salts, or the dissolved gases may impart tastes and odours to the water, *which generally occur together.* Taste and odour may be caused by the presence of: *dissolved gases* such as H_2S, CH_4, CO_2, O_2 etc., combined with organic matter; *mineral substances* like NaCl, iron compounds, carbonates and sulphates of other elements ; and *phenols* and other tarry or oily matter, especially after chlorination. *Some tastes, such as those imparted by dissolved oxygen and dissolved carbon dioxide are generally desirable.* Evidently, for drinking purposes, the water *must not contain* any undesirable or objectionable taste and odour. In order to remove the taste and odour from the raw water, the chemical composition of water must first be ascertained, so as to determine as to what treatment, if any, is required to make its taste and odour acceptable for domestic use.

The extent of taste or odour present in a particular sample of water is measured by a term called ***odour intensity,*** which is related with the *threshold odour.* The threshold odour or **threshold odour number**, as it is generally called, represents the dilution ratio at which the odour is hardly detectable. Water to be tested is, therefore, gradually diluted* with odour free water, and the mixture at which the detection of odour by human observation is just lost, is determined. The number of times the sample is diluted, represents the

*Successive dilution ratio is generally 1 : 4.

threshold odour number. Thus, if 40 ml sample of water is diluted to make 200 ml (till it just looses its taste or odour) ; then the threshold odour number will be 5. For public supplies, the water should generally be free from odour, *i.e.*, the threshold odour number should be 1 and should never exceed 3.

In routine examination of public supplies, odour is generally determined when water is cold ; although, however, it may change with change in temperature, sometimes not being noticeable when cold. In thorough analysis of water, however, odour should be determined at different temperatures, and results produced in tabulated form. Such analysis may help in revealing the biological history of water.

(*iv*) **Temperature.** Testing the temperature of water has usually no practical significance in the sense that it is not usually possible to give any treatment to control the temperature of water.

However, from the academic point of view, it may be mentioned that for potable waters, temperatures of about 10°C are highly desirable, while temperatures above 25°C are considered objectionable.

(*v*) **Specific conductivity of water.** Electrical conductivity of water is a measure of water's ability to conduct electricity. *Pure water is a poor conductor of electricity;* but it shows significant conductivity when ions of dissolved salts are present in it. The electrical conductivity, called the **specific conductivity** or the **specific conductance** of water, therefore, approximately measures the number of ions* present in water; and hence approximately measures the ***total dissolved solids*** in water. The elements whose ionic forms contribute to these measures include Calcium (Ca^{++}), Magnesium (Mg^{++}), Potassium (K^+), Bicarbonate (HCO_3^-), Sulphate (SO_4^{--}) and Chloride (Cl^-), etc. The higher is the concentration of such ionic (dissolved) constituents, the higher will be the conductivity.

However, since some organic molecules and compounds may dissolve in water without ionizing**, they may remain unaccounted while estimating, TDS from the measured electrical conductivity. The measured TDS in such cases (from electrical conductivity measurements) will be somewhat less than the actual TDS.

The specific conductivity of water is measured by a *portable ionic water tester* (fountain pen or ball pen size), called a ***conductivity sensor.*** It measures as to how much electricity is being conducted through a centimetre of water. It is hence expressed in units of mhos per centimeter. [mhos is the unit of conductivity and equals 1 ampere/1 volt]. This unit has been renamed by ISO (International Standards Organization) as Siemen (1 mhos = 1 Siemen). Since mhos is a very large unit, the micromhos *i.e.* microsiemen (μS/cm) is typically used. In clean water, (*i.e.* suface water) the order of magnitude is in the range of μS/cm (10^{-6} S/cm).

*Ions are electrically charged atoms or molecules; both +vely charged (cations) and –vely charged (anions).

The **soluble non-ionised substances may be present in water in the form of molecules of various sizes and formulae; for instance :

(*a*) *Carbon dioxide* is a small molecule with a simple formula CO_2; and

(*b*) *Sugar* is a large molecule with a complicated formula $C_{12}H_{22}O_{11}$.

Since the specific conductivity of water will also increase with temperature*, its measurement is normally standardised at 25°C.

In order to convert the specific conductivity or electrical conductivity of a water sample (μS/cm) into approximate concentration of total dissolved solids (TDS in ppm), the μS/cm is multiplied by a conversion factor, called the **conductivity factor.**

This factor depends on the chemical composition of the TDS, and can usually vary between 0.54 to 0.96**. A value of 0.67 is commonly used as an approximation, if the actual factor is not known. Thus,

TDS (in ppm or mg/l) = [Conductivity of μS/cm] 0.67

8.2.2. Chemical Characteristics. Chemical analysis of water is carried out in order to determine the chemical characteristics of water. This involves tests for determining the total solids and suspended solids, pH value, hardness, chloride content, nitrogen content, iron, manganese and other metal contents, dissolved gases etc. These tests are described below :

(*i*) **Total solids and Suspended solids.** The total amount of solids (suspended as well as dissolved solids) present in water can be determined by evaporating a sample of water and weighing the dry residue left. The suspended solids can be found by filtering the water sample and weighing the residue left on the filter paper. The difference between the total solids and the suspended solids will then represent nothing but the dissolved solids***.

The total permissible amount of solids in water is generally limited to 500 ppm, although higher amounts up to 1000 ppm are also sometimes permitted, but are likely to produce certain psychological effects on human system.

(*ii*) **pH value of water.** The pH value of water indicates the logarithm of reciprocal of hydrogen ion concentration present in water. It is thus an indicator of the acidity or the alkalinity of water.

*Increased temperature provides increased activity or ionic movement, which enables more electiricity to be carried through the solution from one electrode to another.

**A factor of about 0.5 is quite suitable for sea water (sodium chloride based) but generally is incorrect for ground water. The conductivity factors for specific ions is shown in table 8.1 (*a*) below :

Table 8.1(a) Conductivity Factors for Ions Commonly found in Water

Ion	*Conductivity Factor ppm/(μS/cm)*
Potassium	0.65
Sodium	0.47
Calcium	0.38
Magnesium	0.26
Carbonate	0.35
Chloride	0.47
Sulphate	0.65
Nitrate	0.87
Bicarbonate	1.4

***The dissolved solids can also be estimated by physical analysis by determining the specific conductivity of water as explained earlier.

Since the pH is the log of reciprocal of H^+, the higher values of pH means lower hydrogen ion concentrations, and thus represent *alkaline* solutions ; whereas, the lower values of pH means higher hydrogen ion concentrations representing *acidic* solutions.

Truly speaking, pure water is a balanced combination of positively charged hydrogen ions (*i.e.* H^+) and negatively charged hydroxyl ions (*i.e.* OH^-). In **pure or neutral waters,** their number is equal. Further, it has been found that the product of concentration of H ions and concentration of OH ions in a water solution, is constant*. This constant has been found to be equal to 10^{-14} moles/ litre. Therefore, if H ion and OH ion concentrations are equal, virtually each will have a concentration equal to $\sqrt{10^{-14}} = 10^{-7}$ moles per litre. A neutral water will, therefore, have a pH equal to

$$\log_{10}\left(\frac{1}{H^+}\right) = \log_{10}\left(\frac{1}{10^{-7}}\right) = \log_{10} 10^7 = 7$$

If an acid is added to the neutral or pure water, the number of hydrogen ions will increase because of the hydrogen contained in the acid. For example, if hydrochloric acid is dissolved in water, the dissociation becomes

$$H_2O \rightleftharpoons H^+ + OH^-$$
$$HCl \rightleftharpoons H^+ + Cl^-$$

Then the net concentration of hydrogen ions will exceed that of hydroxyl ions, and will be more than the hydrogen ion concentration of neutral water (*i.e.* 10^{-7}) and thus decreasing the pH value to less than 7, and thereby making the water acidic.

Similarly, if an alkali is dissolved in pure water, the concentration of hydroxyl ions will exceed the hydroxyl ion concentration of neutral water, and thus, reducing the concentration of hydrogen ions to less than 10^{-7}. [Their product being constant = 10^{-14} moles/ litre]. For example, if sodium hydroxide is added to water, the dissociation will be

$$H_2O \rightleftharpoons H^+ + OH^-$$
$$NaOH \rightleftharpoons Na^+ + OH^-$$

The net amount of OH ions present in this alkaline water will, therefore, be more than that present in a neutral water, and thus reducing the hydrogen ion concentration to less than 10^{-7}, and thereby increasing the pH above 7, and making the water alkaline.

Hence, if the pH of water is more than 7, it will be alkaline, and if it is less than 7, it will be acidic. The maximum acidity will be at zero value of pH, and the maximum alkalinity will be at a value of pH equal to 14.

Testing the pH value of water, therefore, directly tells us as to whether the water is acidic or is alkaline. Generally speaking, the alkalinity is caused by the presence of bicarbonates of calcium and magnesium ; or by the carbonates or hydroxides of sodium, potassium, calcium and magnesium. Alkalinity caused by bicarbonates is called ***bicarbonate alkalinity*** ; that caused by carbonates is called ***carbonate alkalinity*** ; and that caused by hydroxides is

*According to the law of electrolytes ;

$$\frac{\text{Concentration of cations} \times \text{Conc. of anions}}{\text{Concentration of undissociated molecules}} = \text{Constant ;}$$

Since the dissociation of water is very slight, the concentration of undissociated molecules may be taken as 100%, or unity.

called hydroxide alkalinity or ***caustic alkalinity***. Some, but not all, of the compounds that cause alkalinity also cause *hardness**. Acidity on the other hand, is caused by the presence of mineral acids, free carbon dioxide, sulphates of iron and aluminium, etc. in water.

Measurement of pH. The pH value of water can be measured quickly and automatically with the help of a Potentiometer, which measures the electrical potential exerted by hydrogen ions and thus indicating their concentration. It can also be measured with the help of colour indicators (dyes) which are added to the water, and the colour produced is compared with the standard colours of known pH values. Sets of standard sealed tubes containing coloured waters of known pH values with different indicators are generally kept in the public health laboratories for ready reference. The indicators generally used for different pH ranges are given in Table 8.1.

Table 8.1. Indicators for measuring pH values of water

Indicator	*pH range*	*Original colour of indicator dye*	*Final colour produced in water*
Thymol blue	1.2—2.8	Red	Yellow
Bromophenol blue	3.0—4.6	Yellow	Blue
Methyl orange	2.8—4.4	Red	Yellow
Brom cresol green	3.8—5.4	Yellow	Blue
Methyl red	4.4—6.2	Red	Yellow
Chlorophenol blue	5.4—6.8	Yellow	Red
Brown-thymol blue	6.0—7.6	Yellow	Blue
Phenol red	6.8—8.4	Yellow	Red
Cresol red	7.2—8.8	Yellow	Red
Thymol blue	8.0—9.6	Yellow	Blue
Phenolphthalein red	8.6—10.3	Yellow	Red
Tolyl red	10.0—11.6	Red	Yellow

Permissible pH values. For public water supplies, attempts should be made to keep the pH as close to 7 as possible. The lower values (*i.e.* acidic waters) may cause tuberculation and corrosion, while the higher values (*i.e.*, alkaline waters) may produce incrustation, sediment deposits, difficulties in chlorination, certain psychological effects on human system, etc. However, the permissible pH values for public supplies may range between 6.6 to 8.5.

Example 8.1. *In a water treatment plant, the pH values of incoming and outgoing waters are 7.2 and 8.4 respectively. Assuming a linear variation of pH with time, determine the average pH value of water.*

Solution. By definition of pH value, we have

$$pH = \log \left[\frac{1}{H^+}\right]$$

Using suffix 1 for incoming water and 2 for outgoing water, we have

$$(pH)_1 = 7.2 = \log_{10} \frac{1}{H_1^+}, \qquad (pH)_2 = 8.4 = \log_{10} \frac{1}{H_2^+}$$

*Described in the next article.

Now $\log_{10} \frac{1}{H_1^+} = 7.2$

or $(-) \log_{10} H_1^+ = 7.2$

or $H_1^+ = 10^{-7.2}$

Similarly $H_2^+ = 10^{-8.4}$

$$\text{Average value of } H^+ = \frac{H_1^+ + H_2^+}{2} = \frac{10^{-7.2} + 10^{-8.4}}{2}$$

$$= 10^{-8.4}\left[\frac{(10)^{1.2}+1}{2}\right] = 10^{-8.4}\left[\frac{15.85+1}{2}\right]$$

$$= 10^{-8.4} \times 8.42 = \mathbf{8.42 \times 10^{-8.4}}$$

$\therefore$ Average value of pH

$$= \log_{10}\left(\frac{1}{8.42 \times 10^{-8.4}}\right) = \log_{10}\left(\frac{10^{8.4}}{8.42}\right)$$

$$= \log_{10}\left(10^7 \times \frac{10^{1.4}}{8.42}\right) = \log_{10} 10^7 \left(\frac{25.12}{8.42}\right)$$

$$= \log_{10} 10^7 \times 2.98 = \mathbf{7.474.} \quad \textbf{Ans.}$$

(*iii*) **Hardness of water**. Hardness in water is that characteristic which prevents the formation of sufficient leather or foam, when such hard waters are mixed with soap. It is usually caused by the presence of Calcium and Magnesium salts* present in water, which form scum by reaction with soap.

Hard waters are undesirable because they may lead to greater soap consumption, scaling of boilers, causing corrosion and incrustation of pipes, making foods tasteless, etc.

If bicarbonates and carbonates of calcium and magnesium are present in water, the water is rendered hard *temporarily*, as this hardness can be removed to some extent by simple boiling or to full extent by adding lime to the water. Such a hardness is known as ***temporary hardness*** or ***carbonate hardness.*** When such waters are boiled, carbon dioxide gas escapes out and the insoluble calcium carbonate gets precipitated. Magnesium carbonate, being fairly soluble in water won't get removed by boiling. The temporary hard waters, therefore, do cause deposition of calcium scales in boilers.

On the other hand, if sulphates, chlorides and nitrates of calcium or magnesium are present in water, they cannot be removed at all by simple boiling, and, therefore, such waters require special treatment for *softening***. Such a hardness is known as ***permanent hardness*** or ***non-carbonate hardness.***

Measurement of hardness. Hardness is generally defined as the calcium carbonate equivalent of calcium and magnesium ions present in water, and is expressed in mg/l. After determining the amounts of calcium and magnesium ions present in water by titration with versnate solution*** (*i.e.* EDTA method), the hardness in mg/l can be easily determined as follows :

*In addition to Ca^{++} and Mg^{++}, other divalent metallic ions like Fe^{++}, Mn^{++}, Sr^{++} etc. may also cause hardness, but their presence in most of the waters is generally negligible, and hence hardness for all practical purposes, is attributed to Ca^{++} and Mg^{++} only.

**Described in article 9.24.

***Please refer some book on Titrations in Chemistry for referring details of this titration for computing Ca^{++} and Mg^{++} ions separately, if needed. The EDTA method used for computing total hardness is also explained under article 15.7.

Total hardness in mg/l as $CaCO_3$ (T.H.)

$$= \left[Ca^{++} \text{ in mg/l} \times \frac{\text{Combining wt. of } CaCO_3}{\text{Combing wt. of } Ca^{++}} \right] + \left[Mg^{++} \text{ in mg/l} \times \frac{\text{Combining wt. of } CaCO_3}{\text{Combing wt. of } Mg^{++}} \right] \quad ...(8.1)$$

The combining wt. of Ca^{++} and Mg^{++} and $CaCO_3$ respectively are 20, 12 and 50. Knowing the quantities of calcium and magnesium ions present in water, the hardness in mg/l is easily computed as :

$$\text{Total Hardness (T.H.)} = Ca^{++} \text{ mg/l} \left(\frac{50}{20}\right) + Mg^{++} \text{ mg/l} \left(\frac{50}{12}\right) \quad ...(8.1a)$$

Carbonate hardness is equal to the total hardness or Alkalinity, whichever is lesser.

Non-carbonate hardness is the total hardness in excess of the alkalinity. If the alkalinity is equal to or greater than the total hardness, there is no non-carbonate hardness.

For example, the carbonate and non-carbonate hardness of a water having a total alkalinity of 200 mg/l as $CaCO_3$, and 120 mg/l of Ca^{++}, and 60 mg/l of Mg^{++} as ions, can be computed as follows :

$$\text{Total hardness} = 120 \times \frac{50}{20} + 60 \times \frac{50}{12} = 300 + 250$$
$$= 550 \text{ mg/l as } CaCO_3.$$

The carbonate hardness is the lesser of the total hardness or Alkalinity, and hence equals 200 mg/l.

The non-carbonate hardness is the rest of the total hardness, and hence equals to 550 – 200 = 350 mg/l as $CaCO_3$.

It would also be pertinent to point out here that the **total alkalinity** in water consists of alkalinity caused by CO_3^{--}, alkalinity caused by HCO_3^- and by OH^-. A very little negative alkalinity is also caused by H^+.

Hence, Total Alkalinity = $CO_3^{--} + HCO_3^- + OH^- - H^+$

(All ions expressed in mg/l as $CaCO_3$)

For example, the total alkalinity of a water containing 90 mg/l of CO_3^{--} ions and 61 mg/l of HCO_3^- ions can be computed as follows :

$$CO_3^{--} \text{ as } CaCO_3 = 90 \times \left(\frac{50}{30^*}\right) = 150 \quad ...(i)$$

$$HCO_3^- \text{ as } CaCO_3 = 61 \times \left(\frac{50}{61}\right) = 50 \quad ...(ii)$$

Total alkalinity = (*i*) + (*ii*) = 200 mg/l.

The above calculation of alkalinity has ignored OH ions and H ions, which can be computed only if the pH of water is known, though generally they have a very very small effect on the alkalinity. Say for example, if pH of the above water is say 10, then eventually, H ions = 10^{-10} moles/l and OH ions = 10^{-4} moles/l.

*Combining wt of CO_3^{--} ion $= \frac{12+48}{2} = \frac{60}{2} = 30$

or $H^+ = 10^{-10}$ moles/l

$= (10^{-10} \text{ moles/l})\ (1 \text{ gm/mole})\ (10^3 \text{ mg/gm})$

$= 10^{-7} \text{ mg/l} = 10^{-7} \times \left(\frac{50}{1}\right) \text{ mg/l of } CaCO_3$

$= 5 \times 10^{-6} \text{ mg/l as } CaCO_3 \quad ...(iii)$

and $OH^- = 10^{-4}$ moles/l

$= (10^{-4} \text{ moles/l})\ (17 \text{ gm/mole})\ (10^3 \text{ mg/gm})$

$= 1.7 \text{ mg/l} = 1.7 \times \left(\frac{50}{17}\right) \text{ mg/l of } CaCO_3$

$= 5.0 \text{ mg/l of } CaCO_3 \quad ...(iv)$

It can thus be seen that the OH ions contribute to + 5 mg/l of alkalinity, and H ions contribute to -5×10^{-6} mg/l of alkalinity, which are very small as compared to alkalinity values of 200 mg/l caused by carbonate and biocarbonate ions.

Sometimes, the hardness is represented in degree of hardness. Each British degree of hardness is equal to 14.25 mg/l*. French or French degree is equal to 10 mg/l of hardness (as $CaCO_3$ of course). In general, under a normal range of pH values, waters with hardness upto 75 ppm are considered *soft*, and those with 200 ppm and above are considered *hard*. In between, the waters are considered as *moderately hard*. Underground waters are generally harder than the surface waters, as they do have more opportunity to come in contact with minerals. Their hardness, many a times, exceed 300 ppm, and thus requiring treatment. For boiler feed waters and for efficient cloth washing in laundries, the waters must be soft with hardness less than 75 ppm or so. However for drinking purposes, waters with hardness below 75 ppm are generally tasteless, and hence, the prescribed hardness limit for public supplies ranges between 75 to 115 ppm.

(*iv*) **Chloride content.** Chlorides are generally present in water in the form of sodium chloride (common salt) and may be due to : leaching of marine sedimentary deposits, pollution from sea water, brine or industrial and domestic wastes, etc. Their concentrations above 250 mg/l produce a noticeable salty taste in drinking water and are thus objectionable.

The presence of high quantity of chloride (or sodium chloride) in river or stream waters may indicate pollution of water due to sewage and other human** or industrial wastes. The chloride concentrations of raw waters, being used for public supplies, should, therefore, regularly be tested, so as to immediately detect any sudden increase in its chloride content and the possibility of any organic pollution of water.

The chloride content of water can be measured by titrating the water with standard silver nitrate solution using potassium chromate as indicator. The chloride content of treated water to be supplied to the public should not exceed a value of about 250 ppm.

(*v*) **Nitrogen content.** The presence of nitrogen in water is an indication of the presence of the organic matter, and may occur in one or more of the following forms :

(*a*) Free ammonia or Ammonia nitrogen ;

(*b*) Albuminoid or Organic nitrogen ;

(*c*) Nitrites ; and (*d*) Nitrates.

*One degree of hardness means the soap destroying power produced by 14.25 milligrams of calcium carbonate in one litre of water.

**Considerable salt, of the order of 1 per cent, is excreted by human beings in their urines.

The *free ammonia* or *ammonia nitrogen* indicates the very first stage of decomposition of organic matter and consequent release of free ammonia (thus indicating recent pollution) ; *albuminoid or organic nitrogen* indicates the quantity of nitrogen present in water in the form of undecomposed organic matter (mainly in the form of proteins and amino acids) which on digestion will release ammonia ; the *nitrites* indicate the presence of partly decomposed (not fully oxidised) organic matter ; and *nitrates* indicate the presence of fully oxidised organic matter in water (thus representing old pollution).

The presence of ***free ammonia*** in water indicates the start of decomposition of organic matter present in water; and for potable waters, its value should not exceed 0.15 mg/l. It can be easily measured by simply boiling the water and measuring the liberated ammonia gas, by *distillation process*.

The presence of ***organic nitrogen*** in water also indicates pollution and presence of undecomposed organic matter. For potable waters, it should not exceed a value of about 0.3 mg/l. It can be determined by digestion of the organic matter and measuring the released ammnonia gas. This can be easily done by adding strong alkaline solution of potassium permanganate ($KMnO_4$) to already boiled* water sample and again boiling the same, when ammonia gas is liberated, which is measured, so as to indicate the amount of organic nitrogen present in water. The sum total of ammonia nitrogen and the organic nitrogen is called the **Kjedahl nitrogen**.

The partly oxidised organic matter present in water in the form of ***nitrites*** *is highly dangerous and, therefore, the permissible amount of nitrites present in potable water should be nil.* The ***nitrates***, on the other hand, represent the fully oxidised organic matter, and such waters may not be harmful. However, the presence of too much of nitrate in water may adversely affect the health of infants, causing a disease technically called ***"methemoglobinemia"*** (commonly called **"blue baby disease"**). Children suffering from this disease may vomit, their skin colour may become dark, and may die in extreme cases. The nitrate concentration in domestic water supplies is generally limited to 45 mg/l. The amount of nitrites or nitrates, present in a water sample, can be measured by colour matching methods. For nitrites, the colour is developed by adding sulphonic acid and napthamine ; whereas for nitrates, the colour is developed by adding phenol-di-sulphonic acid and potassium hydroxide. The colour developed in water is finally compared with the standard colours of known concentrations.

Excess presence of nitrogen of any kind indicates pollution, and the form in which it is present indicates the period of this pollution. In olden days, when bacteriological tests of water were not properly developed, the nitrogen tests were of considerable importance, as they were the only methods for detecting pollution caused by sewage or organic waste, and thus indicating the presence or absence of bacteria and other such pollutants. But in modern times, these test are taken only as indicative or as inference tests, and the bacteriological tests are taken as the real confirmatory tests in this respect. Besides serving as inference tests, they also reveal the history of water pollution, if has taken place at any time.

(*vi*) **Metals and other Chemical substances.** Tests are carried out in order to determine the amounts of various metals and other substances, such as iron, manganese, copper, lead, barium, cadmium, arsenic, selenium, fluorine, etc., present in water. The amounts of these minerals or metals in potable waters are then limited to their permissible standard values (Refer Tables 8.8 to 8.10), so as to avoid their harmful effects on human life.

The type of harmful effects that are caused by various elements and compounds when consumed in excess of their permissible values are summarised in Table 8.2 and also indicated in Table 8.10.

*thus removing free ammonia.

Table 8.2. Summary of Harmful Effects Caused by Various Toxic Inorganic Elements and Compounds present in water, when consumed in excess of their specified values

S. No.	*Inorganic contaminant*	*Max. permissible concentration in potable waters as per WHO guide-lines in mg/L*	*Max. permissible concentration as per BIS standards (IS 10500 : 1991) in mg/L*	*Harmful health effects*
(1)	(2)	(3)	(4)	(5)
1.	*Antimony, Sb*	0.005	Not specified (N.S.)	Antimony (Sb) may accumulate in liver and is detrimental to the heart in humans. Can also be accumulated by marine organisms.
2.	**Arsenic, As**	0.01	0.01	May cause cancer, diabetes, and liver damage. Also increases risk of hypertension. Causes acute or chronic arsenic toxicity (called ***arsenicosis***) to humans. Toxic to all life. Contributed as by-product of smelting ores and used in other industries. Ground waters are often found to contain arsenic in specific areas such as in West Bengal-Bangladesh belt.
3.	*Barium, Ba*	0.70	N.S.	Ingested Ba salts are highly toxic to humans. Usually found in trace amounts in natural waters but surface water concentrations are sometimes as high as 0.34 mg/L. May be toxic to plants if present above trace amounts.
4.	*Beryllium, Be*	N.S.	N.S.	Extremely toxic to all life. Usually, naturally present in concentrations less than 0.0001 gm/L in surface waters. Oxides and hydro-xides are insoluble within normal pH ranges.
5.	*Boron, B*	0.3	1.0	No evidence of accumulation in humans. Large amounts may produce digestive difficulties and nerve disorders.
6.	**Cadmium, Cd**	0.003	0.01	Cumulative, highly toxic in humans and livestock. Affects all life. Protects other metals against oxidation ; also used in other industries.

Contd...

(1)	(2)	(3)	(4)	(5)
7.	*Chromium, Cr*	0.05	0.05	Natural Cr is rare. Cr (VI) is the toxic form to humans. Cr (III) is slowly oxidized to Cr (VI) in waters. Toxic to plants. Varying tolerance to Cr salts in aquatic life.
8.	*Cobalt, Co*	NS	NS	Low toxicity to humans, and essential in trace amounts.
9.	*Copper, Cu*	N.S.	0.05	Essential to humans in small daily amounts (totalling 2.0 mg). Higher quantities may, however affect human lungs and respiratory organs.
10.	*Cyanide, CN^-*	0.07	0.05	Cyanide renders tissues incapable of oxygen exchange. It is not cumulative and it is biodegradable in streams. CN^- behaves like halides.
11.	*Fluoride, F*	1.5	1.0	Consumption of higher concentration of fluorides exceeding 1.5 mg/s may cause mottling of teeth and ***fluorosis.*** Excessive consumption over long periods may even cause *bone problems,* including *crippling of bones.* Lower concentrations of fluorides, lesser than 1 mg/l or so in waters may cause ***dental caries*** (tooth decay due to cavity formation) ***Optimum concentrations are 0.7 to 1.2 mg/l.***
12.	*Lead, Pb*	0.01	0.05	Cumulative in humans and livestock. Although human absorption of ingested lead is small; yet larger exposures may cause seizures, mental retardation and behavioural disorders. Fetuses, infants, and children are specially susceptible even to low doses.
13.	*Lithium, Li*	N.S.	N.S.	Higher concentrations cause phytotoxicity.
14.	*Mercury, Hg*	0.001	0.001	Toxic to all forms of life. Mercury is very slowly excreted from the human body. Methyl mercury is 50 times more toxic than inorganic mercury.

Contd...

(1)	(2)	(3)	(4)	(5)
15.	*Molybdenum, Mo*	N.S.	N.S.	Essential micronutrient. Does not accumulate and humans can tolerate large quantities. Plants accumulate Mo in their foliage, but at normal concentrations in water it is not harmful. Some grazing animals, *e.g.*, cattle, sheep, and swine, exhibit sensitivity to this metal.
16.	*Nickel, Ni*	0.02	N.S.	Low oral toxicity to humans; but toxic to plants and marine life.
17.	*Ammonia, NH_3*	N.S.	N.S.	Non toxic to humans at natural levels. Fish cannot tolerate large quantitites of ammonia.
18.	*Nitrates NO_3^-*	11.3	45.0	Toxic to infants at high concentrations, causing "*methemoglobinemia*" popularly called the **"blue baby disease".** Children suffering from this disease may vomit, their skin colour may become dark, and may die in extreme cases.
19.	*Nitrites, NO_2^-*	0.91	N.S.	More toxic than nitrates but less chemically stable than nitrates and generally found in low concentrations.
20.	*Phenolic compounds as phenol* (C_6H_5OH)	N.S.	0.001	They impart bad taste in water, and direct toxicity to fish.
21.	*Selenium, Se*	0.01	0.01	Cumulative poison in humans and animals. Moderately toxic to plants.
22.	*Silver, Ag*	N.S.	N.S.	Cumulative in human tissue resulting in blue-gray discoloration of skin (***argyria***). Toxic to aquatic organisms.
23.	*Sodium, Na*	N.S.	N.S.	Harmful to some persons with cardiac problems. Destructive to soils.
24.	*Thallium, Tl*	N.S.	N.S.	Cumulative poison with sublethal effects such as hair loss and hypertension. Thallium has been shown to inhibit photosynthesis and acts as a neuropoison to fish and aquatic invertebrates.

Contd...

(1)	(2)	(3)	(4)	(5)
25.	*Zinc, Zn*	N.S.	N.S.	Relatively non toxic to humans and animals. Essential nutrient for life. Only at high concentrations has it been found toxic to plants. However, zinc is acutely and chronically toxic to aquatic organisms.

The amount of most of the metals and other substances present in water can be estimated by using ***"colour matching methods"*** by using different indicators. The important metals and the indicators used, are given in Table 8.3. However, the determination of such minerals, when present in water, is the job of chemists, who are generally employed at the treatment plants, and too much detailed study of this topic, therefore, is beyond the scope of this book.

Table 8.3. Commonly Used Indicators

Metal to be measured	*Commonly used indicators*
(1) Iron	1, 10 Phenanthroline
(2) Manganese	Ammonium persulphate + Nitric acid
(3) Fluoride	Zirconium—alizarin

(*vii*) **Dissolved Gases.** The various gases which may get dissolved in water due to its contact with the atmosphere or the ground surface may be nitrogen, methane, hydrogen sulphide, carbon dioxide, and oxygen. The contents of these dissolved gases in water may be worked out, so as to get the idea of the extent of contamination of the water and the treatment required for purifying the same.

The nitrogen gas is not very important. The concentration of methane gas is be studied for its explosive tendency. The hydrogen sulphide gas (H_2S), if present even in small amounts, may give bad taste and odour to the water, and should, therefore, be removed. Presence of carbon dioxide gas indicates biological activity. It may impart bad taste and odour to the water and may also cause corrosion. Oxygen gas is generally absorbed by water from the atmosphere, but is being consumed by unstable organic matter for their oxidation. Hence, if the oxygen present in water is found to be less than its saturation level, it indicates presence of organic matter and consequently making the waters suspicious. The simple ***test which can be performed to determine this oxygen deficiency*** in water is to mix the water sample with 10% acid solution of potassium permanganate and to expose the mixture to atmosphere for 4 hours at 27°C. The quantity of oxygen absorbed can then be calculated by titration. This amount for potable waters should not exceed 5 to 10 ppm.

The extent of organic matter present in water sample can also be easily estimated by supplying oxygen to this sample and finding the oxygen consumed by the organic matter present in water. This oxygen demand is

known as ***Biochemical Oxygen Demand (BOD)*** and is tested for raw waters as well as for treated waters. The BOD of raw water will indicate the extent of organic matter present, thus indicating the extent of treatment required for purifying this water to make it safe and wholesome. The BOD of treated water should be *nil*, so as to make it free from any organic matter. BOD and its testing is explained below :

Biochemical oxygen demand. If sufficient oxygen is present in water, the useful aerobic bacteria* production will flourish and cause the biological decomposition of waste and organic matter, thereby reducing the carbonaceous material from the water. The amount of oxygen required in the process until oxidation gets completed is known as BOD. Polluted waters will continue to absorb oxygen for many months, till the oxidation gets completed, and it is not practically possible to determine this ultimate oxygen demand. Hence, the BOD of water during the first five days at 20°C is generally taken as the standard demand. This is determined by mixing or diluting a known volume of a sample of raw water with a known-volume of pure water saturated with a known quantity of dissolved oxygen (in mg/l) and incubating it for a period of five days at 20°C. The dissolved oxygen is measured after the period of incubation. The difference between the original oxygen content and the residual oxygen content will indicate the oxygen consumed by the water sample in five days (used in oxidising the organic matter present in polluted water sample). The BOD_5 in ppm is then worked out by using the equation :

$$BOD_5 = \text{BOD of 5 days} = \text{Loss of oxygen in mg/l} \times \text{Dilution factor or Dilution ratio}$$

This test is known as the ***dilution method.*** An alternative test by electrolysis is being developed these days.

If BOD of water is zero, it means that no oxygen is required and thus no organic matter is present. Hence the BOD of safe drinking water must be *Nil*. The extent of pollution of sewage and other industrial waste waters is also measured by determining the values of their BOD.

8.2.3. Bacterial and Microscopical Characteristics

8.2.3.1. Bacteria and other microorganisms. Bacteria are the minute single cell organisms possessing no defined nucleous and having no green material (*chlorophyll*) to help them manufacture their own food. They are reproduced by binary fission and may be of various shapes**. They are

*Explained in next article.

**Such as (*i*) spherical shape : called *cocus* (when single), *diplococus* (when in pairs), *streptococus* (when in chains), or *staphylococus* (when in irregular branches) ; (*ii*) straight rod like shape with square or rounded ends, called *bacillius* ; (*iii*) spiral shape called *spirillum* ; and (*iv*) curved shape called *vibrio* (See Fig. 8.5).

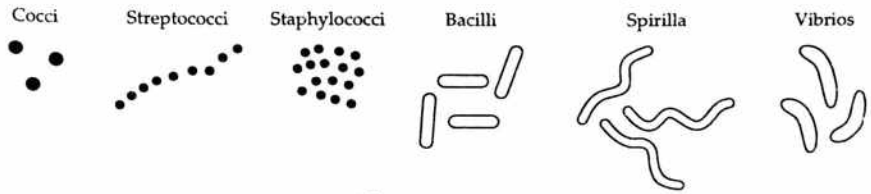


Fig. 8.5. Various types of bacteria.

generally present in raw or contaminated waters. Most bacteria range in size from 0.2 to 2–5 μm in width or diameter, and upto 1—10 μm* in length for non spherical species. The largest known bacteriam, however, is *Thiomargarita namibiensis*, with spheroidal diameters from 100 to 750 μm. Spherical bacteria as small as 50–500 nm (nano metre = 10^{-9} m) in diameter have, however, been reported. They are so small that they cannot be seen with a naked eye and have to be examined under a microscope. Certain other organisms such as viruses** are much smaller and cannot be seen singly even with a microscope. Their presence can, however, be detected with difficulty only by observing their reactions to different stains or cultures.

Most bacteria are harmless, and under certain conditions beneficial to human beings, animals and crops. Such bacteria or micro organisms are called **non-pathogenic bacteria** or ***non-pathogens.*** However, certain other bacteria are the deadly foes of man and animals and may enter their tissues, causing serious water borne diseases, such as cholera, typhoid, infectious hepatitis, etc. Such harmful bacteria or organisms are known as **pathogenic bacteria** or ***pathogens.***

The rain water, as it falls down through the atmosphere, collects bacteria and other types of parasitic organisms***, such as viruses, etc., from the dry dust or smog present in the air or the atmosphere. Since the initial rains wash away most of these dusts, they are likely to be highly contaminated with bacteria, virus and other organisms. The number of bacteria in a teaspoon of rain water may vary from about 5 to about 225. Water falling as snow is likely to contain more bacteria than the water falling as rain, because a snow-flake has a larger surface than a rain drop and, therefore, picks up more dust.

Most of the bacteria present in water require oxygen for their survival. These bacteria consume dissolved oxygen from the water and decompose the refuse and organic matter present in water, thus doing their cleaning-up job. Besides these bacteria which survive on oxygen, certain other types of bacteria which can survive with or without oxygen, are also sometimes present. Depending upon the oxygen necessities of the bacteria, they may be classified as follows :

(*i*) *Aerobic bacteria* — *i.e.* those which require oxygen for their survival.

(*ii*) *Anaerobic bacteria* — *i.e.* those which flourish and thrive in the absence of free oxygen.

(*iii*) *Facultative bacteria* — *i.e.* those which can survive with or without free oxygen.

*1 micron (μ) or 1 μm (micron metre) = 10^{-6} metre = 0.0004 inches ; and
1 milli micron (mμ) = 10^{-9} metre

Viruses are the group of infections or agents which are smaller than ordinary bacteria and are difficult to study. They can be made ineffective by treating the water with chlorine. Most viruses range in size from 5 to 300 nm (nanometre), although some may be as large as 14000 nm (*i.e.* 14 μm).

Five types of **parasitic organisms** (*viz.* bacteria, protozoa, viruses, worms and fungi) are generally known to be infective to man and are found in water. *Bacteria* have been defined earlier. *Protozoa* are single cell animals and are the lowest and the simplest form of animal life. They are bacteria eaters and thus destroy pathogens. However, they form scum and unsightly deposits on porcelain utensils and bath room fittings. They can be directly counted under a microscope. *Viruses* have been defined in the previous para. *Worms* are the larvae of flies ; red blood worms are the larvae of *chironomus* flies, and others are larvae of smaller but allied midgeflies. *Fungi* have been defined at the end of this article. ***The pathogenic bacteria which may be found in wastewaters or contaminated raw waters are given in table 8.4.

Table 8.4. Pathogenic Organisms Commonly found in Raw Contaminated Waters (Waste Waters)

Organism	*Disease*	*Remarks*
(1)	(2)	(3)
Ascaris spp. *Enterobius* spp.	Nematode worms	Danger to man from waste water effluents and dried sludge used as fertilizer
Bacillus anthracis	Anthrax	Found in waste water. Spores are resistant to treatment
Brucella spp.	Brucellosis. Malta fever in man. Contagious abortion in sheep, goats,, and cattle	Normally transmitted by infected milk or by contact. Waste water is also suspected
Entamoeba histolytica	Dysentery	Spread by contaminated waters and sludge used as fertilizer. Common in hot climates
Leptospira iceterohae-morrhagiae	Leptospirosis (Weil's disease)	Carried by sewer rats
Mycobacterium tuberculosis	Tuberculosis	Isolated from waste water and polluted streams. Waste water is a possible mode of transmission. Care must be taken with waste water and sludge from sanatoriums
Salmonella paratyphi	Paratyphoid fever	Common in waste water and effluents in times of epidemics
Salmonella typhi	Typhoid fever	Common in waste water and effluents in times of epidermics
Salmonella spp.	Food poisoning	Common in waste water and effluents
Schistosoma spp.	Schistosomiasis	Probably killed by efficient waste water treatment
Shigella spp.	Bacillary dysentery	Polluted waters are main source of infection
Taenia spp.	Tapeworms	Eggs very resistant, present in waste water sludge and waste water effluents. Danger to cattle on land irrigated or land manured with sludge
Vibrio cholerae	Cholera	Transmitted by waste water and polluted waters
Virus	Poliomyelitis, hepatitis	Exact mode of transmission not yet known. Found in effluents from biological waste water-treatment plants

Although many bacteria are useful and helpful in cleaning natural raw waters, certain other bacteria, often found in polluted waters, produce fatal

diseases. In modern days, these harmful **pathogenic bacteria*** (Table 8.4) can be tested and counted in the laboratories, but only with a great difficulty and with lengthy and complicated tests. These tests are, therefore, generally not performed in routine check up of the water quality. The usual routine tests are however generally conducted only to detect and count the presence of **non-pathogenic indicator organisms**, principally the *coliform group* (*total coliforms* and *fecal coliforms*).

The **coliforms** are the rod-shaped non-pathogenic bacteria (bacilli), whose presence or absence in water indicates presence or absence of fecal pollution and hence of the pathogens.

The coliform group, fecal and total, are the most widely used indicator organisms now and historically. The total coliform group consists of members whose normal habitat is the colon (lower portion of intestines) of humans and warm and cold-blooded animals ; some members are however found in the soil and vegetation as well as in feces. The members which are exclusively found in the feces and which are not found in the soil and vegetation constitute about 96% of all the coliforms of the human feces. Such members are called **fecal coliforms**. They have also recently been named by WHO, as ***thermotolerant coliforms.***

As a matter of fact, the feces of man and animals not only contain coliform group of bacteria, but also contain some other types of bacteria, such as *fecal streptococci* (called *enterococci*), *clostridium perfringens, bacteroides, lactobacilli*, etc., as shown in table 8.5.

Table 8.5. Microbial Flora of human and Animal Feces*

Animal group	*Average density × 10^{-3} per gram*				
	Fecal coliforms	*Fecal strepto-cocci*	*Clostri-dium perfringens*	*Bacter-oides*	*Lacto-bacilli*
Farm animals					
Cow	230	1300	0.2	**	0.25
Pig	3300	84000	3.98	500	251000
Sheep	16000	38000	199	**	79
Horse	12.6	6300	**	**	10000
Duck	33000	54000	—	—	—
Chicken	1300	3400	0.25	**	316000
Turkey	290	2800	—	—	—
Pet animals					
Cat	7900	27000	25100	795000	63000
Dog	23000	980000	251000	500000	39.6
Wild animals					
Mice	330	7700	**	795000	1260000
Rabbits	0.020	47	**	396000	**
Chipmunk	148	6000	—	—	—
Humans	13000	3000	1.58	5000000	630000

*From Geldreich (1978).

**Present in negligible amounts.*

*First of all discovered by Louis Pasteur, (a Frenchman) and Robert Koca (a German) during 18th century.

The coliform bacilli are essentially *lactose fermenters*[1], *non-spore forming*[2], and *gram negative*[3]. The technical definition of this coliform group is : *all of the aerobic and facultative anaerobic gram-negative non-spore forming, rod-shaped bacteria that ferment lactose with gas formation within 48 h at 35°C.* They are *non acid fast*[4] organisms.

The **total coliform group** is widely used as the indicator organism of choice for drinking waters. The major criticism of this group as an indicator is that some of its members (such as *Enterobacter aerogenes*) are widely distributed in the environment. However, when it is assumed that all of its members originated in feces, a safety factor gets provided. This approach is reasonable when applied to provision of safe drinking water but suffers when fecal contamination of a water is suspected.

The **fecal coliform sub group** of the total coliforms is a much more specific indicator of the fecal contamination. **Escherichia Coli (E-Coli)** is the predominant member of the fecal coliform group. **Klebsiella** is another significant member but of minor presence.

The E-Coli, as stated above, is the most abundant coliform, and is further subdivided into *biotype* and *serotype*. Unlike other coliforms, E-Coli is a parasite living only in the human or animal intestines. Voided in feces, it remains visible in the environment only for some days. ***Detection of E-Coli in drinking water, therefore, is taken as evidence of recent pollution with human or animal feces.***

[1]Colan bacilli were first described by Mr. Escherich in 1885, and were called as Bacterium Coli Commune (B-Coli). These colan bacilli not only include the **lactose fermenters** (coliforms) but also include the *late lactose fermenters* (Pathogens like Shigella Sonnei and Paracolons) as well as *non-lactose fermenters* (Pathogens like Salmonella and Shigella).

[2]Some bacteria are able to form *spores* (endospores or free spores) in adverse circumstances. A **spore** is a non-vegetative state that bears little resemblance to the living cell. The protoplasm is covered by many coatings, making it heat resistant and resistant to lack of moisture. Upon finding itself in an agreeable environment, the spore reverts to a vegetative cell. Spores are remarkably heat resistant, which has implications for sterilization. The structure of a spore is much more complex.

[3]**Gram negative bacteria** are those bacteria which leave a **red stain** in a staining technique, which is performed to identify cell components, and called the *gram stain* technique. This technique involves two dyes, iodine and a solvent. The primary dye is crystal violet (purplish colour) and the secondary dye is safranin (red colour). The stain that remains is related to the cell wall characteristics of the bacteria cells. Those cells that are found to be **blue** after the procedure is completed are termed as **gram positive**, while those that are found to be ***red*** upon completion are termed as ***gram negative.*** Most cells of living organisms are gram positive. This is the first test performed on bacteria identification sequence.

[4]Acid fast stain is another important staining technique. An organism is called **acid fast** if it resists de-staining with acids or alcohol, following staining with hot carbol-fuchsin. Other stains also exist to differentiate cell components and inclusions.

Since the harmless organisms of the coliform group live longer in water than the pathogens, and will be in much more number than the pathogens in any waste water, it is generally presumed that the water will be safe and free from pathogenic bacteria, if no (or very little) coliforms are present in it.*

The simplest and the most recent **method adopted for detecting and measuring the presence of coliform bacteria** is to filter the water sample through a sterile membrane of special design (*i.e.* porosity 80%, pore size 5 to 10 mμ), on which the bacteria will be retained, if at all present.

The filter is then rinsed with a sterile buffer solution, placed upon a pad saturated with a suitable nutrient medium, and incubated at an appropriate temperature. The bacteria which are able to grow upon the nutrient medium will produce visible colonies which can be counted, each colony representing one bacterium in the original sample. This incubation process is known as ***culturing through the culture medium***. The medium to be used and the temperature of incubation depend upon the type of bacteria which are to be counted. For *coliform bacteria i.e.* for taking *total coliforms count, M-Endo broth* or *LES Endo Agar* are used with incubation at 35°C for 20 to 22 hours. Coliform bacteria will produce characteristic colonies, which are pink to dark red with golden metallic sheen, often with greenish tint. Non-coliform bacteria which grow in this medium will lack the characteristic sheen. For *fecal coliforms* count, the medium to be used is *M-FC broth* and incubation is done at 44.5°C for 22 hours. The coliform colonies here are blue, and the other bacteria which grow upon this medium are grey to cream coloured. Similarly, to determine the *fecal streptococci* count, the incubation is done on *M-Enterococcus Agar* or *KF-streptococcus* for 48 hours at 35°C. The colonies in this case are light pink to dark red with pink margins. This method of directly counting the coliforms is called the **membrane filter technique or membrane filter test.**

Coliform bacteria, as stated earlier, ferment lactose with formation of gas within a max. of 48 hours at 35°C. This property of coliform organisms has also been used since earlier days to detect their presence in the given water sample. The method used is known as **multiple tube fermentation technique.** In this test, a broth containing lactose and other substance (medium), which inhibit non-coliform organisms, is placed in a series of test tubes. 5 such tubes are then inoculated (mixed) with 10 mL of water sample in each tube. Another 5 test tubes are mixed with 1 mL of water sample in each tube ; and another 5 test tubes are inoculated with 0.1 mL of water sample in each tube. These tubes are finally incubated at 35°C for 24 hours, and formation of gas in tubes is noticed. Presence of gas will give a *positive presumptive test* of the presence of coliform organism in the given tube. In case gas is not found, we may allow further incubation for another 24 hours, and, thus, to notice the said presumptive test at the end of 48 hours, which is the maximum time for release of gas by a coliform bacteria. If no such gas is evolved, it will show a negative test, thereby showing the absence of coliform bacteria in the test tube showing non-evolution of gas.

*Pathogens are contributed only by a few diseased animals (including man), while coliforms are contributed by all, and hence exist in much larger number.

The test tubes showing positive presumptive gas test are further subjected to confirmatory test which will eliminate certain bacteria of non sanitary importance. In one of such several procedures used for this *confirmatory test*, a portion from the fermentation tube (tube evolving gas) is transferred to a tube containing brilliant green bile broth. This is incubated for 48 hours, and if gas is formed, it will be a positive confirmatory test for the presence of coliform bacteria. If no gas is formed, it will be a negative test.

Now, after determining the Nos. of positive test tubes (*i.e.* test tubes containing water having coliform bacteria), the statistical methods are used to determine the bacterial density that has the maximum probability with the given set of constants, which represent nothing but the ***most probable number of the coliforms,*** called *MPN* or *MPN index*. Table 8.6 shows the values of MPN or **MPN index** per 100 mL of given water, when testing is done on 15 tubes (5 tubes each, containing 10 mL, 1 mL and 0.1 mL of water, respectively). This table can be easily used after knowing the combination of positive tubes on three sized samples (*i.e.* 10 mL, 1 mL and 0.1 mL). Say for example, if out of 5 tests on 10 mL samples, 2 are positive ; and all the 5 tests on 1 mL samples are negative ; and say 1 test in 0.1 mL category is +ve, then the combination of +ve tests will be 2-0-1. For this combination value in col (6) of **table 8.6**, the MPN value is read out from col. (7) as 7. Hence, the MPN of the given water will be *7 per 100 mL*, which means 7 coliforms per 100 mL of water.

Although the test requires addition of 10 mL of water in first set of 5 tubes each ; 1 mL of water in second set of 5 tubes each ; and 0.1 mL of water in third set of 5 tubes each ; yet many a times, the test is performed with smaller amounts of water samples, such as 1 mL in first set of tubes, 0.1 mL in the second set of tubes, and 0.01 mL in the third set of tubes. In this case, when MPN value is read out from table 8.6, the said value must be multiplied by 10, because volume of water used is 10 times lesser than the specified sample sizes. Similarly, when water sample sizes used in three sets of tubes are respectively 0.1 mL, 0.01 mL and 0.001 mL, then the value of MPN should be obtained by multiplying the MPN value read out from table 8.6 by 100.

When more than three dilutions are employed in a decimal series of dilutions, the results from only three of these are used in computing the MPN. The three dilutions selected are the highest dilutions giving positive results in all five portions tested (no lower dilution giving any negative results) and the two next succeeding higher dilutions.

The common procedure adopted for MPN determination involves the use of (*i*) sterile pipettes calibrated in 0.1 ml increments ; (*ii*) sterile screw-top dilution bottles containing 99 ml of water ; and (*iii*) a rack containing six sets of five lactose broth fermentation tubes, as shown in Fig. 8.6.

Table 8.6. Most Probable Number (MPN) or MPN Index per 100 mL of Water Sample for Various Combinations of Positive Results When Five Tubes are used per Dilution of 10 mL, 1 mL and 0.1 mL

Combinations of Positives 10 mL – 1 mL – 0.1 mL	*MPN or MPN index / 100 mL*	*Combinations of positives 10 mL – 1 mL – 0.1 mL*	*MPN or MPN index / 100 mL*	*Combinations of Positives 10 mL – 1 mL – 0.1 mL*	*MPN or MPN index / 100 mL*	*Combinations of Positives 10 mL – 1 mL – 0.1 mL*	*MPN or MPN index / 100 mL*	*Combinations of positives 10 mL – 1 mL – 0.1 mL*	*MPN or MPN index / 100 mL*	*Combinations of Positives 10 mL – 1 mL – 0.1 mL*	*MPN or MPN index / 100 mL*
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
0-0-0	< 0	1-0-0	2	2-0-0	5	3-0-0	8	4-0-0	13	5-0-0	23
0-0-1	1.8	1-0-1	4	2-0-1	7	3-0-1	11	4-0-1	17	5-0-1	35
0-0-2	3.6	1-0-2	6	2-0-2	9	3-0-2	13	4-0-2	21	5-0-2	43
0-0-3	5.4	1-0-3	8	2-0-3	12	3-0-3	16	4-0-3	25	5-0-3	58
0-0-4	7.2	1-0-4	10	2-0-4	14	3-0-4	20	4-0-4	30	5-0-4	76
0-0-5	9.0	1-0-5	12	2-0-5	16	3-0-5	23	4-0-5	36	5-0-5	95
0-1-0	1.8	1-1-0	4	2-1-0	7	3-1-0	11	4-1-0	17	5-1-0	33
0-1-1	3.6	1-1-1	6	2-1-1	9	3-1-1	14	4-1-1	21	5-1-1	46
0-1-2	5.5	1-1-2	8	2-1-2	12	3-1-2	17	4-1-2	26	5-1-2	64
0-1-3	7.3	1-1-3	10	2-1-3	14	3-1-3	20	4-1-3	31	5-1-3	84
0-1-4	9.1	1-1-4	12	2-1-4	17	3-1-4	23	4-1-4	36	5-1-4	110
0-1-5	11	1-1-5	14	2-1-5	19	3-1-5	27	4-1-5	42	5-1-5	130
0-2-0	3.7	1-2-0	6	2-2-0	9	3-2-0	14	4-2-0	22	5-2-0	49
0-2-1	5.5	1-2-1	8	2-2-1	12	3-2-1	17	4-2-1	26	5-2-1	70
0-2-2	7.4	1-2-2	10	2-2-2	14	3-2-2	20	4-2-2	32	5-2-2	95
0-2-3	9.2	1-2-3	12	2-2-3	17	3-2-3	24	4-2-3	38	5-2-3	120
0-2-4	11	1-2-4	15	2-2-4	19	3-2-4	27	4-2-4	44	5-2-4	150
0-2-5	13	1-2-5	17	2-2-5	22	3-2-5	31	4-2-5	50	5-2-5	180

Contd...

Table 8.6. Continued

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
0-3-0	5.6	1-3-0	8	2-3-0	12	3-3-0	17	4-3-0	27	5-3-0	79
0-3-1	7.4	1-3-1	10	2-3-1	14	3-3-1	21	4-3-1	33	5-3-1	110
0-3-2	9.3	1-3-2	13	2-3-2	17	3-3-2	24	4-3-2	39	5-3-2	140
0-3-3	11	1-3-3	15	2-3-3	20	3-3-3	28	4-3-3	45	5-3-3	180
0-3-4	13	1-3-4	17	2-3-4	22	3-3-4	31	4-3-4	52	5-3-4	210
0-3-5	15	1-3-5	19	2-3-5	25	3-3-5	35	4-3-5	59	5-3-5	250
0-4-0	7.5	1-4-0	11	2-4-0	15	3-4-0	21	4-4-0	34	5-4-0	130
0-4-1	9.4	1-4-1	13	2-4-1	17	3-4-1	24	4-4-1	40	5-4-1	170
0-4-2	11	1-4-2	15	2-4-2	20	3-4-2	28	4-4-2	47	5-4-2	220
0-4-3	13	1-4-3	17	2-4-3	23	3-4-3	32	4-4-3	54	5-4-3	280
0-4-4	15	1-4-4	19	2-4-4	25	3-4-4	36	4-4-4	62	5-4-4	350
0-4-5	17	1-4-5	22	2-4-5	28	3-4-5	40	4-4-5	69	5-4-5	430
0-5-0	9.4	1-5-0	13	2-5-0	17	3-5-0	25	4-5-0	41	5-5-0	240
0-5-1	11	1-5-1	15	2-5-1	20	3-5-1	29	4-5-1	48	5-5-1	350
0-5-2	13	1-5-2	17	2-5-2	23	3-5-2	32	4-5-2	56	5-5-2	540
0-5-3	15	1-5-3	19	2-5-3	26	3-5-3	37	4-5-3	64	5-5-3	920
0-5-4	17	1-5-4	22	2-5-4	29	3-5-4	41	4-5-4	72	5-5-4	1600
0-5-5	19	1-5-5	24	2-5-5	32	3-5-5	45	4-5-5	81	5-5-5	≥ 2400

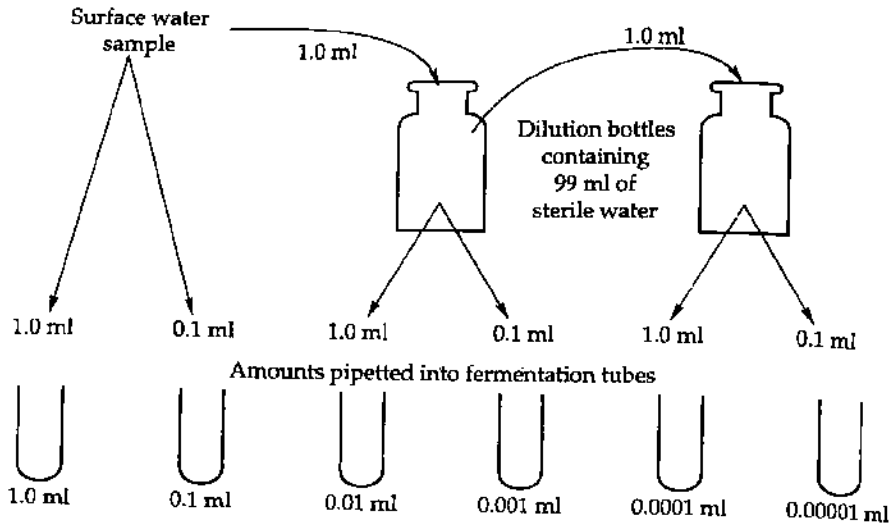


Fig. 8.6. Sketal showing test preparation of the multiple tube fermentation technique for determining MPN of coliform bacteria in a surface water sample.

A sterile pipette is used to transfer 1.0 mL portions of the sample into each of the five fermentation tubes, followed by dispensing 0.1 mL each to a second set of five tubes. For the next dilution, only 0.01 mL of sample water is required, which cannot be taken by the pipette accurately. Therefore 1.0 mL of sample is placed in a dilution bottle (first dilution bottle) containing 99 mL of sterile water and mixed. Now, 1 mL portion of this mixed water from the dilution bottle (*containing 0.01 mL of surface water sample*) can be easily pipetted into the third set of five tubes. The fourth set of tubes receives 0.1 mL from the same dilution bottle, so as to give 0.001 mL of surface water into the 4th set of tubes.

The process is then carried one more step by transferring 1.0 mL from the first dilution bottle into 99 mL of sterile water contained in the second dilution bottle, to provide another 100 fold dilution. The water in the second dilution bottle will thus contain $\frac{0.01\text{ mL}}{100}$ = 0.0001 mL of surface water. 1 mL and 0.1 mL portions from this second bottle are finally pipetted into the fifth and sixth tube sets, respectively, as to provide samples containing 0.0001 mL and 0.00001 mL of surface water. After incubation for 48 hrs at 35°C, the tubes are examined for gas production and the number of positive reactions for each of the serial dilutions are recorded and tabulated in col (3) in table 8.7. The results so obtained shall be used to compute MPN with the help of Table 8.6. This test is known as the **Presumptive MPN Test.**

The presumptive MPN test can be extended to perform *Confirmatory MPN test*, and *Fecal coliform MPN test*, as indicated below :

For performing **confirmatory coliform test**, the growth from each of the positive tubes of the first test (Presumptive MPN test) are transferred aseptically using a wire loop to *brilliant green bile lactose broth, and incubated for 48 hrs at 35°C*. Evolution of gas will confirm the presence of coliform group.

For performing **fecal coliform presence test**, the growth from each of the positive tubes of the first test (Presumptive MPN test) are transferred aseptically using a wire loop to *E.C. medium broth, and incubated for 24 hrs at 44.5°C*. Evolution of gas in this *elevated temperature test* will show the presence of fecal coliform group.

The entire schematic diagram for performing the presumptive MPN test, and for conforming the coliforms, and to determine the presence of fecal coliform group, is shown in Fig. 8.7.

Table 8.7. Observation Table for Coliform Testing under Multiple Tube Fermentation Technique

Serial dilution i.e. set no. of tubes	*Sample portion of surface water (mL)*	*Number of positive reactions out of five tubes*		
		Lactose broth for presumptive test	*Billiant green bile for confirmatory test*	*E.C. medium for fecal coliform test*
(1)	(2)	(3)	(4)	(5)
1	1.0	5	5	5
2	0.1	5	5	4
3	0.01	5	3	1
4	0.001	1	1	1
5	0.0001	0	0	0
6	0.00001	0	0	0

Analysing the Observations Recorded in Table 8.7

After completing the test, the observed data recorded in table 8.7 shall be used to compute *Presumptive Coliform MPN, Confirmed Coliform MPN, and Fecal Coliform MPN,* as given below :

(*a*) **Presumptive MPN Test.** When more than three dilutions are employed in a decimal series of dilutions (as in the present case), we have to use the results from only three of these for computing MPN. The three dilutions to be selected are the highest dilutions giving positive results in all five portions tested, and the two next succeeding higher dilutions.

We can hence choose the first three dilutions, where combination of positive tube is 5-5-5. But for this combination, MPN value (Table 8.6) is not defined properly (result given is >2400); and hence we should choose the second lower dilutions of 0.1 mL, 0.01 mL and 0.001 mL, where combination of +ve tubes is 5-5-1. For this combination, MPN value from Table 8.6 is read out as 350. This value must be multiplied by 100, since surface water volume used are 100 times lesser than the standard values of 10 mL, 1 mL and 0.01 mL.

Hence, *Presumptive Coliform MPN* = **35,000. Ans.**

(*b*) **Confirmed Coliform MPN.** From Table 8.7, col. (2) and (4), we can select the top three dilutions of 1.0 mL, 0.1 mL and 0.01 mL, giving 5-5-3 combination of +ve tubes. For this combination, the value of MPN is read out from Table 8.6 as 920. This value has to be multiplied by 10, since surface water volumes used are 10 times lesser than those of standard test. Hence,

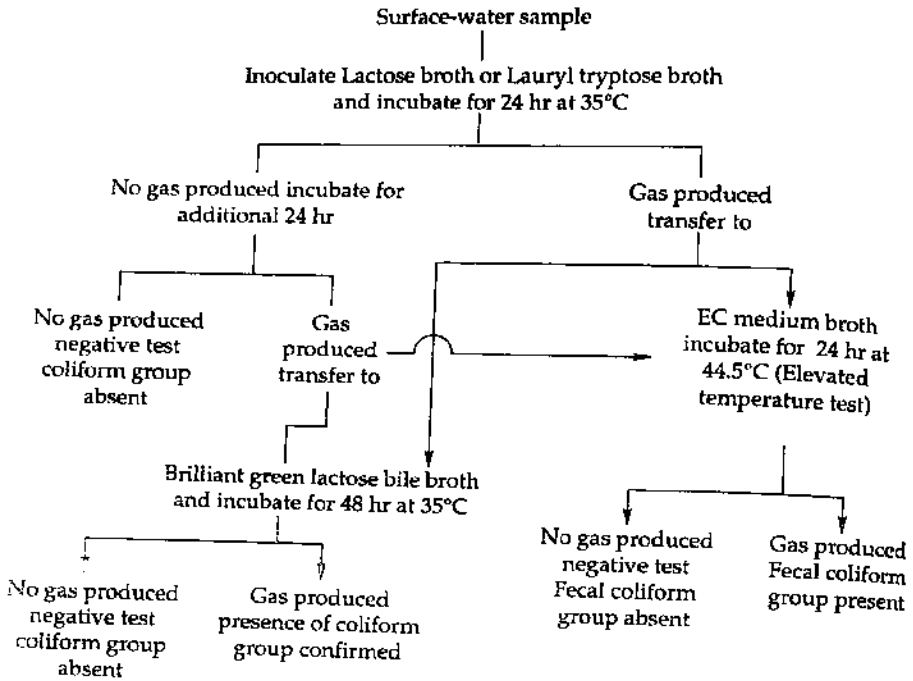


Fig. 8.7. Schematic diagram for performing presumptive coliform MPN test, confirmed coliform MPN test, and the fecal coliforms test in water quality monitoring.

MPN for the test will be **9200**. The second set of dilutions which can be chosen may be 0.1 mL, 0.01 mL and 0.001 mL, where combination of +ve tubes is 5-3-1. For this combination, MPN value from Table 8.6 is read out as 110. This value has to be multiplied by 100 to get MPN test value of 11,000. Since this choice of serial dilutions gives higher value of MPN, it can be selected as the confirmed coliform MPN.

Hence, *confirmed coliform MPN* = **11,000. Ans.**

(*c*) **Fecal Coliform MPN.** Here, the first three serial dilutions of 1.0 mL, 0.1 mL and 0.01 mL are to be chosen, giving combination of +ve tubes as 5-4-1. For this combination, MPN value read from table 8.6, is 170, this value is multiplied by 10 to get test MPN value.

Hence, *fecal coliform MPN* = **1700. Ans.**

Coliform index (C.I.) is another term which was lately used to measure the coliform bacteria present in water sample. It may be defined as the reciprocal of the smallest quantity of a sample which would give a positive coliform test. However, with the advent of *membrane-filter technique test* (described earlier), the actual number of coliform bacteria present in any given sample can be found, and hence the terms such as the most *probable number* or *coliform index* have become obsolete.

In order to ensure that the water is safe for drinking and is free from pathogenic bacteria and other such organisms, it is generally stipulated that,

on an average, not more than one coliform organism be present per 100 ml of water. The (membrane filter test) is generally performed on 5 samples, each of 100 ml, and an average of the coliform organism counts is taken, which should not exceed 1. Moreover, the number of coliform organisms per standard sample must not exceed 4 in any of the five samples. These 5 samples, must be collected from different points of the distribution system, and the water samples should be tested immediately* after they are collected, as otherwise, the bacteria may get reproduced by fission (*i.e.*, cell division) and may increase their number, thus giving false results.

8.2.3.2. Microscopic Plants. In addition to the bacteria and other micro-organisms described above, certain microscopic plants, such as algae, plankton, fungi, etc., are also sometimes present in water, as discussed below :

(*i*) **Algae** is a type of plant that grows in water and flourishes in the presence of sunlight. *Seven types of algae* have been distinguished by colour and shape. All of them contain the green substance ***chlorophyll,*** which permits them to utilise sunlight for their growth by conversion of carbon dioxide, water and other simple substances into organic carbon compounds. Due to this, algae flourishes in sunlight, and hence can be best controlled by covering the reservoir, so as to exclude the sunlight from reaching the water body found infected with algae.

Different types of algae can be broadly divided into the three groups :

(*i*) *Diatomaceae group* ;

(*ii*) *Chlorophylceae group* ; and

(*iii*) *Cyanophyceae group*.

Each of these groups consists of numerous varieties of plants. Say for example, under *Diatomaceae group of algae*, we find plant varieties such as *asterionella, synendrapuchella,* etc. ; under *Chlorophyceae group,* the important variety is *volvox* ; and under *Cyanophyceae group,* the most important variety is *anabaena*.

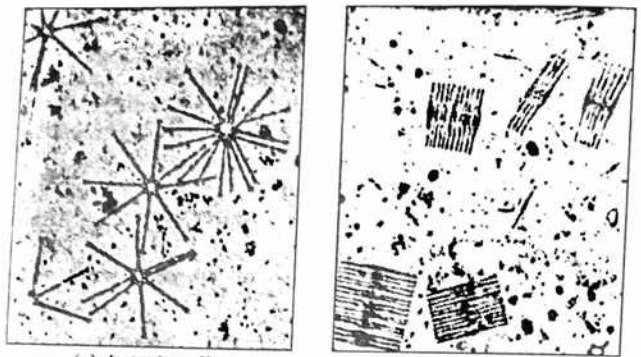
These varieties are exhibited in Fig. 8.8.

(*ii*) **Plankton** are the microscopic plants and animal life that either swim weakly or float in water, and serve as a food for small sea creatures and fish. In most of the cases, they are not injurious to health but they affect the palatability of waters by secreting certain oils, which give bad tastes and odours to the waters. Large quantities of plankton may impart green, brown or yellow hue colour to the water. They sometimes interfere with the smooth working of rapid gravity filters by preventing the proper accumulation of alum flock on sand. Certain types of plankton, if present in water, may cause stains on white porcelain fixtures in kitchens or bathrooms. They may also affect the industries like laundries, dye works, photographic paper, and pump mills, etc., due to colouring tendencies.

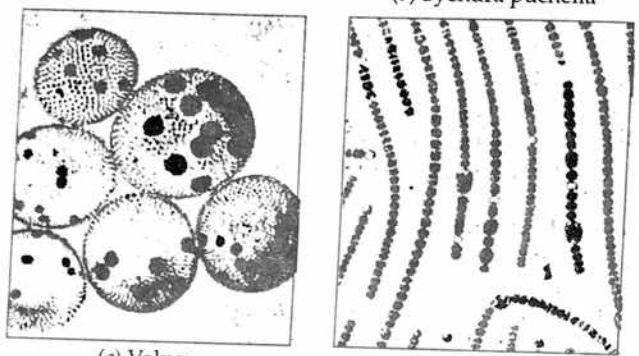
(*iii*) **Fungi** are those plants which grow without sunlight and live on other plants or animals, dead or alive. Some fungi, for example 'toastools' may be fairly large, while other fungi are microscopic**. They may sometimes fest in water-mains and may have to be removed with chlorine treatment.

*within 24 hours.

*which can be seen only under a microscope.



(*a*) Asterionella (*b*) Syendra puchella



(*c*) Volvox (*d*) Anabaena

Fig. 8.8. Types of algae.

8.2.3.3. Collection of water samples for bacteriological testing. The samples of water to be tested for bacteriological and microscopical examination must be collected with great care, as otherwise, the presence of even slight traces of unnoticed pathogens, may spread serious water borne diseases and their epidemics. Special precautions are, therefore, necessary for the collection of water samples, and proper training and experience is essential for testing and evaluating these samples. The testing and examining body may also be consulted for their own specific recommendations before collecting the water samples. In general, the following precautions and procedure must be observed while collecting such samples :

(*i*) Use a *sterile bottle* that will be provided by the laboratory going to examine and test the sample.

(*ii*) Be very careful so that *nothing* except the water to be analysed comes in contact with the inside of the bottle or the cap.

(*iii*) The outside of the tap or faucet at the sample point should be inspected, and if found leaking around the handle, a different sampling point must be chosen.

(*iv*) Clean and dry the outside of the tap or faucet with sterile papers before taking sample.

(*v*) Allow the water to run for at least one half of a minute before collecting the sample.

(*vi*) While filling the bottle, the bottle must be held properly so that no water which contacts the hands enters into the bottle.

(*vii*) The sample must be handed over immediately to the laboratory, as otherwise, extra bacteria may develop (if the delay between collection and examination exceeds 24 hours) by fission, thus giving wrong results.

(*viii*) Sufficient number of samples must be collected from different points of the distribution system, so as to represent a sufficient average value.

8.3. Water Borne Diseases and their Control

Water borne diseases are those diseases which spread *primarily* through contaminated waters ; and the important of these water borne diseases are :

(1) Diseases caused by **bacterial infections** :

(*i*) Typhoid fever and Paratyphoid fever* (caused by *salmonella typhi* bacteria)

(*ii*) Cholera (caused by *vibrio-cholerae* bacteria)

(*iii*) Bacillary dysentery* (caused by *shiga bacillus* or *flexner-bacillus*, or *sonne bacilus*)

(2) Diseases caused by **viral infections** :

(*i*) Infectious hepatitis or infectious jaundice (caused by *hepatitis* virus)

(*ii*) Poliomyelitis (caused by *polio virus*)

(3) Diseases caused by **protozoal infections** :

(*i*) Amoebic dysentery (caused by *entamoeba hystolytic germ*)

All these water borne diseases are infectious diseases in the sense that although they may also spread through direct contact, or through flies or filth, etc. ; yet since water is the main and prime media responsible for the start and spread of these diseases, they are termed as ***water borne diseases.***

The germs or micro-organisms (of different origins as stated above) responsible for spreading these diseases, enter the human body with food or water which has been contaminated by the excreta of patients with the disease, or of carriers of the organisms. Shallow well waters in villages are more likely to get such contaminations due to the leakage of sewage from cesspools, etc. Sometimes, in cities too, the sewer pipe line may leak into a water pipe through some faulty joint, thereby contaminating the same. Since the water pipe lines and sewer lines generally pass through different streets, chances of such contaminations are rare. However, the pollutants can sometimes seep into a leaky water pipe joint, from the dirty surroundings around the water pipe. Sometimes, the water drawn at the waterworks itself may get contaminated due to the inflow of sewage into the source, which may go unnoticed due to the negligence of the waterwork's management. All such situations may contaminate the water and may spread any such water borne disease among its consumers. In hot countries, these diseases may also spread through other means, such as infected dust, or transmission by flies which

*Paratyphoid fever and Bacillary dysentery are far less devastating than the Typhoid fever and the Cholera.

have fed first on the infected excreta and then leaving the infection on human foods or drinks, as pointed out earlier.

Since all these water borne diseases are infectious, the person attending the patient suffering from any of these diseases is liable to be infected and get the disease. He should, therefore, always be careful to thoroughly wash his hands with soap and water every time after attending to the patient and before taking his food or drink. It is also necessary that he takes a preventive inoculation for the particular disease. In order to prevent the spread of water borne diseases and especially in the form of epidemics, the following **preventive or precautionary measures** are recommended :

(1) The water supplies of the town or the city must be thoroughly checked and disinfected before supplying to the public. The supplies of the existing hand pumps or wells in the city, should also be checked and remedial measures taken, where necessary, so as to make them *safe* and *wholesome*.

(2) The water pipe lines should be frequently tested, checked and inspected, so as to detect any leakage and possible source of contamination from nearby surroundings. The leaking joints must be properly sealed.

(3) While laying or designing the water distribution system, attempts should be made as to keep the sewer lines and water lines as far away as possible.

(4) In case of slightest doubts about the supplies, especially in times of rains or floods or in times of low weather flows when surface sources are likely to be most contaminated, people must be forewarned and advised to use boiled water. In such circumstances, an extra chlorine dose must also be added (to serve as disinfectant) to the supplies. The addition of such doses may, no doubt, create bad taste or odour in the supplies, yet it will be better than to infect a serious water borne disease.

(5) The people should also be advised and encouraged to eat hot fresh foods and to avoid raw foods. They should also try to use their own utensils while travelling.

(6) The general habit of cleanliness must be inculcated among the people. People should not be allowed to urinate or defecate as and where desired. Sufficient number of public urinals and latrines should, therefore, be developed, so as to check such tendencies effectively.

(7) The fly nuisance in the city should be checked and reduced to minimum by general cleanliness and by using insecticides.

(8) As soon as some case or cases of such a disease are reported, the people must on a large scale be quickly inoculated and immunised against that disease. They should also be educated to follow the following precautions while handling their relative patients :

(*a*) least or no direct touch with the patient,

(*b*) cleaning and washing of hands with soap after every touch with the patient or his belongings, and also before and after taking any food or drink,

(*c*) making available proper medical care and nutrients to the patient,

(*d*) maintaining proper sanitation around the patient.

8.4. Quality Standards for Municipal or Domestic Supplies

Municipal water required for domestic uses, particularly the water required for drinking, must be colourless, odourless and tasteless. It should be free from turbidity, and excessive or toxic chemical compounds. Harmful micro-organisms and *radio activity** must be absent. The quality of water for municipal supplies is, therefore, generally controlled throughout the world, and even World Health Organisation (W.H.O.) has laid down its standards for potable waters.

The drinking water quality standards as prescribed in U.S.A. by the Environmental Protection Agency (EPA) and included in their National Drinking Water Regulations Act are given in table 8.8.

The WHO's old *European standards for drinking water* (2nd ed—1970) as well as the old *International standards of drinking water* (3rd ed—1971) have since been superceded by the WHO's newly published *Guidelines for drinking water Quality* (1st ed—1984). The 1984 guidelines have also been modified here and there in the latest 1993 edition (2nd ed). These latest guidelines are given in table 8.9. These very old standard guidelines are to be used by various countries for formulation of their own national standards. This may of course lead to the formulation of national standards that may differ appreciably from the prescribed WHO's standard guidelines.

In India, the standards for public water supplies are basically laid down by the recommendations of the *Environment Hygiene Committee* (1949). These very old standard recommendations have further been utilised by the Bureau of Indian Standards in formulating the *Indian Standard Drinking Water Specifications* (1991), which today stand as our national drinking water standards. These standards are given in table 8.10.

When these water standards are not fulfilled, the water may not be 100 per cent fit for drinking, and may be termed as "contaminated". It may not be healthy and tasty, or may cause the spread of some diseases.

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*A possible way of the water getting polluted in the recent atomic age is due to the presence of excessive amounts of radioactive material in it. The raw water falling through an atmosphere contaminated by nuclear tests may increase the content of radioactive materials in raw water. The radioactive materials may also enter water through wash water of plants using radioactive materials. All such possible additions of radioactive materials in water may consequently become more than what the human body can tolerate without being damaged. So for as India is concerned, this problem is still not serious to any extent, as no water has so for been reported to have approached the level of radioactive material considered dangerous to health. But who knows exactly what that danger point is ? A lot of research work has been going on in developed countries like U.S.A. on this topic. On the basis of this research, W.H.O. has now been able to lay down its recommendations, limiting the radioactivity to an amount equal to 0.1 Bq (Bacquerel) per litre of water for α-emitters, and 1 Bq per litre for β-emitters for life time use by large populations.

Table 8.8. U.S. Standards of Drinking Water as of May 1994 (Prescribed by U.S. Environmental Protection Agency)

S. No.	Constituent	Concentrations in mg/l: Maximum contaminant Level Goal MCLG	Concentrations in mg/l: Maximum Contaminant Level (Legally enforceable) MCL
(1)	(2)	(3)	(4)
(1)	**Microbiological Parameters**		
	(*i*) Giardia	0	F.D.*
	(*ii*) Hetrotrophic plate count	—	F.D.
	(*iii*) Legionella	0	F.D.
	(*iv*) *Total coliforms*	0	In P/A (presence or Absence) criteria of coliforms, small systems cannot have more than 1 positive coliform sample per month ; while for large systems, not more than 5% of total coliform samples per month may be positive.
(*v*)	Viruses	0	F.D.
*Various Filtration and Disinfection (F.D.) schemes are specified in U.S. regulations for different water quality conditions.			
(2)	**Inorganic Constituents of Health Significance**		
	(*i*) Antimony	0.006	0.006
	(*ii*) Arsenic	0.0	0.05 based
	(*iii*) Asbestos	7×10^{6}*	7×10^{6}* (* Based on fibres > 10 μm)
	(*iv*) Barium	2	2
	(*v*) Beryllium	0.004	0.004
	(*vi*) Cadmium	0.005	0.005
	(*vii*) Chromium	0.1	0.1
	(*viii*) Cyanide (as CN)	0.2[c]	0.2[c]
	(*ix*) Fluoride	4	4
	(*x*) Lead	0	TT[a] (*a* Treatment Technology specified)
	(*xi*) Mercury (total)	0.002	0.002

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	(*xii*) Nickel	0.1	0.1
	(*xiii*) Nitrate (as N)	10	10
	(*xiv*) Nitrite (as N)	1	1
	(*xv*) Selenium	0.05	0.05
	(*xvi*) Thallium	0.0005	0.002
(2)	**Organic Constituents of Health Significance**		
	Acrylamide	0	TT[a]
	Alachlor	0	0.002
	Aldicarb	0.007[b]	0.007[b]
	Aldicarb sulfone	0.007[b]	0.007[b]
	Aldicarb sulfoxide	0.01	0.01
	Adipate	0.4	0.4
	Atrazine	0.003	0.003
	Benz[*a*]anthracene	0[c]	0.0001[c]
	Benzene	0	0.005
	Benzo[*a*]pyrene	0	0.0002
	Benzo[*b*]fluoranthene	0[c]	0.0002[c]
	Benzo[*k*]fluoranthene	0[c]	0.0002[c]
	Butyl benzyl phthlate	0[c]	0.1[c]
	fluoranthene		
	Carbofuran	0.04	0.04
	Carbon tetrachloride	0	0.005
	Chlordane	0	0.002
	Chloroform		0.08[d]
	Chrysene	0[c]	0.0002[c]
	2, 4, D (2, 4-dichloro-phenoxyacetic acid)	0.07	0.07
	Dalapon	0.2	0.2
	Dibenz[*a, h*]anthra-cene	0[c]	0.0003[c]
	Dichlorobenzene	0.075	0.075
	1, 2-Dichlorobenzene	0.6	0.6
	1, 4-Dichlorobenzene	0.6	0.6
	1, 2-Dibromo-3-chloro-propane (DBCP)	0	0.0002
	1, 2-Dichloroethane	0	0.005
	1, 1-Dichloroethylene	0.007	0.007
		0.07	0.07
	cis-1, 2-Dichloro-ethylene	0.1	0.1

Contd...

Table 8.9. WHO's Latest Standard Guidelines for Potable Waters (1984 guidelines as modified in 1993)

Organism	*Unit*	*Guideline value*	*Remarks*
(1)	(2)	(3)	(4)
(1) **Microbiological Quality**			
A. Piped water supplied			
A.1 Treated water entering the distribution system			
fecal coliforms	number/100 ml	0	Turbidity < 1 NTU ; for disinfection with chlorine, pH preferably < 8.0 ;
coliform organisms	number/100 ml	0	Free chlorine residual 0.2—0.5 mg/litre following 30 minutes (minimum) contact
A.2 Untreated water entering the distribution system			
fecal coliforms	number/100 ml	0	
coliform organisms	number/100 ml	0	In 98% of samples examined throughout the year—in the case of large supplies when sufficient samples are examined
coliform organisms	number/100 ml	3	In an occasional sample, but not in consecutive samples
A.3 Water in the distributionsystem			
fecal coliforms	number/100 ml	0	
coliform organisms	number/100 ml	0	In 95% of samples examined throughout the year—in the case of large supplies when sufficient samples are examined
coliform organisms	number/100 ml	3	In an occasional sample, but not in consecutive samples
B. Unpiped water supplies			
fecal coliforms	number/100 ml	0	
coliform organisms	number/100 ml	10	Should not occur repeatedly ; if occurrence is frequent and if sanitary protection cannot be improved, an alternative source must be found if possible
C. Bottled drinking-water			
fecal coliforms	number/100 ml	0	Source should be free from fecal contamination
coliform organisms	number/100 ml		0

Contd...

Table 8.9. Continued

(1)	(2)	(3)	(4)
D. Emergency water supplies			
fecal coliforms	number/ 100 ml	0	Advise public to boil water in case of failure to meet guideline values
Coliform organisms	number/ 100 ml	0	
Enteroviruses	—	no guideline value set	
(2) **Biological Quality**			
protozoa (pathogenic)	—	no guideline value set	
helminths (pathogenic)	—	no guideline value set	
free-living organisms (algae, others)	—	no guideline value set	
(3) **Inorganic Constituents of Health Significance**			
Antimony	mg/l	0.005	
Arsenic	mg/l	0.01	
Asbestos	—	no guideline value set	
Barium	mg/l	0.7	
Beryllium	—	no guideline value set	
Boron	mg/l	0.3	
Cadmium	mg/l	0.003	
Chromium	mg/l	0.05	
Cyanide as CN	mg/l	0.07	
Fluoride	mg/l	1.5	Natural or deliberately added ; local or climatic conditions may necessitate adaptation
Hardness	—	no health-related guideline value set	
Lead	mg/l	0.01	
Mercury (total)	mg/l	0.001	
Nickel	mg/l	0.02	
Nitrate as N	mg/l	11.3	
Nitrite as N	mg/l	0.91	
Selenium	mg/l	0.01	
Silver	—	no guideline value set	

Contd...

Table 8.9. Continued

(1)	(2)	(3)	(4)
Sodium	—	no guideline value set	
Thallium	—	no guideline value set	
(4) Organic Constituents of Health Significance			
Aldrin and dieldrin	µg/l	0.03	
Benzene	µg/l	10[a]	
Benzo[*a*]pyrene	µg/l	0.01[a]	
Carbon tetrachloride	µg/l	3[a]	Tentative guideline value[b]
Chlordane	µg/l	0.3	
Chlorobenzenes	µg/l	no health-related guideline value set	Odour threshold concentration between 0.1 and 3 µg/l
Chloroform	µg/l	30[a]	Disinfection efficiency must not be compromised when controlling chloroform content
Chlorophenols	µg/l	no health-related guideline value set	Odour threshold concentration 0.1 µg/l
2, 4-D	µg/l	100[c]	
DDT	µg/l	1	
1, 2-dichloroethane	µg/l	10[a]	
1, 1-dichloroethene[d]	µg/l	0.3[a]	
heptachlor and heptachlor epoxide	µg/l	0.1	
hexachlorobenzene	µg/l	0.01[a]	
gamma-HCH (lindane)	µg/l	3	
methoxychlor	µg/l	30	
pentachlorophenol	µg/l	10	
tetrachloroethene[d]	µg/l	10[a]	tentative guideline value[b]
trichloroethene[d]	µg/l	30[a]	tentative guideline value[b]
2, 4, 6-trichlorophenol	µg/l	10[a, c]	odour threshold concentration, 0.1 µg/l
trihalomethanes		no guideline value set	see chloroform

[a] These guideline values were computed from a conservative hypothetical mathematical model which cannot be experimentally verified and values should therefore be interpreted differently. Uncertainties involved may amount to two orders of magnitude (*i.e.*, from 0.1 to 10 times the number).

[b] When the available carcinogenicity data did not support a guideline value, but the compounds were judged to be of importance in drinking-water and guidance was considered essential, a tentative guideline value was set on the basis of the available health-related data.

[c] May be detectable by taste and odour at lower concentrations.

Contd...

Table 8.9. Continued

(1)	(2)	(3)	(4)
[d] These compounds were previously known as 1, 1-dichloroethylene, tetrachloroethylene, and trichloroethylene respectively.			
(5) Parameters controlling Aesthetic Quality of Potable Waters			
Aluminium	mg/l	0.2	
Ammonia	mg/l	1.5	
Chloride	mg/l	250	
Chlorobenzenes and Chlorophenols	—	no guideline value set	These compounds may affect taste and odour
Colour	True colour units (TCU)	15	
Copper	mg/l	1.0	
Detergents	—	no guideline value set	There should not be any foaming or taste and odour problems
Hardness	mg/l (as $CaCO_3$)	500	
Hydrogen sulfide	—	not detectable by consumers	
Iron	mg/l	0.3	
Manganese	mg/l	0.1	
Oxygen—dissolved	—	no guideline value set	
pH		preferably < 8.0	6.5—8.5
Sodium	mg/l	200	
Solids—total dissolved	mg/l	1000	
Sulfate	mg/l	250	
Taste and odour	—	inoffensive to most consumers	
Temperature	—	no guideline value set	
Turbidity	nephelometric turbidity units (NTU)	5	Preferably < 1 for disinfection efficiency
Zinc	mg/l	3.0	
(6) Radioactive Constituents			
Gross alpha activity	Bq/l*	0.1	(*a*) If the levels are exceeded, more detailed radionuclide analysis may be necessary. (*b*) Higher levels do not necessarily imply that the water is unsuitable for human consumption
Gross beta activity	Bq/l*	1	
*1 Bq/l = 0.45 pCi/l			

Table 8.10. Indian Standard Drinking Water Specifications (IS 10500 : 1991)

S. No.	*Substance or Characteristic*	*Requirement (Desirable Limit)*	*Undesirable Effect Outside the Desirable Limit*	*Permissible Limit in the Absence of Alternate Source*	*Remarks*
(1)	(2)	(3)	(4)	(5)	(6)
(1) **Essential Characteristics**					
(*i*)	Colour, Hazen units, *Max*	5	Above 5, consumer acceptance decreases	25	Extended to 25 only if toxic substances are not suspected, and in the absence of alternate sources
(*ii*)	Odour	Unobjectionable	—	—	(*a*) Test cold and when heated (*b*) Test at several dilutions
(*iii*)	Taste	Agreeable	—	—	Test to be conducted only after safety has been established
(*iv*)	Turbidity, NTU, *Max*	5	Above 5, consumer acceptance decreases	10	—
(*v*)	pH value	6.5 to 8.5	Beyond this range, the water will affect the mucous membrane and/or water supply system	No relaxation	—
(*vi*)	Total hardness (as $CaCO_3$) mg/l, *Max*	300	Encrustation in water supply structure and adverse effects on domestic use	600	—
(*vii*)	Iron (as Fe) mg/l, *Max*	0.3	Beyond this limit, taste/appearance are affected, has adverse effect on domestic uses and water supply structures, and promotes iron bacteria	1.0	—
(*viii*)	Chlorides (as Cl) mg/l, *Max*	250	Beyond this limit, taste, corrosion and palatability are affected	1000	—

Contd...

Table 8.10. Continued

S. No.	*Substance or Characteristic*	*Requirement (Desirable Limit)*	*Undesirable Effect Outside the Desirable Limit*	*Permissible Limit in the Absence of Alternate Source*	*Remarks*
(1)	(2)	(3)	(4)	(5)	(6)
(*ix*)	Residual, free chlorine, mg/l, *Min*	0.2	—	—	To be applicable only when water is chlorinated. Tested at consumer end. When protection against viral infection is required, it should be min 0.5 mg/l
(*x*)	Fluoride (as F) mg/l, *Max*	1.0	Fluoride may be kept as low as possible. High fluoride may cause fluorosis	1.5	—
(2) **Desirable Characteristics**					
(*xi*)	Dissolved solids mg/l, *Max*	500	Beyond this, palatability decreases and may cause gastro intestinal irritation	2000	—
(*xii*)*	Magnesium (as Mg), mg/l, *max*	30	Encrustation to water supply structure and adverse effects on domestic use	100	—
(*xiii*)	Calcium (as Ca) mg/l, *Max*	75	Encrustation in water supply structure and adverse effects on domestic use	200	—
(*xiv*)	Copper (as Cu) mg/l, *Max*	0.05	Astringent taste, discoloration and corrosion of pipes, fittings and utensils will be caused beyond this	1.5	—

Contd...

*Introduced vide amendment made in January, 1993.

Table 8.10. Continued

S. No.	Substance or Characteristic	Requirement (Desirable Limit)	Undesirable Effect Outside the Desirable Limit	Permissible Limit in the Absence of Alternate Source	Remarks
(1)	(2)	(3)	(4)	(5)	(6)
(*xv*)	Manganese (as Mn) mg/l, *Max*	0.1	Beyond this limit, taste/appearance are affected, has adverse effect on domestic uses and water supply structures	0.3	—
(*xvi*)	Sulphate (as SO_4) mg/l, *Max*	200	Beyond this, causes gastro intestinal irritation when magnesium or sodium are present	400 (see col 6)	May be extended up to 400 provided magnesium does not exceed 30
(*xvii*)	Nitrate (as NO_3) mg/l, *Max*	45	Beyond this metha-emoglobinemia takes place	100	—
(*xviii*)	Phenolic compounds (as C_6H_5OH) mg/l, *Max*	0.001	Beyond this, it may cause objectionable taste and odour	0.002	—
(*xix*)	Mercury (as Hg) mg/l, *Max*	0.001	Beyond this, the water becomes toxic	No relaxation	To be tested when pollution is suspected
(*xx*)	Cadmium (as Cd), mg/l, *Max*	0.01	Beyond this, the water becomes toxic	No relaxation	To be tested when pollution is suspected
(*xxi*)	Selenium (as Se), mg/l, *Max*	0.01	Beyond this, the water becomes toxic	No relaxation	To be tested when pollution is suspected
(*xxii*)	Arsenic (as As), mg/l, *Max*	0.01*	Beyond this, the water becomes toxic	No relaxation	To be tested when pollution is suspected
(*xxiii*)	Cyanide (as CN), mg/l, *Max*	0.05	Beyond this limit, the water becomes toxic	No relaxation	To be tested when pollution is suspected

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*Amended from 0.05 mg/l (*i.e.* 50 ppb) to 0.01 mg/l (*i.e.* 10 ppb) vide amendment made in Sept. 2003.

Table 8.10. Continued

S. No.	Substance or Characteristic	Requirement (Desirable Limit)	Undesirable Effect Outside the Desirable Limit	Permissible Limit in the Absence of Alternate Source	Remarks
(1)	(2)	(3)	(4)	(5)	(6)
(*xxiv*)	Lead (as Pb), mg/l, *Max*	0.05	Beyond this limit, the water becomes toxic	No relaxation	To be tested when pollution/plumbo-solvency is suspected
(*xxv*)	Zinc (as Zn), mg/l, *Max*	5	Beyond this limit, it can cause astringent taste and an opale-scence in water	15	To be tested when pollution is suspected
(*xxvi*)	Anionic detergents (as MBAS) mg/l, *Max*	0.2	Beyond this limit, it can cause a light froth in water	1.0	To be tested when pollution is suspected
(*xxvii*)	Chromium (as Cr^{6+}) mg/l, *Max*	0.05	May be carcinoge-nic above this limit	No relaxation	To be tested when pollution is suspected
(*xxviii*)	Polynuclear aromatic hydrocarbons (as PAH) g/l, *Max*	—	May be carcinogenic	—	—
(*xxix*)	Mineral oil mg/l, *Max*	0.01	Beyond this limit, undesirable taste and odour after chlorination take place	0.03	To be tested when pollution is suspected
(*xxx*)	Pesticides mg/l, *Max*	Absent	Toxic	0.001	
(*xxxi*)	Radioactive materials : (*a*) Alpha emitters Bq/l, *Max*	—	—	0.1	—
	(*b*) Beta emitters pci/l, *max*	—	—	1	
(*xxxii*)	Alkalinity mg/l, *Max*	200	Beyond this limit, taste becomes unpleasant	600	—

Contd...

Table 8.10. Continued

S. No.	*Substance or Characteristic*	*Requirement (Desirable Limit)*	*Undesirable Effect Outside the Desirable Limit*	*Permissible Limit in the Absence of Alternate Source*	*Remarks*
(1)	(2)	(3)	(4)	(5)	(6)
(*xxxiii*)	Aluminium (as Al), mg/l, *Max*	0.03	Cumulative effect is reported to cause dementia	0.2	—
(*xxxiv*)	Boron, mg/l, *Max*	1	—	5	—

8.5. Quality Standards for Industrial Supplies

Strictly speaking, the quality of water required for industries depends upon the requirements of a particular industry. But in general, it can be stated that about 80 per cent industrial waters are used for cooling and need not be of high quality. The dissolved solids in *cooling water* may, therefore, range from only a few mg/l to 3500 mg/l or so. It need not, also, be free from pollutants either organic or inorganic. It is, however, frequently advantageous to remove the growth of bacteria and other organisms, so as to reduce the fouling of the cooling equipments. Such waters may also sometimes be treated and pH kept not less than 7, so as to reduce or prevent corrosion.

Although the cooling waters required in industries may not be of high standards (as pointed out above), the waters required for *processing* for *boiler feed purposes* must be of *very high quality*.

Boiler feed waters. For example, the boiler feed waters required for raising steam must be soft, as otherwise, the hard waters will cause incrustation on the inside of the boilers, offering resistance to heat transfer and thus reducing their efficiency. Hard waters also cause foaming and priming, and, therefore, the boilers using hard waters do require frequent cleaning and addition of chemicals for treating the hard water. The water required for high pressure boilers (pressure exceeding 7000 kN/m^2 *i.e.* 70 kg/cm^2 or so) should contain very low amounts of dissolved solids (of the order of 0.5 mg/l or so) and dissolved oxygen (preferably less than 0.05 mg/l). Since such high standards can never be met in raw available waters or in public supplies fit for drinking, special suitable treatments are required, which must be given by the industries themselves.

Waters required for processing. Water incorporated into or coming in intimate contact with the manufactured products are subjected to various kinds of quality requirements (depending on the requirements of a particular industry, Refer Table 8.11), which are strictly and rigidly controlled. For example, the waters used in *pulp or paper* industries, must be free from iron, manganese, or hardness. By contrast, water required for *breweries, distilleries*, and *bakeries* should preferably be hard. For *steel rolling mills,* the chloride content of water must be less than what is permitted in drinking

Table 8.11. Average Water Quality Standards for Process waters for Certain Important industries

S. No.	*Industry*	*Turbidity on silica scale*	*Colour on cobalt scale*	*Hardness expressed in mg/l as $CaCO_3$*	*Alkalinity in mg/l*	*pH value*	*Total dissolved solids in mg/l*	*Fe and Mn in mg/l*	*Other properties, and remarks if any*
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
1.	Baking	10	10	...	...	...	...	0.5	Some hardness is desirable ; other properties conforming to standards for potable waters
2.	Brewing								
	(i) light	10	...	...	75	6.5–7	500	0.1	Ca = 100 – 500 ; $CaCO_4$ = 100 – 500 Odour = Low ; other properties confirming to standards of potable waters
	(ii) dark	10	...	...	150	7.0	1000	0.1	
3.	Canning								
	(i) General	10	...	...	...	...	...	0.2	Other properties confirming to standards of potable waters
	(ii) Legumes	10	...	25—75	...	...	...	0.2	-do-
4.	Carbonated beverages	2	10	—	50	...	850	0.3	-do-
5.	Confectionary	...	...	250	...	...	100	0.2	-do-

Table 8.11. Continued

S. No.	Industry	Turbidity on silica scale	Colour on cobalt scale	Hardness expressed in mg/l as $CaCO_3$	Alkalinity in mg/l	pH value	Total dissolved solids in mg/l	Fe and Mn in mg/l	Other properties, and remarks if any
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
6.	Ice making	1—5	...	50	30–50	...	300	0.2	Other properties confirming to standards of potable waters
7.	Laundering	...	...	...	...	...	...	0.2	-do-
8.	Clean plastics	2	2	50	...	...	200	0.02	-do-
9.	Pulp and paper			...					
	(*i*) Ground wood	50	20	180	...	...	...	1.0	-do-
	(*ii*) Kraft	25	15	100	...	...	300	0.2	-do-
	(*iii*) Soda and sulphite	15	10	100	...	...	200	0.1	-do-
	(*iv*) Light paper	5	5	50	...	...	200	0.1	-do-
10.	Rayon (viscose)							0.05	-do-
	(*i*) Production	0.3	5	8	50	...	...	0.2	-do-
	(*ii*) Manufacture	—	—	55	...	7.8—8.3	...	0.2	-do-
11.	Tanning	20	1—100	55—135	133	8.0	...		-do-
12.	Textiles								
	(*i*) General	5	20	50–135	...	...	...	...	-do-
	(*ii*) Dyeing	5	5–20	20	...	...	...	0.25	-do-
	(*iii*) Wool scouring	...	70	20	...	...	...	1.0	-do-
	(*iv*) Cotton bandage	5	5	20	...	...	...	0.2	-do-

water (preferably kept less than 150 mg/l). For producing canned goods such as *milk, meats, beverages, ice, etc.*, the bacteriological quality of water and sanitary requirements even surpass the drinking water standards. The textile industries may require waters which contain low concentrations of dissolved solids and are free from colour, hardness and also from salts such as iron and manganese.

From the above discussion, it becomes very clear that the water quality standards, required for processing different industries may vary with each industry. Under normal circumstances, the drinking water standards are sufficient for processing purposes, but may require further treatment in particular cases. Such treatments are, of course, required for boiler feed purposes. In all such cases, where extra treatments are required, the industries establish their own treatment plants and the municipal public supplies are created only to fulfil the domestic water standards.

PROBLEMS

1. What are the common impurities found in natural sources of water, and explain their effects upon its quality.
2. (*a*) Explain the significance of the following from the point of view of water quality criteria :
   (*i*) Turbidity ;
   (*ii*) Chlorides ;
   (*iii*) Nitrates ;
   (*iv*) Sulphates ; and
   (*v*) Iron.
   (*b*) Explain the significance of E-coil in water analysis.
3. Describe briefly the method of collecting samples of water from different sources for its physical and chemical examination. What special precautions will be required to be observed if biological examination is also to be conducted.
4. Enumerate the common tests that should be carried out in the examination of water at laboratories attached to water treatment plants, and explain the significance of each of them.
5. (*a*) Explain the importance of chemical and bacteriological analysis of water used for domestic purposes.
   (*b*) State the permissible limits for fluorides in water to be supplied for domestic consumption. Mention the ill-effects when they are not in the permissible limits.
6. (*a*) Explain why bacteriological test should be necessary in handing problems of water supply.
   Describe how samples of water should be collected and sent to the laboratory for the above test, indicating the necessary precautions to be observed.
   (*b*) Name one water-borne disease under each of bacterial, viral, and protozoal origin that can be controlled by proper treatment of water.
7. Enumerate the important water-borne diseases in India, and describe how, with your knowledge of various branches of science applicable to sanitary engineering, you would set about to prevent the incidence of these diseases.

8. (*a*) Why are some diseases called water borne ? Which of the following come under this classification :
   (*i*) Typhoid ; (*ii*) Malaria ;
   (*iii*) Jaundice ; (*iv*) Influenza ; and
   (*v*) Cholera.
   (*b*) Name one cause of taste and odour in water supplies.
9. Give the maximum acceptable limits of the following for the public drinking water :
   (*i*) Turbidity ; (*ii*) Colour ;
   (*iii*) Hardness ; and (*iv*) pH.
10. (*a*) What are the maximum acceptable limits of
    (*i*) Turbidity, (*ii*) Fluorides,
    (*iii*) Nitrates, and (*iv*) Phenolic substances, in drinking water.
    (*b*) Which of the following are "water-borne" diseases :
    (*i*) Filaria, (*ii*) Typhoid,
    (*iii*) Chicken-pox, (*iv*) Gastro-enteritis.
11. Enumerate and discuss in brief the various physical, chemical, and bacterial characteristics of testing of raw water supplies. What steps would you take in order to make them fit for drinking.
12. (*a*) Enumerate the various impurities of physical, chemical, micro-organic and radiological origin in raw water supplies, and state their maximum permissible limiting values for obtaining potable municipal supplies.
    (*b*) How does water quality criteria differs for industrial supplies from those for domestic municipal supplies ?
13. "Industries must develop their own water supplies as well as waste treatments, in these modern times". Discuss and debate the statement in the context of Indian conditions.
14. What are B-coli ? Are they harmful to human beings ? Why is their presence tested in water that is supplied for domestic consumption ? Describe briefly the procedure followed in conducting such a test.
15. Write short notes on the following :
    (*i*) pH value of water ; (*ii*) Hardness ;
    (*iii*) B-coli ; (*iv*) M.P.N. ;
    (*v*) Algae ; and (*vi*) Water borne diseases.

—9

Purification of Water Supplies

9.1. General Introduction

The available raw waters must be treated and purified before they can be supplied to the general public for their domestic, industrial or any other uses. The extent of treatment required to be given to a particular water depends upon the characteristics and quality of the available water, and also upon the quality requirements* for the intended use. In general, the public water supplies are mainly designed from the view point of the quality requirements of drinking water, and simultaneously, they should be made reasonably suitable for industrial purposes like steam generation, dyeing, brewing, etc. The available water must, therefore, be made safe, good in appearance, and attractive to human taste and tongue. Various methods which are used to make the water safe and attractive to the consumers are described below. However, the method or the methods adopted for purification depend mostly upon the character of the raw water.

9.2. Methods of Purification of Water

The various methods or the techniques which may be adopted for purifying the public water supplies are :

(*i*) *Screening*
(*ii*) *Plain sedimentation*
(*iii*) *Sedimentation aided with Coagulation*
(*iv*) *Filtration*
(*v*) *Disinfection*
(*vi*) *Aeration*
(*vii*) *Softening*
(*viii*) *Miscellaneous treatments*, such as *fluoridation, recarbonation liming, desalination, etc.*

As pointed out earlier, it may please be noted and kept in mind that all the above techniques may or may not be used for treating a particular water. The necessity of a few or all of these treatments depends solely upon the quality of the available raw water. Before we describe these techniques in details, let us summarise the functions served by each of these techniques in effecting purification of raw water.

Most of the big and visible objects, such as trees, branches, sticks, vegetations, fish, animal life, etc., present in raw waters of surface sources can be removed by *screening*. The coarser suspended materials can then be

*Discussed thoroughly in the previous chapter.

removed by letting the water settle in *sedimentation* basins. The process is called *plain sedimentation*. The effectiveness of sedimentation may however, be increased by mixing certain chemicals with the water, so as to form flocculent precipitate, which carries the suspended particles as it settles. The process is called *chemical coagulation*. The finer particles in suspension, which may avoid settling in sedimentation basins even after using chemical coagulation, may then be removed by filtering the water through filters*. The process is called *filtration*. The filtered water which may still contain pathogenic bacteria, is then made bacteria-proof by adding certain chemicals such as chlorine, etc. This process of killing of germs is called *disinfection*. The resulting water, though now becomes safe, yet may not be attractive to the tongue of the consumers. Unpleasant tastes and odours may then, therefore, have to be removed by adding certain chemical compounds such as activated carbon or by using ozone. This process is called *aeration*. The resulting water may sometimes be much *harder*** than permissible and may, therefore, have to be softened by a process called *softening*. Sometimes, the resulting water may be given further treatment, such as fluoridation (*i.e.* the addition of soluble fluoride for controlling dental caries), *liming* (*i.e.* addition of lime in order to control acidity and reduce corrosive action), *recarbonation* (*i.e.* addition of carbon dioxide so as to prevent deposition of calcium carbonate scale), *desalination* (*i.e.* removal of excess salt, if at all present), etc. etc. All the above techniques are now discussed in details.

SCREENING

9.3. Coarse and Fine Screens

Screens are generally provided in front of the pumps or the intake works, so as to exclude the large sized particles, such as debris, animals, trees, branches, bushes, ice, etc. *Coarse screens* (generally called ***trash racks***) are sometimes placed in front of the *fine screens*. Coarse screens consist of parallel iron rods placed vertically or at a slight slope, at about 2.5 to 5 cm apart. The fine screens are usually made of woven wire mesh with openings not more than 6 mm square. The coarse screens first remove the bigger floating bodies and the organic solids ; and the fine screens then remove the fine suspended solids.

The fine screens normally get clogged, and are to be cleaned frequently. *The fine screens are, therefore, avoided these days,* and the finer particles are separated in "Sedimentation" rather than in "Screening". The coarse screens are also now normally kept inclined at about 45°—60° to the horizontal, so as to increase the opening area to reduce the flow velocity, and thus, making the screening more effective. While designing the screens, clear openings should have sufficient total area, so that the velocity through them is not more than 0.8 to 1 m/sec. The material which is collected on the upstream side of the screens is removed either manually or mechanically. In mechanically cleaned screens, a rake traverses the front of the screen either continuously or intermittently.

*Explained in details a little later.

**Explained in details in the previous chapter.

In mechanically cleaned screens, the cross bars, if present, will obstruct raking, and should, therefore, be avoided as far as possible.

A ***fixed bar type screen*** is shown in Fig. 9.1 and 9.2.

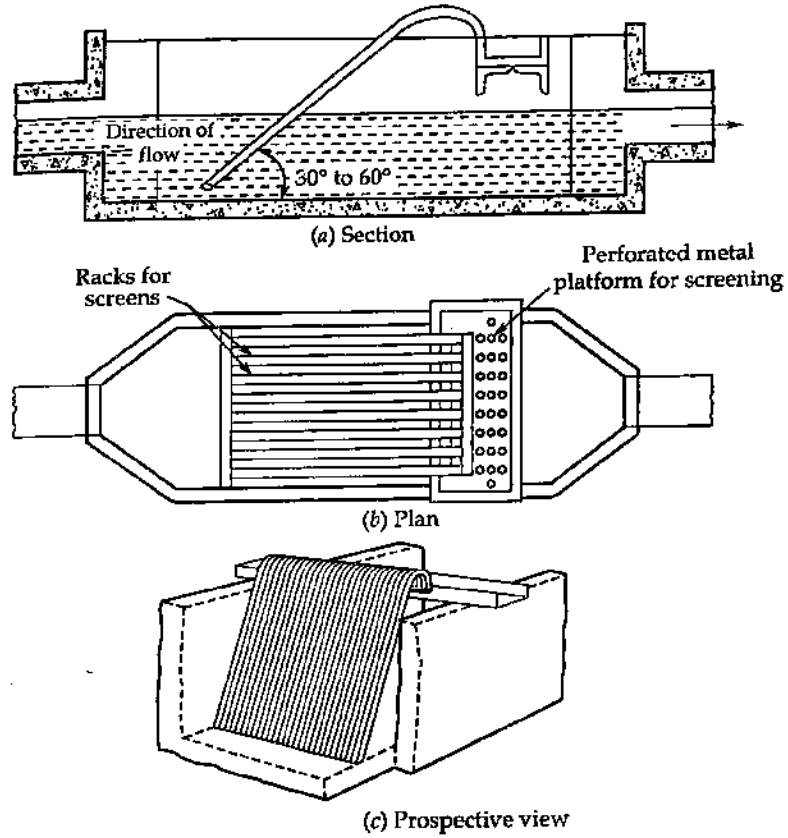


Fig. 9.1. Fixed bar type hand cleaned screen.

Moveable bar type screens or ***Travelling bar screens*** are often used for *fine screens* for passing the water coming out from the coarse screens. Since fine screes are very susceptible to clogging, they must be located at easily accessible places and the design must facilitate cleaning. Although fine screens may be stationary or moveable in operation, yet moveable or travelling fine screens are preferred, as stated above.

A stationary fine screens is cleaned by raising it out of the water, and washing off the collected material. Moveable fine screens are endless belts of fine screening or a series of panels of fine wire mesh mounted on a chain. These are driven slowly by a geared down motor. One type is shown in Fig. 9.3. As the screen is raised above the water level, the debris collected on it are discharged into a trough, usually with the aid of a water spray or a jet. Such screens are usually placed in the inflow channel to the pumps, where they are accessible for maintenance.

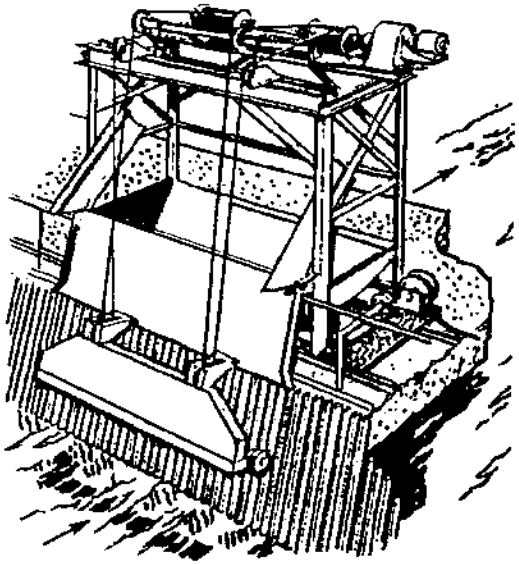


Fig. 9.2. Prospective view of a mechanically cleaned coarse screen (Bar rack with power rake and trash hopper for cleaning)

Fig. 9.3. Mechanically moveable fine screen

PLAIN SEDIMENTATION (Type I Settling)

Most of the suspended impurities present in water do have a specific gravity greater than that of water (*i.e.* 1.0)*. In still water, these impurities will, therefore, tend to settle down under gravity, although in normal raw supplies, they remain in suspension, because of the turbulence in water. Hence, as soon as the turbulence is retarded by offering storage to the water, these impurities tend to settle down at the bottom of the tank, offering such storage. *This is the principle behind sedimentation.*

The basin in which the flow of the water is retarded is called the *settling tank* or *sedimentation tank* or *sedimentation basin* or *clarifier*, and the theoretical average time for which the water is detained in the tank is called the *detention period*.

9.4. Theory of Sedimentation

The settlement of a particle in water brought to rest, is opposed by the following factors :

(*i*) *The velocity of flow* which carries the particle horizontally. The greater the flow area, the lesser is the velocity, and hence more easily the particle will settle down.

(*ii*) *The viscosity of water* in which the particle is travelling. The viscosity varies inversely with temperature. Warm water is less viscous and, therefore, offers less resistance to settlement. However, the temperature of water cannot be controlled to any appreciable extent in "water purification processes" and hence this factor is generally ignored.

(*iii*) *The size shape and specific gravity of the particle.* The greater is the specific gravity, more readily the particle will settle. The size and shape of the particle also affect the settling rate. For example, the weight and volume of the spherically shaped particle, varies with the cube of its diameter (volume being equal to $\frac{\pi d^3}{6}$, where d is the dia) or its size ; and its area varies with the square of the diameter $\left(\text{area being equal to } \frac{\pi d^2}{4}\right)$. Hence, very small sized particles will settle very slowly. It, therefore, clearly follows that the shape and size of the particles do affect their setting velocities.

The settling velocity of a spherical particle is expressed by **Stoke's law**, which takes the above three factors into account. The final Stoke's equation, is expressed as :

$$\boxed{v_s = \frac{g}{18}(G-1)\frac{d^2}{\nu}} \quad \text{for } d < 0.1 \text{ mm} \qquad ...(9.1)$$

[for viscous flow and small sized particles, represented by $R_e < 0.5$]

*The common particles present in turbid waters do have a specific gravity ranging between 2.65 for discrete sand particles to about 1.03 for flocculated mud particles. The particles having specific gravity more than 1.2 can settle easily, while the lighter particles do not settle easily.

where v_s = velocity of settlement of particle (assumed to be spherical) in m/sec.

d = diameter of the particle in m.

G = sp. gravity of the particle.

$$= \frac{\rho_s}{\rho_w} = \frac{\text{density of particle}}{\text{density of water}}$$

ν = kinematic viscosity of water in m^2/sec

Derivation of Stoke's Law. When a solid particle settles down in water, its downward settlement is opposed by the drag force offered by the water. The effective weight of the particle (*i.e.* actual weight—buoyancy) causes the particle to accelerate in the beginning, till it attains a sufficient velocity (v_s) at which the drag force becomes equal to the effective weight of the particle. After attaining that velocity, the particle falls down with that constant velocity (v_s). Now, the drag force offered by the fluid is given by Newton's law, as

$$\text{Drag force} = C_D A . \rho_w . \frac{v^2}{2} \quad ...(1)$$

where C_d = Coefficient of drag

A = Area of particle

ρ_w = Density of water

v = velocity of fall

[**Note.** This drag force increases with the increasing velocity, till it becomes equal to the effective weight of the particle ; and at that time, v becomes equal to v_s.]

The effective weight of the particle

= Total weight – Buoyancy

$$= \frac{4}{3} . \pi r^3 \gamma_s - \frac{4}{3}\pi r^3 . \gamma_w \quad [\because \text{ weight = volume} \times \text{unit weight}]$$

where r = radius of particle

γ_s = unit weight of particle

γ_w = unit weight of water.

$$= \frac{4}{3}\pi r^3 [\gamma_s - \gamma_\omega] \quad ...(2)$$

Eqs. (1) and (2) will become equal when v becomes equal to v_s in Eq. (1).

$$\therefore \quad C_D . A . \rho_w . \frac{v_s^2}{2} = \frac{4}{3}\pi r^3 [\gamma_s - \gamma_\omega]$$

But $A = \pi r^2$

$$\therefore \quad C_D . \pi r^2 . \rho_w . \frac{v_s^2}{2} = \frac{4}{3}\pi r^3 [\gamma_s - \gamma_\omega]$$

or $$v_s^2 = \frac{4}{3} . \frac{(r) . (\gamma_s - \gamma_w) . 2}{\rho_w . C_D}$$

or $$v_s^2 = \frac{\frac{4}{3}\left(\frac{d}{2}\right)(\gamma_s - \gamma_w) . 2}{\rho_w . C_D} \quad \text{or} \quad v_s^2 = \frac{\frac{4}{3} . (\gamma_s - \gamma_w) . d}{\rho_w . C_D} \quad ...(3)$$

Now, $\gamma_s = \rho_s . g$

and $\gamma_w = \rho_w . g$

$$\therefore \quad \gamma_s - \gamma_w = g (\rho_s - \rho_w)$$

$$= g . \rho_w \left(\frac{\rho_s}{\rho_w} - 1\right)$$

$$= g \rho_w . (G - 1)$$

where G = Sp gravity of the particle

Eq. (3) then becomes

$$v_s^2 = \frac{4}{3} . \frac{g . \rho_w (G - 1) d}{\rho_w . C_D} = \frac{\frac{4}{3} . g . (G - 1) . d}{C_D} \quad ...(4)$$

or $$v_s = \left[\frac{\frac{4}{3} g d (G - 1)}{C_D}\right]^{1/2} \quad ...(9.2)$$

The value of coefficient of drag (C_D) changes and depends upon the flow regimes surrounding the particle. The flow regime may be characterised qualitatively as *laminar*, *turbulent* and *transitional*.

In **laminar flow**, the fluid (water) moves in layers, or laminas, one layer gliding smoothly over adjacent layers with only molecular interchange of momentum. In **turbulent flow**, the fluid motion is very erratic with violent transverse interchange of momentum. **Osborne Reynold** developed a quantitative method of describing the different flow regimes, using a dimensionless ratio, called ***Reynold number*** **(Re).** For spheres or spherical particles moving through a liquid (water), this number is defined as :

$$R_e = \frac{v . d}{\nu} \quad ...(9.3)$$

where R_e = Reynold number

d = diameter of the sphere in m or the spherical diameter of the particle

v = velocity of the sphere in m/s

ν = kinematic viscosity of water (liquid) in m^2/s

But $\nu = \frac{\mu}{\rho}$

where μ = dynamic or absolute viscosity of water (liquid) in $N.s/m^2$

ρ = density in kg/m^3

(where N = kgm/sec^2)

The Drag coefficient or Newton's coefficient of Drag (C_D) has been empirically connected with Reynold number (R_e) by *Thomas Camp* by a curve shown in Fig. 9.4.

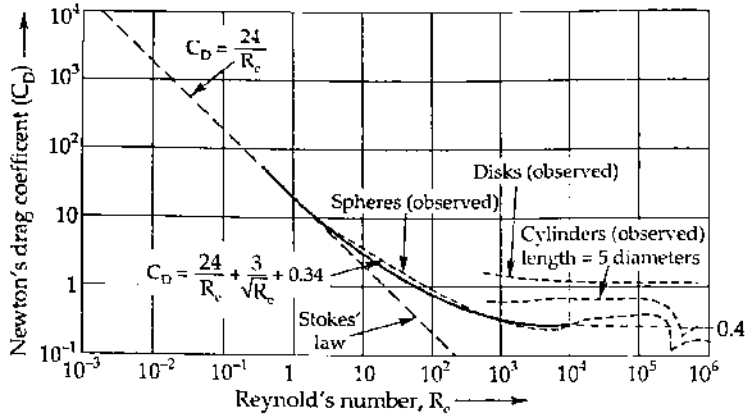


Fig. 9.4. Curve showing relation between C_D and R_e.

It can be seen from the curve of Fig. 9.4 that for *eddying resistance* at higher values of Reynold number exceeding 10^4*, the value of C_D is constant at about 0.4.

Thus, $C_D = 0.4$ for $R_e > 10^4$* (For Turbulent settling, $d > 1.0$ mm) ...(9.4)

For ***viscous resistance at low Reynold number*** ($R_e < 1.0$**), the value of C_D is given by the relation :

$$C_D = \frac{24}{R_e} \text{ for } R_e < 1.0^{**} \text{ (For Laminar settling, } d < 0.1 \text{ mm)} \quad ...(9.5)$$

For *transition region*, R_e lying between 1.0 and 10^4, C_D may be approximated by the relation :

$$C_D = \frac{24}{R_e} + \frac{3}{\sqrt{R_e}} + 0.34 \text{ for } R_e \text{ between 1.0 to 10,000} \quad ...(9.6)$$

(For Transition settling, d between 0.1 mm to 1.0 mm)

Since Equation (9.6) is applicable to Reynold numbers between 1.0 to 10,000 in Transitional zone, which includes almost all situations of interest in sedimentation of water and wastewater, this equation assumes a great importance.

However, for laminar flow region ($R_e < 1.0$), the relation becomes simple as

$$C_D = \frac{24}{R_e} \text{ (as given by Eq. 9.5)}$$

*This value is given to be between 10^3 to 10^4 by Newton. However for sureset turbulent flow existence of higher R_e value of 10^4 is considered.

**Western authors consider this R_e value for Laminar settling to be less than 0.5 (for being on safer side) ; however, we have considered it to be 1.0, based on the value given in GoI, Manual on Water Supply.

Sir George Gabriel **stokes,** while calculating settling velocity for *small spherical particles falling under laminar (quiescent) condition* used equation (9.5) for C_D in Eq. (9.2), to calculate v_s as :

$$v_s = \left[\frac{\frac{4}{3} \cdot g \cdot (G-1) \cdot d}{\frac{24}{R_e}}\right]^{1/2}$$

or $$v_s^2 = \frac{4}{3} \cdot g\,(G-1) \cdot d \cdot \frac{R_e}{24}$$

or $$v_s^2 = \frac{g}{18}(G-1)\,d \cdot \frac{v_s \cdot d}{\nu}$$

or $$v_s = \frac{g}{18}\,(G-1) \cdot \frac{d^2}{\nu}$$

or $$v_s = \frac{g}{18}\,(G-1) \cdot \frac{d^2}{\nu}$$; which is the requisite Stoke's law given in equation (9.1) for settling velocity of small particles and viscous flows.

This equation can be used to determine the settling velocity of spherical particles.

Since the viscosity (ν) is dependent upon temperature*, the above equation can be further modified and written as :

*See Table 9.1 below :

Table 9.1. Important Properties of Water

Temp. °C	*Sp. gravity*	*Unit wt. in kN/m³ i.e. Sp. gravity × 9.807*	*Vapour pressure in kN/m²*	*Viscosity*	
				Dynamic (μ) in N-sec/m²	*Kinematic (ν) in m²/sec** $\nu = \frac{\mu}{\rho}$
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*
0	0.99987	9.805	0.611	17.97×10^{-4}	1.78×10^{-6}
5	0.99999	9.807	0.872	15.17×10^{-4}	1.518×10^{-6}
10	0.99975	9.805	1.225	13.06×10^{-4}	1.310×10^{-6}
15	0.99907	9.798	1.703	11.25×10^{-4}	1.124×10^{-6}
20	0.99810	9.788	2.334	10.03×10^{-4}	1.009×10^{-6}
30	0.99574	9.765	4.609	8.02×10^{-4}	0.804×10^{-6}
40	0.99228	9.731	7.600	6.52×10^{-4}	0.654×10^{-6}
60	0.98338	9.644	19.938	4.70×10^{-4}	0.478×10^{-6}
80	0.97196	9.532	46.583	3.56×10^{-4}	0.366×10^{-6}
100	0.95865	9.401	101.502	2.82×10^{-4}	0.295×10^{-6}

*Units of $\upsilon = \frac{\mu}{\rho}$ is given as :

$$= \frac{\text{N.sec}}{\text{m}^2} \times \frac{1}{\text{kg/m}^3} = \left(\frac{\text{kg.m}}{\text{sec}^2}\right)\frac{\text{sec}}{\text{m}^2} \cdot \frac{\text{m}^3}{\text{kg}} = \frac{\text{m}^2}{\text{sec}}. \qquad [\because\ N = \text{kgm/s}^2]$$

$$v_s = 418\,(G-1)\,.\,d^2\left(\frac{3T+70}{100}\right) \text{ for } d < 0.1 \text{ mm} \qquad ...(9.7)$$

where T = Temp. of water in °C

v_s is in mm/sec.

d is in mm

The above Stoke's equations [Eqn. (9.1) and (9.2)] are valid for particles of size less than 0.1 mm ; in which case, the viscous force predominates over the inertial force. This is called ***streamline settling***.

If however, the settling particles are larger than 1.0 mm, the nature of settling becomes *turbulent settling,* and is governed by ***Newton's equation***, given by :

$$v_s = 1.8.\ \sqrt{gd(G-1)} \text{ for } d > 1.0 \text{ mm} \qquad ...(9.8)$$

Grit particles however, generally lie between 0.1 mm and 1 mm, in the *transition zone*, where settling velocity is given by **Hazen** as :

$$v_s = 418\,(G-1)d\left(\frac{3T+70}{100}\right) \text{ when } d \text{ lies} \qquad ...(9.9)$$

between 0.1 mm and 1 mm

where T = Temp of water in °C.

The above formulas represent the theoretical settling velocities of **discrete*** spherical particles. The actual settling velocities in the sedimentation basins will be much less than those calculated by these formulas, because of : (*i*) the non-sphericity of the particles, (*ii*) the upward displacement of the fluid caused by the settling of other particles, and (*iii*) convection currents.

Example 9.1. *Find the settling velocity of a discrete particle in water under conditions when Reynold's number is less than 0.5. The diameter and specific gravity of the particle is 5×10^{-3} cm and 2.65, respectively. Water temperature is 20°C (Kinematic viscosity ν of water at 20°C = 1.01×10^{-2} cm²/sec).*

Solution. Using Stoke's Eq. (9.1), which is valid for $R_e < 0.5$ (as given), and c.g.s. units, we have

v_s = settling velocity in cm/sec

$$= \frac{g}{18}(G-1)\,.\,\frac{d^2}{\nu} \text{ when } d < 0.1 \text{ mm}$$

where $G = 2.65$

$d = 5 \times 10^{-3}$ cm

$= 0.05$ mm, which is < 0.1 mm (O.K.)

$\nu = 1.01 \times 10^{-2}$ cm²/sec

$g = 981$ cm²/sec

$$\therefore \quad v_s = \frac{981}{18}(2.65-1)\,.\,\frac{(5\times10^{-3})^2}{1.01\times10^{-2}} \text{ cm/sec}$$

*"***Discrete***" or "granular" particles are those which do not change their size, shape and weight. The other type of particles called **"flocculated"** particles are those which change their size, shape and weight, and thus loose their identity.

$$= \frac{981}{18}\times\frac{1.65\times25\times10^{-6}}{1.01\times10^{-2}} \text{ cm/sec} = \mathbf{0.22\ cm/sec.}\ \ \textbf{Ans.}$$

Alternatively, the problem can also be solved by using Eq. (9.7) as :

$$v_s = 418(G-1)\,.\,d^2\,.\left(\frac{3T+70}{100}\right) \text{ for } d < 0.1 \text{ mm}$$

where d is in mm

and v_s is in mm/sec

where $G = 2.65$

$d = 5 \times 10^{-3}$ cm

$= 5 \times 10^{-2}$ mm

$T = 20$°C

$$\therefore \quad v_s(\text{in mm/sec}) = 418(2.65-1)\,.\,(5\times10^{-2})^2\left(\frac{3\times20+70}{100}\right)$$

$$= 418 \times 1.65 \times 25 \times 10^{-4} \times 1.3 = 2.24$$

or $\quad v_s = 2.24$ mm/sec = **0.224 cm/sec. Ans.**

9.5. Sedimentation Tanks

The clarification of water by the process of "sedimentation" can be effected by providing conditions under which the suspended material present in water can settle out. Storage reservoirs may also serve as sedimentation basins, but they cannot effect proper sedimentation, because of factors, such as, the density currents, the turbulences caused by winds, etc. ; and hence they cannot be relied upon. Special basins are, therefore, constructed in order to purify the surface waters of rivers or reservoirs by the process of sedimentation.

But of the three forces, which control the settling tendencies of the particles (enumerated earlier), the two forces, *i.e.*, the velocity of flow, and the shape and size of the particles, are tried to be controlled in these settling tanks. The third force, *i.e.*, the viscosity of water or the temperature of water is left uncontrolled, as controlling the same is not practically possible. The velocity of flow can be reduced by increasing the length of travel and by detaining the particles for a longer time in the sedimentation basin. The size and the shape of the particles can be altered by the addition of certain chemicals in water. These chemicals are called ***coagulants***, and they make the sedimentation quite effective by leading to the settlement of even very fine and colloidal particles. However, their use is not made in "plain sedimentation" but is made under the process titled "sedimentation with coagulation". This has been discussed a little later.

Sedimentation basins are generally made of reinforced concrete, and may be rectangular or circular in plan. Long narrow *rectangular tanks with horizontal flow* (Fig. 9.5) are generally preferred to the *circular tanks with horizontal radial or spiral flow* (Fig. 9.6). The capacity and other dimensions of the tank should be properly designed, so as to effect a fairly high percentage of removal of the suspended materials. A plain sedimentation tank under normal conditions may remove as much as 70% of the suspended impurities present in water.

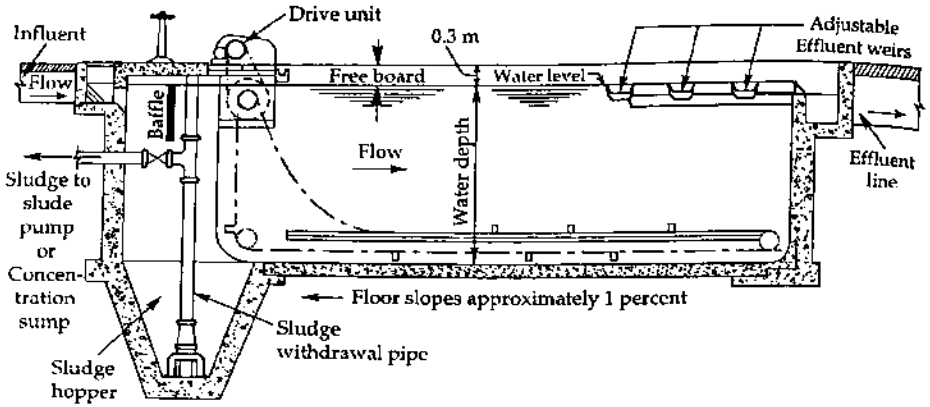


Fig. 9.5. Rectangular sedimentation tank (ideal horizontal flow tank).

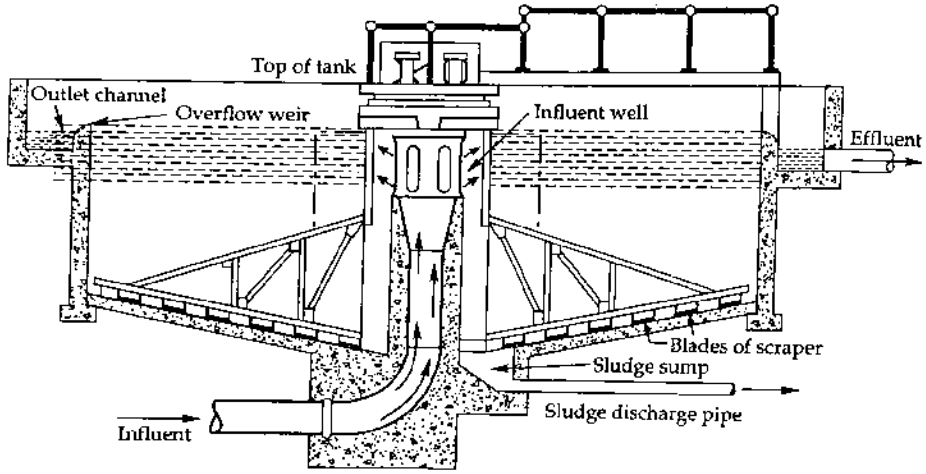


Fig. 9.6. Circular sedimentation tank (Central feed) with radial flow.

9.5.1. Types of Sedimentation Tanks. The sedimentation tanks can basically be divided into two types ; *viz.*

(1) *horizontal flow tanks* ; and

(2) *vertical or up flow tanks.*

These tanks may be rectangular or circular in plan. Both these types of tanks are briefly discussed here.

(1) **Horizontal flow tanks.** In the design of horizontal flow tanks, *the aim is to achieve, as nearly as possible, the ideal conditions of equal velocity at all points lying on each vertical line in the settling zone.* The direction of flow in the tank is substantially horizontal.

Among the horizontal flow tanks, we may have different types of designs, such as :

(*i*) ***Rectangular tanks with longitudinal flow***, such as the one shown in Fig. 9.5. They may be provided with *mechanical scrapping devices*, to scrap the sludge to the *sludge pit* located usually towards the influent end, from where it is continuously or periodically removed, without stopping the working of the tank. Such tanks are known as ***continuous flow type of sedimentation*** tanks. In such a tank, the flow velocity is only reduced and the water is not brought to complete rest.

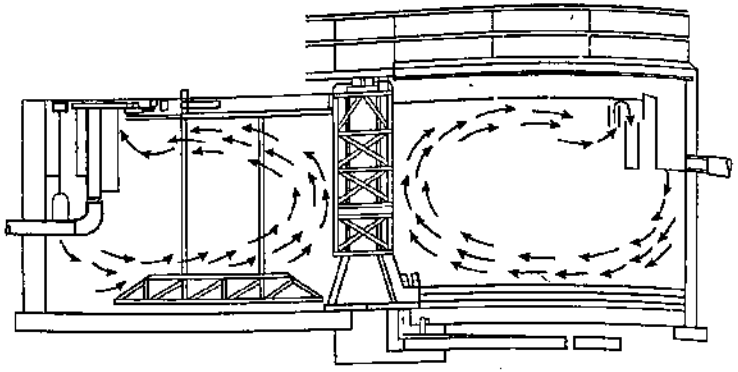
In other types of such tanks, mechanical scrappers may not be provided and the tank may have to be cleaned by stopping the operation of the tank. In such ***intermittent type of sedimentation tanks***, raw water is simply stored and kept at rest for a certain period, say about 24 hours. During this rest period, the suspended particles settle down to the bottom of the tank. The clear water from the tank is finally taken out, and the tank is cleaned off the settled silt. The tank is again filled with raw water to continue the next operation. This type of tank, thus, functions intermittently, as a period of about 30 to 36 hours is usually required to put the tank again in working order. This necessitates the commissioning of atleast two tanks. Such intermittent tanks are usually not preferred in modern treatment plants, as they lead to wastage of time and labour, requiring more units.

Continuous flow type of tanks are, therefore, invariably used these days. The working of such a tank is simple, as the water enters from one end and comes out from the other end. The velocity is sufficiently reduced by providing sufficient length of travel. The velocity is so designed that the time taken by the sand particle to travel from one end to another is slightly more than the time required for settlement of that particle.

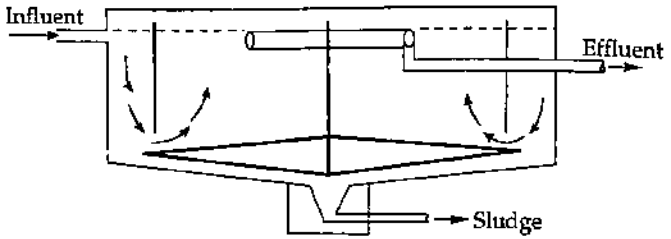
(*ii*) ***Circular tanks with radial flow, with central feed***, such as the one shown in Fig. 9.6. In such a tank, the water enters at the centre of the tank into a circular well provided with multiple ports, from which it emanates out to flow radially outwards in all directions equally. The water, thus, flows horizontally, and radially from the centre towards the periphery of the circular tank. *The aim here is to provide uniform radial flow with decreasing horizontal velocity towards the periphery*, from where the water is withdrawn from the tank through the effluent structure (overflow weir, etc.). The sludge is scrapped to the central sump mechanically and continuously, from where it is withdrawn during operation. The sludge removal mechanism consists of scraper blades mounted on two or four arms revolving slowly.

The second type of circular tanks, are provided with *peripheral feed*. These tanks differ from the central feed circular tanks in that, the raw water here enters the tank from the periphery or the rim. It has been demonstrated that the average detention time is greater in peripheral feed basins, leading to better performance. Two possible arrangements of such a tank are shown in Fig. 9.7 (*a*) and (*b*).

(2) **Vertical or Upflow settling tanks.** Vertical flow tanks usually combine sedimentation with flocculation, although they may also be used for plain sedimentation. They may be square or circular in plan, and may have *hopper bottoms*. (Refer Fig. 9.8). The influent enters at the bottom of the unit. The upflow velocity decreases with the increased cross-sectional area of the tank. The clarified water is withdrawn through the circumferential or central weir.



(*a*) Peripheral feed circular sedimentation tank with effluent and influent channels separated by a skirt.



(*b*) Peripheral feed circular sedimentation tank with effluent weirs near the centre of the basin.

Fig. 9.7. Circular Peripheral feed sedimentation tank.

When used with coagulants, the flocculation takes place in the bottom of the tank leading to formation of blanket of floc through which the rising floc must pass. Because of this phenomenon, these tanks are also called the ***Upflow sludge blanket clarifiers.***

9.5.2. Design Concepts in Plain Sedimentation Tanks (Type I Settling). In sedimentation tanks, we use a term, called ***overflow velocity***, or **overflow rate**, or ***surface overflow rate (SOR)***, or *surface loading*, which may be represented by the symbol (v_o). It is that flow velocity at which the tank is designed to operate. The other term relating to sedimentation has already been defined, and is called the **settling velocity** (v_s) of the particle. The importance of these two terms can be understood easily by considering an upflow sedimentation tank, as shown in Fig. 9.8.

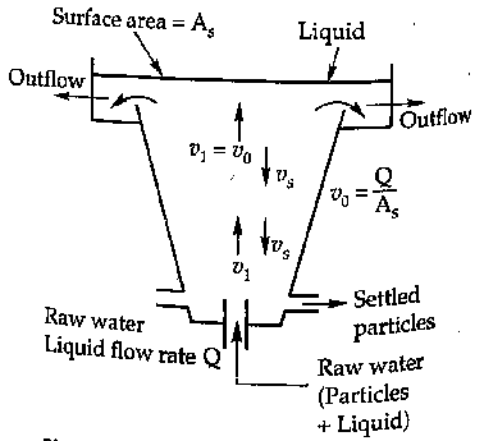


Fig. 9.8. Sedimentation in an upflow clarifier.

In this design, the particles fall downward and the water rises vertically upward. The rate at which the particle is settling is v_s (*i.e.* the particle-settling velocity), and the velocity of the liquid rising is v_o (*i.e.* the overflow rate). Evidently, if the particle is to be removed from the bottom of the tank and not go out in the clarified water, then the particle settling velocity (v_s) must be greater than the liquid rise velocity v_o. In other words, v_s should be equal to or greater than v_o. If this happens, *i.e.* when $v_s \geq v_o$, then one would expect 100% removal of particles ; and if v_s is less than v_o, one would expect 0 percent removal. Hence, in the successful design of a sedimentation tank, the aim would be to determine the particle settling velocity (v_s), and fix the overflow rate at some lower value. Often v_o is kept at about 80% of v_s, for an *upflow clarifier*.

The liquid rise velocity (v_o) is called the overflow rate, because the water is flowing over the top of the tank into the weir system. This term is also referred to as ***surface loading rate***, because its units are : discharge per unit of plan area, *i.e.* $m^3/d.\ m^2$, the unit m^3/d representing discharge or flow of water in the tank, and m^2 is the surface area of the tank (A_s)*. This term, v_o, can be thought of as the amount of water that goes through each m^2 of tank surface area per day, which is similar to a loading rate.

We also know that the velocity of flow is always equal to the flow rate divided by the area through which the water flows. Hence, an overflow rate is the same as the liquid velocity, since

$$v_o = \frac{\text{Discharge}}{\text{Surface area}} = \frac{\text{Volume/Time}}{\text{Surface area}}$$

$$= \frac{(\text{Surface area} \times \text{Depth})/\text{Time}}{\text{Surface area}}$$

$$= \frac{\text{Depth}}{\text{Time}} \text{ (i.e. m/sec)} = \text{Liquid velocity} \qquad \text{...(9.10)}$$

It can therefore be concluded that as long as v_s is greater than v_o, the particles will settle downward, and be removed from the bottom of the tank, regardless of the depth of the tank. *The particle removal being independent of the depth of the sedimentation tank.* Sedimentation zones vary from a depth of a few centimetres to 6 m or greater**.

In the case of a **horizontal sedimentation tank**, we can similarly show that particle removal is likewise dependent only upon the overflow rate. Such a horizontal sedimentation tank is designed based on the following **three assumptions** :

(*i*) Particles and their velocity vectors are evenly distributed across the tank cross-section. This is the function of the inlet zone.

(*ii*) The liquid moves as an ideal slug, down the length of the tank.

(*iii*) Any particle hitting the bottom of the tank is removed.

---

*For a circular tank of dia. D, $A_s = \frac{\pi}{4} D^2$; while for a rectangular tank of width B and length L, $A_s = B \cdot L$.

**Tube settlers are designed with a very shallow settling zone.

Let us now consider such a ***horizontally flowing rectangular tank (continuous of course)*** as shown in Fig. 9.9. Let the water containing uniformly distributed sediment enters this tank with a uniform velocity v. If Q is the discharge entering the basin, the flow velocity v is given as :

$$v = \frac{Q}{B.H} \qquad ...(9.11)$$

where B = Width of the tank

H = Depth of the water in the tank.

Now, let us consider a discrete particle which enters the tank at the top of the basin at point A (Fig. 9.9) at full water height H from the bottom. Such a particle will have a horizontal flow velocity v and a downward vertical velocity v_s. Its resultant path is given by the vector sum of its flow velocity (v) and the settling velocity (v_s). Obviously, if this particle has to settle down in the tank, this resultant must in extreme case, move along the line AC, so that the particle just touches the tank bottom before reaching the outlet zone. If this particle entering at the top of the tank is so removed, all particles with the same settling velocity will be removed even if they enter the tank at lesser height, as their trajectory will bring them down to the bottom somewhere between the points B and C.

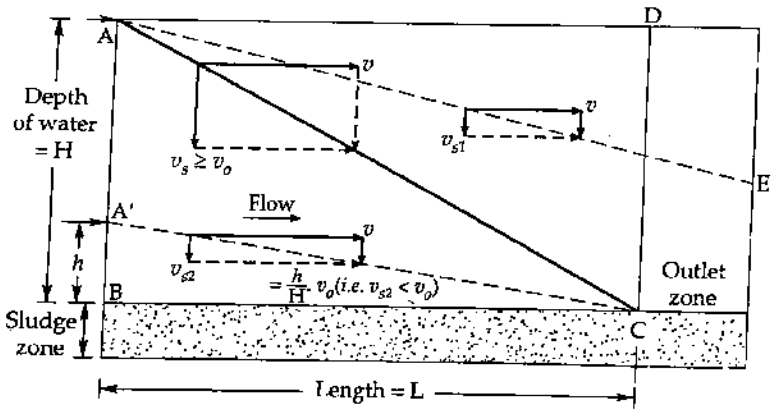


Fig. 9.9. Elevation of a rectangular sedimentation tank (line diagram).

Considering the extreme case of slope vector from A to C, and the geometry of the basin, we can write

$$\frac{v}{v_s} = \frac{L}{H} \qquad ...(9.12)$$

or $$v_s = v \cdot \frac{H}{L}$$

or $$v_s = \frac{Q}{BH} \cdot \frac{H}{L} \qquad \text{(From Eq. 9.11)}$$

or $$v_s = \frac{Q}{BL} = \frac{Q}{A_s} \qquad ...(9.13)$$

where A_s = surface area of the tank (BL).

$\frac{Q}{A_s}$ is nothing but the overflow rate (v_o).

Thus, the surface overflow rate can be said to be representing the settling velocity of the slowest settling particles, which are 100% removed.

If the settling velocity say v_{s_1}, would have been less than v_o as would happen in the case of lighter particles, then the particle entering at the top of the tank would not have reached the bottom of the tank and gone out in the outlet zone, as shown by its resultant vector line AE. It can, therefore, be concluded that all those particles having settling velocity equal to or greater than v_o, will settle down and be removed entirely. In other words, no particle having a settling velocity more than or equal to v_o will remain suspended in such a tank.

The above conclusion that a particle having settling velocity greater than or equal to v_o will be removed is infact true when we have considered a particle entering the tank at full height H. Truly speaking, even a smaller particle having settling velocity, say v_{s_2}, lower than v_o, will settle down if it happens to enter the tank at some lower height, say h. Such a particle entering at height h and having its settling velocity $\geq \frac{h}{H} \cdot v_o$ will infact settle down in such an ideal tank. Such a particle entering at lesser height will infact find a correct trajectory to reach the bottom, and hence removed. This leads us to the concept of **partial removal of lighter particles** in a horizontally flowing rectangular tank (ideal settling tank).

Unlike in an upflow clarifier, therefore, a horizontal rectangular clarifier will thus cause not only the removal of heavier particles (i.e. particles having $v_s \geq v_o$), but will also cause partial removal of the smaller sized particles having $v_s < v_o$.

For example, let us consider particles having settling velocity 0.5 v_o entering uniformly into the settling zone, as shown in Fig. 9.10. This figure clearly shows that 50% of these particles (*i.e.* those entering below the depth $h = 0.5H$) will hit the bottom of the tank before being carried to the outlet zone, because they have to settle only 0.5 H depth of the tank. Likewise, 25% of the

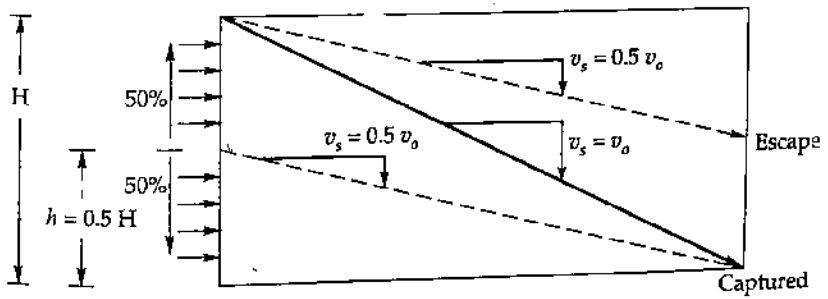


Fig. 9.10. Partial removal of solids in a horizontally flowing settling tank.

particles having settling velocity of 0.25 v_o will be removed. The percentage (%) of such lighter particles having $v_s < v_o$, which shall get removed, will thus be equal to $\frac{v_s}{v_o} \times 100$,

or
$$P_e = \frac{v_s}{v_o} \times 100 \qquad ...(9.14)$$

where P_e = %age of lighter particles (with settling velocity less than v_o), which shall be removed in an ideal settling basin.

Such a horizontally flowing rectangular settling tank, where the raw water enters uniformly distributed along the entire height, thus helps us in partial removal of even smaller sized particles (*i.e.* particles smaller than that given by its overflow rate) ; and is hence considered to be an ***ideal sedimentation tank***. *No such partial removal of lighter particles will occur in an upflow tank, as stated earlier.*

To determine the efficiency of removal of suspended solids in such an ideal tank will therefore become a little bit difficult since in a typical sedimented raw water, a large gradation of particle sizes may occur. To determine the efficiency of removal of sediment load for a given settling time or overflow rate in such an ideal tank, it is necessary to consider the entire range of the settling velocities present in the system. This job can be done by performing the *sieve analysis and hydrometer testing* to work out the %ages of particles of various stated sizes. The settling velocities for all such sizes can be computed by using Stoke's low (Eq. 9.1). The %ages of particles with less than the stated settling velocities can then be used to plot a curve, known as the ***settling velocity analysis curve for discrete particles***, as shown in Fig. 9.11.

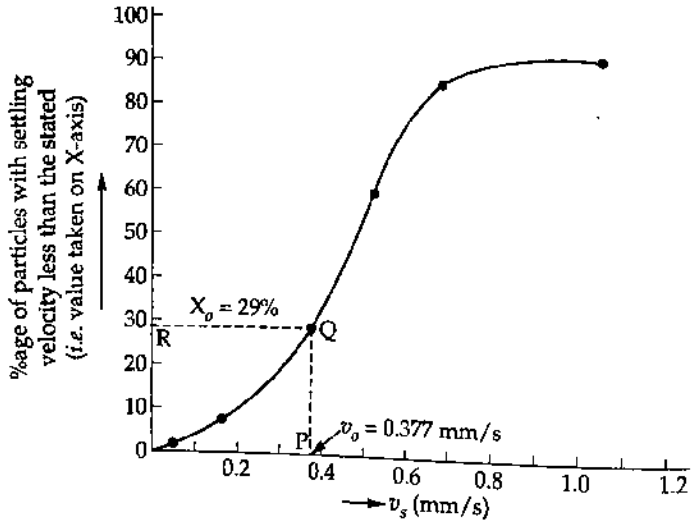


Fig. 9.11. Settling velocity analysis curve for a discrete particle in an ideal settling tank (plotted for data of Example 9.9).

In all, the particles having settling velocity greater than or equal to v_o will be fully removed ; and the remaining particles, say X_o % will be removed in the ratio of $\frac{v_s}{v_o}$.

The total %age of particles removed

$$= (100 - X_o) + \int_{X=0}^{X=X_o} \left(\frac{v_s}{v_o} \times 100\right) dX \qquad ...(9.15)$$

where X_o corresponds to v_o.

Since mathematical integration of the curve is not feasible, the summation is done by dividing the enclosed area into finite strips, each of interval ΔX, say ; with corresponding v_s values read out from the curve. The summation of their product $v_s \Delta X$ can be done, as to approximate Eq. (9.15) as :

Total percentage of particles removed

$$= (100 - X_o) + \frac{1}{v_o} \Sigma v_s \cdot \Delta X \qquad ...(9.16)$$

The overflow rate (v_o), in any case, remains the controlling factor for the removal of particles in all types of settling basins. ***Normally adopted values of overflow rates vary between 12—18 m^3 per day per m^2 of plan area* for plain sedimentation tanks (i.e. Type I settling) ; and between 24—30 m^3 per day per m^2 of plan area for sedimentation tanks using coagulants as aids (Type II settling).***

Decreasing the overflow rate will lead to the settlement of even those particles which are having lower values of their settling velocities. *Hence, smaller particles will also settle down, if the overflow rate is reduced.* Moreover, with a given Q, the overflow rate can be reduced by increasing the plan area of the basin. It, therefore, follows that an increase in the plan area (*i.e.* width × length) will increase the efficiency of the sedimentation tank ; and *theoretically speaking, depth does not have any effect on the efficiency of sediment removal.* However, it is important for practical considerations and also for making allowances for deposition of sludge and silt. Usual values of depth ranges between 3.0 to 4.5 m, with 1.8 m as minimum and 6 m as maximum.

Another important term which is used in connection with the design of the sedimentation basins is its *detention time* or *detention period* or *retention period.* The **detention period** (t) of a settling tank may be defined as the average theoretical time required for the water to flow through the tank length. It is that time which would be required by the flow of water to fill the tank, if there were no outflows. In other words, it is the average time for which the water is detained in the tank. Hence, it is the ratio of the volume of the basin to the rate of flow (or discharge) through the basin.

---

*12–18 $m^3/d/m^2$ means $\frac{12 \times 1000}{24} = 500$ to $\frac{18 \times 1000}{24} = 750$ l/hr/m^2

∴ *Detention time t for a Rectangular tank*

$$= \frac{\text{Volume of the tank}}{\text{Rate of flow}} = \frac{BLH}{Q} \qquad ...(9.17)$$

Similarly, *detention time for a Circular tank*

$$= \frac{d^2(0.011d + 0.785H)}{Q} \qquad ..(9.18)$$

where d = Dia of the tank

H = Vertical depth at wall or side water depth.

This formula is based on the fact that the volume of the type of circular tank used for sedimentation, *i.e., the one having a cone-shaped bottom with 1 : 1 slope,* is found to have a volume equal to $d^2(0.011\,d + 0.785H)$.

The detention time usually ranges between 4 to 8 hours for plain sedimentation, and from 2 to 4 hours when coagulants are used. The width of the tank is normally kept equal to 10 m and not allowed to exceed 12 m or so. The length of the tank is not generally allowed to exceed four times the width, though it may vary from one to six times its width. The cross-sectional area of the sedimentation tank is such as to provide a horizontal flow velocity ranging between 0.15 to 0.9 m/minute, normally kept at about 0.3 m/minute. The total amount of flow from the tank within 24 hours, generally equals the maximum daily demand of water.

For the efficient removal of sediment in the sedimentation tanks, it is necessary that the flow is uniformly distributed throughout the cross-section of the tank. If currents, on the other hand, permit a substantial portion of the water to pass directly through the tank without being detained for the intended time, the flow is said to be ***short circuited.*** Properly designed inlets and outlets near the entrance and exit may reduce the short circuiting tendencies and distribute the flow more uniformly. Moreover, long relatively narrow tanks are less affected by the inlet and outlet disturbances, and by the currents caused by breezes.

But, however, in actual practice, certain amount of short circuiting always exist and, therefore, the actual average time which a batch of water takes in passing through a settling tank, called the **flowing through period**, is always less than the detention period which is the corresponding theoretical time. The ratio of the 'flowing through period' to the 'detention period' is called the *displacement efficiency*. Therefore,

$$\textbf{Displacement efficiency} = \frac{\text{Flowing through period}}{\text{Detention period}} \qquad ...(9.19)$$

It generally varies between 0.25 to 0.50 in normal sedimentation basins.

9.5.3. Check against Scour of Deposited Particles. So far, we have ensured to limit the overflow velocity in the sedimentation tank as to cause the settlement of all the particles smaller than the size d. However, we have also to confirm that this flow velocity should not be such as to cause scour of the settled or deposited particles. In other words, the flow velocity should not be more than the scour velocity, as to cause the uplift of the particles which have settled and reached the sludge zone. Hence, the flow velocity in the tank (v) should be less than the *scour* or *displacement velocity* (v_d). Camp (1946) has

given the following equation for computing the displacement or **scour velocity**, which velocity represents the minimum velocity required to start the motion of the settled particles of size d :

$$\textit{Scour velocity} = v_d = \sqrt{\frac{8\beta}{f'} \cdot g \cdot (G-1)\, d} \qquad ...(9.20)$$

where β = 0.04 for unigranular sand and 0.06 or more for non-uniform (inter-locking) sticky material.

f' = Darcy Weisbach friction factor

= 0.025 to 0.03 for settling tanks.

g and G have the usual meaning.

9.5.4. Inlet and Outlet Arrangements in Sedimentation Tanks. A very suitable type of an inlet for a rectangular settling tank is in the form of a channel extending to full width of the tank with a submerged weir type baffle wall, as shown in Fig. 9.12.

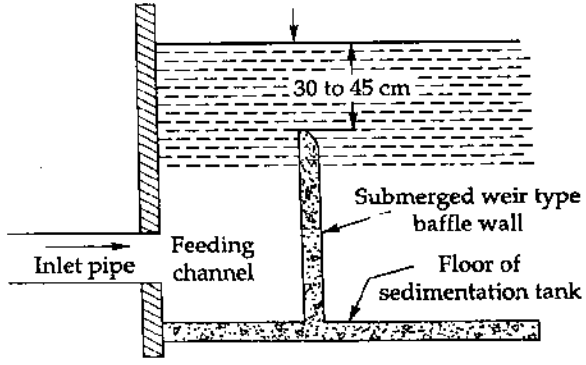


Fig. 9.12. Section of a submerged type or a weir type inlet.

A similar type of outlet arrangement is also used these days. It consists of an outlet channel extending for full width of the tank and receiving the water after it has passed over a weir, as shown in Fig. 9.13.

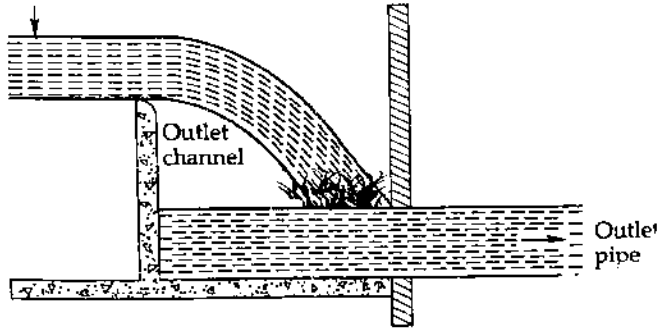


Fig. 9.13. Section of a 'submerged type' or a 'weir type' outlet.

9.5.5. Cleaning and Sludge Removal from Sedimentation Tanks. The suspended materials contained by the raw supplies settle down at the bottom of the sedimentation tank, and have to be removed periodically. The removal of the deposited sludge and sediment is necessary not only because it reduces the capacity of the tank and its detention period, but also because it leads to the formation and evolution of certain foul gases due to the decomposition of the settled organic matter. The sedimentation tanks are, therefore, cleaned from time to time at frequent intervals, either manually or they are provided with mechanical arrangements for cleaning. The tanks are, therefore, provided in duplicate for facility of cleaning. The manual cleaning may be carried out as follows :

The tank is, first of all, put out of service, and the supply of raw water is stopped. The already contained water is drained off till the depth remains about 30 cm or so. The sludge is now stirred and removed as a slurry, through a separate pipe provided with a gate valve at the bottom of the tank. *Pick axes* and *phaorahs* may be used for displacing hard deposits. The removed sludge is normally thrown into the municipal sewers.

In modern water treatment plants (WTPs), sedimentation tanks are generally fitted with mechanical cleaning devices. Say for example, in Fig. 9.5, the sludge is scrapped and brought to the hopper at the outlet end, and is removed daily or oftener. Similarly, in the circular tank of Fig. 9.6, the sludge is scrapped and brought to the centre and likewise removed. For tanks without mechanical sludge removal equipment, an additional minimum depth of about 0.8 to 1.2 m should be provided for storage of sediment, and is called the ***sludge zone.***

9.5.6. Design Examples on Sedimentation Tanks

Example 9.2. *The maximum daily demand at a water purification plant has been estimated as 12 million litres per day. Design the dimensions of a suitable sedimentation tank (fitted with mechanical sludge removal arrangements) for the raw supplies, assuming a detention period of 6 hours and the velocity of flow as 20 cm per minute.*

Solution. Quantity of water to be treated in 24 hours

$$= 12 \times 10^6 \text{ litres}$$

Quantity of water to be treated during the detention period of 6 hours

$$= \left[\frac{12\times10^6}{24}\times 6\right] \text{ litres} = 3 \times 10^6 \text{ litres}$$

$$= 3 \times 10^3 \text{ cubic metres} = 3000 \text{ cu. m.}$$

$\therefore$ The capacity of the tank required = 3000 cu. m. **Ans.**

Velocity of flow to be maintained through the tank

$$= 20 \text{ cm/minute} = 0.2 \text{ m/minute}$$

The length of the tank reqd.

$$= \text{Velocity of flow} \times \text{Detention period}$$

$$= 0.2 \times (6 \times 60) = 72 \text{ m.}$$

Cross-sectional area of the tank required

$$= \frac{\text{Capacity of the tank}}{\text{Length of the tank}} = \frac{3000}{72} \text{ m}^2 = 41.67 \text{ m}^2, \text{ say } 41.7 \text{ m}^2.$$

Assuming the water depth in the tank as 4 m*, the width of the tank required $= \frac{41.7}{4} = 10.42$ m, say 10.5 m.

Using a free-board of 0.5 m, the overall depth

$$= 0.5 + 4.0 = 4.5 \text{ m.}$$

Hence, a rectangular sedimentation tank with an overall size of 72 m × 10.5 m × 4.5 m can be used. **Ans.**

Example 9.3. *Two million litres of water per day is passing through a sedimentation tank which is 6 m wide, 15 m long and having a water depth of 3 m. (a) Find the detention time for the tank. (b) What is the average flow velocity through the tank ? (c) If 60 ppm is the concentration of suspended solids present in turbid raw water, how much dry solids will be deposited per day in the tank, assuming 70% removal in the basin, and average specific gravity of the deposit as 2. (d) Compute the overflow rate.*

Solution. (*a*) The capacity of the tank = $L.B.D$

$$= 15 \text{ m} \times 6 \text{ m} \times 3 \text{ m} = 270 \text{ m}^3$$

Discharge passing through the tank

$$= Q = 2 \text{ million litres per day}$$

$$= 2 \times 10^6 \text{ litres/day} = \frac{2\times10^6}{24} \text{ litres/hr}$$

$$= 83.33 \times 10^3 \text{ litres/hr.} = 83.33 \text{ cu.m./hr.}$$

$$\text{Detention time} = \frac{\text{Capacity of the tank}}{\text{Discharge}}$$

$$= \frac{270}{83.33} \text{ hours} = 3.24 \text{ hours.} \quad \textbf{Ans.}$$

**Alternatively*

Instead of assuming the depth, we may assume an overflow rate, say as 600 litres/hr/m^2.

$\therefore$ $\frac{Q}{BL} = 600$ l/hr. m^2 [*i.e.* between 500 to 750 l/hr/m^2]

But $Q = \frac{12\times10^6}{24}$ litres/hr $= 0.5 \times 10^6$ litres/hr.

$\therefore$ $BL = \frac{Q}{600} = \frac{0.5\times10^6}{600} = 833$ m^2 or $B = \frac{833}{72} = 11.6$ m.

$\therefore$ the depth $= \frac{3000}{72\times11.6} = 3.6$ m.

Hence, we can alternatively use a tank of dimensions

$$= 72 \text{ m} \times 11.6 \text{ m} \times (3.6 + 0.5) \text{ m overall depth.}$$

i.e. 72 m × 11.6 m × 4.1 m size. **Ans.**

(*b*) Average velocity of flow through the tank

$$= \frac{\text{Discharge}}{\text{Cross-sectional area, } i.e.\ BH}$$

$$= \frac{83.33}{6 \times 3} \text{ m/hr} = \frac{83.33}{6 \times 3} \times \frac{100}{60} \text{ cm/minute}$$

$$= 7.72 \text{ cm/minute.} \quad \textbf{Ans.}$$

(*c*) Quantity of water passing per day

$$= 2 \text{ million litres} = 2 \times 10^6 \text{ litres}$$

Concentration of suspended solids = 60 ppm.

∴ Quantity of suspended solids entering the tank per day

$$= 2 \times 10^6 \times \frac{60}{10^6} \text{ litres} = 120 \text{ litres} = 0.12 \text{ cu.m.}$$

Given the average specific gravity of the deposited material as 2, we have its density as 2000 kg/m³.

∴ Mass of suspended solids deposited (with 70% removal) per day

$$= [0.12 \times 0.7][2000] \text{ kg} = 168 \text{ kg.} \quad \textbf{Ans.}$$

(*d*) Overflow rate = Discharge per unit plan area

$$= \frac{Q}{B.L} = \frac{83.33 \times 10^3 \text{ litres/hr}}{6 \times 15 \text{ m}^2} = 926 \text{ litres/hr/m}^2. \quad \textbf{Ans.}$$

Example 9.4. *A rectangular settling tank without mechanical equipment is to treat 1.8 million litres per day of raw water. The sedimentation period is to be 4 hours, the velocity of flow 8 cm/minute, and the depth of the water and sediment 4.2 m. If an allowance of 1.2 m for sediment is made, what should be (a) the length of the basin ; (b) the width of the basin ?*

Solution. Supply of water to be treated per day

$$= 1.8 \times 10^6 \text{ litres}$$

Supply of water to be treated during the detention period of 4 hours, *i.e.* the capacity of the tank

$$= \frac{1.8 \times 10^6}{24} \times 4 = 0.3 \times 10^6 \text{ litres} = 300 \text{ cu.m.}$$

Flow velocity = 8 cm/minute

Length of the tank = Flow velocity × Detention time

$$= 8 \times (4 \times 60) \text{ cm} = 1920 \text{ cm} = \textbf{19.2 m.} \quad \textbf{Ans.}$$

The cross-sectional area of the tank

$$= \frac{\text{Capacity of the tank}}{\text{Length of the tank}} = \frac{300}{19.2} \text{ m}^2 = 15.63 \text{ m}^2$$

The depth of sediment and water = 4.2 m.

The depth of sediment = 1.2 m.

The water depth = 3 m.

$$\text{The width of the tank} = \frac{\text{Cross-sectional area}}{\text{Water depth}} = \frac{15.63}{3} = 5.21 \text{ m.} \quad \textbf{Ans.}$$

Example 9.5. *A circular sedimentation tank fitted with standard mechanical sludge removal equipment is to handle 3.5 million litres per day of raw water. If the detention period of the tank is 5 hours, and the depth of the tank is 3 m, what should be the diameter of the tank ?*

Solution. Quantity of the raw water to be treated per day

$$= 3.5 \times 10^6 \text{ litres}$$

∴ Quantity of the raw water to be treated during the detention period of 5 hours, *i.e.* the capacity of the tank

$$= \frac{3.5 \times 10^6 \times 5}{24} \text{ litres}$$

$$= 728 \times 10^3 \text{ litres} = 728 \text{ cu.m.}$$

The capacity of a circular tank of depth H and dia d is given by

$$\text{Volume} = d^2 (0.011\, d + 0.785\, H) \qquad \text{Here } H = 3 \text{ m (given)}$$

$$\therefore \quad 728 = d^2 (0.011\, d + 0.785 \times 3)$$

$$= d^2 (0.011\, d + 2.255)$$

Solving this equation by Hit and Trial, we get

$$d = \textbf{17.3 m.} \quad \textbf{Ans.}$$

Example 9.6. *In a continuous flow settling tank 3 m deep and 60 m long, what flow velocity of water would you recommend for effective removal of 0.025 mm particles at 25°C. The sp. gravity of particles is 2.65, and kinematic viscosity* ν *for water may be taken as 0.01 cm²/sec.*

(Engineering Services, 1979 ; Civil Services, 1985)

Solution. The settling velocity v_s for particles of 0.025 mm (*i.e.*, < 0.1 mm) dia is given by eqn. (9.1), as

$$v_s = \frac{g}{18} (G - 1) \frac{d^2}{\nu}$$

where $g = 981 \text{ cm/sec}^2$

$G = 2.65$ (given)

$d = 0.025 \text{ mm} = 0.0025 \text{ cm}$

$\nu = 0.01 \text{ cm}^2/\text{sec}$

$$\therefore \quad v_s = \frac{981}{18} (2.65 - 1) \frac{(0.0025)^2}{0.01} \text{ cm/sec.}$$

or $\quad v_s = \textbf{0.0562 cm/sec.}$

Now, using Eqn. (9.12), we have

$$\frac{v}{v_s} = \frac{L}{H}$$

$$\therefore \quad v = v_s \cdot \frac{L}{H}$$

where L = Length of the tank = 60 m

H = Ht. of water in the tank ; assuming 0.5 m free-board from out of the total depth of 3 m of the tank, we have the water depth in the tank = H = 2.5 m

$$\therefore \qquad v = \left[0.0562 \times \frac{60}{2.5}\right] \text{ cm/sec} = 1.35 \text{ cm/sec}$$

Hence, in order to ensure effective removal of particles upto 0.025 mm, the flow velocity in the settling tank should not be more than 1.35 cm/sec. **Ans.**

Check against the scour of deposited particles

Use : $v_d = \sqrt{\frac{8 \cdot \beta}{f} \cdot g\,(G-1)d}$ (*i.e.* Eq. 9.20)

$$= \sqrt{\frac{8 \times 0.04}{0.03} \times 981\,(2.65-1)0.0025} \text{ cm/s}$$

$$= 6.56 \text{ cm/s}.$$

Since the flow velocity (v) is only 1.35 cm/s, it is much less than the scour velocity (v_d) of 6.56 cm/s, the design flow velocity is O.K.

Example 9.7. *Two primary settling basins are 26 m in diameter with a 2.1 m side water depth. Single effluent weirs are located on the peripheries of the tank.*

For a water flow of 26,000 m^3/d, calculate :

(i) Surface area and volume ;

(ii) Overflow rate in $m^3/m^2.d$;

(iii) Detention time in hours ; and

(iv) Weir loading in m^3/m.d. (*Civil Services, 1981*)

Solution. Using eqn. (9.18), we have

$$\text{Detention period} = \frac{d^2(0.011d \times 0.785H)}{Q}$$

where d = dia of tank = 26 m

H = Side water depth = 2.1 m

Q = Half the total discharge as two tanks are in operation

$$= 13000 \text{ m}^3/\text{day} = \frac{13000}{24} \text{ m}^3/\text{hr}.$$

$$\therefore \quad \text{Detention period} = \frac{(26)^2\,[0.011 \times 26 + 0.785 \times 2.1]}{\frac{13000}{24}} \text{ hr.} = 2.41 \text{ hr.}$$

Qty. of water to be treated during the detention period of 2.41 hr.

$$= \frac{13000}{24} \times 2.41 \text{ m}^3 = 1308 \text{ cu.m.}$$

(*i*) *Capacity* (*volume*) of each tank reqd. = 1308 cu. m. **Ans.**

Surface area of tank reqd.

$$= \frac{\text{Vol.}}{\text{Depth of the water}} = \frac{1308}{2.1} \text{ m}^2 = 623 \text{ m}^2. \quad \textbf{Ans.}$$

(*ii*) *Overflow rate, i.e.* discharge per unit of surface area

$$= \frac{Q}{\text{Surface area}} = \frac{13000}{623} \text{ m}^3/\text{m}^2.\text{day} = 20.87 \text{ m}^3/\text{m}^2.\text{day}. \quad \textbf{Ans.}$$

(*iii*) *Detention time* is already determined as 2.41 hr. **Ans.**

(*iv*) *Weir loading*

Length of the weir along periphery of the tank

$$= \pi \cdot d = 3.14 \times 26 = 81.64 \text{ m}.$$

Weir loading per day

$$= \frac{\text{Discharge per day}}{\text{Length of weir}} = \frac{13000}{81.64} \text{ m}^3/\text{m.day}$$

$$= 159.23 \text{ m}^3/\text{m.day}. \quad \textbf{Ans.}$$

Example 9.8. *A rectangular sedimentation basin is to handle 10 million litres/day of raw water. A detention basin of width to length ratio of $\frac{1}{3}$ is proposed to trap all particles larger than 0.04 mm in size. Assuming a relative density of 2.65 for the particles and 20°C as the average temperature, compute the basin dimensions. If the depth of the tank is 3.5 m, calculate the detention time.*

(*Civil Services, 1990*)

Solution. Settling velocity

$$= v_s = 418\,(G-1)\,d^2 \left(\frac{3T+70}{100}\right) \text{ for } d < 0.1 \text{ mm} \quad (\textit{i.e.} \text{ Eq. 9.7})$$

$$= 418\,(2.65-1) \times (0.04)^2 \left(\frac{130}{100}\right) \text{ mm/sec.}$$

$$= 1.435 \text{ mm/sec} = 0.1435 \text{ cm/sec.}$$

Also, $\frac{v}{v_s} = \frac{L}{H}$ (*i.e.* Eq. 9.12)

$$\therefore \qquad v = 0.1435\,\frac{L}{H} \text{ cm/sec.}$$

where v is the max. flow vel. in the tank.

Also L = Flow vel. × Detention time (t)

or $$L = \left[0.1435\,\frac{L}{H}(t \times 60 \times 60)\right] \text{ cm}$$

where t is in hr.

or $$L = \left[0.1435\,\frac{L}{H} \times \frac{3600\,t}{100}\right] \text{ m}$$

or $$L = 5.164\,\frac{L\,t}{H}$$

or $$t = \frac{H}{5.164} \qquad \ldots(i)$$

Also, the capacity of the tank of t hr detention period

$$= \frac{10 \times 10^6}{10^3} \times \frac{t}{24} \text{ m}^3 = 416.67\,t$$

or B.L.H. $= 416.67\,t$

where $L = 3\,B$ (given) $\qquad \ldots(ii)$

$$\therefore \quad 3 \cdot B^2 \cdot H = 416.67\,t$$

Putting values of t from (i) into (ii), we get

$$3\,B^2\,H = 416.67 \times \frac{H}{5.164}$$

$\therefore$ $B = 5.19$ m ; say 5.2 m.
and $L = 15.6$ m. **Ans.**

When H = Depth, *i.e.* Height of the tank = 3.5 m (given)
t = ?

$$t = \frac{H}{5.164} = \frac{3.5}{5.164} = 0.68 \text{ hr.} = \mathbf{41\ minutes.}\quad \mathbf{Ans.}$$

Example 9.9. *A settling basin is designed to have a surface overflow rate of 32.6 m/day. Determine the overall removal obtained for a suspension with size distribution given below. The specific gravity of the particles is 1.2 and the water temperature is 20°C, at which the dynamic viscosity is 1.027 centipoise and the density 0.997 g/cm³.*

Particle size, mm	*0.10*	*0.08*	*0.07*	*0.06*	*0.04*	*0.02*	*0.01*
Weight fraction greater than size (per cent)	*10*	*15*	*40*	*70*	*93*	*99*	*100*

(Engineering Services, 1992)

Solution. The values of the settling velocity (v_s) for the given particle sizes of 0.1 mm, 0.08 mm, 0.07 mm, 0.06 mm, 0.04 mm, 0.02 mm and 0.01 mm are first of all computed using Eq. (9.1) (*i.e.* Stoke's law—applicable, since particle size is ≤ 0.1 mm). From Eq. (9.1), we have

$$v_s = \frac{g}{18}(G-1)\frac{d^2}{\nu}$$

Here $g = 9.807$ m/s²
$G = 1.2$

ν = kinematic viscosity = $\dfrac{\mu}{\rho_w}$

Here $\mu = 1.027$ Centi Poise
$= 1.027 \times 10^{-2}$ Poise
But 10 Poise = 1 Ns/m²

$$\therefore\quad \mu = \frac{1.027\times10^{-2}}{10} \text{ Ns/m}^2$$
$$= 1.027 \times 10^{-3} \text{ Ns/m}^2$$
$$\rho_w = 0.997 \text{ g/cm}^3 \text{ (given)}$$
$$= 997 \text{ kg/m}^3$$
$$\therefore\quad \nu = \frac{\mu}{\rho_w} = \frac{1.027\times10^{-3} \text{ Ns/m}^2}{997 \text{ kg/m}^3}$$
$$= 1.03 \times 10^{-6} \frac{\frac{\text{kgm}}{\text{s}^2}\cdot\frac{\text{s}}{\text{m}^2}}{\text{kg/m}^3} \quad (\because\ 1\text{ N} = 1\text{ kgm/s}^2)$$
$$= 1.03 \times 10^{-6} \text{ m}^2/\text{s}$$

$$\therefore\quad v_s = \left[\frac{9.807}{18}(1.2-1)\frac{d^2}{1.03\times10^{-6}}\right] \text{ m/s} \qquad \text{(where } d \text{ is in m)}$$

or $$v_s = 0.1058 \times 10^6\, d^2 \qquad \text{(where } d \text{ is in m)}$$

If d is in mm, then the above eqn. becomes

$$v_s = 0.1058\, d^2_{mm} \text{ (m/s)}$$
$$= 0.1058 \times 10^3\, d^2_{mm} \text{ (mm/s)}$$

or $$\boxed{v_s = 105.8\, d^2}$$

where v_s is in mm/s and d is in mm

The values of v_s are now computed for all the values of particle sizes (d in mm) using eqn. $v_s = 105.8\, d^2$, as shown in table 9.2.

Table 9.2

Particle size in mm (d)	0.10	0.08	0.07	0.06	0.04	0.02	0.01
Wt. fraction greater than the stated size (%)	10	15	40	70	93	99	100
Settling velocity (v_s) in mm/s = 105.8 d^2	1.058	0.677	0.518	0.381	0.169	0.042	0.011
Wt. fraction less than the stated size or stated settling velocity (%)	90	85	60	30	7	1	0

The cumulative distribution curve is now plotted for the values of v_s and X (wt. fraction less than the stated size or stated values of v_s), as shown in Fig. 9.9.

The value of v_o *i.e.* surface overflow rate, given as 32.6 m/day, is now computed in units of mm/s, as

$$v_s = 32.6 \text{ m/day}$$
$$= \frac{32.6\times1000}{24\times60\times60} \text{ mm/s} = \mathbf{0.377\ mm/s.}$$

This value of v_0 is marked on the plotted curve (Fig. 9.9) and a vertical line is drawn to measure $X_0 = 29\%$. The %age of particles, having settling velocity greater than 0.377 mm/s, will, thus, be = 100 – 29 = 71%. Hence, minimum of 71% of suspended particles will be removed in the settling tank (whether it be an upflow one or of any other type). However, additional partial removal of smaller particles will also occur in the case of an ideal horizontal rectangular tank. Let us calculate that removal also.

The %age of smaller particles removed is given as :

$$\int_{X=0}^{X=X_0=29\%} \frac{v_s}{v_o} \times 100\ .\ dX, \text{ which approximately equals to}$$

$$\frac{1}{v_o}\ \Sigma\, v_s\ .\ \Delta X$$

Let us divide X values between 0 to 29% into intervals of say 5% each, giving ΔX

$$\therefore \quad \Delta X = 5, 5, 5, 5, 5, 4.$$

The values of v_s corresponding to X values of 5, 10, 15, 20, 25 and 29% are read out from the plotted curve as 0.14, 0.21, 0.27, 0.32, 0.35, 0.377 (mm/s). The values of $v_s \cdot \Delta X$ are now computed as :

$$0.14 \times 5 = \mathbf{0.7}, 0.21 \times 5 = \mathbf{1.05}, 0.27 \times 5 = \mathbf{1.35},$$

$$0.32 \times 5 = \mathbf{1.60}, 0.35 \times 5 = \mathbf{1.75} \text{ and } 0.377 \times 4 = \mathbf{1.51}.$$

Their sum total is

$$= \Sigma v_s \cdot \Delta X = 0.7 + 1.05 + 1.35 + 1.60 + 1.75 + 1.51 = 6.45$$

$$\therefore \quad \frac{1}{v_o} \cdot \Sigma v_s \cdot \Delta X = \frac{6.45}{0.377} = 17\%.$$

It means that lighter solids, amounting to a total 17% (out of 29% in all) will also get removed in an ideal settling basin, making overall solids removal = 71% + 17% = 88%. **Ans.**

9.6. Tube Settlers

In the analysis of the settling of the discrete particles in a sedimentation tank, we have seen that the settling efficiency is primarily dependent upon the surface area of the tank and is independent of its depth. Therefore, the sedimentation basin should be made as shallow as possible. However, due to land constrictions, the surface area has to be limited, and obviously, the tank volume is made up by providing sufficient water depth in the tank. Due to such practical problems, water depth of 3 to 5 m is usually provided in sedimentation tanks.

Attempts have, therefore, been made to place horizontal trays in the tank, to divide the tank height into compartments, to reduce the depth and to provide multiple surface areas. Such an arrangement, infact, will increase the surface area, thereby reducing the surface overflow rate (v_o), and hence causing more particle settlement. Placement of such wide shallow trays inserted within the conventional basins, have however, not been very successful, as they had developed a number of problems, like clogging and odours due to biological growth, build up of oil and grease, etc.

However, very small diameter tubes having high wetted perimeter relative to wetted area, providing laminar flow conditions and low surface loading rate, have shown good promise. Such tube settling devices, called **tube settlers**, have been found to provide excellent clarification with detention times of equal to or less than 10 minutes.

The tubes to be used may be square or circular or of any other section, and may be made of thin plastic sheets (1.5 mm thick), black in colour, though plastic and asbestos cement pipes have also been used. A large number of such tubes, of say 50 mm × 50 mm or smaller in size, of ht. say 1 m or so, are placed together side by side to make a bundle, either rectangular or circular or square in shape, called a *module* or a ***tube settler module***. Its dimension may be say 5 m × 5 m, or so. The height of this module may be of the order of 1 m or so. Tubes can be placed either horizontally or steeply inclined. In inclined tubes (about 60° to horizontal), continuous gravity drainage of the settleable

material can be obtained ; whereas in horizontal tubes (normally inclined at 5°), auxiliary scouring of settled solids will become necessary. The achieved advantage of course, decreases with the increase in the angle of inclination (θ) of the tubes with the horizontal (See Fig. 9.14), and the efficiency decreases rapidly at angles greater than 40° ; but the sludge will not slide automatically with angles less than 60°. Hence, 60° inclined tube settlers are preferred, and have been found to be quite effective in improving the performance of the existing plants as well as in making new plants. It has been claimed that if one-fifth of the outlet end of the basin is covered with tube settlers, the effective surface loading on the tank is nearly halved, or the flow through the tank can be doubled without impairing the effluent quality.

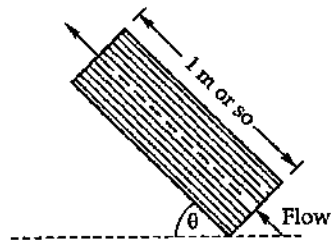


Fig. 9.14

A number of proprietory devices, such as a **Lamella clarifier** (Fig. 9.15), have been built to take advantage of the same principle.

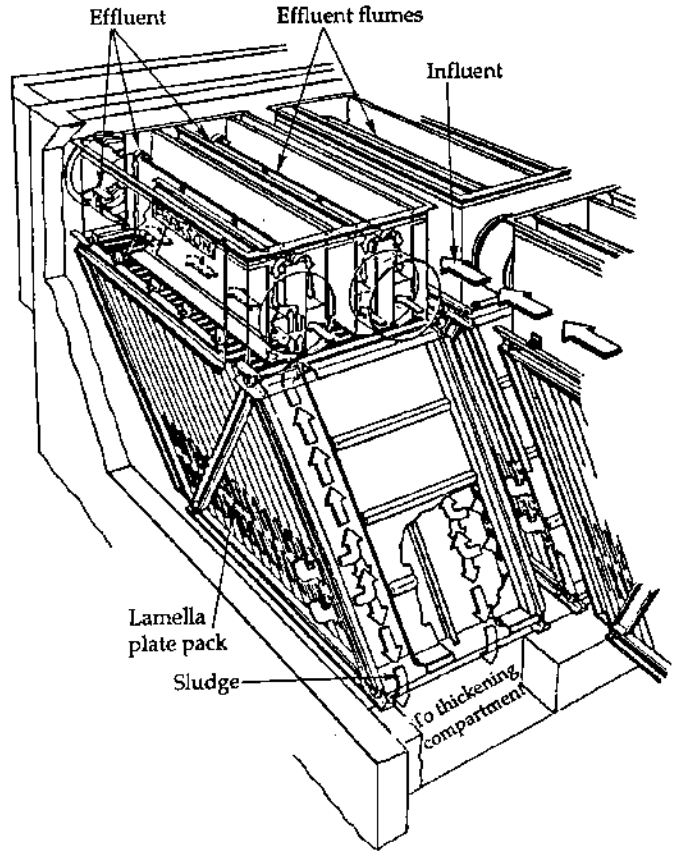


Fig. 9.15. Lamella clarifier.

9.6.1. Design of Tube Settlers. The performance of a tube settler is normally estimated by a parameter, S, defined as :

$$S = \frac{v_s}{v_o} (\sin \theta + L \cos \theta) \qquad ...(9.21)$$

where v_s = settling velocity of the particle in a vertically downward direction

v_o = velocity of flow along the tube settler

θ = angle of inclination of the tube settler with the horizontal

L = relative effective length of the settler (dimensionless)

$= \frac{l}{d}$; where

l = length of the tube settler

d = dia (width) of the tube settler.

If the value of S equals or exceeds a critical value (S_c) for any particle, it is completely removed in the tube settlers under ideal conditions. For laminar flow regime in tube settlers, the value of S_c have been determined to be $\frac{4}{3}$, $\frac{11}{8}$ and 1 for circular, square, and parallel plates type of tube settlers, assuming uniform flow.

It is also found that their performance is significantly improved with L values of upto 20, and insignificantly beyond 20. It will, therefore, be desirable to design the tube settlers with L values around 20 but less than 40.

It has further been recommended to increase the dimensionless length L of the tube settlers by an additional amount L', to account for transition zone near inlet to change to fully developed laminar flow. Thus, $L + L' = \frac{l}{d}$; L' being given by Eqn.

$$L' = 0.058 R_e$$

where R_e = Reynold number $= \frac{v_o \cdot d}{\nu}$

$$L' = 0.058 \frac{v_o \cdot d}{\nu} \qquad ...(9.22)$$

This factor reduces the effective length of settler $(L) = \left(\frac{l}{d} - L'\right)$

Design Example 9.10. *Design a tube module of square cross-section with the following data :*

(i) Average output required from the tube settler = 200 m^3/hr

(ii) Loss of water in desludging = $2\frac{1}{2}$ % of output required.

(iii) Settling velocity of smallest particle to be removed = 120 m/d.

(iv) Cross-section of square tubes = 50 mm × 50 mm.

(v) Lengths of tubes = 1 m.

(vi) Angle of inclination of tubes = 60°.

(vii) Kinematic viscosity of water = 1.01×10^{-6} m^2/s.

Solution.

Average output reqd = 200 m^3/hr

Loss of water in desludging = 2.5% of reqd output

= 2.5% × 200 m^3/hr = 5 m^3/hr

∴ Average design flow = 200 + 5 = 205 m^3/hr.

Relative length of settler $= \frac{l}{d} = \frac{1 \text{ m}}{0.05 \text{ m}} = 20$

Let us increase L by L', where

$$L' = 0.058 \cdot \frac{v_o \cdot d}{\nu} \qquad (i.e. \text{ Eq. } 9.22)$$

$$= \frac{0.058 \times v_o \times 0.05}{1.01 \times 10^{-6}} \text{ ; where } v_o \text{ is in m/s}$$

$$= 2.87 \times 10^3 v_o \text{ ; where } v_o \text{ is in m/s}$$

$$= \frac{2.87 \times 10^3 v_o}{24 \times 60 \times 60} \text{ ; where } v_o \text{ is in m/day}$$

$$= 0.033 v_o$$

∴ $L' = 0.033 v_o$

where v_o is flow-through velocity for tube settler in m/d.

Effective relative length of the settler

$$L = \frac{l}{d} - L'$$

or $L = 20 - 0.033 v_o$.

Now, using Eq. (9.21),

$$S_c = \frac{v_s}{v_o} [\sin \theta - L \cos \theta], \text{ we have}$$

The critical value of S for square type of tube settlers is $\frac{11}{8}$ (as stated earlier in the text).

$$\therefore \quad \frac{11}{8} = \frac{120 \text{ m/d}}{v_o} [\sin 60° + (20 - 0.033 v_o) \cos 60°]$$

or $$\frac{11}{8} \times \frac{v_o}{120} = [0.866 + (20 - 0.033 v_o)\, 0.5]$$

or $$0.01146 v_o = [0.866 + 10 - 0.0165 v_o]$$

or $$0.02796 v_o = 10.866$$

$$v_o = 388.6 \text{ m/d.}$$

Total entrance area of tubes

$$= \frac{Q}{v_o} = \frac{205 \times 24 \text{ m}^3/\text{d}}{388.6 \text{ m/d}} = 12.66 \text{ m}^2$$

No. of tubes required (size 0.05 m × 0.05 m)

$$= \frac{12.66}{0.05 \times 0.05} = 5064.$$

Let us provide 72 tubes along the length and 72 tubes along the width of a square module, with total tubes = 72 × 72 = 5184 tubes.

Length of tube module

= No. of tubes [inside dimension of square tube + 2 thickness of each tube] using 1.5 mm thick tubes

= 72 [0.05 + 2 × 0.0015] m

= 3.816 m ; say 3.82 m

Height of tube module for 1 m length of tubes inclined at 60°

= 1 sin 60° = 0.866 m ; 0.87 m (say).

∴ Overall dimension of tube module = 3.82 m × 3.82 m × 0.87 m. **Ans.**

SEDIMENTATION AIDED WITH COAGULATION

As pointed out earlier, very fine suspended mud particles and the colloidal matter present in water cannot settle down in plain sedimentation tank of ordinary detention period. Such particles can, however, be removed easily by increasing their size by changing them into flocculated particles. For this purpose, certain chemical compounds, called ***coagulants***, are added to the water, which on thorough mixing, form a *gelatenous precipitate* called ***'floc'***. The very fine colloidal particles present in water, get attracted and absorbed in these flocs, forming the bigger sized flocculated particles.

The colloidal particles do, infact, possess surface charges resulting from preferential adsorption or from ionisation of chemical groups on the surface. Most of the colloidal particles in water or waste water are negatively charged. The stationary charged layer on the surface is surrounded by a bound layer of water, as shown in Fig. 9.16.

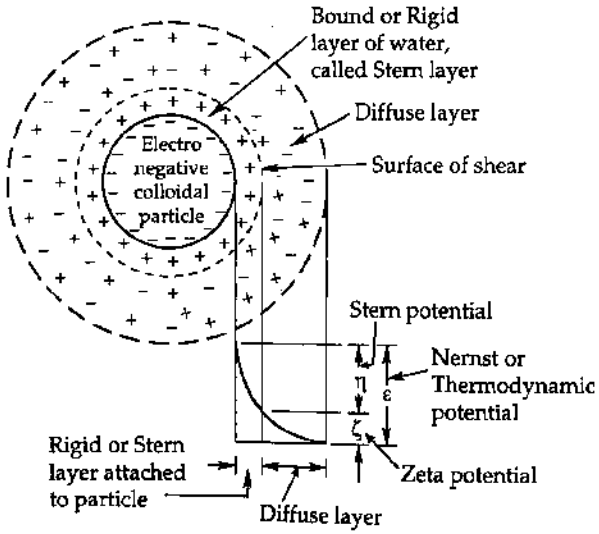


Fig. 9.16. Model of a colloidal particle.

In this bound layer, called the *Stern layer*, ions of opposite charge drawn from the bulk solution, produce a rapid drop in potential, called the Stern potential (η). A more gradual drop, called the *Zeta potential* (ζ) occurs between the shear surface of the bound water layer and the point of electro neutrality in the solution, as shown in Fig. 9.16.

The surface charge on colloidal particles gives them long-term stability ; and hence the particles which might otherwise settle or coalesce are mutually repelled by their like charges. **Coagulation** is a chemical technique which is directed towards the destabilisation of the charged colloidal particles. **Flocculation,** on the other hand, is the slow mixing technique which promotes the agglomeration of the destabilised particles.

For all practical purposes, however, the entire process of addition of chemicals (coagulants) and mixing (flocculation) is usually referred to as coagulation. The coagulated water is finally made to pass through the sedimentation tank, where the flocculated particles settle down, and are thus removed.

The use of coagulants is generally necessary for clarifying raw waters containing turbidities greater than 30 to 50 mg/l, but in actual practice, plain sedimentation is rarely used these days, and *the coagulation before sedimentation is almost universally adopted in all the major water treatment plants, and is followed by rapid sand filtration**.

9.7. Analysis of Flocculent Settling (Type 2 Settling)

Flocculent particles such as those resulting from coagulation, will agglomerate (coalesce) while settling, with a result in increase in the particle size. The density of the composite particle (floc) may decrease due to the inclusion of water, but the overall result is generally an increase in the settling velocity. A suggested equation for the increase in the floc density with particle size is :

$$\rho_f = \rho_w + Kd^{-0.7} \qquad ...(9.23)$$

where ρ_f = density of the floc

ρ_w = density of water

d = dia of particle, or dia of sphere with equivalent surface area

K = a coefficient depending upon the characteristics of the water and the chemicals used.

To determine the removal efficiency of a flocculent suspension, adequate mathematical equations do not exist ; and ***settling column analysis*** of such suspensions are performed. This analysis is done in a test cylinder (settling column) of atleast 300 mm diameter and having a depth equal to the proposed settling tank. The column, usually, has ports (openings) at 0.6 m interval for withdrawal of samples, as shown in Fig. 9.17. The flocculent suspension for which the settling characteristics are to be determined, is introduced into the column in such a way that a uniform distribution of particle size occurs from top to bottom. The settling is allowed to occur under quiescent conditions and

*Discussed later under **FILTRATION**.

at constant temperature to eliminate convection currents. Samples are withdrawn at various selected time intervals from different depths and analysed to determine the S.S. Concentrations. The %age removals of suspended solids (S.S.) are computed at different times and depths, and plotted between time and depth for a fixed value of percentage removal (such as 40%, 50%, 60% etc.). Different curves will, thus, be plotted for different removal percentages, as shown in Fig. 9.18. These curves are infact plotted like contours. From this *generalised plot*, the overall removal of solids (%age) for a given settlement time t and depth h (usually 100%) can be computed by using the relation :

% removal

$$= \left[\frac{R_1 + R_2}{2} \cdot \frac{\Delta h_1}{h} + \frac{R_2 + R_3}{2} \frac{\Delta h_2}{h} + \ldots\ldots + \frac{R_n + R_{n+1}}{2} \cdot \frac{\Delta h_n}{h}\right] \quad \ldots(9.24)$$

where $R_1, R_2, \ldots\ldots R_{n+1}$ are percent removals such as 100%, 80% ..... at partial depths, such as

0%, $\frac{\Delta h_1}{h}$%, $\frac{\Delta h_2}{h}$% ..... 100%, corresponding to the given detention time, t. Eventually, R_{n+1} is the percent removal after time t and at 100% depth.

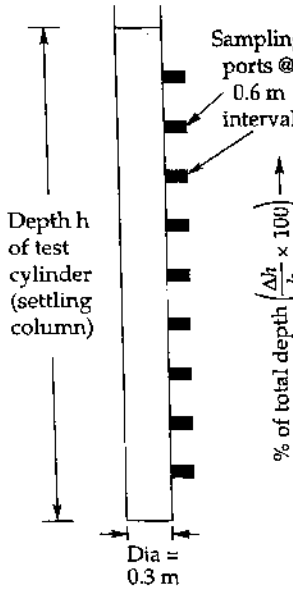


Fig. 9.17. Settling column in test cylinder.

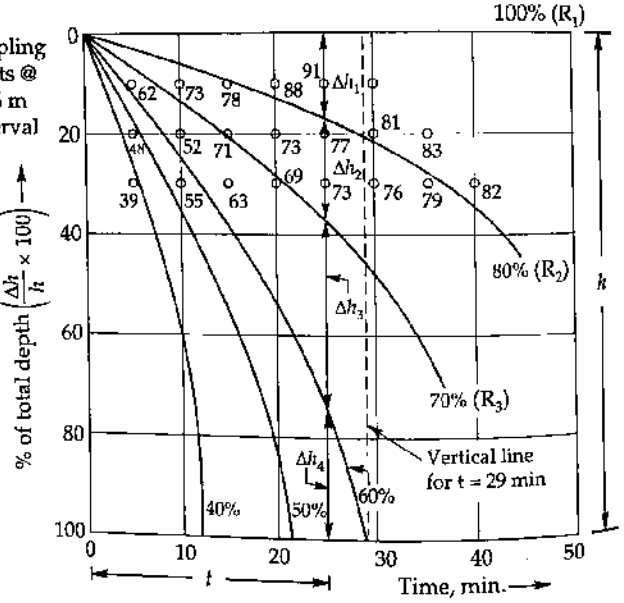


Fig. 9.18. Settling curves for flocculent particles.

The curves can also be used to determine the detention period t, depth and surface overflow rate to obtain a given percentage of removal of flocculent particles.

Since the optimum conditions will not prevail in the field as in the laboratory, the laboratory results obtained in the above test should be modified to reduce the design surface overflow rate by a dividing factor of 1.5, and increase the detention time by a factor of 2.

Example 9.11. *Using the results of the settling test shown in Fig. 9.18, determine the overall removal of solids if the detention time is 29 minutes and the tank depth equals the depth of the test column (100% depth).*

Solution. Using Fig. 9.18, we will first of all erect a vertical line on the point (X) representing time = 29 minutes on X-axis. We will then find out which one of the given curves (such as curves of 40%, 50%, 60% removals .....) do intersect the X-axis (100% depth), or is likely to intersect the X-axis. For t = 29 min, we find that the curve of 60% removal does exactly intersect the X-axis ; and hence R_{n+1} will be equal to 60%.

[**Note.** For other values of t where the curves donot exactly intersect the X-axis, their value has to be interpolated and approximated.]

Thus, after a time t = 29 min and at 100% depth, the percentage removal, *i.e.* R_{n+1} = 60% at point X.

The vertical line is erected over this point X, $\Delta h_1, \Delta h_2, \ldots\ldots \Delta h_n$ are measured in terms of total depth h as :

$$\therefore \quad \frac{\Delta h_1}{h} = 20.5\%$$

(between 100% (R_1) and 80% (R_2) removal curves)

$$= 0.205$$

$$\frac{\Delta h_2}{h} = 47\% \quad \text{(between 80\% } (R_2) \text{ and 70\% } (R_3) \text{ removal curves)}$$

$$= 0.47$$

$$\frac{\Delta h_3}{h} = 100 - 47 = 53\%$$

(between 70% and 60% (R_4) removal curves)

$$= 0.53$$

The overall removal of solids assuming linear variation between contours is given as :

$$= \left[\sum \frac{R_n + R_{n+1}}{2} \cdot \frac{\Delta h_n}{h}\right]$$

$$= \left[\frac{R_1 + R_2}{2} \cdot \frac{\Delta h_1}{h} + \frac{R_2 + R_3}{2} \cdot \frac{\Delta h_2}{h} + \frac{R_3 + R_4}{2} \cdot \frac{\Delta h_3}{h}\right]$$

$$= \frac{100\% + 80\%}{2} \times 0.205 + \frac{80\% + 70\%}{2} \times 0.47 + \frac{70\% + 60\%}{2} \times 0.53$$

$$= 90\% \times 0.205 + 75\% \times 0.47 + 65\% \times 0.53$$

$$= 88.15\% \text{ ; say } 88\%.$$

Hence, the total removal of particles in quiescent settling of the tested flocculent suspension = **88%. Ans.**

9.8. Chemicals Used for Coagulation

Various chemicals, such as alum ; iron salts like ferrous sulphate, ferric chloride, ferric sulphate ; etc., are generally used as coagulants. *These chemicals are most effective when water is slightly alkaline.* In the absence of

such an alkalinity in raw supplies, external alkalies like sodium carbonate, or lime, etc. are added to the water, so as to make it slightly alkaline, and thus to increase the effectiveness of the coagulants. The important coagulants and the chemical reactions associated with them are described below in details :

(1) **Use of Alum as Coagulant.** Alum is the name given to the aluminium sulphate with its chemical formula as $Al_2(SO_4)_3.18\,H_2O$. The alum when added to raw water, reacts with the bicarbonate alkalinities, which are generally present in raw supplies, so as to form a gelatinous precipitate (floc) of aluminium hydroxide. This floc attracts other fine particles and suspended matter (colloids), and thus grows in size, and finally settles down to the bottom of the tank. The chemical equation is :

$$Al_2(SO_4)_3.18H_2O + 3Ca(HCO_3)_2 \longrightarrow 3CaSO_4 + \underset{\text{Al. hydroxide ppt.}}{2Al(OH)_3\downarrow} + 6CO_2\uparrow$$

From the above equation, it becomes evident that the addition of alum to water imparts permanent hardness to it, in the form of calcium sulphate. The carbon dioxide gas, which is evolved, causes corrosiveness.

From the above equation, it can also be seen that the alum reacts with bicarbonate alkalinity, although total alkalinity is a measure of bicarbonate (HCO_3^-), carbonate (CO_3^{--}), and hydroxide (OH^-) ions. Since the alkalinity in water in the form of bicarbonate alkalinity is typical at pH below 8.3* and above the pH of 4.5* or so, it *fixes a limit on maintaining an optimum pH value of about 6.5 to 8.3, which is required for the formation of a good sweep floc.*

The amount of alum required for coagulation depends on the turbidity and colour of the raw water. The use of the optimum amount of a coagulant is indicated by the formation of large feathery flakes ; and can be approximately determined by laboratory testing** which is adjusted with the actual results obtained at the treatment plant. The dose of alum may vary from 5 mg/l for relatively clear waters to 85 mg/l for highly turbid waters. The average normal dose is about 17 mg/l.

Since the molecular mass of alum is 600, and that of the 3 molecules of bicarbonate alkalinity is 300, it becomes evident that about 600 mg of alum will require about 300 mg of alkalinity, Hence, 1 mg/L of commercial alum will consume about 0.5 mg/L of alkalinity. Moreover, atleast 5–10 mg/L of alkalinity must remain after the reaction occurs to keep the pH near optimum (*i.e.*, between 6.5 to 8.3). The alkalinity of raw water must therefore, be atleast equal to half of the expected alum dose plus 5–10 mg/L. This is the thumb rule for minimum alkalinity's presence. However, if raw supplies are not sufficiently alkaline, then the external alkalies like lime [*i.e.* $Ca(OH)_2$] or soda ash [*i.e.* Na_2CO_3] are generally added, and the following reactions take place :

(*i*) $Al_2(SO_4)_3.18H_2O + 3Ca(OH)_2 \longrightarrow 3CaSO_4 + 2Al(OH)_3\downarrow + 18H_2O.$

(*ii*) $Al_2(SO_4)_3.18H_2O + 3Na_2CO_3 \longrightarrow 3Na_2SO_4 + 2Al(OH)_3\downarrow + 3CO_2\uparrow + 15H_2O.$

Alum or *filter alum* (as it is sometimes called) has proved to be a very effective coagulant, and is now extensively used throughout the world. It is quite cheap, forms an excellent stable floc, and does not require any skilled supervision for handling. The water obtained is also quite clear, as it helps in reducing taste and colour of raw water in addition to removing its turbidity.

*Pl. see Fig. 15.2.

**The jar test, normally adopted for this testing, is described in article 9.11.

The main problem in using alum till recently was that it is difficult to dewater the sludge formed and is not easy to dispose it of, as it is found unsuitable for filling low lying lands. But the recent research has, however, shown that it is possible to recover alum from this sludge, and it can be re-used for coagulation. The cost of recovery is also one-fourth of the cost of the recovered alum. It has thus solved a vital problem of its disposal. *The only drawback in the use of alum now is, that the effective pH range for its use is small, i.e. 6.5 to 8.3, and may, in many cases, require the addition of external alkali salts, thereby rendering it costlier.*

(2) **Use of Copperas as Coagulant.** Copperas is the name given to ferrous sulphate with its chemical formula as $FeSO_4.7H_2O$. *Copperas* is generally added to raw water in conjunction with lime. Lime may be added either to copperas *or vice-versa*. When lime is added first, the following reaction takes place :

(*i*) $\underset{\text{Copperas}}{FeSO_4.7H_2O} + \underset{\text{Hydrated lime}}{Ca(OH)_2} \longrightarrow CaSO_4 + \underset{\text{Ferrous hydroxide}}{Fe(OH)_2} + 7H_2O$

Similarly, when copperas is added earlier to lime, the reaction that takes place is :

(*ii*) $FeSO_4.7H_2O + \underset{\text{Alkali present in raw water}}{Ca(HCO_3)_2} \longrightarrow Fe(HCO_3)_2 + CaSO_4 + 7H_2O$

and $Fe(HCO_3)_2 + \underset{\text{Hydrated lime}}{2Ca(OH)_2} \longrightarrow \underset{\text{Ferrous hydroxide}}{Fe(OH)_2} + 2CaCO_3 + 2H_2O$

The ferrous hydroxide formed in either case, further gets oxidised, forming ferric hydroxide, as given below :

$$4Fe(OH)_2 + O_2 + 2H_2O \longrightarrow \underset{\text{Ferric hydroxide ppt.}}{4Fe(OH)_3\downarrow}$$

The ferric hydroxide forms the floc, and thus, helps in sedimentation.

Copperas is extensively used as a coagulant for raw waters that are not coloured. It is generally cheaper than alum, and functions effectively in the pH range of 8.5 and above. For coloured raw waters, it is however, not used, as it does not give satisfactory results. *The quantity of copperas required is almost the same as that of alum.*

(3) **Use of Chlorinated Copperas as Coagulant.** When chlorine is added to a solution of copperas (*i.e.*, ferrous sulphate), the two react chemically, so as to form ferric sulphate and ferric chloride. The chemical equation is as follows :

$$\underset{\text{Copperas}}{6(FeSO_4.7H_2O)} + 3Cl_2 \longrightarrow \underset{\text{Ferric sulphate}}{2Fe_2(SO_4)_3} + \underset{\text{Ferric chloride}}{2FeCl_3} + 42H_2O$$

The resultant combination of ferric sulphate and ferric chloride is known as *chlorinated copperas,* and is a valuable coagulant for removing colours, especially where the raw water has a low pH value.

Both the constituents of the chlorinated copperas along with lime are effective coagulants, and their combination is often quite effective. The chemical reactions that take place are given below :

$$\underset{\text{Ferric sulphate}}{Fe_2(SO_4)_3} + \underset{\text{Hydrated lime}}{3Ca(OH)_2} \longrightarrow 3CaSO_4 + \underset{\text{Ferric hydroxide ppt.}}{2Fe(OH)_3\downarrow}$$

$$\underset{\text{Ferric chloride}}{2FeCl_3} + \underset{\text{Hydrated lime}}{3Ca(OH)_2} \longrightarrow 3CaCl_2 + \underset{\text{Ferric hydroxide ppt.}}{2Fe(OH)_3}\downarrow$$

The resulting Ferric hydroxide forms the floc, and helps in sedimentation.

Ferric sulphate is quite effective in the pH range of 4 to 7 and above 9, whereas Ferric chloride is quite effective in the pH range of 3.5 to 6.5 and above 8.5. *The combination has, therefore, proved to be a very effective coagulant for treating low pH waters.*

(4) Use of Sodium Aluminate as a Coagulant. Besides alum and iron salts, sodium aluminate ($Na_2Al_2O_4$) is also sometimes used as a coagulant. This chemical when dissolved and mixed with water, reacts with the salts of calcium and magnesium present in raw water, resulting in the formation of precipitates of calcium or magnesium aluminate. The chemical reactions that are involved are :

$$Na_2Al_2O_4 + Ca(HCO_3)_2 \longrightarrow \underset{\text{Cal. aluminate}}{CaAl_2O_4}\downarrow + Na_2CO_3 + CO_2\uparrow + H_2O$$

$$Na_2Al_2O_4 + CaCl_2 \longrightarrow CaAl_2O_4\downarrow + 2NaCl$$

$$Na_2Al_2O_4 + CaSO_4 \longrightarrow CaAl_2O_4\downarrow + Na_2SO_4$$

This coagulant is about $1\frac{1}{2}$ times costlier than alum, and is therefore, generally avoided for treating ordinary public supplies, but, however, it is very useful for treating waters which do not have the natural desired alkalinity, and thus cannot be treated with pure alum.

As is evident from the above equations, this chemical further reduces the temporary as well as the permanent hardness present in raw supplies, rather than increasing the same as is done by alum. This coagulant is, therefore, widely used for treating boiler feed waters, which permit very low values of hardness.

9.10.1. Comparison of Alum and Iron Salts (as Coagulants). The alum and the iron salts are having their own advantages and disadvantages, as summarised below :

(*i*) Iron salts produce heavy floc and can, therefore, remove much more suspended matter than the alum.

(*ii*) Iron salts, being good oxidising agents, can remove hydrogen sulphide and its corresponding tastes and odours from water.

(*iii*) Iron salts can be used over a wider range of pH values.

(*iv*) Iron salts cause staining and promote the growth of iron bacteria in the distribution system.

(*v*) Iron salts impart more corrosiveness to water than that which is imparted by alum.

(*vi*) The handling and storing of iron salts require more skill and control, as they are corrosive and deliquescent. Whereas, no such skilled supervision is required for handling alum.

Note. Iron salts are used as coagulants more frequently for treating sewage, and alum is used more frequently for treating raw waters.

Example 9.12. *Determine the quantity of alum required in order to treat 13 million litres of water per day at a treatment plant, where 12 ppm of alum dose is required. Also determine the amount of carbon dioxide gas which will be released per litre of water treated.*

Solution.

Quantity of water to be treated = 13×10^6 litres/day.

Alum dose required = 12 ppm, *i.e.*, 12 mg/l.

Amount of alum reqd. per day = $(13 \times 10^6 \times 12)$ mg. = **156 kg.** **Ans.**

The chemical reaction which is involved in treating water with alum is given by

$$Al_2(SO_4)_3.18H_2O + Ca(HCO_3)_2 \longrightarrow 2Al(OH)_3\downarrow + 3CaSO_4 + 18H_2O + \underset{\text{Carbon dioxide}}{6CO_2}\uparrow$$

Now, the molecular mass of alum

$= 2[26.97] + 3[32.066 + 4 \times 16] + 18[2 \times 1.008 + 16]$

$= 666.426 = \mathbf{666}$ (say).

The molecular mass of carbon dioxide

$= (12.01) + 2(16) = 44.01 = \mathbf{44}$ (say).

Now, it means that 666 mg of alum, if used, will release

$= 6 \times 44$ mg of CO_2

$\therefore$ 12 mg of alum will release

$$= \frac{6 \times 44}{666} \times 12 \text{ mg of } CO_2 = 4.76 \text{ mg.}$$

Since 12 mg of alum is required for treating 1 litre of water, the quantity of CO_2 evolved per litre of water = **4.76 mg.** **Ans.**

Example 9.13. *8 mg of copperas is consumed with lime at a coagulation basin, per litre water. Determine the quantity of copperas and the quick lime required to treat 10 million litres of water.*

Solution.

Quantity of copperas required per litre of water = 8 mg

Quantity of copperas required per 10 million litre of water

$= 8 \times 10 \times 10^6$ mg = **80 kg.** **Ans.**

The chemical reactions that are involved are

$$\underset{\text{Quick lime or lime}}{CaO} + H_2O \longrightarrow \underset{\text{Hydrated lime}}{Ca(OH)_2}$$

$$\underset{\text{Copperas}}{FeSO_4.7H_2O} + \underset{\text{Hydrated lime}}{Ca(OH)_2} \longrightarrow Fe(OH)_2 + CaSO_4 + 7H_2O$$

Now, molecular mass of copperas

$= [55.85 + 32.066 + 4 \times 16 + 7(2 \times 1.008 + 16)]$

$= 278.028 = \mathbf{278}$ (say)

Similarly, molecular mass of quick lime (CaO)

$= [40.08 + 16] = 56.08 = \mathbf{56}$ (say)

Since one molecule of copperas requires one molecule of lime, 278 mg of copperas will require 56 mg of quick lime.

∴ 278 mg of copperas need = 56 mg quick lime

or 1 mg of copperas need = $\frac{56}{278}$ mg quick lime

or 80 kg of copperas need = $\frac{56}{278} \times 80$ kg of quick lime

= 16.12 kg of quick lime

Hence, the quantity of quick lime required at the plant

= 16.12 kg. Ans.

Example 9.14. *A coagulation-sedimentation plant clarifies 40 million litre of water every day. The quantity of filter alum required at the plant is 18 mg/l. If the raw water is having an alkalinity equivalent to 5 mg/l of $CaCO_3$, determine the quantity of filter alum and the quick lime (containing 85% of CaO) required per year by the plant. Given the molecular weights as :*

[Al = 27, S = 32, O = 16, H = 1, Ca = 40, C = 12].

Solution. Quantity of water to be treated = 40×10^6 litres/day

∴ Quantity of filter alum required per day @ 18 mg/l

$$= 18 \times 40 \times 10^6 \text{ mg} = \frac{18 \times 40 \times 10^6}{10^6} \text{ kg} = 720 \text{ kg}.$$

∴ Quantity of filter alum required per year

= 720 × 365 kg = 262.8 Tonnes. Ans.

The chemical reactions that take place are

$$Al_2(SO_4)_3.18H_2O + 3Ca(HCO_3)_2 \longrightarrow 2Al(OH)_3 \downarrow + 3CaSO_4 + 18H_2O + 6CO_2 \uparrow$$

$$CaCO_3 + H_2O + CO_2 \longrightarrow Ca(HCO_3)_2$$

$$CaCO_3 \longrightarrow CaO + CO_2$$

The molecular masses are worked out as below :

Mol. mass of $Al_2(SO_4)_3.18H_2O$ $= 2 \times 27 + 3(32 + 64) + 18(2 + 16)$

$= 54 + 288 + 324 = \mathbf{666}$

Mol. mass of $Ca(HCO_3)_2$ $= 40 + 2(1 + 12 + 48) = \mathbf{162}$

Mol. mass of $CaCO_3$ $= 40 + 12 + 48 = \mathbf{100}$

Mol. mass of CaO $= 40 + 16 = \mathbf{56}$

Now, from the above equations, it is clear that alum requires (3 × 162) parts of natural alkalinity as $Ca.(HCO_3)_2$ for every 666 parts of alum. (3 × 162) parts of alkalinity as $Ca(HCO_3)_2$ is equivalent to (3 × 100) parts of alkalinity required as $CaCO_3$.

Hence, the alkalinity required as $CaCO_3$ for the water containing alum of 18 mg/l

$$= \frac{3 \times 100}{666} \times 18 \text{ mg/l} = 8.108 \text{ mg/l}.$$

Natural alkalinity available as $CaCO_3$ = 5 mg/l.

∴ Additional alkalinity reqd. to be added in the form of lime

$= 8.108 - 5 = 3.108$ mg/l as $CaCO_3$

Now, 100 parts of $CaCO_3$ is produced by 56 parts of CaO.

Hence, the quantity of CaO required

$$= \frac{3.108 \times 56}{100} \text{ mg/l} = 1.74 \text{ mg/l}.$$

Since quick lime contains 85% of CaO, the quick lime required

$$= \frac{1.74 \times 100}{85} \text{ mg/l} = 2.05 \text{ mg/l}.$$

∴ The quantity of lime required for treating 40×10^6 litres of water/day

$= 2.05 \times 40 \times 10^6$ mg/day $= 2.05 \times 40$ kg/day $= 82$ kg/day.

Hence, the yearly consumption of quick lime

= 82 × 365 kg = 29,930 kg = 29.93 Tonnes. Ans.

Example 9.15. *A coagulation treatment plant with a flow of 0.5 m^3/sec is dosing alum at 23 mg/L. No other chemicals are being added. The raw water suspended solids concentration is 37 mg/L. The effluent suspended solids concentration is measured as 12 mg/L. The sludge content is 1 percent and the specific gravity of sludge solids is 3.01. What volume of sludge must be disposed of each day ? Based on your result, remark with reference to water treatment process.* (*Engineering Services, 1994*)

Solution. (*i*) Alum added = 23 mg/L

Alum reacts with alkali in raw water to produce $Al(OH)_3$ solid precipitate, by the equations :

(*i*) $\underset{\text{(Alum)}}{Al_2(SO_4)_3.18H_2O} + 3Ca(OH)_2 \longrightarrow CaSO_4 + 2Al(OH)_3 \downarrow + 18H_2O$

and (*ii*) $\underset{\text{(Alum)}}{Al_2(SO_4)_3.18H_2O} + 3Na_2CO_3 \longrightarrow 3Na_2SO_4 + 2Al(OH)_3 \downarrow + 3CO_2 + 15H_2O$

It eventually means that 1 mol of *Alum* produces 2 moles of $Al(OH)_3$. The molecular mass of Alum is 666 gm and that of $Al(OH)_3$ is 78 gm.

Hence, 666 gm of alum produces

$= 2 \times 78$ gm of $Al(OH) \downarrow$ as solid sludge

∴ 1 gm of alum will produce = $\frac{2 \times 78}{666} = 0.24$ gm of $Al(OH)_3 \downarrow$

Hence, 1 mg/l of Alum will produce 0.24 mg/L of solid sludge.

(*i*) ∴ 23 mg/L of Alum will produce

$= 23 \times 0.24$ mg/L of solid sludge ...(*i*)

= 5.52 mg/l of solid sludge

(*ii*) Suspended solids (turbidity) removed ...(*ii*)

= 37 mg/L − 12 mg/l = 25 mg/L

∴ Total dry sludge (solids) removed = (*i*) + (*ii*)

= 5.52 + 25 = 30.52 mg/L

Total flow in the plant = 0.5 m^3/s

= 0.5 × 24 × 60 × 60 m^3/day = 43,200 m^3/day

Total dry sludge solids produced per day

= 43,200 m^3/d × 30.52 mg/L

= 43,200 m^3/d × 30.52 gm/m^3

= 1318464 gm/day = 1.318 t/day ...(*iii*)

∴ Volume of sludge solids produced

$$= \frac{1.318 \text{ t/d}}{\text{Density of sludge solids}} = \frac{1.318 \text{ t/d}}{3.01 \text{ t/m}^3}$$

(Sp. gr. of sludge solids = 3.01, given)

= 0.44 m^3/day ...(*iv*)

Since the sludge content is stated to be 1%, the produced wet sludge will contain 1% solids and 99% water, by weight. It means that 1 tonne of sludge solids will contain 99 tonnes of water ; and hence 1.318 t/day of sludge solids will contain water = 1.318 t/day × 99 = **130.48 t/day.**

∴ Volume of water contained in the wet sludge

$$= \frac{130.48 \text{ t/day}}{1.0 \text{ t/m}^3} \quad (\because \text{ Density of water} = 1 \text{ t/m}^3)$$

= 130.48 m^3/day ...(*v*)

Total volume of wet sludge = (*iv*) + (*v*)

= (0.44 + 130.48) m^3/day = **130.92 m^3/day. Ans.**

Disposal of such a heavy quantity of water sludge daily will be a difficult task, and hence it would be advisable to take this sludge to the *clarifier thickener*. The thickened sludge can then be dewatered and disposed of with or without *chemical recovery*.

It may also be mentioned here that 12 mg/L of suspended solids (about 30%) are still left in the effluent from the *clariflocculators,* and hence it must be subjected to *filtration*. The filter waste sludge can also be taken to the *clarifier-thickener* along with the above sludge of clariflocculator.

The filtered water shall finally be chlorinated for *disinfection*, before supplying to the public. **Ans.**

Note. It can be seen from the above computations that the solids account for a negligible fraction of the total volume of sludge ; and that is why sludge dewatering and drying becomes an important requirement for the safe disposal of the sludge from a water treatment plant.

9.9. The Constituents of a Coagulation Sedimentation Plant

The coagulation sedimentation plant, sometimes called simply *a coagulation plant* or *a* **clariflocculator**, contains the following four units :

(1) Feeding device ;

(2) Mixing device or mixing basin ;

(3) Flocculation tank or flocculator ;

(4) Settling or sedimentation tank.

A clariflocculator containing all these four units, as manufactured by the Dorr Co., is shown in Fig. 9.19.

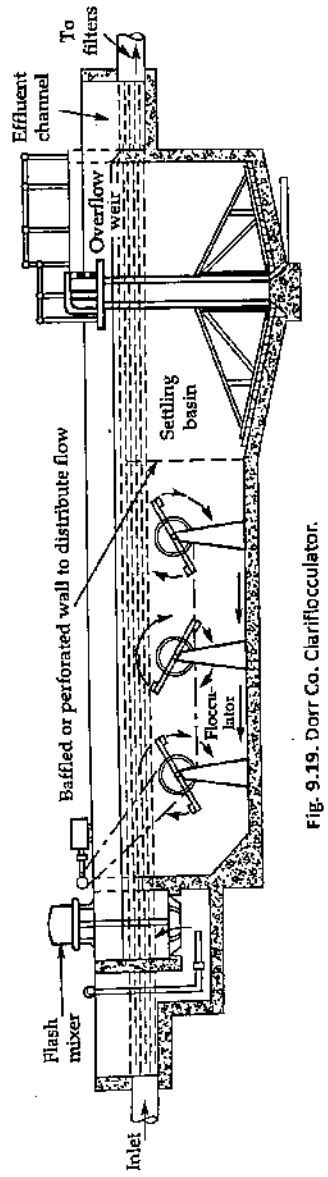


Fig. 9.19. Dorr Co. Clariflocculator.

The chemical coagulant is, first of all, fed (either dry or in solution form) into the raw water through the *feeding device*. This mixture is then thoroughly mixed and agitated in the *mixing basin*. The 'floc', which is formed as a result of the chemical reaction taking place in the mixing basin, is then allowed to consolidate in the *flocculation tank*. The flocculated water is finally passed into the *sedimentation tank* where these flocculated particles settle down and be removed. The resultant water of low turbidity can be taken out through the outlet of the sedimentation tank, directly to the rapid gravity filters, which do not function well with turbid waters expected from plain sedimentation. *The complete process of coagulation-sedimentation may help in removing turbidities up to as low values as 10—20 mg/l. It may also help in reducing the bacteria from the water and, thus, to reduce the B-Coli index by as much as 70%.*

The requirements, design and other details of the different units of a coagulation-sedimentation basin are described below in details :

9.9.1. Feeding Devices. As pointed out earlier, the chemical coagulant may be fed into the raw water either in a powdered form or in a solution form. The former is known as *dry feeding*, and the latter is known as *wet feeding*. Wet feeding equipments are generally costlier than the dry feeding equipments, but they have the advantage that they can be easily controlled and adjusted. The choice between these two types of equipments depends upon the following factors :

(*a*) *The characteristics of the coagulant and the convenience with which it can be applied.* Chemicals which clog or which are non-uniform in composition cannot be fed by dry feeding. For example, alum being fairly fine and uniform in size, can be fed easily by dry feeding, but copperas may give trouble in dry feeding, because the water of crystallisation present in it, may change with temperature, thereby turning it into a solid or a sticky mass. Similarly,

hydrated lime cannot be fed by dry feeding, because it may bridge the orifices by taking the atmospheric moisture.

On the other hand, there are certain chemicals like activated carbon, which cannot be fed into the solution form, and dry feeding has to be adopted for using such chemicals.

(*b*) *The amount of the coagulant to be used.* The amount of the coagulant to be used is an important factor in choosing the type of feeding arrangement. For example, if the dose of the coagulant is very small, then for reasons of accuracy, it must be fed in a solution form.

(c) *The cost of the coagulant and the size of the plant.* In a plant which uses a great deal of coagulant, the chemicals should be purchased in its cheapest form and the plant should be equipped to use the chemical in that form. The cost of the feeding machine is, therefore, less important as compared to the cost of the coagulant in a large plant. Whereas, if the plant is small, the cost of the feeding equipment may become the governing factor, and in that case, the chemicals may be purchased in the dry form, because dry fed machines are cheaper.

The wet feeding equipments, in addition, require making and preparing solutions of required strength, and to keep them available when needed. Sufficient watch and ward staff is, therefore, required for this additional work, which is easily and cheaply available at a large plant. *Hence, large plants naturally use wet feeding, and smaller plants utilise dry feeding, unless objected to by the characteristics of the coagulant.*

The two types of feeding devices are now discussed below :

(*i*) **Dry feeding devices.** The common devices which are used for dry feeding of the coagulants are shown in Fig. 9.20 (*a*) and (*b*).

They are in the form of a tank with a hopper bottom. Agitating plates are placed inside the tank, so as to prevent the arching of the coagulant. The coagulant, in the powdered form, is filled in the tank, and is allowed to fall in the mixing basin. Its dose is regulated by the speed of a toothed wheel [Fig. 9.20 (*a*)], or a helical screw [Fig. 9.20 (*b*)]. The speed of the toothed wheel or the helical screw is, in turn, controlled by connecting it to a venturi-device

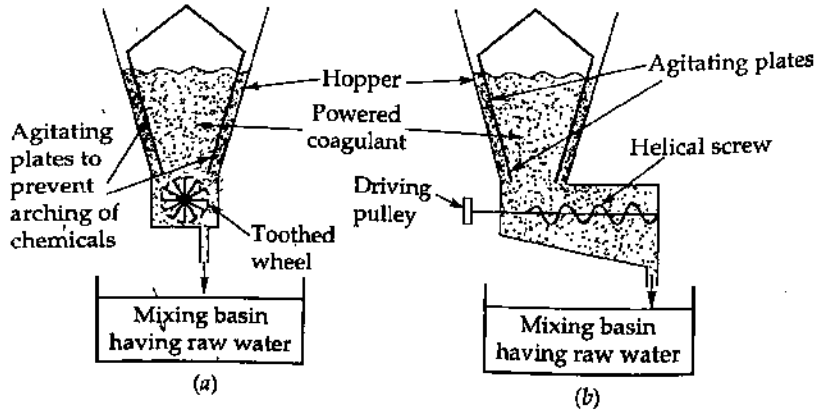


Fig. 9.20. Dry feeders.

installed in the raw water pipes bringing water to the mixing basin. The quantity of the coagulant released is, thus, controlled in proportion to the quantity of the raw water entering the mixing tank.

(*ii*) **Wet feeding devices.** In wet feeding, the solution of required strength of coagulant is prepared and stored in a tank, from where it is allowed to trickle down into the mixing tank through an outlet. The level of coagulant solution in the coagulant feeding tank is maintained constant by means of a float controlled valve, in order to ensure a constant rate of discharge for a certain fixed rate of raw water flow in the mixing basin. When the rate of inflow of raw water changes, the rate of outflow of coagulant must also change. In order to make these two flows in proportion to each other, 'a conical plug type arrangement' such as shown in Fig. 9.21, may be used.

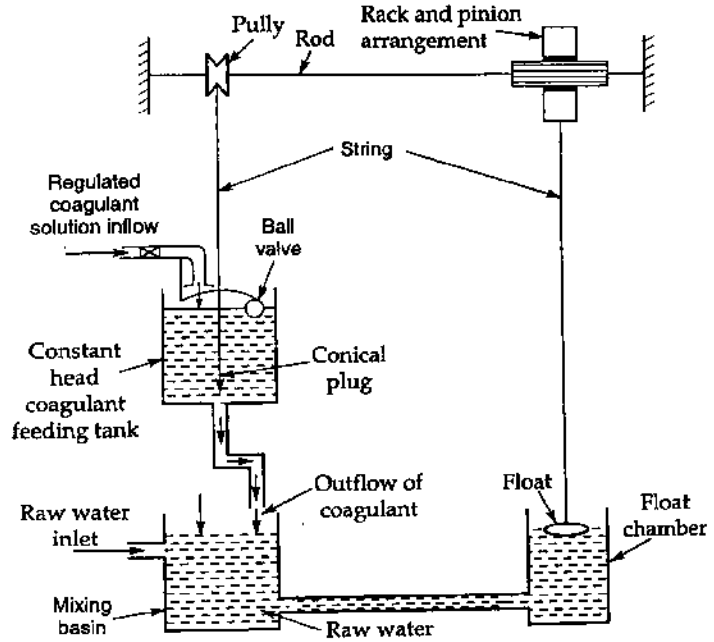


Fig. 9.21. Wet feeding by Conical plug and float arrangement.

The working of a conical plug type arrangement is very simple. The mixing basin and the float chamber are interconnected together, so that the water level remains the same in both of them. As the flow of raw water increases, the depth of water, and therefore, its level increases in the mixing tank. Correspondingly, the water level in the float chamber increases and thereby, lifting the float of the float chamber. As the float rises, the *pinion and pulley* rotates in the same direction, thereby lifting the conical plug and allowing more quantity of coagulant solution to fall down into the mixing basin. When the flow of water decreases, the conical plug, descends down and allows the feeding to continue at a lower rate. The *constant head coagulant feeding tank*, thus, automatically controls the dose of coagulant.

9.9.2. Mixing Devices. After the addition of the coagulant to the raw water, the mixture is thoroughly and vigorously mixed, so that the coagulant gets fully dispersed into the entire mass of water. This violent agitation of water can be achieved by means of *mixing devices*, such as, centrifugal pumps, compressed air, mixing basins, etc. Out of these devices, mixing basins are most important and normally adopted.

There are two types of **mixing basins**, *viz.*

(*a*) *mixing basins with baffle walls* ; and

(*b*) *mixing basins equipped with mechanical devices.*

They are described below in details :

9.9.2.1. Mixing basins with baffle walls. *The baffle type* mixing basins are rectangular tanks which are divided by baffle walls. The baffles may either be provided in such a way as the water flows horizontally *around their ends* (as shown in Fig. 9.22) ; or they may be provided as to make the water move vertically *over and under* the baffles (as shown in Fig. 9.23).

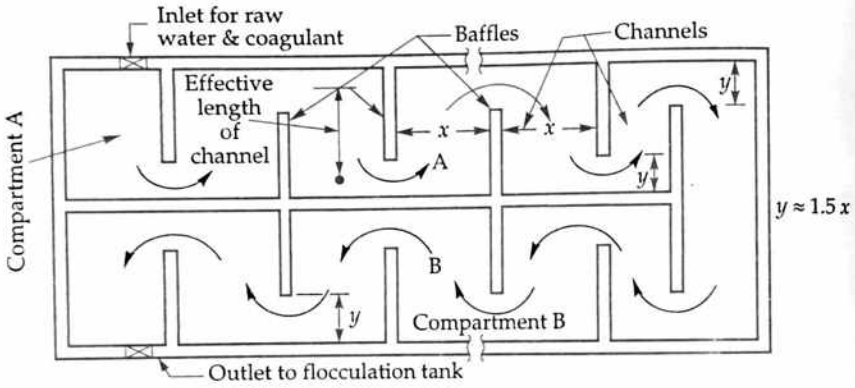


Fig. 9.22. Plan of "Around the end baffle type" (*i.e.* Horizontal flow type) mixing basin.

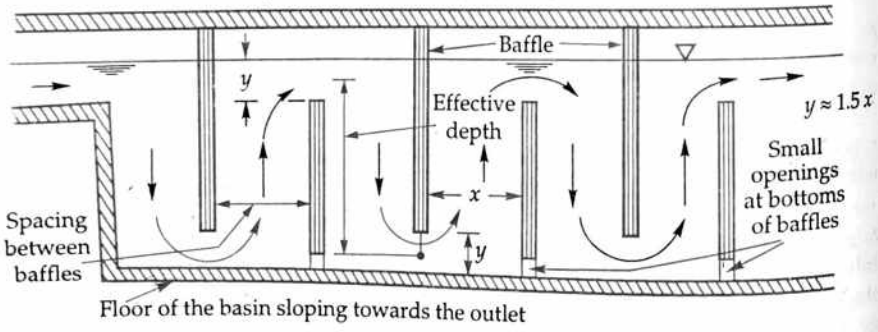


Fig. 9.23. Sectional elevation of "Over and under the baffle type" (vertical flow type) mixing basin.

The hindrances and the disturbances created by the provision of baffles in the path of flow, give it sufficient agitation, as to cause necessary mixing to develop the floc. The flocculation energy is, thus, derived primarily from the

180° change in the direction of flow at each baffle. The basis of design for both types of baffled basins remain the same. *Yet however, the first type, i.e. horizontal flow baffled basins are preferred due to ease of cleaning.*

Design Criteria. The velocity of flow in the channels between baffles is controlled to a value of 0.15 to 0.45 m/sec*. The detention period is normally kept between 20 to 50 minutes. Knowing these two values, the length of flow is known. The capacity of the tank for the required detention period is also known from the water demand. The capacity divided by the length will fix up the cross-sectional area of the channel. Total length of flow can be divided by the *effective flow length of each channel* to obtain the number of channels required. (Pl. see Solved Example 9.16).

To permit easy cleaning of the channels, the distance between baffles (x) should not be less than 45 cm or so. The clear opening between the end of each baffle and the tank wall (or roof or top water surface or the floor in the case of vertical floor type basins) i.e. y should be kept about 1.5 times the distance between the baffles (x), subject to a minimum value of 60 cm or so.

The baffle spacing may sometimes be increased gradually with length to achieve tapered flocculation. The floor of the channel should slope towards the outlet. The head loss across a baffled channel (basin) is in the range of 0.5–1.0 m.

For *around the end baffle* type basins, the depth can be obtained by dividing the cross-sectional area of each channel by the distance between the baffles. Since a depth less than 1 m is normally not allowed because of considerable variations in flow velocity with even slight variations in water depth, "*around the end baffles" should not be used when the computed depth* works out to be less than 1 m or so.

The width of a channel in a basin provided with "over and under baffles" is obtained by dividing the cross-sectional area of each channel by the spacing selected for the baffles. The effective depth (*i.e.* the effective length of flow of each channel) is normally kept 2 to 3 times the distance between the baffles.

The only advantage of the baffled basins is their simplicity, because there is no moving equipment and near plug flow conditions occur with low short-circuiting. Their main disadvantage lies in the fact that most head loss occurs at the 180° bends, and therefore, the value may be too high at the bends, and inadequate in the straight channel for good flocculation. Moreover, the baffle types of mixing basins are not flexible as they can operate efficiently only within narrow limits of velocity and volume of flow. *The head loss through such a basin is much greater than that in the basin equipped with mechanical mixers.* So much so that for each 180° turn, the loss of head has been calculated to be approximately equal to 3.2 times the velocity head in the channel. *Moreover, such basins often require greater quantities of coagulants as they provide less efficient mixing than what can be obtained with properly designed mechanical mixers.*

*Higher value to be adopted for higher turbidity waters and lower value for lower tubridity waters.

Baffle type mixing basins are, therefore, not used in modern days or especially on large plants. They may, however, be used on smaller plants, as they require less skilled labour in their operation.

Example 9.16. *Design the water depth for a mixing basin having around the end baffles, in order to treat 48 million litres per day of water. The tank, may be divided into two similar compartments by providing a longitudinal partition wall, and each half may have a clear width of 8 m. Assume suitable values of detention period and the flow velocity through the basin. The clear distance between the baffles may be kept as equal to min. permissible. Mention the number of channels in the tank and also the overall inside length of the tank.*

Solution. The total volume of water to be treated per day

$$= 48 \times 10^6 \text{ litres/day}$$

Assume a detention period of 30 minutes in the basin (*i.e.* somewhere between 20 to 50 minutes).

∴ The volume of water required to be

treated in 30 minutes or $\frac{1}{2}$ hr. = Capacity of the tank

$$= \frac{48 \times 10^6}{24} \times \frac{1}{2} \text{ litres}$$

$$= 1 \times 10^6 \text{ litres} = 1 \times 10^3 \text{ cu. m.} = 1000 \text{ cu. m.}$$

Further, assume the velocity of flow through the channels as 0.3 m/sec. (*i.e.* somewhere between 0.15 to 0.45 m/sec).

Now, the length of flow = Velocity × Detention period

$$= 0.3 \times (30 \times 60) \text{ m} = \mathbf{540 \text{ m}}.$$

∴ The required cross-sectional area of each channel between the baffles

$$= \frac{\text{Capacity of the tank}}{\text{Total length of flow}} = \frac{1000}{540} \text{ m}^2 = 1.85 \text{ m}^2.$$

The distance between the baffles is kept equal to the minimum permissible, *i.e.* 0.45 m.

∴ The depth of the water in the basin

$$= \frac{\text{Cross-sectional area of each channel}}{\text{Distance between baffles, } i.e. \text{ the width of each channel}}$$

$$= \frac{1.85}{0.45} \text{ m} = 4.12 \text{ m say } \mathbf{4.2 \text{ m}}.$$

The clear opening between the end of each baffle and the wall may be taken as equal to 1.5 times the distance between the baffles = 1.5 × 0.45 m = 0.67 m; say 0.7 m.

The effective length of each channel may now be taken as the average distance travelled by the water in the channel (See Fig. 9.22).

∴ The effective flow length of each channel

= clear width of either half of the tank − 2 × $\frac{1}{2}$ the opening

$$= 8 \text{ m} - 2 \times \tfrac{1}{2} \times 0.7 \text{ m} = 8 - 0.7 = 7.3 \text{ m} = \mathbf{7.3 \text{ m}}.$$

∴ No. of channels required

$$= \frac{\text{Total flow length}}{\text{Effective flow length of each channel}} = \frac{540}{7.3} = 74. \quad \textbf{Ans.}$$

Hence, there will be $\frac{74}{2} = 37$ channels in each half of the tank.

The clear length of the tank excluding the baffle walls and side walls

= The No. of channels × 0.45 m

$$= 37 \times 0.45 \text{ m} = 16.63 \text{ m}$$

Assuming the thickness of each baffle as 7.5 cm,

The overall inside length of the tank

$$= 16.63 + \left(\frac{7.5}{100}\right) \times \text{No. of baffles}$$

$$= 16.63 + \frac{7.5}{100} \times (37 - 1) = 16.63 + \frac{7.5}{100} \times 36$$

$$= 16.63 + 2.7 = \mathbf{19.33 \text{ m}}. \quad \textbf{Ans.}$$

9.9.2.2 Mixing basins equipped with mechanical devices. *The mechanically agitated* mixing basins provide the best type of mixing as also the flocculating devices. The chemical added to the raw water is vigorously mixed and agitated by a *flash mixer* for its rapid dispersion in raw water, and the water is then transferred to a *flocculation tank* provided with a *slow mixer*.

Mixing therefore involves high degree of turbulence and power dissipation.

A typical **mixing basin provided with a flash mixer** is shown in Fig. 9.24. It consists of a rectangular tank which is provided with an impeller fixed to an impeller shaft. The impeller is driven by an electric motor, and it revolves at a high speed inside the tank.

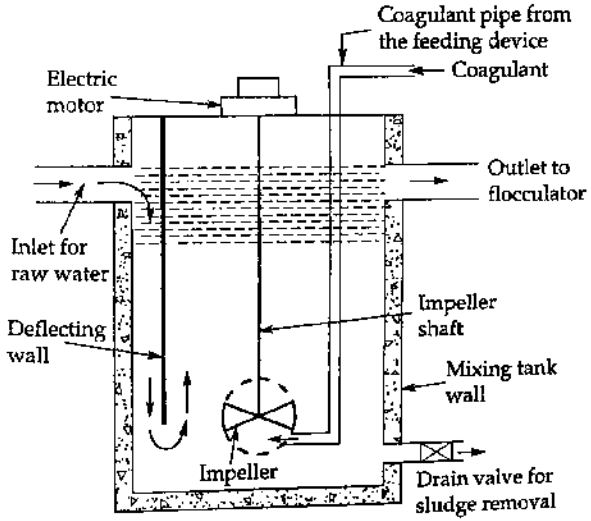


Fig. 9.24. Mixing basin with a flash mixer.

The coagulant is brought by the coagulant pipe and is discharged just under the rotating fan. The raw water is separately brought from the inlet end, and is deflected towards the moving impeller by a *deflecting wall*. The thoroughly mixed water is taken out from the outlet end. A drain valve is also provided to remove the sludge from the bottom of the flash mixer.

The impeller's speed is generally kept between 100 to 120 r.p.m. (revolutions per minute), and the usual values of detention period may vary between $\frac{1}{2}$ to 2 minutes.

Power required in flash mixing may vary from 2 to 5 kW per m^3 per minute. Power input in mixing and flocculation is frequently expressed in terms of *temporal mean* **velocity gradient, G',** expressed by the equation :

$$G' = \left[\frac{P}{\mu . V}\right]^{1/2} \qquad ...(9.25)$$

where G' = Velocity gradient (per second)

P = Power dissipated in watts *i.e.* N.m/s

μ = Absolute or dynamic viscosity of raw water in Ns/m^2

V = Volume of raw water to which P is applied, in m^3.

The units of G' will evidently be

$$\sqrt{\frac{N.m}{s}\frac{m^2}{N.s}.\frac{1}{m^3}} = \frac{1}{s} \text{ ; } i.e. \text{ per second.}$$

For rapid mixing, the product $G'T$ (unitless) should be 30,000 to 60,000, where T is the detention period or mixing period, usually 60 to 120 seconds.

9.9.3. Flocculation Tank or a Flocculator. As was pointed out earlier, the best floc will form when the mixture of water and coagulant are violently agitated followed by a relatively slow and gentle stirring to permit build up and agglomeration of the floc particles. From the mixing basin, the water is, therefore, taken to a ***flocculation tank*** called a ***flocculator***, where it is given a slow stirring motion. Rectangular tanks fitted with paddles operated by electric motors can best serve this purpose, although even plain flocculation chambers with controlled flow velocities are also possible. Various patented flocculators are now-a-days available in the market. A typical flocculator fitted with slowly moving paddles is shown in Fig. 9.25 (*a*) and (*b*). The water coming out from the flocculator is taken to the sedimentation tank.

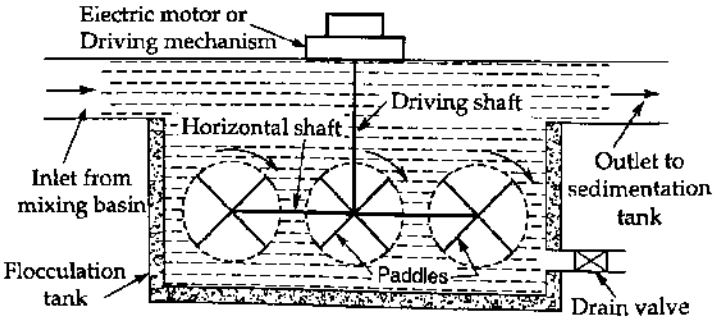


Fig. 9.25. (*a*) A typical Flocculator fitted with paddles.

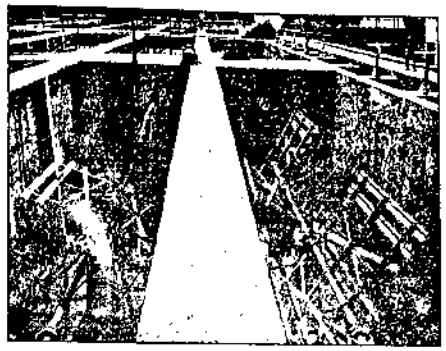


Fig. 9.25. (*b*) Photographic view of a Rotary type paddle flocculator.

The paddles usually rotate at a speed of about 2 to 3 rpm. The usual values of detention period for this tank ranges between 20 to 60 minutes (30 minute as the normal value) and value of velocity gradient (G') ranges between 20 to 80 s^{-1}. The clear distance between the paddles and the wall or the floor of the tank is about 15 to 30 cm. The velocity of flow through such a flocculator is unimportant, because the paddles provide a rolling motion which prevents the floc from settling.

9.9.4. Sedimentation Tank. The function, design and other details of this tank are the same as those discussed under "Plain Sedimentation". This tank is designed on the same assumptions as a plain sedimentation tank, except that, a lower value of detention period (say about 2 to 4 hours)* is generally sufficient here. Also higher value of surface loading (or the overflow rate) varying between 24-30 $m^3/d/m^2$ of plan area, *i.e.* 1000—1250 litres/hr/m^2 of plan area** is generally permitted.

9.10. Combined Coagulation-cum-Sedimentation Tanks

It has been possible to combine the flocculation tank along with the sedimentation tank, as shown in Fig. 9.26. Such a tank is known as a *coagulation*

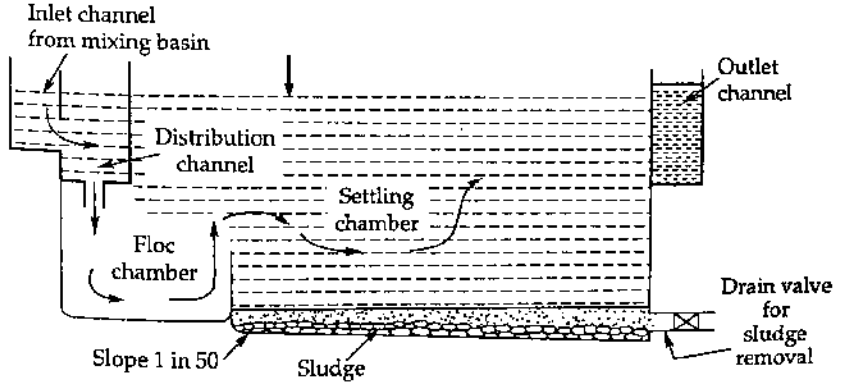


Fig. 9.26. Coagulation-Sedimentation tank.

*as against a detention period of 4 to 8 hours for plain sedimentation tanks

**as against 500-750 l/hr/m^2 for plain sedimentation tanks.

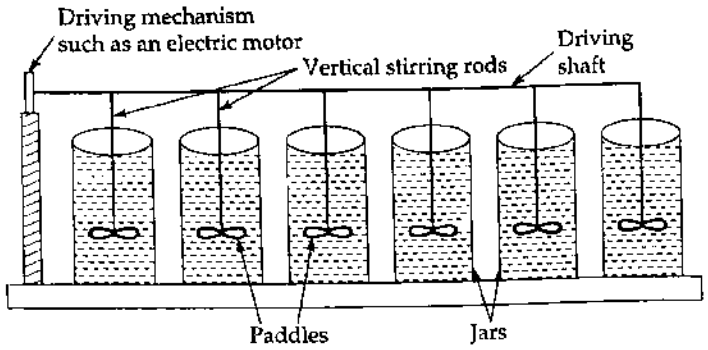


Fig. 9.28. Apparatus for jar test for determining approximate optimum coagulant dosage.

are thus made to rotate. The formation of the floc in each jar is noted. The amount of coagulant in the jar which produces a *good floc* with the least amount of coagulant, indicates the optimum dosage. The speed of paddles and the time of mixing may also be varied for different tests during determining this least optimum dosage.

9.12. Production of Sludge in the Coagulation-Sedimentation Process

The suspended solids of raw water along with the precipitates of metal coagulants settle down in a coagulation-sedimentation plant. This settled mass, called sludge, is to be periodically removed and safely disposed of.

The quantity of this sludge has also to be estimated for deciding the depth of the sludge zone, and periodicity of its removal. The sludges withdrawn from the coagulation plants are largely composed of water. As much as 98% of the sludge mass may be water. In other words, 2 gm of solids will constitute 100 gm of wet sludge, by combining with 98 gm of water.

By knowing the suspended solids concentrations of the raw and the sedimented waters, along with estimating the sludge contributed by the chemical coagulants added to the water in the form of metallic precipitates, we can estimate the volume of the sludge produced by the plant, if other factors like flow discharge, % age of water contained in the sludge, Sp. gr. of the sludge or of solids, etc. are predetermined, as computed in solved example 9.18.

Alum, which is chiefly used as a coagulant has been found to be generating sludge equal to 0.24* times of its own mass, as shown in Example 9.15. The addition of other miscellaneous chemicals like clay, carbon, polymers, etc.

*Each mole of alum (mol. mass = 666) is supposed to produce 2 mol of aluminium hydroxide $[Al(OH)_3]$ precipitate, (mol. mass = 78). In other words, 666 kg of alum produces 2×78 kg of sludge precipitate. 1 kg alum thus produces = $\frac{2 \times 78}{666}$ = 0.24 kg of sludge, as considered in Example 9.15. Mackenzie L Davis and David A. Cornwell in their book "Introduction to Environmental Engineering, however opine that the alum produces a complex Aluminium hydroxide, having a molecular formula of $Al(H_2O)_3(OH)_3$, with molecular mass = $27 + 3 \times 16 + 3 \times 17 = 132$. Accordingly, *1 kg of alum will produce* $\frac{2 \times 132}{666}$ = *0.44 kg of sludge.*

during the coagulation process may further produce sludge equal to the amounts of these substances. The suspended solids removed from the water will also produce equal amount of sludge. These three factors, contributing to the sludge, can be written down in the form of a mathematical equation, as :

Dry sludge produced in mg/day

$$= Q \text{ in L/day} \left[\begin{array}{l} \text{Suspended solids removed in mg/L} + 0.24 \\ \times \text{ Alum dose in mg/L} + \text{Misc. chemicals added} \\ \text{to raw water like carbon, clay, polymers, etc.} \\ \text{in mg/L} \end{array} \right] \quad ...(9.26)$$

Example 9.18. *A rectangular sedimentation tank following coagulation-flocculation is to treat a flow of 3000 m^3/day with a detention time of 6 hours. It is to be hand cleaned of sludge at 6 week intervals. The suspended solids concentration of the water is reduced from 250 mg/L to 5 mg/L by coagulation-flocculation. The settled sludge includes 40 mg/L (based on water flow) of metallic precipitate and has a moisture content of 85% and specific gravity of 1.24. Determine the volume of sludge produced between cleanings and the basic dimensions of the tank if the water depth just before cleaning is 3 m and its length is twice its width.* (*GATE 1995*)

Solution.

Initial suspended solids concentration of water = 250 mg/L

Final suspended solids concentration of water = 5 mg/L

Suspended solids removed in the tank

= 250 mg/L – 5 mg/L = 245 mg/L

These removed suspended solids will settle down as sludge along with the metallic precipitates formed during coagulation. The concentration of such metallic precipitates in sludge is given as 40 mg/L.

Hence, the total solids concentration in sludge

= 245 mg/L + 40 mg/L = 285 mg/L.

Discharge or water flow in the tank = 3000 m^3/day.

∴ Total solids removed as sludge per day

$$= 285 \text{ mg/l} \times 3000 \text{ m}^3\text{/day}$$

$$= 285 \text{ mg/L} \times \frac{(3000 \times 1000) \text{ L}}{\text{day}}$$

$$= \frac{285}{10^6} \times (3000 \times 1000) \text{ kg/day} = \mathbf{855 \text{ kg/day}}.$$

Total solids removed in 6 weeks (*i.e.* 6 × 7 days)

= 855 × (6 × 7) kg = **35,910 kg.**

Since the produced sludge has a m.c. of 85%, we have :

15 kg of solids (dry sludge) will make

= 100 kg of wet sludge

∴ 35910 kg of solids (dry sludge) will make

$$= \frac{100}{15} \times 35{,}910 \text{ kg of wet sludge}$$

= 2,39,400 kg of wet sludge

= **239.4** tonne of wet sludge.

Volume of wet sludge *produced per 6 week interval*

$$= \frac{\text{Mass of wet sludge}}{\text{Density of wet sludge}}$$

where density of wet sludge

= sp. gr. of wet sludge × density of water

= 1.24 × 1 t/m^3 = 1.24 t/m^3

$$= \frac{239.4 \text{ t}}{1.24 \text{ t/m}^3} = \mathbf{193.06\ m^3}. \quad \textbf{Ans.}$$

Dimensions of the Tank

L = Length of the tank = 2 B, where B is the width of the tank (given)

D_w = Water depth = 3 m (since the given "water depth just before cleaning" is only the water depth, and does not include the sludge depth)

Q = Flow in the tank = 3000 m^3/day

t = Detention time = 6 hrs.

Capacity of the tank reqd. *i.e.* Qty. of water to be treated per 6 hr

$$= 3000 \times \frac{6}{24} \text{ m}^3 = 750 \text{ m}^3$$

But capacity $= L \times B \times D_w = (2B)\,(B)\,(3) = 750$

$$\therefore \quad B = \sqrt{\frac{750}{6}} = \mathbf{11.2\ m} \quad \text{(say)}$$

L = 22.4 m, B = 11.2 m

$D = D_w$ + Sludge depth (av.) + Free board ; where

$$\text{Sludge depth (av.)} = \frac{\text{Sludge volume}}{\text{Plan area of the tank}} = \frac{193.06 \text{ m}^3}{22.4 \text{ m} \times 11.2 \text{ m}}$$

= **0.77 m** ; say 0.8 m.

Assuming that a free-board of 0.5 m is provided, we have the total depth of the tank = D = 3 + 0.8 + 0.5 = **4.3 m.**

Hence, the *tank size to be used* shall be

22.4 m × 11.2 m × 4.3 m. Ans.

9.12.1. Dewatering and Disposal of Sludge from Water Treatment Plants. Wet or watery sludge is produced in a water treatment plant in its following units :

(*i*) *Sedimentation-flocculation tank* or *Clariflocculator*

(*ii*) *Filters.*

The sludge obtained from the above units have often been discharged into rivers in the past, but with the alarming increase in the pollution of river waters, advanced methods for disposing of these wastes have to be adopted.

These wastes are sometimes disposed of by ponding in artificial or natural **lagoons**, which are constructed by excavation or enclosing an area by dikes. Lagoons do provide a popular, acceptable and economical method for dewatering, thickening, and temporary storage of waste sludge, provided sufficient open land is available in the vicinity of the plant. Detailed description of lagoons is given in article 9.26 of Vol. II.

Sand drying beds are sometimes constructed for temporary storage of wet sludge, to allow its filtration through the beds of sand and gravel, to effect dewatering by gravity drainage. Such beds are, however, possible only for small plants and where sufficient land area is available. Detailed description of such contact beds is given in article 9.25.1 in Vol. II.

Gravity thickeners may sometimes be used to reduce the volume of sludge for its easy handling and disposal. Such a thickener unit consists of a deeper settling tank (to accommodate greater volume of sludge), containing a heavier raking mechanism than that provided for a conventional settling tank. Polymers or other coagulant aids may be applied to improve the settleability of the sludge solids. The thickened sludge, having reduced moisture content, is finally sent to sludge drying and disposal. Detailed description of a gravity thickener is given in article 9.35.4 of vol. II.

High speed centrifuges, are often used in advanced countries for dewatering and drying of sludge. **Pressure filtration** is another method, which is adopted for dewatering sludges of water treatment plants on a large scale in advanced countries. *Vacuum filtration* is also sometimes adopted for dewatering sludges of water treatment plants, although of course, vacuum filters are usually preferred for processing the sludge wastes of Sewage treatment plants. Such mechanical methods of dewatering sludge are described in article 9.25.2 of Vol. II.

The dewatered sludge is finally disposed of either by *incineration* or by *burial in trenches*, as discussed in article 9.25.4 of Vol. II.

FILTRATION

Screening and sedimentation removes a large percentage of the suspended solids and organic matter present in raw supplies. The percentage of removal of the fine colloidal matter increases when coagulants are also used before sedimentation. But however, the resultant water will not be free of impurities, and may contain some very fine suspended particles (discrete, or flocculated when coagulation is used) and bacteria present in it. To remove or to reduce the remaining impurities still further, and to produce potable and palatable water, the water is *filtered* through the beds of fine granular material, such as sands, etc. *The process of passing the water through the beds of such granular materials (called filters) is known as filtration.* Filtration may help in removing colour, odour, turbidity, and some pathogenic bacteria from the water.

Two types of filters are commonly used for treating municipal water supplies. They are

(*i*) *The slow sand gravity filters* ; and

(*ii*) *The rapid sand gravity filters.*

A third type of a rapid sand filter works under pressure and is known as a *pressure filter*. This type of filters are generally used for small plants, such as for individual industrial supplies, or for swimming pools ; and are generally not adopted for treating large scale municipal supplies.

The *slow sand gravity* filters* often called *slow sand filters* are useful in the sense that they can remove much larger percentage of impurities and bacteria from the water, as compared to what can be removed by *rapid sand gravity filters* (often called *rapid gravity filters*). However, slow sand filters yield a very slow rate of filtration (about $\frac{1}{30}$ th of that given by *rapid gravity filters*) and require large areas, and are costly.

With the advancement of disinfection techniques, the necessity of too much purification and that of the maximum removal of bacteria (as is achieved by the slow sand filters) has decreased, and therefore, the slow sand filters are becoming obsolete these days. In the modern treatment plants, rapid gravity filters are now-a-days almost universally adopted. *The water from the coagulation-sedimentation plant is directly fed into the rapid gravity filters, and the resultant supplies are disinfected for complete killing of germs and colour removal.*

Before we discuss the theory and operation of the various types of filters in details, it shall be worthwhile to summarise, as to how these so-called filters purify the water.

9.13. Theory of Filtration

The filters, in fact, purify the water under four different processes. These processes or actions are summarised below :

(*i*) **Mechanical straining.** The suspended particles present in water, and which are of bigger size than the size of the voids in the sand layers of the filter, cannot pass through these voids and get arrested in them. The resultant water will, therefore, be free from them.

Most of the particles are removed in the upper sand layers. The arrested particles including the coagulated flocs forms a mat on the top of the bed, which further helps in straining out the impurities.

(*ii*) **Flocculation and sedimentation.** It has been found that the filters are able to remove even particles of size smaller than the size of the voids present in the filter. This fact may be explained by assuming that the void spaces act like tiny coagulation-sedimentation tanks. The colloidal matter arrested in these voids is a gelatinous mass and, therefore, attract other finer particles. These finer particles thus settle down in the voids and get removed.

(*iii*) **Biological metabolism.** Certain micro-organisms and bacteria are generally present in the voids of the filters. They may either reside initially as coatings over sand grains, or they may be caught during the initial process of filtration. Nevertheless, these organisms require organic impurities (such as algae, plankton, etc.) as their food for their survival. These organisms, therefore, utilise such organic impurities and convert them into harmless compounds by the process of biological metabolism. The harmless compounds so formed, generally form a layer on the top, which is called **schmutzdecke** or ***dirty skin***. This layer further helps in absorbing and straining out the impurities.

*The term "gravity" indicates that the water passes through the filter under the mere force of gravity, *i.e.* the water standing over the filter bed is exposed to the atmospheric pressure.

(*iv*) **Electrolytic changes.** The purifying action of filter can also be explained by the theory of ionisation. According to this theory, a filter helps in purifying the water by changing the chemical characteristics of water. This may be explained by the fact that the sand grains of the filter media and the impurities in water, carry electrical charges of opposite nature. When these oppositely charged particles and the impurities come in contact with each other, they neutralise each other, thereby changing the character of the water and making it purer. After a certain interval, the electrical charges of sand grains get exhausted and have to be restored by cleaning the filter.

9.14. Filter Materials

Sand, either fine or coarse, is generally used as filter media. The layers of sand may be supported on *gravel*, which permits the filtered water to move freely to the under-drains, and allows the wash water to move uniformly upward.

Sand. The filter sand should generally be obtained from rocks like quartzite, and should contain the following properties :

(*i*) It should be free from dirt and other impurities.

(*ii*) *It should be uniform in nature and size.*

(*iii*) It should be hard and resistant.

(*iv*) It should be such as not to loose more than 5% of its weight after being placed in hydrochloric acid for 24 hours.

The *size of the sand* is measured and expressed by the term called *effective size*. The ***effective size, i.e.*** D_{10} may be defined as the size of the sieve in mm through which ten per cent of the sample of sand (by mass) will pass. The selection of the correct effective size is very important, because too smaller size will lead to very frequent clogging of filters, and will give very low filtration rates. Similarly, too large size will permit the suspended particles and bacteria to pass through it, without being removed. The value of D_{10} for sand to be used in *slow sand filters* varies from 0.2 to 0.4 mm, while for *rapid gravity filters*, this value varies from 0.35 to 0.55 mm.

The *uniformity in size* or degree of variations in sizes of particles is measured and expressed by the term called *uniformity coefficient. The* ***uniformity coefficient,*** *i.e.* $\left(\frac{D_{60}}{D_{10}}\right)$ may be defined as the ratio of the sieve size in mm through which 60 per cent of the sample of sand will pass, to the effective size of the sand. The value of $\frac{D_{60}}{D_{10}}$ for sand to be used in *slow sand filters* varies from 1.8 to 2.5; while for *rapid gravity filters*, this value varies from 1.3 to 1.7.

Gravel. The gravel which may be used below the sand should be hard, durable, free from impurities, properly rounded, and should have a density of about 1600 kg/m^3. Gravel of different sizes are usually placed in 3-4 layers, each of 15-20 cm Depth, with the coarsest size (20 to 60 mm) placed in the bottom-most layer, and the finest size (3 to 6 mm) in the top most layer.

Other materials. Instead of using sand, sometimes, *anthrafilt* is used as filter media. Anthrafilt is made from anthracite, which is a type of coal-stone that burns without smoke or flames. The use of anthrafilt as a filter media in two or multi layers have proved very successful in foreign countries. It is cheaper and has been able to give a high rate of filtration. However, this type of coal is not available in India. Research is, therefore, being conducted at Central Public Health Research Institute, Nagpur, to find out the suitability and usefulness of the bituminous coal* as a substitute to anthracite. The results obtained so far have been very promising. But so long as this substitute filter medium comes out of the laboratories, sand continues to form the principle filter medium.

9.15. Types of Filters and Their Classifications

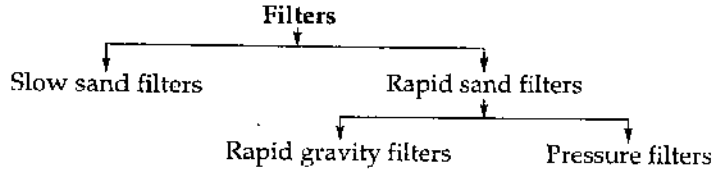
As pointed out earlier, there are mainly three types of filters, *viz.*

(*i*) *Slow sand filters* :

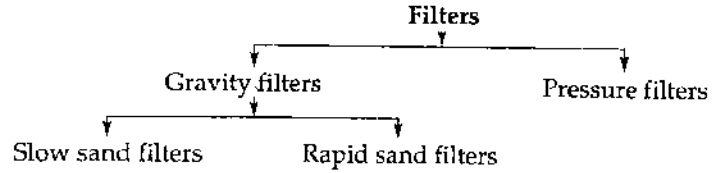
(*ii*) *Rapid gravity filters ; and*

(*iii*) *Pressure filters.*

However, when classified on the basis of the rate of their filtration, the filters can be divided as follows :



Similarly, their classification, when based upon the considerations of gravity and pressure, is indicated below.



9.16. Slow Sand Filters

Slow sand filters were first devised when the process of filtration was invented in England by James Simpson (who was working for Chelsa Water Co.) in 1829. They were widely used since then, till the last decade of the 19th century, when the rapid gravity filters were invented. *Their use has since decreased and they are becoming obsolete these days. However, they may still be preferred on smaller plants at warm places, where covers on filters are not required to protect the filters from freezing.* Slow sand filters normally utilise effluents from the plain sedimentation tanks, and are used for relatively clearer waters.

*Which is easily and cheaply available in India.

9.16.1. Construction of Slow Sand Filters. A typical section of a slow sand filter is shown in Fig. 9.27. The various parts of this filter are discussed below.

(*i*) **Enclosure tank.** It consists of an open water-tight rectangular tank, made of masonry or concrete. The bed slope is kept at about 1 in 100 towards the central drain. The depth of the tank may vary from 2.5 to 3.5 m. The plan area of the tank may vary from 100 to 2000 sq. m or more, depending upon the quantity of water to be treated.

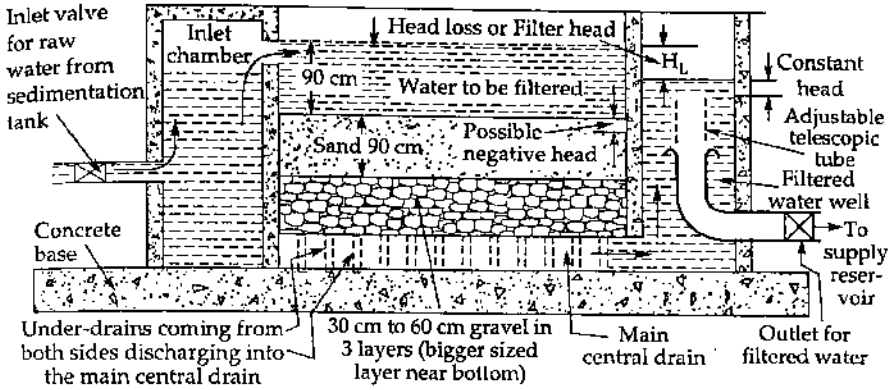


Fig. 9.29. Typical section of a slow sand filter.

(*ii*) **Filter media.** The filtering media consists of sand layers about 90 to 110 cm in depth, and placed over a gravel support. The effective size (D_{10}) of the sand varies from 0.2 to 0.4 mm and the uniformity coefficient $\left(\frac{D_{60}}{D_{10}}\right)$ varies from 1.8 to 2.5 or 3.0. The top 15 cm layer of this sand is generally kept of finer variety than that of the rest, which is generally kept uniform in grain size. However, if different gradations of sand are used (though not necessary), then the coarsest layer should be placed near the bottom, and the finest towards the top. The finer the sand used, the purer will be the obtained water, as more impurities and bacteria will be removed.

(*iii*) **Base material.** The base material is gravel, and it supports the sand. It consists of 30 to 75 cm thick gravels of different sizes, placed in layers. Generally, three to four layers each of 15-20 cm depth are used. The coarsest gravel is used in the bottom most layer, and the finest gravel is used in the topmost layer. The size of gravel in the bottom-most layer is generally kept 40 to 65 mm ; in the intermediate layers, as varying between 20 to 40 mm, and 6 to 20 mm (when two intermediate layers are used) ; and in the top-most layer as, 3 to 6 mm.

(*iv*) **Under-drainage system.** The gravel support is laid on the top of an under-drainage system. The under-drainage system consists of a central drain and lateral drains, as shown in Fig. 9.30. The laterals are open jointed pipe drains or some other kind of porous drains placed 3 to 5 m apart on the bottom floor and sloping towards a main covered central drain. The laterals collect the

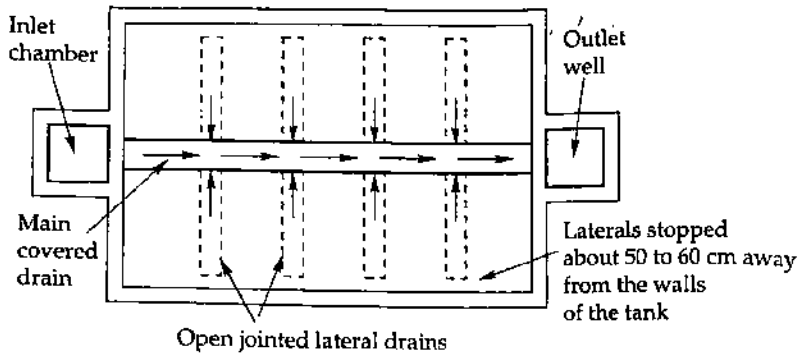


Fig. 9.30. Plan of under drainage system for slow sand filter.

filtered water and discharge it into the main drain, which leads the water to the filtered water well. Sometimes, instead of placing it in the centre, the main drain is placed along one side of the tank, and the laterals slope towards it.

(*v*) **Inlet and Outlet arrangements.** An inlet chamber, is constructed for admitting the effluent from the plain sedimentation tank without disturbing the sand layers of the filter and to distribute it uniformly over the filter bed.

A 'filtered water well' is also constructed on the outlet side in order to collect the filtered water coming out from the main under-drain. *In order to maintain a constant discharge through the filter, an* ***adjustable telescopic tube*** *(as shown in Fig. 9.29) is generally used.*

Inlets and outlets are generally governed by automatic valves.

(*vi*) **Other appurtenances.** Besides these arrangements, certain other appurtenances are provided for the efficient functioning of these filters. For example, *vertical air pipe* passing through the layers of sand may be provided, and may help in proper functioning of the filtering layers. Similarly, arrangements are made in order to control the depth of water above the sand layer (1 to 1.5 m). This depth is not allowed to undergo large variations. In addition, a *meter* to measure the flow, and a *gauge* to measure the loss of head are usually installed. The loss of head caused by the resistance offered by the sand grains to the flow of water through it, is usually called ***filter head*** or ***filtering head***. It is the difference of water levels between the filter tank and the filtered water well. For a freshly cleaned filter unit, the resistance offered is less, and, therefore, the filter head is usually small, say 10 to 15 cm, but goes on increasing as the filter layers get clogged, and telescopic tube is adjusted by manual labour, so as to maintain a uniform discharge. But when this loss of head becomes high (0.7 to 1.2 m or so) the filter unit must be put out of *service* and be cleaned.

9.16.2. Operation and Cleaning of Slow Sand Filters. The treated water from the sedimentation tank is allowed to enter the inlet chamber of the filter unit and get distributed uniformly over the filter bed. The water percolates through the filter media and gets purified during the process of filtration. The water then enters the gravel layers and comes out as the filtered water. It gets collected in the laterals through the open joints, which discharge into the main drain. The main drain finally discharges into the 'filtered water well', from

where the filtered water is taken to the storage tanks for supplies. The rate of discharge or the rate of filtration is kept constant by arrangements like that of a telescopic tube, described earlier.

It may also be noted that the water entering the slow sand filter should not be treated by coagulants. This is due to the fact that the dirty skin formed by the floc and carried to the filter considerably affects the economical working of the filter.

The depth of water on the filter should also be decided carefully and should not be allowed to undergo large variations. This depth should neither be tool large nor it should be too small. It is generally kept equal to the depth of the filter sand.

The loss of head called ***filter head*** or *filtering head* or *filtration head* (defined earlier) is generally limited to a maximum value of about 0.7 to 1.2 m. When this limiting value, which is roughly kept as 0.7 to 0.8 times the depth of the filter sand, is reached, the filter unit must be put out of service and the filter be cleaned.

The ***cleaning of slow sand filters*** is not done by back washing as is done for rapid gravity filters, but is done by scrapping and removing the 1.5 to 3 cm of top sand layer. The top surface is finally raked, roughened, cleaned and washed with good water. The amount of wash water required is generally small, say of the order of 0.2 to 0.6 per cent of the total water filtered. Cleaning is repeated as often as necessary until the sand depth is reduced to about 40 cm or so. Then more sand is added. *A lot of manual labour is required in cleaning such filters, although very small quantities of wash waters are needed.*

After each cleaning, the filter is again used and raw water is admitted into it. But the effluents that will be obtained in the beginning will not be pure and are not used for about 24 to 36 hours until formation of a film of arrested impurities around the sand grains (*i.e.* the formation of **schmutzdecke**) has taken place. Since the filtering action of slow sand filters depends largely upon the formation of this film, the effluents obtained in the beginning, when such a film is absent, shall not be pure.

The interval between the two successive cleanings depends mainly on the nature of the impurities present in water and also on the size of the filtering sand used in the filter. This interval however, normally ranges between one to three months.

9.16.3. Rate of Filtration or Rate of Loading for Slow Sand Filters. The rate of filtration that can be obtained from the normal slow sand filters is small and usually ranges between 100 to 200 litres per hour per m^2 of filter area.

9.16.4. Efficiency and Performance of Slow Sand Filters. Slow and filters are highly efficient in removing bacteria and other suspended solids from raw waters. *The extent of bacteria removal is up to 98 to 99 per cent or more.* These filters will also remove the odours and tastes, particularly those caused by the presence of organic impurities, such as algae and plankton. However, these filters are less efficient in removing colours of raw waters. Moreover, they can remove turbidities only upto about 50 mg/l or so, and are,

therefore, not suitable for sedimented waters having turbidities greater than 50 to 60 mg/l or so.

9.16.5. Uses of Slow Sand Filters. Slow sand filters are best suited for smaller plants and for purifying waters with low colours, low turbidities and low bacterial contents. However, because of their smaller rate of filtration, they require huge surface areas and large volumes of filtering materials. This makes them costly and uneconomical, especially for treating large scale supplies. They are, therefore, being replaced by rapid gravity filters at all the major cities and towns. They may, however, be preferred for smaller plants for village supplies or for individual industrial supplies, especially in hotter and under-developed countries like India.

Example 9.19. *Design six slow sand filter beds from the following data :*

Population to be served = *50,000 persons.*
Per capita demand = *150 liters/head/day.*
Rate of filtration = *180 litres/hr./sq. m*
Length of each bed = *Twice the breadth.*
Assume max. demand as 1.8 times the average daily demand.
Also assume that one unit, out of six, will be kept as stand-by.

Solution.

Average daily demand

= Population × Per capita demand
= 50,000 × 150 litres/day
= 7.5×10^6 litres/day

Max. daily demand = $1.8 \times 7.5 \times 10^6 = 13.5 \times 10^6$ litres/day

Rate of filtration per day = (180 × 24) litres/sq m/day

Total surface area of filters reqd.

$$= \frac{\text{Max. daily demand}}{\text{Rate of filtration per day}}$$

$$= \frac{13.5 \times 10^6}{180 \times 24} \text{ sq. m} = 3.125 \times 10^3 \text{ sq. m.} = 3125 \text{ sq. m.}$$

Now, six units are to be used ; out of them, one is to be kept as stand-by, and hence only 5 units should provide the necessary area of filter required.

∴ The area of each filter unit

$$= \frac{1}{5} \times \text{Total area reqd.} = \frac{1}{5} \times 3125 = 625 \text{ sq. m.}$$

Now, if L is the length and B is the breadth of each unit, then $L = 2.B$ (given)

$$\therefore \quad 2B\,B = 625 \text{ sq. m.}$$

or $$B^2 = 312.5 \text{ sq. m}$$

or $$B = 17.7 \text{ m ; say } \mathbf{18\ m}$$

$$\therefore \quad L = 2\,(17.75) = \mathbf{36\ m.}$$

Hence, use 6 filter units with one unit as stand-by, each unit of size 36 m × 18 m, arranged in series with 3 units on either side. **Ans.**

RAPID SAND FILTERS OR MECHANICAL SAND FILTERS

It was pointed out earlier that the slow sand filters can filter water at a very slow rate, and thus require huge areas for their installation. In addition to requiring huge quantities of filtering materials, such filters will, therefore, require huge space which may prove to be very costly, particularly in congested cities and industrial towns. In order to reduce the requirement of space and to increase the rate of filtration, a lot of research was conducted, which finally led to be development of rapid sand filters, or sometimes called *mechanical sand filters.*

The rapid sand filters are of **two types** : *viz.*

(*i*) One which utilise comparatively larger sized sand particles, which allow greater rate of filtration as compared to that of slow sand filters. They are called **rapid gravity filters.**

(*ii*) One which utilise the development of pressure over the filtering water and thereby increasing the rate of filtration. They are called **pressure filters.**

Rapid gravity filters are used for large municipal supplies, whereas pressure filters, being more handy, may be used for small installations, such as for industrial plants and swimming pools.

These two types of filters will be discussed in the subsequent pages.

9.17. Rapid Gravity Filters

Rapid gravity filters were developed in the last decade of the nineteenth century through the efforts of G.W. Fuller at Louisville U.S.A. These filters employ coarser sand, with effective size as 0.5 mm or so. On an average, these filters may yield as high as 30 times the yield given by the slow sand filters. Waters from the coagulation-sedimentation tanks are used in these filters, and filtered water is treated with *disinfectants,* so as to obtain potable supplies.

9.17.1. Construction of Rapid Gravity Filters. A typical section of a rapid gravity filter is shown in Fig. 9.31 (*a*) and (*b*). An isometric view of such a filter is also shown in Fig. 9.32. The **various parts** of this filter are described below :

(1) **Enclosure tank.** It consists of an open water-tight rectangular tank, made of masonry or concrete. The depth of the tank may vary from 2.5 to 3.5 m. In order to achieve uniform distribution of water, the area of the filter units should not be kept larger, and is generally limited to about 10 to 80 m^2 for each unit.

The number of units at a filter plant may be roughly estimated by the equation developed by Morrell and Wallace, which states that

$$N = 1.22\sqrt{Q} \qquad \ldots(9.27)$$

where N = no. of filter units
Q = plant capacity in million litres per day.

There should be at least two filter units in any plant. And for a plant of more than 9 million litres per day capacity, no single unit should have a capacity greater than one-fourth of the capacity of that plant.

(2) **Filter media.** The filtering media consists of sand layers, about 60 to 90 cm in depth, and placed over a gravel support. The effective size (D_{10}) of the sand varies from 0.35 to 0.55 mm and the uniformity coefficient $\left(\frac{D_{60}}{D_{10}}\right)$

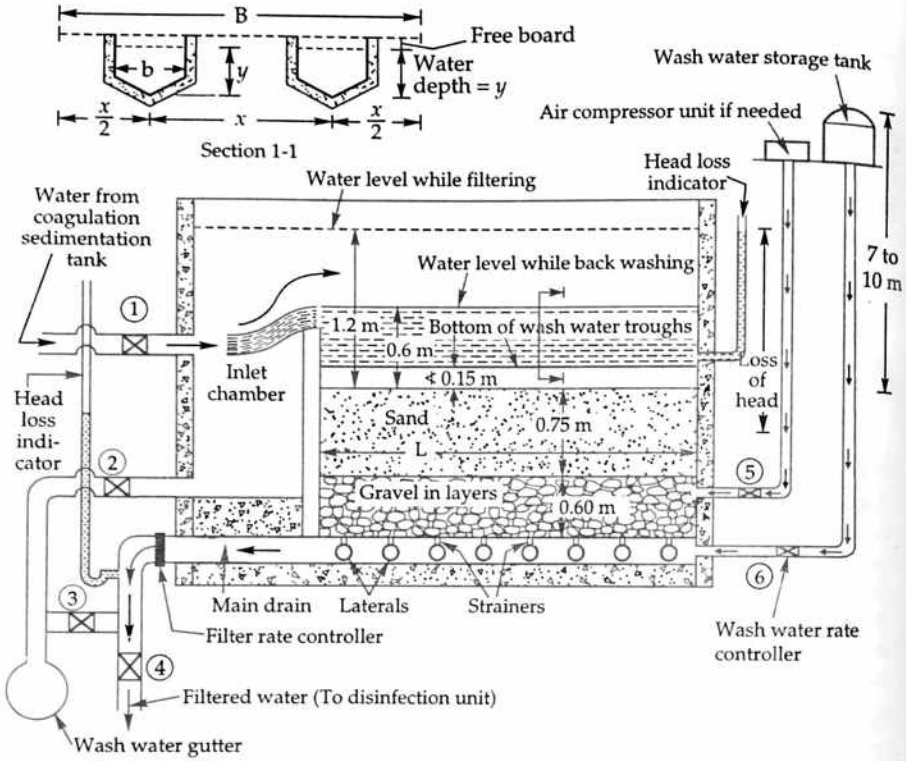


Fig. 9.31(a). Typical section of a Rapid Gravity filter.

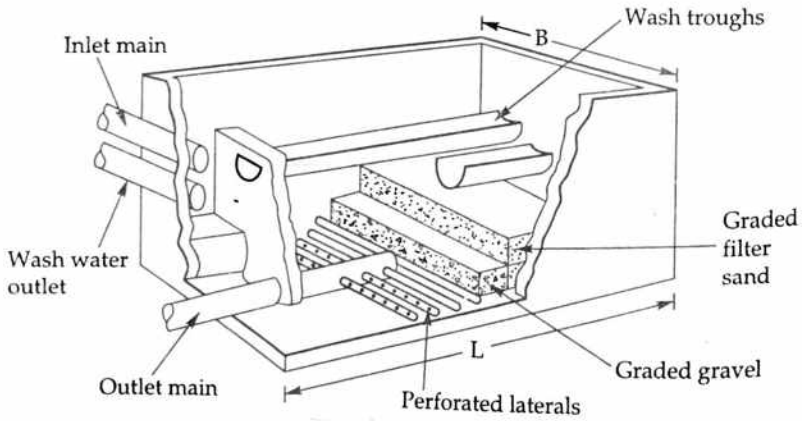


Fig. 9.31 (b). Cut away section of a Rapid Gravity Filter

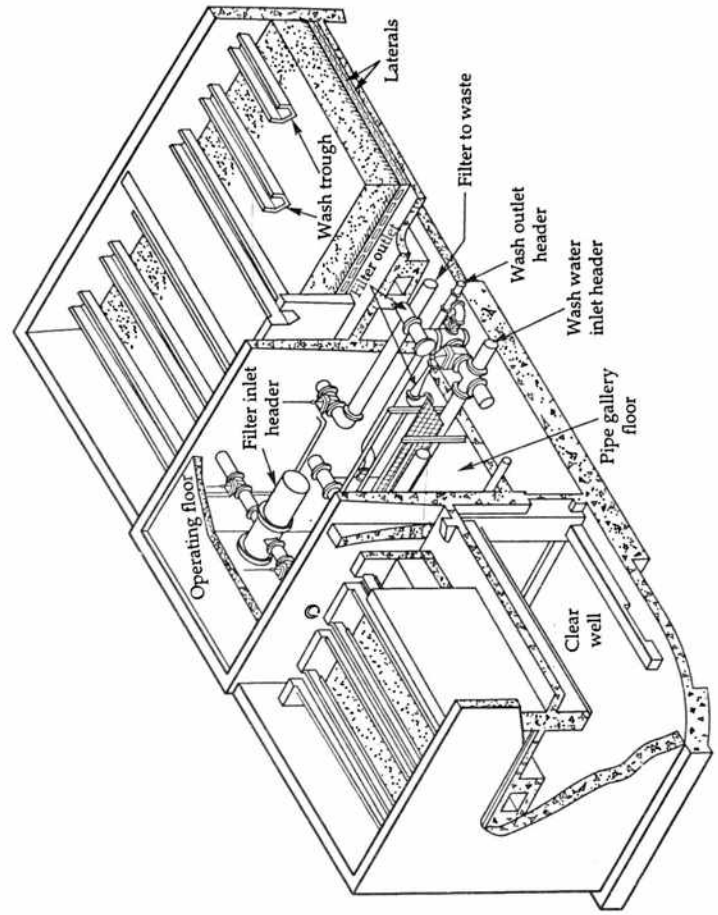


Fig. 9.32. Isometric view of a Rapid Gravity filter.

generally ranges between 1.3 to 1.7. The sand must be laid in layers. The finer variety should be used towards the top and coarser variety towards the bottom.

(3) **Base material.** In slow as well as rapid gravity filters, the base material is gravel, and it supports the sand. But in a rapid gravity filter, in addition to supporting the sand, it distributes the wash water. It consists of 60 to 90 cm thick gravels of different sizes, placed in layers. Generally, five to six layers, each of 10 to 15 cm in depth are used. The coarsest gravel (about 40 mm in size) is used in the bottom-most layer, and the finest gravel (about 3 mm in size) is used in the top-most layer. The size of the gravel in the bottom-most layer is thus generally kept between 20 to 40 mm ; in the intermediate layers, between 12 to 20 mm, and 6 to 12 mm (when two intermediate layers are used) ; and in the top most layer, between 3 to 6 mm. *In a rapid gravity filter, the distribution of the wash water is the critical function of the gravel layer and hence careful grading and equally careful placing of the materials is important.*

(4) **Under-drainage system.** In slow sand-filters, the under-drainage system was provided only to receive and deliver the filtered water. Whereas, in rapid gravity filters, the under drainage system serves two purposes ; *viz.* (*i*) to receive and collect the filtered water, and (*ii*) to allow the back washing for cleaning of filter.

The under drainage system should, therefore, be designed in such a way that in addition to collecting the filtered water during its downward journey, it should be capable of passing the wash water upward at sufficiently high velocity. Back washing in fact consists of passing filtered water upward through the bed at such a velocity that it causes the sand bed to expand until its thickness is 25 to 40% greater than during filtering, depending upon the media. The grains move through the rising water, rub against each other and are cleaned of deposits. A bed is usually washed when head loss through the filter reaches 2 to 3 m. The bank wash water should usually move up at a high rate of about 300—900* litres/minute/m^2 of filter area, or more, depending on the design, while applying it evenly and uniformly over the under portion of the gravel or sand bed. Since the rate of application of wash water is so much higher than the rate of filtration (*i.e.* about 6 to 18 times the rate of filtration which is of the order of 50 to 100 litres/minute/sq. m), the design of the under-drainage system is primarily governed by the considerations of even and uniform distribution of wash water.

Various forms of under-drainage systems, such as (*i*) Manifold and lateral system (*ii*) The wheeler bottom (*iii*) the porous plate bottom, etc. have been designed for rapid gravity filters, and are patented by their manufacturers. However, *the two forms of 'manifold and lateral system'* are most widely used in India, and are described below.

A **manifold and lateral system** type of installation consists of about 40 cm diameter manifold pipe running lengthwise along the centre of the filter bottom. Taking off from the Manifold in both directions at right angles to it would be 10 cm dia laterals. The laterals are placed at about 15 to 30 cm apart, as shown in Fig. 9.33.

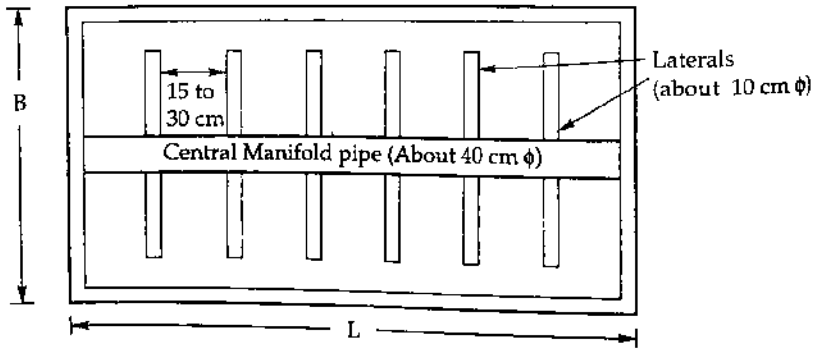


Fig. 9.33. Plan of a 'manifold and lateral' under drainage system for a Rapid Gravity filter.

*The rate of application of wash water should not exceed the settling velocity of the smallest particle to be retained in the filter, which usually ranges between 0.3 to 0.9 m/minute ; thereby giving a back wash rate of about 300 to 900 litres/minute/m^2 of filter area.

(*i*) In ***perforated pipe type of this system***, the lateral drains are provided with holes at the bottom side. These holes are 6 to 13 mm in diameter and make an angle of 30° with the vertical, as shown in Fig. 9.34. The holes are spaced at about 7.5 to 20 cm centre to centre for 6 to 13 mm dia pipes, respectively. They may sometimes be staggered on either side instead of being continuous. These perforations (holes) are sometimes lined with brass bushings to prevent corrosion or rusting of the surfaces of the holes. Placing the openings on the underside of the laterals requires supporting the laterals on concrete blocks 40-50 mm above the filler bottom. The advantage is that the jet action of wash water is discouraged.

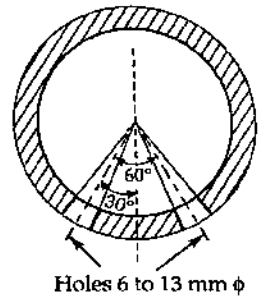


Fig. 9.34. X-section of a lateral pipe drain with perforations.

(*ii*) In ***pipe and strainer type of this system***, the laterals are attached to the Manifold as in the above system, but holes are not drilled into the laterals. Instead of drilling holes, *strainers* are placed on the lateral drains, as shown in Fig. 9.35. A strainer, in its simple form, is a small brass pipe closed at its top by a perforated cap. Such strainers are placed at about 15 cm apart on lateral drains instead of drilling holes into them. Various forms of strainers have

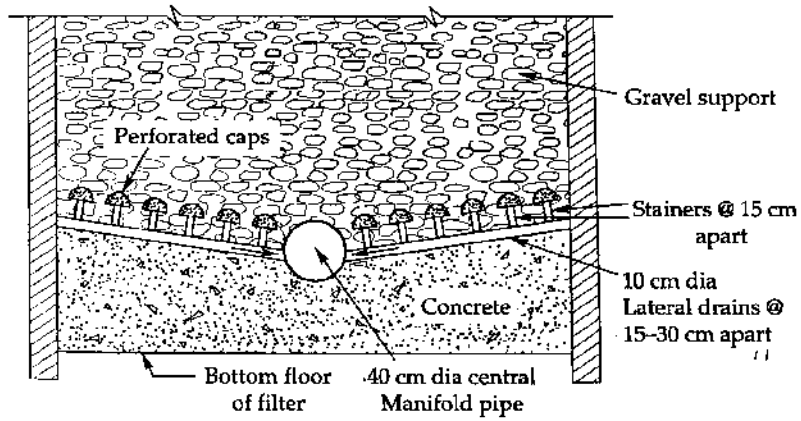


Fig. 9.35. Section of the 'manifold and lateral' system, in which laterals are provided with strainers.

been devised by different manufacturers of filter units. In certain cases, the strainers are even fixed directly on the central drain without any laterals.

When pipe and strainer system is provided, compressed air is used while back washing the filter. This results in the saving of the wash water.

When no strainers are used (*i.e.*, in the perforated pipe system), the agitation of sand particles is done by water-jet, and the back wash is, therefore, required at a high velocity. The rate of water application being of the order of 600 to 800 litre/minute/sq. m of filter area ; but when strainers are used (*i.e.* in pipe and strainer system) wash water is required at low rate of

about 250 to 300 litres/minute/ sq. m of filter area, because compressed air assists in agitating the sand particles. The former is known as **high velocity wash**, and the latter is known as **slow velocity wash.**

The following points may also be considered and kept in mind, while designing the sizes of the pipes to be used in the above system :

(*a*) The total cross-sectional area of perforations should be about 0.2 per cent of the total filter area.

(*b*) The cross-sectional area of each lateral should be about 2-4 times the total cross-sectional area of perforations in it, for diameter of perforations 10 mm and 5 mm, respectively.

(*c*) The cross-sectional area of the manifold should be about *twice* the cross-sectional areas of the lateral drains.

(*d*) $\frac{\text{Length of each lateral}}{\text{Diameter of the lateral}} \nmid 60.$

(*e*) Maximum permissible velocity in the manifold to provide the required amount of wash water, is about 1.8 to 2.4 m/sec.

(5) **Other appurtenances.** The more important among them are :

(*i*) Wash water troughs (*ii*) Air compressor

(*iii*) Rate controller (*iv*) Miscellaneous accessories.

These appurtenances are briefly described below :

(*i*) ***Wash Water Troughs.*** The rising dirty wash water which comes out of the filter after cleaning it, is collected in *wash water troughs* or *gutters*, and carried to the main gutter. These gutters may be square, V-shaped, or semi-circular in section. They may be constructed of concrete, steel, or fibre glass. Concrete troughs have vertical sides and V-shapes bottom. The lips of the troughs are horizontal and are all placed at the same height. They should be set at such a height (of about 0.4-0.6 m) that the overflow lip will be at, or somewhere above the top of the sand rise, otherwise, sand may be washed out of the filter. Their tops are, therefore generally kept atleast above the sand-bed by a distance of about half the depth of sand*, and their bottoms kept atleast 5 cm above the top of the sand surface. Flat bottom troughs, particularly with clearance above the sand less than half of the depth of sand, cause *sand boiling* during washing, and are, therefore, avoided.

Since the wash water should not travel more than 1 m laterally in order to minimize pressure difference in the filler, these troughs are generally spaced at about 1.5 to 2.0 m (x) apart, which spacing, limits the horizontal distance travelled by the wash water before entering the trough to 0.75 to 1.0 m. The water surface during washing is hence maintained, as nearly as possible, at a uniform level. For efficient working, these troughs should be large enough and should be laid at suitable slopes. They must be made of such capacity that their sides will act as free-falling weirs for the wash water.

(*ii*) ***Air Compressor.*** During back washing the filter, the sand grains are agitated either by water jet, or by compressed air, or by mechanical rakes. When compressed air is used, air compressor unit having the required

*Sand is likely to expand by about 50% during back washing.

capacity, must be installed. Generally, the compressor should be able to supply compressed air for about 4 minutes or so, at a rate of about 600 to 800 litres/minute/sq m of filter area. The pressure of the compressed air should be sufficient to overcome the frictional resistances offered by the air pipes and the column of water lying above the air distribution system. The compressed air is supplied either through the laterals or through separate pipe system.

(*iii*) ***Rate Controller.*** In order to obtain automatically, a uniform rate of filtration irrespective of the head loss through the filter, rate controllers are required to be fitted at the outlet end of each filter unit. The most commonly used type of a controller is a **venturi-rate controller**, which works on the principle of a venturimeter. A typical of such a controller is a *simplex rate of flow controller* and is shown in Fig. 9.36. It consists of a balanced disc valve

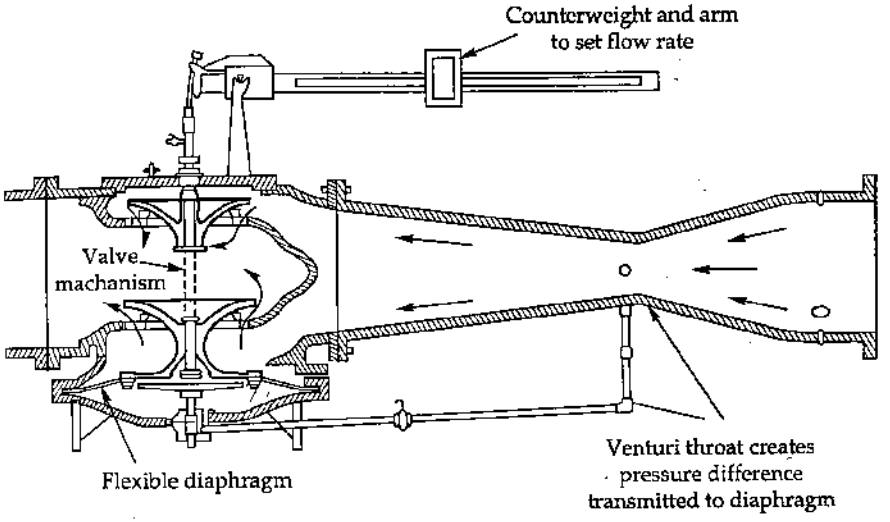


Fig. 9.36. Simplex-rate of flow controller.

connected to a flexible diaphragm below, and to a lever and a weight above. The water enters this valve through a venturitube. A small pipe transmits pressure at the throat of the tube to the under-side of the diaphragm. When the rate of flow increases, the pressure difference between the upper and the lower side of the diaphragm increases. The upper side will be exerted upon by higher pressure because of its connection to the larger end. This will push down the valve till the pressure difference becomes small enough to balance the pull of the lever and the weight. As the filter clogs with its use, and the flow decreases, the reverse action takes place, and thereby, giving a constant rate of outflow.

(*iv*) ***Miscellaneous Accessories.*** Besides the above appurtenances, other miscellaneous devices, such as *head loss indicators*, *meters* for measuring rates of flow, etc. are also needed. A **head loss indicator** is nothing but a differential type of a mercury gauge, one end of which is connected to the water resting on the sand bed and the other to the effluents coming from the filter. Arrangements can also be made for recording the readings on a time

scale. Meters are installed for measuring discharges at entry and outlet, and also at backwash.

9.17.2. Working and Cleaning of Rapid Gravity Filters. The working of a rapid gravity filter is explained below with reference to Fig. 9.31. In this figure, various valves have been numbered as enumerated in Table 9.3 :

Table 9.3

Valve No.	*Name of Valve*
1	Inlet valve.
2	Waste water valve to drain water from inlet chamber.
3	Waste water valve to drain water from main drain.
4	Filtered water supply valve.
5	Compressed air valve.
6	Wash water supply valve.

Valve 1, is, first of all, opened, which leads the effluents of the coagulation sedimentation basin, to enter the inlet chamber of the filter. This water gets filtered through the filter beds, and the filtered water can be taken out from the main drain by opening valve 4. This filtered water can be taken to the disinfection unit. Thus, when filter is in working condition, only these two valves (*i.e.* valve 1 and 4) shall be kept open, and all other valves kept closed.

Back washing. When sand becomes dirty, as is indicated by the excessive loss of head, the filter must be cleaned and washed. For cleaning, the raw supplies as well as the filtered supplies have to be cut off, bed drained down, and wash water sent back upward through the filter beds. This forced upward movement of wash water and compressed air (if used) will agitate the sand particles, and thus removing the suspended impurities from it. This can be accomplished as follows :

Valves 1 and 4 are closed, and valves 5 and 6 are opened. The wash water and compressed air are thus forced upward from the under-drainage through the gravel and sand beds. Valve 5 is closed after supplying the required amount of air. The dirty water resulting from washings, overflows into the wash water troughs, and is removed by opening valve 2, through the inlet chamber into the wash water gutter. The process of washing the filters and removing the dirty water is generally continued for a period of 3 to 5 minutes.

After the washing of the filter has been completed, valves 2 and 6 will be closed, and valves 1 and 3 are opened. This restores the inlet supplies through valve 1, but the filtered water, in the beginning, is not collected, and washed for a few minutes through valve 3 to the gutter. This is necessary because the remains of the wash water must be removed from the voids of the filter and a surface mat must be allowed to be formed on sand. Ultimately, valve 3 is closed and valve 4 is opened to get the filtered supplies again.

The entire process of back washing the filters and re-maintaining filtered supplies, takes about 15 minutes ; and the filter unit remains out of operation for this much of time. The amount *of water required for washing a rapid gravity filter may vary from 2 to 5 per cent of the total amount of water filtered.*

The rapid gravity filters get clogged very frequently and have to be washed every 24 to 48 hours.

The rate of washing may vary between 15—90 cm rise per minute. A figure of 45 cm/minute is quite acceptable. This rate can be provided by a flow of 500 l/m^2 of bed area/minute. Normally 10 to 15 minutes are required in washing, but in recommissioning, a total of about 30 minutes may be consumed. The pressure, at which wash water is supplied, is about 40 kN/m^2.

Large volumes of wash water containing low concentration of solids (100-1000 mg/L), will, thus, be evolved from the water treatment plants, using filters. The filter-back wash, may contain large fraction of organic solids, much larger than that contained in the sludges of the sedimentation flocculation tanks, because filters do support biological growth. The filter sludge will have to be suitably disposed of after thickening, to ensure its safe disposal. The plant authoritics however do not care much for safe disposal of filter backwash and have been found to be disposing it of in natural drainage channels or water courses, which not only causes siltation of such drains but also promotes biological activity within the drainage channel, causing atmospheric pollution and promoting pollution of rivers and streams. As a matter of fact, this is happening even in Delhi, the capital city of India, where the sullage waste from Bhagirathi Water Treatment Plant is openly being discharged through a local drain into river Yamuna, and the plant authorities are just not taking any action to develop advanced methods of dewatering and sludge disposal, such as those pointed out in article 9.14.1.

The rapid gravity filters are conventionally cleaned by back washing, as explained above. However, the recent research and developments have shown that these filters can also be washed easily and efficiently by **surface wash.** In this process, clean filtered water is applied to the sand bed from above, by means of nozzles of special design. The rate of application of wash water may range from 200 to 600 litres/minute/sq m, and it may be applied under a pressure of 10 to 20 m head of water (*i.e.* 100—200 kN/m^2). The nozzles may either be of the fixed or of swivelling type. The water, letting out of the nozzles, breaks up the filter surface, and scours the sand particles, thereby cleaning the top layers easily. Since the top layers normally contain the major trapped impurities, which cannot be easily cleaned by back washing without causing troubles such as formation of mud balls*, etc., surface wash may be used to supplement the back wash. This will increase the efficiency and quality of cleaning operations.

Loss of head and necessity of cleaning. The water percolating through the filter moves downward under the force of gravity. This motion is opposed by the resistance offered by the sand grains and the impurities arrested in them. The percolating water, therefore, looses some of its head. The loss of head can be easily computed by knowing the water level in the filter and the pressure of water in the outlet pipe. The difference between the two heads will give the loss of head, called ***filter head*** or ***filtration head***.

This loss of head is measured by inserting two piezometers, one in the water standing over the filter and the other in the outlet pipe, as shown in Fig. 9.31.

*Explained in the next article under "Operational Troubles".

The difference in the readings of these two piezometers will give the loss of head.

When the filter is newly commissioned, the loss of head, usually called ***initial head loss***, is generally very small, and is of the order of 15 to 30 cm. However, the loss of head goes on increasing as the time passes, and as more and more impurities get trapped into it. A stage is finally reached when the frictional resistance offered by the filter media exceeds the static head of water above the sand bed. Most of this resistance is caused by the top 10 to 15 cm sand layer. The bottom sand then acts like more or less a vacuum, and water is sucked through the filter media rather than getting filtered through it. The fall of mercury level in the piezometer inserted in the outlet pipe below the centre line of the pipe, indicates the presence and extent of *negative pressure*. The ***negative pressure*** so developed, tends to release the air dissolved in water. It causes the formation of bubbles, which stick to the sand grains, and thereby seriously affecting the working of the filter. The phenomenon is known as **air binding**, as the air binds the filter and stops its functioning, thereby reducing the rate of filtration considerably.

The escape of air into the filter media to form bubbles, leading to *air binding*, may in fact, result from : (*i*) negative head ; (*ii*) from increase of water temperature as it passes through the bed ; (*iii*) and occasionally from oxygen released by algae. Trouble is specially likely to occur if the water is saturated or super-saturated with air in solution. Serious interference with filtration may result and when the filter bed is washed, the rising air-bubbles may cause uneven washing and a loss of media sand. **The remedial measures to avoid air binding are :**

(*i*) by *avoiding development of excessive negative head*, by frequently cleaning the filter, particularly as soon as the loss of head exceeds the optimum allowable values. On an average, the loss of head is generally limited to 2.5 m to 3.5 m, and the negative head is about 1.2 m.

The depth of water during filter operation may also be increased by 15-20 cm as compared to the normal depth to increase static head (to compensate the lost head) and, thereby reducing the negative head.

(*ii*) by *controlling the growth of algae.*

(*iii*) by *remedying conditions that super-saturate the water with air.*

(*iv*) by *taking precautions if possible against the water getting warmed, as it passes through the plant.*

9.17.3. Operational Troubles in Rapid Gravity Filters. Besides *air binding* (explained above), the two other common troubles which are generally encountered during the operation of rapid gravity filters, are :

(1) Formation of mud balls ; and

(2) Cracking of filters.

They are explained below :

(1) **Formation of mud balls.** The mud from the atmosphere usually accumulates on the sand surface, so as to form a dense mat. During inadequate washing of the filter, this mud may sink down into the sand bed. This mud then sticks to the sand grains and other arrested impurities,

thereby forming *mud balls*. These mud balls slowly and steadily go on increasing in size and weight. They may then sink down into the gravel, thus interfering with the upward movement of wash water during cleaning. They cause turbulence around them, and thus hinder with the uniform application of wash water. The high velocities created around the edges of these balls, also displace the gravel, and thereby forming mounds ; while above the balls, the sand is improperly washed and mud accumulates. Thus, when once the mud ball formation starts, they go on increasing in number, until the entire space in the filter box gets filed up with them.

The various measures which may be adopted to control mud ball formation are given below :

(*i*) Mud balls may be broken with some *mechanical rakes*, and the broken mud particles are washed off.

(*ii*) Mud balls can also be broken by *water stream* by using a 10 mm ϕ pipe having a pointed closed end and two perforations a bit above the point. The pipe is connected to a rubber hose furnishing water under pressure.

(*iii*) Compressed *air scour* during back washing for about 4 minutes supported with *manual surface raking* may help in effective removal of mud balls.

(*iv*) The bed of the filter is first washed, then water is drawn off to 10 cm depth above the bed, and *caustic soda* is applied. After 12 hrs of soaking, the sand is thoroughly agitated, preferably with air wash, and 8 hrs later it is washed until the water runs clear.

The use of surface wash technique inplace of old conventional back washing technique has ofcourse proved very successful in eliminating mud ball formations.

(*v*) When the filter gets badly clogged and damaged, then it may have to be totally replaced.

(2) **Cracking of filters.** The fine sand contained in the top layers of the filter bed, shrinks and causes the development of shrinkage cracks in the sand bed. These cracks are more prominent near the wall junctions. With the use of filter, the loss of head and, therefore, the pressure on the sand bed goes on increasing, which further goes on widening these cracks. The floc, mud and other impurities arrested in the filter, penetrate deep into the filter through these cracks, and thus impairing both the washing of the filter and the efficiency of filtration.

The remedial measures for these consequent troubles are the same as are adopted for preventing mud ball formations.

9.17.4. Rate of Filtration or Rate or Loading for Rapid Gravity Filters. The rate of filtration that can be obtained from rapid gravity filters is very high, and is generally of the order of 3000 to 6000 litres/hr/sq. m of filter area (*i.e.* 50 to 100 litres/minute/sq. m). This high rate of filtration leads to considerable saving of space as well as filter materials.

9.17.5. Efficiency and Performance of Rapid Gravity Filters. Rapid gravity filters, compared to slow sand filters, are less efficient in removing bacteria and turbidities. They can remove about 80 to 90 per cent of bacterial

load present in water. The remaining bacteria are removed in disinfection units. They can remove turbidities to about 35 to 40 mg/l. But since the waters entering these filters are given pretreatments in coagulation sedimentation tanks, they are comparatively less turbid. Such turbidities, can be easily removed by these filters and brought to permissible limits.

As far as colour removal is concerned, they are highly efficient and can bring down the colour to as low value as 10 on cobalt scale.

9.17.6. Uses of Rapid Gravity Filters. Rapid gravity filters are the best and most economical and, therefore, invariably used for treating public supplies, especially for large towns and cities. The treated waters are, however, not so much safe as those obtained from slow sand filters, and need further treatment before they can be supplied to the general public.

Example 9.20. *Design the approximate dimensions of a set of rapid gravity filters for treating water required for a population of 50,000 ; the rate of supply being 180 litres per day per person. The filters are rated to work 5000 litres per hour per sq. m. Assume whatever data are necessary, and not given.*

Solution. Assume that the given rate of supply is an average demand, and also assuming that the max. daily demand is 1.8 times the average daily demand, we have, by ignoring wash water requirements :

The max. water demand per day

= Population × Max. daily rate of supply

= 50,000 × 1.8 (180) = 16.2×10^6 litres

= 16.2 million litres

Water demand per hour (ignoring time lost in cleaning)

$$= \frac{16.2 \times 10^6}{24} \text{ litres/hr} = 675 \times 10^3 \text{ litres/hr}$$

Rate of filtration = 5000 litres/hr/sq. m

∴ Area of filter beds required

$$= \frac{\text{Water demand}}{\text{Rate of filtration}} = \frac{675 \times 10^3}{5000} \text{ sq. m} = 135 \text{ sq. m}$$

Since two units are required to be designed,

the area of each unit $= \frac{135}{2} = 67.5$ sq. m

Assuming $L = 1.5\,B$, we have

$$1.5\,B^2 = 67.5$$

or $B^2 = 45$ or $B = 6.75$ m.

Choose 6.75 m width and 10 m length. Hence, two units of size 10 m × 6.75 m are reqd. One additional unit as stand-by may also be provided for breakdowns, repairs, or cleaning operations.

Example 9.21. *Design a rapid sand filter unit for 4 million litres per day of supply, with all its principal components.*

Solution. Water required per day = 4 M.L.

Assuming that 4% of filtered water is required for washing of the filter, every day, we have

Total filtered water required per day

$$= \frac{4 \text{ ML}}{0.96} = 4.167 = 4.167 \text{ M.L./day.}$$

Now, assuming that 0.5 hour is lost everyday in washing the filter, we have

Filtered water required per hour

$$= \frac{4.167}{23.5} \text{ M.L/hour} = 0.177 \text{ M.L./hr.}$$

Now, assuming the rate of filtration to be 5000 litres/hr/sq. m, we have

The area of filter required

$$= \frac{0.177 \times 10^6}{5000} \text{ m}^2 = 35.46 \text{ m}^2.$$

Now, assuming the length of the filter bed (L) as 1.5 times the width of the filter bed (B), and two beds, the total area provided

$$2 \times (L.B) = 35.46$$

or $2 \times (1.5\,B)\,(B) = 35.46$

or $B^2 = \frac{35.46}{3} = 11.82$

or $B = 3.44$ m

∴ $L = 1.5\,B = 1.5 \times 3.44 = 5.16$

say 5.2 m.

or Use the length of the filter bed as = 5.2 m, and

$$B = \frac{35.46}{2 \times 5.2} = 3.4 \text{ m.}$$

Hence, adopt 2 filter units, each of dimensions

5.2 m × 3.4 m. **Ans.**

Design of under-drainage system. Let a "manifold and lateral system" be provided below the filter bed, for receiving the filtered water, and to allow back washing for cleaning the filter, as shown in Figs. 9.33 and 9.34. This consists of a central *manifold pipe*, with *laterals* having perforations at their bottom.

To design this system, let us assume that the total area of the perforations in all the laterals is 0.2% of the total filter area.

∴ Total area of the perforations

= 0.2% × Filter area

$$= \frac{0.2}{100} (5.2 \times 3.4) \text{ m}^2 = \mathbf{0.035\ m^2.}$$

Now, assuming the area of each lateral

= 2 times the area of perforations in it

(for 13 mm dia perforations), we have

Total area of laterals = 2 × Total area of perforations

$= 2 \times 0.035 \text{ m}^2 = 0.070 \text{ m}^2$.

Now, assuming the area of the manifold to be about *twice* the area of laterals, we have

The area of manifold

$= 2 \times 0.070 = 0.14 \text{ m}^2$.

∴ Dia of manifold (d) is given by

$$\frac{\pi}{4} \cdot d^2 = 0.14 \quad \text{or} \quad d = \sqrt{\frac{0.14 \times 4}{\pi}} = 0.42 \text{ m}.$$

Hence, use a 45 cm dia manifold pipe laid length wise along the centre of the filter bottom. Laterals running perpendicular to the manifold (*i.e.* widthwise) eminating from the manifold may be laid at a spacing of say 15 cm (max. 30 cm). The number of laterals is then given as

$$= \frac{5.2 \times 100}{15} = 34.6 \text{ ; say } 35$$

on either side of the manifold. Hence, use 70 laterals in all, in each unit.

Now, length of each lateral

$$= \frac{\text{Width of filter}}{2} - \frac{\text{Dia of manifold}}{2}$$

$$= \frac{3.4}{2} - \frac{0.45}{2} = \frac{2.95}{2} = \mathbf{1.475 \text{ m}}.$$

Now, adopting 13 mm dia perforations in the laterals, we have

Total area of perforations = 0.035 m²

$$= 350 \text{ cm}^2 = x \cdot \frac{\pi}{4} (1.3)^2$$

where x = Total no. of perforations in all 70 laterals.

$$\therefore \quad x = 350 \times \frac{4}{\pi} \frac{1}{(1.3)^2} = 263.8 \text{ ; say } 264.$$

∴ No. of perforations in each lateral

$$= \frac{264}{70} = 3.8 \text{ ; } \mathbf{\text{say } 4}.$$

∴ Area of perforations per lateral

$$= 4 \times \left[\frac{\pi}{4} \times (1.3)^2\right] \text{ cm}^2 = 5.30 \text{ cm}^2.$$

Now, area of each lateral

= 2 × Area of perforations per lateral

$= 2 \times 5.30 = 10.60 \text{ cm}^2$

∴ Dia of each lateral

$$= \sqrt{10.60 \times \frac{4}{\pi}} = 3.67 \text{ cm ; say } 3.7 \text{ cm}.$$

Hence, use 70 laterals each of 3.7 cm dia, @ 15 cm c/c, each having 4 perforations of 13 mm size, with 45 cm dia manifold. **Ans.**

Check :

$$\frac{\text{Length of each lateral}}{\text{Dia of lateral}} = \frac{1.475 \text{ m}}{3.7 \text{ cm}} = \frac{147.5}{3.7} = 39.9$$

(which is less than 60, and hence O.K.)

Now, let us assume that the rate of washing of the filter be 60 cm rise/minute or 0.60 m/minute.

∴ The wash water discharge

$$= \frac{0.60 \times (5.2 \times 3.4)}{60} \text{ m}^3/\text{sec} = 0.177 \text{ m}^3/\text{sec}$$

∴ Velocity of flow in the laterals for wash water

$$= \frac{0.177}{70 \times \left[\frac{\pi}{4} \times \left(\frac{3.7}{100}\right)^2\right]} = \frac{0.177 \times 10000}{70 \times 10.75} \text{ m/sec}$$

$$= \frac{1770}{70 \times 10.75} = 2.35 \text{ m/sec}.$$

Similarly, velocity of flow in the manifold

$$= \frac{\text{Discharge}}{\text{Area}} = \frac{0.177}{\frac{\pi}{4} \times (0.45)^2} = \frac{0.177}{0.158}$$

= 1.12 m/sec ; which is less than 1.8 to 2.4 m/sec (max. permissible), and hence O.K.

Design of wash water troughs. Wash water troughs, as said earlier, are generally kept at about 1.5–2 m apart. So in a width of 3.4 m of filter bed, let us provide 3 troughs, at $\frac{3.4}{3} = 1.13$ m apart.

Now, the total wash water discharge of 0.177 m³/sec enters in these 3 troughs.

∴ Discharge in each trough

$$= \frac{0.177}{3} = 0.059 \text{ m}^3/\text{sec}.$$

The dimensions of a concrete V-bottom trough are now designed by using an empirical formula

$$\boxed{Q = 1.376\, b\, y^{3/2}} \qquad \ldots(9.21)$$

where Q = discharge in m³/s

b = width of trough in m = y (assume)

y = water depth in the trough in m

Let

$$\therefore \quad 0.059 = 1.376 \times y \times y^{3/2}$$

or $y^{5/2} = \frac{0.059}{1.376} = 0.0429$

$\therefore \quad y = (0.214)^{2/5} = 0.28 \text{ m} = 28 \text{ cm.}$

Keeping 5 cm freeboard, adopt the depth of trough

$= 28 + 5 = \mathbf{33 \text{ cm.}}$

Hence, 3 No. wash water troughs of size 27 cm × 33 cm may be used, as shown in Fig. 9.37. **Ans.**

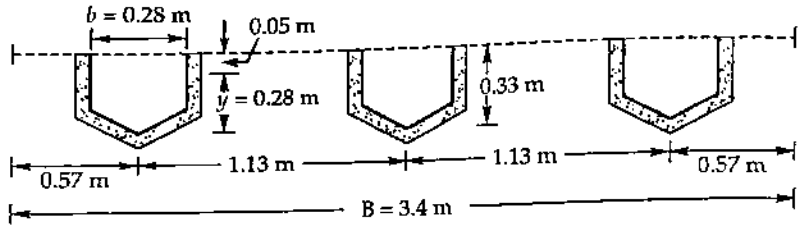


Fig. 9.37. Section of washwater trough.

Example 9.22. *A filter unit is 4.5 m by 9.0 m. After filtering 10,000 cubic metre per day in 24 hour period, the filter is back washed at a rate of 10 l/sq m/sec. for 15 min. Compute the average filtration rate, quantity, and percentage of treated water used in washing, and the rate of wash water flow in each trough. Assume 4 troughs.* (*Engg. Services, 1980 ; Civil Services, 1985*)

Solution. Area of filter $= 4.5 \times 9.0 \text{ m}^2 = 40.5 \text{ m}^2$

Filtered qty. in 24 hrs $= 10{,}000$ cum/day.

$$\text{Area of filter} = \frac{\text{Water filtered}}{\text{Rate of filtration}}$$

Rate of filtration in *l*/hr/sq m.

$$= \frac{10{,}000 \times 1000}{24 \times 40.5} \ l/\text{hr/m}^2$$

$$= 10{,}288 \ l/\text{hr/m}^2$$

Av. rate of filtration (by counting $\frac{1}{2}$ hr. lost in cleaning)

$$= \frac{10{,}288}{24.5} \times 24 = 10{,}078 \ l/\text{hr/m}^2. \quad \textbf{Ans.}$$

Amount of water used in cleaning @ 10 *l*/m²/sec for 15 min

$$= 10 \times (4.5 \times 9.0) \times (15 \times 60) \ l$$

$= 364500$ litres $= \mathbf{364.5 \text{ cum. Ans.}}$

Qty. of wash water expressed as percentage of total filtered water

$$= \frac{364.5 \times 100}{10{,}000} = 3.645\%. \ \textbf{Ans.}$$

Wash water discharge through each trough

$$= \frac{\text{Total wash water discharge through the filter}}{\text{No. of troughs}}$$

$$= \frac{10 \ l/\text{m}^2/\text{sec} \times (4.5 \times 9) \text{ m}^2}{4}$$

$= \mathbf{101.25 \ l/sec. \ Ans.}$

Example 9.23. *A surface water treatment plant coagulates a raw water having a turbidity of 9 Jackson candle units by applying an alum dosage of 30 mg/l. Estimate the total sludge solids produced in grams per cubic metre of water processed. Compute the volume of sludge from the settling basin and filter backwash water using 1.0% solid concentration in the sludge and 500 mg/l of solids in the wash-water. Assume that 30% of the total solids are removed in the filter. Use the following equation :*

$$\textit{Sludge solids (mg/l)} = \frac{\textit{Alum dosage (mg/l)}}{4} + \textit{raw water turbidity in Jtu.}$$

(*Engineering Services, 1979*)

Solution.

Total sludge solids produced

$$= \left(\frac{30}{4} + 9\right) \text{ mg/l} = 16.5 \text{ mg/l.}$$

$$= 16.5 \times \frac{1000}{1000} \text{ gm/cu m of water processed}$$

$= 16.5$ gm/cu m of raw water. **Ans.**

Consider 1 cu m of raw water processed

Sludge solids removed in the filter

$= 30\%$ of total solids.

$$= \frac{30}{100} \times 16.5 \text{ gm} = 4.95 \text{ gm.}$$

Filter back wash contains 500 mg/l of solids *i.e.* 500 gm/cu m of solids, *i.e.*, 500 gm of solids/cu m of wash water

$\therefore$ Vol. of filter wash

$$= \frac{1}{500} \times 4.95 \text{ cu m.}$$

$= \mathbf{0.0099 \text{ cu. m} = 9.9 \text{ litres}}$

Hence, the Vol. of sludge from the filter back wash

$= 9.9$ litres. **Ans.**

Total solids removed in the settling basin

$= 70\% \times 16.5 \text{ gm} = 11.55 \text{ gm.}$

Sludge contains 1% solids mean that 100 gm of wet sludge will contain 1 gm of solids and 99 gm of water.

Wt. of wet sludge in the settling basin

$= 1155$ gm (∵ 1 gm of solids in the wet sludge mean 100 gm of wet sludge)

$= 1.155$ kg

Since this sludge largely contains water (99%) its unit wt. can be assumed to be that of water *i.e.* 1000 kg/m^3.

∵ Vol. of this wet sludge

$$= \frac{1.155 \text{ kg}}{1000 \text{ kg/m}^3} = \frac{1.155}{1000} \text{ m}^3$$

$$= \mathbf{0.001155 \text{ cu m.}} = 1.155 \text{ litres :}$$

say 1.16 litres

Hence, Vol. of sludge produced per cu m of raw water :

(*i*) In the settling tank = 1.16 litres/m^3 of raw water
(*ii*) In the filter wash = 9.9 litres/m^3 of raw water
Ans.

9.18. Design of Filtering Sand Media for Rapid Gravity Filters

The size of sand to be used as filter media, as pointed out in article 9.14, is measured and expressed by the term called the *Effective Size* (E.S.), which is D_{10} *size, defined as the size (mm) of the sieve through which ten percent of the sand sample by mass will pass.* The selection of the correct effective size is very important, because too smaller size will lead to very frequent clogging of filter and will give very low filtration rate and higher head losses ; while too large size will permit the suspended particles and bacteria to pass through it, without getting removed or absorbed in the sand layers.

Another important parameter used in filter sand design is the **Uniformity Coefficient** (U), which is the ratio of D_{60} size and D_{10} size ; *i.e.*

$$U = \frac{D_{60}}{D_{10}} \quad \text{or} \quad D_{60} = U \,.\, D_{10} \qquad ...(9.28)$$

Experience has suggested that for rapid gravity filters, the effective size (D_{10}) of silica sand to be used should be in the range of 0.35 to 0.55 mm, with a maximum value of about 1.0 mm. The uniformity coefficient should range between 1.3 to 1.7. *It may be noted that as the uniformity coefficient increases, the sand becomes less uniform.*

In order to provide a less uniform sand media, the uniformity coefficient should be sufficiently higher than 1 (at which value, the sand will be totally uniform). Practically, since very uniform sand can not be produced by manufacturers, it becomes practically difficult to produce sand of uniformity coefficient below 1.3 ; while a value above 1.5, will have cost premium associated with it. Hence, inorder to obtain less expensive and less uniform sand media, the locally available stock sand is used, the characteristics of which are determined by performing sieve analysis, as to plot *the grain size distribution curve* for the stock sand. This sand is finally used for the filter media after screening out the coarser particles and washing away the finer ones. The coarser and finer quantities that are required to be removed from the stock sand, as to obtain sand of the designed values of D_{10} and D_{60}, are worked out as indicated below :

In a stock sand that does not meet the required specifications of D_{10} and D_{60} sizes, there is a usable portion (P_{use}), a portion that is too fine (P_f), and a portion that is too coarse (P_c). Therefore

$$P_{use} + P_f + P_c = 100 \qquad ...(9.29)$$

All of the sand that lies between the specified D_{10} size and D_{60} size is usable.

Let $P_{st(10)}$ represents the "percentage finer than (P)" of stock sand corresponding to the designed D_{10} size. Similarly, let $P_{st(60)}$ represents the "percentage finer than (P)" of stock sand corresponding to the specified D_{60} size. These values can be read out from the grain size distribution curve of the stock sand.

The stock of sand that lies between D_{10} and D_{60} sizes comprises 50% of the specified sand. The total usable sand is

$$P_{use} = 2\left[P_{st(60)} - P_{st(10)}\right] \qquad ...(9.30)$$

Ten percent of the usable sand can be below the specified D_{10} size. The percentage of usable stock sand below the D_{10} size is, thus, equal to 0.1 P_{use}. The percentage of stock sand that is too fine is thus, given as :

$$P_f = P_{st(10)} - 0.1\, P_{use} \qquad ...(9.31)$$

or
$$P_f = P_{st(10)} - 0.2\left[P_{st(60)} - P_{st(10)}\right] \qquad ...(9.32)$$

For the portion of the stock sand that is smaller than the specified D_{10} size, it would be most desirable to remove the smallest sizes and retain the larger sizes in the amounts required.

The percentage of stock sand that is too coarse (P_c) is the remaining portion determined from equation (9.29) as :

$$\therefore \quad P_c = 100 - P_f - P_{use}$$

$$= 100 - \left[P_{st(10)} - 0.2\left(P_{st(60)} - P_{st(10)}\right)\right] - 2\left[P_{st(60)} - P_{st(10)}\right]$$

or
$$P_c = \left[100 - P_{st(10)} - 1.8\left(P_{st(60)} - P_{st(10)}\right)\right] \qquad ...(9.33)$$

Again, it is desirable to retain sand as close as possible to $P_{st(60)}$ size. The coarse portion P_c to be removed, will consist of the largest grain size.

Note. *In rapid gravity filters, the removal of "fines" is usually accomplished by stratifying the filter bed through backwashing and then scrapping of the layer that includes the unwanted finest sand.*

Example 9.24. *Filter sand is to be prepared for a rapid sand filter from the stock sand for which the details of sieve analysis are given below :*

Sieve size mm	*Mass retained (g)*	*Sieve size mm*	*Mass retained (g)*
2.0	0	0.50	168
1.5	100	0.40	159
1.2	125	0.30	165
1.0	150	0.20	95
0.9	210	0.10	120
0.8	115	0.08	51
0.75	100	0.06	33
0.70	170	Finer	04
0.60	235		

The required effective size and the uniformity coefficient of the filter media is 0.3 mm and 2.5, respectively.

Determine amount of usable sand and percentage and sizes above and below which the sand is too coarse or too fine.

Solution. The required sand for filter should have the following specifications :

$D_{10} = 0.3$ mm (given effective size)

$D_{60} = U \times D_{10} = 2.5 \times 0.3 = 0.75$ mm.

The sieve analysis of the given stock sand is now analysed as shown in Table 9.4 to work out "Percentage (P), finer than each of the given size (d).

Table 9.4

Sieve size d (mm)	*Mass retained g*	*% age mass retained* $\frac{(2)}{2000} \times 100 = \frac{(2)}{20}$	*Cumulative % mass retained*	*Cumulative % age finer than d size* (P_{st}) *100 – (4)*
(1)	*(2)*	*(3)*	*(4)*	*(5)*
2.0	0	0	0	100
1.5	100	5.00	5.00	95
1.2	125	6.25	11.25	88.75
1.0	150	7.50	18.75	81.25
0.9	210	10.50	29.25	70.75
0.8	115	5.75	35.00	65.00
0.75	100	5.00	40.00	**60.00**
0.70	170	8.50	48.50	51.50
0.60	235	11.75	60.25	39.75
0.50	168	8.40	68.65	31.35
0.40	159	7.95	76.60	23.40
0.30	165	8.25	84.85	**15.15**
0.20	95	4.75	89.60	10.40
0.10	120	6.00	95.60	4.40
0.08	51	2.55	98.15	1.85
0.06	33	1.65	99.80	0.20
Finer	04	0.20	100.00	0
Σ	2000	= 100.00		

These values are designation P_{st} as against the ordinary designation P, since st represents P values for stock sand :

A grain size distribution curve can now be plotted between values of size, d (col. 1) and %age finer than, P_{st} (col 5) on a semi log graph paper by taking values of d on x-axis and values of P_{st} on y-axis, so as to read out P_{st} values corresponding to the specified size values of $D_{10} = 0.3$ mm and $D_{60} = 0.75$ mm. However, since these values can be directly read from the tabulated values of Table 9.4, plotting of such a grain size distribution curve is not necessary at this stage. It can hence be read out that

$P_{st(10)}$ = %age finer than 0.3 mm size = **15.15%**

$P_{st(60)}$ = %age finer than 0.75 mm size = **60.00%**

Now, using Eq. (9.30), we have

$$P_{use} = 2\left[P_{st(60)} - P_{st(10)}\right]$$

$$\therefore \quad P_{use} = 2[60 - 15.15] = \mathbf{89.7\%}$$

Thus, 89.7% of stock sand can be used and the remaining **10.3%** *will have to be screened or washed away.* **Ans.**

Too fine stock sand (P_f) to be removed is computed by Eq. (9.31) as :

$$P_f = P_{st(10)} - 0.1\, P_{use}$$
$$= 15.15 - 0.1 \times 89.7 = 15.15 - 8.97 = \mathbf{6.18\%}$$

Hence 6.18% of finest sand will have to be washed out. **Ans.**

Too coarse stock (P_c) to be removed is computed from Eq. (9.29) as :

$$P_{use} + P_f + P_c = 100$$

$$\therefore \quad 89.7 + 6.18 + P_c = 100$$

or $$P_c = 100 - 89.7 - 6.18 = \mathbf{4.12\%}$$

Hence, 4.12% of the coarsest sand will have to be screened out. **Ans.**

To determine the sizes above and below which the sand is too fine and too coarse, it is necessary to plot the grain size distribution curve, which is plotted as shown in Fig. 9.38.

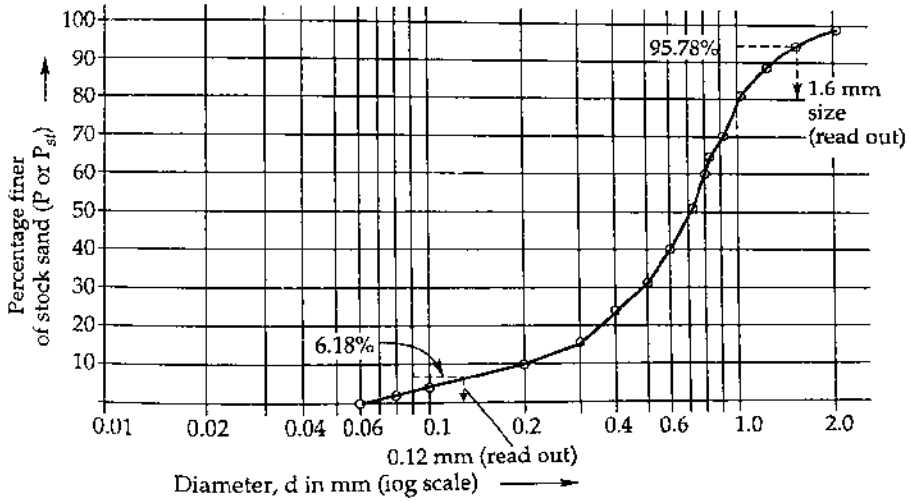


Fig. 9.38. Grain size distribution curve for Solved Example 9.24.

To determine size of 4.12% coarsest sand, we read particle size (d) corresponding to P or $P_{st} = 100 - 4.12 = 95.78\%$. This size is read out as about 1.6 mm. Hence, sand coarser than 1.6 mm size will be screened out. **Ans.**

Similarly, to determine size of 6.18% finest sand, we read the size for P or $P_{st} = 6.18\%$ as 0.12 mm. Hence, sand finer than 0.12 mm has to washed out. **Ans.**

or $$\boxed{D_e = \frac{(1-n)D}{(1-n_e)}} \quad ...(9.39)$$

The expanded bed depth can thus, be computed by Eq. (9.39).

This computation will help in determining the placement of the backwash troughs above the filter bed, as the bottom of these troughs should be kept at least 0.15 m above the expanded bed to prevent loss of filter material.

In case, the sand media is not uniformly represented by a representative grain size d, and consists of a stratified bed of non-uniform particles, then equation (9.39) can be modified to compute value of D_e as :

$$D_e = (1-n)D \,.\, \sum \frac{f}{1-n_e} \quad ...(9.39a)$$

where f = mass fraction of sand of various sizes in the sand (as per sieve analysis)

The quantity n_e is a function of the terminal settling velocity of the particles and the **backwash velocity (upflow velocity)**. An increase in backwash velocity will result in a greater expansion of the bed. The expression commonly used to relate the bed expansion to backwash velocity (v_b) and particle settling velocity (v_s) is given as :

$$n_e = \left(\frac{v_b}{v_s}\right)^{0.22*} \quad ...(9.40)$$

where n_e = porosity of expanded bed

v_b = backwash velocity (*i.e. backwash flow divided by the total filter area) in m/s*

v_s = settling velocity of the particle of given size d (in m) in m/s.

The computation of D_e from equation (9.39) and (9.40), however, is not straight forward. From Eq. (9.40), it becomes obvious that the expanded bed porosity (n_e) is a function of the settling velocity (v_s). The particle settling velocity (v_s) is determined by equation (9.2) as

$$v_s = \left[\frac{\frac{4}{3} g d \,.\, (G-1)}{C_D}\right]^{1/2} \quad ...(i.e.\ \text{Eq. } 9.2)$$

To solve this equation (*i.e.* Eq. 9.2), we must calculate the drag coefficient (C_D). The drag coefficient is a function of the Reynold number R_e (Eqns. 9.4 to 9.6), which, inturn, is a function of settling velocity $\left(R_e = \frac{v_s d \phi}{\nu}\right)$. We, thus, need the value of the settling velocity (v_s) to find the value of R_e, and then the value of C_D, and finally to compute the value of the settling velocity (v_s).

*Although this value had been commonly reported near 0.22 for sand, yet Amirtharajah (1978) has reported that a value of 0.4 to 0.5 is more correct for sand filter.

Knowing the sand grain diameter (d) and specific gravity (G), we can use Fig. 9.39 to obtain a first hand estimate for the settling velocity (v_s) to use in calculating the Reynold number (R_e), and to finally calculate C_D and v_s. This refined value of v_s can again be used to repeat the calculations, to arrive at near solution.

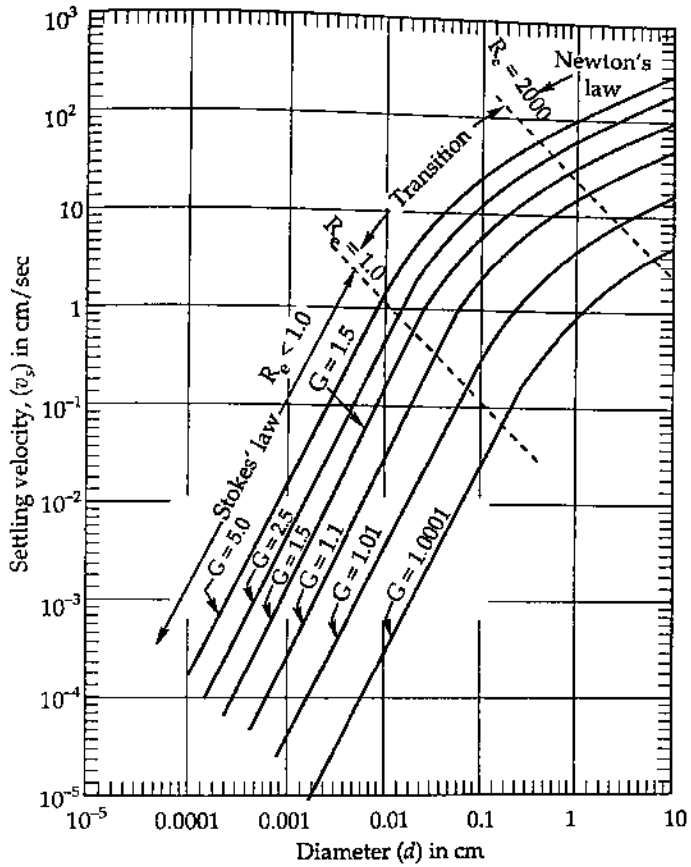


Fig. 9.39. Settling and Rising velocities of discrete spherical particles in quiescent water at 10°C.

For other temperatures, multiply the Stokes values by $\frac{\nu}{1.31\times10^{-2}}$, where ν is the kinematic viscosity of water at the given temperature (values given in table 9.1).

Backwash rates normally vary between 15 cm/min to 90 cm/min, with preferred value of about 45-60 cm/min. However, the limiting factor in choosing a backwash rate is the terminal settling velocity of the smallest sand grains that are to be retained in the filter. Since the filter backwashing process is effectively an upflow clarifier, the backwash rate becomes the overflow rate that determines whether a particle is retained in the filter or is washed out through the backwash trough.

Example 9.25. *In a water treatment plant, raw water is passed through a filter bed of uniform sand at a velocity of 5 m/hour. The filter is made of sand*

grains of diameter = 0.4 mm, shape factor = 0.85, and specific gravity = 2.65. The depth of the bed is 0.67 m and the porosity is 0.4. Determine the head loss through the bed (Take density of water = 998 kg/m³, and dynamic viscosity = 1.0 × 10⁻³ kg/ms.) *(Civil Services, 2002)*

Solution. The *head loss through a clean stratified sand filter with uniform porosity* and having representative particle size d, is given by *Rose's equation*, as :

$$h_L = \frac{1.067\,(v)^2 D}{\phi \cdot g \cdot n^4}\left[\frac{C_D}{d}\right] \qquad \text{...(i.e. Eq. 9.34a)}$$

where h_L = head loss through the filter in m
v = approach velocity or filtration velocity is m/s
D = depth of filter in m
ϕ = shape factor for non spherical sand particles
g = acceleration due to gravity in m/s²
n = Porosity
C_D = Drag coefficient, given by the equations 9.4 to 9.6.

The equation to be used for computing C_D (out of Eq. 9.4 to 9.6) depends upon the value of Reynold No. (R_e), which is first calculated by using Eq. (9.3) as :

$$\text{Reynold's No.} = \frac{v \cdot d}{\nu}$$

where v = velocity of flow in m/s
ν = kinematic viscosity of water in m²/s
d = dia. of spherical sand particles
= ϕd for non spherical particles
where ϕ is shape factor

$$\therefore \qquad R_e = \frac{v \cdot \phi d}{\nu}$$

Substituting given values, we get

v = filtration velocity in m/s

$$= 5 \text{ m/hr (given)} = \frac{5 \text{ m}}{(60 \times 60) \text{ sec}} = 1.389 \times 10^{-3} \text{ m/s}$$

d = sand grains dia. = 0.4 mm = 0.4×10^{-3} m

$\phi d = 0.85 \times 0.4 \times 10^{-3} = 0.34 \times 10^{-3}$ m

$$\nu = \frac{\mu}{\rho_\omega} = \frac{1.0 \times 10^{-3}}{998 \dfrac{\text{kg}}{\text{m}^3}}\left(\frac{\text{kg}}{\text{m} \cdot \text{s}}\right)$$

$$= 1.002 \times 10^{-6} \text{ m}^2/\text{s}$$

$$\therefore \qquad R_e = \frac{(1.389 \times 10^{-3}) \times (0.34 \times 10^{-3})}{1.002 \times 10^{-6}}$$

$$= \mathbf{0.471} < 0.5$$

Since $R_e < 0.5$, the coefficient of drag force (C_D) can be calculated by using Eq. (9.5) as :

$$C_D = \left[\frac{24}{R_e}\right] = \frac{24}{0.471} = 50.92$$

$$\text{Now,} \qquad h_L = \frac{1.067 \cdot (1.389 \times 10^{-3})^2 \times (0.67)}{0.85 \times 9.81 \times (0.4)^4}\left[\frac{50.92}{0.4 \times 10^{-3}}\right]$$

$$= 822.5 \times \frac{10^{-6}}{10^{-3}} = \mathbf{0.822 \text{ m.}} \quad \textbf{Ans.}$$

Example 9.26. *A rapid sand filter is to be provided in a water treatment plant, to process the water for a town with a population of 2,75,000. The water demand is 200 litres/capita/day. The rate of filtration is 15 m³/m²/hour. Allow 5% of filtered water for storage to meet the backwash requirements. Each backwashing period is of 30 min. Determine the number of filters required allowing one as a standby unit. The available surface area configuration of filter unit is 10 m × 4 m. Also compute the up-flow velocity and headloss to expand the bed to 0.66 m from its original undisturbed depth of 0.6 m. The porosity of the bed is 0.50. Specific gravity is 2.5. The average particle size is 0.6 mm. The drag coefficient is 5.02. The flow is assumed to be transitional flow.*

Solution. Water required daily

= Population × Per capita demand
= 2,75,000 × 200 L/c/d
= 55 × 10⁶ L/d = 55 MLD ...(*i*)

Filtered water required for backwashing

= 5%
= 5% × 55 MLD = 2.75 MLD ...(*ii*)

∴ Daily water demand of filtered water

= (*i*) + (*ii*) = 55 + 2.75
= **57.75 MLD.**

Since 30 minutes (*i.e.*, 0.5 hr) is lost daily in back washing the filters, the effective time left for working of filter units

= 24 − 0.5 = **23.5 hr.**

∴ Filtered water required per hour = $\frac{57.75}{23.5}$ = **2.457 ML/h**

Now, filtration rate = 15 m³/m².h (given)

∴ Area of filter required

$$= \frac{\left[\dfrac{2.457 \times 10^6}{10^3}\right] \text{m}^3/\text{h}}{15 \text{ m}^3/\text{m}^2.\text{h}} = 163.83 \text{ m}^2$$

Size of filter (given) = 10 m × 4 m

Area of one filter unit = 40 m^2

No. of units required = $\frac{163.83}{40}$ = 4.01 ; say 4.

(*i*) Using one filter unit as stand by unit, total number of filters required = **5. Ans.**

Use eqn. (9.2) for settling velocity (v_s) as

$$v_s = \sqrt{\frac{\frac{4}{3} \cdot g \cdot d \cdot (G-1)}{C_D}}$$

$$= \sqrt{\frac{\frac{4}{3} \times 9.81 \times (0.6 \times 10^{-3})\,(2.5-1)}{5.02}}$$

$$= \mathbf{0.048\ m/s}$$

Now, use Eq. (9.39) as :

$$\frac{D_e}{D} = \left(\frac{1-n}{1-n_e}\right) \qquad \text{where } n = 0.5 \text{ (given)}$$

$$\therefore \quad \frac{0.66\ \text{m}}{0.60\ \text{m}} = \frac{1-0.5}{1-n_e}$$

or $$(1-n_e) = \frac{0.5 \times 0.6}{0.66} = 0.455$$

or $$n_e = \mathbf{0.545}$$

Now, use Eq. (9.40) as :

$$n_e = \left(\frac{v_b}{v_s}\right)^{0.22}$$

$$0.545 = \left(\frac{v_b}{0.048}\right)^{0.22}$$

or $$v_b = 3.04 \times 10^{-3}\ \text{m/s} = 0.304\ \text{cm/s} = 18.2\ \text{cm/min.}$$

Hence, the backwash upflow velocity = 18.2 cm/min. **Ans.**

Head loss through the expanded medium is given by Eq. (9.37) as :

$$h_{Le} = D_e(1-n_e)(G-1)$$

$$= (1-0.545)\ 0.66\ (2.5-1)$$

$$= 0.455 \times 0.66 \times 1.5$$

$$= \mathbf{0.45\ m.} \quad \textbf{Ans.}$$

9.20. Slow Sand Filters *versus* Rapid Gravity Filters

The differences between these two types of filters are brought out in Table 9.5.

Table 9.5. Comparison of Slow Sand and Rapid Gravity filters

S. No. (1)	*Item* (2)	*Slow sand filters* (3)	*Rapid gravity filters* (4)
1.	Pre-treatment requirements	Effluents either from plain sedimentation tanks or raw waters without any treatment are generally fed into them ; and coagulation is not at all required.	Coagulation, flocculation, and sedimentation is a must.
2.	Base material	The gravel base supports the sand. It varies from 3 to 65 mm in size and 30 to 75 cm in depth.	The gravel base supports the sand and also distributes the wash water uniformly on the surface of sand. It varies from 3 to 40 mm in size and its depth is slightly more, *i.e.* about 60 to 90 cm.
3.	Filter sand	The effective size of filter sand ranges between 0.2 to 0.4 mm and uniformity coefficient between 1.8 to 2.5 or 3.0.	The effective size of the filter sand ranges between 0.35 to 0.55 and uniformity coefficient between 1.2 to 1.8.
		The grain size distribution is generally uniform throughout the depth of filter media,, except that the top 10 to 15 cm may be laid of finer variety.	The sand is laid in layers with smallest grain size at the top and coarsest grain size at the bottom.
4.	Under-drainage system	Laid in order to receive filtered water.	Laid in order to receive filtered water and also to pass water for back washing at a very high rate.
		Open jointed pipes or drains covered with blocks may be used.	Perforated pipe laterals discharging into mains or diffuser plate bottom may be used.
5.	Size of each unit	Large, such as (30 m × 60 m). The area varying from 100 to 2000 sq m or more.	Small, such as 5 m × 8 m. The area varying from 10 to 80 sq. m.
6.	Rate of filtration	Small, such as 100 to 200 litres per hour per sq. m of filter area.	Large, such as 3000 to 6000 litres per hour per sq. m of filter area.
7.	Economy	High initial cost of both land and materials, but low cost of operation and maintenance.	Low initial cost, but higher cost of operation and maintenance. Overall, it is cheaper and economical.
8.	Depreciation cost	Relatively low.	Relatively high.

Contd..

Table 9.5. Continues

S. No. (1)	Item (2)	Slow sand filters (3)	Rapid gravity filters (4)
9.	Efficiency	Very efficient in removing bacteria (98 to 99 per cent) but less efficient in removing colour. Overall turbidity removal in these filters using plain sedimentation is also low. They cannot handle turbid waters containing turbidities more than 50 mg/l.	Less efficient in removing bacteria (80 to 90 per cent) but very efficient in removing colour. The overall turbidity removal in these filters using coagulation sedimentation, is also high. They can, therefore, handle very turbid waters.
10.	Flexibility	Not flexible for meeting variations in demand.	Quite flexible for meeting reasonable variations in demand.
11.	Suitability and adaptability	May be adopted for treating smaller village supplies or for individual industrial supplies, especially at hotter places where no covers are required to protect the filters from freezing. They are however, becoming obsolete.	They are widely and almost universally adopted for treating public supplies, especially at all major cities and towns.
12.	Post treatment, required, if any	Almost pure water is obtained. However, it *may be disinfected slightly* to make it completely safe. Other miscellaneous treat-ments may or may not be reqd.	*Disinfection is a must* and some other miscellaneous treatments may be given, if needed.
13.	Ease in construction	Simple	Complicated, as underdrainage is to be properly designed.
14.	Skilled supervision, if reqd.	Not required.	Essential.
15.	Loss of head	Approximately 0.1 m is the initial loss, and 0.8 to 1.2 m is the final limit when cleaning is required.	Approximately 0.3 m is the initial loss, and 2.5 to 3.5 m is the final limit when cleaning is required.
16.	Method of cleaning	(*a*) Scrapping and removing the top 1.5 to 3 cm thick layer, and washing down by hoses.	(*a*) Agitating the sand grains and back washing with or without compressed air.
		(*b*) Laborious method.	(*b*) Short and easy method.
17.	Qty. of wash water required	Very small amount of wash water, varying from 0.2 to 0.6 per cent of the total water filtered is generally needed.	Large amount of wash water varying from 1 to 5 per cent of the total water filtered, is generally required.
18.	Period of cleaning	Cleaned at intervals of 1 to 3 months.	Cleaned frequently at intervals of 1 to 3 days.

9.21. Pressure Filters

Pressure filters are just like small rapid gravity filters placed in closed vessels, and through which water to be treated is passed under pressure. Since water is forced through such filters at a pressure greater than the atmospheric pressure, it is necessary that these filters are located in air tight vessels. The raw water is pumped into the vessels by means of pumps. The pressure so developed may normally vary between 30 to 70 metre head of water, *i.e.* 300 to 700 kN/m^2.

9.21.1. Construction of Pressure Filters. The filter vessel may be installed either in a horizontal or in a vertical position, depending upon which, they may be classified as *horizontal pressure filters* or *vertical pressure filters*. Typical cross-sections of both these types of filters are shown in Figs. 9.40 (*a*) and 9.40 (*b*), respectively. Steel cylinders are used as pressure vessel and may

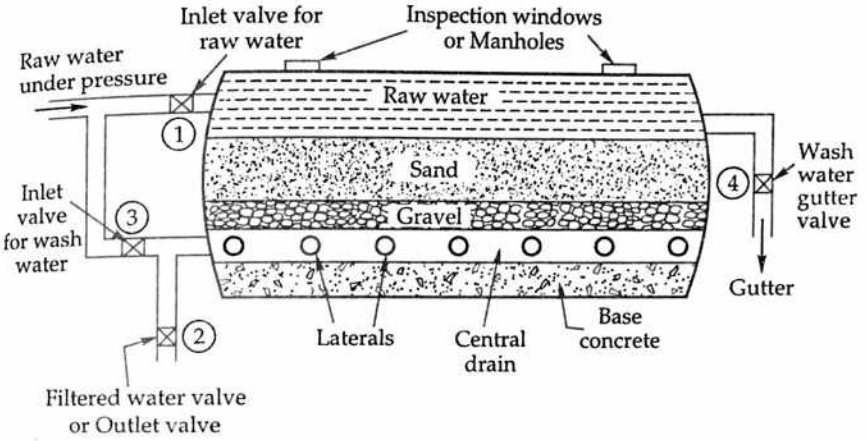


Fig. 9.40. (*a*) Horizontal pressure filter.

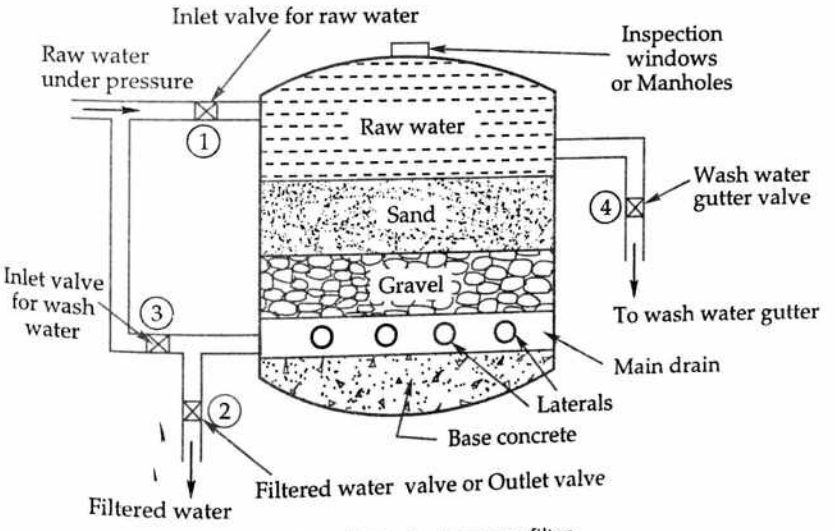


Fig. 9.40. (*b*) Vertical pressure filter.

be riveted or welded. Their diameters generally vary between 1.5 to 3 m, and their lengths or heights may vary from 3.5 to 8 m. Inspection windows are provided at top for inspection purposes. The positions of various valves are also clearly shown in the figures.

9.21.2. Working and Operation of Pressure Filters. A pressure filter is operated like an ordinary rapid gravity filter except that the raw coagulated water is neither flocculated nor sedimented before it enters the filter. The flocculation takes place inside the pressure filter itself. Under normal working conditions, the coagulated water under pressure enters the filter vessel through the inlet valve 1, and the filtered water comes out of the outlet valve 2. Hence, under this condition, only these two valves are kept open and all other valves are kept closed. The commonly used coagulant is alum and is kept in a pressure container connected to the influent line to the filter (not shown in figure). Little time is thus available for this coagulant to get mixed properly or to form floc outside the filter vessel.

The *cleaning* of the filter may be carried out by back washing as is done in a normal rapid gravity filter. The compressed air (though not shown in figure) may also be used, if designed, in order to agitate the sand gains. For cleaning, the inlet and outlet valves [*i.e.* valves 1 and 2] are closed and the wash water valve 3 and wash water gutter valve 4 are opened. After the completion of cleaning, these valves may be closed, and raw supplies restored. However, the filtered supplies should not be collected for a little time and wasted through valve 4, as is done in a rapid gravity filter.

These filters are cleaned when the loss of head due to clogging, exceeds a certain fixed value. Pressure filters may require a slightly more frequent cleaning as compared to that required by rapid gravity filters, because the impurities, which are removed in the sedimentation tank in case of rapid gravity filters, are also removed by the filter in the case of pressure filters. Compact automatic pressure filters, in which the cleaning is done automatically at a predetermined interval of time or loss of head, are now-a-days available.

9.21.3. Rate of Filtration of Pressure Filters. The pressure filters can yield filtered water at rates much higher, *i.e.* 2 to 5 times than what can be obtained from rapid gravity filters. Their rate of filtration normally ranges between 6,000 to 15,000 litres per hour per sq. m of filter area. The lower rates are used for domestic supplies, medium rates for industrial supplies, and higher rates for recirculating swimming pool supplies.

9.21.4. Efficiency and Suitability of Pressure Filters. The pressure filters are less efficient than the rapid gravity filters, in removing bacteria and turbidities. The quality of their effluent is poorer and they are, generally, not used for public supplies. But since a pressure filter provides a compact and an easy handling machine, they may be preferred for treating smaller quantities of comparatively clearer waters. Hence, they may be installed for colonies of few houses, individual industries, private estates, swimming pools, railway stations, etc.

9.21.5. Advantages and Disadvantages of Pressure Filters. The relative advantages and disadvantages of pressure filters over those of rapid gravity filters are given below :

Advantages

(*i*) A pressure filter is a compact machine and can be handled easily. Even completely automatic units have been designed.

(*ii*) It requires lesser space and lesser filtering material for treating the same quantity of water, because the rate of filtration is higher (2 to 5 times that of a rapid gravity filter).

(*iii*) Sedimentation and coagulation tanks are avoided.

(*iv*) They are more flexible, as the rate of filtration can be changed by changing the pumping pressure.

(*v*) When installed on a large scale for treating turbid waters, pressure filters, though prove costlier, yet may prove economical for treating smaller quantities of comparatively clearer waters. Hence, they can be best adopted for supplying waters to co-operative housing societies, individual industrial plants, private estates, swimming pools, etc.

(*vi*) Since the water coming out of the filter possesses sufficient residual head (as the only loss of head is that reqd. to force the water through the filter), the repumping of the filtered water is not required, as is the case with the rapid gravity filters.

Disadvantages

(*i*) Although the rate of filtration is high, the filter unit being smaller, the overall capacity of the plant is small.

(*ii*) They are less efficient in removing bacteria and turbidities, and hence, the quality of the filtered effluents is poorer.

(*iii*) They are costlier, particularly for treating large scale municipal supplies.

(*iv*) Since the process of filtration, as well as that of back washing, takes place in a closed tank, proper inspection and quality control is not possible.

(*v*) Inspection, cleaning and replacement of sand, gravel and under-drainage system is difficult.

(*vi*) Because of their inherent curved shapes, properly designed wash water gutters which can trap the washed impurities without flushing them back to other portions of the sand bed, cannot be provided easily.

(*vii*) Since these filters are operated under pressure, the normal tendency is to pump the water at higher rates, and thus obtaining still poorer quality of effluents.

Due to these disadvantages, pressure filters are generally not used for treating municipal surface supplies, particularly for any city or major industrial town. Rapid gravity filters are almost invariably and universally used under all such conditions.

9.22. Other Types of Filters

The following two other types of filters are also used in certain special cases/ situations :

(1) *Roughening filters*

(2) *Diatomaceous earth filters*

Both these types of filters are described below :

9.22.1. Roughening Filters and Double Filtration. The slow sand filters normally yield a very small rate of filtration (100 to 200) litres per hour per sq. m) due to trapping of large amounts of suspended impurities. The rate of filtration of such filters can be considerably increased (almost doubled) by using a preliminary filter (a rapid gravity filter of a higher filtration rate) which is called the *roughening filter*. The raw water (generally without coagulants) is, therefore, first of all, passed through the roughening filter which partly removes impurities. The effluents are then passed through the ordinary slow sand filter, so as to get a good quality supply and also at a higher rate. The water is thus filtered twice.

The roughening filter is a rapid gravity filter, utilising coarser sand than what is used in an ordinary rapid gravity filter ; thus giving a comparatively higher rate of filtration, of the order of 7000 litres per hour per sq. m. The roughening filter reduces the load on the slow sand filter, thereby increasing its filtration rate.

The double filtration with roughening filters increases the efficiency of slow sand filters, and also produces a better quality water. It may, therefore, be adopted at places where land available for the installation of slow sand filters is restricted.

Sometimes, some other combinations of double filtration may be adopted. For example, water may sometimes be passed through a rapid gravity filter and again passed through another rapid gravity filter. Similarly sometimes a coarser filter may be used preliminary to a rapid gravity filter. The double filtration no doubt produces a better quality water, free from colour, turbidity, bacteria, etc., but may prove to be uneconomical for treating large supplies. They may, however, be adopted in some smaller individual projects.

9.22.2. Diatomaceous Earth Filters or Diatomite Filters. This type of filters were basically developed by the Army for their field use to remove chlorine resistant dysentery cysts, and the cercariae of schistosomiasis which even passes through slow sand filters. They require very little space and hence can be used as portable units for field applications, or for industrial uses. They can also, though rarely, be used for treating municipal supplies for very small towns or colonies, besides their accepted use for swimming pools.

The filter Screen or Element consists of rigid cylindrical metal covered by a closely wound noncorrosive wire or fabric of metal, plastic or other material ; or they may be cylinders of fused alumina or porous stone (Fig. 9.41). While starting the operation of such a filter, a *precoat* layer of *diatomaceous earth* (*diatomite*) in the amount of about 0.3 kg/m^2 of filter area is allowed to get deposited on the filter screen to serve as an effective filter. The diatomite is kept in a precoat pot and raw water is allowed to enter through that pot, as shown in Fig. 9.41. The diatomite mixed raw water, passing through the filter screen, will thus, help in providing a precoat layer of diatomite on the filter screen. The water coming out of the filter is wasted, till such a coating layer gets fully established on the filter screen or filter media. Thereafter, diatomite

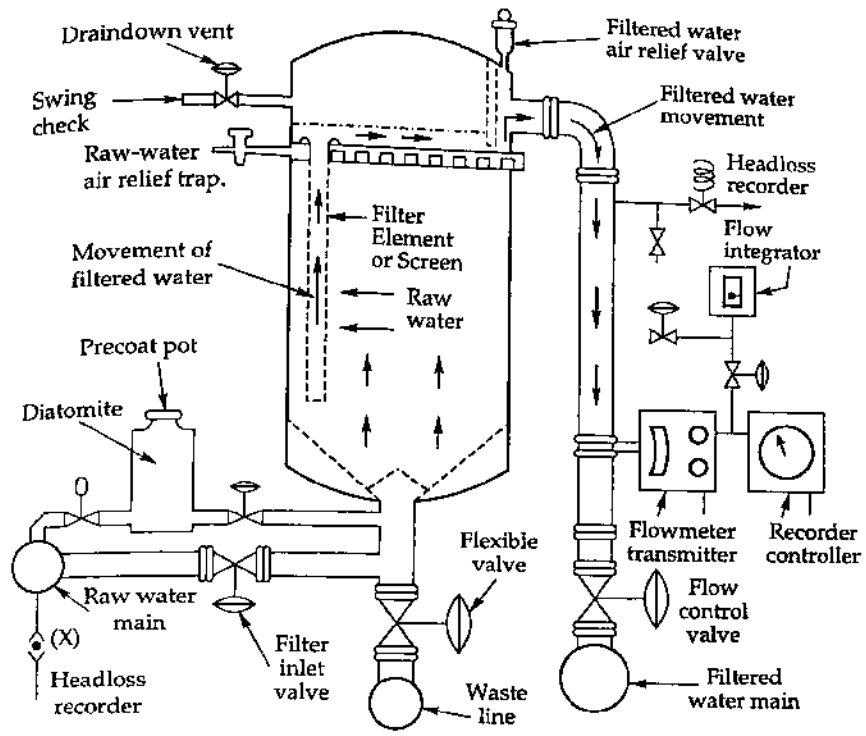


Fig. 9.41. Schematic diagram of a diatomite filter.

is added continuously as a slurry to the raw water to form a body coat. This keeps the mixture of turbidity etc. (removed from the water in filter) in porous condition, and allows longer runs between backwashings. Body coat feed quantity is based on the turbidity present in raw water, and the diatomite used for this purpose may be about 2 to 3 mg/l per mg/l of turbidity contained in raw water. The raw water mixed with this quantity of diatomite is thus, allowed to pass through the filter element formed by the deposition of initial layer of diatomite on the filter medium/screen. Body coat, thus, helps in continuously building the filter, so that surface clogging is minimised. When maximum pressure drop* or minimum filtration rate is reached, the filter medium is backswashed by using filtered water by reversing the flow, discharging the *dirty filter cake* to wasted water. Backwashing may be done with or without the use of auxiliary water jets. Some manufacturers of such filters provide "air bumps" to dialodge the dirty material from the filter element. Runs between washings may vary from a few hours to several days, depending upon the characteristics (primarily turbidity) of raw water and operation skill used in back washings.

Diatomite filters are not adapted to waters of high turbidity. The upper limit usually fixed on turbidity is 30 mg/l, although some what higher

*When the difference in pressure between the influent and effluent sides of a filter reduces to about 70 kPa.

turbidities have been successfully treated. *The raw water should also generally be chlorinated before application to the filter.*

Most state Govts. and municipalities do not approve the use of such filters for public supplies, since there exists only a thin layer of diatomite on the filter element, and raw water might pass through. Moreover, only a metal wall separates the raw and filtered water, and there are chances of their mixing, particularly due to carelessness in backwashing of the filter unit.

DISINFECTION OR STERILISATION

The filtered water which is obtained either from the slow sand filters or rapid gravity filters, may, normally contain some harmful disease producing bacteria in it. *These bacteria must be killed in order to make the water safe for drinking.* The chemicals used for killing these bacteria are known as *disinfectants,* and the process is known as *disinfection* or *sterilisation**.

This process of purification is the most important, because the bacterially contaminated water may lead to the spread of various diseases and their epidemics, thus causing disasters to public life. The presence of turbidities, colour, or minerals, etc., may not be dangerous, but the presence of even a single harmful organism will definitely prove dangerous, thereby making 'disinfection' as the most important process. The importance of 'disinfection' can also be visualised from the fact that in case of emergencies, such as for water supplies to armies in war times when complete treatment including sedimentation, filtration, etc. cannot be given, the disinfection is the only treatment which is given to the raw water. However, in normal cases, it is the final safeguard, and is given after filtration.

The 'disinfection' not only removes the existing bacteria from the water at the plant, but also ensures their immediate killing even afterwards, in the distribution system. The contamination of water during its transit from the treatment plant to the place of its consumption is also thus, prevented by the residual quantities of the disinfectant used. The chemical which is used as a disinfectant must, therefore, be able to give this ***residual sterilising effect*** for a long period, thus affording some protection against recontamination. In addition to giving this residual effect to prevent recontamination, the disinfectant chemical should be harmless, unobjectionable to tongue, economical, and measurable by simple tests. ***Chlorine*** has been found to be the best and the most ideal disinfectant and is now invariably used through out the world. The term ***Chlorination*** is hence, commonly used to indicate broadly that the water has been treated with a sterilising agent. *Disinfection, chlorination, and sterilisation are thus used as synonymous to each other.* Besides chlorine, there are, however, other minor methods of disinfection. We will discuss these methods before taking up 'chlorination', separately.

*Strictly speaking, ***disinfection*** means killing of only the disease producing bacteria ; whereas, ***sterilisation*** means killing of all the bacteria of all types. However, the two terms are generally used as synonymous to each other, in Water Supply Engineering, both meaning, the killing of harmful bacteria.

9.23. Minor Methods of Disinfection

The following are the minor methods of disinfection :

(1) Boiling of water ;
(2) Treatment with excess lime ;
(3) Treatment with ozone ;
(4) Treatment with iodine and bromine ;
(5) Treatment with ultra-violet rays ;
(6) Treatment with potassium permanganate ; and
(7) Treatment with silver, called *Electro-Katadyn process.*

These methods are summarised below :

(1) **Boiling of Water.** The bacteria present in water can be destroyed by boiling it for a long time. It is an effective method of disinfection, but it is not practically possible to boil huge amounts of public water supplies. Moreover, it can only kill the existing germs but cannot take care of the future possible contaminations. This method is hence, not at all used for disinfecting public supplies. However, during water borne epidemics, public is advised to drink water only after boiling it in their houses.

(2) **Treatment with Excess Lime.** Lime is generally used at a water purification plant for softening* (*i.e.* reducing hardness) the supplies. But it has been found that if excess lime is added to the water, it can in addition, kill the bacteria also. So much so, that an addition of 14 to 43 ppm of excess lime has been found to remove the bacterial load by about 99.3 to 100% from highly polluted waters.

The excess lime when added to water, in fact, raises the pH value of water, making it extremely alkaline. This extreme alkalinity (and even acidity) has been found detrimental to the survival of bacteria, thus killing them partially or completely. When the lime raises the pH of water to about 9.5, 99.9 to 100% bacteria are removed.

This method, though quite practical for complete removal of bacteria, yet needs the removal of excess lime from the water before it can be supplied to the general public. Treatment like *recarbonation* for lime removal should, therefore, be used after disinfection, when this method is used for disinfection. Moreover, it cannot protect the water from the possible dangers of recontamination, and hence not used these days. It was used in olden days, but with the advent of chlorine, its use has become obsolete.

(3) **Treatment with Ozone.** Ozone gas is a faintly blue gas of pungent odour, and is an excellent disinfectant. Ozone gas is nothing but an unstable allotropic form of oxygen, with each of its molecule containing three oxygen atoms. It can be produced by passing a high tension electric current through a stream of air in a closed chamber, under the following chemical reaction :

$$\underset{\text{Oxygen molecule}}{3O_2} \xrightarrow[\text{arc voltage}]{\text{under high electric}} \underset{\text{Ozone molecule}}{2O_3}$$

*Explained under "Softening" in article 9.25.

Because of its instability, ozone readily breaks down into normal oxygen, and releases *nascent oxygen* as below :

$$\underset{\text{Ozone}}{O_3} \longrightarrow \underset{\text{Oxygen}}{O_2} + \underset{\text{Nascent Oxygen}}{O}$$

The nascent oxygen, so produced, is a powerful oxidising agent and removes the organic matter as well as the bacteria from the water.

During the treatment, ozone gas is manufactured and bubbled through water contained in a sterilising chamber having inlets and outlets for water as well as an inlet for ozone. The ozonised water comes out through the outlet at the top.

About 2 to 3 ppm of ozone is generally consumed, so as to obtain a residual ozone of 0.10 ppm after a contact period of about 10 minutes. The residual ozone left in water can be easily measured by testing it with Orthotolidine test*. 0.1 ppm of residual ozone will give the same yellow colour as is given by 0.15 ppm of residual chlorine.

Ozone and chlorine are quite competitive disinfectants, each having its merits and demerits over the other. Chlorine is, however, preferred and almost universally adopted, although ozone is also largely used in France and Russia, and cities like Chandigarh in India.

The advantages and disadvantages of using ozone as disinfectant are discussed below :

Advantages of ozone

(*i*) Ozone being unstable, nothing remains in water, by the time it reaches the distribution system.

(*ii*) Ozone removes the colour, taste and odour from water, in addition to removing the bacteria from it.

(*iii*) The ozonised water becomes tasty and pleasant unlike the chlorinated water which becomes bitter to tongue.

Disadvantages or Limitations of ozone

(*i*) It is very costly ; much costlier than chlorination.

(*ii*) Ozone needs electricity for its manufacture, and hence it can be used only when electricity is available easily and cheaply.

(*iii*) No residuals can be maintained because it is highly unstable, and thus its use does not ensure safety against possible future contaminations.

(*iv*) Complicated ozone manufacturing apparatus called *ozoniser* is required to be installed at the treatment plants, because it cannot be supplied in cylinders as chlorine can be.

(*v*) It is less efficient than chlorine, in killing bacteria.

(4) **Treatment with Iodine and Bromine.** The addition of iodine or bromine to water can help in killing the pathogenic bacteria, and thereby disinfecting the same. The quantity of these disinfectants may be limited to about 8 ppm and a contact period of 5 minutes is generally enough. These disinfectants are now-a-days also available in the form of pills and are thus

*Described under "Chlorination" in article 9.24.5.

very handy. They are not used for treating any large scale public supplies, but may be used for treating small water supplies for army troops, private plants, swimming pools, etc.

(5) **Treatment with Ultra-Violet Rays.** Ultra-violet rays are the invisible light rays having wave lengths of 1000 to 4000 mμ. They are basically found in sun light, but can also be produced by passing electric current through mercury enclosed in quartz bulbs. Mercury vapour lamps enclosed in quartz bulbs can therefore, be used as a good source of such rays.

These rays are highly effective in killing all types of bacteria, thus yielding a truly sterilised water. The water to be treated with ultra-violet rays should, however, be less turbid and low in colour. Normally, it should be colourless and turbidity should not exceed 15 mg/l. For treatment, the water is passed several times around the quartz bulbs emitting such rays. The depth of water over the bulbs should not generally exceed 10 cm or so, because these rays can effectively penetrate to this much of distance only.

Sterilisation with ultra-violet rays does not impart any additional taste or odour to the water, as no chemicals are added. But it is very costly, needs technical know-how, and possesses possibilities of interruption due to failure of electricity. This method is, hence, unsuitable for treating large scale public supplies. The method, however, possesses ample scope for treating small quantities of water in hospitals, and dispensaries for surgical uses, or for drinking purposes at places where the cost is a minor factor. This method is also used for treating swimming pool waters in developed countries.

(6) **Treatment with Potassium Permanganate.** Potassium permanganate is used as a popular disinfectant for disinfecting well water supplies in villages which are generally contaminated with lesser amounts of bacteria. Besides killing bacteria, it also helps in oxidising the taste producing organic matter. It is, therefore, sometimes added in small doses (such as 0.05 to 0.10 mg/l) even to filtered and chlorinated water. It has also been used as an algicide and for removing colour and iron from water.

For treating well water supplies, small amount of potassium permanganate ($KMnO_4$) is dissolved in a bucket of water and is mixed with the well water, thoroughly. The addition of potassium permanganate to water, produces pink colour. However, if the pink colour disappears, it shows that organic matter is present in water, and more quantity of potassium permanganate should be added, until the pink colour stands. The well should not be used for atleast 48 hours after the addition of potassium permanganate.

The normal doses of this disinfectant varies between 1 to 2 mg/l with a contact period of 4 to 6 hours.

Potassium permanganate, though cheap, handy and quite useful, yet cannot guarantee 100% removal of bacteria. It can remove about 98% bacteria. It can possibly remove 100% organisms causing cholera, but is of little use against other disease organisms. Moreover, the water treated with potassium permanganate, with the passage of time, produces a dark brown precipitate, which is noticeable as a coating on porcelain vessels and is difficult to remove without scouring. This method of disinfection is also, therefore, not satisfactory and not recommended except for some limited use in rural areas.

(7) **Treatment with Silver or Electro-Katadyn Process.** In this method of disinfection, metallic silver ions are introduced into the water by passing it through a tube containing solid silver electrodes which are connected to a D.C. supply of about 1.5 volts. The so introduced silver ions have a strong germicidal action, and thus act as disinfectant. The recommended silver dose may very between 0.05 to 0.1 mg/l, and the required contact period may vary between 15 minutes to 3 hours.

The silver treatment neither imparts any taste and odour to the water nor it produces any harmful effect on human body. But the use of silver is very costly, and hence not adopted for treating public supplies. Moreover, the suspended organic matter and hydrogen sulphide should be removed by pre-treatment before using this disinfectant, as the same interferes with the efficient bacteria removal. The method, however, removes algae, and its germicidal property is retained for a considerable time, thus allowing some safeguard against future contamination.

9.24. Chlorination

Chlorine in its various forms is invariably and almost universally used for disinfecting public water supplies. It is cheap, reliable, easy to handle, easily measurable, and above all, it is capable of providing residual disinfecting effects for long periods, thus affording complete protection against future recontamination of water in the distribution system. Its only disadvantage is that when used in greater amounts, it imparts bitter and bad taste to the water, which may not be liked by certain sensitive-tongued consumers.

9.24.1. Disinfecting Action of Chlorine. When chlorine is added to water, it forms hypochlorous acid or hypochlorite ions, which have an immediate and disastrous effect on most forms of microscopic organisms. The reactions that take place are :

$$Cl_2 + H_2O \xrightarrow{pH>5} \underset{\text{Hypochlorous acid}}{HOCl} + HCl$$

The hypochlorous acid is unstable and may break into hydrogen ions and hypochlorite ions

$$HOCl \underset{pH<7}{\overset{pH>8}{\rightleftharpoons}} \underset{\text{Hydrogen ions}}{H^+} + \underset{\text{Hypochlorite ions}}{OCl^-}$$

The above reaction is reversible and depends upon the pH value of water. The dissociation of hypochlorous acid into ions is more effective at high pH values and *vice versa*. Thus, at pH values greater than 10, only OCl ions are found ; while in pH values of less than 7 (more than 5), HOCl will generally exist without dissociating into OCl ions ; and in the pH range of below 5, chlorine does not react and remains as elemental chlorine. The variation of existence of chlorine species with pH and temperature is shown in Fig. 9.42.

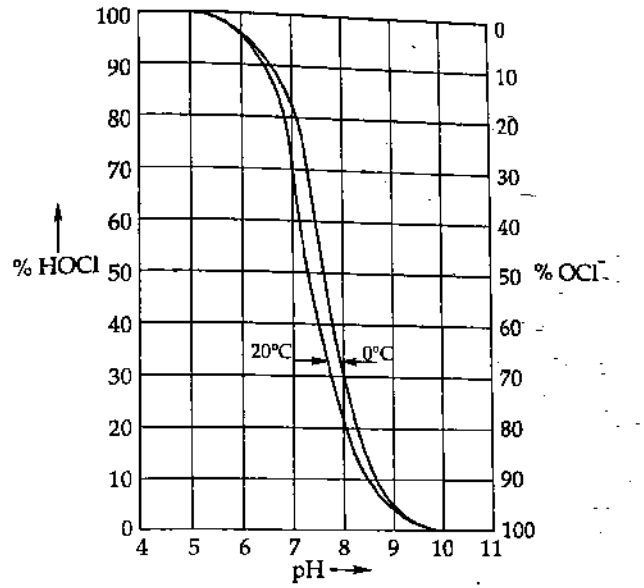


Fig. 9.42. Effect of pH on the distribution of hypochlorous acid (HOCl) and hypochlorite ions (OCl^-) in water.

Out of these forms of ***free available chlorine****, the hypochlorous acid is the most destructive, being about 80 times more effective than the hypochlorite ions. For this reason, the pH value of water during chlorination is generally maintained slightly less than 7, so as to keep the dissociation of HOCl to minimum, and thereby keeping more HOCl in solution compared to OCl ions.

Moreover, the chlorine will immediately react with ammonia present in water to form various chloramines, as given below :

$$NH_3 + HOCl \longrightarrow \underset{\text{Monochloramine}}{NH_2Cl} + H_2O$$

$$NH_2Cl + HOCl \longrightarrow \underset{\text{Di-chloramine}}{NHCl_2} + H_2O$$

$$NHCl_2 + HOCl \longrightarrow \underset{\substack{\text{Nitrogen}\\\text{Trichloramine}}}{NCl_3} + H_2O$$

The mono-chloramine (NH_2Cl) predominates at pH value of over 7.5 ; di-chloramine ($NHCl_2$) at pH range of 5 to 6.5 ; and nitrogen trichloramine (NCl_3) at pH below 4.4. Thus, in the usual chlorine treatment, when pH is kept slightly less than 7, *dichloramine formation is the most predominant.*

The chloramines so formed are stable and are found to possess disinfecting properties. They can also remove odour from water but only to a certain extent.

*The sum of hypochlorous acid, hypochlorite ions, and molecular chlorine existing in a sample of water is termed as free available chlorine.

When the added chlorine has consumed all the ammonia available in water, then it would persist as free chlorine. The combined chlorine with ammonia in the form of chloramines is called the ***combined chlorine***, and is much less effective in causing disinfection compared to the free chlorine, being about 25 times less effective.

The germicidal action of free chlorine or combined chlorine is explained by the recent theory of *Enzymatic hypothesis*, according to which, the chlorine enters the cell walls of bacteria and reacts with and kills those enzymes, which are essential for the metabolic processes of living organisms, thus bringing about the disinfection.

9.24.2. Doses of Chlorine. The amount of chlorine required for a water depends upon the inorganic and organic impurities present in it, because when chlorine is added to a water, it first of all reacts with the *inorganic impurities* (reducing agents like S^{--}, Fe^{++}, Mn^{++}, NO_2^-, etc.) that convert the chlorine into chloride, which has no residual oxidising power. Excess chlorine after this point is consumed by ammonia to form *Chloramines* (chlorine compounds representing combined chlorine). Simultaneously, chlorine will also react with *organic impurities* present in water. Some organic compounds will completely oxidise chlorine, while some chloro-organics formed will have some oxidising power. Reactions of chlorine with organics to form combined chlorine (compounds of chlorine) residuals are usually not significant as compared to the ammonia reactions. *The chlorine consumed in all the above reactions represent the chlorine demand of water.* When once it gets satisfied, the chlorine will appear as *free chlorine.*

The free chlorine as well as the combined chlorine (chloramines) will cause germicidal action on bacteria and pathogens. The free chlorine will instantaneously kill the pathogens, while the combined chlorine will provide long term germicidal effect.

In general, most of the waters are satisfactorily disinfected if the free chlorine residual is about 0.2 mg/l, 10 minutes after the chlorine is applied. A larger chlorine residual may cause bad tastes, whereas, a smaller residual may not be relied upon. The effectiveness of chlorine also depends upon the pH of the water, since chlorine is most effective if the pH of water is slightly less than 7.

The optimum dose of chlorine for a given water is, therefore, generally determined experimentally by adding varying amounts of chlorine to a given sample and observing the residual* left after a **contact period** of about 10 minutes. The dose which leaves a residual of about 0.2 mg/l is then selected. This total dose (in mg/l) minus the free residual will automatically represent the *chlorine demand of water.*

The chlorine dose must generally be increased during rainy season and epidemics. Moreover, the supply of water, to reach the first consumer should take atleast about 30 minutes, after the chlorine is mixed in water, so as to ensure sufficient disinfecting action on pathogens and viruses present in water.

*Tested by DPD and other tests, described in article 9.24.5. and in article 15.8.

Chlorine doses required for killing isolated viruses. With the isolation of various types of organisms, it has been possible to study the effects of chlorine on each kind, and to determine its amount required for killing each kind of virus individually. According to these studies, the following quantities are required for different viruses :

Table 9.6

Type of virus to be killed	*Quantity of free chlorine required in mg/l with about 30 minutes contact period for water of pH lower than 7 or so*
Poliomyelitis virus	0.1
Hepatitis virus	0.4
Cysts of E. histolytica, *i.e.* the organism causing amoebic dysentery	3.0 or even lower
Tuberculosis organisms	3.0
Coxsaickie Virus	Very huge dose varying from 21 to 138 mg/l

9.24.3. Various Forms in which Chlorine can be Applied. Chlorine is generally applied in the following forms :

as free chlorine

(1) In the form of liquid chlorine or as chlorine gas.

as combined chlorine

(2) In the form of hypochlorites or bleaching powder.

(3) In the form of chlorine tablets.

(4) In the form of chloramines, *i.e.* a mixture of ammonia and chlorine.

(5) In the form of chlorine dioxide.

The application of chlorine in these forms is discussed below :

(1) **Use of chlorine as liquid chlorine or chlorine gas.** The molecular chlorine (Cl_2) can be carried and applied to the water to be treated either in a liquid form or in a gaseous form. *The liquid form is generally preferred and almost exclusively used these days.*

Chlorine in its gaseous form is a greenish yellow gas, about $2\frac{1}{2}$ times heavier than air, and gets dissolved in water, though slightly. The maximum solubility being 1% at 10°C, which reduces to 0.61% at about 40°C. The gaseous form of chlorine gets converted into the *liquid form,* when subjected to a pressure of about 700 kN/m². The exact amount of pressure required for this conversion will of course depend on the temperature (pressure required increases with temperature). *The liquid chlorine so formed is an amber coloured oily fluid, which is* $1\frac{1}{2}$ *times heavier than water, and is generally available in steel cylinders weighing from 50 to 100 kg under a pressure of 700 to 1000 kN/m².* When required for use, the pressure is released and the liquid

chlorine gets converted into gas, which escapes out of the cylinder and gets dissolved in water to make a solution in the chlorinator, which is fed into the water to be treated. When the liquid chlorine is drawn from the cylinder as a gas, cooling occurs in the cylinder. This cooling may sometimes cause troubles, because if the temperature goes below 10°C, the liquid chlorine in the cylinder will get frozen into icy crystals, which will stick and choke the lines feeding the liquid chlorine. This trouble can be prevented by keeping the supply cylinders warm at about 38 to 42°C.

Chlorine gas neither burns nor explodes and can, therefore, be safely stored in steel cylinders either in a gaseous form or in a liquid form. Steel, however, burns in dry chlorine brightly above a temperature of 90°C. Steel cylinders storing chlorine should, therefore, be guarded against such high temperatures and kept in well ventilated rooms.

Chlorine gas causes irritation to the lungs and to the membranes of the nose and throat. So much so, that a chlorine content of about 10 ppm in air can be easily noticed and felt ; 20 ppm causes inconvenience, and 1000 ppm may cause death just within 5 minutes of exposure. Moreover, chlorine forms a very explosive mixture when mixed with carbon monoxide gas.

For these reasons, chlorine must be stored properly and carefully in independent well ventilated rooms, and should be carefully handled so as to avoid any undue leakage. Gas masks designed to protect against chlorine should also be kept available for emergencies. Provision should also be made for forcing air into the store rooms from the top, so as to drive out the heavier chlorine through the ventilators provided at the floor level.

Chlorine is applied to the water to be treated, through an instrument called **chlorinator** *or chloronome.* ***Solution feed chlorinators,*** using liquid chlorine, are almost invariably used these days. An ordinary solution-feed chlorinator takes the liquid chlorine from the container, meters it, and mixes it with a small amount of water to form a strong chlorine solution. This solution is fed into the distribution mains by means of a *water injector*. A typical solution feed chlorinator installation is shown in Fig. 9.43. All medium or large sized chlorinators are generally of this type.

Another type of a small sized chlorinator using liquid chlorine feeds the chlorine gas to the water through a diffuser by utilizing the pressure of the chlorine in the cylinder. Such chlorinators using chlorine gas directly without first mixing it with water are called *direct feed chlorinators*, and may be used on small supplies.

The operation of a chlorinator should be automatic in the sense that the amount of chlorine fed to the water is proportional to the volume of inflow and the chlorine demand of water. Moreover, the chlorine should be applied at such a point in the water system that good mixing of the chlorine with the water is ensured and atleast 20 to 30 minutes of *contact period* remains available between the point of chlorine mixing and the point where the first consumer is served. This enough contact period will not only ensure full-fledged killing of pathogens but also reduces the tastes and odours of the water which may be developed due to possible overdoses of chlorine. This contact period will thus depend upon the desired amount of *residual chlorine*

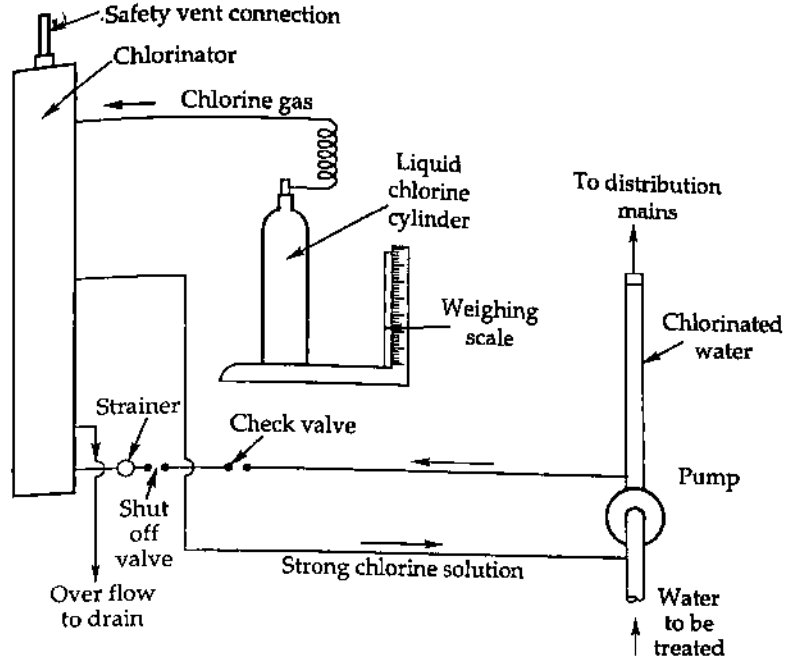


Fig. 9.43. Line diagram of a typical solution feed chlorinator installation.

(*i.e.* the free chlorine left when the water reaches the consumers). For keeping low *residuals*, a longer contact period is required.

All these provisions must be satisfied by properly maintaining and installing the chlorinator-installation.

Advantages of Using Free Molecular Chlorine as a Disinfectant

The liquid chlorine is now-a-days invariably and universally adopted for disinfecting public supplies, owing to its following advantages :

(*i*) It can be easily stored for a long period without any risk of its getting deteriorated.

(*ii*) It is quite cheap and easily available.

(*iii*) It occupies less space for storage.

(*iv*) It can be easily and cheaply transported.

(*v*) The chlorine dose to be added can be easily measured, and hence the chances of overdosing or under-dosing are minimum.

(*vi*) The initial cost of installing the chlorination plant is quite low.

(*vii*) It is a very powerful disinfectant, and may remain in water as residual for good enough time ; especially if ammonia also happens to be present in water.

(*viii*) It can be uniformly applied to the entire body of water.

(*ix*) The operation of the plant does not require *much skilled* supervision ; although the storage of chlorine and its use may need *careful* handling.

(*x*) No sludge is formed in its application, as may be formed by using hypochlorites or chloramines.

(2) **Use of Hypochlorites and Bleaching Powder.** Calcium and sodium hypochlorites may be used for chlorinating small public supplies. When such a hypochlorite is dissolved in water, it dissociates, as to form hypochlorite ions, as given below :

$$\underset{\substack{40+(16+35)2=142\\ \text{Calcium hypochlorite}}}{Ca(OCl)_2} \xrightleftharpoons{H_2O} \underset{\substack{\text{Calcium}\\ \text{ions}}}{Ca^{++}} + \underset{\substack{2\times(16+35)=102\\ \text{Hypochlorite ions}}}{2OCl^-}$$

The hypochlorite ions may further combine with the hydrogen ions present in water, so as to form hypochlorous acid.

$$OCl^- + H^+ \underset{pH>8}{\overset{pH<7}{\rightleftharpoons}} \underset{\text{Hypochlorous acid}}{HOCl}$$

This process is known as *hypochlorination.* The hypochlorite ions as well as hypochlorous acid both causing disinfection of water.

A 100 per cent pure hypochlorite should contain free available chlorine equal to the OCl^- value of the compound. For example, calcium hypochlorite should contain free available chlorine as equal to twice the OCl^- value ; or 142 parts of calcium hypochlorite will contain 102 parts of free chlorine, *i.e.* about 70%.

Commercially available calcium hypochlorite, called **High Test Hypochlorite (HTH)** or ***Perchloron***, generally contains 60% to 70% of available chlorine. Hence, to treat water with an approximate dose of 0.7 mg/L of chlorine, we shall require about 1 mg of HTH per litre of water, or about 1 kg of HTH for 1 million litre of water.

Another chlorine compound, which is sometimes used for disinfecting water is **bleaching powder** or *chlorinated lime* or *calcium oxychlorite*, having molecular formula as $CaOCl_2$. This compound is a white amorphous powder with a pungent smell of chlorine. When freshly made, it contains about 30% of available chlorine. It is, however, an unstable compound, and on exposure to air, light and moisture, it rapidly looses its chlorine content.

Its strength should, therefore, first be ascertained in the laboratory before using it for chlorination. All this makes it a tedious, laborious, inefficient and expensive disinfectant and, therefore, not used these days. The *high test hypochlorites* may, however, sometimes be preferred and used owing to their following *advantages* :

(*i*) They are available in small packets in powdered form.

(*ii*) Their chlorine content does not decrease with storage.

(*iii*) They can be applied to water in dry condition or as solution.

Nevertheless, hypochlorites and bleaching powder are generally *not used* in modern days for treating public water supplies, mainly because (*i*) they raise the pH value of water on account of their lime contents, (*ii*), they contain very low amounts of chlorine. They may, however, be adopted for treating small supplies for swimming pools, etc. They may also be used for disinfecting water mains, filter sands, storage reservoirs, wells and tubewells, etc., and also as emergency disinfectants.

The hypochlorites like bleaching powder can be added to the water for disinfection by using ***Gravity feed chlorinators*** (dry and wet types) as are used for feeding coagulants, and as shown in Figs. 9.20 and 9.21. They may also be injected in water pipe in solution form by means of non-corrosive ***dosing pump***, which is inter- locked with the main water pump, so that it injects chlorine when water is flowing in the pipe line.

Displacement Dosers. They are pressure vessels in which a rubber bag containing the dosing solution (bleaching powder solution) is placed. An orifice flange installed on the water pipe line near the doser creates a difference of pressure. A connection from the upstream side of the orifice having higher pressure is made to the pressure vessel, while the connection from the downstream side of the orifice having lower pressure is made to the outlet of the rubber bag having dosing solution, as shown in Fig. 9.44. The difference in pressure squeezes the rubber bag injecting the dosing solution into the water line. This injection of solution occurs only during the flow of water in the pipe line. By adjusting the concentration of the dosing solution in the bag, the dosing is made proportional to the water used.

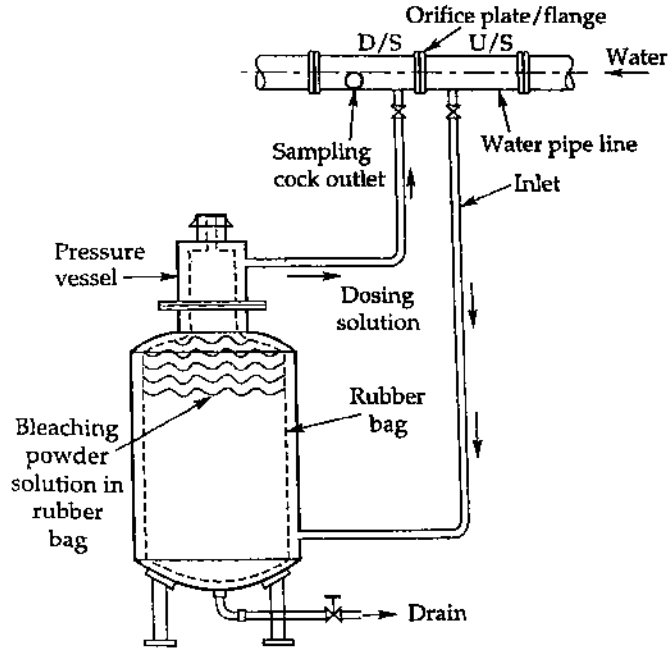


Fig. 9.44. Displacement doser.

Example 9.27. *It is required to supply water to a population of 20,000 at a per capita demand of 150 litres per day. The disinfectant used for chlorination is bleaching powder which contains 30 per cent of available chlorine. Determine how much of bleaching powder is required annually at the waterworks, if 0.3 ppm of chlorine dose is required for disinfection.*

Solution.

Average daily water demand

= Population × Per capita demand

= 20,000 × 150 litres

= 30,00,000 litres = 3×10^6 litres.

∴ Amount of chlorine required *daily* (based on average annual consumption)

$$= 0.3 \frac{\text{mg}}{l} \times 3 \times 10^6 \, l$$

$$= 0.9 \times 10^6 \text{ mg} = 0.9 \text{ kg}.$$

Since the chlorine content in bleaching powder is 30%, it means that 30 kg of chlorine is contained in 100 kg of bleaching powder.

∴ Amount of bleaching powder required daily (based on average annual consumption)

$$= \frac{0.9 \times 100}{30} = 3 \text{ kg}.$$

∴ Annual consumption of bleaching powder

= 3 × 365 kg

= 1095 kg = 1.095 tonnes. **Ans.**

Example 9.28. *Chlorine usage in the treatment of 20,000 cubic metre per day is 8 kg/day. The residual after 10 min. contact is 0.20 mg/l. Calculate the dosage in milligrams per litre and chlorine demand of the water.*

(Engineering Services, 1980)

Solution.

Water treated per day = 20,000 cum.

= $20,000 \times 10^3$ litres

= 20×10^6 litres = 20 M.*l*.

Chlorine consumed per day = 8 kg = 8 M.mg

∴ Chlorine used per litre of water

$$= \frac{8 \text{ M . mg}}{20 \text{ M . } l} = 0.4 \text{ mg}/l$$

∴ Hence, the given chlorine dosage = 0.4 mg/*l*.

Now, residual chlorine left = 0.2 mg/*l* (given)

Hence, actual chlorine dosage, which has reacted in water, *i.e. chlorine demand of water*

= 0.4 − 0.2 = 0.2 mg/*l*. **Ans.**

(3) Use of Chlorine Tablets. Chlorine tablets may also some times be used to disinfect small quantities of water, but they are costly. They are available in the market under various trade names, such as *halazone tablets*. The National Environmental Engineering Research Institute, Nagpur has formulated a new type of chlorine tablet, which is 15 times better than the ordinary *halazone tablets*. These tablets are manufactured in various strengths and are now available in plenty in the Indian market at a cheap rate. A single tablet of 0.5 g is sufficient to disinfect about 20 litres of water. These tablets can be used for treating overhead water storage tanks, or for treating ground water supplies, and even for retreating the public supplies at consumer's end during water borne epidemics, or during the floods when public supplies are liable to be contaminated.

(4) Use of Chloramines or Use of Chlorine with Ammonia. Chloramines are the disinfectant compounds, which are formed by the reactions between ammonia and chlorine, as given earlier. These compounds are quite stable and can remain in water as residuals for a sufficient time, contrary to the unstable chlorine which evaporates after some time. Hence, they can provide a greater safeguard against future pollution, although they are comparatively much weaker disinfectants compared to free chlorine. The other advantages of using chloramines as disinfectants are :

(*i*) they do not cause bad tastes and odours when left as residuals, as is caused by free chlorine.

(*ii*) they are very useful when phenols are present in water ; because the reactions of phenols with chlorine alone will result in the creation of disagreeable tastes, but no such tastes develop with chloramines.

Hence, chloramines may be used as disinfectants. For producing them, ammonia is added to the filtered water before adding chlorine. Doses of ammonia and chlorine used will depend upon the local characteristics of the water and the water system. But under normal conditions, the amount of ammonia used generally varies from $\frac{1}{3}$ to $\frac{1}{4}$th of the amount of chlorine. However, when phenol tastes are to be controlled, the proportion of ammonia may be higher.

The ammonia should be properly applied and mixed with water, about 20 minutes to 1-2 hours earlier than applying chlorine. This contact period of ammonia being kept higher when phenols are present in water. The ammonia is applied by a separate machine (just like that of a chlorinator) called ***ammoniator***. Sometimes, mechanical mixing is adopted, because ammonia, although highly soluble in water, does not disperse easily in it. Separate machines for feeding ammonia and chlorine are a must, because the two chemicals react differently with materials in the feeders.

Since chloramines (*i.e.* the mixture of ammonia and chlorine) are weaker disinfectants (only about $\frac{1}{25}$ times stronger than chlorine) ; either higher dose or a longer contact period of about 1 to 2 hours must be used in order to achieve the same results, as could be obtained with chlorine.

(5) Use of Chlorine-Dioxide Gas. Chlorine dioxide gas (ClO_2) is a very effective and powerful disinfectant, about 2 $\frac{1}{2}$ times stronger than chlorine. It is produced by passing chlorine gas through sodium chlorite, as follows :

$$\underset{\text{Sodium chlorite}}{2NaClO_2} + Cl_2 \longrightarrow 2NaCl + \underset{\text{Chlorine dioxide gas}}{2ClO_2} \uparrow$$

This chlorine dioxide gas is costly, and very unstable and has to be used immediately after its production. For these reasons, it is generally not used for

treating ordinary public supplies. However, because of its stronger disinfecting powers, it may be used for treating waters containing larger amounts of organic impurities. It may also be used for treating waters containing phenolic compounds which may develop unwanted tastes and odours with the use of chlorine alone. Since the action of chlorine dioxide remains unaffected by the high pH values (6 to 10), it can be advantageously used for treating highly alkaline waters with pH between 8 to 10. The normal dose of chlorine dioxide usually ranges between 0.5 to 1.5 mg/l.

9.24.4. Types of Chlorination and Certain Important Definitions. Depending upon the quantity of chlorine added, or the stage at which it is added, or upon the results of chlorination, various technical terms in relation to the chlorination are used. They are :

(*i*) Plain chlorination
(*ii*) Pre-chlorination
(*iii*) Post-chlorination
(*iv*) Double chlorination
(*v*) Break point chlorination
(*vi*) Super chlorination
(*vii*) Dechlorination

These terms are discussed and defined below :

(*i*) **Plain chlorination.** This term is used to indicate that *only the chlorine treatment and no other treatment* has been given to the raw water. Under plain chlorination, therefore, raw water is fed into the distribution system after giving chlorine treatment only. This helps in removing bacteria, organic matter, and colour from the raw water. This technique may be used for treating relatively clearer waters (with turbidities less than 20 to 30 mg/l) obtained from lakes, reservoirs, deep tube wells, etc. It may also be used during emergencies, when full-fledged treatment cannot be given, such as for supplying water to army troops during war times. The used quantity of chlorine required for plain chlorination is about 0.5 mg/l or more.

(*ii*) **Pre-chlorination.** Pre-chlorination is the process of *applying chlorine to the water before filtration or rather before sedimentation-coagulation.* It helps in improving coagulation, and reduces the loads on the filters. It also reduces the taste, odour, algae and other organisms. The chlorine dose should be such that about 0.1 to 0.5 mg/l of residual chlorine comes to the filter plant. The normal doses required are as high as 5 to 10 mg/l. Pre-chlorination is, however, always followed by post chlorination, so as to ensure the final safety of water.

(*iii*) **Post chlorination.** Post chlorination or sometimes simply called ***chlorination*** is the normal standard process of *applying chlorine in the end,* when all other treatments have been completed. While treating normal public supplies, the post chlorination is adopted after filtration and before the water enters the distribution system. The dosage of chlorine should be such as to leave a residual chlorine of about 0.1 to 0.2 mg/l, after a contact period of about 20 minutes. This residual will ensure the disinfection of water, if at all any future recontamination occurs in the distribution system.

(*iv*) **Double chlorination.** The term double chlorination is used to indicate that the water has been *chlorinated twice.* The pre-chlorination and post chlorination are generally used in double chlorination. Post chlorination, however, is generally always used, while the pre-chlorination is also used when the waters are highly turbid and contaminated. Since the double chlorination uses pre-chlorination prior to the normal post chlorination, its advantages are the same as those of pre-chlorination. In addition, the second unit of chlorination plant required for per-chlorination may serve as a stand-by unit for emergencies.

(*v*) **Break-point chlorination.** Break point chlorination is a term which gives us an idea of the extent of chlorine added to the water. *In fact, it represents, that much dose of chlorination, beyond which any further addition of chlorine will equally appear as free residual chlorine.*

When chlorine is added to the water, it first of all, generally reacts with the ammonia present in the water, so as to form chloramines. These chloramines respond to the D.P.D. test* in the same manner as does free chlorine. *Therefore, the D.P.D. test will indicate the quantum of total residual chlorine "combined" as well as "free".* Hence, if chlorine is slowly added to the water and the residual is tested, it will be found that the residual will go on increasing with the addition of chlorine. However, some chlorine is consumed for killing bacteria, and thus the amount of residual chlorine shall be slightly less than that added, as shown by the curve *AB* in Fig. 9.45. If the addition of

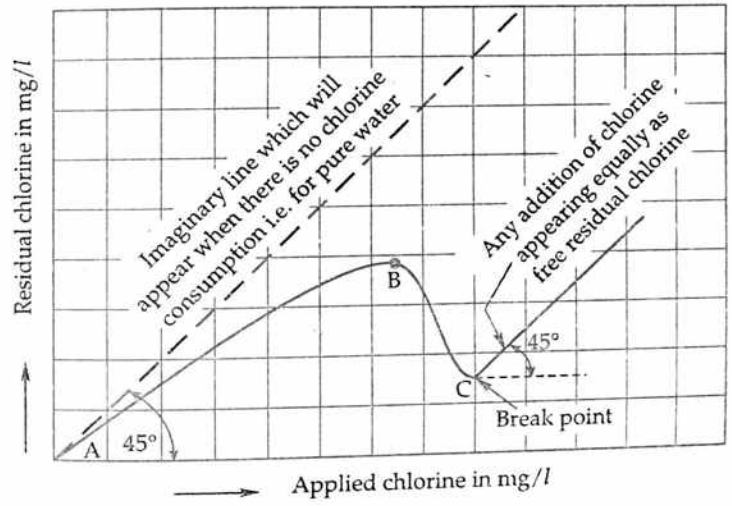


Fig. 9.45. Figure indicating break-point chlorination

chlorine is continued beyond the point *B*, the organic matter present in water starts getting oxidised, and, therefore, the residual chlorine content suddenly falls down, as shown by the curve *BC*. The point *C* is the point beyond which any further addition of chlorine will appear equally as free chlorine, since nothing of it shall be utilised. This point "*C*" is called the ***break point***, as any

*Test conducted to measure residual chlorine ; it is explained in article 9.24.5. and article 15.8.

chlorine that is added to water beyond this point, breaks through the water, and totally appears as residual chlorine. The addition of chlorine beyond break point is called ***break point chlorination***.

At the point *B*, when oxidation of organic matter starts, a bad smell and taste generally appears ; which disappears at the break point *C*, when the oxidation has been completed.

It is a general practice to add chlorine beyond break point, and thus to ensure a residual of 0.2 to 0.3 mg/l of free chlorine. This residual of free chlorine, appearing after the break point, is not easily removed except by sun light, and, therefore, it takes care of the future recontamination of water in the distribution system.

(*vi*) **Super chlorination.** Supper chlorination is a term which indicates the addition of excessive amount of chlorine (*i.e.* 5 to 15 mg/l) to the water. This may be required in some special cases of highly polluted waters, or during epidemics of water borne diseases. It may be used when there is a reason to believe that the water contains cysts of *histolytica* (*i.e.* the organism which causes amoebic dysentery).

The huge quantity of chlorine which is added in superchlorination is such as to give about 1 to 2 mg/l of residue beyond the break point, in the treated water. Sometimes, even higher doses may be used and the resultant water is dechlorinated* after the end of the desired contact period, by using *dechlorinating agents* such as sodium thiosulphate, activated carbon, sulphur dioxide, etc. This ensures the removal of bad tastes and odours caused by the presence of excess chlorine, but may render the water free from chlorine and susceptible to recontamination. Hence, in such cases, it is desirable to again chlorinate the water by a dose of about 0.1 to 0.2 mg/l.

(*vii*) **Dechlorination.** As indicated above, the dechlorination means *removing the chlorine from water.* This is generally required when super-chlorination has been practised. The dechlorination process may either be carried out to such an extent that sufficient residual chlorine (0.1 to 0.2 mg/l) do remains in water after dechlorination ; or otherwise, if full chlorine has been removed, additional chlorine will generally be added to maintain such residues.

The dechlorination may be carried out be adding certain chemicals to water or by simply aerating the water. These chemicals are called *dechlorinating agents*.

The common **dechlorinating agents** are : *Sulphur dioxide gas* ($SO_2\uparrow$), *Activated carbon, Sodium thiosulphate* ($Na_2S_2O_3$), *Sodium metabisulphate* ($Na_2S_2O_5$), *Sodium sulphite* (Na_2SO_3), *Sodium bisulphite* ($NaHSO_3$), *and Ammonia* as NH_4OH.

SO_2 gas may be applied to water somewhat in the same manner as chlorine, with a minimum contact period of about 10 to 15 minutes, with a dose of about 0.3 to 0.6 ppm, depending upon the excess chlorine present in water. The required SO_2 to excess chlorine ratio is 1.12 : 1 with an additional 25% of SO_2 over this theoretical requirement.

*Explained more in the next para.

Sulphites are granular solids, which may be added to water by dry or solution feed methods. Although sodium sulphite is cheaper in per kg cost, yet sodium thiosulphate proves overall cheaper, since lesser quantity of sodium thiosulphate is required to obtain the same results.

Ammonia may also be useful and economical dechlorinator, because of its reaction with chlorine to form chloramines.

9.24.5. Testing of Chlorine Residuals. The amount of residual chlorine left in the chlorinated water after the required contact period, can be experimentally determined by using any of the following tests :

(1) Orthotolidine test ;

(2) D.P.D. test ;

(3) Chlorotex test ; and

(4) Starch-iodide test.

These tests are briefly discussed below :

(1) **Orthotolidine test.** In this test, 10 ml of chlorinated sample of water is taken after the required contact period, in a glass tube. To this is added 0.1 ml of orthotolidine solution. The colour formed is noted after 5 minutes. The formation of a yellow colour normally indicates the presence of chlorine (either combined or free) in the water. The more yellow the colour, the greater is the chlorine residual. Hence, the amount of residual chlorine can be ascertained by comparing the above noted colour in the glass tube with the standard colours already kept in the laboratory. This test is, therefore, very simple and does not require much technique or time. Under normal conditions, a lemon yellow colour is satisfactory for public supplies.

The orthotolidine test will normally give the total residual chlorine present in water. However, it may be adjusted so as to give separately the quantities of 'free' as well as 'combined' chlorine. Truly speaking, the free residual chlorine forms the yellow colour during the first 5 seconds of the addition of orthotolidine, while the combined residual chlorine goes on forming the colour for about 5 minutes. Hence, the colour after 5 seconds will give us the quantity of free chlorine, and the colour after 5 minutes will give us the summation of free as well as combined chlorine. The quantity of free chlorine when subtracted from the total quantity will easily give us the quantity of the combined chlorine.

The orthotolidine test, however, is not fool-proof, because the impurities such as iron, manganese, nitrites, etc. are likely to cause a false yellow colour, and thereby indicating wrong and increased chlorine residuals. For such waters, orthotolidine-arsenite test is used in place of simple orthotolidine test.

Orthotolidine—Arsenite (OTA) test. In this test, sodium arsenite is first of all added to the chlorinated sample of water (after the required contact period). This will dechlorine the sample ; and orthotolidine is then added, as above. The colour formed is noted. This will indicate the intensity of colour caused not by chlorine but by interfering agents like nitrites, iron, manganese, etc. Let this intensity of colour corresponds to a chlorine concentration of R_1.

Now, another sample of water is taken in another test tube and orthotolidine solution is added first, and just after 5 seconds, sodium arsenite is added. The sodium arsenite will arrest the colour to be formed by the 'combined' chlorine. Hence, the noted colour at the end of the experiment will be due to the free chlorine plus due to the interfering colour causing compounds of iron, manganese, nitrites, etc. Let this intensity of colour corresponds to a chlorine concentration of R_2.

Then, a third test is conducted on a third sample of water. In this, only the orthotolidine solution is added to the given sample of water, and the colour is noted after 5 minutes. The noted colour will evidently be due to the free and combined chlorine plus due to the interfering colour causing compounds. Let this intensity of colour corresponds to a chlorine concentration of R_3.

Now, the different chlorine residuals can be easily worked out as follows :

The quantity of 'free' residual chlorine $= R_2 - R_1$

The quantity of 'combined residual chlorine $= R_3 - R_2$

In the orthotolidine test as well as in the orthotolidine-arsenite test, it is also required to control the temperature of water up to room temperature. Hence, in certain special cases, when the water sample is very cold, it may have to be slightly warmed, so as to bring it to room temperature. If this precaution is not observed, the colour formation will be slow and uncertain.

When chloramines are used as disinfectants, the ordinary orthotolidine test can be used for determining the total chlorine residuals, with the only difference that the colour of water should be noted after 15 minutes instead of 5 minutes. This is because, chloramines takes a little more time in developing full yellow colour with orthotolidine as compared to chlorine.

When the water to be tested is highly alkaline, a blue tinge is produced in the orthotolidine test. This may interfere with the measurement of yellow colour. To remove this tinge, the quantity of orthotolidine should be doubled. Hence, in such cases, the ordinary orthotolidine test, but with 2 ml of orthotolidine in 100 ml of water sample, will be able to give satisfactory results.

The orthotolidine methods described above have, however, become obsolete these days, because the chemical is producing carcinogenic compounds (compounds producing cancer) and the chemical itself has carcinogenic effect. *So this chemical (reagent) is now-a-days not used for chlorine detections, and has almost been replaced by the DPD reagent test.*

(2) **DPD test.** This test is widely used in modern days, and makes use of Palin's DPD* reagent, which is available as DPD chlorine tablets. The testing is generally carried out in a **BDH**Comparator**, so as to easily compare the developed *red colour* with the standard given colours. The comparator is provided with a special dulling screen provided with a disc which must be fitted to the right hand viewing window of the comparator. The BDH comparator discs will be available in two alternatives, for two different chlorine-concentration-ranges.

*i.e. (NN'-Diethly-p-Phenylene-Diamine]

**British Drug House.

(*a*) 3/40/A 0.1 to 10 mg/l of chlorine

(*b*) 3/40/B 0.2 to 4.0 mg/l of chlorine.

In order to carry out this test, 10 ml of water sample is taken in a tube, and it is placed in the left compartment of the comparator. A reagent tablet (DPD chlorine tablet) is placed in another tube, to which is added about 1 cm depth of water, and the tablet is allowed to dissolve. More water is now added to this, until the tube contains 10 ml, and it is placed in the right hand compartment. After 2 minutes, the colours are viewed and compared through the viewing windows and the disc is rotated till the colours match. The residual chlorine amount can then be directly read in the window in the lower right hand corner of the instrument.

A detailed description of Laboratory test for DPD testing is given in article 15.8.3.

Similar comparator instrument, called **Chloroscope**, is generally used even in the orthotolidine testing, so as to facilitate easy comparison of developed colour with the standard colours, and to know the actual chlorine concentration.

(3) **Chlorotex test.** This is a very simple test and uses BDH Chlorotex reagent, which is provided with standard colour matching card. No comparator is used in this method.

In this test, 50 ml sample of water is taken in a clean glass vessel. To this is added, 5 ml of chlorotex reagent. The immediate development of a colour will indicate the presence of chlorine. The developed colours and their representating chlorine concentrations are indicated in Table 9.7.

Table 9.7

Colour developed with chlorotex reagent	*Chlorine concentration indicated, in mg/l*
White milky fluorescence	Nil
Faintly pink and milky	0.1
Pink	0.2
Red	0.5
Purple	0.6
Violet	0.8
Blue	1.0 or more

(4) **Starch Iodide test.** In this test, one litre of water sample is collected in a heat-proof earthen ware vessel (called *casserole*), to which 10 ml of potassium iodide solution is added. The contents are thoroughly mixed, and 5 ml of starch solution is added, which produces blue colour. This blue colour is removed by titrating this water sample against sodium thiosulphate solution of normality N/100. The amount of chlorine can then be easily ascertained by using the simplified titration equation :

$$\begin{bmatrix}\text{Quantity of chlorine}\\ \text{in mg/l in the original}\\ \text{sample of water}\end{bmatrix} = 0.355 \begin{bmatrix}\text{Number of ml of}\\ \text{thiosulphate reqd.}\\ \text{to remove the}\\ \text{blue colour}\end{bmatrix}$$

A detailed description of this laboratory testing is given in article 15.8.2.

Starch iodide test is laborious and costly, and hence generally not used for testing public supplies.

Example 9.29. *Results of chlorine demand test on a raw water are given below :*

Sample No.	*Chlorine dosage mg/l*	*Residual chlorine after 10 min contact (mg/l)*
1	*0.2*	*0.19*
2	*0.4*	*0.36*
3	*0.6*	*0.50*
4	*0.8*	*0.48*
5	*1.0*	*0.2*
6	*1.2*	*0.4*
7	*1.4*	*0.6*
8	*1.6*	*0.8*

Sketch a 'chlorine demand curve'. What is the 'break point dosage' and what is the 'chlorine demand' at dosage of 1.2 mg/l ? *(Civil Services, 1980)*

Solution. The given data is used to plot the chlorine demand curve (strictly speaking chlorine residual curve) as shown in Fig. 9.46.

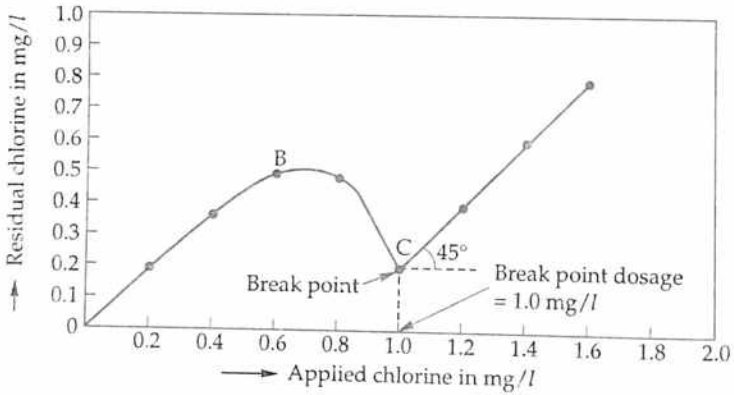


Fig. 9.46

Break point is represented by the point C, and hence the *break point dosage* is 1.0 mg/l. **Ans.**

Chlorine demand is the difference between applied chlorine and residual chlorine, and it becomes equal to 1.0 – 0.2 = 0.8 mg/l at break point. This chlorine demand becomes constant thereafter, and all the added chlorine appears as free chlorine.

Thus, at any dosage above 1.0 mg/l, the chlorine demand will remain study, and equal to 0.8 mg/l.

Hence, the chlorine demand at a dosage of 1.2 mg/l will be equal to 0.8 mg/l. This tallies with the given data of chlorine residual of 0.4 mg/l with a dose of 1.2 mg/l, giving chlorine demand = 1.2 – 0.4 = 0.8 mg/l. **Ans.**

WATER SOFTENING

The reduction or removal of hardness* from water is known as ***water softening***. It is not always essential to soften raw public supplies to make the water safe for public uses, since the normally prevalent hardness in raw waters are not too high to cause any major harm to human health. The advantage of softening, however, lies chiefly in the reduction of soap consumption, lowered cost in maintaining plumbing fixtures, and improved taste of food preparations. Hence whether or not the hardness of a water supply should be reduced, depends on the relation between the cost of treatment and the obtained resultant saving and satisfaction to the consumers. However, for industrial supplies, the softening is more important, because the hard waters are likely to cause scaling troubles in boilers and interfere in the working of dyeing systems.

It was explained earlier in chapter 8, that the **temporary** or **carbonate hardness**, *which is caused by the carbonates and bicarbonates of calcium and magnesium ; can be removed by boiling, or by adding lime* ; whereas, the **permanent** or **non-carbonate hardness** *which is caused by the sulphates, chlorides and nitrates of calcium and magnesium***, *has to be removed by special methods of water softening.*

It may also be recalled here that the permissible hardness for public supplies normally ranges between 75 to 115 mg/l (where 14.25 mg/l is equivalent to one degree of hardness).

9.25. Methods of Removing Temporary Hardness

The temporary hardness can be removed either by boiling or by adding lime to the water, as explained below :

(1) **Boiling.** Calcium carbonate, being only slightly soluble in water, will, however, usually exist in water as calcium bicarbonate, because it easily dissolves in natural water containing carbon dioxide. When such a water is boiled, the carbon dioxide gas will get out, leading to the precipitation of $CaCO_3$, which can be sedimented out in settling tank. The reaction can be represented as :

$$\underset{\text{Cal. bicarbonate}}{Ca(HCO_3)_2} + \text{Heat} \longrightarrow \underset{\substack{\text{Cal. carbonate}\\ \text{(insoluble)}}}{CaCO_3\downarrow} + CO_2\uparrow + H_2O$$

The magnesium bicarbonate and magnesium carbonate, however, cannot be satisfactorily removed under the above type of reaction, since $MgCO_3$ is

*Please refer article 8.2.2 (*iii*).

**In addition to Ca^{++} and Mg++, other divalent metallic ions like Fe^{++}, Mn^{++}, Sr^{++} etc. ; will also cause hardness, but their presence in most of the waters is generally negligible, and hence hardness for all practical purposes is attributed to Ca^{++} and Mg^{++} only.

fairly soluble in water. Boiling cannot therefore satisfactorily remove the temporary hardness caused by magnesium. Moreover, large scale boiling of public supplies is practically unfeasible and hence boiling is not adopted for softening. On the other hand, magnesium carbonate and magnesium bicarbonate are removed by precipitating them as insoluble $Mg(OH)_2$, by treating the hard water with lime, which also helps in the removal of calcium bicarbonate as calcium carbonate by supplying OH ions, as discussed below.

(2) **Addition of lime.** Lime (CaO), generally hydrated lime $[Ca(OH)_2]$ is added to the water. The following reactions take place :

(*ia*) $\underset{\text{Mag. carbonate (soluble)}}{MgCO_3} + \underset{\text{Hydrated lime}}{Ca(OH)_2} \longrightarrow \underset{\text{Mag. hydroxide (insoluble)}}{Mg(OH)_2\downarrow} + \underset{\text{Cal. carbonate (insoluble)}}{CaCO_3\downarrow}$

$\underset{\text{Mag. bicarbonate}}{Mg(HCO_3)_2} + \underset{\text{Hydrated lime}}{Ca(OH)_2} \longrightarrow Ca(HCO_3)_2 + \underset{\text{Mag. hydroxide}}{Mg(OH)_2\downarrow}$

$\underset{\text{Cal. bicarbonate}}{Ca(HCO_3)_2} + Ca(OH)_2 \longrightarrow 2CaCO_3\downarrow + 2H_2O$

The calcium carbonate and magnesium hydroxide are precipitated, and can be removed in the sedimentation tank.

This method is generally adopted for softening waters which contain only temporary hardness. However, the waters which contain temporary as well as permanent hardness are softened by special methods of water softening.

9.26. Methods of Removing Permanent Hardness

The permanent hardness is more permanent and difficult to remove. It can, however, be removed by certain special methods, generally called *water softening methods*. The three methods, which are commonly adopted for softening waters containing either permanent hardness, or permanent as well as temporary hardness both, are

(1) *Lime-Soda process ;*

(2) *Base-Exchange process*, generally called *Zeolite process* ; and

(3) *Demineralisation process.*

These methods are described below :

9.26.1. Lime-Soda Process for Removing Hardness. In this process, lime $[Ca(OH)_2]$ and soda ash $[Na_2CO_3]$ are added to the hard water ; which react with the calcium and magnesium salts, so as to form insoluble precipitates of calcium carbonate and magnesium hydroxide $[Mg(OH)_2]$. These precipitates can be sedimented out in a sedimentation tank. The chemical reactions which may be involved are :

(*i*) $\underset{\text{Carbonate hardness of Ca}}{Ca(HCO_3)_2} + \underset{\text{Lime}}{Ca(OH)_2} \longrightarrow 2CaCO_3\downarrow + 2H_2O$

(*iia*) $\underset{\text{Carbonate hardness of Mg}}{Mg(HCO_3)_2} + \underset{\text{Lime}}{Ca(OH)_2} \longrightarrow \underset{\text{Carbonate hardness of Ca}}{Ca(HCO_3)_2} + Mg(OH)_2\downarrow$

(*iib*) $\underset{\text{Carbonate hardness of Mg}}{MgCO_3} + \underset{\text{Lime}}{Ca(OH)_2} \longrightarrow Mg(OH)_2\downarrow + CaCO_3\downarrow$

(*iii*) $\underset{\text{Non-carbonate hardness of Mg}}{MgCl_2} + \underset{\text{Lime}}{Ca(OH)_2} \longrightarrow Mg(OH)_2\downarrow + \underset{\text{Non-carbonate hardness of Ca}}{CaCl_2}$

(*iv*) $\underset{\text{Non-carbonate hardness of Mg}}{MgSO_4} + \underset{\text{Lime}}{Ca(OH)_2} \longrightarrow Mg(OH)_2\downarrow + \underset{\text{Non-carbonate hardness of Ca}}{CaSO_4}$

(*v*) $\underset{\text{(Free Dissolved } CO_2)}{CO_2} + \underset{\text{Lime}}{Ca(OH)_2} \longrightarrow CaCO_3\downarrow + H_2O$

(*vi*) $\underset{\text{Non-carbonate hardness of Ca}}{CaCl_2} + \underset{\text{Soda}}{Na_2CO_3} \longrightarrow CaCO_3\downarrow + 2NaCl$

(*vii*) $\underset{\text{Non-carbonate hardness of Ca}}{CaSO_4} + \underset{\text{Soda}}{Na_2CO_3} \longrightarrow CaCO_3\downarrow + Na_2SO_4$

From the above reactions, it can be easily seen that lime *helps in removing the entire carbonate hardness* (*i.e.* carbonate hardness caused by Ca as well as that caused by Mg), and *it reacts with non-carbonate hardness of Mg* to convert the same into non-carbonate hardness of Ca. The non-carbonate hardness of Ca is finally removed by soda. Lime also helps in removing the free dissolved carbon dioxide, if and when it exists in the given water, as per reaction No. (*v*) given, above.

The sodium salts which are finally formed in the above method, are soluble in water, but are generally not objectionable in the amounts resulting from the softening process. Most of the calcium carbonate and magnesium hydroxide which is formed, get precipitated and can be sedimented out in the sedimentation tank. However, a little quantity may remain as finely divided particles, and may cause troubles by getting deposited on the filter to cause enlargement of the sand grains called **incrustation of filter media** ; or in the pipes of the distribution system. To prevent this, it is generally necessary that the water be ***recarbonated*** by passing carbon dioxide gas through it, as it leaves the sedimentation tank. In the **recarbonation process,** the insoluble carbonates combine with the carbon dioxide to again form the soluble bicarbonates, as given below :

(*i*) $\underset{\text{Insoluble Cal. carbonate}}{CaCO_3\downarrow} + CO_2 + H_2O \longrightarrow \underset{\text{Soluble Cal. bicarbonate, causing carbonate hardness of cal.}}{Ca(HCO_3)_2}$

(*ii*) $\begin{bmatrix} Mg(OH)_2 + CO_2 & \longrightarrow & MgCO_3 + H_2O \\ MgCO_3 + CO_2 + H_2O & \longrightarrow & Mg(HCO_3)_2 \end{bmatrix}$ Carbonate hardness of Mg

The carbon dioxide gas to be blown in water can be produced by burning coke (Fig. 9.47), gas or oil, or delivered to the plant as a pressurised liquid.

By the recarbonation process, even though the water regains some of its carbonate (temporary) hardness, yet recarbonation is advisable.

The amount of lime and soda required for the softening, depends upon the chemical quality of the water and the extent of hardness removal desired (Refer examples 9.30 to 9.35). Many of the hard waters may primarily contain the carbonate hardness and very low amounts of non-carbonate hardness. Hence, for treating all such waters, lime is often the only chemical required.

The *equipment* required for the lime soda treatment is similar to that required for chemical coagulation. The lime and soda ash are added to the raw water, either separately or together, and are well mixed with water. Flocculation and sedimentation will then follow. The feeding, mixing, and settling devices used, are similar to those which are used in the process of coagulation-cum-sedimentation. However, for obtaining good results, the detention time for settling basin may be kept slightly more than that required in an ordinary coagulation-sedimentation basin, and may vary between 2 to 4 hours.

The water coming out of the sedimentation tank after lime-soda treatment is recarbonated by passing carbon dioxide gas, through it, to remove insoluble carbonates and magnesium hydroxide precipitates which do not settle out in sedimentation. Fig. 9.47 shows a typical installation used for producing and feeding carbon dioxide gas.

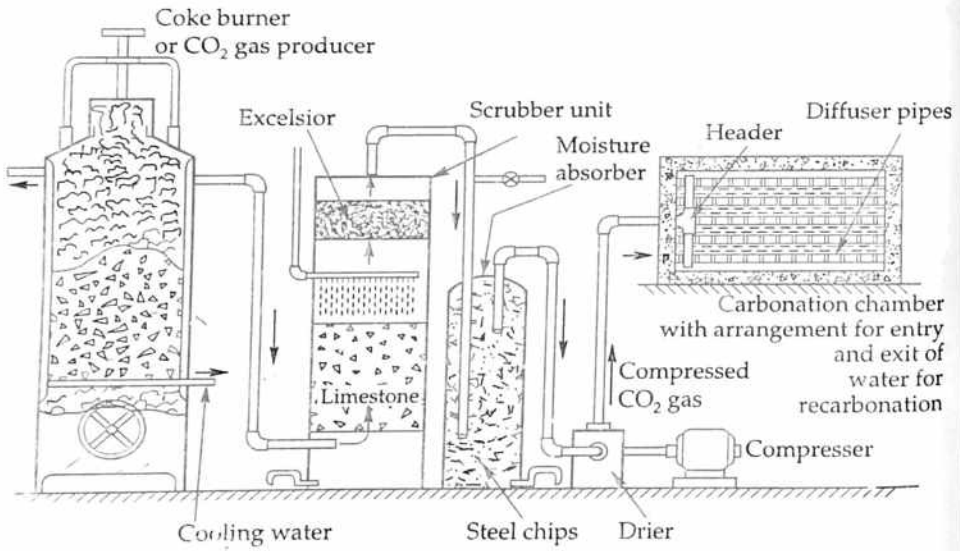


Fig. 9.47. Recarbonation Plant.

Carbon dioxide gas can be produced by burning coke or it can be purchased in cylinders. Essential equipment for recarbonation includes a *gas producer*, a *scrubber*, a *moisture trap*, a *driver*, a *compressor*, and a *diffuser*. The compressed carbon dioxide gas is applied under a low pressure to the water by means of a grid of pipes perforated with holes 2-3 mm in dia and about 15 cm apart, or by some other type of diffusers. A short contact period between the

carbon dioxide and water is sufficient. A tank having a depth of about 3 m and a displacement period of 15-30 min will be satisfactory.

The effluents from the recarbonation plant are finally passed through a rapid gravity filter.

Alum is often added with the lime and soda to combine chemical coagulation and water softening into a single process. Special compact units combined with mixing, flocculation, clarification, etc. into a single unit have been designed these days. Such units are relatively less costly and can give good results, if operated properly.

The Lime-Soda process is widely used because of the following *advantages* :

(*i*) The process is economical.

(*ii*) It can be easily combined with the usual water treatment methods, without much extra trouble or cost.

(*iii*) When lime and soda are added in addition to the coagulants, during the process of softening-cum-coagulation, lesser quantity of coagulant is generally required.

(*iv*) This treatment leads to an increase in the pH value of water, thus reducing the corrosion of the distribution pipes.

(*v*) The increased causticity may also sometimes help in killing pathogenic bacteria. This will happen when the alkalinity caused by calcium and magnesium hydroxide upto 20 to 50 mg/l is retained in the treated water for about 4 to 5 hours.

(*vi*) This treatment helps in reducing the total mineral content of water.

(*vii*) It helps in removing iron and manganese from the water, although only to a certain extent.

There are, however, four distinct *limitations* or *disadvantages* of lime soda treatment ; they are :

(*i*) A large quantity of sludge (*i.e.* the insoluble precipitates of calcium carbonate and magnesium hydroxide) is formed in this process, which must be disposed of by some suitable method. This sludge may be discharged into sewers or may sometimes be used for raising low lying areas after drying.

Theoretically, each mg/l of calcium hardness removed (expressed as $CaCO_3$) will produce 1 mg/l of sludge ($CaCO_3 \downarrow$)* ; each mg/l of magnesium hardness removed (expressed as $CaCO_3$) will produce 0.58 mg/l of sludge ($Mg(OH)_2$* $\downarrow$) ; and each mg/l of added lime (expressed as $CaCO_3$) will produce 1 mg/l of sludge ($CaCO_3 \downarrow$)*. The *theoretical computation of sludge* can thus be made by

*A visual inspection of the reactions involved in the lime soda treatment will reflect that each mol of added lime produces one mol of $CaCO_3 \downarrow$ (sludge), along with precipitating one mol of $Mg(OH)_2 \downarrow$ or $CaCO_3 \downarrow$ for each mol of magnesium or calcium hardness, respectively. Therefore when such calcium hardness is expressed in terms of $CaCO_3$, then naturally 1 mg/L of this hardness will cause 1 mg/L of $CaCO_3 \downarrow$ sludge. Similarly, each mg/L of added lime (expressed as $CaCO_3$) will contribute to each mg/l of $CaCO_3 \downarrow$ sludge. In case of magnesium hardness (expressed as $CaCO_3$ of molecular mass = 100)), the produced sludge is $Mg(OH)_2$ of molecular mass = 58 ; and hence, 1 mg/L of magnesium hardness (expressed as $CaCO_3$) will produce 0.58 mg/L of $Mg(OH)_2 \downarrow$ sludge.

using the following equation :

Dry sludge produced in mg/l

$$= \left[C_{aR} + 0.58\, M_{gR} + L_{iA} \right] \quad \text{...(9.41)}$$

where C_{aR} = Calcium hardness removed in mg/l (expressed as $CaCO_3$)

M_{gR} = Magnesium hardness removed in mg/l (expressed as $CaCO_3$)

L_{iA} = Lime added in mg/l (expressed as $CaCO_3$)

When water softening is combined with coagulation, additional sludge from coagulation of suspended materials and precipitation of metal coagulants, as computed in article 9.12 by eqn. (9.26) shall be produced, in addition to the sludge volume produced by softening under eqn. (9.41) above.

(*ii*) Careful operation and skilled supervision is required in order to get good results.

(*iii*) Incrustation of pipe walls of the distribution system will result, if the water is not properly recarbonated. The total omission of recarbonation will certainly cause troubles in the filter operations in addition to the incrustation of pipes.

(*iv*) This process cannot help in producing waters of zero hardness. This is because of the fact that calcium carbonate, though slightly, is soluble in water (the solubility being of the order of 30 mg/l). Hence, the lime soda process can remove hardness only to about 50 mg/l. However, this limitation of the method is no disadvantage for treating public supplies, because certain hardness is always desired in drinking waters from health considerations and also for the protection of distribution pipes and plumbing fixtures.

Example 9.30. *Calculate the amount of lime and soda required to treat two million litre (M.l.) of hard water, containing carbonate hardness equal to 15° French, and magnesium hardness equal to 9° French, and a total hardness of 24° French.*

Solution. ***1° French of hardness = 10 mg/l of hardness as $CaCO_3$.***

Chemical reactions involved in lime soda process are given in article 9.26.1. A visual inspection of these reactions shows that *lime* converts the entire magnesium hardness into calcium hardness in 1 : 1 mol ratio ; and carbonate hardness of calcium finally uses lime in 1 : 1 mol ratio. The original carbonate hardness of calcium, and that of magnesium (on conversion into calcium) will, thus, require lime. *The entire carbonate hardness thus requires lime, in addition to the lime required by the magnesium hardness (Mg^{++}).*

The non-carbonate hardness of calcium, and that of magnesium (of course on conversion into calcium) require soda. Hence, soda is required to remove the entire non-carbonate hardness.

In the given question,

Magnesium hardness	= 9° F = 90 mg/l	(as $CaCO_3$)
Carbonate hardness	= 15° F = 150 mg/l	
Total hardness	= 24° F = 240 mg/l	

Total hardness = Carbonate hardness + non-carbonate hardness

$\therefore$ 240 mg/l = 150 mg/l + non-carbonate hardness

or Non-carbonate hardness = 240 – 150 = 90 mg/l as $CaCO_3$

Lime is required to remove hardness

= Carbonate hardness + Magnesium hardness.

= 150 mg/l + 90 mg/l = **240 mg/l as $CaCO_3$**

(i) Qty. of lime required to remove 240 mg/l (as $CaCO_3$) of hardness is computed as follows :

From molecular masses, we find that 100 parts of $CaCO_3$ requires 56 parts of pure lime (CaO) for treatment, or

100 *mg/l* of $CaCO_3$ requires = 56 *mg/l* of CaO (quick lime)

$\therefore$ 240 *mg/l* of $CaCO_3$ will require = $\frac{56}{100} \times 240$ *mg/l* of CaO

= 135 *mg/l* of quick lime

Hence, quick lime needed = 135 *mg/l*.

Since 2 million litres of water is to be treated, total quick lime required

= 135 *mg/l* × 2 × 10^6 *l*

= 135 × 2 × 10^6 mg = **270 kg. Ans.**

(*ii*) The Quantity of soda is required to react with the entire non-carbonate hardness of 90 mg/l (as $CaCO_3$), which is computed below :

From molecular masses, we have

100 parts of $CaCO_3$ requires = 106 parts of Na_2CO_3

$\therefore$ 90 mg/l of $CaCO_3$ will require = $\frac{106}{100} \times 90$ mg/l of Na_2CO_3

= 95.4 mg/l of soda.

Total soda required in 2 Ml of water

= 95.4 mg/l × 2 × 10^6 *l*

= 95.4 × 2 × 10^6 mg

= 190.8 kg

say 191 kg. Ans.

Example 9.31. *Compute the quantities of lime and soda required to soften raw water containing alkalinity of 220 mg/l ; hardness as $CaCl_2$ = 40 mg/l, and as $MgSO_4$ = 60 mg/l. Water required to be treated is 1 million litres.*

Solution. Before solving this numerical question, we will like to explain alkalinity and its relationship with hardness and pH value of water, as given below :

Alkalinity may be defined as the quantity of ions in water that will react to neutralise the hydrogen ions (H^+). It will, thus, represent the ability of water to neutralise acid.

Alkalinity in water is chiefly caused by the presence of bicarbonates (HCO_3^-), carbonates (CO_3^-), and hydroxides (OH^-). Free H^+ evidently cause negative alkalinity. The alkalinity is therefore expressed by the equation

$$\textbf{Alkalinity} = [HCO_3^-] + [2CO_3^{--}]^* + [OH^-] - [H^+] \quad ...(9.42)$$

where [ ] refers to the concentration in moles/L

In most natural waters (pH = 6 to 8), the H^+ and OH^- are negligible**, *and hence for usually occurring waters, alkalinity can be considered to be equal to the sum of bicarbonate and carbonate ions. These bicarbonate and carbonate ions in water are usually given by Ca, Mg and Na metals*, and if there is no sodium bicarbonate or sodium carbonate present in water, then naturally the entire alkalinity shall be caused by the bicarbonates and carbonates of Ca and Mg, which is nothing but the carbonate hardness of water. *The carbonate hardness can thus be considered to be equal to the alkalinity, provided there is no sodium alkalinity in water*. And, the sodium alkalinity will be absent, if sulphates or chlorides of calcium or magnesium (*i.e.* salts causing non-carbonate hardness) are at all present in water, because such salts react with sodium alkalinity to convert it into calcium or magnesium alkalinity. Such salts can therefore be present themselves, only after they exhaust and convert the sodium alkalinity into calcium or magnesium alkalinity. The presence of such 'non-carbonate hardness' salts in a water will indicate the absence of sodium alkalinity. *It therefore follows that if any non-carbonate hardness (NCH) is present in a water, then its carbonate hardness will be equal to its alkalinity, since sodium alkalinity will be zero.*

However, if a water does not contain any non-carbonate hardness, then there is a possibility of the presence of sodium alkalinity, and hence the alkalinity may exceed the carbonate hardness ; but in such a case, the carbonate hardness will be equal to the total hardness (TH). It can hence be concluded that *the carbonate hardness is equal to the alkalinity or the total hardness, whichever is less.*

By convention, alkalinity is always expressed in mg/L of $CaCO_3$, unless otherwise specified.

Experimental tests on alkalinity have further established the following relation between the pH of water and the different types of alkalinity.

(*i*) When pH of water is above 11.5, the entire alkalinity is in the form of carbonate (CO_3^{--}) and OH^-. OH ions are not negligible in this pH range.

(*ii*) When pH of water is between 8.3 to 7.5, the entire alkalinity is in the form of bicarbonates (HCO_3^-).

(*iii*) When pH of water is less than 4.5, the entire alkalinity is in the form of carbonic acid (H_2CO_3), and the alkalinity will be negative (due to H^+).

The above indicated variation of alkalinity species *Vs* pH of water is shown in Fig. 9.48.

*The CO_3^{--} is multiplied by 2 because it can accept 2 protons.

**[$H^+ = 10^{-6}$ and $OH^- = 10^{-8}$ for pH = 6,

$H^+ = 10^{-7}$ and $OH^- = 10^{-7}$ for pH = 7], and

$H^+ = 10^{-8}$ and $OH^- = 10^{-6}$ for pH = 8.

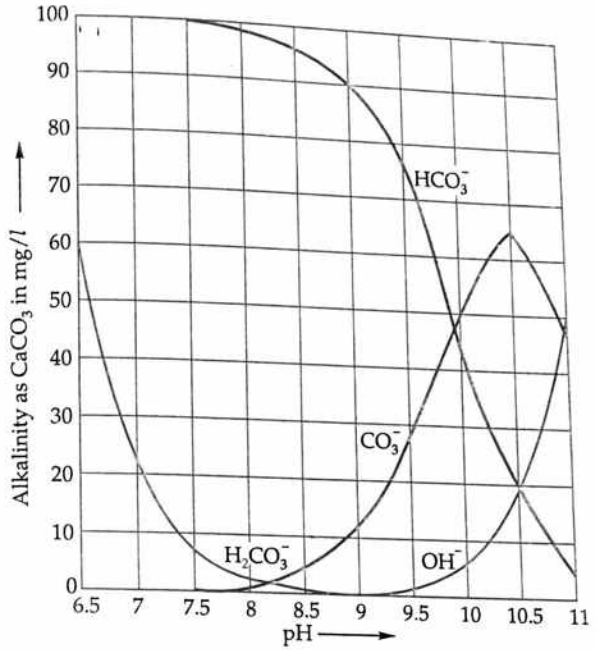


Fig. 9.48. Constituent species of alkalinity *Vs* pH of water.

In the given question, since the water contains $CaCl_2$ and $MgSO_4$, it is evident that the sodium alkalinity is not present (as otherwise $CaCl_2$ and $MgSO_4$ would not have existed independently), as the same would have been utilised in reacting with the carbonates and bicarbonates of sodium. Hence, the alkalinity of 220 mg/l (as $CaCO_3$ of course) shall be equal to the Carbonate hardness of the given water.

Lime required. Lime is now required to react with this carbonate hardness of 220 mg/l (as $CaCO_3$), and the non-carbonate hardness due to magnesium salts (*i.e.* 60 mg/l of $MgSO_4$). The quantity of lime required is computed below :

(*i*) *lime required to react with carbonate hardness of 220 mg/l (as $CaCO_3$) :*

100 mg/l of $CaCO_3$ (alkalinity) needs = 56 mg/l of CaO

$$\therefore \quad 220 \text{ mg/l of } CaCO_3 \text{ (alkalinity) needs} = \frac{56}{100} \times 220$$

$$= 123.2 \text{ mg/l of CaO} \quad ...(i)$$

(*ii*) *lime required to react with 60 mg/l of $MgSO_4$*

120 mg/l of $MgSO_4$ needs = 56 mg/l of CaO

$$\therefore \quad 60 \text{ mg/l of } MgSO_4 \text{ needs} = \frac{56}{120} \times 60 \text{ mg/l of CaO}$$

$$= 28 \text{ mg/l of CaO} \quad ...(ii)$$

Total pure lime (CaO) required = (*i*) + (*ii*)
= 123.2 + 28 = 151.2 mg/l
= 151.2 kg/Ml

Hence, lime required for 1 Ml of raw water = **151.2 kg. Ans.**

Soda required. Soda is required to react with the entire non-carbonate hardness, consisting of 60 mg/l of $MgSO_4$ and 40 mg/l of $CaCl_2$, as computed below :

(*a*) 120 mg/l of $MgSO_4$ needs = 106 mg/l of Na_2CO_3

$\therefore$ 60 mg/l of $MgSO_4$ needs = $\frac{60}{120} \times 60$ mg/l of Na_2CO_3
= 53 mg/l of Na_2CO_3 ...(*iii*)

(*b*) 110 mg/l of $CaCl_2$ needs = 106 mg/l of Na_2CO_3

$\therefore$ 40 mg/l of $CaCl_2$ needs = $\frac{106}{110} \times 40$ mg/l of Na_2CO_3
= 38.6 mg/l of Na_2CO_3 ...(*iv*)

Total soda required = (*iii*) + (*iv*)
= 53 + 38.6 mg/l = 91.6 mg/l

Soda required for treating 1 Ml of water
= 91.6 mg/l × Ml = **91.6 kg. Ans.**

Example 9.32. *The analysis of a hard water shows the following compositions :*

Free carbon dioxide	= *3 mg/l*
Alkalinity	= *68 mg/l*
Non-carbonate hardness	= *92 mg/l*
Total magnesium	= *15 mg/l.*

Assume that it is possible to remove all but 35 mg/l of carbonate hardness with lime, and that the treated water is to have a total hardness of 80 mg/l. Determine the amount of hydrated lime and soda required for treatment per million litre of raw water.

Solution. Total hardness to be left = 80 mg/l (given)
Carbonate hardness to be left = 35 mg/l (given)
$\therefore$ Non-carbonate hardness to be left = 80 – 35 = 45 mg/l.
Non-carbonate hardness of raw water = 92 mg/l (given)
$\therefore$ Non-carbonate hardness to be removed from raw water
= (92 – 45) mg/l = **47 mg/l.**

Now, we know that the lime is required to remove carbonate hardness, free carbon dioxide, and magnesium present in water ; whereas soda is required to remove non-carbonate hardness. Their quantities are computed as below :

Lime requirement. *Lime is required for three reactions*, as computed below :

(*i*) *Lime required for free carbon dioxide.* Since the molecular mass of CO_2 is 44, and that of lime (CaO) as 56, we have

44 mg/l of CO_2 requires = 56 mg/l of CaO

$\therefore$ 3 mg/l of CO_2 will require = $\frac{56}{44} \times 3$ mg/l of CaO
= 3.82 mg/l of CaO

$\therefore$ CaO required for 1 Ml of water
= **3.82 kg** ...(*i*)

(*ii*) *Lime required for carbonate hardness, which equals to the given alkalinity* of 68 mg/l.*

Since the molecular mass of $CaCO_3$ is 40 + 12 + 48 = 100 gm, and that of CaO is 40 + 16 = 56 gm, we have

Carbonate hardness (CH) of 100 mg/l of $CaCO_3$ requires
= 56 mg/l of CaO

$\therefore$ Carbonate hardness of 68 mg/l of $CaCO_3$ will require
= $\frac{56}{100} \times 68$ mg/l of CaO
= 38.08 mg/l of CaO

$\therefore$ CaO required for 1 Ml of water = **38.08 kg** ...(*ii*)

(*iii*) *Lime required for magnesium.* Since 24 mg/l of magnesium (in magnesium salt) reacts with 56 mg/l of pure lime (CaO), we have

24 mg/l of Mg requires = 56 mg/l of CaO

$\therefore$ 15 mg/l of Mg will require = $\frac{56}{24} \times 15$ mg/l of CaO = 35 mg/l of CaO

$\therefore$ Lime required for 1 Ml of water = 35 kg ...(*iii*)

Hence, total pure lime (CaO) required
= (*i*) + (*ii*) + (*iii*)
= 3.82 + 38.08 + 35 = **77.90 kg**

Also, 56 kg of pure lime (CaO) is equivalent to 74 kg of hydrated lime *i.e.*, $Ca(OH)_2$, we have

Hydrated lime (Impure lime) required per M.l. of raw water

= 77.90 × $\frac{74}{56}$ = **102.94 kg. Ans.**

Quantity of Soda Required. Soda (Na_2CO_3), *is required for non-carbonate hardness*, as follows :

Non-carbonate hardness to be removed
= 47 mg/l (as $CaCO_3$)

Now,

NCH of 100 mg/l of $CaCO_3$ requires = 106 mg/l of Na_2CO_3

$\therefore$ NCH of 47 mg/l of $CaCO_3$ will require = $\frac{106}{100} \times 47$ mg/l of Na_2CO_3
= 49.8 mg/l

$\therefore$ Soda reqd. for 1 Ml of water = **49.8 kg. Ans.**

*Since the water contains certain non-carbonate hardness, its alkalinity is equal to its carbonate hardness.

Example 9.33. *Calculate the annual cost of slaked lime and soda far treating 70,000 litres of water per day, if the water contains the following impurities :*

$CaCO_3 = 280$ ppm	$Mg(HCO_3)_2 = 100$ ppm
$NaCl = 35$ ppm	$CaSO_4 = 110$ ppm
$MgCl_2 = 138$ ppm	$MgSO_4 = 80$ ppm
$Fe_2O_3 = 55$ ppm	$SiO_2 = 40$ ppm.

Purity of slaked lime is 85% and that of soda is 100%. Slaked lime costs ₹ 80 per quintal and soda for ₹ 400 per quintal.

[*Atomic weights in gm : Ca = 40, Mg = 24, S = 32, Cl = 35.5, C = 12, O = 16, Na = 23, Fe = 56, and Si = 28*].

Solution. Magnesium bicarbonate reacts with lime in two steps, with total equation as :

$$Mg(HCO_3)_2 + 2Ca(OH)_2 \longrightarrow 2CaCO_3 \downarrow + Mg(OH)_2 + 2H_2O$$

It shows that magnesium bicarbonate needs two equivalents of lime, as against the one equivalent of lime needed by others like calcium bicarbonate, magnesium chloride, and magnesium sulphate. Calcium sulphate does not need any lime.

(A) **Requirement of hydrated lime** is thus worked out as below :

(*i*) *To react with the $CaCO_3$ (of course existing as bicarbonate) of 280 ppm*.*

100 ppm of $CaCO_3$ needs hydrated lime = 74 ppm

$\therefore$ 280 ppm of $CaCO_3$ will need hydrated lime $= \frac{74}{100} \times 280$ ppm

$= 207.2$ ppm ...(*i*)

(*ii*) *To react with the $Mg(HCO_3)_2$ of 100 ppm.*

146 ppm of $Mg(HCO_3)_2$ needs hydrated lime = (2) × 74 ppm

$\therefore$ 100 ppm of $Mg(HCO_3)_2$ will need hydrated lime $= \frac{2 \times 74}{146} \times 100$

$= 101.4$ ppm ...(*ii*)

(*iii*) *To react with the $MgCl_2$ of 138 ppm.*

95 ppm of $MgCl_2$ needs hydrated lime = 74 ppm

$\therefore$ 135 ppm of $MgCl_2$ will need hydrated lime $= \frac{74}{95} \times 138$ ppm

$= 107.5$ ppm ...(*iii*)

(*iv*) *To react with the $MgSO_4$ of 80 ppm.*

120 ppm of $MgSO_4$ needs slaked lime = 74 ppm

$\therefore$ 80 ppm of $MgSO_4$ will need slaked lime $= \frac{74}{120} \times 80$ ppm

$= 49.3$ ppm ...(*iv*)

*For usual waters having pH ≤ 8.3, the cal. carbonate alkalinity (280 ppm here) will exist in the form of calcium bicarbonate alone, as explained earlier.

Total lime required

$= (i) + (ii) + (iii) + (iv)$

$= 207.2 + 101.4 + 107.5 + 49.3$

$= 465.4$ ppm ...(1)

$\therefore$ Daily lime required for treating 70,000 litres of raw water

$= \frac{465.4 \times 70,000}{10^6}$ kg

$= 32.58$ kg.

Since the purity of lime is 85%, the available lime required daily

$= \frac{32.58}{0.85}$ kg $= 38.32$ kg.

$\therefore$ Lime required annually

$= 38.32$ kg × 365 $= 13986.8$ kg

$= 139.868$ quintal $= 140$ quintal (say).

Since lime costs ₹ 80.00 per quintal, we have the annual cost of lime

$=$ ₹ 80.00 × 140

$=$ ₹ 11,200 ...(A)

(B) **Requirement of soda** is worked out as below :

Soda (Na_2CO_3) is required for reacting with $MgCl_2$, $CaSO_4$, and $MgSO_4$, as calculated below :

(*i*) *To react with $MgCl_2$ of 138 ppm (of course after $MgCl_2$ has been converted into equivalent $CaCl_2$ by lime)*

95 ppm of $MgCl_2$ needs $Na_2CO_3 = 106$ ppm

$\therefore$ 138 ppm of $MgCl_2$ will need $Na_2CO_3 = \frac{106}{95} \times 138$ ppm

$= 154$ ppm ...(*i*)

(*ii*) *To react with $CaSO_4$ of 110 ppm.*

136 ppm of $CaSO_4$ needs $Na_2CO_3 = 106$ ppm

$\therefore$ 110 ppm of $CaSO_4$ needs $Na_2CO_3 = \frac{106}{130} \times 110$ ppm

$= 85.7$ ppm ...(*ii*)

(*iii*) *To react with $MgSO_4$ of 80 ppm (of course after $MgSO_4$ has been reduced into equivalent $CaSO_4$ by lime)*

120 ppm of $MgSO_4$ needs $Na_2CO_3 = 106$ ppm

$\therefore$ 80 ppm of $MgSO_4$ will need $Na_2CO_3 = \frac{106}{120} \times 80$ ppm

$= 70.7$ ppm ...(*iii*)

Total soda required

$= (i) + (ii) + (iii)$

$= 154 + 85.7 + 70.7 = 310.4$ ppm $= 310.4$ mg/l

Daily soda required to treat 70,000 litres of water

$$= \frac{310.4 \times 70,000}{10^6} \text{ kg} = 21.73 \text{ kg.}$$

∴ Annual consumption of soda

$$= 21.73 \text{ kg} \times 365 = \frac{21.73 \times 365}{100} \text{ quintal} = 79.3 \text{ quintal.}$$

∴ Annual cost of soda

$$= ₹\ 400 \times 79.3 = ₹\ 31,720 \quad \text{...(B)}$$

Hence, annual cost of lime and soda

$$= A + B$$

$$= ₹\ 11,200 + ₹\ 31,720 = ₹\ \mathbf{42,920.}\ \textbf{Ans.}$$

Example 9.34. *Calculate the requirement of lime and soda for cold softening of 2,00,000 litres of raw water, found to have the following chemical composition :*

Analysis of raw water

Dissolved $CO_2 = 39.6$ *mg/l*

$Ca^{++} = 44$ *mg/l*

$Mg^{++} = 18$ *mg/l*

$Na^{+} = 16$ *mg/l*

Alkalinity $(HCO_3^-) = 122$ *mg/l.*

Solution. The problem can be easily solved by converting constituent quantities in mg/l into m.eq/l by dividing mg/l by the equivalent mass, as shown below in table 9.7 :

Table 9.7

S. No.	*Constituents*	*mg/l*	*Eq. mass (gm)*	$m.eq/l = \frac{mg/l}{Eq.\ mass} = \frac{Col.(3)}{Col.(4)}$
(1)	*(2)*	*(3)*	*(4)*	*(5)*
1.	CO_2 (Dissolved)	39.6	22	39.6/22 = 1.8
2.	Ca^{++}	44	20	44/20 = 2.2
3.	Mg^{++}	18	12	18/12 = 1.5
4.	HCO_3^-	122	61	122/61 = 2

Requirement of slaked lime i.e. $Ca(OH)_2$. Lime is required to neutralise : (*i*) free CO_2, (*ii*) Carbonate hardness (*i.e.* HCO_3^-), and (*iii*) Mg^{++}. This requirement is thus given as

$$= [\text{m.eq/l of } CO_2 + \text{m.eq./l of } HCO_3^- + \text{m.eq/l of } Mg^{++}] \times [\text{Eq. mass of } Ca(OH)_2]$$

$$= (1.8 + 2 + 1.5)\ 37 \text{ mg/l} = \mathbf{196.1\ mg/l.}$$

Total Lime required for treating 2,00,000 litres of raw water

$$= \frac{196.1 \times 2,00,000}{10^6} \text{ kg} = \mathbf{39.22\ kg.}\ \textbf{Ans.}$$

Requirement of Soda (Na_2CO_3). Soda is required to neutralise the entire non-carbonate hardness which eventually equals the sum of Ca^{++} and Mg^{++} (T.H.) lessened by HCO_3^- (C.H.). This requirement is thus given as

$$= [\text{m.eq./l of } Ca^{++} + \text{m.eq./l of } Mg^{++} - \text{m.eq/l of } HCO_3^-] \times [\text{Eq. mass of } Na_2CO_3]$$

$$= (2.2 + 1.5 - 2)\ (53) \text{ mg/l} = \mathbf{90.1\ mg/l}$$

Total Soda required for treating 2,00,000 litres of raw water

$$= \frac{90.1 \times 2,00,000}{10^6} \text{ kg} = \mathbf{18.02\ kg.}\ \textbf{Ans.}$$

Example 9.35. *The result of chemical analysis of a sample of raw water is given below :*

$Ca^{++} = 80$ *mg/l*

$Na^{+} = 2.5$ *m.eq./l*

Total alkalinity $= 80$ *mg/l as* $CaCO_3$

Total hardness $= 120$ *mg/l as* $CaCO_3$

$SO_4^{--} = 20$ *mg/l as* $CaCO_3$

$Cl^- = 140$ *mg/l as* $CaCO_3$

$NO_3^- = 5$ *mg/l as* $CaCO_3$

(*i*) *Prepare a bar diagram for the raw water*

(*ii*) *Estimate the quantity in kg/day of CaO (90% pure) and Soda ash (95% pure) required to soften 4 million litres per day of this water.*

(*Civil Services, 1993*)

Solution. A bar chart of water analysis is the graphical presentation of the water analysis, and is generally obtained by plotting milli equivalent per litre values of various radicals, to a scale. In order to plot such a chart, it is therefore necessary to compute milli equivalent/litre (m.eq./l) values for all the cations (positive radicals) as well as anions (negative radicals). The sum of the m.eq./l values for cations must be equal to the sum of the m.eq./l values for anions. In perfect evaluation, they would exactly be the same, because water in equilibrium is electrically balanced.

The values of different radicals in mg/l as given in the question, are therefore to be converted into m.eq./l values, except for Na^+, where concentration is directly given in m.eq./l. The visual inspection of the given data also reflects that the concentrations of different radicals are given in mg/l as $CaCO_3$, whose equivalent mass is 50. Hence, the given values of mg/l as $CaCO_3$ will have to be divided by this equivalent mass of $CaCO_3$ in order to obtain their concentrations in m.eq./l.

However, the concentration of Ca^{++} is given as 80 mg/l, which cannot be its concentration as Ca^{++}, since the total hardness, as $CaCO_3$ = $\left(Ca^{++} \times \frac{50}{20} + Mg^{++} \times \frac{50}{12}\right)$ is given to be 120 mg/l. Had Ca^{++} been equal to 80 mg/l, then eventually, the total hardness would have been something more than $80 \times \frac{50}{20}$ = 200 mg/l (as against the given value of 120 mg/l). It, therefore, transpires that Ca^{++} present in water also equals to 80 mg/l as $CaCO_3$.

Mg^{++} present in this water (as $CaCO_3$) can then be computed as :

= Total hardness – Ca^{++} (as $CaCO_3$)
= (120 – 80) mg/l (as $CaCO_3$)
= 40 mg/l (as $CaCO_3$)

Hence, **Mg^{++} = 40 mg/l as $CaCO_3$.**

Since the alkalinity of 80 mg/l is lesser than the total hardness of 120 mg/l, it transpires that the carbonate hardness is 80 mg/l (being lesser of the two values). Hence, the entire alkalinity can be considered to be bicarbonate alkalinity, causing carbonate hardness.

All the given values of radicals (as $CaCO_3$) are now listed in Col. (3) of table 9.9. These values are converted into m.eq./l values by dividing mg/l (as $CaCO_3$) by 50 (*i.e.* eq. mass of $CaCO_3$), as shown in Col (4) of this table.

Table 9.9

S. No.	*Name of radical*	*mg/l as $CaCO_3$*	*$m.eq/l = \frac{mg/l \text{ as } CaCO_3}{Eq. mass \text{ of } CaCO_3 \text{ in gm}} = \frac{col.(3)}{50}$*	*Remarks*
(1)	*(2)*	*(3)*	*(4)*	*(5)*
1.	Ca^{++}	80	1.6	Water analysis is perfect, as it is getting balanced
2.	Mg^{++}	40	0.8	
3.	Na^{+}	—	2.5 (directly given)	
			Total for cations = **4.9**	
4.	SO_4^{--}	20	0.4	
5.	Cl^{--}	140	2.8	
6.	NO_3^{-}	5	0.1	
7.	HCO_3^{-} (Alkalinity)	80	1.6	
			Total for anions = 4.9	

The bar chart is now plotted as shown in Fig. 9.49.

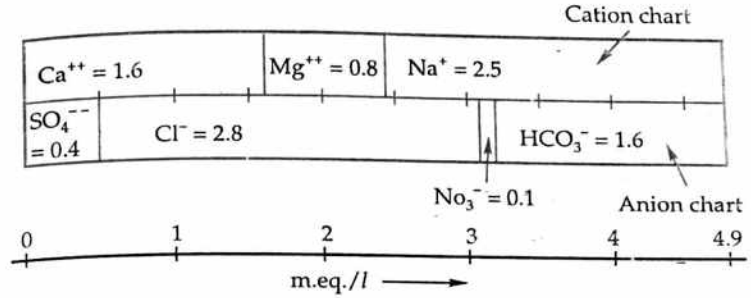


Fig. 9.49. Bar chart for the raw water given in example 9.35.

(*a*) ***Computing Qty. of Lime (CaO) required for Softening.*** Lime is required for neutralising the carbonate hardness (*i.e.* alkalinity) plus Mg^{++}, and hence is given as

= [m.eq/l of HCO_3^- + m.eq/l of Mg^{++}] × [Eq. mass of CaO]
= (1.6 + 0.8) 28 mg/l
= 67.2 mg/l

Qty. of impure lime reqd.

$= \frac{67.2}{0.90}$ mg/l (∵ Purity = 90%)
= 74.67 mg/l

Total qty. of lime required to treat 4 Ml/day of raw water
$74.67 \times 4 \times 10^6$ mg/day
= 74.67 × 4 kg/day = **298.67 kg/day. Ans.**

(*b*) ***Computing Qty. of Soda reqd. for Softening.*** Soda ash (Na_2CO_3) is required to neutralise non-carbonate hardness, which equals

$Ca^{++} + Mg^{++} - HCO_3^-$.
T.H. – C.H.

Hence, Soda reqd.

= [m.eq/l of Ca^{++} + m.eq/l of Mg^{++} – m.eq/l HCO_3^-] × (Eq. mass of Na_2CO_3)
= (1.6 + 0.8 – 1.6) 53 mg/l
= 42.4 mg/l

Qty. of Impure Soda reqd.

$= \frac{42.4}{0.95}$ mg/l (∵ Purity = 95%)
= 44.63 mg/l

Total Qty. of Soda required to treat 4 Ml/day of raw water

$= 44.63 \times (4 \times 10^6)$ mg/day

$= 178.5$ kg/day. **Ans.**

9.26.2. Zeolite *or* Base-Exchange *or* Cation-Exchange Process for Removing Hardness. Zeolites are the natural* salts or clays*, which are hydrated silicates of sodium and aluminium, having the general formula as $Na_2O\ Al_2O_3x.SiO_2\ y.H_2O$. The usual value of x is 2 or more, and that of y of ranging amounts. Naturally occurring zeolite like substances can also be manufactured synthetically, and they are known as **Resins****. *The zeolites or Resins have the excellent property of exchanging their cations ; and hence, during softening operation, the sodium ions of the zeolite get replaced by the calcium and magnesium ions present in hard waters.* The chemical reactions which may be involved are given in the following equation, where Z stands for the complex zeolite radical.

$$Na_2Z + \begin{matrix}Ca\\Mg\end{matrix}\left\{\begin{matrix}(HCO_3)_2\\SO_4\\Cl_2\end{matrix}\right. \longrightarrow Na_2\left\{\begin{matrix}(HCO_3)_2\\SO_4\\Cl_2\end{matrix}\right. + \begin{matrix}Ca\\Mg\end{matrix}\left\{\begin{matrix}\\Z\\\end{matrix}\right.$$

Sod. zeolite or active zeolite — Cal. or Mg. salt — Sod. salts which don't cause hardness — Ca or Mg zeolite *i.e.* exchanged or used zeolite

The calcium and magnesium zeolite can be regenerated into active sodium zeolite by treating it with 5—10 per cent solution of sodium chloride. The exchange reactions that take place during regeneration can be represented as :

$$\left.\begin{matrix}Ca\\Mg\end{matrix}\right\} Z + 2NaCl \longrightarrow Na_2Z + \left.\begin{matrix}Ca\\Mg\end{matrix}\right\} Cl_2$$

Used zeolite — Sod. chloride solution — Regenerated zeolite

A zeolite softener (or *a cation exchange unit*) resembles a sand filter in which the filtering medium is a zeolite rather than sand. The hard water enters through the top, and is evenly distributed on the entire zeolite bed. The softened water is collected through the strainers at the base. When a significant portion of the sodium in the zeolite has been replaced by calcium and magnesium, it is regenerated by first washing it with water by reversing the flow, and then treating it with 10 per cent solution of brine (NaCl). The excess brine solution retained in the zeolite after the treatment is removed by

*The most common naturally available zeolite is green in colour and generally termed as **green sand.**

The most common synthetic resin is a white coloured substance, called ***Permutit* ; manufactured from felspar, kaolin, clay, and soda.

again washing it with good water. The regenerated zeolite can be used afresh for softening.

Zeolite softeners may be either gravity or pressure filters ; **pressure filter type zeolite softeners,** such as shown in Fig. 9.50, being more common. The unit consists of a closed steel cylinder containing a bed of zeolite, which may be from 0.75 m to 2 m thick. The thicker bed has a greater capacity per unit area, and hence takes up less space and requires less water when washing is needed. The rate of filtration through a zeolite softener is about 100-300 litres per sq. m per minute [*i.e.* 0.1 to 0.3 m/minute] for bed depths of 0.75 m – 2 m (Deeper beds can be loaded on higher side).

Washing of Zeolite is done at rates of 0.2 to 0.4 m/min. The softner may operate either with downward flow (as explained above) or with upward flow. The advantage claimed for upward flow is that compaction of zeolite is prevented, greater efficiency is obtained, and washing of the softner is not required.

The zeolite process will result in a water of zero hardness, which generally, is not suitable for public supplies. Therefore, only a portion of the water passing through the treatment plant is to be softened, and then mixed with unsoftened water to obtain the desired water quality.

The advantages and disadvantages of this method are given below :

Advantages

(*i*) *Water of zero hardness can be obtained,* and hence, useful for specific uses in textile industries, boilers, etc.

(*ii*) The plant is compact, automatic and easy to operate.

(*iii*) No sludge is formed, and hence, there is no problem of sludge disposal.

(*iv*) The RMO (Running, maintenance and operation) cost is quite less.

(*v*) It also removes ferrous iron and manganese from water.

(*vi*) There is no difficulty in treating water of varying quality.

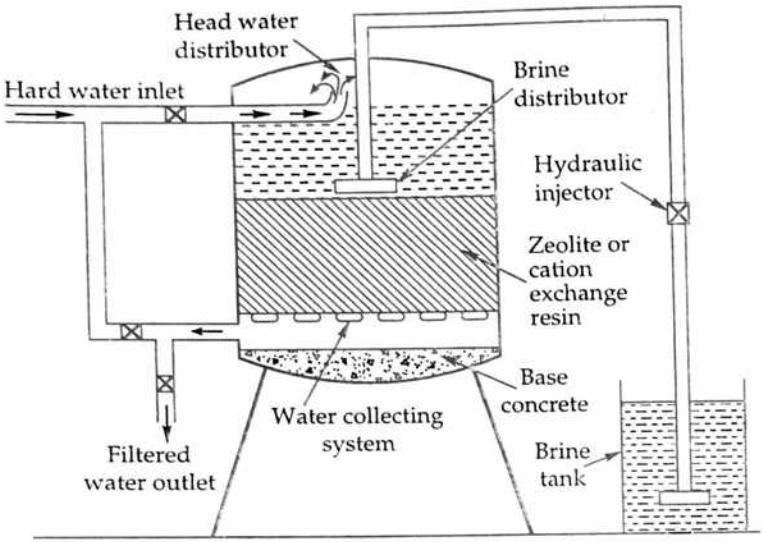
(*vii*) There is no problem of incrustation of pipes of the distribution system, as is there in the lime soda process.

Disadvantages

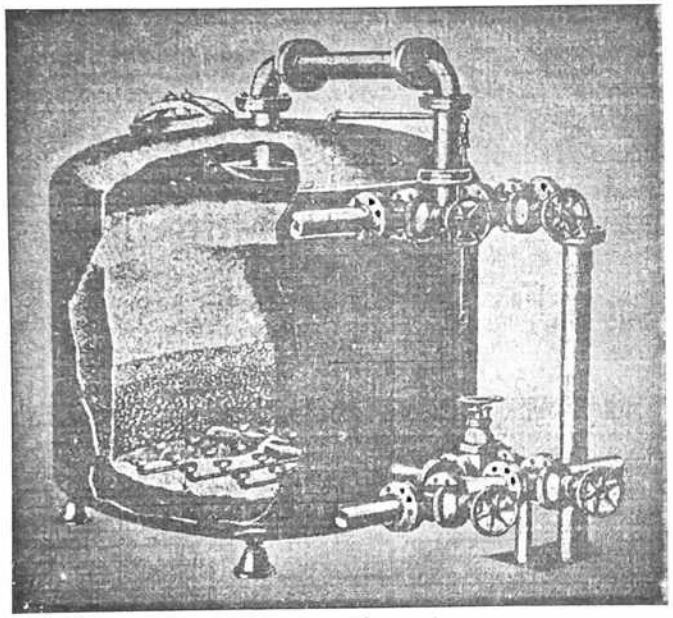
(*i*) This process is not suitable for treating highly turbid waters, because the suspended impurities get deposited around the zeolite particles, and thus cause obstruction to the working of the zeolite.

(*ii*) The process leaves sodium bicarbonate in water, which causes priming and foaming in industrial or boiler feed waters.

(*iii*) The zeolite process is costlier and unsuitable for treating waters containing iron and manganese. This is because of the fact that the iron zeolite or manganese zeolite formed during the chemical reactions, cannot be regenerated into sodium zeolite. The zeolite is thus wasted, although the iron and manganese are removed from the water.



(*a*) Sectional view



(*b*) Cutaway Photo view

Fig. 9.50. Zeolite water softener of the Permutit Co. (pressure filter type).

9.26.3. Demineralisation Process for Removing Hardness. As its very name implies, demineralisation means removing the minerals from the water. Since this process helps us in completely removing or reducing the mineral content by any desired extent, it is very suitable for producing water of any desired hardness or even mineral free water. This *demineralised water*, sometimes called *de-ionised water*, is as pure as distilled water, and is very

suitable for industrial purposes, especially for steam raising in high pressure boilers.

This complete removal of minerals present in water can be carried out by first passing the water through a bed of cation exchange resins, and then through a bed of anion exchange resins.

The process of passing the water through cation exchange resins produces almost similar effects, as are produced in the previous zeolite method, except that, hydrogen (instead of sodium) is exchanged for the basic metallic ions. The ***cation exchange resins***, in fact, are phenol aldehyde condensation products, which on sulphonation produces resinous mass having base exchange properties. Their chemical formula may be represented by H_2R, where H represents the hydrogen ions and R represents the organic part of the substance. The chemical reactions involved during the process may be represented as :

$$\underset{\text{Fresh cation exchange resin}}{Ca(HCO_3)_2 + H_2R} \longrightarrow \underset{\text{Exhausted resin}}{CaR} + 2H_2O + 2CO_2 \uparrow$$

$$CaCl_2 + H_2R \longrightarrow CaR + 2HCl$$

$$MgSO_4 + H_2R \longrightarrow MgR + H_2SO_4$$

$$2NaCl + H_2R \longrightarrow Na_2R + 2HCl$$

The water coming out of the cation-exchanger will now contain diluted carbonic acid, hydrochloric acid, sulphuric acid, etc. (instead of originally present minerals and salts) ; and can be removed by passing the water through a bed of anion exchange resin.

The ***anion-exchange resins*** are formed by the condensation of amines with formaldehyde, and are capable of replacing the anions (such as Cl^- or SO_4^- ions) with their hydroxyl ions. The chemical formula for such a resin may be written as ROH ; where OH represents the hydroxyl ions and R represents the organic part of the substance. The chemical formulae that may be involved are :

$$\underset{\text{Fresh anion exchange resin}}{ROH} + HCl \longrightarrow \underset{\text{Exhausted resin}}{RCl} + \underset{\text{Water}}{HOH}$$

$$\underset{\text{Fresh anion exchange resin}}{2ROH} + H_2SO_4 \longrightarrow \underset{\text{Exhausted resin}}{R_2SO_4} + \underset{\text{Water}}{2HOH}$$

The water coming out from this anion exchanger will then be free from minerals. The extent of removal will depend upon the strength and freshness of the resins used. The completely demineralised water can sometimes be added to raw water, so as to obtain the resultant supplies of any desired mineral content.

When the use of the resins have been in progress for a sufficient time, they become exhausted, and can be regenerated as follows :

Regeneration of cation exchange resins. The exhausted cation exchange resins can be regenerated by treating them with dilute hydrochloric acid or sulphuric acid. The reactions likely to take place are

$$\left.\begin{matrix}Ca\\Mg\\Na_2\end{matrix}\right\} + R + 2HCl \longrightarrow H_2R + \left.\begin{matrix}Ca\\Mg\\Na_2\end{matrix}\right\} Cl_2$$

Exhausted cation exchange resin — Regenerated cation exchange resin

$$\left.\begin{matrix}Ca\\Mg\\Na_2\end{matrix}\right\} + R + H_2SO_4 \longrightarrow H_2R + \left.\begin{matrix}Ca\\Mg\\Na_2\end{matrix}\right\} SO_4$$

Exhausted cation exchange resin — Regenerated cation exchange resin

Regeneration of anion-exchange resins. The exhausted anion exchange resins can be similarly regenerated by treating them with sodium carbonate solution. A typical likely reaction will be

$$\underset{\text{Exhausted anion resin}}{RCl} + Na_2CO_3 + 2H_2O \longrightarrow \underset{\text{Regenerated anion resin}}{2ROH} + 2NaCl + CO_2\uparrow + H_2O$$

9.26.4. Comparison of Lime-Soda and Zeolite Processes of Softening Water Supplies. The comparative merits and demerits of both these methods are summarised in Table 9.10.

Table 9.10. Lime Soda process Vs. Zeolite process of Softening Water Supplies

S. No. (1)	*Item* (2)	*Lime soda process* (3)	*Zeolite process* (4)
1.	Size of plant.	Bulky and large.	Compact and small.
2.	Skilled supervision, if required.	Careful and skilled supervision is necessary for obtaining results.	Automatic and easy to operate.
3.	Sludge troubles, if any.	Large quantity of sludge is formed, thereby posing problems for finding out space for its disposal.	No sludge is formed and therefore, there in no problem of sludge disposal.

Contd..

Table 9.10. Continues

S. No. (1)	*Item* (2)	*Lime Soda process* (3)	*Zeolite process* (4)
4.	Post treatment, if needed.	Recarbonation is a must after sedimentation and filtration. If recarbonation is not done,, incrustation of pipe walls of distribution system may occur.	No such post treatment is required.
5.	Results obtained.	This can produce waters of hardness not less than about 50 mg/l. Therefore, useful for public supplies only.	Water of zero hardness can be obtained. Therefore, useful for industrial supplies also.
6.	Removal of colour due to iron and manganese.	Can remove the colour due to iron and manganese, but only to a very small extent.	Can remove the colour due to iron and manganese, although very costly for treating such waters, because the exhausted Mn and Iron zeolites cannot be regenerated.
7.	Effects on bacteria.	The increased causticity may help in killing pathogenic bacteria, especially when alkalinity caused by calcium, magnesium or sodium hydroxides of 20 to 50 mg/l is maintained for 4 to 5 hours.	No such advantage is offered by this process.
8.	The pH of the treated water.	Increases the pH value of water, which reduces the corrosion of distribution pipes.	The pH value of the water is not affected.
9.	Care needed in handling materials	Careful handling is required because the materials *i.e.* (lime and soda) are corrosive.	Brine solution is required to be handled, which does not require much care.
10.	Economy	Process is economical and can be easily combined with usual water treatment methods at slight extra cost.	Process is costlier.
11.	Hardness which can be treated.	Excessively hard waters, particularly those which are high in magnesium hardness can be treated.	Raw waters with hardness greater than 800 mg/l cannot be easily and economically treated.
12.	Allowable turbidities in raw waters.	Highly turbid and acidic waters can be treated.	Highly turbid waters are difficult to be treated, because the suspended impurities deposited around the zeolite particles cause obstruction to the working of the zeolite.

Example 9.36. *Design a Zeolite-Softener for an industrial concern, using the following data :*

(i) Qty. of soft water required per hour = 25,000 litres

(ii) Hardness present in raw water as $CaCO_3$ = 400 ppm.

(iii) Hardness to be obtained in the treated supplies = 50 ppm.

(iv) Ion exchange capacity of the zeolite = 10 kg of hardness per cu. m. of zeolite

(v) Salt required for regeneration of exhausted zeolite = 50 kg per cu. m. of zeolite.

Also assume that the industry works for 2 shifts of 8 hours each, per day. Make suitable assumptions where needed, if not given.

Solution.

Qty. of soft water reqd. per shift of 8 hrs.

$= 25,000 \times 8$ litres $= 2,00,000$ litres

$= 2 \times 10^5$ litres.

Now, the hardness reqd. to be removed is upto 50 ppm out of a total hardness of 400 ppm. This means that 350 ppm *i.e.* $\frac{350}{400} \times 100 = 87.5\%$ of hardness has to be removed. Now, assuming that part of the total requirement will be softened to zero degree hardness, and then raw water will be added to get the water of the desired hardness (50 ppm), we can say that only 87.5% of the raw water needs to be softened to zero degree hardness, and the balance 12.5% can be added as raw water to obtain 50 ppm of hardness.

∴ The qty. of water to be treated per shift

$= 2 \times 10^5 \times 0.875$ litres $= \mathbf{1.75 \times 10^5}$ **litres.**

Let us also assume that the regeneration of the zeolite shall be carried out once in 8 hours shift, and that the regeneration process will take one hour. Hence, the zeolite softener will remain out of service for one hour out of 8 hours ; thus giving a useful service of 7 hours per shift. A balancing tank of 1 hr. capacity will hence be provided to supply water during regeneration period.

Now, the amount of hardness to be removed per shift

$$= \begin{bmatrix}\text{Qty. of water to be treated} \\ \text{per shift in litres}\end{bmatrix} \times \text{Hardness in mg/l}$$

$= (1.75 \times 10^5) \times 400$ mg

$= 700 \times 10^5$ mg

$= 70 \times 10^6$ mg $= 70$ kg

∴ The qty. of resin reqd.

$$= \frac{\text{Hardness to be removed in kg}}{\text{Iron-exchange capacity of resin in kg/cu. m}}$$

$$= \frac{70}{10} = \mathbf{7\ cu.\ m.}$$

Provide 6 units of 1.4 cum each ; thus giving one unit as stand by. Each unit of 1.4 cum may have 1 sq. m. plan area and 1.4 m depth. **Ans.**

Regeneration

After the softener has remained in operation for 7 hours, its zeolite beds will be regenerated by passing 10% solution of sodium chloride (NaCl) through it. The storage in the balancing tank will provide water during this one hour interval. The stand by unit will help during any break downs, etc.

Now, the qty. of salt reqd. for regeneration

$= 50$ kg/cu. m of zeolite

$= 50 \times 7$ kg.

$= 350$ kg (without additional unit).

Using 10% brine solution, means that 10 kg salt will be dissolved in water to make up 100 kg of solution.

∴ 350 kg of salt will produce

$$= \frac{350 \times 100}{10} = 3500 \text{ kg of water solution.}$$

$$= \frac{3500 \text{ kg}}{1000 \text{ kg/m}^3} = 3.5 \text{ cum.}$$

Hence, two tanks of 1.75 cu. m. capacity each can be provided. If 1.2 m is the dia. of such a tank, then the height reqd.

$$= \frac{1.75}{\frac{\pi}{4}(1.2)^2} = 1.53 \text{ m} = 1.55 \text{ m (say)}$$

Using 0.15 m free-board, the overall tank size will be 1.2 m dia × 1.7 m height. **Ans.**

Check for the contact period

The flow rate over the softener beds

$$= \frac{\text{Volume of water to be treated per shift}}{\text{7 hrs working of the softener}}$$

$$= \frac{1.75 \times 10^5}{7} \text{ litres/hr} = 25,000 \text{ litres/hr.}$$

Average flow velocity

or Rate of filtration $= \dfrac{\text{Rate of volume of water to be passed}}{\text{Surface area}}$

$$= \frac{25,000}{5 \times 1.0} \text{ litres/m}^2\text{/hr} = 5000 \text{ l/m}^2\text{/hr}$$

$$= \frac{5000}{60} \text{ l/m}^2\text{/min} = 83.3 \text{ l/m}^2\text{/min.}$$

$= 0.083$ m/minute.

(Ok, it is the less than about 0.3 m/min)

Since each bed is 1.4 m deep, the average time of travel through the bed, or the contact period

$$= \frac{1.4 \text{ m}}{0.083 \text{ m/min}} = \mathbf{16.9} \text{ minutes.}$$

Since the contact time available for the ion exchange is sufficient (> 7.5 minutes or so), the design is O.K.

MISCELLANEOUS TREATMENTS

Besides the normal treatment processes, such as, coagulation sedimentation, filtration and disinfection (or chlorination) ; certain other special treatments are sometimes required in order to remove the special minerals, tastes, odours, colours, etc. from water. For example, minerals such as iron and manganese may not be removed in the normal treatment processes, and hence have to be removed by some special treatment method. Similarly, dissolved gases such as carbon di-oxide (CO_2) or hydrogen sulphide (H_2S), etc., may sometimes be present in water, and may impart certain tastes and odours, which are not removed under the conventional treatments. Dissolved carbon di-oxide gas will also make the water corrosive. Organic matter derived from algae and other micro-organism may also impart tastes and odours which may not be easily removed by the ordinary conventional methods. Similarly, the industrial waste waters may contain organic matter, such as phenols, etc. which may create bad tastes and odours after chlorination. All these aspects must be looked into, and steps taken to remove all the tastes and odours from water, so as to make it attractive to the consumer's tongue. *Further, the fluoride content of the water should neither be less as to cause dental caries (early tooth decay) in children, during the calcination of their permanent teeth ; nor it should be too high as to cause fluorosis (discolouration of teeth and even deformation of bones in extreme cases).*

9.27. Removal of Colours, Odours and Tastes from Waters

The special treatments which may be given to the water under special conditions for removing colours, tastes and odours from it, are described below :

9.27.1. Aeration. Under the process of *aeration*, water is brought in intimate contact with air, so as to absorb oxygen and to remove carbon dioxide gas. It may also help in killing bacteria to a certain extent. It also helps in removing H_2S gas, and iron and manganese to a certain extent, from the treated water. The aeration of water can be carried out in one of the following ways :

(*i*) **By using spray nozzles.** In this method, water is sprinkled in air or atmosphere through special nozzles which breaks the water into droplets, thus permitting the escape of dissolved gases. Carbon di-oxide gas is thus considerably removed (up to 90% or so) in this method. However, considerable head of water is required for the working of these nozzles, which function efficiently at a pressure of 10 to 14 m head of water.

(*ii*) **By permitting water to trickle over cascades.** In this method, the water is made to fall through a certain height (1 to 3 m) over a series of steps (three to ten) with a fall of about 0.15 to 0.3 m in each step. The structure so formed is known as a **freefall aerator**. The simplest type of a freefall aerator is known as a **cascade aerator**. Fig. 9.51 (*a*) shows the design of a cascade aerator. Such aerators are widely used as water features. They will take large quantities of water in a comparatively small area at low head, are simple to be kept clean

and can be made of robust and durable materials with a long life. The plates can be made of cast iron, or of RCC, or of timber, or even of glass. The aerator

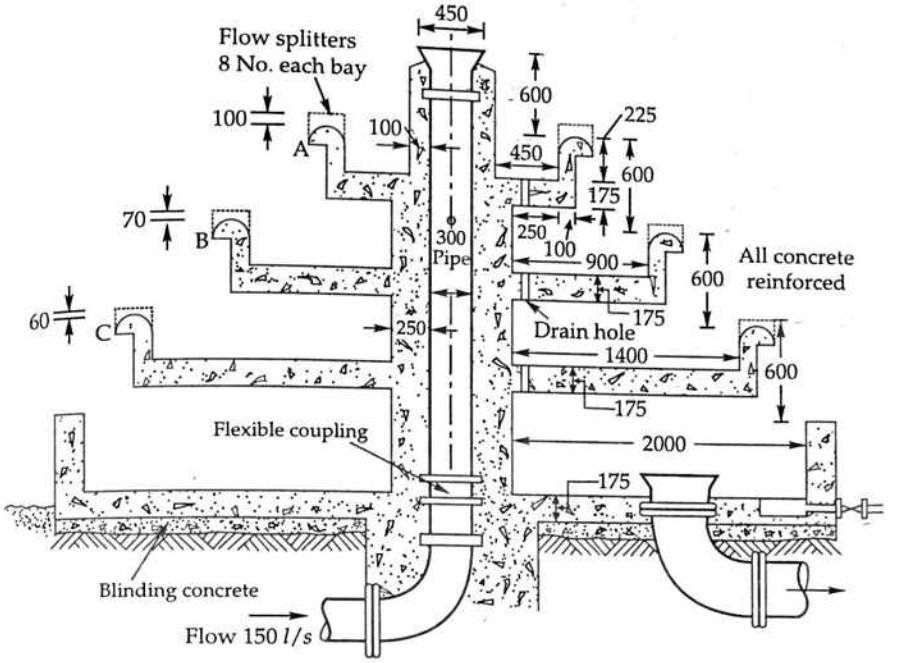


Fig. 9.51. (*a*) Section through a circular Cascade aerator.

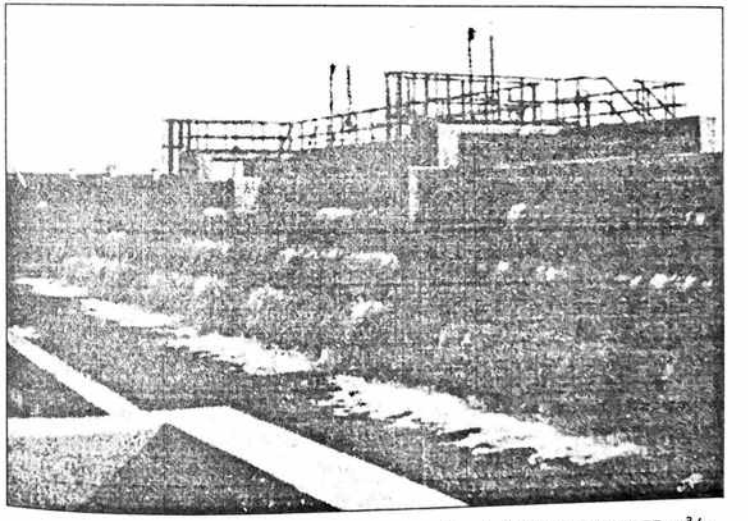


Fig. 9.51. (*b*) Photoview of a cascade aerator (each fall 0.5 m) for 2.75 m^3/s flow rate at Izmit Water Treatment Works, Turkey.

should preferably be installed in open air. However, for protection against air pollution, freezing and algal growth, it can be installed in a small house having plenty of louvered air inlets. The cascade aerators are efficient in raising dissolved oxygen content of water, but not for CO_2 removal, which is removed only in the range of 60 to 70%.

Weirs and waterfalls of any kind are, of course, cascade aerators.

Where a stream passes over an artificial or naturally occurring obstacle, a large contribution occurs to the self-purification of the river water, due to increase in DO, which accelerates the process of decomposition of organic matter. From the work carried out by the former U.K. Water Pollution Control Laboratories, an empirical relationship has been developed for the ratio, r, which is the ratio of the oxygen deficit just above a weir to the oxygen deficit just below the weir ; this relationship is given as :

$$r = 1 + 0.38abh(1 - 0.11h)(1 + 0.046T) \quad ...(9.43)$$

where $r = \dfrac{\text{Oxygen deficit of the stream just above a weir}}{\text{Oxygen deficit of the stream just below the weir}}$

$$= \frac{D_s - D_u}{D_s - D_d} \quad ...(9.44)$$

where D_s = Saturation D.O. at the given stream temperature

D_u = D.O. concentration upstream of weir

D_d = D.O. concentration down-stream of weir

a = a constant, having a value of 1.25 for slightly polluted water ; 1.00 for moderately polluted water ; and 0.85 for sewage effluents

b = a constant having a value of 1.00 for free fall weir, and 1.30 for a stepped weir

h = the height of the fall in metres

T = water temperature in °C.

Using the above formula, and knowing the fall height h, the increased DO downstream of fall *i.e.* D_d, can be calculated, if required.

In a cascade aerator, usually, the rate of flow may vary between 20 to 100 m^3/h. per m length of weir. To allow entrained air to mix in the water, each receiving basin should have a pool of water of depth 0.3 to 0.5 m. Weirs with serrated edges perform better, as they help to break water flow into separate jets. [Please see Fig. 9.51 (*b*)]. If the water is allowed to cling to the steps, specially at low discharge rates, the efficiency is reduced. The space requirement is typically of the order of 0.5 m^2 per 1 m^3/h water treated. The oxygen transfer efficiency, which is defined as the oxygen transferred (kg/h) per unit of power input, should be as high as 2.5 kg O_2/kWh.

(*iii*) **By air diffusion.** In this method, compressed air is bubbled through the water, so as to thoroughly mix it with water. Perforated pipes are, therefore, installed at the bottom of the settling tanks, and the compressed air is blown through them. The compressed air is thus bubbled up from the bottom of the tank. During its upward movement through the water body, it gets thoroughly

mixed up with the water contained in the tank, thereby completing the aeration process.

(*iv*) **By using trickling beds.** In this method, the water is allowed to trickle down the beds of coke, supported over the perforated bottomed trays, and arranged vertically in series, as shown in Fig. 9.52. Generally, three beds are used, the depth of each being about 0.6 m with a clear distance of about 0.45 m in between. The water is applied from the top through perforated distribution pipes and allowed to trickle down, up to the bottom bed. During this downward motion, the water gets mixed up with air, and *aeration* takes place. The size of the coke to be used, usually ranges between 50 to 75 mm. This method gives better results than what can be obtained by *'cascades'*, but is less effective than the method of *spray nozzles*.

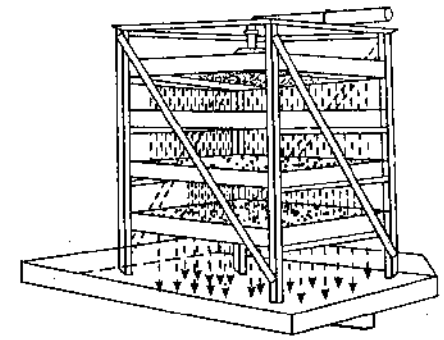


Fig. 9.52. Trickling bed aeration.

Aeration, though helpful in removing iron and manganese besides removing volatile gases, such as carbon dioxide and hydrogen sulphide, yet cannot be relied upon to remove or even reduce the tastes and odours of all kinds. Moreover, it cannot completely remove the carbon dioxide, and its residuals (such as 3 to 5 mg/l) always remain in practice even after aeration. However, the oxidation, made possible by aeration, helps in removing oils and decomposing products of algae and other aquatic vegetation, and thus helps in removing the odours, tastes and colours due to their presence.

Aeration should, however, be used only to a limited extent because too much of absorbed oxygen will make the water corrosive and may necessitate the *de-aeration process*. The **de-aeration** process can be carried out by allowing the water to trickle down through the gravel layers in a closed vessel at a pressure of about $\frac{1}{30}$th of an atmosphere. Dissolved oxygen can also be removed by chemical methods or by boiling, but the latter is undesirable for treating public supplies.

9.27.2. Treatment with Activated Carbon. Activated carbon is a specially treated carbon, which possesses the property of absorbing and attracting impurities, such as gases, liquids, and finely divided solids. Because of its excellent property of absorbing impurities, it is widely used for removing tastes and odours from public supplies.

It can be manufactured by heating or *charring* wood or sawdust or some other similar carbonaceous material at about 500°C in a closed vessel, and then slowly burning it (or oxidising it or activating it) under very closed controlled conditions at 800°C, thus removing hydrocarbons (which may interfere with the absorption of organic matter) from it. *The activated carbon is available under various patented trade names, such as Darco, Nuchar, etc.* It is available in granular as well as powder forms. It is highly porous and absorptive, and thus very useful for removing phenol type impurities.

The activated carbon is mostly used in the powdered form, and may be added to the water either *before or after the coagulation, but before filtration*. The most common method adopted is to add a portion in the mixing tank, and to add the remaining portion in water just before it enters the filter. This method of using activated carbon at two stages is called *split method*. Activated carbon may be applied at a constant rate or at varying rate. It may be applied either in a dry form or as a suspension or as a slurry. Its use as a suspension in water and then fed into the water to be treated as slurry, is generally preferred. Its use in powdered form may crate problems, such as the likelihood of being blown off due to winds. Nevertheless, in both the cases, the feeding equipments are similar to those used for adding coagulants. *The usual dose of activated carbon varies from 5 to 20 mg/l, and optimum dose may be determined first in the laboratory and then in the field.*

Activated carbon may also be used in the granular form as a filter media, instead of using sand in the rapid gravity filters or preferably in pressure filters. With the passage of time, it looses its absorptive powers and requires to be rejuvenated. This regeneration is carried out by forcing the live steam upward through the perforated pipes placed in the gravel bed of filters. This process of rejuvenation may have to be carried out at intervals of 1 month to 1 year, depending upon the quality of water being treated.

The use of activated carbon may thus serve the following *advantages* :

(*i*) When used in powdered form before coagulation, it aids in coagulation.

(*ii*) It reduces the chlorine demand of treated water.

(*iii*) It removes the organic matter present in water.

(*iv*) It removes the tastes, odours and colours caused by the presence of iron, manganese, phenols, excess chlorine, hydrogen sulphide, etc.

(*v*) Its overdose is not harmful.

9.27.3. Treatment with Copper Sulphate. Copper sulphate ($CuSO_4.7H_2O$) helps in removing colours, tastes and odours from water. It may be added to the treated water in the distribution pipes, but its main advantage is obtained by adding it to the open reservoirs or lakes. *The chief function served by copper sulphate in the reservoirs is to kill the algae or to rather check the growth of algae, even before its production.* These algae, if permitted to complete their life cycle, or if killed after attaining growth, give off oils and other decomposing products which impart highly disagreeable tastes and odours to the water. The use of copper sulphate at the proper time will help in preventing the growth of such algae, and thereby keeping the water free from the bad tastes and odours likely to be caused by them.

The usual doses of copper sulphate vary between 0.5 to 0.65 mg/l. Such quantities, if added to the treated water in the distribution pipes, will not make the water unfit for domestic or industrial purposes. *The solution of copper sulphate may be prepared and added just at the entry of water into the distribution system.*

Since copper sulphate may sometimes prove detrimental to fish life, it is necessary to work out carefully its dose requirements for reservoirs and lakes; keeping in view the concentration required for killing the types of algae present and also the tolerance limits of fish present in that water. While applying

copper sulphate to the lakes or reservoirs, the crystals of copper sulphate may be grounded to powdered form, which may be sprinkled over the surface of the reservoir by means of a power operated rotating disk. In another method of application, the copper sulphate crystals may be placed in porous bags which may be pulled behind a boat being towed through the reservoir water. The boat may be moved along parallel paths, 6 to 10 m apart, or along two sets of parallel paths that are at right angles to each other. In still another method, the solution of copper sulphate of the required strength is prepared, and is sprinkled over the surface of reservoirs from a moving boat.

9.27.4. Treatment with Oxidising Agents. The colours, odours and tastes from the water may also be removed by oxidising the organic matter responsible for them. The oxidising agents commonly used are potassium permanganate, chlorine, ozone, etc. The usual doses of potassium permanganate may vary between 0.05 to 0.1 mg/l. Chlorine also helps in removing the organic matter, provided sufficient doses (*i.e.* beyond break point, or super chlorination followed by dechlorination) are used.

Chlorine dioxide gas, ozone, etc., may also be used as oxidising agents for obtaining good tasty water, but their use has not been found economical, as yet.

9.28. Removal of Salt and Dissolved Solids from Water ; i.e. Desalination

The water which contains common salt or sodium chloride dissolved in it, has a peculiar salty or brackish taste, and therefore, it is named as *salt water* or *brackish water*. Since the salt waters have a brackish taste, they cannot be used for drinking, unless the salt content is removed or reduced. The process of removing this salt from water is known as *desalination,* and the resultant water which is free from salt is known as ***fresh water***.

9.28.1. Classification of Salty Water. Although ***desaliation*** strictly means removal of salt, yet since common salt (NaCl) cannot be removed in isolation, this term is practically used to describe process, which is used for the reduction of total dissolved solids (TDS) in water, which is measured in mg/l. Sometimes, conductivity is measured and used as a measurement of total dissolved solids, as it can be easily measured in the field with a conductivity meter. The S.I. unit for conductivity is *milli Siemens per m*. For most purposes, the TDS value of a water can be obtained by multiplying the conductivity value by 0.66, except for certain special cases, *e.g.* mine waters.

In broad terms, natural waters can be classified according to their TDS values, as follows :

Table 9.11. Classification of Salty Waters

S. No.	*Type of water*	*TDS value in mg/l*
1.	Sweet waters	0—1000
2.	Brackish waters	1000—5000
3.	Moderately saline waters	5000—10,000
4.	Severely saline waters	10,000—30,000
5.	Sea water	Above 30,000

9.28.2. Occurrence of Salt-Water. Sea water is the most evident example of salt water. One can taste this water while sailing on a sea and one will immediately reject it saying that the water is highly salty. ***Sea water averages 3.5 per cent salt.*** Land-locked lakes without known outlets are found to be more salty than the oceans. This is because they are more shallow than the oceans. For the same surface area, the evaporation of water will be more for a shallow lake than for a deep ocean ; and, therefore, the salt concentration of a lake will be more than that of an ocean. Such confined bodies of water are termed as salt-lakes. The famous "Great Salt Lake" in Utah city of USA is about seven times more salty than the oceans, as it contains about 23% salt.

The sea, and great salt lakes are not the only places where we can find salt water. Salt water can also be found in rivers that pass over salt beds and rocks, containing large amounts of salts. Water of the Gila River in Arizona contains about 2% salt.

9.28.3. Need for Utilization of Brackish Waters. Salt water is not usable because of its high salt content. We may need water for drinking, for irrigation, for industrial utilization, etc. Depending upon the need, the salt content of the water must be removed or reduced. A very pertinent question that one can ask will be as to "why do we want to use salt water or sea water, when fresh water is available in rivers and streams ? The answer is two fold. One point, is that fresh water may not be available in *sufficient quantity*, and the second point is, that the fresh water may not be available at the required place, and importing fresh water may either prove to be costly or may not be feasible due to political or other considerations.

By 'Sufficient quantity', we mean the quantity sufficient enough to meet all our demands, including domestic demand irrigation demand, industrial demand, etc. With the development of civilization, our water demand is going on increasing, while the supply remains almost the same. Therefore, it becomes evident that the available fresh water will become scarce in future, even if sufficient water is available today.

There are various places in the world where water is in great scarcity even today. Hong Kong, for example, faces acute shortage of fresh water, and one has to pay 10 cents for a glass of drinking water. An ordinary Indian, if happens to face this situation, will definitely call it a 'robbery'. But in fact this is not a robbery, because water has to be treated or imported in Hong Kong, which makes it so costly. Hong Kong is a fine natural harbour and is surrounded by sea. If this sea water can be converted into fresh water at a cheap rate, the major problem of Hong Kong can be solved.

Hong Kong is not the only country facing water shortage. At least sixty of the under-developed countries belonging to the United Nations, are now facing, or will soon face water problems that cannot be solved by using naturally available fresh water.

Even in our own country, shortage of fresh water is creating serious problems in certain thickly populated areas, such as Chennai (Madras), Kandla, and Greater Calcutta. In certain other areas, such as greater portions of Kathiawar and Kutch, shortage of water appears to be a major obstacle in settlement and development.

Shortage of water is one of the causes of the conflicts between Israel and the Arab states. The point of conflict is, as to how much water each country may take from the River Jordan, that flows between their boundaries. Tremendous areas of these countries are covered with deserts ; the Negev Desert covering about two-thirds of the country of Israel, and the Sahara Desert covering about 11.5 million sq. km of the Arab states. Israel can very easily obtain salt water from the Gulf of Aquba, and if Israel could find a way to desalt this water inexpensively, she could reclaim the nearby Negev-desert. She could also provide water for her industries and to the people living in cities like Tel Aviv, whose population buys distilled water at a very high rate. Finding new cheaper sources of water can always help us in improving the living standards of the people residing in deserts.

9.28.4. History of Desalination. The problem of an abundance of sea water that cannot be used, and shortage of usable fresh water is as old as man. This problem attracted the attention of the modern man, when he was travelling in ships. Whenever, men went in a ship, they always loaded water along with the cargo, so as to use it for their personnel needs. This water carried by them became short at many times, causing troubles.

One can easily imagine the fate of the sailors who were adrifted at sea, the whole stock of fresh water was exhausted, and in the end died of thirst, crying for water :

Water, water everywhere !
Not a drop to drink

The panic of those sailors who had been ship-wrecked and set afloat on rafts or life boats can be best quoted in the verse given above. Today, however, small kits have been invented that enable sailors to convert salt water into drinking water.

Ships too, these days, find it more practical to covert sea water into fresh water than to set aside cargo or passenger space to carry fresh water. *In fact, a desalting unit takes up less space than the fresh water it produces in one day.*

9.28.5. Cost Aspects of Desalination. Desalination is a costly process. The water obtained by desalination proves much costlier than the naturally available-treated water. Some years ago, the cost of desalination was too high to be feasible. The cost has been reduced considerably during the past few years, but it is still very much higher than the cost of treating natural water for public use. With modern equipments, the cost of producing fresh water from sea water has been reduced to about seven times, but still it is about three to ten times costlier than the cost of treating natural water. *What a tremendous price gap to close ?*

At present, desalting is not used for irrigation, because the cost of conversion is too high to make it practical to produce very large amounts. The greatest cost factor in producing desalted water is the fuel needed to run the huge stills. The expenses of building, equipping and maintaining plants, and labour charges are also considerable. All those expenses have to be counted while evaluating the cost of each litre of water.

Intensive research is being conducted on desalination, in order to reduce its cost. Even in India, some leading research organisations, such as Nagpur

Research Institute, National Chemical Laboratory (Poona), and the Central Arid Zones Research Institute, (Jodhpur) are busy on this subject. U.S.A. no doubt is the leading country in this field.

Even inspite of its high cost, desalination, has been used at various places. Numerous commercial de-salting plants are functioning these days, around the world. Many other privately owned smaller plants for industries are also in existence. U.S.A. itself is having about 200 commercial plants. Some islands too, do have their own desalting plants. Israel having many such desalting plants functioning to their full capacity, still finds herself amidst water-shortage. Singapore has also constructed a desalination plant at a cost of US$ 200 million (₹ 1120 crore) to treat 135 ML/day of sea water.

9.28.6. Economics of Desalination. Desalination is not a problem of pure economics but is a problem of applied economics. It is a socio-economic project, and the high cost and expenditure involved on such a project, which may not he justified from purely mathematical stand point, has to be sometimes borne in such a case. What we mean to say is that, when such a project is needed for the survival of the society, then it must be carried out even if it is costlier. *The question of cost has to be kept aside when there is a question of life and death.*

Upto now, man has been paying a low price for suitable water. But if the water has to be obtained after desalination, it may prove to be costly. And one may have to pay higher price for water in future, as the need for exhausting sea-water on a large scale arises. However, water is an essential commodity without which man cannot survive, and one will have to purchase it, at all costs, Desalination must be visualised in this light. In other words, even if desalination is economically not defensible, it is a means of survival, and hence has to be adopted.

9.28.7. Methods of Desalination. The various methods which are generally adopted for the conversion of salt water into fresh water are enumerated below :

(1) Desalination by evaporation and distillation ;

(2) Electrodialysis method ;

(3) Reverse osmosis method ;

(4) Freezing process ;

(5) Solar distillation method ; and

(6) Other methods.

These methods, their merits and demerits, are explained below in details :

(1) **Desalination by evaporation and distillation.** To turn salt water into fresh water by "distillation" is not a new discovery. Two thousand years ago, Greek sailors used to get fresh water from sea water by distilling sea water on ships, in their crude 'stills'—nearly ball shaped glass flasks with long bent necks. They used to boil sea water in their stills, turning it into water vapour, which was then caught in cooled flasks. The vapour was, thus, condensed to obtain pure water.

Water obtained by this process is the purest form of water which man can think to produce on any large scale.

Modern distillation plants are operated on the same principles that the Greek sailors used. Sea water is boiled in giant stills called *evaporators*, and the water vapour is caught and condensed into fresh water. The water produced is nearly pure and can be used for industries and other needs, provided its high cost is tolerated.

A modern salt water 'evaporator' works on the principle of a 'still', but is more efficient. In its simplest form, it consists of a metal box in which the salt water (generally called brine) is heated by a nest of pipes, carrying very hot steam. Heat passes from the steam through the pipe walls and boils the brine. The boiling brine evaporates and the vapour is led into a second box, where another nest of pipes, filled with cold brine, condenses it to fresh and pure water. The heat coming out from the vapour, during cooling operation, warms the cold brine, which is then sent to the evaporator.

This method of evaporating water has been used, and is used till today on oceans, by sailors. This kind of evaporators, generally called ***marine evaporators***, work well, but poses the problem of 'scaling'. When the sea water is heated in the metal box evaporator, layers of hard crust get quickly deposited on the metal due to deposition of certain chemical compounds. The hotter the water, and faster it forms. This ***scale*** is found to consists of lime stone, chalk and plaster of paris. It is as hard as rock and cannot be removed easily. In olden days, it was a serious trouble to the ship engineers, and the only answer was to keep some extra evaporators, and to employ labour to scrape and to chip off this scaling. The problem of scale has been reduced these days by modern techniques in new evaporators. Sometimes, the brine is chemically treated before it is heated up in the evaporator, and sometimes the evaporator tubes are lined with a chemical substance called *teflon*, which is said to give better heat transfer, and so makes the water to boil faster, and the scale to cling less tenaciously. Scale, however, is still a problem and needs further research.

Since the need for converting sea water into drinkable water has been increasing, it has become imperative to find out means and ways to reduce its cost. The research conducted on this subject has led to the development of various new methods for cutting down the cost of distillation. Essentially, these methods involve cleverer ways to use the same heat again and again, or ways to boil water at lower temperatures, thus reducing the cost of fuel.

Water can be boiled at low temperatures, by reducing the pressure. Thus, hot brine can be vaporised to a certain extent at a certain pressure, colder brine left can then, also, be vaporised further at still lower pressures and so on. This principle is used in 'multi-stage' evaporators, *i.e.* water is forced to evaporate and condense again and again in various stages. These modern evaporators throw away very little heat energy. They use it again and again by 'staging' the evaporator, until it is so low in temperature that it is too week to be useful. These techniques have led to a 16-fold improvement in efficiency.

A multi-stage evaporator, producing fresh water from salt water, is sketched in Fig. 9.53 (*a*). In this evaporator, important use is made of condenser vacuum by enabling the whole operation to go on at low temperatures and below or near the atmospheric pressure. By dividing the process into stages, the heated brine can evaporate some of its water, which

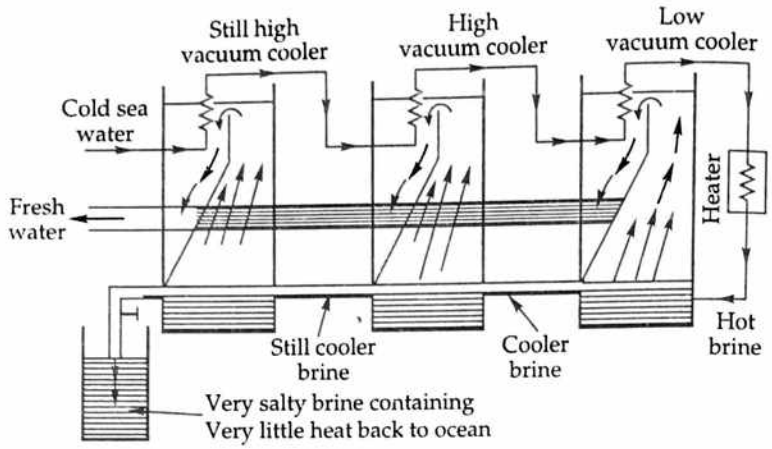


Fig. 9.53. (*a*) Multi-stage evaporator.

is condensed into another compartment. The colder brine goes to another compartment, where pressure is still less, thus causing the evaporation of some of it, which is also condensed and collected. The process is repeated until the vacuum is as high as is practically feasible, and all possible evaporation has taken place. Finally, when the brine becomes so strong and temperature and pressure so low that it cannot be evaporated and the remaining heat cannot be extracted, then it is thrown into the oceans. Only this little heat is wasted in this method.

An important advancement was made in this field, when the ideas of '*flash evaporation*' and '*multiple-flash evaporation*' came into existence.

Flash evaporation simply means the effect produced when hot salt water is suddenly cooled under less pressure. In this way, water is vaporised suddenly like a flash and the actual boiling point is never reached.

One form of flash evaporation consists in spraying the hot brine under pressure through a nozzle into a chamber which is at a lower pressure and is cooler. The sudden change produces the flash action and the water will evaporate immediately. The produced vapour is taken to a condenser to form pure water. This happens in each stage in a 'multi-flash stage evaporator'.

Many variations of flash process such as 'vapour compression', and various other improved methods such as **'long tube vertical method** (LTV method)', etc. have been used at certain places. But the question of finding a perfect evaporator is still unsolved. Perfection is difficult to achieve but still improvements are going on.

A photoview of a 5 stage 150 ML/day sea water flash distillation plant installed at Ras Abu Fontas Qatar in 1999, is shown in Fig. 9.53 (*b*).

(2) **Desalination by Electrodialysis method.** We are all aware of the fact that in a salt-water, H_2O molecules are bonded together with the sodium and chlorine ions. These hydrogen-bonds between the H_2O molecules and Na^+ and Cl^- ions must be broken up, in order to separate the salt from water. These bonds were broken by heat in the "method of distillation" ; while in the "electrodialysis method", these bonds are broken with the help of electricity.

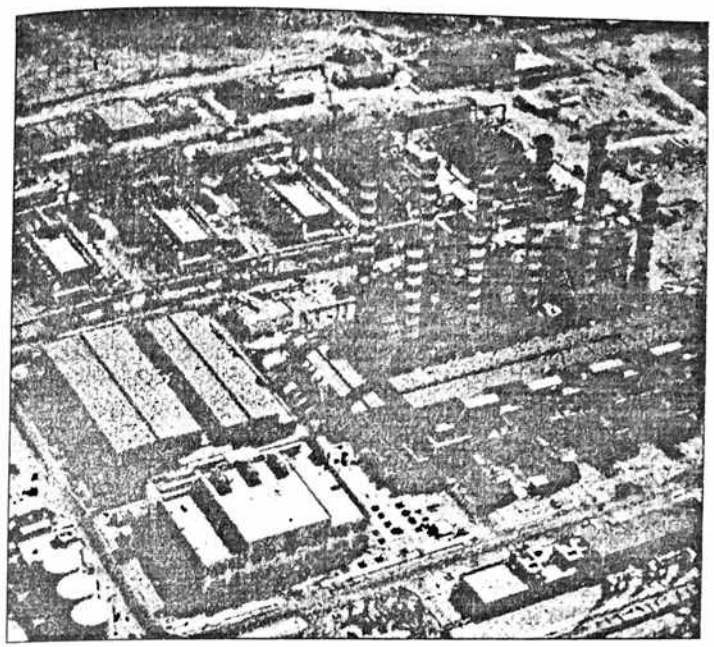


Fig. 9.53. (*b*) Photoview of a combined 150 ML/day five-stage seawater flash distillation plant installed at Ras Abu Fontas, Qatar in 1999. (Engineers : Mott MacDonald, Brighton UK. Contractors : Weir Westgarth and ABB, UK).

If an electric-current is passed through the salt-solution, the sodium and chlorine ions get freed from water molecules, and they start moving towards their oppositely charged electric poles. In other words, *the + vely charged* sodium ions will move towards the – ve pole, *i.e.* cathode, and the – vely charged chlorine ions will move towards the + ve pole, *i.e.* anode. If these cations (+ ve ions) and anions (– ve ions) are allowed to segregate in different compartments, what is left, is fresh-water.

The segregation is achieved by means of thin plastic like sheets called '**membranes**'. They are made of peculiar chemical substances called 'ion exchange resins' and they are very very thin, say about $\frac{1}{180}$ th of a cm. These thin membranes can be economically made and have the ability to accept or reject certain molecules in solution.

By choosing the right kind of resin, membranes could be made that would pass either positive or negative ions but not both. Fortunately, it has been possible to manufacture such membranes which conduct electricity enough, and have reasonably long life.

A simple sketch, using a single cell, on this principle, is shown in Fig. 9.54 (*a*).

Such individual cells consisting of a pair of cation and anion membranes installed in parallel between a pair of electrodes is known as a *cell* (Fig. 9.54 *a*). When a number of membrane pairs are installed parallel between a pair of electrodes (with spacers between each membrane), what is formed is known as a *stack*, as shown in Fig. 9.54 (*b*).

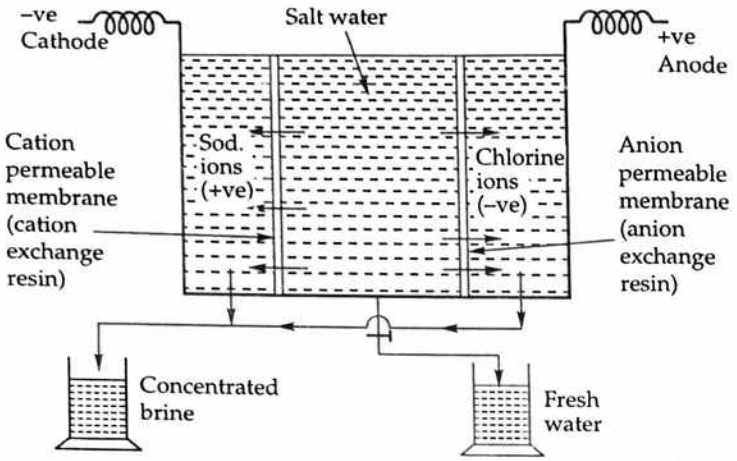


Fig. 9.54. (*a*) Principle of electrodialysis showing one cell.

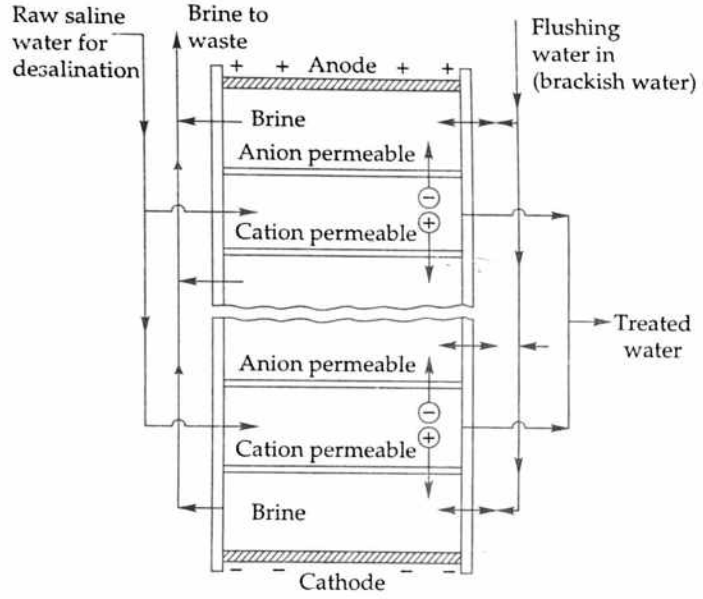


Fig. 9.54. (*b*) Principle of electrodialysis showing a stack.

For a big plant, using the above principle, therefore, stacks are hydraulically connected together, usually in parallel (although sometimes in series, also), as shown in Fig. 9.54 (*c*). In such a plant, therefore, a large number of small separations are hooked up together in parallel ; and alternate kinds of membranes are stacked with thin spacers in between each pair of opposites. Pores give excess at each level, so that salt water could be forced in between the membranes, while less salty water could be pumped out at the other side, which can be recirculated through other stacks of membranes. Since one stack of membranes usually removes 50% salinity, very fine water could be produced in this manner, using three to four stacks.

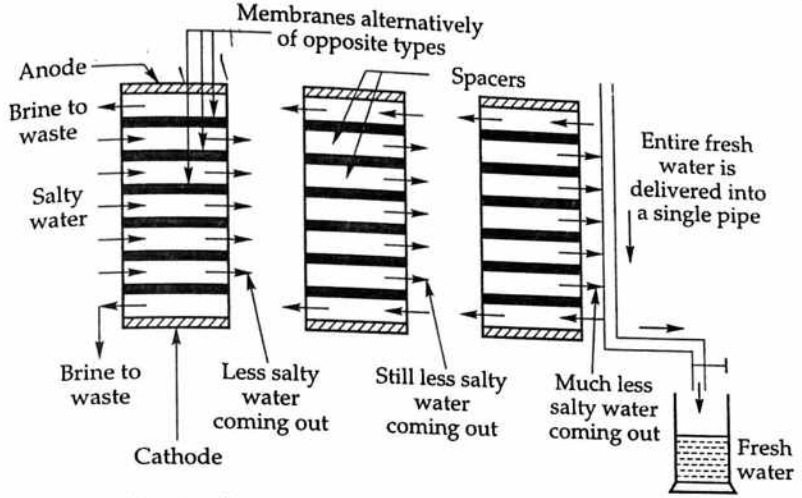


Fig. 9.54. (*c*) Line sketch showing stacking of membranes in a desalination plant working on the principle of electrodialysis.

Merits and Advantages of Electrodialysis method are :

(*i*) When too much of purification is not required, this method is the best of all the available methods. For moderate desalting problems varying between 500—3000 mg/l (TDS), there is nothing that can beat the electrodialysis method.

(*ii*) It is a compact machine.

(*iii*) The cost of buying and erecting the plant is small.

(*iv*) It is easy to operate. It can be started and stopped in a few minutes.

(*v*) This is an ideal machine for purifying water in a small town or at a remote place, where water obtained from wells may be contaminated with calcium, iron, fluoride, arsenic, and other unwanted ingredients, that spoil the taste and cause health hazards. All such minerals and metals can be removed together by this method of desalination, which infact, causes **demineralisation**, *i.e.* removal of all minerals from water.

(*vi*) They can be assembled for different outputs just by changing the number of units added together.

(3) **Desalination by Reverse osmosis process.** In this method of desalination, the water molecules and the salt ions are separated by *forcing* the salt solution against a 'semi-permeable membrane' barrier, which permits the flow of water through itself but stops the salt, as shown in Fig. 9.55 (*b*).

We know from our knowledge of chemistry that in ordinary osmosis when salt solution is separated from pure water by a semipermeable membrane, the pure water flows across the membrane until the pressure on the pure water side become equal to the osmotic pressure of the salt solution, as shown in Fig. 9.55 (*a*). But in

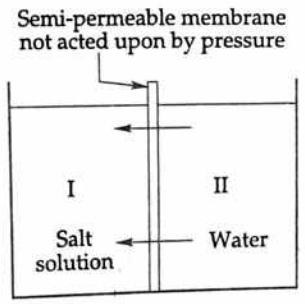


Fig. 9.55. (*a*) Osmosis.

reverse osmosis process, the natural osmotic pressure is opposed by exerting an external pressure on the side containing the salt solution. In other words, the osmotic pressure is reversed, and this external high pressure working in opposition to nature's low-osmotic pressure, forces pure water from the salt solution to move across the membrane towards the side containing water (Fig. 9.55 *b*).

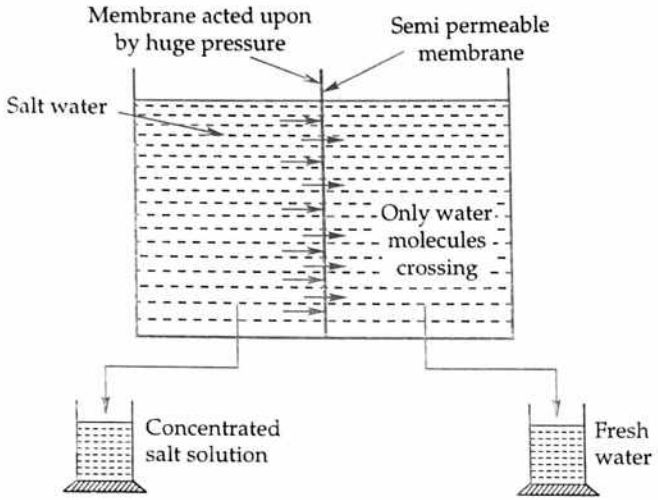


Fig. 9.55. (*b*) Principle sketch showing Reverse Osmosis.

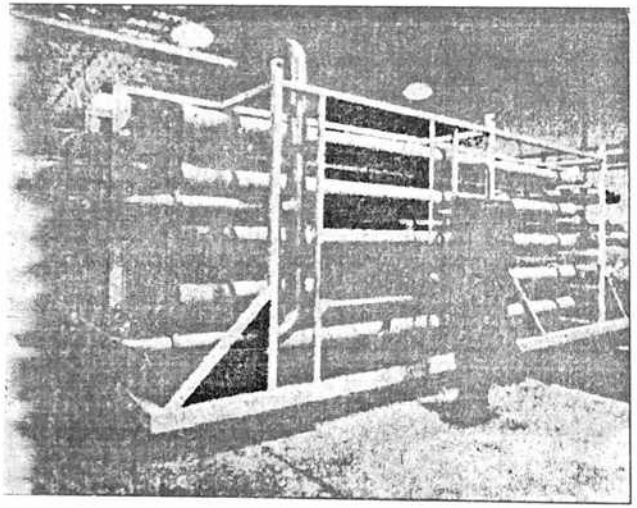


Fig. 9.55. (*c*) Photoview of a 11 ML/day Reverse Osmosis desalination plant installed at Kill Devil Hills, North Carolina USA in 1989. (Engineers : Black & Veatch USA).

The **osmotic pressure** is proportional to the TDS of the water, and a pressure of atleast twice the osmotic pressure is required to achieve an economically feasible flow. The semi-permeable membrane used in this process is hence thin but dense and strong enough to withstand the high external pressure. It is supported by a grid, and the salty water circulates against one surface of it. This surface has a thick and a tough skin, while the body of the membrane is softer and less dense. Reverse osmosis does not work below 60,000 kN/m^2 (600 kg/cm^2) and is usually operated at about 1,00,000 kN/m^2 (1000 kg/cm^2).

As a matter of fact, the semi-permeable membranes, like the ones found in numerous numbers in human body, have been used for separating the materials according to their physical and chemical properties, when a pressure differential or electrical potential difference (as in electrodialysis) is applied across the membrane. Pressure driven processes can be broadly classified according to the membrane pore size and size of particles removed, as shown in Fig. 9.56. These processes are :

(*i*) *Micro filtration* (MF)

(*ii*) *Ultra filtration* (UF)

(*iii*) *Nano filtration* (NF) ; and

(*iv*) *Reverse osmosis* (RO).

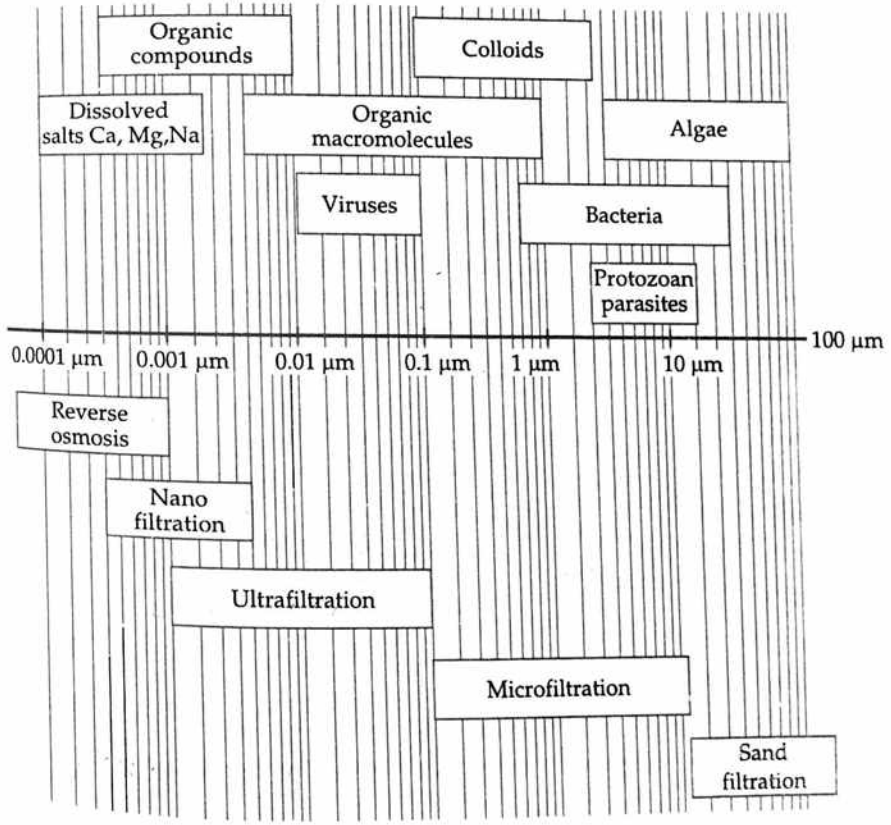


Fig. 9.56. Various types of pressure driven filtration processes

It can be observed from Fig. 9.56 that *nano filtration* as well as *reverse osmosis* helps in filtration of dissolved salts. Hence **nano filtration** (NF) is considered to be a part of reverse osmosis (RO), and is known as **low-pressure reverse osmosis**. Both these processes *i.e.* NF and RO help in removal of ions (dissolved salts) by osmosis, while Micro Filtration (MF) and Ultra filtration (UF) are micro porous membranes which remove suspended small size particles by physical separation. MF and UF are, therefore, low pressure processes, while NF and RO are high pressure processes. In reverse osmosis, separation is by diffusion through a dense membrane. Nano filtration follows the same principle but utilises a micro-pore membrane. RO rejects particles of size as small as 0.0001 μm, whereas NF rejects particles of size greater than 0.001 μm (nano meter).

Reverse osmosis can be used for desalination with low pressure membranes (as Nano filtration) of moderately salty waters (TDS of 1000 to 10000 mg/l); and with high pressure membranes for severely saline waters containing TDS above 10000 mg/l. Sufficiently good quality water containing TDS within 500 mg/l, can be obtained by this method.

Use of reverse osmosis for desalination is gaining a lot of popularity these days, as a large number of such commercial plants have been set up in North America and Middle East countries. A photoview of 11 Ml/day desalination plant based on R.O. process installed in North Carolina USA, as late as in 1989, is shown in Fig. 9.55 (*c*).

Due to acute shortage of water for drinking as well as for industries, India is also now feeling the necessity of desalinating its sea water, especially in coastal cities like Chennai (Madras), Kanda, Bhavnagar (Gujarat), etc. Desalination is, therefore, receiving a lot of attention even in this country from various research institutions, particularly the Central Salt Research Institute Bhavnagar, to find out new means and ways for reducing the capital and RMO cost of desalination plants. Bharat Heavy Electricals Ltd (BHEL) has been able to develop a compact desalination unit, which can desalt sea water at a reduced rate, on the basis of reverse osmosis process. But still, however, the cost of desalination is quite prohibitive as compared to the cost of treating ordinary surface supplies. So much so that the desalted water is still costlier by about 100 times than the treated surface water supplies. Say for example, here in Delhi, the cost of drinking water, drawn and supplied (with proper treatment) from Yamuna river, works out to ₹ 0.30 per kilolitre as against the general cost of ₹ 30.00 per kilolitre of desalted water.

(4) **Desalination by Freezing process.** This method is based upon the principle that when salt water freezes, the ice formed in the beginning is almost free from salt. This ice, when melted, can give us good water. A plant using this principle has been built in United States at Wrights Ville beach, North Carolina. The quality of water obtained is satisfactory, but the cost of production is high and prohibitive.

(5) **Desalination by Solar distillation method.** This is an ancient method but remained ignored for many years, because it seemed to be an impractical method for providing large quantities of water at reasonable cost.

Solar heat is a free heat, and everyday the sun vaporises billions of tonnes of sea water and lifts the vapour into clouds. If this vapour can be caught and distilled to form pure water, we can definitely get cheaper water.

The solar energy has been used in vaporising the water of a 'huge still'—a pond or a shallow basin covered with glass or plastic covers. It is a costlier process and the investigations are still going on, in order to make it cheaper.

(6) **Other methods of desalination.** A new chemical method is under investigations. In this method, propane gas is allowed to combine with salt water under controlled conditions of temperature and pressure. A chemical reaction takes place between salt water and propane gas at temperatures higher than the freezing point of water, forming ice like crystals. These crystals reject the salt and accept only pure water in their composition.

These crystals are separated from the brine, washed and decomposed to form water. The propane gas released during the decomposition of the crystals can be re-used for forming new crystals. This seems to be a promising method.

Various other methods are also under investigations. A lot of research is going on in this field throughout the world, and let us hope that a day will soon come when we will be able to convert sea water into useful water at a very moderate cost; and the huge quantities of about 155 billion cubic kilometres of open oceans will be at our disposal to get good and wholesome water.

9.28.8. Selection of a Particular Method of Desalination. As stated above, several methods have been developed for desalination of high TDS waters, and the selection of the correct process, especially in the 500—5000 mg/l range (TDS), requires careful evaluation of process efficiency, plant capital and running cost. As a general guide, the most frequent application of the various desalination processes has been in the following categories of TDS waters (Table 9.12).

Table 9.12. Choice of Desalination Process for Various Grades of Salty Water

S. No.	*Desalination process*	*TDS value of salty water to be treated in mg/l*
1.	Ion exchange	Up to 500
2.	Electrodialysis	500—3000
3.	Low pressure Osmosis including nano filtration	1000—10000
4.	High pressure *Reverse osmosis*	10,000 and above
5.	*Distillation*	Above 30,000

Out of all the desalination methods described above, *Distillation* and *Reverse osmosis* have infact, been largely used on commercial basis throughout the world. So much so that in terms of installed capacity, almost 75% of all desalination installations are based on *Distillation*, with largest concentration of plants in Middle East and North America. In recent years, however, *Reverse Osmosis Systems* have also been devcloped for desalting sea water, and the installed capacity of such plants is steadily increasing.

9.29. Removal of Iron and Manganese from Water

Iron and manganese salts are generally found dissolved together in well water or anaerobic reservoir waters, in invisible dissolved state. When exposed to

air, these reduced forms slowly transform to insoluble visible oxidized ferric iron and manganic manganese. When their contents exceed about 0.3 mg/l, and 0.05 mg/l respectively, they become objectionable due to the following reasons :

(*i*) they cause discolouration of clothes washed in such waters due to deposition of red/brown coloured oxides of iron/ manganese ;

(*ii*) they cause incrustation of the water-mains due to deposition of ferric hydroxide and manganese oxide ;

(*iii*) they make the water unpleasant in taste ;

(*iv*) the reduced iron in water promotes the growth of autotrophic bacteria in the distribution mains. Periodic flushing of small distribution pipes may be effective in removing accumulations of rust particles. However, elimination of iron bacteria is generally difficult and expensive.

The iron and manganese may be present in water either in combination with organic matter or without such combination. When present *without combination with organic* matter, they can be easily removed by *aeration*, followed by coagulation, sedimentation and filtration. During aeration, the soluble ferrous and manganous compounds present in the water may get oxidised into insoluble ferric and manganic compounds, which can be sedimented out easily.

On the other hand, when iron and manganese are present in *combination with organic matter*, it becomes difficult to break the bond between them and to cause their removal. However, when once this bond is broken, they can be removed as above. This bond may be removed either by adding lime, and thereby increasing the pH value of water to about 8.5 to 9 ; or by adding chlorine or potassium permanganate.

Manganese zeolite, a natural green sand, coated with manganese dioxide, can also be used for removing soluble iron and manganese from solution. After the zeolite becomes saturated with metal ions, it can be regenerated by back washing with potassium permanganate.

9.30. Addition to and Removal of Fluorides from Water

Fluorides in drinking water, as stated earlier, must neither be totally absent nor should exceed an optimum value of about 1 mg/l. To ensure this, fluorides are added to waters found deficient in fluoride concentrations, under a process known as *Fluoridation*. When, however, the fluoride concentration in a given water exceeds the limiting value of 1 to 1.5 mg/l, the fluorides are removed from water under a process known as *Defluoridation*. Both these processes are described below :

9.30.1 Fluoridation. For decades, it has been believed that when fluoride concentration in water supply is less than an optimum value of about 1 mg/l, the water proves harmful, as it may result in *dental caries* in children. It has, thus, been believed that during the formation of permanent teeth in children, scarcity of fluoride in consumed water, may lead to formation of weeker tooth enamel leading to early tooth decay. It had also been widely suggested that fluoride also proves beneficial to older people in reducing hardening of the arteries ; and as fluoride stimulates bone formation, it is helpful in the

treatment of osteoporosis. Due to such age old beliefs about the advantages of fluorides, its presence in public water supplies has been ensured by the addition of extra outside dose of fluoride, wherever the water was found to be deficient in fluoride concentration. It is believed that optimum benefits of fluoride are obtained, when fluoride concentration is achieved at about 1 mg/l.

This addition of external fluoride to public water supplies is known as fluoridation. Today, over 250 million people, drink over the world, artificially fluoridated water ; which practice is largely adopted in Western Countries. In the U.S. alone, for example, over half of the water supplies are fluoridated, since the natural waters in this region are usually found to contain fluoride concentrations to be much below the optimum level of 1 mg/l.

Most recent research has, however, suggested that fluoridation does not make a very significant difference in the incidence of tooth decay (USEPA, 1991). The primary explanation given being, is the present wide spread use of fluoridated tooth paste, fluoride supplements and better dental hygiene in the community. In any case, after the age of 12 years, calcification of teeth ceases, and fluoride provides no further benefits. Therefore, it does seem more appropriate to provide fluoride medication solely for children, rather than its mass-medication approach by fluoridation of public water supplies.

These considerations have now a days reduced fluoridation practices. In Western Continental Europe, the fluoridation has virtually been terminated. Even in U.K., at present, hardly 10% public supplies are being fluoridated (Deptt. of Environment, 1997). Northern Ireland is not at all fluoridating its supplies.

The fluoridation is achieved by the addition of outside fluoride compounds. The compounds which may be used for adding fluoride to the water are : sodium fluoride (NaF), sodium silico fluoride ($Na_2Si\ F_6$), hydro-fluosilicic acid ($H_2Si\ F_6$), etc. Of these compounds, **sodium fluoride** is most widely used, and is generally fed under pressure, as a solution, through a *chemical feed equipment*, shown in Fig. 9.57.

However, the dose of fluoride should be carefully worked out, because the presence of excess fluoride (above 1 to 1.5 mg/l) is also harmful, as to cause spotting and discolouration (*mottling*) of teeth. The long term exposure may result in permanent grey to black discolouration of the enamel (*dental fluorosis*). Children who drink fluoride in excess of 5 mg/l may also develop severe pitting of the enamel. Still larger and continued use of excess fluoride (30 to 50 mg/l) may lead to gastroenteritis, skin irritations, deformation of bones and other skeletal abnormalities. With the increasing exposure of children to fluoride, the advantage of water fluoridation may be out-weighed by the risk of fluorosis from over-exposure. Hence, fluoridation is increasingly becoming outdated and obsolete.

9.30.2 Defluoridation. Fluoride mainly enters the human body through drinking water. 96-99% of it combines with bones, since fluoride has affinity for calcium phosphate in the bones. Excess intake of fluoride can lead to *dental fluorosis*, *skeletal fluorosis*, or *non-skeletal fluorosis*. **Dental fluorosis** is characterised by discoloured, blackened, mottled, or chalky white teeth. Skeletal fluorosis leads to severe and permanent bone and joint deformations.

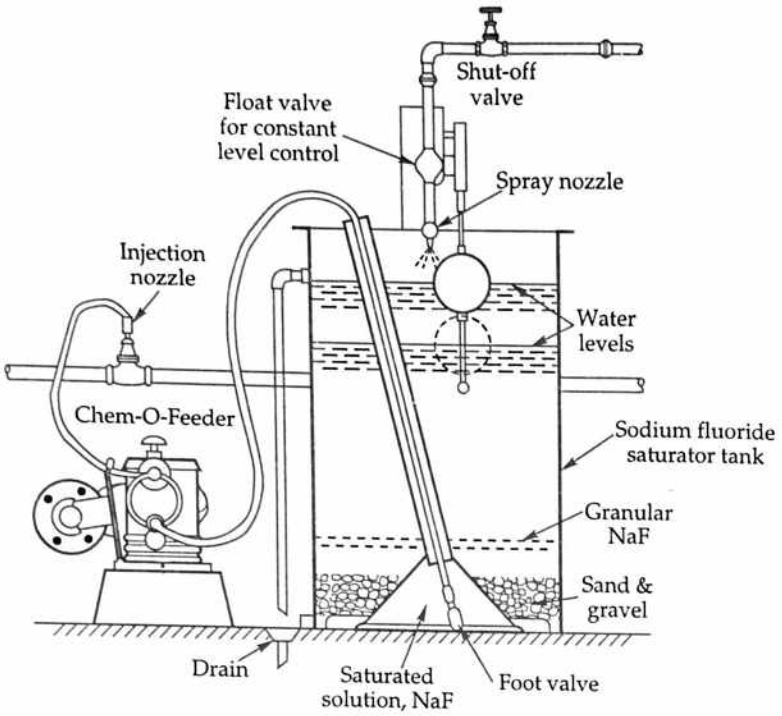


Fig. 9.57. Feeding arrangement for fluoridation.

Non-skeletal fluorosis leads to gastro-intestinal problems and neurological disorders. Fluoride can damage a foetus, and adversely affect the IQ of children.

Fluorosis can be detected in the neck, spine, knee, pelvis, shoulder and small joints of hands and feet, Gastro-intestinal symptoms include abdominal pain, diarrhoea, constipation ; while some of the neurological manifestations include nervousness, excessive thirst, and tendency to urinate frequently.

Inspite of all these symptoms, fluorosis commonly remains undiagnosed for a long time. Often, patients get misdiagnosed with arthritis, spondylitis, or non-specific backache.

A 2001 study about fluorosis notes down as under *"the occurrence of fluorosis can vary widely among different locations having almost the same fluoride concentrations in drinking water and can be affected by factors such as climate, individual susceptibility and biological response"*. This study also conclude that *poor nutrition also plays an important role in aggrevating endemic fluorosis*, thus explaining as to why poor people are often the worst affected. ***Fluorosis is infact an irreversible disease and it has no cure.***

Inorder to avoid contracting such a serious disease, it is necessary to provide drinking water to the people, which does not contain fluoride in excess of its permissible value of 1-1.5 mg/l (preferably 1 mg/l). However, when the available water supply contains fluoride in excess of this limit, it becomes imperative to remove the excess fluoride from the water before the same is

consumed by people. Whereas the public water supplies in western countries are found to be deficient in fluoride, the Indian water supplies, which are usually drawn through wells and tubewells, are found to contain heavy doses of fluoride. Since the ground waters in countries like India and Bangladesh are found to contain large quantities of fluoride, it becomes imperative to remove fluoride from water, whereever the waters are found to be polluted with excess fluorides, as to make the water safe for drinking. *The technique of removal of fluoride from water is known as defluoridation.*

9.30.2.1 Methods of defluoridation. The following technologies are generally used for removing fluorides from water :

(1) Absorption by *Activated Alumina (AA)*, Commonly known as *Prasanti Technology* ;

(2) *Ion Exchange Adsorption Method* ;

(3) *Nalgonda Technique* ; and

(4) *Reverse Osmosis Process.*

A comparison of these four treatment methods is given in Table 9.13, while the methods are individually briefly discussed below :

(1) **Prashanti Technique using Adsorption by Activated Alumina (AA).** In this method, the raw water containing high contents of fluoride, is passed (percolated) through the insoluble granular beds of substance like **Activated Alumina (AA),** or *Bone Char*, or activated *carbon*, or *serpentinite*, or *activated bauxite* ; which adsorbs fluoride from the percolating water, giving out defluoridated water.

Out of the above mentioned adsorption media, all except activated alumina, have limitations which would make the process uneconomical. Activated alumina is, however, found to be an excellent medium for removal of excess fluoride. It is highly selective to fluoride in the presence of sulphates and chlorides, when compared to synthetic ion exchange resins. In the presence of bicarbonates, although the fluoride level is reduced, yet the absorption capacity shows a major decline. Silica is known to interfere with the adsorption of fluoride.

The adsorption process is best carried out under slightly acidic conditions (pH = 5 – 7) ; the lower value of pH is more effective for its removal.

The activated alumina, after becoming saturated with adsorbed fluoride, can be cleaned and regenerated by back washing with 1% caustic soda solution (NaOH).

(2) **Ion Exchange Adsorption Method.** The ion exchange process is almost similar to that used for removing hardness from water, discussed in article 9.26.2. The process here, however, uses a *strong base anion exchange resin (zeolite) in the chloride form.* As the water passes through the bed of the resin contained in a pressure vessel (See Fig. 9.50), fluorides and other anions like arsenic, nitrates, etc., present in the water are exchanged with the chloride ions of the resin, thus releasing chlorides into water and adsorbing fluoride ions into the resin. The arsenic and nitrate ions also get removed in the process. When the resin gets saturated with anions like fluoride, nitrate, arsenic, etc. as indicated by their increased concentrations in the out flowing

Table 9.13. Comparative Table of Available Technologies for Defluoridation

Name of technology	*Developer and process*	*Advantages*	*Disadvantages*
1. Adsorption by activated Alumina (AA) Commonly known as Prasanti technology	• Satya Sai University for Higher Learning, Andhra Pradesh • Costs ₹ 35,000 at community level and ₹ 1,300-1,700 at household level, Recurring cost of AA replenishment • In this process, raw water is passed through AA which adsorbs fluoride, passing out defluoridated water	• Useful at community and household level • Filters can be attached with handpumps or standposts • Low sludge formation	• Cost is inhibiting for villagers • Regeneration of AA poses problem • Poor after sales service in villages
2. Ion Exchange Adsorption Method	• Ion Exchange (India) Ltd. • In this process, resin is used to adsorb fluoride from water • Capacity ranges from 500 liters per hour to 5,000 liters per hour	• Useful at community and household level • High fluoride removal efficiency	• Regular replacement of resin • Large amount of salts are involved in regeneration • Poor after sales service in villages
3. Nalgonda Technology	• National Environmental Engineering and Research Institute, Nagpur in 1975 • In this process, alum and lime is added to raw water and stirred for 10 minutes. Fluoride gets adsorbed by alum, and flocs are formed. These flocs are allowed to settle down for 90 minutes, after which water is filtered.	• Useful at community and household level • Cost effective • Indigenous technology • Low capital cost	• Technical expertise required • Time consuming process • Disposal of flocs is a problem • Alum dose needs regular calculation
4. Reverse	Osmosis • In this process, water is passed through a membrane which blocks fluoride flow, allowing only defluoridated water to pass. • 60-litre per day filter costs ₹ 20,000 • Annual maintenance costs about ₹ 3,000	• Different kind of membranes available • No chemicals added • High fluoride removal efficiency	• Cost inhibitive for villagers • Poor after sales service in villages

water, the same can be cleaned and regenerated with 5-10% sodium chloride solution (brine), and the bed is returned to service.

To ensure that a flow is maintained during regeneration, ideally 100% standby units should be provided.

During regeneration, the exchange process gets reversed, as the anions absorbed on the resin get replaced by chloride ions and discharged to waste water with excess chloride ions.

The capacity of a plant based on this technology may range from 500 l/h to 5000 l/h. Ion Exchange (India) Ltd. specialises in providing such treatment units. Although the method ensures high efficiency of fluoride removal (besides removing nitrates, arsenic, etc.), yet it requires regular replacement of resin, and large amount of salt (NaCl) for regeneration of resin saturated with fluorides. The method is hence found to be very costly and good "after sales service" in villages do not become easily available. The safe disposal of waste water from regeneration, containing high concentrations of toxic fluoride, nitrate and arsenic ions, etc., again poses serious problems.

(3) **Nalgonda Technique.** In India and particularly in rural areas, ground water containing excess fluoride is treated by *Nalgonda technique*. This technique is found to be simpler and economical than the above mentioned *fixed bed ion-exchange processes*, since it does not involve regeneration of media, and employs chemicals which are readily available, and easy to operate and maintain, using local skills. Nalgonda technique, not only helps in the removal of fluoride, but also helps in removing colour, odour, turbidity, bacteria and organic contaminants from raw supplies.

Nalgonda technique uses aluminium salt (alum) for removing fluoride. The raw water is firstly mixed with adequate amount of lime (CaO) or sodium carbonate (Na_2CO_3) and thoroughly mixed. *Alum solution* is then added, and water is stirred slowly for about 10 minutes, and allowed to settle for nearly one hour. The precipitated sludge is discarded, and the clear supernatant containing permissible amount of fluoride is withdrawn for use. The line diagram for the process is shown in Fig. 9.58.

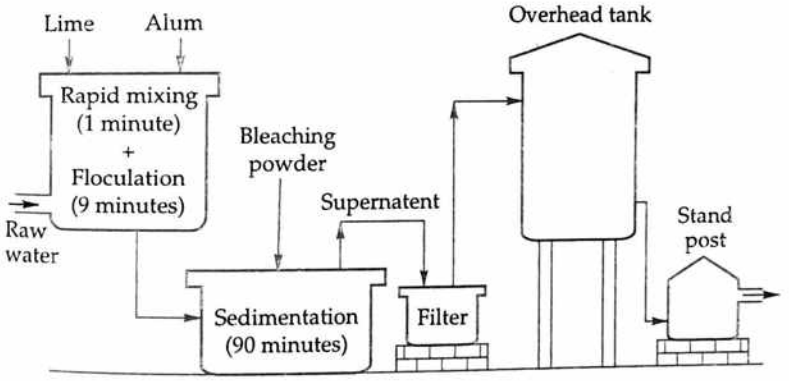


Fig. 9.58. Line diagram for Nalgonda Technology.

The added lime or sodium carbonate helps to ensure adequate alkalinity required for effective hydrolysis of aluminium salts, so that the residual aluminium does not remain in the treated water. Bleaching powder is also generally added with lime prior to the addition of alum, to achieve simultaneous disinfection of treated water, and also to keep the system free from undesirable biological growth.

Detailed description of plants based on this technology is given in article 12.6, under the chapter on Rural Water Supplies.

(4) **Reverse Osmosis Process.** In this method, the raw water is passed through a semi permeable membrane barrier, which permits the flow of clear water through itself and blocks the flow of salts including fluorides. This method is generally adopted for desalination for removing salt from water and has been thoroughly explained under 'Desalination' in article 9.28.7(3). *This method is however rarely used solely for defluoridation of village water supplies due to prohibitive high cost and poor after sales service of machines using such a treatment technology, although the method is capable to produce high fluoride removal efficiency, without the use of any chemicals.*

Choosing a particular defluoridation method. When once excess fluoride is detected in water, the obvious step is to defluoridate the same. The technology to be adopted depends upon the fluoride levels in water and the volume of water to be fluoridated. The various merits and demerits of the four methods described above, are to be kept in view, while choosing any one of them for adoption, since none are fool-proof. Machines based on principles of ion exchange and Reverse Osmosis are not only cost intensive but also need a lot of 'after sales service', which is difficult to be provided in towns and villages. Prashanti technology *i.e.* AA method may need frequent changes of defluorident beds and may not be able to reduce fluoride to non-toxic levels. *Defluoridation plants based on Nalgonda technique, though quite popular in rural India, have failed several times because of improper management and paucity of funds to meet the Running Operation and Maintenance Costs (RMO Costs).* Say for example, two defluoridation plants based on Nalgonda technique were installed in Gujarat in Tawadiya village and Thakarasen village of Paten District in 1988 and 1992, respectively. Thakarasen plant had a capacity of 60 M cum and costed around ₹ 15 lakhs. Its annual RMO cost was found to be around ₹ 1.83 lakh. The cost of treated water worked out to be ₹ 7.69 p/kL. The Tawadiya plant was set up for a capacity of 10 Mcum at a cost of about ₹ 3.5 lakh. Its annual RMO cost was found to be around ₹ 1.81 lakh, and the treated water cost was about of 8.26 p/kL. Since neither the local people could afford to meet such a huge cost for treated water nor the Govt. could provide enough funds and sufficient managerial personnel to run these plants, both the plants were shut down within a few months of their commissioning. Similarly, a few years back, Gujarat Water Supply and Sewerage board (GWSSB) also installed a number of defluoridation plants based on Nalgonda technique in Mehsana district at a huge cost of ₹ 12-15 lakh per plant. These plants were again shut down within a few months, due to lack of proper management and monitoring. Such failure of plants can, however, not be considered as cases of failures of Nalgonda technology, since these failures have occurred due to the negligence of the local govt. and municipalities. As a matter of fact, several such plants have proved to be quite successful, wherever local people and NGO's have come forward to run them. The case of Methan Village in Gujarat is a classic example. Here, Muniwar Abad Charitable Trust, an NGO, succeeded in involving the local community to share the RMO cost of such a plant, and hence could successfully run the plant for 4 years before shutting down the same in the year 2002, when

municipal piped water supply through Dharoi Surface Water Scheme became available to them.

The experiment of distribution of individual house hold kits to remove fluoride from their individual hand pump water in Gujarat has also again failed in Lathililiya village, since the poor village folk found the process to be too much time consuming.

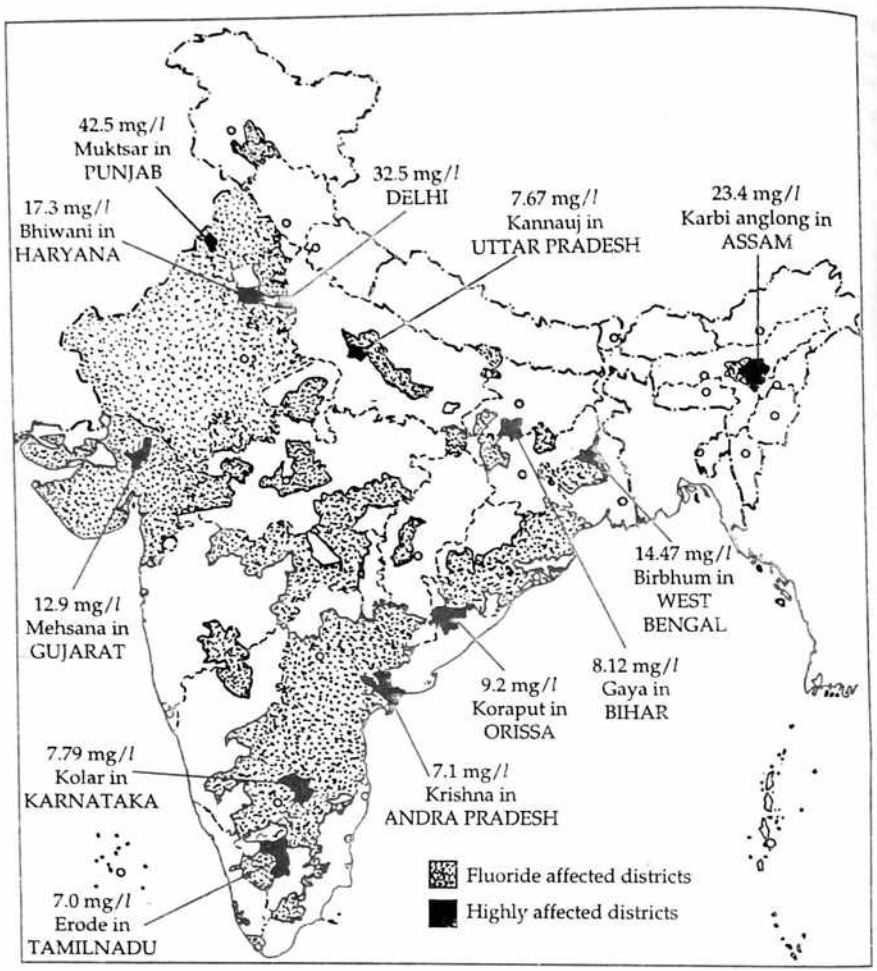
All such failures can, however, be avoided by concerted and honest efforts of the local Panchayat and municipalities. The State and Central Govt. also must not provide funds only for the plan developments of such plants, but should also provide enough Non-Plan funds to run these plants, besides providing sufficient managerial and engineering personnel, who should be made responsible for failures and pilferages.

9.30.2.2. The Fluoride Contents of Indian Ground Waters. The presence of fluorides in Indian ground waters is, infact, increasing over time, due to excess exploitation of ground water leading to excavation of deeper bore holes for drawal of ground water. In its bid to supply germ free water supplies, the municipalities in Indian towns and villages, have drilled deep tubewells to supply drinking water. The continuous withdrawal of ground water and reduced recharge due to development of proper drainage system for draining rain runoff, the ground water-table has been falling continuously almost every where in the country. Such deep tubewells are found withdrawing water containing excess fluorides, since the fluoride concentrations have generally been found to be increasing with depth in Indian aquifers, due to the presence of fluoride bearing minerals found present at deeper depths, although ofcourse, the presence of such fluoride bearing minerals will vary at different locations and will actually depend on the geology of the area and composition of its bed rocks.

A recent survey (1999) has shown that the **excess fluoride*** has been found to be present in ground water supplies of various towns and villages in 19 States of India. It has also identified 59,111 villages with fluoride levels above 1.5 mg/l. Fluorosis is, thus, found to be affecting a large rural population, consuming high fluoride water. About 65 million people including 6 million children are today estimated to be suffering from fluorosis. The areas found to be affected with excess fluoride in the country are shown in **Fig. 9.59.** The highly affected districts are shown in this Fig. in dotted area, which represents the presence of very heavy and harmful levels of fluorides in Muktsar in Punjab (42.5 mg/l), Delhi (32.5 mg/l), Karbi Anglong in Assam (23.4 mg/l), Bhiwani in Haryana (17.3 mg/l), Birbhum in West Bengal (14.47 mg/l), Mehsana in Gujarat (12.9 mg/l), Koraput in Orissa (9.2 mg/l), Gaya in Bihar (8.12 mg/l), Kolar in Karnataka (7.79 mg/l), Kannauj in U.P. (7.67 mg/l), Krishna in Andhra Pradesh (7.1 mg/l), Erode in Tamil Nadu (7 mg/l), etc. As a matter of fact, virtually more than half of the country is suffering from fluorides, in excess of BIS permissible value of 1 mg/l.

In the absence of development and exploitation of proper surface water sources, the ground water is being largely consumed in the country, without checking its safety for the presence of excess fluoride or even arsenic. There is

*Fluoride in excess of its safe limit of 1-1.5 mg/l or so.



- Fluorosis is endemic in 19 states of India. 65 million people, including six million children, are today affected.
- Fluoride levels in India's groundwater vary from 1 mg/1 to 48 mg/l. (The world health organization guideline for maximum permissible level of fluoride in drinking water is 1.5 mg/l)
- A 1999 study by New Delhi-based Fluorosis research and Rural Development Foundation has identified 59,111 problem villages with fluoride levels above 1.5 mg/l.
- A 1987 study by the Rajasthan Voluntary Health Association shows that almost 35,000 people in the state are consuming water having more than 10 mg/l of fluoride.
- A 2001 study by S.L. Choubisa of M.L. Sukhadia University in Rajasthan shows high prevalence of dental fluorosis in various districts of the state: 63.4 percent in Banswara 77.2 percent in Dungarpur, and 72.5 percent in Udaipur.
- A recent white paper prepared by the Gujarat Government, Water quality problems in Gujarat, shows that 2,826 villages have fluoride levels above 1.5 mg/l.
- Out of the total 1,152 villages in Andhra Pradesh, 624 have high fluoride in the drinking water.
- In village Tilaipani of Mandla district in Madhya Pradesh, 74.4 percent of children and adolescents below age 20 have dental mottling, and 51.2 percent are affected with genu valgum (knock kness).
- Groundwater surveys in Unnao district of up conducted by UP Jal Nagam (1992) and UNICEF (1997) have identified 500 problem villages with fluoride levels above one mg/l.

Fig. 9.53. Areas affected by Excess Fluoride in India.

no standard protocol to test fluoride in water, and there does not exist sufficient laboratories to periodically test waters of more than 40 lakh handpumps, which draw drinking water supplies in the country. The Indian government has failed to provide-and administer appropriate technology either to detect fluoride or to clean fluoride-ridden water. Emergent measures are needed to be taken to install proper treatment plants to treat ground water supplies, before they are supplied to public, to make them safe against the presence of fluorides.

Moreover, immediate measures to recharge ground water through developing water harvesting structures at the cost of Govt. supported welfare schemes, to improve water quality, need be initiated on a large scale. Besides, we must exploit surface water on a larger scale through construction of large dams, for its utilisation for drinking, even at the cost of deficient irrigation supplies.

9.31. Arsenic Contamination and its Removal from Waters

Arsenic is a metalloid element, brittle in nature and gray or tin white in colour. It is not found in nature as a free element, but exists in combinations with other elements, as compounds of oxygen, chloride, hydrogen, lead, mercury, gold and iron. There are as many as 150 arsenic-bearing minerals found existing in nature. However, only three of them are considered as arsenic ores (the amount of arsenic in each being very high). These three are : ***Realgar*** *i.e.* Arsenic di-sulphide ; ***Orpiment*** *i.e.* Arsenic tri-sulphide ; and ***Arsenopyrite*** *i.e.* Ferrous Arsenic sulphide. Presence of these arsenic ores in ground aquifers is likely to contaminate the ground water with arsenic toxicity.

Arsenic contamination has been noticed on a large scale in West Bengal in India as well as in Bangladesh. **Arsenopyrite** has primarily been identified as the main source of arsenic pollution in this belt.

Chemically, arsenic compounds are of two types ; *i.e.* (*i*) organic ; and (*ii*) inorganic. ***Inorganic arsenic is more toxic than organic arsenic.*** **Inorganic arsenic** is found in two forms ; *i.e.* (*a*) *trivalent arsenite* ; and (*b*) *pentavalent arsenate*. The trivalent arsenite is usually expressed by the symbol As (III) ; while the pentavalent arsenate is symbolised as As (V). As (III) usually occurs in soluble form under anaerobic conditions predominantly in ground waters ; while As (V) usually occurs in soluble form under aerobic conditions more commonly in surface waters. The ground waters of West Bengal, however, are found to contain both forms of arsenic.

9.31.1. Toxic and Ill Effects of Arsenic Contamination. The toxic effects of arsenic contamination are found to be somewhat erratic. To unacclimatised individuals, even small doses (as little as 100 mg) may produce serious adverse health effects ; whereas, even higher doses can be easily digested by persons who become acclimatised to consuming arsenic. **The maximum permissible limit (guideline value for arsenic in drinking water) specified by WHO as well as BIS is 0.01 mg/l (10 ppb).** Hence, when a water contains arsenic more than 0.01 mg/l (10 ppb), it may be termed as contaminated with arsenic.

Arsenicosis or Arsenic toxicity usually develops in human beings after two to five years of exposure to arsenic contaminated drinking water, depending on the amount of water consumed and arsenic concentration in the consumed water. Initially, the skin begins to darken (called *diffuse melanosis*). This happens first in the palms. Diffuse melanosis leads to *spotted melanosis*, when darkened spots begin to appear on the chest, back and limbs. Sometimes, this may be the early symptoms of arsenicosis. At a later stage, *leucomelanosis* sets in, wherein the body begins to show black and white spots. It is commonly found in persons who have stopped drinking arsenic-contaminated water, but previously had spotted melanosis.

Keratosis is the middle stage of arsenicosis. The skin, in portions, becomes hard and fibrous, as if the body has broken out into hard boils or ulcers. *Diffuse* or *nodular keratosis* on the palm of the hand or sole of the foot is a sign of moderately severe toxicity. Rough dry skin, often with *palpable nodules* on hands, feet and legs, means severe arsenic toxicity. This can ultimately lead to the formation of *gangrene* and *cancer*. Besides, arsenic poisoning may bring with it other health complications like *enlargement of liver and spleen ; cirrhosis of liver ; myocardial degeneration and cardiac failure ; peripheral neuropathy affecting primary sensory functions, diabetes mellitus* and *goitre*; *skin cancers ;* etc. Three types of skin cancers have been observed : *Bowen's disease* (form of squamous cell carcinoma) ; *basal cell carcinoma*, and *squamous cell carcinoma*. These skin cancers develop primarily from keratosis.

9.31.2. Extent of Arsenic Contamination Present in Ground Waters of India and Bangladesh. The ground water at various locations in West Bengal (India)—Bangladesh belt have been noticed to be contaminated with arsenic, much in excess of its permissible value of 0.01 mg/l. So much so that a study conducted in May 2002 by the Researchers of the School of Environmental Studies, Jadavpur University, Kolkata, reportedly found a very serious grim situation, as brought out below :

For Bangladesh

"The present situation is that in 2000 villages, in 50 out of a total of 64 districts in **Bangladesh**, ground water contains arsenic above 0.05 mg/l, and more than 25 million people are drinking (poisoned) water".

For West Bengal (India)

"2700 villages have so far been identified and more than 6 million people from 9 affected districts are drinking water containing more than 0.05 mg/l of arsenic. 3 lakh (0.3 Million) people may have visible skin lesions. Samples indicate that many more may be subclinically affected. Children in arsenic-affected villages may be in special danger".

As a matter of fact, the arsenic contamination of ground water in the region is spreading fast, day by day, since the ground water-table is going on depleting, causing more wells to spill out water contaminated with arsenic, because almost all the wells in the area tapping water from depth of 20 m to 100 m are found to contain excess arsenic ; whereas shallow wells tapping water from less than 20 m depth are found to be safe and clean because they contain mostly rain water or water flowing swiftly through the sediments.

Deeper wells in 20 to 100 m depth range, have, however, been found to be giving water containing excess arsenic, because of the presence of arsenopyrite layers in the underground aquifer strata at this depth range, as shown in Fig. 9.60.

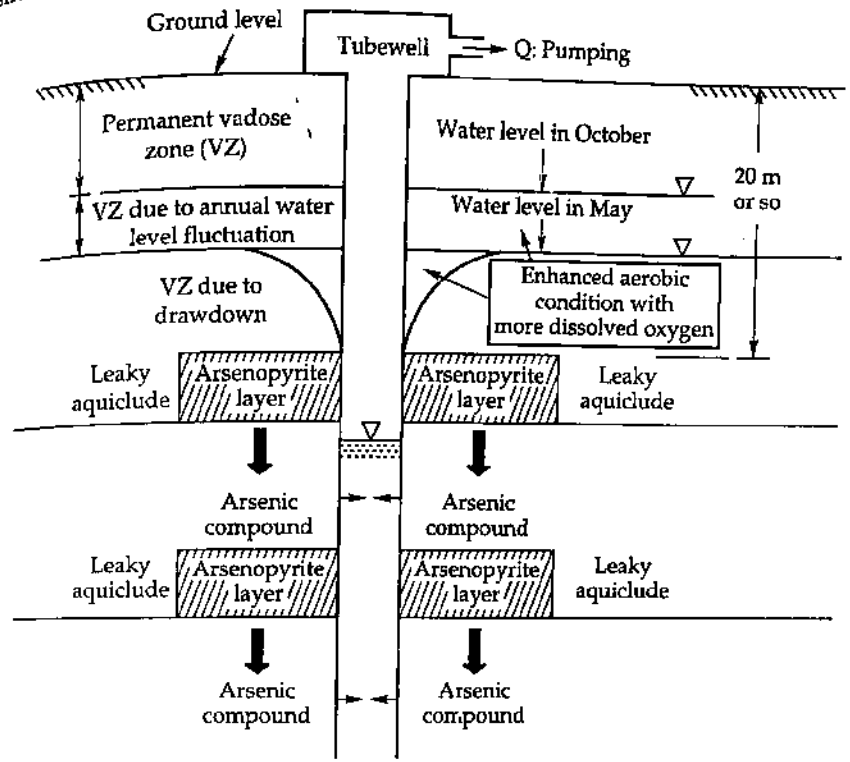


Fig. 9.60. Presence of Arsenopyrite layers in deeper aquifers in West Bengal—Bangladesh belt.

Source : Md Abul Fazal et al 2001, Extent and severity of groundwater arsenic contamination in Bangladesh, in Water International, Vol. 26, No. 3, p 376.

Prof. Dipankar Chakraborti, Director, School of Environmental Sciences, Jadavpur University, Kolkata has recently stated that the number of people at risk of arsenic poisoning from drinking water from sunken wells is infact considerably higher than previously thought of. Almost **330 million people may be at risk in India and Bangladesh** compared to 150 million, as believed earlier.

Recent investigations have further shown that the available water in the village of Semria Ojha Patti in Bhojpur district of Bihar is also found to be similarly contaminated with excess arsenic (25 times above the guideline value of 0.01 mg/l). This finding is greatly disturbing, and hence, investigation of ground water in the States of Bihar and U.P. have become imminent, because it is feared that the entire upper and middle stretches of Gangetic plains, might be affected by arsenic, being the extensive areas of Precambrian era (last 570 M. years) with geological formations having metamorphic rocks and granites with widespread *sulphide mineralisation*.

9.31.3. Removal of Arsenic from Water. No large scale efforts have so far been made in India or at international level to provide specific treatments for removing arsenic from public water supplies, because excess arsenic was usually not found present in public water supplies till about a decade back. As a matter of fact, it was only in 1998 that the international community finally accepted that the problem of existence of excess arsenic is there. The World Bank has therefore, announced an emergency three year programme to identify tubewells splitting arsenically poisoned water in Bangladesh and West Bengal in India. However, still there is no strategic plan to address this important public issue, and to install treatment plants for removal of arsenic. The problem is still being considered as a local problem, with local Govts or WHO not doing much to protect the poor village folk from drinking poison.

Technically, the arsenic from water can be removed by the following three methods :

(1) *Coagulation-precipitation technique by using Aluminium and Ferric Salts* ;

(2) *Adsorption technique by using Activated Alumina (AA), or Ion Exchange Resins ;*

(3) *Membrane technology* like *Reverse Osmosis* and *Electrodialysis.*

The first removal method is generally adopted for surface waters, while the others are used for ground waters. Certain other processes like **biological oxidation** and **photo-oxidation,** both in the presence of iron, are also being developed for removal of arsenic.

These techniques are briefly discussed below :

(1) **Co-precipitation or Coagulation-Precipitation Technique.** Both the forms of Arsenic *i.e.* As (III) as well as As (V) can be effectively removed by coagulation followed by sedimentation and filtration to separate the solid precipitates and the clear treated water.

As (V) form is removed with equal efficiency by using both aluminium and iron salt as coagulants at pH < 7.5. However, iron coagulants are found to be more effective than aluminium coagulants in removing As (V) at pH > 7.5 and also As (III). Alternatively, As (III) could be more effectively removed by oxidising it to As (V) by pre-chlorination followed by coagulation using either of the two coagulants.

One drawback of this method is that the arsenic gets deposited on filter sand, which requires frequent intermittent backwashing with caustic soda.

Lime softening when used for treating hard waters can also help in removing arsenic, since when this treatment is carried out at pH = 11 to ensure removal of entire magnesium, will also ensure removal of arsenic.

(2) **Adsorption by Activated Alumina and Ion Exchange Technique.** Ordinary Activated Alumina adsorption, as well as Ion exchange method remove only As (V), although a new grade of activated alumina has now been discovered which helps in removal of As (III) equally well. As (V) adsorption on activated alumina is affected by silicates (at pH > 7) and phosphate (at pH > 7). The effect of silicates (at pH > 5) and phosphate (at pH > 6) is much adverse on removal of As (III). Other ions which reduce the removal efficiency are

sulphates and bicarbonates. The optimum pH range for adsorption is 5.5—6.5, although satisfactory performance has been achieved at a pH of 7.5. Because of the effect of sulphate on arsenic removal, any pH correction should be made by hydrochloric acid instead of by sulphuric acid.

In **AA method,** the raw water containing arsenic is passed (percolated) through the insoluble granular beds of Activated alumina, which adsorbs arsenic from the percolating water, giving out water with largely reduced arsenic concentration. The activated alumina after becoming saturated with arsenic, can be cleaned and backwashed with raw water and 1% solution of caustic soda (NaOH). For this regeneration, the bed is backwashed to give above 50% bed expansion using raw water, which is then subjected to co-current regeneration using 1% caustic soda solution, and counter current reactivation using sulphuric acid, with co-current rinsing phase in between. Such a regeneration process can help to recover almost 80% of the bed capacity.

The spent regenerate waste water would of course be highly toxic, as it will contain heavy concentrations of arsenic. It should therefore be disposed of properly and carefully, after necessary treatment. Usually, this waste water can be treated by aluminium or iron salts to form insoluble arsenates, or by lowering the pH to 5—6.5 to form insoluble hydroxides.

The **Ion Exchange method** used for removal of arsenic is exactly the same as was described for removal of fluoride in the previous article. This method, of course, can be applied only to waters containing dissolved solids of less than 500 mg/l, and sulphates of less than 25 mg/l as SO_4^{--}. The process uses a strong *base anion exchange resin of the chloride form* at a pH between 8 and 9, which removes As (V). The exchange capacity of chloride resin to exchange its chloride anions with arsenic ions is usually low, but it does not exhaust rapidly, because arsenic concentrations in most ground waters are very low.

The spent or exhausted resin can be regenerated by 5—10% solution of sodium chloride (NaCl). During regeneration, the exchange process gets reversed, as the anions of arsenic absorbed in the resin get replaced with chloride ions. The arsenic ions are, thus, washed off in the spent regenerate waste water along with excess chloride ions. The regenerate waste water, containing very high concentrations of arsenic, can be treated before disposal with aluminium or iron salts to form insoluble arsenates, or by lowering the pH to 5—6.5 to form insoluble hydroxides.

(3) **Membrane Technologies.** Membrane processes used are *Reverse Osmosis* (RO) and *Electrodialysis Reversal* (EDR)—usually known as *Electrodialysis (ED).* These have been thoroughly discussed under "Desalination" in article 9.29.6 (3) and 9.29.6 (2). Both these processes help in virtual **demineration** of water, since these methods remove arsenic alongwith other dissolved salts, as the membranes cannot be selected for removal of ions of only a particular metal/mineral.

Membranes are also prone to fouling, particularly by dissolved organic material, iron, manganese and suspended matter. Also, since it is a concentration process, *sulphates* and *carbonates of calcium* may increase the levels in the concentrate, at which they begin to precipitate and *scale the*

membranes. Hence, a water which is to be demineralised, must be pre-treated to remove a substantial proportion of the *fouling contaminants*.

Additionally, acid and sequestering agents such as sodium hexametaphosphate may be added to avoid scaling. Precipitation of sulphate is prevented by limiting the ratio of product water to raw supply, which is normally in the range of 75—90%, depending on the sulphates and carbonates present in the water.

The waste volume generated in membrane processes is consequently high and can vary between 10—25% of the feed supply. This is a major drawback of the process for inland installation. The process may also be uneconomical, if extensive pre-treatment of the feed is required.

A comparative table of the advantages and disadvantages of the various removal methods is given in table 9.14.

Table 9.14. Comparative Table of Available Technologies for Removing Arsenic from Water

Process	*Advantages*	*Disadvantages*
1. Co-precipitation In this process, aluminium and ferric salts are added to precipitate arsenic in raw water. These precipitates, after settling down, are removed.	• Can be used at household and community level • Chemicals easily available • Low capital cost	• Problem of disposal of toxic sludge • Requires trained operators • Medium removal of toxic trivalent arsenic
2. Adsorption In this process, chemicals such as activated alumina, iron coated sand, ion exchange resins are used to adsorb arsenic.	• Commercially available • High arsenic removal efficiency	• Needs regular replenishment of chemicals • pH needs to be monitored regularly • Poor after sales service in villages • Sludge disposal is a problem
3. Membrane technologies In this process, raw water is passed through a membrane, which filters out arsenic. Some common technologies are reverse osmosis and electrodialysis.	• High arsenic removal efficiency • Low space requirement	• High initial investment • High operation and maintenance cost • Requires pretreatment of water

It may, however, be mentioned that no treatment units have so far been installed to remove arsenic from the waters of problem village areas, as it is a very costly and difficult exercise to install thousands of units at local village levels. The only remedial action to bring safety to such areas is to promote rain water harvesting, ground recharge and exploitation of surface water supplies on a large scale, as is true for waters containing high fluorides.

9.32. Removal of Radio-activity from Water

Dangerous radioactive substances, such as uranium, thorium, stronium, caesium, etc. from the wastes of the atomic reactors or from the atomic explosions, may sometimes pollute the water sources. Such pollutions are almost nil in a developing country like India at present, but may occur in future as more and more use of atomic energy is made in industry, research or medicines. The radioactive pollutions of water sources must, however, be controlled, even with the greater use of radioactive materials, by properly controlling their waste discharge. A lot of research is being undertaken for safely discharging these wastes, and thus, to prevent any major health hazard.

The radio-activity present in the water can be measured these days with the help of some very sensitive instruments and detectors. The permissible limits for radio-activity in water have already been given in chapter 8.

Radioactive materials, if present in water, can be removed *only partly* by coagulation and filtration. Only up to 80 to 90% removals are expected. *The radioactive substances, such as Stronium and Caesium, which are held in true solution in water, cannot be removed either by coagulation-sedimentation or filtration.*

The ***lime soda softening method***, however, is quite effective in bringing down the radioactivity present in water. With relatively high doses of lime-soda, such as 200 mg/l or so, up to 99 per cent of radioactivity can be removed. Similarly, the ***zeolite softeners*** are also very effective in removing radioactivity from water, especially if *vermiculite* zeolite (which is a complex silicate of magnesium, iron and aluminium) is used.

Research is also being undertaken to find out other methods of removing radioactivity from water. Certain promising methods which are studied for removing radioactivity from water are :

(*i*) By phosphate coagulation method

(*ii*) By electrodialysis method

(*iii*) By addition of clay materials to water

(*iv*) By addition of metallic dusts to water.

However, none of these methods has so far been developed to be used to any larger extent. In fact, the removal of radioactivity is generally not needed for treating ordinarily available supplies, and therefore, the subject remains more or less a topic of research, and not of much practical value.

9.33. Domestic Methods of Treating Water Supplies

In our homes or offices, we generally get treated municipal water supplies, which usually are safe and disinfected. However, the stored supplies in overhead or underground tanks are liable to get recontaminated and become unsafe. Even the direct piped supplies may often contain dirt and mud, making the water unhealthy and unaesthetic, besides being contaminated on some occasions due to faulty distribution system. Moreover, many a times, ground water is lifted by pumps and is used in our houses, which may contain sand, mud, or even bacteria or viruses. In order to obtain better quality safe drinking water, many households are now-a-days using their own water

treatment devices, like **filters** and ultraviolet treatment units like **Aquaguards**.

Sometimes, **boiling of water** is also adopted in houses, which needs proper boiling and is thus expensive and time consuming. Such boiled waters are also liable to recontamination during subsequent cooling, storage and handling.

Sometimes, **chlorine tablets** are added in domestic storage tanks to safeguard from the storage contamination. Apart from giving peculiar taste and odour, chlorine is a toxic chemical if consumed in excess. Hence, the use of chlorine tablets in our houses do require utmost care and caution, particularly because extended exposure to high levels of chlorine is carcinogenic.

The domestic filters and Aquaguard units need a more elaborate description, and hence are discussed below :

9.33.1. Domestic Filter Containers. Domestic filters are usually made of stainless steel in the form of two circular containers placed one over the above. The upper container's base fits in the rim of the lower container. The upper container is fitted with vertically standing *porous ceramic filter candles*, usually two to three in number, and is covered by a lid from the top. The water from the top or storage tank is filled in the upper container, which slowly gets filtered through the filter candles, and falls down in the lower container, which usually is provided with a water tap (bib cock) at its bottom to take out the filtered water. Bajaj stainless steel water filters in different sizes are easily available in the market. Such filters, however, take a long time in filtering water, and one filter may hardly suffice to meet even the drinking water requirements of a family. The candles also often get clogged and need to be frequently cleaned (once a weak or so). If not cleaned regularly, the filter candles may serve as the breeding ground for more bacteria. Such bacteria multiplying in the clogged filter may flow out along with the filtered water. Moreover, such filters donot trap viruses, which are too minute to be trapped by the filter candles, and hence such filters are claimed to be removing 98-99% contamination only. These filters are largely used in individual homes, and their efficiency is enhanced by adding chlorine tablets or chlorine solution to disinfect the water.

9.33.2. On-Line Filters. A variety of domestic filters fixed directly on a water tap (bib cock) are now-a-days available in the market, claiming *zero bacteria* water at the outlet. These filters (Fig. 9.61) use *granulated filter media, activated carbon,* and *crystals of silver iodide*. The filter media traps the suspended particles, while the activated carbon removes colour, odour and gases from the water. The silver iodide finally causes disinfection, as it is a very

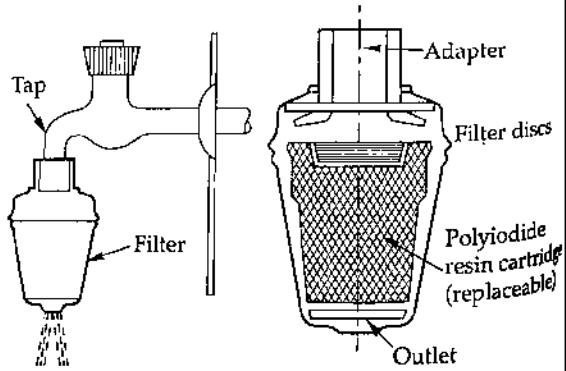


Fig. 9.61. Typical on-line tap filter.

effective oxidizing agent and is available in the form of crystals. Silver iodide therefore causes effective disinfection when water passes over these crystals. Silver iodide of course is costly, and may make such filters to be somewhat costlier.

The filter media provided in such filters may sometimes be in the form of a cartridge having a limited life, and hence needs regular replacement. Some of such filters may have some other filter media which may need back-washing.

Although such tap filters provide safe water, yet it becomes difficult for the user to know as to when the filter may cease to function efficiently and continued use of water from such filters may rather add to contamination hazards.

9.33.3. Domestic Ultra Violet Purification Systems like Aquaguards. As discussed earlier, ultra-violet (UV) rays generated from special electric lamps cause effective disinfection. Excellent sterilisation occurs when UV rays of sufficient intensity irradiate a thin film of water. When exposed to UV rays, most living cells (bacteria and viruses) either die or loss their capability to reproduce in the same water. However, ultra violet sterilization does not have a residual effect like that of chlorine, and is therefore considered to be an effective disinfectant when installed immediately before the point of use. The water should also be absolutely clear and free from suspended matter, as otherwise the penetration of U.V. rays in water gets affected. *That is why, in a U.V. purification system, the water is initially passed through a filter candle, then through a bed of activated carbon (for removing dissolved gases and foul odours), and the resultant clear and sparkling water is finally subjected to ultra violet rays treatment in a U.V. sterilizer* (Fig. 9.62). The rate of flow of water is controlled through the U-tube of the sterilizer (when it is exposed to U.V. radiation) to ensure complete disinfection. To prevent untreated water from passing through the system, an electrically operated automatic shut off valve is provided. Such a system or a unit or a machine requires water under good pressure, such as a supply from an overhead tank ; or water lifted by a pump

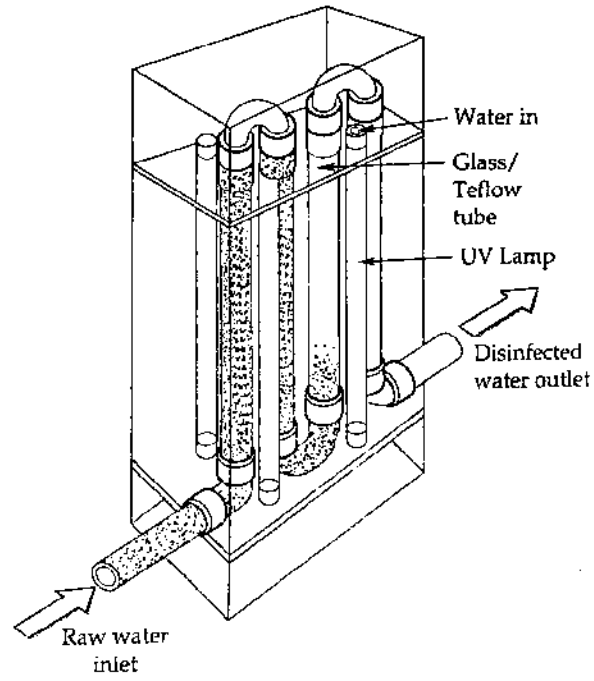


Fig. 9.62. U.V. Sterilizer.

from the underground tank, etc. It also requires electricity and regular maintenance, besides being a costly unit.

An **AQUAGUARD** is a patented device, which works on the above principle, and represents a marvel of high technology. This device eventually works on the principle of ultra violet treatment, and contains a prefilter unit, which helps to remove the physical impurities, such as dirt, mud, dust, etc. from the raw water moving through the filter-bed. After pre-filtration, the water in such a machine, is made to pass through a bed of activated carbon, which helps in reducing colour, odour and organic impurities from the water. Finally, the water is subjected to ultraviolet rays treatment, which helps in killing bacteria and viruses. The finally produced water is, thus, quite good and safe for drinking purposes.

Such a machine needs electricity, though the electricity consumption is low ; and needs water moving under some minimum head of about 4 m water depth (40 kN/m^2). In case such pressure is not available, a small tullu lift pump (to lift the water such as from the ground storage tank), will have to be installed in the line. These two additional requirements of this machine make it somewhat more complicated than the simple filter containers, which donot need any electricity or water moving under pressure, since the filter can be filled with a bucket. Such a machine also needs regular servicing (after two to three months) as its filter may need cleaning and activated carbon cartridge replacement.

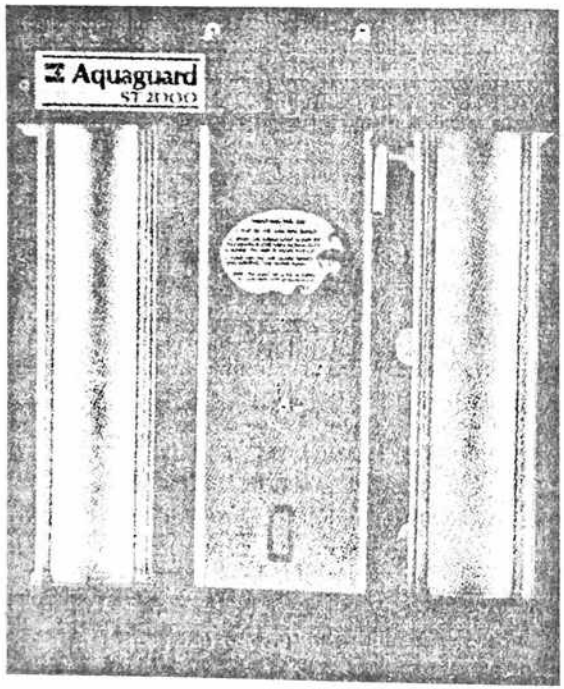
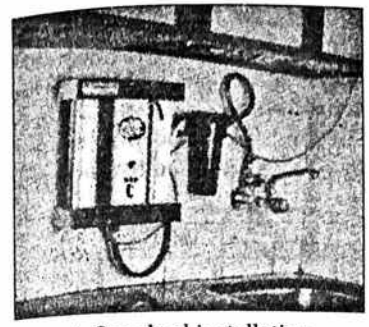
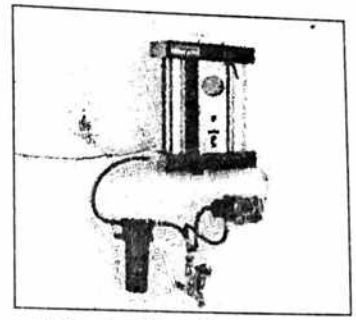


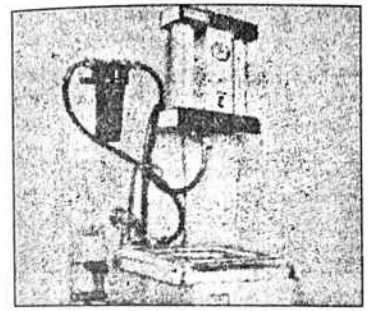
Fig. 9.63. (*a*) Photoview (front view) of an Aquaguard ST 2000 model.



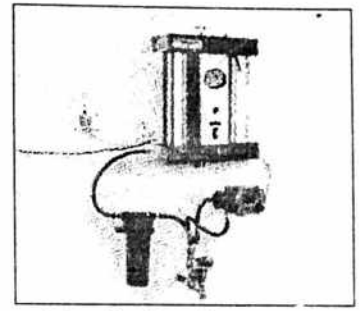
★ Standard installation



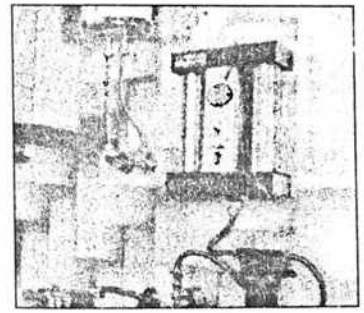
★ Standard installation with a pressure pump (for low water pressure areas)



★ Pre-Cooler installation



★ Standard installation with a pressure reducing valve (P.R.V.) (for high water pressure areas)



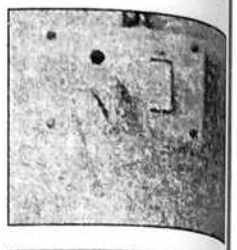
★ Hot and cold installation

Fig. 9.63. (*b*) Standard installation of Aquaguard near the source of tap water.

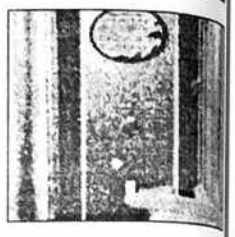
Such regular servicing is, however, easily provided by the company on annual service contract or on casual calls. Such a unit can however provide us enough quantity of water to serve our drinking, cooking and cleaning demands, easily.

A photographic view of a standard Aquaguard ST-2000 model is shown in Fig. 9.63 (*a*) ; while its complete standard installation on a wall is shown in Fig. 9.63 (*b*), use of Aquaguard, step by step, is shown in Fig. 9.63 (*c*). The unit will have to be switched to electricity supply of 220/230 V and 5 Amp point.

Step 1. Fit plug into the socket. (5 amp. plug)



Step 2. Click on the main red switch. The monitor will glow red for 2-5 seconds.

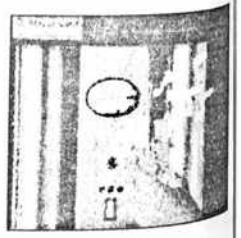


Step 3. Directly thereafter an amber light will glow for 60 seconds indicating the unit is processing water.

Step 4. When the indicator lamp glows green, a musical note is heard and the unit is ready to deliver clear, safe drinking water.

Step 5. Switch on the water control switch (silver) and you will find water flowing out of your AQUAGUARD ST 2000.

Step 6. While you fill your glass, a pleasant musical tune will be heard. Please note that music is only an indicator and has no relation to the purification process.



... Fig. 9.63 (c) contd.

Step 7. If the unit has not been used for a day or two, allow two to three glasses of water to flow out before using the water.

Step 8. If the unit is not to be used for the day make it a practice to turn main switch off and water supply off.

Fig. 9.63. (*c*) How to use your Aquaguard step by step.

The front and back view diagrams are shown in Fig. 9.64.

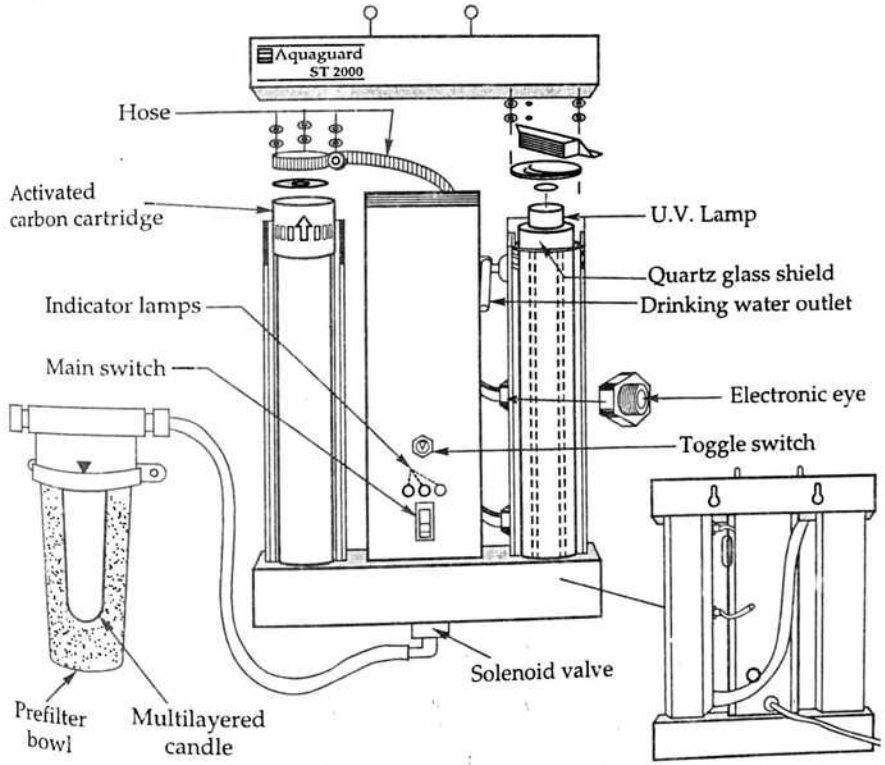


Fig. 9.64. Diagram of a Aquaguard ST 2000 showing front view and back view.

The technical specifications of this branded unit are given below :

Dimensions	:	360 × 100 × 300 (mm) ($H \times D \times W$)
Net Mass	:	4.75 kg
Ultra violet Lamp	:	8 watts.
Power rating	:	30 watts.
Voltage	:	160—270 V
Water pressure		
minimum	:	40 kN/m^2 (4 m head of water)*
maximum	:	200 kN/m^2 (20 m head of water)*
Fuse	:	0.25 Amp.

The above mentioned aquaguard ST-2000 model is being manufactured by:

Eureka Forbes Ltd.
Bhupesh Gupta Bhavan
85, Sayani Road
Prabhadevi Bombay
Pin : 400025

This model has the following four major components :

(*i*) *The Pre filter* ;

(*ii*) *The Activated carbon chamber* ;

(*iii*) *The Ultra violet (U.V.) chamber* ; and

(*iv*) *The Electronic monitoring system.*

These components are briefly discussed below :

Stage 1—The Prefilter. The water first passes through the prefilter, which incorporates a specially developed multilayered cartridge which strains out physical impurities present in the water, such as dust, dirt and mud.

Stage 2—The Activated Carbon. The water then passes through especially treated activated carbon, which reduces colour, odour, organic impurities and free gases like chlorine.

Stage 3—U.V. Chamber. The water then moves into a separate ultra violet chamber where it is subjected to the ultra violet treatment, which has been documented as a proven method for eliminating water-borne disease-causing bacteria and viruses. Infact the manufacturer of AQUAGUARD ST 2000 claims that its ultraviolet germicidal dosage, even at its lowest, is 1.2 times of that required to destroy all the known water borne disease causing organisms.

Stage 4—The Electronic Monitoring System. Finally, the Electronic monitoring system monitors the purification process and stops the flow of water immediately if the level of purification is inadequate.

AQUAGUARD ST 2000 unit is thus claimed to be an effective water treatment unit for domestic water consumption, as it works without the use of

*In case the water pressure in your tap water is lower than the specified, attach a tullu pump. If the pressure is higher than the max. specified (such as from very high storage tanks), a reducing valve is to be attached in the circuit. Both the above accessories are available with the sellers of aquaguards.

any chemicals and purifies water without adding anything to it, while detaining important natural salts and minerals in the water. It is time saving and economical, and is reliable with its electronic monitoring system. It is built with superior electronics and even has a built in-voltage stabiliser.

The method to backwash the filter unit, which is regularly required in this device, is shown in Fig. 9.65.

Regular backflushing of the prefilter will increase the life of the candle. However, the frequency of backflushing will depend on :

(*a*) the quality of the water in terms of physical impurities ; and

(*b*) the quantity of input water passed through the unit.

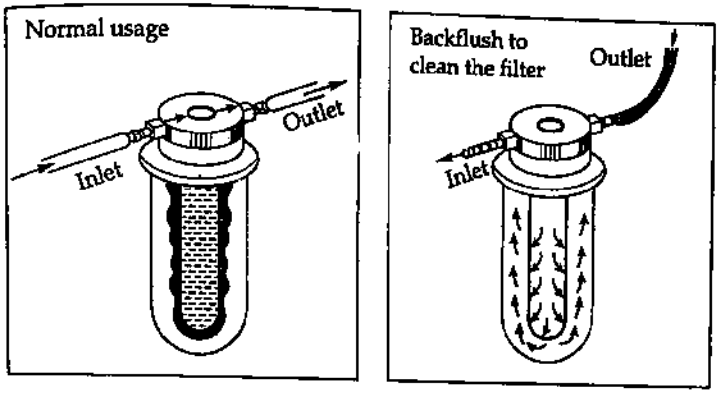


Fig. 9.65. Back-washing of Aquaguard machine.

How to Backflush the Prefilter

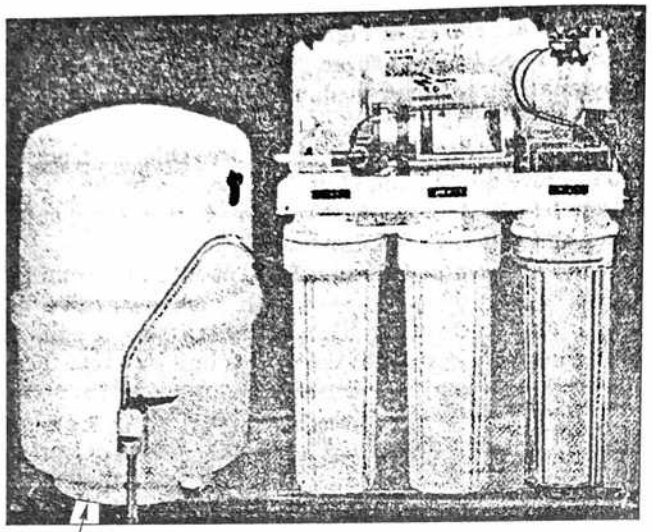
Disconnect the PVC tubes from the inlet and outlet sides of the prefilter (as marked).

- Connect the pipe from the tap to the outlet nozzle.
- Leave the other nozzle of the prefilter open.
- Turn the tap on to maximum pressure.
- Let the water flow into the sink for about 10 minutes.
- Reconnect the inlet and outlet nozzle as before. Do not by mistake interchange the inlet and outlet of the Prefilter, as this may clog the candle.
- You are now ready to use the AQUAGUARD ST 2000 once again.

AQUAGUARD ST 2000 has PVC Nozzles. These are easy to remove, thus making your task of backflushing simple.

9.31.4. Domestic Reverse Osmosis Filter Systems. Reverse-Osmosis (R.O.) process used for salination has also been used for purifying water in individual houses, by incorporating this technology in domestic filters, called **R-O Systems.** A photographic view of such a *homeline basic unit*, initially developed by a private firm, known as Kent R-O Systems of C-15 Sector-7, Noida, U.P.-201301, India, is shown in Fig. 9.66. Such a filter system helps in removing hardness from water (which cannot be removed by an aquaguard type of a filter) alongwith removing other impurities and bacteria. The rate of

production of filtered water, however, is on lower side, being only of the order of 7 litres/hr.



14 litre tank for storing purified water for availability of water on demand

Fig. 9.66. Photoview of the initially developed basic Reverse-Osmosis Domestic Filter System.

Features :

- Five stage water purification through Reverse Osmosis Technology employing a semipermeable membrane.
- Removes Salinity/Hardness/TDS from Raw Water, besides eliminating impurities and viruses.
- Enhances taste of water by removing undesirable salts.
- Even Tubewell/Hard/Municipal supplies/Overhead tank water is processed to best quality drinking water.
- 7 Lt./Hr. water production capacity.
- No consumable chemical is used and easy to maintain.
- Easy to install (even under the kitchen cabinet) with 14 litres water storage tank for availability of water on demand.
- Easy to operate with automatic control which stops the maker when tank is full and starts automatically when tank is empty.
- Pressure Boost Pump providing constant output and proper operations regardless of inlet water pressure.
- Auto Flush for minimising scaling of R-O Membrane and enhancing its life.

Stages in Purification :

The five purification stages involved in such a system are pointed out below, and are shown in Fig. 9.67.

(*i*) **Stage I—Sediment filter.** The filter uses 10 micron polypropylene filter to remove sediments to improve quality of water and protect membrane.

(*ii*) **Stage II—Pre-carbon filter.** It removes chlorine from raw/tap water to improve quality and to protect the membrane.

(*iii*) **Stage III—Carbon block.** It removes bacteria and suspended impurities and enhances colour/odour.

(*iv*) **Stage IV—Reverse-Osmosis (R.O.) membrane.** Semi purified water from carbon block is passed through a reverse osmosis membrane having 0.0001 micron pores. Dissolved salts, heavy metals and impurities are reduced by this unit by about 90%.

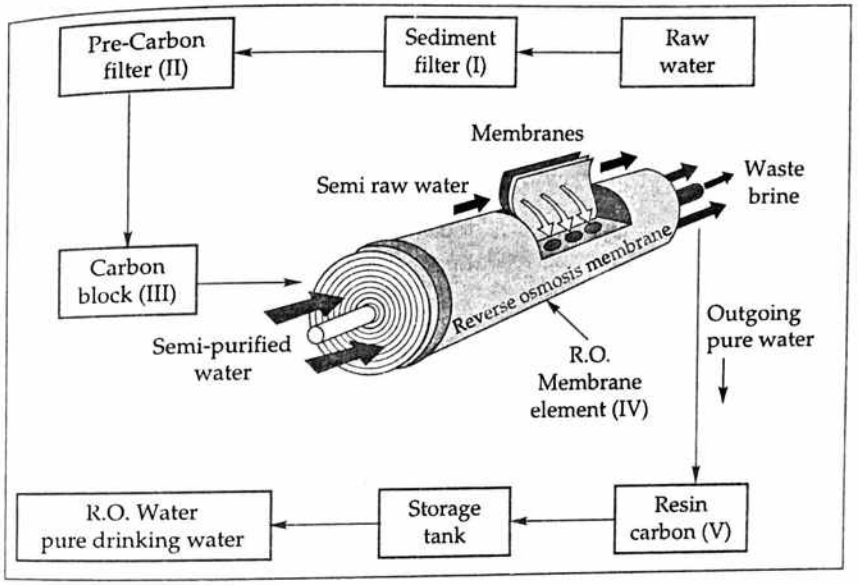


Fig. 9.67. Figure showing various stages involved in the purification process of a Reverse-Osmosis homeline filter unit.

(*v*) **Stage V—Resin carbon.** In this unit, the left over bacteria/ virus are trapped and destroyed, and all chances of their regeneration are removed.

Such a filter unit can be of good help in individual houses, where municipal supplies are not available, and the hard salty tubewell waters are to be used even for drinking and cooking purposes.

Uses :

- Switch from bottled water to Kent R-O Water.
- Purify Saline/Hard Water to get tastiest water.
- Process Municipal Water/Overhead Tank water to get purest and safe drinking water.

Specifications :

Model Name	:	**Kent R-O Water Maker-Homeline-Excell**
Dimensions	:	(54 × 48 × 21) cm

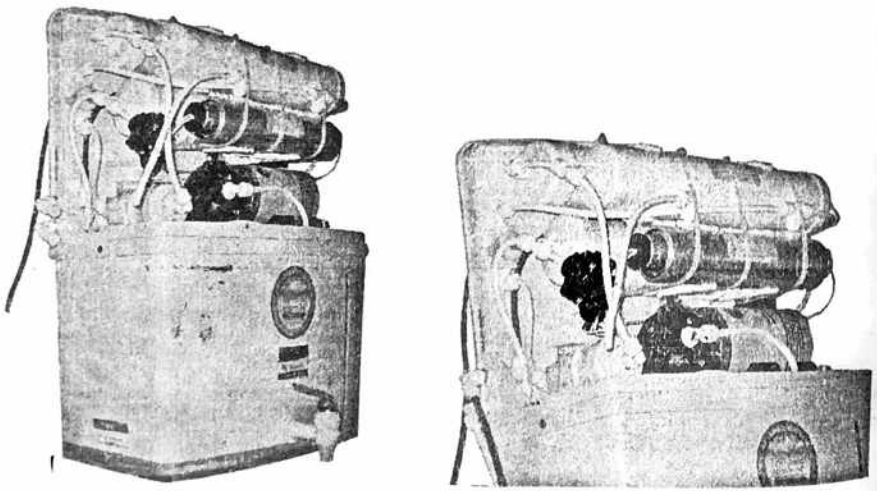
Water Tank	:	Hydrostatic 14 Lt. Plastic Tank
Purified Water		
Production Capacity	:	7 Lt./Hr.
Inlet Water		
Pressure (min)	:	0.3 Kg/cm^2
Voltage	:	20 ± 10% 1ph 50 Hz
Booster Pump	:	24 V Ac 350 mA (Inbuilt Transformer)
Filter Cartridge	:	Sediment, Pre Carbon, Carbon Block, Post Carbon, R.O. Membrane
Membrane	:	R.O. Membrane

Safety Protections :

Low Pressure Switch	:	Automatically shuts off, if raw water pressure is low.
Tank Switch	:	Automatically stops, when tank is full and starts when tank is half empty.
Flushing Switch	:	Automatically flushes the membrane for 1 minute, whenever the maker starts.

Improved models of the R.O. domestic filters, incorporating passage of water through an ***Ultra-violet (U.V.) chamber*** to completely kil! to pathogens and viruses, have also now been developed. The latest model, developed by Kent RO Systems Ltd, Noida, is known under the brand name of '***Kent Grand Plus Mineral RO***'. Such a model not only incorporates the U.V. treatment but also introduces a ***TDS control Valve***, which helps to regulate the amount of total dissolved solids (TDS) to be kept in the treated water.

Two photoview of such a model is shown in Fig. 9.68 (*a*) and (*b*) ; while the water flow diagarm is shown in Fig. 9.68 (*c*).



(*a*) (*b*)

Fig. 9.68 (a) and (b) Two photoviews of a Kent Gand Plus Mineral R.O. Filter.

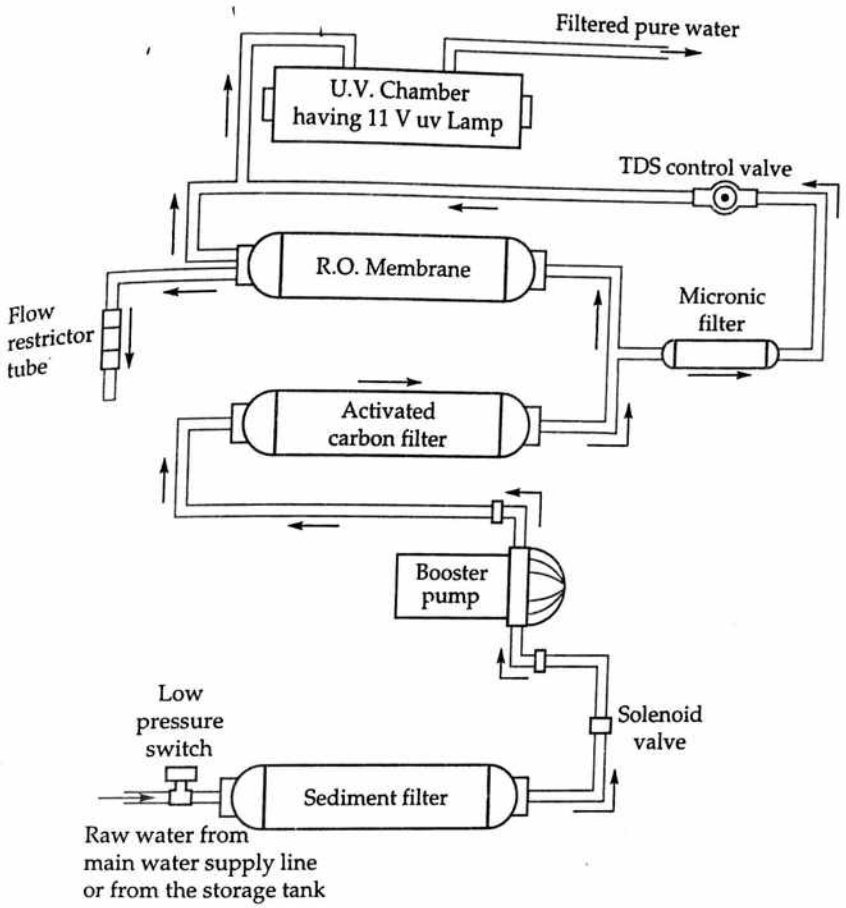


Fig. 9.68 (c) Water flow diagram of a Kent Grand Plus Mineral R.O. Filter.

A photoview of such a model is shown in Fig. 9.68 (*a*) and (*b*) ; while the water flow diagarm is shown in Fig. 9.68 (*c*).

9.34. Packaged Natural Mineral Waters

Packed mineral water in bottles and glasses has become a high sale item in modern society, since it is widely used, even in a developed country like ours, for drinking purposes, by travellers and in marriages and other parties on occasions of mass consumption.

The production of the packaged natural mineral water in India is guided by the specifications given in BIS code *IS : 13428 : 1998—Packaged Natural Mineral Water Specifications*. In practical life, however, most of the saleable brands of the mineral waters, usually, neither, fulfil these specifications, nor are they naturally available mineral waters.

Inspite of the fact that about 20 brands of packaged drinking bottled waters are available in India, there are virtually only one or two brands, like *Catch** or *Himalayan*, which are natural mineral waters, carrying the BIS certification mark under IS : 13428 for packaged mineral waters. All other important brands, including *Bisleri*, do not fulfil the specified standards of the BIS code of packaged mineral waters. Considering the non-availability of natural mineral waters in India, separate BIS standards for drinking packaged waters have been framed under BIS code, IS : 14543-1998 : *Packaged Drinking Water (Other Than Packaged Natural Mineral Water)*. Most of Indian bottled waters have now obtained BIS certification under this IS code.

The companies making ordinary bottled drinking water (other than the mineral water) can easily obtain BIS certification under BIS-14543.

The BIS standards for both the packaged mineral water as well as for ordinary drinking water (non mineral water) are, infact, legally enforceable in India under the *Prevention of Food Adulteration Act, 1954 (PFA)*, since packaged water falls under the definition of "food or a drink for human consumption other than drugs and water", which come within the preview of this Act (Section 2V of PFA). Since non-packaged water is not included within the preview of PFA Act, BIS Water Quality Standards given for ordinary potable drinking waters to be supplied by local municipalities or even by Private Tankers, as given in Table 8.10, remain voluntary in nature, and can not be legally enforced. Although pressure has been put on the Govt. of India to include drinking water supplies under the PFA Act as 'food', so as to make these BIS standards to be legally enforceable and binding, yet nothing tangible has so far happened. There stands an urgent need to bring potable drinking water supplies also within the preview of the PFA Act.

The BIS standards for packaged mineral water are described in this article, while those for non-mineral water are described in the next article.

9.34.1. Definition of Natural Mineral Water. According to BIS code 13428 : 1998, the natural mineral water may be defined as the *water which is clearly distinguishable from ordinary drinking water because :*

(*a*) *it is obtained directly from natural or drilled sources from underground water bearing strata for which all possible precautions should be taken within the protected parameters to avoid any pollution of, or external influence on, the chemical and physical qualities* ;

(*b*) *it is characterized by its content of certain mineral salts and their relative proportions and the presence of trace elements or of other constituents*;

(*c*) *of the constancy of its composition and the stability of its discharge and its temperature, due account being taken of the cycles of minor natural fluctuations* ;

(*d*) *it is collected under conditions which guarantee the original microbiological purity and chemical composition of essential components* ;

(*e*) *it is packaged close to the point of emergence of the source with particular hygienic precautions ; and*

*"Catch" brand of natural packaged mineral water is being sourced at Raison Springs, Kullu, H.P., India.

(*f*) *It is not subjected to any treatment other than those permitted by the BIS Standard* IS : 13428 : 1998.

Packaged Natural Mineral Water. Natural mineral water filled into hermetically sealed containers of various compositions, forms and capacities, that is suitable for direct consumption without further treatment, is known as packaged natural mineral water.

9.34.2. Treatment and Handling of Packaged Natural Mineral Water. The following guidelines are given :

(*i*) Treatments permitted include : *separation from unstable constituents*, such as, compounds containing iron, manganese, sulphur, arsenic by **decantation** and/or **filtration**, if necessary, accelerated by previous **aeration**.

(*ii*) Naturally available mineral water may be *naturally carbonated, non-carbonated*, or *decarbonated* or *fortified with carbon dioxide* from the source, on the condition that the mineral content of the water is not modified in its essential constituents, which give the water its properties.

(*iii*) The **transport** of natural mineral waters in bulk containers for packaging, or for any other process before packaging, is prohibited.

(*iv*) Natural mineral water may be **disinfected** to a level that will not lead to contamination of natural mineral water.

The *disinfection* in this context is defined as the reduction by means of chemical agents and/or physical methods, of the number of micro-organisms to a level that does not compromise food safety or suitability.

The natural mineral water shall be collected, processed, handled, packaged and marketed in accordance with the hygienic practices, specified in the said BIS standards.

9.34.3. Quality Requirements of Packaged Mineral Waters. The quality requirements of packaged natural mineral waters are given in tables 9.15 (*a*), (*b*), (*c*) and (*d*), which tables indicate the *physical parameters, general parameters relating to various substances, parameters concerning toxic substances*, and *parameters relating to radio activity residues*, respectively.

Other than these prescribed standards for various parameters, the packaged mineral waters should fulfil the *microbiological standards* shown in table 9.15 (*e*).

Table 9.15 (a) : Standards for Packaged Mineral Waters

Physical and Organoleptic Parameters

S. No.	*Characteristics*	*Requirements*	*Method of Tests Ref. to IS*
(1)	(2)	(3)	(4)
(*i*)	Colour, colour unit, *Max*	2	3025 (Part 4)
(*ii*)	Odour	Agreeable	3025 (Part 5)
(*iii*)	Taste	Agreeable	3025 (Part 8)
(*iv*)	Turbidity, NTU, *Max*	2	3025 (Part 10)
(*v*)	Total dissolved solids, mg/l,	150 to 700	3025 (Part 16)
(*vi*)	pH	6.5 to 8.5	3025 (Part 11)

Table 9.15 (b) : Standards for Packaged Mineral Waters

General Parameters Concerning Substances-Undesirable in Excess Amounts :

S. No.	Characteristics	Requirements	Method of Tests, Ref. to	
			Annex of this Standard (IS : 13428)	Other Standards
(1)	(2)	(3)	(4)	(5)
(i)	Nitrate (as NO_3), mg/l, *Max*	50	—	IS 3025 (Part 34)
(ii)	Nitrite (as NO_2), mg/l, *Max*	0.02	—	IS 3025 (Part 34)
(iii)	Sulphide (as H_2S), mg/l, *Max*	0.05	—	IS 3025 (Part 29)
(iv)	Manganese (as Mn), mg/l, *Max*	2.0	—	35 of IS 3025
(v)	Copper (as Cu), mg/l, *Max*	1.0	—	IS 3025 (Part 42)
(vi)	Zinc (as Zn), mg/l, *Max*	5	—	IS 3025 (Part 49)
(vii)	Fluoride (as F), mg/l, *Max*	2.0	—	23 of IS 3025
(viii)	Barium (as Ba), mg/l, *Max*	1.0	F	—
(ix)	Antimony (as Sb), mg/l, *Max*	0.005	G	—
(x)	Borate (as B), mg/l, *Max*	5	H	—
(xi)	Silver (as Ag), mg/l, *Max*	0.01	J	—
(xii)	Chloride (as Cl), mg/l, *Max*	200		IS 3025 (Part 32)
(xiii)	Sulphate (as SO_4), mg/l, *Max*	200	—	IS 3025 (Part 24)
(xiv)	Magnesium (as Mg), mg/l, *Max*	50	—	IS 3025 (Part 46)
(xv)	Calcium (as Ca), mg/l, *Max*	100	—	IS 3025 (Part 40)

Contd...

(1)	(2)	(3)	(4)	(5)
(xvi)	Sodium (as Na), mg/l, *Max*	150	—	IS 3025 (Part 45)
(xvii)	Alkalinity (as HCO_3), mg/l,	75 to 400	—	IS 3025 (Part 23)
(xviii)	Selenium (as Se), mg/l, *Max*	0.05	—	28 of IS 3025
(xix)	Mineral oil	Not detectable	—	IS 3025 (Part 39)
(xx)	Phenolic compounds (as C_6H_5OH)	Not detectable	—	54 of IS 3025
(xxi)	Anionic surface active agent.	Not detectable	K	—

Table 9.15 (c) : Standards for Packaged Mineral Waters

Parameters Concerning Toxic Substances

S. No.	Characteristics	Requirements	Method of Test, Ref. to	
			Annex of this Standard (IS : 13428)	Other Standards
(1)	(2)	(3)	(4)	(5)
(i)	Arsenic (as As), mg/l, *Max*	0.05	—	IS 3025 (Part 37)
(ii)	Cadmium (as Cd), mg/l, *Max*	0.003	—	IS 3025 (Part 41)
(iii)	Cyanide (as CN), mg/l, *Max*	0.07	—	IS 3025 (Part 27)
(iv)	Chromium (as Cr), mg/l, *Max*	0.05	J	—
(v)	Mercury (as Hg), mg/l, *Max*	0.001	—	IS 3025 (Part 48)
(vi)	Lead (as Pb), mg/l, *Max*	0.01	—	IS 3025 (Part 47)
(vii)	Nickel (as Ni), mg/l, *Max*	0.02	L	—
(viii)	Polychlorinated biphenyle (PCB)	Not detectable	M	—
(ix)	Polynuclear aromatic hydrocarbons	Not detectable	*See* Note	—

NOTE—The test for polynuclear aromatic hydrocarbon should be kept in abeyance till a test method is incorporated in the standard.

Table 9.15 (d) : Standards for Packaged Mineral Waters

Parameters Concerning Radioactive Residues

S. No.	Characteristics	Requirements	Method of Test Ref. to
(1)	(2)	(3)	(4)
(*i*)	'Alpha' emitters, Bq/l (*max*)	0.1	IS 14194 (Part 2)
(*ii*)	'Beta' emitters, Bq/l (*max*)	1	IS 14194 (Part 1)

Table 9.15 (e) : Standards for Packaged Mineral Waters

Microbiological Requirements

S. No.	Microbial parameter	Limiting Standards
(1)	(2)	(3)
1.	*Escherichia-Coli (E-coli)*	*E-coli or Thermotelerant bacteria* shall be absent in any 250 ml sample, when tested in accordance with method given in IS : 5887 (Part 1).
2.	*Coliforms*	Shall be absent in any 250 ml sample when tested in accordance with the method given in IS : 5401.
3.	*Faecal Streptococci and Straphylococcus aurenus*	Shall be absent in any 250 ml sample, when tested in accordance with the method given in IS : 5887 (Part 2).
4.	*Sulphite reducing anaerobics*	Shall be absent in 50 ml sample, when tested in accordance with the method given in IS : 13428 (Annexure-C).
5.	*Pseudomonas aeruginosa*	Shall be absent in 250 ml sample, when tested in accordance with the method given in IS : 13428 (Annexure-D).
6.	*Anaerobic microbial count*	The total viable colony count shall not exceed 100 per ml at 20°C to 22°C in 72 hrs on agar-agar, or on agar-gelatine mixture ; and 20 per ml at 37°C in 24 hrs on agar-agar when tested in accordance with the methods given in IS : 5402
7.	*Yeast and Mould*	Shall be absent in 250 ml sample, when tested in accordance with the method given in IS : 5403.
8.	*Salmonella and Shigella*	Shall be absent in any 250 ml sample when tested in accordance with the method given in IS : 5887 (Part 3).
9.	*Vibrio Cholera and V. Parahaemolyticus*	Shall be absent in 250 ml sample, when tested in accordance with the method given in IS : 5887 (Part 5).

In addition to the above microbiological requirements, the packaged mineral water should contain Pesticide residues as covered under the relevant Rule of Prevention of Food Adulteration Act, 1954, *to be below the detectable limits*, when tested in accordance with the relevant methods (as specified by various BIS codes for testing 32 No. pesticides and mentioned in Annexure N of BIS Code 13428 : 1998). The mentioned 32 pesticides to be tested and their corresponding BIS codes for testing procedure, are indicated in Table 9.15 (*f*) :

Table 9.15 (f) : BIS Codes for Testing Pesticide Residues for Packaged Mineral Water

S. No.	Pesticide	BIS code giving methods for residue analysis
1.	Malathion	IS 5863 : 1985
2.	DDT	IS 5864 : 1983
3.	Parathian	IS 5952 : 1972
4.	BHC (HCH)	IS 6169 : 1983
5.	Fenitrothion	IS 10168 : 1982
6.	Carbaryl	IS 10169 : 1982
7.	Aldicarb	IS 10629 : 1983
8.	Methyl Parathion	IS 10630 : 1983
9.	Carbofuran	IS 11020 : 1984
10.	Dimethoate	IS 11021 : 1984
11.	Phosalone	IS 11301 : 1985
12.	Monocrotophos	IS 11374 : 1985
13.	Ethion	IS 11773 : 1986
14.	Dichlorvos	IS 11774 : 1986
15.	Propoxor	IS 12335 : 1988
16.	Diazinon	IS 12336 : 1988
17.	Chlorpyrifos	IS 12365 : 1988
18.	Fenthion	IS 12609 : 1989
19.	Phosphamidon	IS 12610 : 1989
20.	Endosulfan	IS 12611 : 1989
21.	Cypermethrin	IS 12616 : 1989
22.	Deltamethrin	IS 12616 : 1989
23.	Fenvalerate	IS 12616 : 1989
24.	Permethrin	IS 12616 : 1989
25.	Atrazine	IS 13829 : 1993
26.	Simazine	IS 13829 : 1993
27.	Captafol	IS 13830 : 1993
28.	Acephate	IS 13831 : 1993
29.	Dithiocarbamate	IS 14311 : 1995
30.	Metaloxyl	IS 14161 : 1994
31.	Fosetylal	IS 14162 : 1994
32.	Lindane (Gamma BHC)	IS 14311 : 1995

9.35. BIS Standards for Packaged Drinking Water Other than Packaged Natural Mineral Water

The standards specified by BIS code IS 14543 : 1998 for Packaged Drinking water other than the packaged natural mineral water are briefly discussed below :

9.35.1. Definition of Drinking Water and Packaged Drinking Water Other than the Natural Mineral Water. Drinking water is defined as the water from any potable water source including public drinking water supply systems (See IS : 10500). Drinking water filled in hermetically scaled containers of various compositions, forms and capacities that is suitable for direct consumption without further treatment is defined as the packaged drinking water.

9.35.2. Treatment and Handling of Packaged Drinking Water (Other than the Natural Mineral Water). The drinking water meant for packaging may be subjected to treatment namely decantation, filtration, combination of *filtration, aeration, filtration with membrane filter*, depth filter, cartridge filter, *activated carbon filtration, demineralisation, reverse osmosis*. The drinking water may also be *disinfected* to a level that will not lead to harmful contamination in the drinking water.

The drinking water shall be collected, processed, handled, packaged and marketed in accordance with the hygienic practices given in Annexure B of the said BIS code.

9.35.3. Quality Requirements of Packaged Drinking Water (Other than Natural Mineral Water). The quality requirements of the packaged drinking water are almost the same, as for the packaged mineral water, and are discussed below :

(1) Physical and organoleptic properties. These requirements are shown in Table 9.16 (*a*), and are the same as for mineral water and given in Table 9.15 (*a*) except in relation to requirement of TDS, which is limited to 500 mg/l here, as against the value of 150 to 700 mg/l specified for packaged mineral waters.

Table 9.16. (a) Standards for Packaged Drinking Water
Physical and Organoleptic Parameters

S. No.	*Characteristics*	*Requirements*	*Method of Tests Ref. to IS Code No.*
(*i*)	Colour, Colour unit, Max	2	3025 (Part 4)
(*ii*)	Odour	Agreeable	3025 (Part 5)
(*iii*)	Taste	Agreeable	3025 (Part 8)
(*iv*)	Turbidity NTU, Max	2	3025 (Part 10)
(*v*)	Total dissolved solids, mg/l	500	3025 (Part 16)
(*vi*)	pH	6.5 to 8.5	3025 (Part 11)

(2) General parameters concerning substances undesirable in excessive amounts. These requirements are shown in table 9.16 (*b*), and are vastly different from those specified for Mineral waters, Pl. compare table 9.15 (*b*) with table 9.16 (*b*).

Table 9.16. (b) Standards for Packaged Drinking Waters
General Parameters Concerning Substances Undesirable in Excess Amounts

S. No.	*Characteristic*	*Requirement*	*Method of Test, Ref. to*
(*i*)	Barium (as Ba), mg/l, *Max*	1.0	Annex F of IS 13428
(*ii*)	Copper (as Cu), mg/l, *Max*	0.05	IS 3025 (Part 42)
(*iii*)	Iron (as Fe), mg/l, *Max*	0.1	32 of IS 3025
(*iv*)	Manganese (as Mn), mg/l, *Max*	0.1	35 of IS 3025
(*v*)	Nitrate (as NO_3), mg/l, *Max*	45	IS 3025 (Part 34)
(*vi*)	Nitrite (as NO_2), mg/l, *Max*	0.02	IS 3025 (Part 34)
(*vii*)	Fluoride (as F), mg/l, *Max*	1.0	23 of IS 3025
(*viii*)	Zinc (as Zn), mg/l, *Max*	5	IS 3025 (Part 49)
(*ix*)	Silver (as Ag), mg/l, *Max*	0.01	Annex J of IS 13428
(*x*)	Aluminium (as Al), mg/l, *Max*,	0.03	M of IS 3025
(*xi*)	Chloride (as Cl), mg/l, *Max*	200	IS 3025 (Part 32)
(*xii*)	Selenium (as Se), mg/l, *Max*	0.01	28 of IS 3025
(*xiii*)	Sulphate (as SO_4), mg/l, *Max*	200	IS 3025 (Part 24)
(*xiv*)	Alkalinity (as HCO_3), mg/l, *Max*	200	IS 3025 (Part 23)
(*xv*)	Calcium (as Ca), mg/l, *Max*,	75	IS 3025 (Part 40)
(*xvi*)	Magnesium (as Mg), mg/l, *Max*	30	IS 3025 (Part 46)
(*xvii*)	Sodium (as Na), mg/l, *Max*	200	IS 3025 (Part 45)
(*xviii*)	Residual free chlorine, mg/l, *Max*	0.2	IS 3025 (Part 26)
(*xix*)	Phenolic compounds (as C_6H_5OH), mg/l, *Max*	0.001	54 of IS 3025
(*xx*)	Mineral oil, mg/l, *Max*	0.01	IS 3025 (Part 39)
(*xxi*)	Anionic surface active agents (as MBAS), mg/l, *Max*	0.2	Annex K of IS 13428

(3) Requirements for parameters concerning toxic substances. These requirements are given in table 9.16 (*c*). These requirements are generally the same as are given for mineral waters, except for cadmium and cyanide Pl. compare table 9.16 (*c*) with table 9.15 (*c*).

Table 9.16 (c) Standards for Packaged Drinking Water
Parameters Concerning Toxic Substances

S. No.	*Characteristic*	*Requirement*	*Method of Test Ref. to*
(*i*)	Mercury (as Hg), mg/l, *Max*	0.001	IS 3025 (Part 48)
(*ii*)	Cadmium (as Cd), mg/l, *Max*	0.01	IS 3025 (Part 41)
(*iii*)	Arsenic (as As), mg/l, *Max*	0.05	IS 3025 (Part 37)
(*iv*)	Cyanide as (CN), mg/l, *Max*	0.05	IS 3025 (Part 27)
(*v*)	Lead (as Pb), mg/l, *Max*	0.01	IS 3025 (Part 47)
(*vi*)	Chromium (as Cr), mg/l, *Max*	0.05	Annex J of IS 13428
(*vii*)	Nickel (as Ni), mg/l, *Max*	0.02	Annex L of IS 13428
(*viii*)	Polychlorinated biphenyle (PCB)	Not detectable	Annex M of IS 13428
(*ix*)	Polynuclear aromatic hydrocarbons	Not detectable	See Note

Note. The test for polynuclear aromatic hydrocarbons should be kept in abeyance till a test method is incorporated in the standard.

(4) Requirements for parameters concerning radioactive residues. These requirements for packaged drinking water remain totally the same as were given for packaged mineral water in table 9.15 (*d*). Hence, are not being repeated.

(5) Microbiological requirements. Microbiological requirements for packaged drinking water also remain totally the same, as were given for packaged mineral water in table 9.15 (*e*). Hence are not being repeated.

(6) Requirements for Pesticides. Pesticide requirements for packaged drinking water (other than the packaged mineral water), till recently, were the same as existing for mineral water and already described in the previous article. These existing provisions about **the pesticides to be below the detectable limits** was found to be insufficient, as the Centre for Science and Environment, New Delhi showed through its study published in "Down to the Earth" magazine (vol. 11, No. 18, February 15, 2003) that most brands of bottled water sold in India under BIS certification mark were pesticide laiden, when tested in accordance with international standards. As a consequence to this study, the Indian Govt. and BIS were forced to reconsider the present provisions of BIS code as well as of PF Act, about the limits imposed on Pesticides. The Ministry of Health and Family Welfare GoI, vide its notification dated July 18, 2003, under *Prevention of Food Adulteration (1st Amendment) Rules, 2003,* has now specified that in all the items covered under the definition of food & drinks under PF Act, all the 32 known pesticides

shall be tested separately, and *pesticide residues considered individually shall not be more than 0.0001 mg/l, while total pesticide residue shall not be more than 0.0005 mg/l. (The testing analysis shall be conducted by using internationally established test methods meeting the residue limits specified herein). These standards shall be enforceable wef. January 1, 2004, and shall be applicable to packaged drinking water but not to packaged mineral waters drawn from natural springs.* The BIS Code IS : 14543 : 1998 has also specified these very specifications for pesticide levels wef April 1, 2003 ; which deadline, was later extended to January 1, 2004, following representations from the industry.

PROBLEMS

1. Enumerate and discuss briefly the various methods which are adopted collectively for treating public water supplies drawn from a perennial river.
2. Explain the sedimentation process used in a water treatment plant. Draw a neat sketch of a sedimentation tank in which coagulant is used.
   Compute the dimensions of continuous flow rectangular settling tank for a population of 20,000 persons with a daily per capita water allowance of 120 litres. Assume detention period to be 6 hours.
3. Which are the discrete particles removed in an ideal sedimentation tank under the following conditions :
   (*a*) horizontal flow ; and
   (*b*) Vertical flow.
4. Give the approximate reasonable values of the detention time in hours of the sedimentation tank as noted below :
   (*i*) Sedimentation tank followed by coagulation and flocculation ; and
   (*ii*) Sedimentation tank followed by lime soda process.
5. (*a*) Define 'flowing through period' and 'detention period' in a sedimentation basin.
   (*b*) Prove that the area and overflow rates rather than the detention period govern the design of a settling tank.
   (*c*) Design a rectangular sedimentation tank to treat 3780 m^3/day of coagulated water. Make necessary assumptions. Sketch the inlet, outlet and sludge removal arrangements.
6. (*a*) Prove theoretically that the surface loading and not the depth is a measure of effective removal of particles in an ideal sedimentation tank. Mention the assumptions made in theory.
   (*b*) The population of a city is 50,000 and the per day capita consumption is 130 *l*/day. Calculate the following in respect of the rapid sand filter for the above data :
   (*i*) Total area of filters ;
   (*ii*) Number and dimension of each filter bed ;
   (*iii*) Quantity of air for air wash per filter bed ;
   (*iv*) Back wash water per filter bed after air wash.
7. Describe briefly the various constituents of a coagulation-sedimentation plant.
8. (*a*) Enumerate the chemicals which are used for coagulation. Discuss their comparative merits and demerits.
   (*b*) How will you determine the optimum coagulant quantity by jar test.

9. Distinguish between slow sand and rapid sand filters with reference to :
   (*i*) Rate of filtration ;
   (*ii*) Filter media of sand ;
   (*iii*) Period of cleaning ; and
   (*iv*) Loss of head.

10. Distinguish between slow sand and rapid sand filters with reference to
   (*i*) Rate of filtration ;
   (*ii*) Quantity of wash water ;
   (*iii*) Method of cleaning ;
   (*iv*) Effective size of sand ; and
   (*v*) Depth of bed for gravel and sand.

11. (*a*) Show that the settling velocity of a spherical particle in a liquid under condition when Reynold's number is less than 0.5 may be given by expression

$$V_s = \frac{g}{18}(S_s - 1)\frac{d^2}{\nu}$$, where S_s is sp. gravity of the particle

   (*b*) Draw a typical layout sketch of water treatment plant using rapid gravity filters.

   (*c*) Design slow sand filter for the following details :

Population	1 lakh
Per capita demand	140 litres/day

   Total maximum demand 1.5 times average

   Indicate general arrangement of filter beds.

12. Draw a plan, cross-section and longitudinal section of a slow sand filter bed, with dimensions, noting the sizes and depths of filtering materials used.

   Design six slow sand filter beds from the following data and show the arrangements of bed in plan :
   (*i*) population to be served = 50,000 persons.
   (*ii*) Quantity of water to be supplied = 200 litres per head per day.
   (*iii*) Rate of filtration = 300 litres per sq. m per day.
   (*iv*) Length of each bed is twice the breadth.

13. Draw a neat sketch of a rapid gravity filter and describe how it works. What are its advantages over the slow sand filter.

   Design a set of rapid filters for treating water required for a population of 50,000 ; the rate of supply being 180 litres per day. The filters are rated to work 5,200 litres per square metre.

14. Describe with a neat sketch the working of a pressure filter. What are the relative advantages and disadvantages of this type over those of the gravity filters.

15. Discuss the relative merits of Rapid Gravity filters and Pressure filters, indicating the conditions favourable for the choice of each.

   Describe the working of any one of them with the aid of neat sketches.

16. Describe with a sketch, the working of a pressure filter bringing out the circumstances under which it can be used.

17. Indicate in appropriate space what effect an increase in variable A will have on variable B.

S. No.	*Variable A*	*Variable B*	*Increase*	*Decrease*	*Net Effect*
(*i*)	Depth of tank	Overflow rate			
(*ii*)	Temperature of water	Settling velocity			
(*iii*)	Oxygen contents	Rate of absorption of oxygen			
(*iv*)	Water temperature	Head loss in filter			

18. Design rapid sand filtration units for a population of 100,000 to be served by a 200 litres/capita/day water supply. Assume the following :

Rate of filtration	$= 3 \times 10^5$ m³/ha/day
Amount of wash water	= 5% of filtered water/day
Filter dimensions of each unit	= 17.5 m × 10 m.

   The filter needs back washing once in 24 hours.
   Assume any other data not given.

19. (*a*) Give the chemical reactions when ferrous sulphate is used as a coagulant along with lime to remove turbidity.
   (*b*) Find the settling velocity of a discrete particle in water under conditions when Reynold's number is less than 0.5. The diameter and specific gravity of the particle is 5×10^{-3} cm and 2.65 respectively. Water temperature is 20°C [kinematic viscosity = ν of water at 20°C = 1.010×10^{-2} cm²/sec].

20. What is meant by 'disinfection" in treating public water supply ? What is its importance ? What are the chemicals which are used as disinfectants and what are their comparative merits and demerits ?

21. Discuss the use of chlorine as disinfecting agent with reference to :
   (*i*) Its disinfecting action ; (*ii*) Its doses ;
   (*iii*) Its forms ; and (*iv*) Testing its residuals.

22. What is meant by sterilization of water ? Describe briefly any *three* methods of doing so and state their relative merits.

23. (*a*) Give three major requirements of a disinfectant.
   (*b*) Give chemical equations for precipitation of iron when iron bearing water is aerated and precipitation of calcium when hard water containing calcium bicarbonate is treated with lime.

24. Discuss how a slow sand filter differs from a rapid sand filter.

   Find the area of slow sand filter required for a town having a population of 20,000 with average rate of demand as 150 l.p.h.d.

   What is break point chlorination and under what conditions is it advocated ?

25. (*a*) Explain briefly the following processes :
      (*i*) Break point chlorination ;
      (*ii*) Super chlorination.
   (*b*) Calculate the amount of bleaching powder required to treat 3.0 M.l. of water/day. The chlorine required is 0.5 p.p.m. to maintain a residual chlorine of 0.15 p.p.m. If high strength calcium hypochlorite is used in place of bleaching powder, calculate the difference in amount required. Make your own assumptions.

26. (*a*) Differentiate between temporary and permanent hardness.
(*b*) Mention any three methods of softening water. Describe 'Zeolite process' of softening water in detail.

27. Give the range of values for the following in water treatment :
(*i*) Minimum chlorine residual at 20°C and pH between 7 and 8.
(*ii*) pH range to use alum as coagulant.
(*iii*) Maximum surface loading or overflow rate for settling alum or iron flocks.
(*iv*) Minimum detention time for setting alum or iron flocks in a 3.3 m deep sedimentation tank.
(*v*) Loss of head in rapid sand filter.

28. Discuss and debate the various methods which are adopted for treating public supplies in order to remove colour and taste from it.

29. Write short notes on any four of the following :
(*i*) Chlorination
(*ii*) Water Aeration
(*iii*) Fluoridation
(*iv*) Desalination
(*v*) Removal of radioactivity from water
(*vi*) Chemical coagulation
(*vii*) Negative head in a filter
(*viii*) Break point method of chlorination
(*ix*) Clarflocculator
(*x*) Carbonate hardness
(*xi*) Non-carbonate hardness
(*xii*) Jar test.

30. Differentiate between the following pairs :
(*a*) 'Peri-kinetic flocculation' and 'ortho-kinetic flocculation' ;
(*b*) 'Loss of head' and 'negative head' in rapid sand filters ;
(*c*) 'Lime soda process' and 'Zeolite process' in water softening.

31. Match the words under "Process" with the corresponding appropriate words under "Impurity removed" :

Process	*Impurity Removed*
(1) Aeration	(*a*) Larger suspended solids
(2) Screening	(*b*) Dissolved solids
(3) Plain settling	(*c*) Pathogenic microbes
(4) Filtration	(*d*) Colloids and microbes
(5) Disinfection	(*e*) Taste and odour
(6) Zeolite Process	(*f*) Fine suspended solids
(7) Settling plus coagulation	(*g*) Floating solids
(8) Chemical treatment plus filtration	(*h*) Hardness

[**Ans.** 1—*e* ; 2—*g* ; 3—*a* ; 4—*b* ; 5—*c* ; 6—*h* ; 7—*f* ; 8—*d*]

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Distribution System

10.1. General Introduction

After the water has been properly treated and made safe and wholesome, it has to be supplied to the consumers in their individual homes. The water has, therefore, to be taken from the treatment plant to the roads and streets in the city, and finally to the individual houses. This function of carrying the water from the treatment plant to the individual homes is accomplished through a well planned *distribution system*. A distribution system may, therefore, consists of *pipe lines** of various sizes for carrying the water to the streets ; *valves* for controlling the flow in the pipes, *hydrants* for providing connections with the water mains for releasing water during fires ; *meters* for measuring discharges ; service connections (called *services*) to the individual homes ; *pumps* for lifting and forcing the water into the distribution pipes, distribution or *service reservoirs* for storing the treated water to be fed into the distribution pipes ; etc. Further, the water may be supplied to the public either *continuously* for all the 24 hours of the day, or it may be supplied intermittently during certain fixed hours of the day. Also, the water may either be pumped directly into the distribution pipes, or it may first be stored in a distribution reservoir and then fed into the distribution pipes.

10.2. Requirements** of a Good Distribution System

The various requirements for proper functioning of a distribution system are :

(*i*) It should be capable of supplying water at all the intended places within the city with a reasonably sufficient pressure head.

(*ii*) It should be capable of supplying the requisite amount of water for fire fighting during such needs.

(*iii*) It should be cheap with the least capital construction cost. The economy and the cost of installing the distribution system is a very important factor, because the distribution system is the most costly item in the entire water supply scheme. So much so, that it gobbles up, upto about 70% of the total cost of the scheme.

(*iv*) It should be simple and easy to operate and repair, thereby keeping the RMO cost and troubles to the minimum.

(*v*) It should be safe against any future pollution of water. This aim may be achieved by keeping the water pipe lines above and away from the sewerage and drainage lines by sufficient amounts, and also by improving the general

*It includes the mains, submains, branches or laterals (also called feeders) which carry the water to the streets.
**i.e., the "Factors affecting the design of a distribution system".

26. (*a*) Differentiate between temporary and permanent hardness.
(*b*) Mention any three methods of softening water. Describe 'Zeolite process' of softening water in detail.

27. Give the range of values for the following in water treatment :
(*i*) Minimum chlorine residual at 20°C and pH between 7 and 8.
(*ii*) pH range to use alum as coagulant.
(*iii*) Maximum surface loading or overflow rate for settling alum or iron flocks.
(*iv*) Minimum detention time for setting alum or iron flocks in a 3.3 m deep sedimentation tank.
(*v*) Loss of head in rapid sand filter.

28. Discuss and debate the various methods which are adopted for treating public supplies in order to remove colour and taste from it.

29. Write short notes on any four of the following :
(*i*) Chlorination
(*ii*) Water Aeration
(*iii*) Fluoridation
(*iv*) Desalination
(*v*) Removal of radioactivity from water
(*vi*) Chemical coagulation
(*vii*) Negative head in a filter
(*viii*) Break point method of chlorination
(*ix*) Clarflocculator
(*x*) Carbonate hardness
(*xi*) Non-carbonate hardness
(*xii*) Jar test.

30. Differentiate between the following pairs :
(*a*) 'Peri-kinetic flocculation' and 'ortho-kinetic flocculation' ;
(*b*) 'Loss of head' and 'negative head' in rapid sand filters ;
(*c*) 'Lime soda process' and 'Zeolite process' in water softening.

31. Match the words under "Process" with the corresponding appropriate words under "Impurity removed" :

Process	*Impurity Removed*
(1) Aeration	(*a*) Larger suspended solids
(2) Screening	(*b*) Dissolved solids
(3) Plain settling	(*c*) Pathogenic microbes
(4) Filtration	(*d*) Colloids and microbes
(5) Disinfection	(*e*) Taste and odour
(6) Zeolite Process	(*f*) Fine suspended solids
(7) Settling plus coagulation	(*g*) Floating solids
(8) Chemical treatment plus filtration	(*h*) Hardness

[**Ans.** 1—*e* ; 2—*g* ; 3—*a* ; 4—*b* ; 5—*c* ; 6—*h* ; 7—*f* ; 8—*d*]

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Distribution System

10.1. General Introduction

After the water has been properly treated and made safe and wholesome, it has to be supplied to the consumers in their individual homes. The water has, therefore, to be taken from the treatment plant to the roads and streets in the city, and finally to the individual houses. This function of carrying the water from the treatment plant to the individual homes is accomplished through a well planned *distribution system*. A distribution system may, therefore, consists of *pipe lines** of various sizes for carrying the water to the streets ; *valves* for controlling the flow in the pipes, *hydrants* for providing connections with the water mains for releasing water during fires ; *meters* for measuring discharges ; service connections (called *services*) to the individual homes ; *pumps* for lifting and forcing the water into the distribution pipes, distribution or *service reservoirs* for storing the treated water to be fed into the distribution pipes ; etc. Further, the water may be supplied to the public either *continuously* for all the 24 hours of the day, or it may be supplied intermittently during certain fixed hours of the day. Also, the water may either be pumped directly into the distribution pipes, or it may first be stored in a distribution reservoir and then fed into the distribution pipes.

10.2. Requirements** of a Good Distribution System

The various requirements for proper functioning of a distribution system are :

(*i*) It should be capable of supplying water at all the intended places within the city with a reasonably sufficient pressure head.

(*ii*) It should be capable of supplying the requisite amount of water for fire fighting during such needs.

(*iii*) It should be cheap with the least capital construction cost. The economy and the cost of installing the distribution system is a very important factor, because the distribution system is the most costly item in the entire water supply scheme. So much so, that it gobbles up, upto about 70% of the total cost of the scheme.

(*iv*) It should be simple and easy to operate and repair, thereby keeping the RMO cost and troubles to the minimum.

(*v*) It should be safe against any future pollution of water. This aim may be achieved by keeping the water pipe lines above and away from the sewerage and drainage lines by sufficient amounts, and also by improving the general

*It includes the mains, submains, branches or laterals (also called feeders) which carry the water to the streets.
**i.e., the "Factors affecting the design of a distribution system".

sanitary conditions of the area through which the distribution pipes have to pass.

(*vi*) It should be safe as not to cause the failure of the pipe lines by bursting, etc.

(*vii*) It should be fairly water-tight, as to keep the "losses due to leakage" to the minimum.

10.3. Arrangement of Distribution Pipes and Other Accessories

The distribution pipe system consists of supply mains, submains, branches and laterals ; usually made of cast iron and jointed by means of "spigot and socket joints". The service connections are made of galvanised cast iron pipes. These water mains and submains are usually laid sloping from the high level to the low level areas, so as to achieve the maximum advantage of the available head, and thus to keep their sizes minimum.

Sluice valves are placed along the straight length of the pipes at suitable intervals, and also at all the junctions and branching off points, so as to control the flow of water into the different sections. *Drain valves* are placed at all the low points in the distribution pipe system, so as to drain off the water from the pipes for carrying out any repairs, etc. These drain valves are connected properly to the sewers through check valves, etc., so as to avoid contamination by back flow. *Air valves* are placed at all the high points, so as to remove air from the pipe during the filling operations, and also to admit air while emptying the pipe. The description and detailing of all these valves have been given earlier in chapter 6, and may again be referred to.

In smaller cities and towns, the ***distribution mains*** generally take off from the treatment plant itself, and distribute the water into the different branches and laterals. However, large cities are generally divided into various zones, and each zone may be served with a separate distribution reservoir. In such cases, the distribution mains will take off from these service reservoirs, and carry the water to the different localities of the zone.

The distribution pipes are generally laid on one side of the roads and the streets, usually below the footpaths*, and at about 2 m above (*i.e.* vertically) and 3 m away (*i.e.* horizontally) from the sewers, so as to avoid any future contamination of water through the leaky joints. Generally, sewer lines run on one side of the streets, and the water pipes run on the other side of the streets.

In wider streets, however, a *subsidiary lateral* may also be provided on the other side of the street, which is cross-connected to the main lateral at suitable intervals, as shown in Fig. 10.1. This will avoid the otherwise needed long house connections.

The sizes of the distribution pipes mainly depend upon the amount of flow to be carried, and the permissible loss in the pressure head. The methods of solving the pipe networks for determining their sizes shall be dealt later. *It may, however, be mentioned here that the smallest lateral should normally be not less than 8 to 10 cm in diameter ; and if it has to supply water to a fire hydrant, it should be at least 15 to 20 cm in diameter.*

*Laying the pipes below the footpaths instead of laying them below the carpetted roadway helps in easy excavations during repairs.

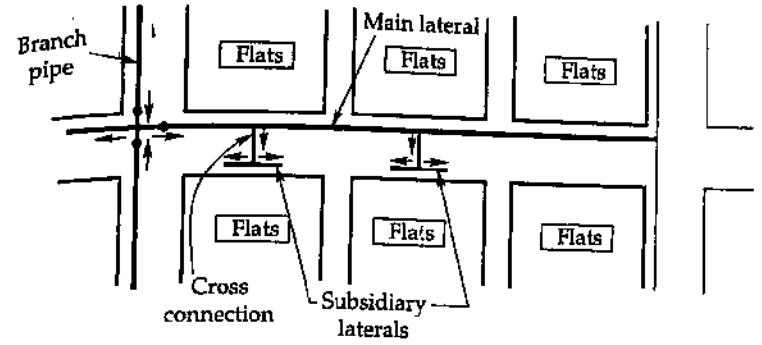


Fig. 10.1. Laying of pipe lines on both sides of street for wider streets.

10.4. Layouts of Distribution Networks

As pointed out earlier, the distribution pipes are generally laid below the road pavements, and as such, their layouts will generally follow the layouts of the roads. There are, in general, four different types of pipe networks ; any one of which, either singly or in combinations, can be used at a particular place, depending upon the local conditions and orientation of roads. These systems are :

1. *Dead end system* ;
2. *Grid iron system* ;
3. *Ring system* ; and
4. *Radial system.*

The different types of distribution networks are described below. Their designs and calculations for working out the sizes of various pipes shall, however, be dealt later.

(1) **Dead-end System.** In the dead end system, which is also sometimes called ***Tree system***, there is one *main* supply pipe, from which originates (generally at right angles) a number of submain pipes. Each submain, then divides into several branch pipes, called laterals. From the laterals, service connections are given to the consumers. A typical plan of such a network is shown in Fig. 10.2.

This type of layout may have to be adopted for older towns which have developed in a haphazard manner, without properly planned roads. The water supply mains have then to be taken along the main roads, and branches taken off wherever needed, thus resulting in the formation of a number of ***dead ends****** as shown. This system is, therefore, suitable for localities which expand irregularly, and where the water pipes have to be laid at random due to the absence of any planned full fledged road network.

The ***advantages*** of this system are :

(*i*) The distribution network can be solved easily, and it is possible to easily and accurately calculate the discharges and pressures at different points in the system.

*The termination or end point of the pipe is known as a dead end.

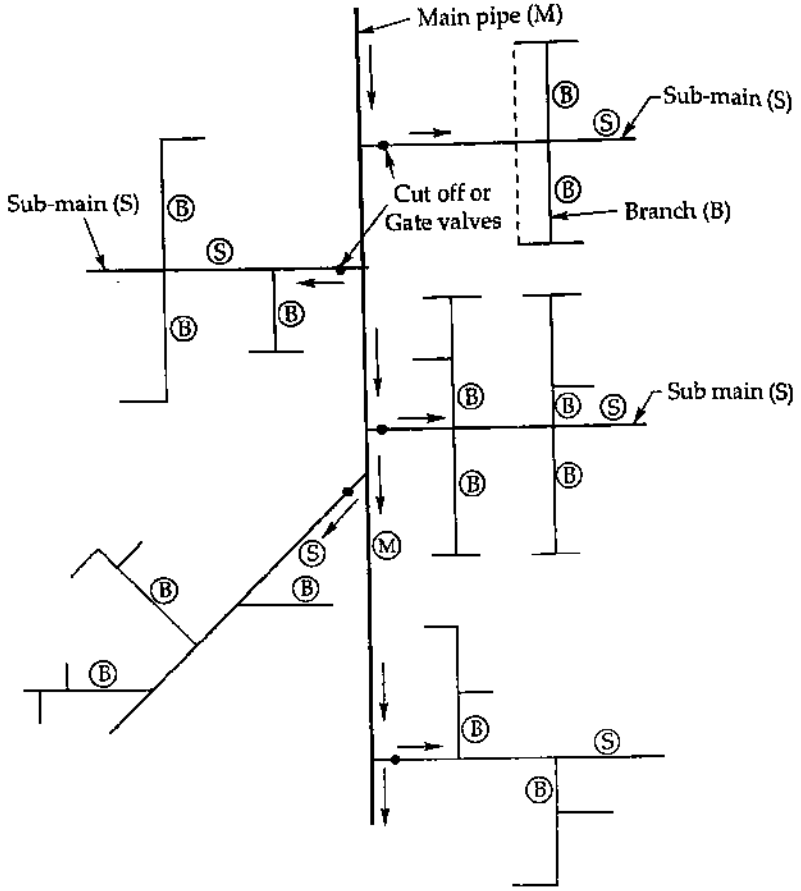


Fig. 10.2. Dead-end or Tree system.

(*ii*) Lesser number of cut-off valves (*i.e.* sluice valves) are required in this system.

(*iii*) Shorter pipe lengths are needed, and the laying of pipes is easier.

(*iv*) It is cheap and simple, and can be extended or expanded easily.

The ***disadvantages*** of this system are :

(*i*) Since, in this method, water can reach a particular point only through one route, any damage or repair in any pipe line will completely stop the water supply in the area being fed by that pipe. Thus, if damage occurs or repair is taking place in a main pipe or in a submain pipe, considerable areas may get affected, thereby causing great inconvenience to the consumers.

(*ii*) *There are numerous dead ends in this system*, which prevent the free circulation of water. This stagnation of water may lead to the degradation in its quality. This stale water should, therefore, be removed periodically at all the dead ends, by providing scour valves at each dead end. This will result in greater wastage of treated water, and will necessitate careful attendance at each valve.

(*iii*) Since in this system, the discharge is reaching a point from only one direction, the supplies during fire fighting cannot be increased by diverting any other supplies from any other side. Hence, it can give only limited supplies and may sometimes prove to be a serious handicap.

(2) **Grid-iron System.** In this system, which is also known as *Interlaced system or* ***Reticulation system***, the mains, sub-mains and branches are all inter-connected with each other, as shown in Fig. 10.3. In fact, in a well planned city or a town, the roads are generally developed in a grid-iron pattern, and the pipe lines in such places can follow them easily. Hence, this system is more suitable for well planned towns and cities, and has been used in Chandigarh.

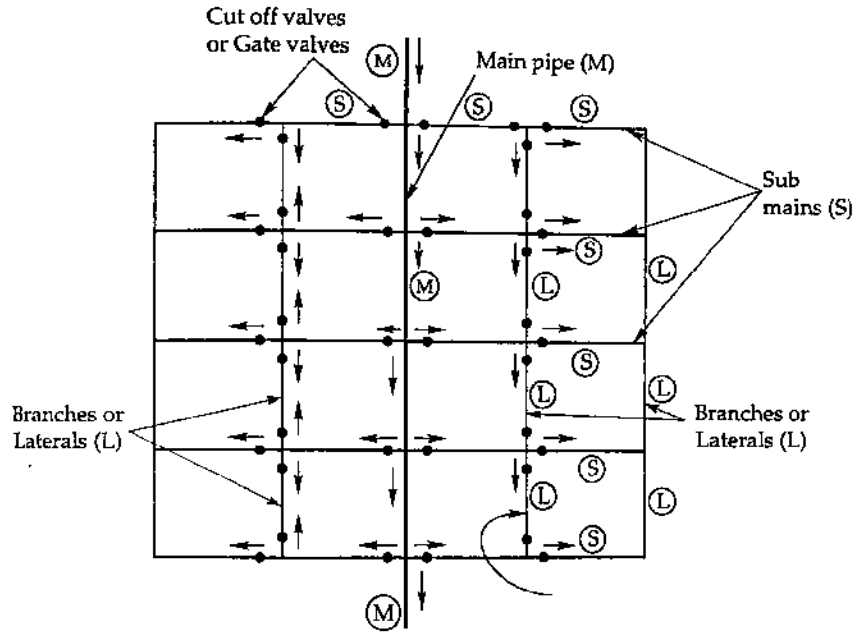


Fig. 10.3. Grid-iron system.

Nevertheless, the principle of grid-iron system can be applied to the dead end system also, by closing the loop, and thus removing the dead ends, as shown by a dotted line in Fig. 10.2.

The ***advantages*** of this system are :

(*i*) Since the water reaches at different places through more than one route, the discharge to be carried by each pipe, the friction loss, and the size of the pipe, therefore, get reduced.

(*ii*) In case of repairs, very small area will be devoid of complete supply, as atleast, some supply will be reaching at the point from some other route.

(*iii*) Because of the different inter-connections, the dead ends are completely eliminated, and, therefore, water remains in continuous circulation, and hence not liable to pollution due to stagnation.

(*iv*) During fire, more water can be diverted towards the affected point from various directions by closing and manipulating the various cut-off valves.

The ***disadvantages*** of this system are :

(*i*) This system requires more length of pipe lines, and a larger number of sluice valves (*i.e.* cut-off valves).

(*ii*) Its construction is costlier.

(*iii*) The design is difficult and costlier, the calculations for determining accurately the sizes of the pipes and the pressures at various key points, is a real tedious job, and may require the services of design experts and even computers.

(3) **Ring System.** This system is also sometimes called ***Circular system.*** In this system, a closed ring, either circular or rectangular, of the main pipes, is formed around the area to be served, as shown in Fig. 10.4. The distribution area is divided into rectangular or circular blocks, and the main water pipes are laid on the periphery of these blocks. The sub-mains may be placed as shown. *The ring system is very suitable for towns and cities having well planned roads.* Sometimes, this system is used as a "looped feeder placed centrally around a high demand area" along with the grid iron system. In such a case, this enhances the capacity of the grid iron system, and will improve the pressures at various points. The advantages and disadvantages of this system are the same as that of the grid iron system.

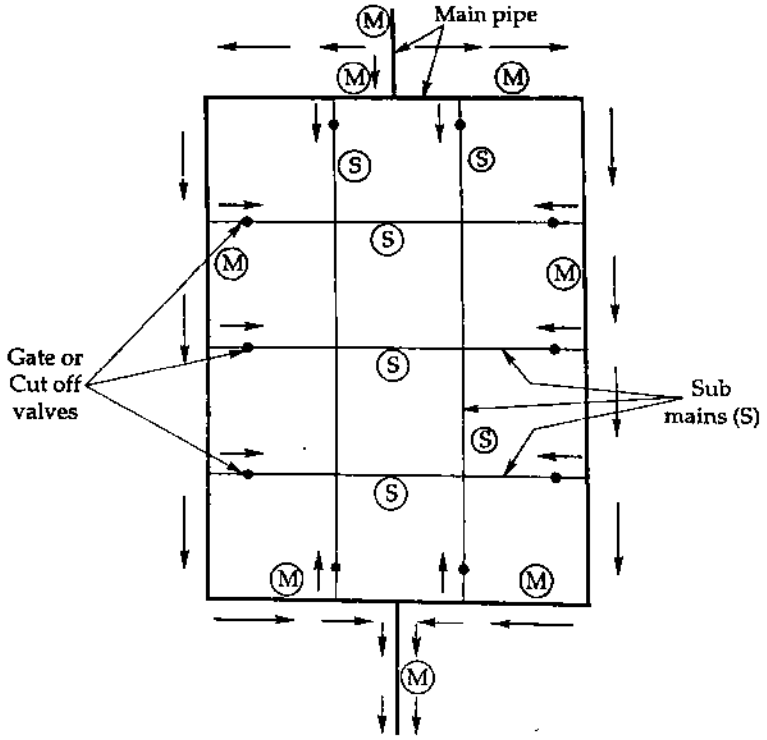


Fig. 10.4. **Ring system.**

(4) **Radial System.** If a city or a town is having a system of radial roads emerging from different centres, the pipe lines can be best laid in a radial

method by placing the distribution reservoirs at these centres. In this system, water, is therefore, taken from the water mains, and pumped into the distribution reservoirs placed at different centres, as shown in Fig. 10.5. The water is then supplied through radially laid distribution pipes. This method ensures high pressures and efficient water distribution. The calculations for design of sizes are also simple.

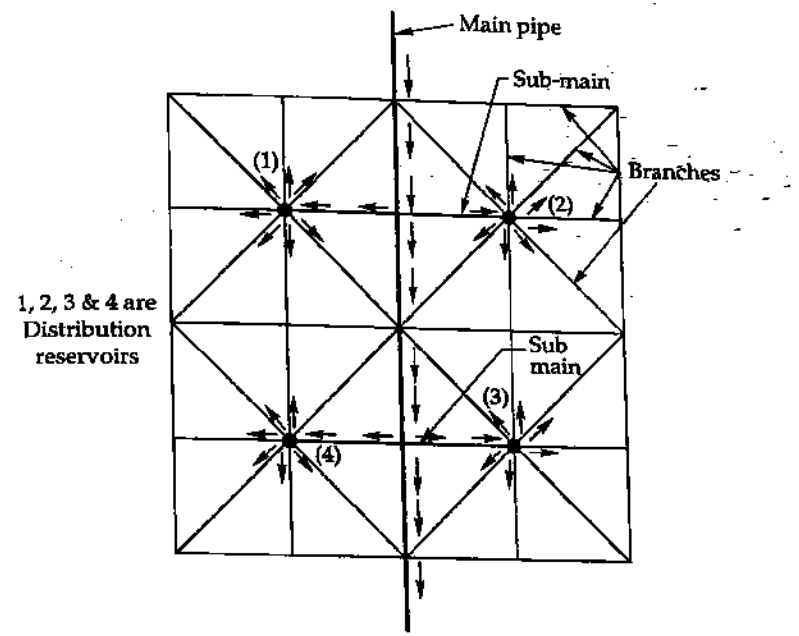


Fig. 10.5. Radial system.

10.5. Methods of Distribution

The main object of a distribution system is to develop adequate water pressures at various points of the consumer's taps. Depending upon the level of the source of water and that of the city, topography of the area, and other local conditions and considerations, the water may be forced into the distribution system in the following three ways :

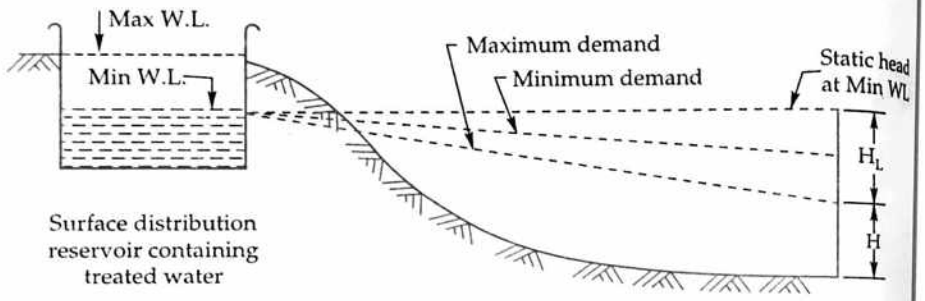
1. *By gravitational system* ;
2. *By pumping system* ; and
3. *By combined gravity and pumping system.*

These systems are described below :

(1) **Gravitational System.** In this system, the water from the high levelled source is distributed to the consumers at lower levels, by the mere action of gravity without any pumping. For proper functioning of this system, the difference of head available between the source (rather the distribution reservoir at the treatment plant) and the localities, must be sufficient enough, as to maintain adequate pressure at the consumer's door-steps, after allowing the frictional and other losses in the pipes. This method is the most economical and reliable, since no pumping is involved at any stage. However, it needs a

lake or a reservoir as a source of supply. In Mumbai, the water which is brought from the high lakes situated in the hills, is distributed to the consumers by this method.

From the above discussion, it becomes evident that such a system can be adopted for cities which are situated at the foot-hills and the source of supply is available somewhere in the hills at sufficient elevation in relation to the elevation of the city. Fig. 10.6 show the gravitational system with hydraulic gradients during maximum and minimum demands.



where H_L = Head loss
H = Head available to consumers.

Fig. 10.6. Gravitational distribution system.

In gravitational system, the pumping is normally not required at any stage. However, in a case, where the source, such as a lake is situated at a hill, and the treatment plant is also situated almost at the same level on the hill itself, then the water may have to be conveyed from the source to the treatment plant by low lift pumping. However, the treated water will be distributed to the low levelled consumers situated at the toothill by mere gravity. Hence, no pumping will be required in the distribution system.

The gravitational system is designed so as to leave only the minimum permitted available head to the consumers, and the rest is consumed in frictional and other losses. This will keep the leakages and the wastages to the minimum, and will also reduce the required sizes of the pipes. ($\because$ greater is H_L, lesser dia. is reqd. to pass a given discharge). However, this will necessitate the use of *motor pumps,* so as to develop sufficient pressure during fires.

(2) **Pumping System.** In the pumping system, the treated water is directly pumped into the distribution mains without storing it anywhere. For this reason, this system is also sometimes called ***pumping without storage system.*** High lift pumps are required in this system, which have to operate at variable speeds, so as to meet the variable demand of water*. Thus, a continuous attendance is needed at the pumping station, so as to ensure the desired flow in the distribution system. Moreover, if the power supply fails, there will be complete stoppage of water supply, and if by chance, a fire breaks out at such a time, it will bring disaster. *This method is, therefore, generally not used.* However, the only advantage of this method is that during fires, it can force

*Either the pumps should function at variable speed or a number of pumps should be installed, and are to be switched on or switched off, depending upon the requirements.

large volumes of water under high pressure in the required direction, so that the motor pumpers may be eliminated. Fig. 10.7 shows the pumping system with hydraulic gradients during maximum or minimum demands.

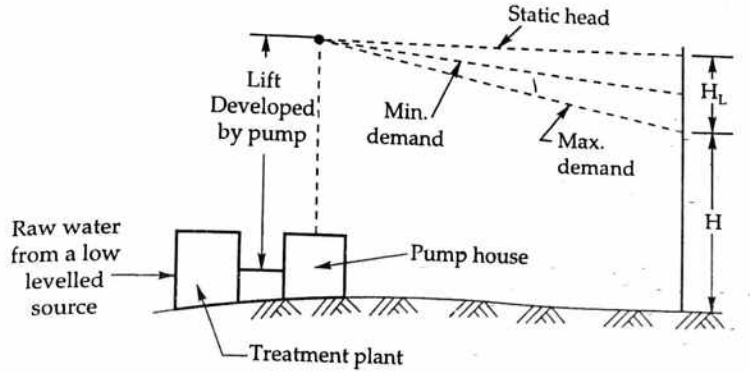


Fig. 10.7. Pumping system for water distribution.

(3) **Combined Gravity and Pumping System.** In this system, the treated water is pumped at a constant rate and stored into an elevated distribution reservoir, from where it is distributed to the consumers by the mere action of gravity. Sometimes, the entire water is first of all pumped into the distribution reservoir, and many a times, it is pumped into the distribution mains and reservoirs, simultaneously. This method thus, combines pumping as well as gravity flow, and is sometimes called ***pumping with storage system.***

The excess water during low demand periods gets stored in the reservoir and gets supplied during high demand periods. The pumps are worked at a constant rate, which is adjusted in such a way that the excess quantity of water stored in the reservoir during low consumption nearly equals the extra demand during high consumption. This system helps in operating the pumps at constant speeds at their rated capacities, thus increasing their efficiency and reducing their wear and tear. Fig. 10.8 shows this system with the hydraulic gradient lines for minimum and maximum demands.

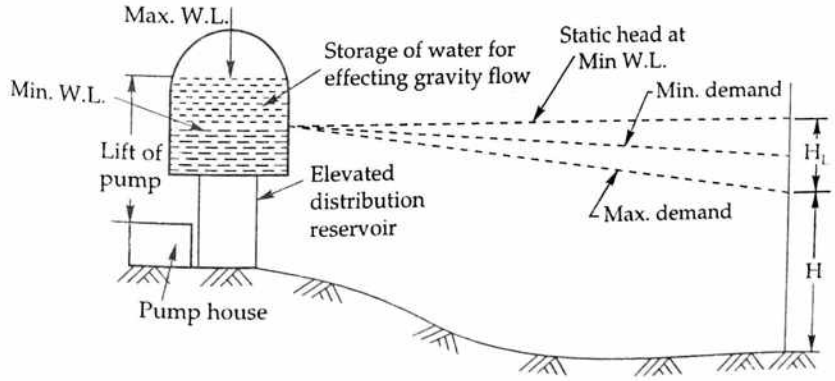


Fig. 10.8. Combined gravity and pumping system for water distribution.

This type of system is invariably and almost universally adopted because of its following advantages :

(*i*) The balancing reserve of the distribution reservoir can be supplied to the places of fire. However, the necessary pressure required to be developed for fire fighting can be achieved by closing down the supply of some localities or by using motor pumpers. In all modern fire fighting squads, motor pumps are, therefore, provided.

(*ii*) The pumps are to be worked at uniform rate and, thereby operating them to their rated capacities. This increases their efficiency and also reduces the wear and tear of the pumps. The attendance and supervision required for operating these pumps is much less compared to the case when they are operated at variable speeds. In smaller towns, this method of distribution enables us to operate the pumps only for a part of the day, *i.e.* for 8 to 16 hours.

(*iii*) This method is quite reliable because even during the power failure or pumps failure, certain amount of water can be supplied from the storage or service reservoir.

(*iv*) This system proves to be overall cheap, efficient and reliable, and hence adopted practically everywhere.

10.6. Pressures in the Distribution System

From the time the water enters the distribution mains, and up to the time it comes out at a consumer's tap, water-head is constantly lost in overcoming friction of the pipes (H_L), and also in the fittings, meters, valves, etc. (H_{LM}). The net available head at a consumer's door step is thus reduced by these losses. This net available head or effective head (H) is the most important pressure, because it will make the water rise upto the different storey heights. The greater is this pressure, the greater will be the height up to which water will be able to rise. For example, if this pressure is 0.6 kg/cm^2 ($\approx$ 60 kN/m^2), then naturally, water can, at the most, rise by 6 m, and not above that. Hence, in cases when the adequate pressures are not maintained in the pipe lines, the persons living on upper storeys (*i.e.*, 2nd, 3rd or 4th) may not get water in their homes. For this reason, it is necessary that sufficient pressures are maintained in the distribution system, so as to supply water to the consumers up to the desired heights. It is also necessary to keep the losses in the distribution pipes and fittings to optimum minimum.

From the above discussion, it appears that the pressures should be maintained as high as possible, so as to give higher flow and also to serve the upper storey consumers. But from economic considerations, it is not possible to permit very high pressures ; because, this will necessitate the use of higher lift pumps and thicker and stronger pipes, thereby increasing the operational as well as the initial cost of installation. Moreover, it will increase the water lost due to leakage and waste. The choosing of a proper value for the design pressures to be maintained in the distribution pipes is, therefore, a matter of cost *versus* convenience. *The greater is the design pressure, the costlier it will be, but will cause more convenience to the consumers.*

The value of the design pressure may also sometimes be guided by the fire-fighting requirements. In case of large cities, where the fighting squads are

generally equipped with motor pumps for lifting the water at the site itself, this factor may not govern the chosen value of the design pressure. However, in small towns, where fire fighting squads are not equipped with motor pumpers, higher pressures, sufficient enough to throw water at 2nd or even 3rd storey heights, must be maintained in the distribution pipes.

The provision of meters in the individual house connections also make it obligatory to choose higher value of design pressure, because the meters cause unnecessary hinderance in the flow and the consequent loss of head.

The municipal water supplies in India, wherever existing, are mostly under-developed, because they ensure the water supply only to the ground floors, or at the most to the first floor consumers at the time of maximum demand. The 2nd and upper storey consumers face a lot of troubles and have to install their own private booster pumps. The distribution system in Indian cities is thus required to take a maximum pressure head of hardly 8 to 15 metres ; whereas in America, the normal minimum pressure maintained is of the order of 20 to 30 metres. *Under ordinary Indian circumstances, it will thus be enough to design the city's distribution system for a minimum pressure head of 14 metres, (≈ 140 kPa) so as to let the water rise three storeys and overcome the frictional resistance of the house connection pipes.* The multi-storeyed buildings may then have to develop their own boosters and elevated storage tanks. Ideally, however, if all the houses are metered, and the fire-fighting squads are not equipped with motor pumps, it will be desirable to have a pressure head of at least 28 metres (≈ 280 kPa). Pressures in excess of 70 m in the distribution system are generally undesirable and maximum allowable pressure is of the order of 100 m, which generally equals the design pressure for plumbing fixtures, like pipe fittings, etc.

In hilly areas, where there may be wide variations in the elevations of the various houses, it is desirable and economical to divide the town or the city into a number of zones at different levels (with differences of 30 metres or so) ; and then feeding each zone by a separate distribution reservoir. Pressure relief valves should also be placed at all places where the pipes pass through low levelled areas, so as to ensure the safety of the pipe lines against bursting due to high pressure.

10.7. Systems of Supply

The water may be supplied either *continuously* for all the 24 hours of the day or may be supplied *intermittently* only for the peak periods during morning and evening. The second, *i.e.* the ***intermittent supply system*** may sometimes lead to some saving in water consumption due to losses occurring for lesser time and more vigilant use of water by consumers. This method may, therefore, be adopted at places where there is a shortage of water at the source. However, the intermittent system should not be continued as a long term policy, and be replaced by the ***continuous supply system*** at the earliest opportunity, because of the inherent limitations of this system. These limitations or *disadvantages of the intermittent system* are discussed below.

(*i*) Even the very first argument advocated in favour of the use of the intermittent system, that is that "saving in water is obtained" is debatable. At

many places, the intermittent system may fail to give any saving of water over the continuous system because of the reasons discussed in article 2.4 (8).

(*ii*) Intermittent system of supplies do not cover fire risks during non-supply periods. Any fire break-out during such periods may bring disaster and immense damage to life and property.

(*iii*) This system causes great inconvenience to consumers, keeping them on their toes for receiving and collecting water as soon as the supply is restored.

(*iv*) Since in this system, the supply of water for the whole day has to be pumped during 6 to 8 hours only, it will necessitate the use of bigger sized distribution mains, if the water is supplied to the whole town at the same time.

(*v*) When the supply of water is stopped and the water from the pipe is drawn off, a partial vacuum may be created in the pipe. This will induce suction through the leaky joints ; and if the pipes are surrounded by dirts on the ground, the same may get into the pipes. This will contaminate the initial supplies as and when the supply is restored.

(*vi*) A number of air valves and sluice valves are required to be fitted in the distribution system. These valves will have to be operated daily while opening or closing the supply. This will necessitate additional watch and ward staff and high maintenance cost.

Inspite of these limitations and disadvantages, the intermittent supply system is largely employed in India*. This system may or may not help in saving water, but may definitely help in supplying water to high level areas with adequate pressures, by zoning the city. For improving the pressures, the entire city is divided into a number of zones, and different zones are supplied with water during different hours, thus obtaining better pressures**. This system may also facilitate the repair works, as the same can be done during non-supply hours.

DISTRIBUTION RESERVOIRS

Distribution reservoirs, also called ***service reservoirs,*** are the storage reservoirs, which store the treated water for supplying water during emergencies (such as during fires, break-downs, repairs, etc.) and also to help in absorbing the hourly fluctuations in the normal water demand. The main functions served by the distribution reservoirs are enumerated below.

10.8. Functions of Distribution Reservoirs

(*i*) They absorb the hourly variations in demand, and allow the water treatment units and pumps to operate at a constant rate. This will reduce the RMO costs and improve efficiency.

(*ii*) They help in maintaining constant pressure in the distribution mains. In their absence, the pressure will fall down as the demand of water increases and if the pumps don't immediately respond to it.

*Even in Delhi, the Capital of India, intermittent supply system is followed.

**For lesser discharge and for the same horse power of the pumps, the lift can be increased thus giving better pressures.

(*iii*) The pumping of water in shifts is made possible by them without affecting the supply. Thus, 8 to 16 hours of pumping can be carried out, so as to pump the whole day's demand.

(*iv*) The water stored in these reservoirs can be supplied during emergencies, such as break-down of pumps, heavy fire demand, etc.

(*v*) They lead to an overall economy by reducing the sizes of pumps, pipe lines, and treatment units.

10.9. Types of Distribution Reservoirs

The distribution reservoirs may be made of steel, R.C.C., or masonry. Depending upon their elevation with respect to the ground, they may be classified into the following two types :

1. *Surface reservoirs* ; and
2. *Elevated reservoirs.*

Both these types of reservoirs are discussed below :

10.9.1. Surface Reservoirs. Surface reservoirs are circular or rectangular tanks, constructed at ground level or below the ground level. They are, therefore, also called, ***ground reservoirs.*** They are generally constructed at high points in the city. If a city has more than one high point, more than one reservoir may be provided. In that case, the city will be divided into several zones, and a separate reservoir is provided in each zone for distribution of water.

In a gravitational type of distribution system, water is stored in the *ground service reservoir*, and then directly sent from there into the distribution system. However, in a combined gravity and pumping system of distribution, the treated water is first of all stored in a *ground reservoir*, and then pumped into an elevated service reservoir, from where it can be supplied into the distribution mains.

A typical section of a surface reservoir is shown in Fig. 10.9. It is divided into two compartments, so that one may be cleaned and repaired, while the

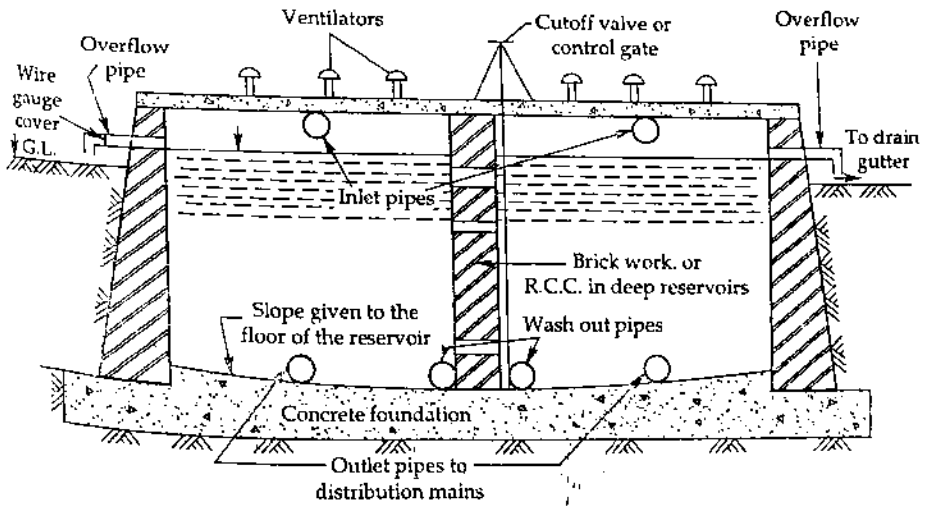


Fig. 10.9. Typical section of a ground reservoir.

other is in use. The two compartments are connected with each other by shut off valve or sluice valves. Overflow pipes are provided at full supply level, so as to maintain a constant water level.

Ventilators are provided in the roof slab so as to affect free circulation of air. Although the stored water is treated, yet some sludge may settle down due to storage, and hence has to be removed. Such a reservoir is, therefore, cleaned through the washout pipes, at suitable intervals. The cement concrete floor may, therefore, be sloped towards the central washout pipes, as shown.

10.9.2. Elevated Reservoirs. Elevated reservoirs are the rectangular, circular, or elliptical *overhead* tanks erected at a certain suitable elevation above the ground level and supported on towers. They are constructed where the pressure requirements necessitate considerable elevation above the ground surface, and where the use of ***stand pipes**** becomes impracticable. They are constructed in areas where the combined gravity and pumping system for water distribution is adopted. Water is pumped into these elevated tanks from the filter units or from the surface reservoirs, and then supplied to the consumers. A typical sketch of a simple rectangular tank is shown in Fig. 10.10.

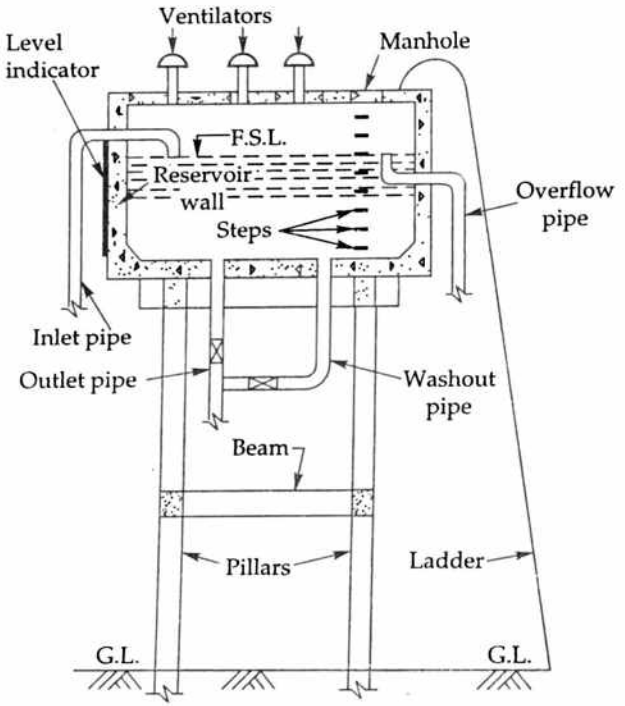


Fig. 10.10. Rectangular elevated tank (reservoir).

These tanks may be made of R.C.C., steel, or prestressed concrete. With the advancement in structural analysis, it has been possible to construct them in any shape, so as to suit the architectural requirements. The R.C.C. tanks are now-a-days preferred to the steel tanks, because they are cheaper, do not

*Described in the next article.

corrode, and require less maintenance. An R.C.C. tank of **Intz** type, such as shown in Fig. 10.11, has become very popular these days, because the same is structurally sound and quite economical. Prestressed concrete elevated tanks are also coming up in the modern days, and can give considerable savings in steel (up to 60%) and concrete (up to 40%). Their design computations can, however, be found in any standard book on R.C.C. Designs :

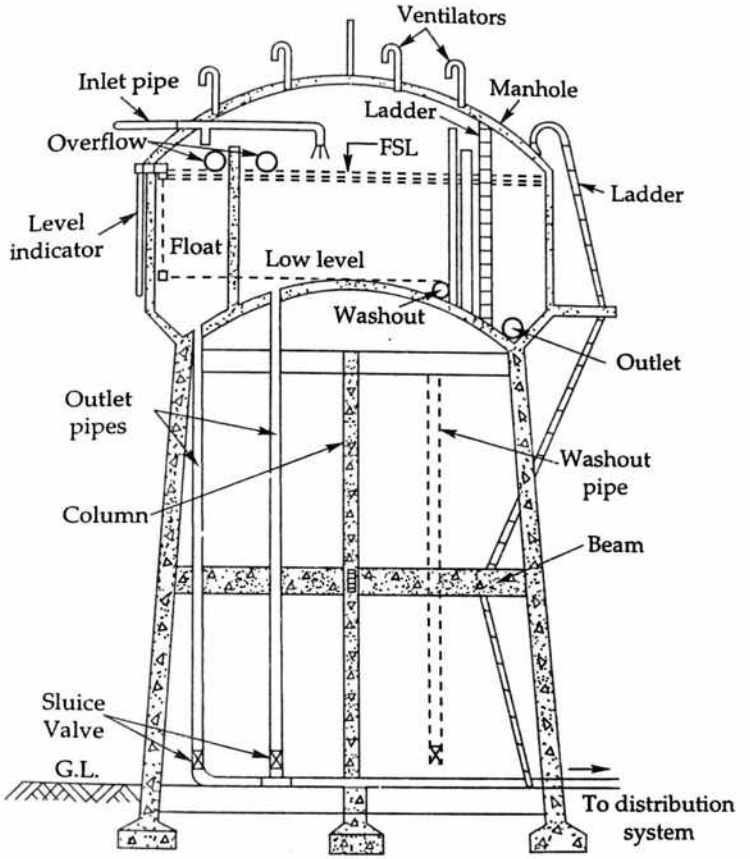


Fig. 10.11. Intz tank.

These elevated reservoirs are generally very costly and, hence, are not designed for capacities* of more than 6 to 8 hours of the average daily supply of the city ; and possibly situated in the heart of the city. They are generally covered at top by the roof slabs in order to avoid the contamination of water due to dirts, birds, insects, etc. Roofs should, however, be provided with ventilators, so as to allow free circulation of air. Various other accessories of such a reservoir are :

(*i*) Inlet pipe for the entry of water.

(*ii*) Outlet pipe connected to the distribution mains for the exit of water.

*Discussed under "Storage Capacities of Distribution Reservoirs" a little later, under article 10.11.

(*iii*) Overflow pipe discharging into drain gutters and maintaining constant level.

(*iv*) A float gauge or an indicator for indicating the depth of water which can be read from outside.

(*v*) A wash-out pipe (or drain pipe) for removing water after cleaning of the reservoir.

(*vi*) Automatic devices to stop pumping when the tank is full.

(*vii*) Ladders to reach the top of the reservoir and then upto the bottom of the reservoir for inspection.

(*viii*) Manholes for providing entry into the tank for inspection purposes.

(*ix*) Ventilator for fresh air circulation (as pointed out earlier also).

These accessories have been shown in sketches 10.10 and 10.11.

10.10. Stand Pipes

Stand pipes are a kind of elevated tanks without any erected towers for resting the tank body. They are thus tall cylindrical shells resting *directly* on the ground, as shown in Fig. 10.12. These stand pipes may be 15 to 30 m high, and 10 to 15 m in diameter.

They may be made of R.C.C. or preferably of steel containing 0.2% copper (so as to avoid corrosion). Because of their large heights, the Hoope's tension developed in the bottom portion is very high. R.C.C. is difficult to bear such high stresses and, therefore, R.C.C. stand pipes may not remain water-tight under heads above 15 m or so. R.C.C. stand pipes are therefore, generally not used for heights above 15 m or so.

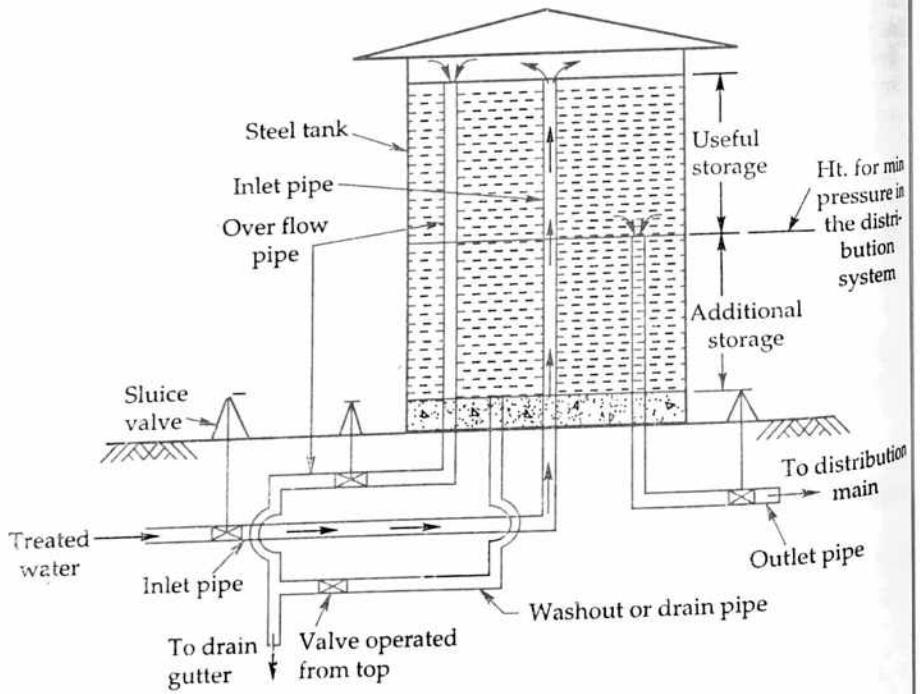


Fig. 10.12. Stand pipe.

The useful storage capacity of a stand pipe is the volume above the elevation required to give the requisite pressure in the distribution system. The water stored below this elevation can also be utilised, especially during fire-fighting, with the help of booster pumps. Like elevated reservoirs, stand pipes are also provided with inlet pipe, outlet pipe, drain or wash-out pipe, overflow pipe, and other miscellaneous facilities, such as manholes, ladders, etc. for inspection, repairs and maintenance.

10.11. Storage Capacity of Distribution Reservoirs

The total storage capacity of a distribution reservoir is the summation of :

1. *Balancing storage* (or *equalising* or *operating storage*) ;
2. *Breakdown storage* ; and
3. *Fire storage*.

They are described below :

(1) **Balancing Storage or Equalising Storage.** The main and primary function of a distribution reservoir is to meet the fluctuating demand with a constant rate of supply from the treatment plant. The quantity of water required to be stored in the reservoir for equalising or balancing this variable demand against the constant supply is known as the *balancing reserve* or *balancing storage* or *the storage capacity of a balancing reservoir*.

This balancing storage can be worked out by utilising the hydrographs of inflow and outflow, either by mass curve method or by using an analytical tubular solution.

Mass Curve Method. **A mass diagram** is the plot of accumulated inflow (*i.e.* supply) or outflow (*i.e.* demand) *versus* time. The mass curve of supply (*i.e. supply line*) is, therefore, first of all, drawn ; and is superimposed by the demand curve. The amount of balancing storage can then be easily determined by adding the maximum ordinates between the demand and the supply lines. To construct such diagrams for a particular water supply project, we have to proceed as follows :

(*a*) From the past records, determine the hourly demand (or draft) for all 24 hours for typical days (maximum, average and minimum).

(*b*) Calculate and plot the cumulative demand against time, and thus plot the ***mass curve of demand*** (Fig. 10.13 or 10.14).

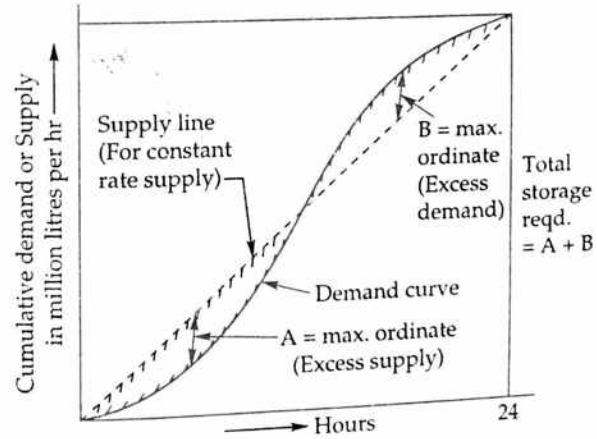


Fig. 10.13. Mass diagram for 24 hours pumping.

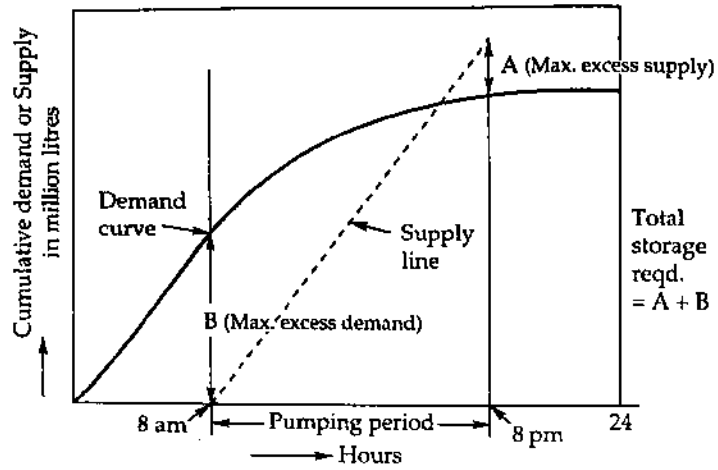


Fig. 10.14. Mass diagram for pumping during limited period.

(*c*) Draw the cumulative supply also against time* to plot the ***supply line*** or ***mass curve of supply*** as shown in Fig. 10.13 or 10.14.

(*d*) Read the storage required, as the sum of the two maximum ordinates between demand and supply line, as shown in Figs. 10.13 and 10.14.

(*e*) Repeat the procedure for all the typical days (maximum, average and minimum), and determine the maximum storage required for the worst day.

Analytical Solution. In this method, the cumulative hourly demand and cumulative hourly supplies are tabulated for all the 24 hours. The hourly excess of demand as well as the hourly excess of supply (cumulative of course) are then worked out. The summation of maximum of the excess of demand and the maximum of excess of supply will give us the required storage capacity.

These methods will become more clear when we solve the following examples :

Example 10.1. *A town with a population of 1 million has a continuous water supply. Average supply is 270 lpcd, the water being supplied by direct pumping. The total supply of 270 lpcd is phased as follows :*

Time	*lpc*
5 A.M. to 11 A.M.	*90*
11 A.M. to 3 P.M.	*54*
3 P.M, to 9 P.M.	*81*
9 P.M. to 12 midnight	*27*
12 midnight to 5 A.M.	*18*

Water is supplied from the treatment plant at a uniform rate of 11.25 million litres per hour, for all the 24 hours. Find out the capacity of the reservoir

*If pumping is continued for all the 24 hours of the day, the supply line will be a straight line extending diagonally across the diagram for the whole day, as shown in Fig. 10.13 ; and if the pumping is resorted to only for certain fixed hours, then supply lines can be drawn from the beginning of the pumping period to its end, as shown in Fig. 10.14.

required for distribution of water. Assume no loss or drawal from the trunk main. (*A.M.I.E., May 1963*)

Solution.

Total daily supply = Rate of supply × Population
= 270×10^6 litres = 270 M. litres

Now, the given demand pattern is converted into cumulative demand, as shown in Table 10.1.

Table 10.1

Time	*Daily per capita consumption in lpc*	*Consumption in M. litres for 1 million population (2) × 1 M*	*Cumulative demand in M. litres*
(1)	*(2)*	*(3)*	*(4)*
5 A.M. to 11 A.M.	90	90	90
11 A.M. to 3 P.M.	54	54	144
3 P.M. to 9 P.M.	81	81	225
9 P.M. to 12 (midnight)	27	27	252
12 (midnight) to 5 A.M.	18	18	270

The mass curve of demand is now plotted from the data of col. (1) and col. (4) of the above table, as shown in Fig. 10.15. The supply from the water-works is obtained at a rate of 11.25 M. litres per hour. The supply line on the mass

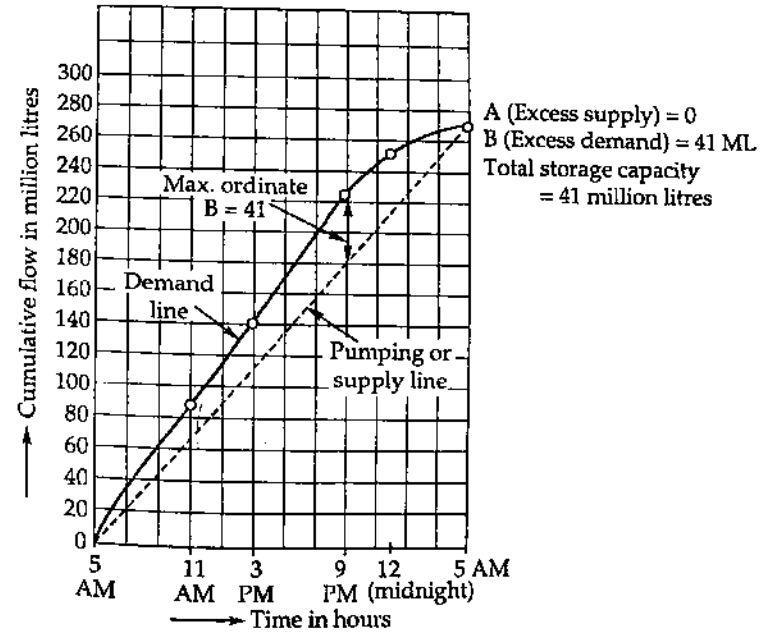


Fig. 10.15

diagram will, therefore, be a straight line with its slope of 11.25 M. litres per hour, as shown in Fig. 10.15. The two maximum ordinates A and B enclosed between the demand line and the supply line are read out from Fig. 10.15, as :

A = 0 million litres

B = 41 million litres

∴ Total storage reqd. = $A + B$ = 41 million litres. **Ans.**

Example 10.2. *For the water supply of a small rural town with the daily requirement of 2,25,000 litres, it is proposed to construct a distribution reservoir.*

The pattern of draw off is as under :

7.00 A.M.—8.00 A.M. 30% of day's supply

8.00 A.M.—5.00 P.M. 35% of day's supply

5.00 P.M.—6.30 P.M. 30% of day's supply

6.30 P.M.—7.00 A.M. 5% of day's supply

The pumping is to be done at a constant rate of 8 hours per day (8 A.M. to 4 P.M.).

Determine the storage capacity of the reservoir. (*Punjab University, 1967*)

Solution. Total daily requirement = 2,25,000 litres = 225×10^3 litres.

Now, the given demand pattern is converted into cumulative demand, as shown in table 10.2.

Table 10.2

Period	*Rate of Demand*	*Demand in litres col. (2) × (225 × 10³)*	*Cumulative demand in 10³ litres*
(1)	*(2)*	*(3)*	*(4)*
7.00 A.M.–8.00 A.M.	30% of total	67.5×10^3	67.50
8.00 A.M.–5.00 P.M.	35% of total	78.75×10^3	146.25
5.00 P.M.–6.30 P.M.	30% of total	67.5×10^3	213.74
6.30 P.M.–7.00 A.M.	5% of total	11.25×10^3	225.00

The mass curve of demand is now plotted from the data of cols. (1) and (4) of the above table, as shown by the firm line in Fig. 10.16.

The total supply is obtained in 8 hours at a rate

$$= \frac{225 \times 10^3}{8} \text{ litres/hr.} = 28.125 \times 10^3 \text{ litres/hr.}$$

The supply line on the mass diagram will, therefore, be a straight line with its slope as 28.125×10^3 litres per hour, as shown by the dotted line in Fig. 10.16. The two maximum ordinates A and B enclosed between the demand and the supply lines are read out from Fig. 10.16, as :

$A = 88 \times 10^3$ litres

$B = 67.5 \times 10^3$ litres

Total storage reqd. $= 155.5 \times 10^3$ litres

$= 1,55,500$ litres. Ans.

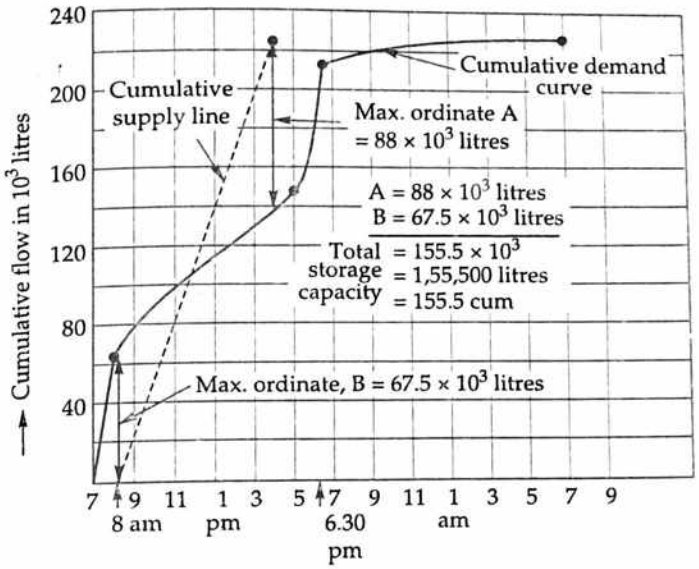


Fig. 10.16

Example 10.3. *A city with a population of 2 lakhs has to be supplied with water at 180 litres per person per day. The probable hourly variation in the rate of demand is given in table 10.3.*

Table 10.3

Period of day in hours	*Percentage of average hourly flow expected*
(1)	*(2)*
0—1	*15*
1—2	*15*
2—3	*15*
3—4	*20*
4—5	*25*
5—6	*40*
6—7	*80*
7—8	*120*
8—9	*180*
9—10	*220*
10—11	*220*
11—12	*150*
12—13	*100*
13—14	*80*
14—15	*60*
15—16	*110*
16—17	*150*
17—18	*180*
18—19	*180*
19—20	*160*
20—21	*140*
21—22	*80*
22—23	*45*
23—24	*15*

Determine the capacity of the balancing reservoir to be provided for balancing the variable demand against a constant rate of pumping, (a) if the pumping is to be done for all the 24 hours. (b) if the pumping is to be done only from 5 A.M. to 11 A.M. and 2 P.M. to 8 P.M. Also mention the rate of pumping required in both cases. Solve this problem by using (i) mass curve method, (ii) analytical method.

Solution. Using mass curve method.

Average daily supply = 180 × 2,00,000 litres = 36 million litres

Average hourly demand $= \frac{36 \text{ Ml}}{24} = 1.5 \text{ Ml}$

The given demand pattern of Table 10.3 is converted to cumulative demand, as shown in table 10.4.

Table 10.4

Period in hours	*Demand in units of av. hourly demand*	*Cumulative demand in terms of av. hourly demand*	*Cumulative demand in million litres = col. (3) × 1.5 Ml*
(1)	*(2)*	*(3)*	*(4)*
0	—	—	—
1	0.15	0.15	0.225
2	0.15	0.30	0.45
3	0.15	0.45	0.675
4	0.20	0.65	0.975
5	0.25	0.90	1.35
6	0.40	1.30	1.95
7	0.80	2.10	3.15
8	1.20	3.30	4.95
9	1.80	5.10	7.65
10	2.20	7.30	10.95
11	2.20	9.50	14.25
12	1.50	11.00	16.50
13	1.00	12.00	18.00
14	0.80	12.80	19.20
15	0.60	13.40	20.10
16	1.10	14.50	21.75
17	1.50	16.00	24.00
18	1.80	17.80	26.70
19	1.80	19.60	29.40
20	1.60	21.20	31.80
21	1.40	22.60	33.90
22	0.80	23.40	35.10
23	0.45	23.85	35.775
24	0.15	24.00	36.00

Case (a) When pumping is done for all the 24 hours of the day.

The mass curve of demand is now plotted from the data of col. (1) and col. (4) of table 10.4, as shown by firm line in Fig. 10.17.

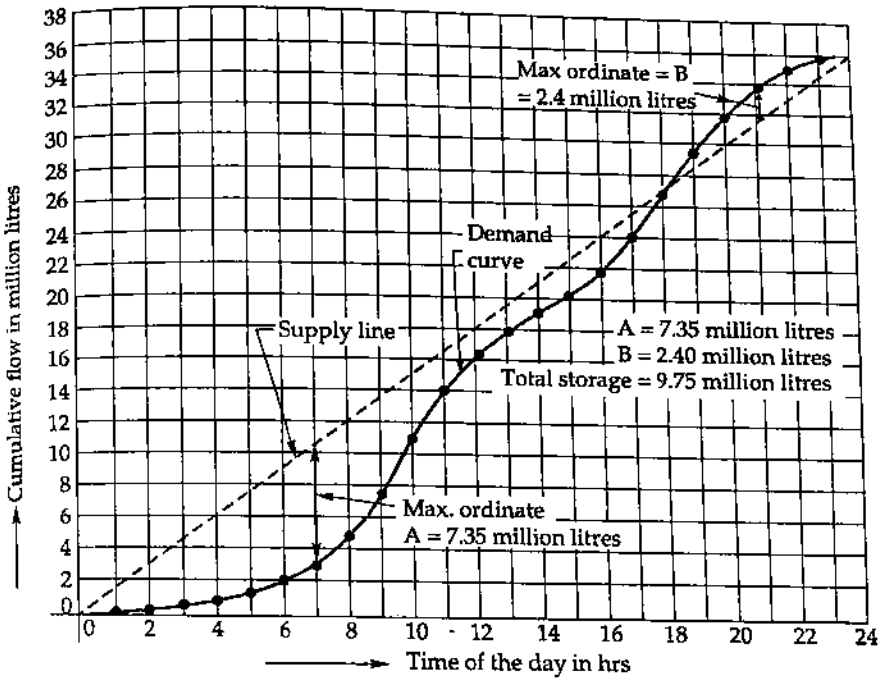


Fig. 10.17

The rate of supply or pumping in this case is given as :

$$= \frac{36}{24} \qquad [\because \text{ pumping is done for 24 hrs.}$$

= 1.5 million litres/hr. **Ans.**

The supply line is also drawn, as shown by the dotted line in Fig. 10.17. The two maximum ordinates A and B, enclosed between the demand and the supply line, are read out from Fig. 10.17, as below :

A = 7.35 million litres

B = 2.40 million litres

Total storage reqd. = 9.75 million litres. **Ans.**

Case (b) When pumping is done for a limited period.

The demand curve is drawn in Fig. 10.18 with the help of col. (1) and col. (4) of table 10.4, as was drawn in Fig. 10.17. Now, when the pumping is resorted

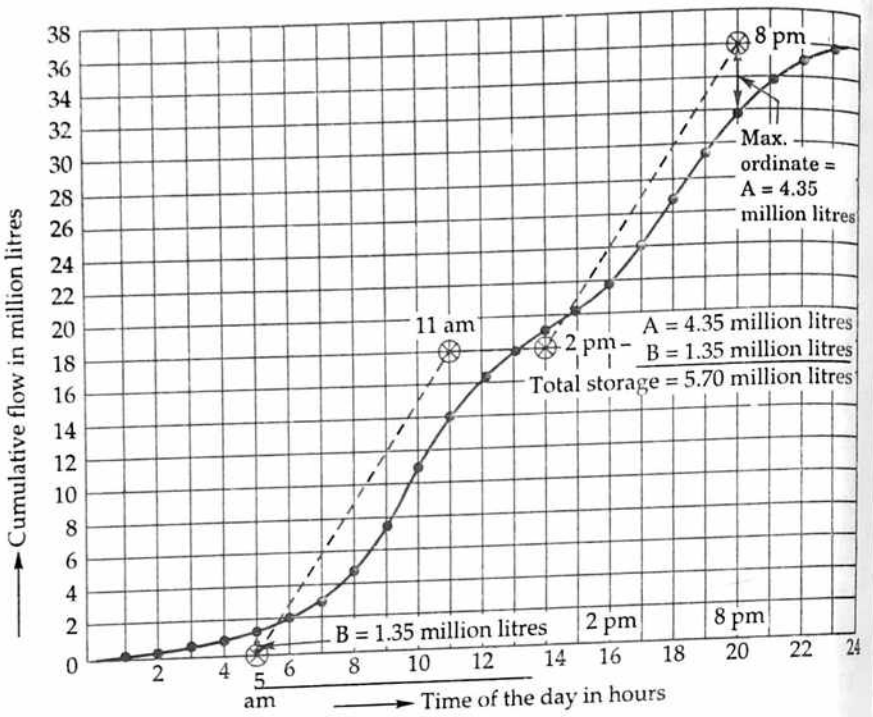


Fig. 10.18

to for only 5 A.M. to 11 A.M. and 2 P.M. to 8 P.M. (*i.e.* for 12 hours),

$$\text{the rate of supply} = \frac{36}{12} = 3 \text{ million litres/hr.} \quad \text{Ans.}$$

The supply line is therefore drawn, as shown by the dotted line in Fig. 10.18. The maximum ordinates A and B enclosed between the supply line and the demand curve are read out from Fig. 10.18, as below :

$A = 4.35$ million litres

$B = 1.35$ million litres

Total storage reqd. = 5.70 million litres. **Ans.**

Using Analytical Method

Case (*a*) *When pumping is done for 24 hours.*

The analytical solution is carried out as shown in Table 10.5. The table is otherwise self-explanatory.

Table 10.5. Analytical solution when pumping is continued for 24 hours

Time in hours of the day	*Cumulative demand as per col. (4) of Table 10.4 in million litres*	*Constant pumping rate in million litres per hour*	*Cumulative pumping in million litres*	*Excess of demand in million litres = col. (2)–col. (4) ; only +ve values*	*Excess of supply in million litres = col. (4)–col. (2) only +ve values*
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*
1	0.225	1.5	1.5	—	1.275
2	0.45	1.5	3.0	—	2.55
3	0.675	1.5	4.5	—	3.825
4	0.975	1.5	6.0	—	5.025
5	1.35	1.5	7.5	—	6.15
6	1.95	1.5	9.0	—	7.05
7	3.15	1.5	10.5	—	**7.35**
8	4.95	1.5	12.0	—	7.05
9	7.65	1.5	13.5	—	5.85
10	10.95	1.5	15.0	—	4.05
11	14.25	1.5	16.5	—	1.25
12	16.50	1.5	18.0	—	1.50
13	18.00	1.5	19.5	—	1.50
14	19.20	1.5	21.0	—	1.80
15	20.10	1.5	22.5	—	2.40
16	21.75	1.5	24.0	—	2.25
17	24.00	1.5	25.5	—	1.50
18	26.70	1.5	27.0	—	1.30
19	29.40	1.5	28.5	0.90	—
20	31.80	1.5	30.0	1.80	—
21	33.90	1.5	31.5	**2.40**	—
22	35.10	1.5	33.0	2.10	—
23	35.775	1.5	34.5	1.275	—
24	36.00	1.5	36.0	—	—

From table 10.5, it can be observed that

the maximum excess of demand = 2.4 million litres

and maximum excess of supply = 7.35 million litres

Hence, the total storage reqd. = 9.75 million litres

(same as worked out earlier). **Ans.**

Case (*b*) *When pumping is done for limited period*

The analytical solution in this case is carried out as shown in Table 10.6. The table is otherwise self-explanatory.

Table 10.6. Analytical solution when pumping is resorted only for the limited period

Time in hours of the day	*Cumulative demand as per col. (4) of Table 10.4 in million litres*	*Pumping in million litres*	*Cumulative pumping in million litres*	*Excess of demand in million litres = col. (2)–col. (4) ; only +ve values*	*Excess of supply in million litres = col. (4)–col. (2) only +ve values*
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*
1	0.225	—	—	0.225	—
2	0.45	—	—	0.45	—
3	0.675	—	—	0.675	—
4	0.975	—	—	0.975	—
5	1.35	—	—	**1.35**	—
6	1.95	3.0	3.0	—	1.05
7	3.15	3.0	6.0	—	2.85
8	4.95	3.0	9.0	—	4.05
9	7.65	3.0	12.0	—	**4.35**
10	10.95	3.0	15.0	—	4.05
11	14.25	3.0	18.0	—	3.75
12	16.50	—	18.0	—	1.50
13	18.00	—	18.0	—	—
14	19.20	—	18.0	1.20	—
15	20.10	3.0	21.0	—	0.9
16	21.75	3.0	24.0	—	2.25
17	24.00	3.0	27.0	—	3.00
18	26.70	3.0	30.0	—	3.30
19	29.40	3.0	33.0	—	3.60
20	31.80	3.0	36.0	—	4.20
21	33.90	—	36.0	—	2.10
22	35.10	—	36.0	—	0.90
23	35.775	—	36.0	—	0.225
24	36.00	—	36.0	—	—

From Table 10.6, it can be observed that

the maximum excess of demand = 1.35 million litres

and the maximum excess of supply = 4.35 million litres.

Hence, the total storage required = 5.70 million litres

(same as worked out earlier). **Ans.**

Example 10.4. *Calculate the storage required to supply the demand shown in the following table if the inflow of water to the reservoir is maintained at a uniform rate throughout 24 hours.*

Table 10.7

Time	*00—04*	*04—08*	*08—12*	*12—16*	*16—20*	*20—24*
Demand in million litres	*0.48*	*0.87*	*1.33*	*1.00*	*0.82*	*0.54*

(*Engineering Services, 1974*)

Solution. Total demand during the day = 5.04 million litres

(*i.e.* the summation of all the demands)

∴ Total supply during the day = Total demand

= 5.04 million litres

∴ Constant hourly supply = $\frac{5.04}{24}$ = 0.21 million litres

∴ 4 hourly supply = 0.21 × 4 = 0.84 million litres.

Now, using the given data, the analytical solution is carried out in table 10.8, which is self explanatory.

Table 10.8

Time in hrs.	*Demand in million litres*	*Cumulative demand in million litres*	*Pumping in million litres*	*Cumulative pumping in million litres*	*Excess of demand col. (3) – col. (5) +ve values only*	*Excess of supply; col. (5) – col. (3); + ve values only*
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*	*(7)*
0–4	0.48	0.48	0.84	0.84	Nil	**0.36**
4–8	0.87	1.35	0.84	1.68	Nil	0.33
8–12	1.33	2.68	0.84	2.52	0.16	Nil
12–16	1.00	3.68	0.84	3.36	**0.32**	Nil
16–20	0.82	4.50	0.84	4.40	0.10	Nil
20–24	0.54	5.04	0.84	5.04	Nil	Nil

From Table 10.8, it can be observed that

the maximum excess of demand = 0.32 million litres

and the maximum excess of supply = 0.36 million litres.

Hence, the total storage reqd. = 0.32 + 0.36

= **0.68 million litres. Ans.**

Example 10.5. *Water is pumped into a distribution reservoir from wells at a uniform rate of 1.3 cumecs. The estimated hourly water requirement for the*

worst day are given in Table 10.9. Determine mathematically, the capacity of the distribution reservoir in cubic metres required to fulfil these demands.

Table 10.9

Time in hours	*1*	*2*	*3*	*4*	*5*	*6*	*7*	*8*	*9*	*10*	*11*	*12*
Demand in million litres per hour	*2.5*	*2.3*	*2.2*	*2.1*	*2.3*	*2.5*	*3.0*	*4.5*	*5.9*	*6.3*	*6.4*	*6.3*
Time in hours	*13*	*14*	*15*	*16*	*17*	*18*	*19*	*20*	*21*	*22*	*23*	*24*
Demand in million litres per hour	*6.3*	*6.3*	*6.2*	*6.3*	*6.4*	*6.7*	*7.0*	*7.0*	*6.7*	*5.8*	*4.2*	*3.5*

Solution. Uniform pumping rate

= 1.3 cumecs (*i.e.* 1.3 m^3/sec)

= 1.3 × 60 × 60 × 1000 litres/hr. [∵ 1000 litres = 1 m^3]

$$= \frac{1.3 \times 36}{10} \text{ million litres/hr.} = 4.68 \text{ million litres/hr.}$$

The given data is now analysed in table 10.10, which is self-explanatory.

Table 10.10

Time in hr.	*Demand in million litres*	*Cumulative demand in million litres*	*Pumping in million litres*	*Cumulative pumping in million litres*	*Excess of demand in million litres col. (3) – col. (5) ; (+ve values only)*	*Excess of supply in million litres col. (5) – col. (3) ; (+ ve values only)*
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*	*(7)*
1	2.5	2.5	4.68	4.68	Nil	2.18
2	2.3	4.8	4.68	9.66	Nil	4.56
3	2.2	7.0	4.68	14.04	Nil	7.04
4	2.1	9.1	4.68	18.72	Nil	9.62
5	2.3	11.4	4.68	23.40	Nil	12.00
6	2.5	13.9	4.68	23.08	Nil	14.18
7	3.0	16.9	4.68	32.76	Nil	15.86
8	4.5	21.4	4.68	37.44	Nil	**16.04**
9	5.9	27.3	4.68	42.12	Nil	14.82
10	6.3	33.6	4.68	46.80	Nil	13.50
11	6.4	40.0	4.68	51.48	Nil	11.48
12	6.3	46.3	4.68	56.16	Nil	9.86
13	6.3	52.6	4.68	60.84	Nil	8.24
14	6.3	58.9	4.68	65.52	Nil	6.62

Contd...

(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*	*(7)*
15	6.2	65.1	4.68	70.20	Nil	5.10
16	6.3	71.4	4.68	74.88	Nil	3.48
17	6.4	77.8	4.68	79.56	Nil	1.76
18	6.7	84.5	4.68	84.24	0.26	Nil
19	7.0	91.5	4.68	88.92	3.58	Nil
20	7.0	98.5	4.68	93.60	4.90	Nil
21	6.7	105.2	4.68	98.28	6.92	Nil
22	5.8	110.2	4.68	102.64	**8.04**	Nil
23	4.2	115.2	4.68	107.64	7.56	Nil
24	3.5	118.7	4.68	112.32	6.38	Nil

From Table 10.10, it can be observed that :

The maximum excess of demand = 8.04 million litres

and, the maximum excess of supply = 16.04 million litres

Hence, the total storage reqd. = 8.04 + 16.04

= 24.08 million litres

$$= \frac{24.08 \times 10^6}{10^3} \text{ cu.m.}$$

= 24,080 cubic metres. Ans.

Example 10.6. *An overhead tank is to be provided for a city water supply. Given the following data, calculate the minimum capacity of the tank without any fire demand storage. Assume the tank to be empty between 12 and 15 hours. The draw-off factors, water pumped, and water consumed, etc. are shown in the table 10.11 below.*

Table 10.11

No.	*Time (hr)*	*Hours*	*Draw-off factor*	*Water pumped*	*Water consumed*	*Accumulation (+) Draw-off (–)*	*Water in service reservoir (S.R.)*
	(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*	*(7)*
A	*0—4*	*4*	*0.2*	*0*	*13,333*		
B	*4—6*	*2*	*1.2*	*50,000*	*40,000*		
C	*6—10*	*4*	*1.8*	*1,00,000*	*1,20,000*		
D	*10—12*	*2*	*1.0*	*50,000*	*33.333*		
E	*12—15*	*3*	*0.5*	*0*	*25,000*		
F	*15—16*	*1*	*0.5*	*25,000*	*8,333*		
G	*16—20*	*4*	*1.6*	*1,00,000*	*1,06,666*		
H	*20—23*	*3*	*1.0*	*75,000*	*50,000*		
I	*23—24*	*1*	*0.2*	*0*	*3,333*		

Compute columns 6 and 7 of the table to calculate the storage capacity of the tank.

Given : (*i*) *Average water supply* = *16,666 litres / hr.*
(*ii*) *Rate of pumping* = *25,000 litres / hr.*
(*iii*) *Hours of pumping* = *4–12 and 15–23.*

(*Engg. Services, 1981*)

Solution. The summation of the values given in col. (4) is 4,00,000 litres, which confirms 8 (*i.e.* 4—12 hrs) + 8 (*i.e.* 15—23) = 16 hours of pumping @ 25,000 litres/hr. The summation of values in col. (5) is 3,98,998, which is 2 less than, 4,00,000 ; and hence the given values in col. (5) need slight adjustment, to increase the total values by 2, which can be done by changing the value infront of *F* from 8,333 to 8,334, and the value infront of *I* to 3,334 from 3,333. The corrected values are shown in table 10.12.

The slightly adjusted and corrected values in col. (5) and those of col. (4) are then utilised to work out "Accumulation" values in col. (6), by subtracting col. (5) from col. (4), as shown in Table 10.12. The col. (6) is thus completed. Now, the value in col. (7), infront of S. No. *E* (*i.e.* between time 12—15 hr is now written as 0 (S.R. is empty during this interval). The values in this column are then worked out downward by adding the accumulation to the previous value in this col ; as shown in table, till we reach at *I*, and then proceed to S. No. *A*, *B*, *C* and *D* till we get 25,000 infront of S. No. *D*, and confirming 0 infront of S. No. *E*. The max. value in col. (7) is **35,000 litres,** which is the max. water stored in the service reservoir, and is hence the reqd. min. capacity of this reservoir.

Table 10.12

S. No.	Time	Hours	Draw off factor	Water pumped in litres	Water consumed (corrected and adjusted values in litres	Accumulation (+) Draw-off (–) col. (4) – col. (5) in litres	Water in service reservoir in litres
	(1)	(2)	(3)	(4)	(5)	(6)	(7)
A	0—4	4	0.2	0	13,333	(–) 13,333	18,333***
B	4—6	2	1.2	50,000	40,000	(+) 10,000	28,333
C	6—10	4	1.8	1,00,000	1,20,000	(–) 20,000	8,333
D	10—12	2	1.0	50,000	33,333	(+) 16,667	25,000
E	12—15	3	0.5	0	25,000	(–) 25,000	0
F	15—16	1	0.5	25,000	8,334	(+) 16,666	16,666*
G	16—20	4	1.6	1,00,000	1,06,666	(–) 6,666	10,000**
H	20—23	3	1.0	75,000	50,000	(+) 25,000	35,000
I	23—24	1	0.2	0	3,334	(–) 3,334	31,666
				Σ = 4,00,000	4,00,000	0	

*0 + 16,666 = 16,666
**16,666 – 6,666 = 10,000
***31,666 – 13,333 = 18,333.

Hence, the min. reservoir capacity reqd.
= 35,000 litres = **35 cum. Ans.**

Example 10.7. *A town with a population of one lakh is to be supplied with water daily at 200 litres per head. The variation in demand is as follows :*

6 am to 9 am	*— 40% of total*
9 am to 12 noon	*— 10% of total*
12 noon to 3 pm	*— 10% of total*
3 pm to 6 pm	*— 15% of total*
6 pm to 9 pm	*— 25% of total.*

Determine the capacity of the service reservoir assuming pumping to be at uniform rate and the period of pumping to be from 6 a.m. to 6 p.m. Neglect fire demand. Solve by the mass curve method. (*Engg. Services, 1982*)

Solution. The question is similar to *Q.* 10.2, and is solved below.

Total daily requirement
= 1,00,000 × 200 litres
= 2×10^7 litres = 20 M.l.

The cumulative demand is worked out in table 10.13.

Table 10.13

Period	Hours	Rate of demand	Demand in Ml	Cumulative demand in Ml
(1)	(2)	(3)	(4)	(5)
6 am to 9 am	3	40% of 20 Ml	8	8
9 am to 12 Noon	3	10% of 20 Ml	2	10
12 Noon to 3 pm	3	10% of 20 Ml	2	12
3 pm to 6 pm	3	15% of 20 Ml	3	15
6 pm to 9 pm	3	25% of 20 Ml	5	20
		Σ = 100%	20 Ml	

A graph is now plotted between time and cumulative demand, to obtain the demand mass curve, as shown by firm line in Fig. 10.19. The total supply is obtained in 12 hours (*i.e.* from 6 am to 6 pm) by pumping at a uniform rate, *i.e.* at the rate of

$$= \frac{20}{12} \text{ Ml/hr} = 1.667 \text{ Ml/hr.}$$

Time of the day in hr.

The supply line on the mass diagram will therefore be a st. line with its slope as 1.667 Ml/hr, between 6 AM to 6 PM, as shown by dotted

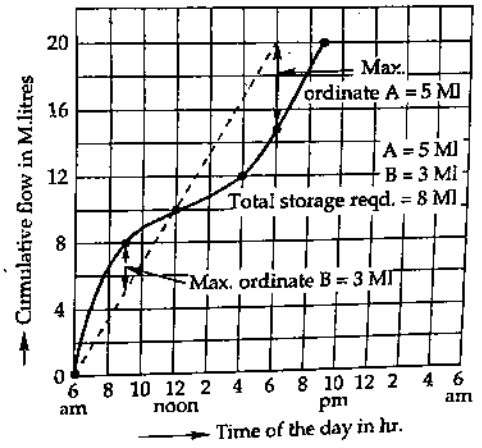


Fig. 10.19

line in Fig. 10.19. The two maximum ordinates A and B, enclosed between the demand and the supply lines are read out from Fig. 10.19 as :

$A = 5$ Ml, $B = 3$ Ml

Total storage reqd = 8 Ml **Ans.**

Alternatively, the question can be solved by mathematical analysis as given in table 10.14.

Table 10.14

Time	*Demand Ml*	*Cumulative demand Ml*	*Supply @ 5/3 Ml per hr i.e. 5 Ml per 3 hr*	*Cumulative Supply Ml*	*Excess of demand col. (3) –col. (5) +ve values only*	*Excess of supply in Ml col. (5) –col. (3) +ve values only*
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*	*(7)*
6 am–9 am	8	8	5	5	**3**	—
9 am–12 N	2	10	5	10	—	—
12 N–3 pm	2	12	5	15	—	3
3 pm—6 pm	3	15	5	20	—	**5**
6 pm–9 pm	5	20	Nil	20	—	—

From table 10.14,

The max. excess of demand = 3 Ml and

The max. excess of supply = 5 Ml

Hence, total storage reqd. = 3 + 5 = **8 Ml. Ans.**

Example 10.8. *A clear water tank is receiving water from the treatment plant at a rate of 200 m^3/hr for 24 hours. The high lift pumps are lifting water from the same tank at the following rates : 4—14 hrs @ 120 m^3/hr, 15—24 hrs @ 400 m^3/hr. Determine the capacity required for the clear water tank.*

(Civil Services, 1982)

Solution. The given data is analysed in table 10.15.

Table 10.15

Time in hr	*Demand from clear reservoir, lifted by high lift pumps in m^3*	*Cumulative demand from clear reservoir in m^3*	*Supply in the clear reservoir in m^3 @ 200 m^3/h for 24 h*	*Cumulative supply in m^3*	*Excess of supply in m^3 col. (5)– col. (3) +ve values only*	*Excess of Demand in m^3 col. (5)– col. (3) +ve values only*
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*	*(7)*
0–4	Nil	Nil	200 × 4 = 800	800	800	—
4–14	120 m^3/h × 10 h = 1200	1200	200 × 10 = 2000	2800	1600	—
14–15	Nil	1200	200 × 1 = 200	3000	**1800**	—
15–24	400 m^3/h × 9 h = 3600	4800	200 × 9 = 1800	4800	—	—

From table 10.15, it can be observed that :

The max. excess of demand from the clear water reservoir = Nil

and

The max. excess of supply to the clear water reservoir = 1800 m^3

Hence, the total storage reqd. = 0 + 1800

= **1800 m^3. Ans.**

(2) **Breakdown Storage.** The breakdown storage or often called *emergency storage* is the storage preserved in order to tide over the emergencies posed by the failure of pumps, the electricity or any other mechanism driving the pumps. The amount of provision to be made for this factor is very difficult to assess, because it depends upon the frequency and extent of the failures, and also upon the time required for carrying out the repairs. For this reason, a lump sum provision is generally made for this storage. A value of about 25 per cent of total storage capacity of the reservoir, or about $1\frac{1}{2}$ to 2 times of the average hourly supply, may be considered as enough provision for accounting this storage, under all normal circumstances. However, if the supply lines of equipments are expected to be out of operation for long times, higher allowances must be made.

(3) **Fire Storage.** The third component of the total reservoir storage is the fire storage. This provision takes care of the requirement of water for extinguishing fires.

In case of fires, sufficient amount of water must remain available in the reservoir for throwing it over the fire, in case the fire coincides with pumps or electricity failure at the waterworks.

The fire fighting requirements were discussed in details in chapter 2 in article 2.1. It was mentioned there that the total volume of water required for fire fighting is generally small, say of the order of 1 to 5 litres per day per person. It was also mentioned that for extinguishing a fire, three jet streams each giving a discharge of about 1100 litres/minute must be thrown on the fire. Thus, if a provision for 10 hours of fire fighting per day is desired, the volume of water required to be stored

$= (3 \times 1100) \times (10 \times 60)$ litres/day

$= 198 \times 10^4$ litres/day

say about 2 million litres/day.

The storage for fire fighting, in fact, is dependent upon the chances of fire break-outs in the considered city. The National Board of Underwriters of America recommends provisions for extinguishing a serious fire of 10 hours at a rate of about 10,000 to 50,000 litres/minute for towns with populations more than 6000 and upto 2 lakhs. For smaller communities with populations of 4000, 2000 and 1000, the provision may be made for extinguishing a serious fire of 8, 6 and 4 hours, respectively. These provisions *are too high* to be adopted by a poor developing country like India. *Under normal conditions in India, we may store about 1 to 4 litres per person per day as the necessary fire storage, depending upon the importance of the city.*

The *total reservoir storage* can finally be obtained by adding all the three storages, *viz*, balancing storage, emergency storage, and fire storage.

10.12. Location and Height of the Distribution Reservoirs

The following points must be thoroughly considered before deciding the locations of the distribution reservoirs :

(*i*) They should be located in the heart of the city, so as to command the maximum area all around. Also, they should preferably be placed as close as possible to the points of heaviest demand. Such locations will ensure lesser head loss due to friction, because water from the distribution reservoir will have to travel smaller distances in reaching the different points of the distribution network.

(*ii*) They should be located at high elevations, so that adequate pressure is maintained in the distribution system. They should, therefore, be located at the natural ridges of the area, if they suit otherwise.

(*iii*) With respect to the position of the pumping stations and the distribution area, the reservoirs may be located in two different ways ; *viz.*, (*a*) they can be placed between the pumping station and the distribution area ; (*b*) they can be placed at the farther end of the distribution area, as shown in Fig. 10.20. In the former case, the elevation of the reservoir has to be high and the water to be supplied will first always pass through the reservoir, while in the latter case, the elevation can be low and the supply can be made available directly from the pumps as well as from the reservoir, during high demand. The length of the rising main carrying the water from the pumping station (i.e. waterworks) to the reservoir will be less in the former case, and *vice versa*. Since the pipe length in the latter case is more, the head loss will be more. Thus, the total head against which the pumping has to be done will remain the same in both the cases. The hydraulic gradient line for the functioning of the reservoir of (*b*) type is shown in Fig. 10.20.

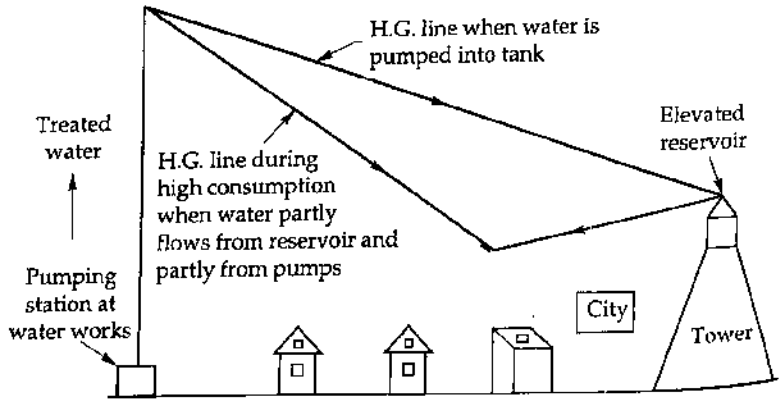


Fig. 10.20. Functioning of the elevated reservoir located at the farthest end of the distribution area.

Example 10.9. *A water supply system consisting of a reservoir with lift pumps, elevated storage tank, piping, and withdrawal points is shown in the diagram (Fig. 10.21 a).*

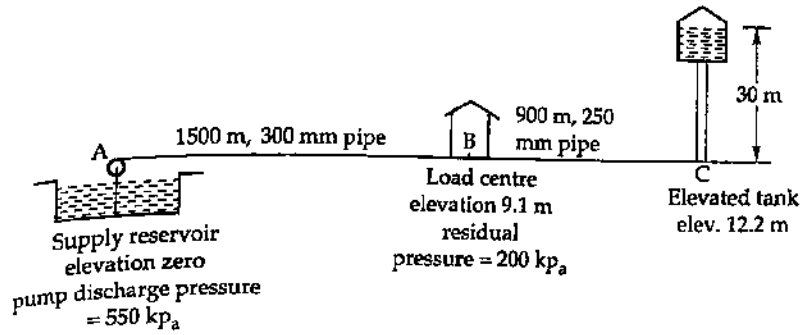


Fig. 10.21 (*a*)

(*i*) *Based on the following data, sketch the hydraulic gradient for the system.*

$Z_A = 0$ *m*, $P_A = 550$ *kPa*, $Z_B = 9.1$ *m*, $P_B = 200$ *kPa*,

$Z_C = 12.2$ *m*, $P_C = 30$ *m (water level in the tank).*

(ii) For these conditions, compute the flow available at point B from both the supply pumps and elevated storage. Use C = 100 and pipe sizes as shown in the diagram. Hydraulic values are given below.

Table of Hydraulic Values for C = 100

d (mm)	h_L *(m/1000)*	*Q (l/s)*	*V (m/s)*
300	*17.8*	*133*	*1.7*
250	*14.1*	*117*	*1.5*
200	*20.0*	*48*	*1.6*
150	*30.0*	*30*	*1.7*
100	*40.0*	*13*	*1.5*

(*Engg. Services, 1980*)

Solution. The pressure given at A and B is in S.I. units of pressure. The S.I. unit of pressure is kilo pascal (kPa). 1 pascal is equal to 1 N/m^2.

$\therefore$ $1 \text{ kPa} = 1 \text{ kN/m}^2$

But 1 m of water head $= 9.81 \text{ kN/m}^2$

$\therefore$ $1 \text{ kN/m}^2 = \dfrac{1}{9.81}$ m of water head

$\therefore$ $100 \text{ kPa} = \dfrac{100}{9.81}$ m of water head

$= 10.1937$ m of water head.

In the given question,

Pressure at A $= P_A = 550$ kPa

$= \dfrac{550}{100} \times 10.1937$ m (of water)

$= 56.065$ m ; say **56.07 m**

Similarly, the residual pressure at B

$= P_B = 200$ kPa

(*iv*) **By using waste detection meters.** The name given to these meters is 'misleading' because these meters do not measure the actually wasted water, but only measure any unusually high flow passing through a water main during the period of low consumption, such as during night or early hours of morning. This *unnatural excess flow* from a portion of the pipe will indicate the leakage of the water through that section of the pipe, provided it is reasonably established that no unusual high consumption has actually occurred in the nearby area. This test is generally conducted at night times. The method consists in, first of all isolating a particular suspected locality, and closing down the supplies of all areas except that of the head of the chosen locality. The meter is then fitted at the head of the main supply pipe, and water is thus allowed to flow into the locality after getting recorded in the meter*. Now, the various *laterals* or the *branches* of the locality are progressively shut off. The recorded flow through the meter will, therefore, drop at each shut off. The shut off which indicates a large and a disproportionate drop in the recorded flow (when compared to the size of the pipe and consumption in the area being served by that pipe) will indicate, that the particular pipe is leaking. Steps can then be taken to rectify the defects.

Deacon's waste water detection meters are widely used, as they are quite sensitive and accurate. The water passing through such a meter forces down a brass disc otherwise balanced by a counter weight. This movement of the disc is conveyed through a rod to a pencil point which moves vertically up and down the graph paper mounted on a drum and moving horizontally with time. The quantity of the water flowing through the meter automatically gets recorded. The graph paper can be removed for analysis after the test is performed.

The above method of using waste detection meters for detecting leakage in the distribution pipes is quite scientific and useful, and hence widely used. The method is, however, costlier because of higher operational crew charges, and the necessity of installing additional valves and cutouts for inserting meters in the pipe lines.

DESIGN OF DISTRIBUTION NETWORKS

Depending upon the various factors, such as the relative levels of the different zones of the city, the layout of the roads, etc., the type of the distribution network (*i.e.* the arrangement of the distribution pipes) to be adopted is first of all, decided. The four important types of the network, namely, the dead-end system, the grid-iron system, the circle or ring system and the radial system have already been discussed in this chapter. Any one of them, or more commonly a combination of the two or more, may be adopted for a particular town or a city.

10.14. Fixing the Sizes of Pipes of a Simple Distribution System

After the layout plan of the distribution pipes and the locations of other appurtenances (such as valves, hydrants, etc.) is finalised, the *sizes of the pipes* are computed. These sizes should be such as to ensure availability of

*The meter is provided with a pencil point and a square paper wrapped round a drum, so as to automatically register the flows at different times.

water at the end points of the pipes, with the minimum allowable pressure at the time of maximum demand. The *terminal pressures* in all the pipes should, therefore, remain above the *minimum allowable pressure* chosen for the design, and should also not exceed the *maximum allowable pressure* chosen for the design of the distribution system.

No direct method is available to compute the pipe sizes of a distribution network, and hence a *trial procedure* is adopted, wherein the sizes are assumed and terminal pressures are computed to compare them with the designed minimum and maximum pressures. If the required matching is not obtained, then the sizes are changed, and the procedure is repeated till the required matching is obtained.

In order to compute the terminal pressures in all the pipes, the head loss through each pipe will need evaluation. The head loss in a pipe can be easily computed by Darcy Weisbatch or Hazen William formula, provided the discharge flowing through the pipe is known. Hence, the head losses through the pipes can be computed if the discharges flowing through the different pipes are known or are computed by equating the inflow and outflow at each point of the pipe network.

Strictly speaking, the total head loss in a pipe consists of the head lost in the pipe friction (H_L), and the *minor losses** in valves and fittings, etc. These minor losses are usually ignored unless the computers are available for computations. The frictional head loss (H_L) given by the Hazen-William's formula is, thus, considered to be equal to the total head loss in the pipe. This loss is given by eqn. (6.12) and (6.12 *a*), as :

$$V = 0.85\, C_H \,.\, R^{0.63} \,.\, S^{0.54} \qquad \text{(Refer eq. 6.12)}$$

$$\text{where } S = \frac{H_L}{L}$$

and

$$H_L = \frac{1}{0.094}\left(\frac{Q}{C_H}\right)^{1.85} \frac{1}{d^{4.87}} \qquad \text{(Refer eq. 6.12 } a\text{)}$$

This formula may be applied for calculating the head loss in each pipe of a trial diameter. The residual pressures at the terminal points can finally be worked out, and compared with the designed minimum and maximum allowable pressures, to finalise the pipe sizes.

However, since the design involves the use of various trial values of the pipe diameters, and that too for various pipes of the network, it is often convenient to use a **Hazen-William's chart**, such as shown in Fig. 10.22 instead of using the formula. This will avoid cumbersome calculations. Since the pipes of the distribution system are made of cast iron, for which $C_H = 100$, the chart given in Fig. 10.22 has been drawn for a C_H value of 100. Such charts are available in the design office and can be conveniently used. The method of using this chart is simple. A straight edge is placed on any two known values, such as, discharge and diameter**, and the two other unknown values (such as loss of head per 1000 m, and velocity) can be directly read out. The loss of head per 1000 m so obtained when multiplied by the length of the pipe in thousand

*Discussed earlier in chapter 6 in Article 6.4, Table 6.3.

**Pipe diameter is assumed within 10 to 40 cm to obtain optimum velocity.

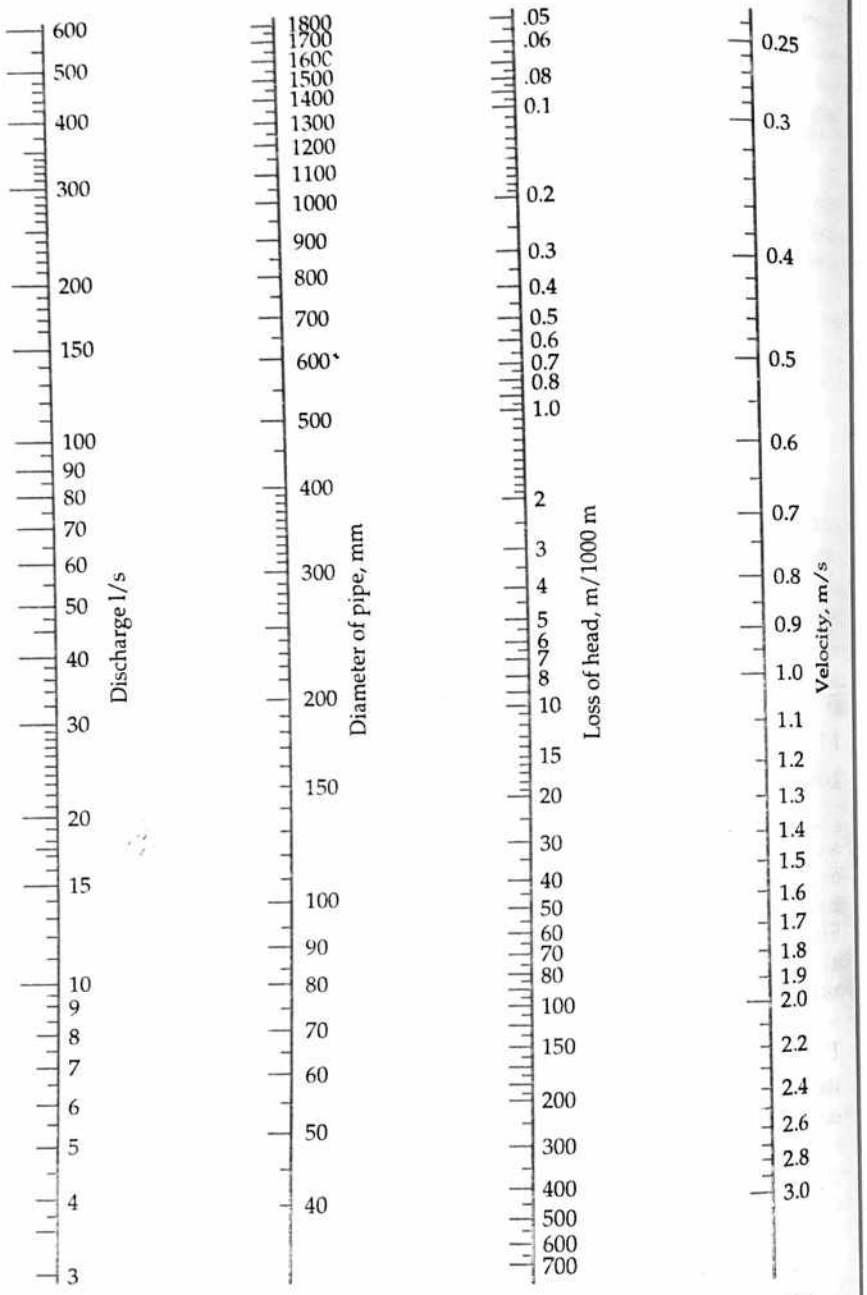


Fig. 10.22. Hazen's-William's Nomogram for Cast iron pipes (Eq. 6.12 for $C_H = 100$).

metres will indicate the head loss in the pipe of that diameter. The calculated head losses in different pipes will then be used in working out the terminal pressures by starting from the reservoir or pumping end (where the pressure is known). If the terminal pressure in any zone is found to be less than the

minimum permissible or more than the maximum permissible, then the size of the pipe leading to that zone is suitably increased or decreased, *and the whole system is again analysed*. The process is continued till the required pressures are obtained.

Efforts are made to achieve the optimum velocities varying between 0.9 to 1.8 m/sec for pipe diameters varying between 10 cm to 40 cm, as shown in table 10.16*. The pipe diameters are therefore assumed by intelligently looking at the monogram, as to obtain optimum velocity for the given discharge and the assumed diameter.

Table 10.16. Optimum velocities in pipes of different sizes*

Pipe dia in cm.	*Approximate velocity in m/sec*
10	0.9
15	1.2
25	1.5
40	1.8

*Strictly speaking, the distribution mains should be designed for carrying the maximum hourly draft of the maximum day (i.e. about 2.7 times the average daily demand), or for the coincident draft with fire, whichever is more (as discussed in article 2.6.1 and 2.6.2). However, for normal cases, the distribution pipes are designed for a maximum demand of 3 times the average ; and the service pipes in the streets are designed for 2 times the average. The minimum size of the distribution pipes is kept 10 cm*** and that of the service pipes as 19 mm ($\frac{3}{4}''$), except when the pressure is very high when it may be kept as 13 mm ($\frac{1}{2}''$).

The simple method of calculating the head losses in the different pipe sections, and then calculating the terminal pressures at various points by moving from the known pressure point, can be used easily for solving the simple distribution networks (such as those of dead-end system). An example has been solved below on such a network. However, for complicated networks, in which the water at different points reaches from the different places (such as in the grid iron system), other methods of analysis, such as *Equivalent pipe method, Method of sections*, the *Hardy Cross method*, etc. will have to be used. These methods are described in article 10.15.

Example 10.10. *A dead end type of a distribution pipe network has been adopted for supplying water to a certain zone of the city having different blocks and their populations, as shown in Fig. 10.23. The average rate of supply being 200 litres per head per person. Design the sizes of the distribution pipes with the following assumptions :*

(i) The populations in different blocks may be assumed as shown as P in Fig. 10.23.

*If velocities are kept higher in smaller pipes, excessive head loss will occur.

Even a 10 cm distribution pipe is rarely adopted, because such a pipe will though provide enough water for domestic needs of 1000 persons, yet can not supply water to even a single fire hose. Hence, **15 to 20 cm dia is generally used as the minimum size for the distribution pipes.

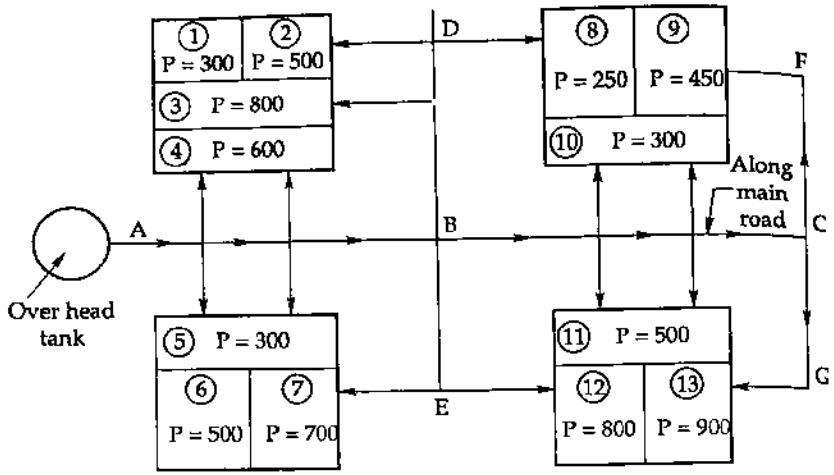


Fig. 10.23

(ii) The R.L. of the bottom of the storage tank may be taken as 225.00 m.

(iii) The R.L. of points along main road be taken as :

A—210.00 m, B—200.00 m, C—195.00 m.

(iv) The lengths AB and BC may be taken as 500 m and 400 m respectively.

(v) The distribution system may be designed for a maximum demand equal to 2.7 times the average demand.

(vi) The minimum pressure head to be maintained at any point in the distribution system may be assumed as 12 m.

Solution. The main pipe line *AC* is having two portions, *i.e. AB* and *BC*, for which the sizes are to be determined. Now, starting backward from the point *C* towards the overhead tank at *A*, we will work out the population served by each portion (*e.g. CB* and *BA*) of the pipe line *AC*, as shown in Fig. 10.24 and col. (4) of Table 10.17.

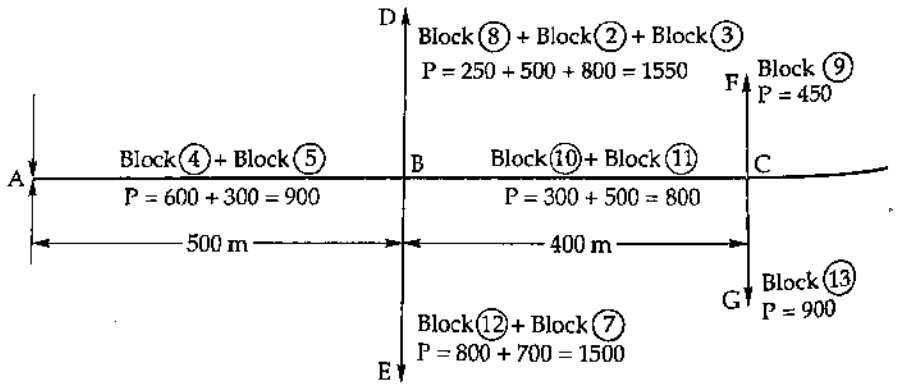


Fig. 10.24

The calculations are carried out in Table 10.17. The maximum demand *i.e.* design discharge (taken as 2.7 times the average) is worked out in col. (5) for

Table 10.17. Computation Table for Example 10.10 (First and Last Trial)

Pipe Section	Population served: Previous	Population served: Local	Population served: Total P	Max. demand = design discharge $= \frac{2.7 \times 200 \times P}{24 \times 60 \times 60}$ litres / sec.	Assumed dia. of pipe in mm	Head loss: Rate per 1000 m*	Head loss: Length in metres	Head loss: Rate × length = Actual head loss	Hydraulic level worked out from the last point i.e. pt A in metres	Ground level in metres	Net head available in metres at the pt indicated in col. (10) = [Col. (10) − Col. (11)]
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
CB (i.e. BC)	450 + 900 = 1350	800	2150*	13.4	120	20	400	8	*C*–207.50***	195.00	12.5, O.K. being less than the min reqd. value of 12.0 m
BA (i.e. AB)	2150* + 1550 + 1500 = 5200	900	6100	38.1	180	19.0	500	9.5	*B*–215.50** **A–225.00** (given)	*B*–200.00 *A*–210.00	15.5, O.K. being less than 12.0. 15.0, O.K. being less than 12.0

*Read out from the Monogram (Fig. 10.22) for the discharge value of col. (5) and the dia value of col. (6)

**225.00 − 9.5 = 215.50

***215.50 − 8 = 207.50

the population of col. (4). The sizes of pipes are now assumed in col. (6) as to obtain optimum velocities. The rate of loss of head per 1000 m is then read out from the Hazen's nomogram (Fig. 10.22) for the discharge values of col. (5) and the dia values of col. (6), and are tabulated in col. (7). The actual loss in a certain given length is then determined in col. (9). The hydraulic levels of different points are then worked out by starting from the last point A, after completing the calculations for both the pipes AB and BC. They are compared with the actual ground levels, and the available pressure heads are worked out in col. (12).

Since the available pressure heads computed in col. (12) are more than the minimum required value of 12 m, the assumed values of dia of both pipes (AB and BC) are O.K., and hence may be adopted. Hence, use 18 cm dia pipe for length AB, and 12 cm dia pipe for length BC. **Ans.**

Note : If the available head (with 120 mm dia BC pipe) at point C would have been less than the minimum permissible head of 12.0 m (given), then the pipe dia for the length BC should have been increased from a value of 120 to 140 mm (say) in the second trial, so as to reduce the head loss in this pipe, needing revised calculations, till the results are found to be O.K.

10.15. Analysis of Complex Pipe Networks

In any pipe network, the following two conditions must be satisfied :

(*i*) *The algebraic sum of the pressure drops around a closed loop must be zero, i.e.* there can be no discontinuity in pressure.

(*ii*) *The flow entering a junction must be equal to the flow leaving the same junction ; i.e.* the law of continuity must be satisfied.

Based upon these two basic principles, the pipe networks are generally solved by the methods of successive approximation, because any direct analytical solution is not possible, as the same will involve various equations to be solved simultaneously and many of which are non-linear. The important procedures or methods used for such solutions are given below :

10.15.1. Hardy-Cross Method. The procedure suggested by Hardy and Cross requires that the flow in each pipe is assumed by the designer (in magnitude as well as in direction) in such a way that the principle of continuity is satisfied at each junction (*i.e.* the inflow at any junction becomes equal to the outflow at that junction).*

A correction to these assumed flows is then computed successively for each pipe loop in the network, until the correction is reduced to an acceptable magnitude.

If Q_a is the assumed flow and Q is the actual flow in the pipe, then the correction Δ is given by

$$\Delta = Q - Q_a$$

or $$Q = Q_a + \Delta.$$

Now, expressing the head loss (H_L) as

$$H_L = K.Q^x$$

*Since the method involves balancing the quantities of inflow and outflow at each junction, it is also called as the **quantity balanced method**.

we have, the head loss in a pipe

$$= K \,.\, (Q_a + \Delta)^x$$

$$= K \,.\, \left[Q_a^{\,x} + x \,.\, Q_a^{\,x-1}\, \Delta + \text{..... Negligible terms containing higher powers of } \Delta \right]$$

$$= K \left[Q_a^{\,x} + x Q_a^{\,x-1}\, \Delta \right]$$

Now, around a closed loop, the summation of head losses must be zero.

$$\therefore \quad \Sigma K \left[Q_a^{\,x} + x \,.\, Q_a^{\,x-1} \,.\, \Delta \right] = 0$$

or $$\Sigma K.\, Q_a^{\,x} = -\, \Sigma K x\, Q_a^{\,x-1} \,.\, \Delta$$

Since Δ is the same for all the pipes of the considered loop, it can be taken out of the summation. Therefore,

or $$\Sigma K.Q_a^{\,x} = -\, \Delta \,.\, \Sigma K x\, Q_a^{\,x-1}$$

or $$\Delta = -\, \frac{\Sigma K \,.\, Q_a^{\,x}}{\Sigma x \,.\, K Q_a^{\,x-1}}$$

Since Δ is given the same sign (or direction) in all pipes of the loop, the denominator of the above equation is taken as the absolute sum of the individual items in the summation. Hence

or $$\Delta = -\, \frac{\Sigma K \,.\, Q_a^{\,x}}{\Sigma \left| x \,.\, K Q_a^{\,x-1} \right|} \qquad \text{...(10.1)}$$

or $$\Delta = \frac{-\, \Sigma H_L}{x \,.\, \Sigma \left| \dfrac{H_L}{Q_a} \right|} \quad (\because \; KQ^x = H_L \text{ and } x = \text{constant}) \qquad \text{...(10.2)}$$

where H_L = head loss for the assumed flow Q_a

The numerator of the above equation is the algebraic sum of the head losses in the various pipes of the closed loop computed with the assumed flow. Since the direction and magnitude of flow in these pipes is already assumed, their respective head losses with due regard to sign* can be easily calculated after assuming their diameters. The absolute sum of respective $K.Q_a^{\,x-1}$ or H_L/Q_a is then calculated. Finally, the value of Δ is found out for each loop, and the *assumed flows in each pipe are corrected by using eqn. (10.2). Pipes common to two loops will receive both corrections with due attention to sign.*

After correcting the flows in the entire pipe network once, the second correction can be applied to the once corrected flows, and the recorrected flows are again worked out in the entire network (consisting of one or more loops). *The flows in pipes, common to two loops, should be corrected for the computed corrections of both the loops, as stated earlier.* The procedure can be repeated to obtain more accuracy and precision.

The value of x in Hardy Cross method is assumed to be constant (*i.e.* 1.85 for Hazen-William's formula, and 2 for Darcy-Weisbach formula) and is not varied as the precise application of Darcy-Weisbach formula would require. The minor losses are generally neglected, although they can be introduced by

*The head loss in clockwise direction may be taken as + ve, and that in the anti-clockwise direction as – ve.

substituting an equivalent length of the pipe. While analysing the given distribution system, *the system must be divided into two or more loops, such that each pipe in the network is included in atleast one loop.*

The entire procedure to be adopted in Hardy Cross method can be summed up as follows :

(1) Assume any internally consistent distribution of flow. The sum of the flows entering any junction must equal the sum of the flows leaving that junction.

(2) Compute the head loss in each pipe by means of an equation or diagram. Conventionally, clockwise flows are positive and produce positive head loss, and *vice versa*.

(3) With due attention to sign, compute the total head loss around each circuit, *i.e.* $\Sigma K \,.\, Q_a^{\,2}$.

(4) Compute without regard to sign, for the same circuit, the sum of $\Sigma x.K.Q_a^{\,x-1}$.

(5) Apply the corrections obtained from Eq. (10.1) or (10.2) to the flow in each pipe. *Pipes common to two loops will receive both corrections with due regard to sign.*

A few examples have been solved below to illustrate the use of this method, since this method is the most important and widely used for the design of distribution systems.

Example 10.11. *Determine the distribution of flow in the pipe network shown in Fig. 10.25. The head loss, h_L, may be assumed as KQ^n. The flow is turbulent and pipes are rough. The value of K for each pipe is indicated in the figure. Use Hardy-Cross method.*

(*Engg. Services 1996*)

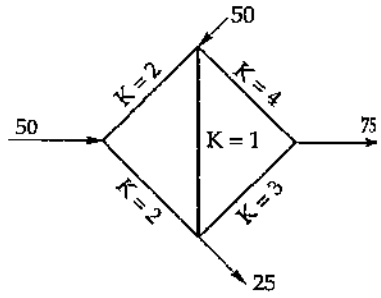


Fig. 10.25

Solution. First of all, the magnitudes as well as directions of the possible flows in each pipe are assumed keeping in consideration the law of continuity at each junction (*i.e.* the input equals the off-take at each junction). These assumed flows are shown in Fig. 10.26.

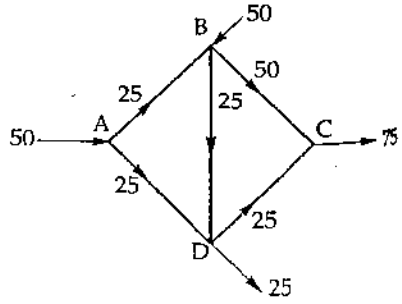


Fig. 10.26

The head loss in each pipe is to be taken as equal to $K \,.\, Q^n$. Assume $n = 2$ for using Manning's formula (Eq. 6.11) or Darcy-Weisbach's formula (Eq. 6.1) for rough pipes and turbulent flow. Hence,

$$H_L = K \,.\, Q^2$$

The various K values for different pipes are given in the figure (Fig. 10.25), which can be used while analysing the two loops ; *i.e.* loop *ABDA* and loop *BCDB*. Both these loops are now analysed by Hardy Cross procedure in Table 10.18.

Table 10.18. Hardy Cross Procedure for 1st Correction

Pipe	*Assumed Flows* Q_a *l/s*	*K (given)*	$H_L = K.Q_a^{\,2}$	$\left\lvert \frac{H_L}{Q_a} \right\rvert = \frac{col.(4)}{col.(2)}$	*Corrected Q after first correction* $= Q_{a_1} = Q_a + \Delta_1$* $= col.(2) + \Delta_1$ *l/s*
(1)	(2)	(3)	(4)	(5)	(6)
For Loop ABDA (Loop I)					
AB	25	2	1250	50	(+) 22.5
BD (Common pipe)	25	1	625	25	+ 35.0*
DA	– 25	2	(–) 1250	50	(–) 27.5
		Σ	(+) 625	125	

*where $\Delta_1 = \dfrac{(-)\,\Sigma H_L}{x \,.\, \Sigma\left(\dfrac{H_L}{Q_a}\right)}$ ... (*i.e.* Eq. 10.2)

or $\Delta_1 = \dfrac{625}{2 \times 125} = (-)\,\mathbf{2.5}$

For Loop BCDB (Loop II)					
BC	50	4	10000	200	37.5
CD	– 25	3	(–) 1875	75	(–) 37.5
DB (Common pipe)	– 25	1	(–) 625	25	(–) 35.0*
		Σ	7500	300	

Δ_1' in 2nd Loop $= (-)\,\dfrac{7500}{2 \times 300} = (-)\,\mathbf{12.5}$

The corrected flows after 1st correction in various pipes of both the loops are thus worked out in col. (6) of Table 10.19 as :

*Corrected flow in common pipe BD
$= 25 + \Delta_1 + \Delta_1'$
where Δ_1 for pipe BD = (–) 2.5 ; and Δ'_1 for pipe DB is (–) 12.5 and hence Δ_1' for pipe BD will be (+) 12.5.

$\therefore$ $Q_{BD} = 25 - 2.5 + 12.5 = \mathbf{35}$ **l/s.**

Flows after first correction

Pipe	Corrected discharge after first correction l/s
For Loop ABDA	
AB	22.5
BD	35.0
(Common pipe)	
DA	(–) 27.5
For Loop BCDB	
BC	37.5
CD	(–) 37.5
DB	(–) 35.0
(Common pipe)	

These discharges are now again used to reanalyse both the loops for the second correction in Table 10.19.

Table 10.19. Hardy Cross Procedure for Second Correction

Pipe	Assumed Flows (Q_{a_1}) l/s	K	$H_L = K.Q_a^2$	$\left\|\frac{H_L}{Q_a}\right\| = \frac{col.(2)}{col.(4)}$	Corrected flows after 2nd correction ; $Q_{a_2} = Q_{a_1} + \Delta_2$ col. (2) + Δ_1
(1)	(2)	(3)	(4)	(5)	(6)
For Loop ABDA (Loop I)					
AB	22.5	2	1012.5	50	19.8
BD	35.0	1	1225.0	25	32.6*
(Common pipe) *DA*	(–) 27.5	2	(–) 1512.5	50	(–) 30.2
		Σ	725	135	
$\Delta_2 = \frac{(-)\,725}{2 \times 135} =$ **(–) 2.7**					
For Loop BCDB (Loop II)					
BC	37.5	4	5625	150	37.2
CD	(–) 37.5	3	(–) 4218.75	112.5	(–) 37.8
DB (Common pipe)	(–) 35.0	1	(–) 1225	35	(–) 32.6*
		Σ	181.25	297.5	

Δ_2 in 2nd Loop = $(-)\frac{181.25}{2 \times 297.5}$ = **(–) 0.3**

*Corrected flow in common pipe *BD*
= 35 – 2.7 – (– 0.3) = 35 – 2.7 + 0.3 = **32.6 l/s**

The corrected flows obtained in col. (6) of Table 10.20 after 2nd correction are now plotted on the pipe network in Fig. 10.27 to check balancing of flow at each point, to verify the answer.

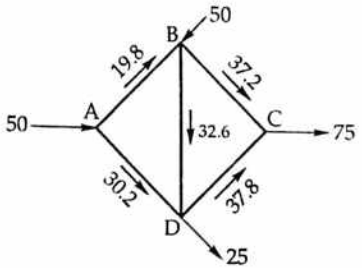


Fig. 10.27 (Corrected flows).

Example 10.12. *Calculate the head losses and the corrected flows in the various pipes of a distribution network shown in Fig. 10.28. The diameters and the lengths of the pipes used are given against each pipe. Make use of Hardy-Cross method with William Hazen's formula. Compute the corrected flows after two corrections.*

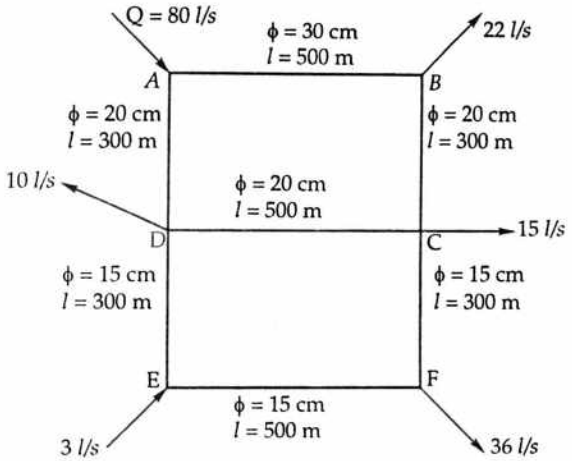


Fig. 10.28

Solution. First of all, the magnitudes as well the directions of the possible flows in each pipe are assumed keeping in consideration the law of continuity at each junction (*i.e.* the input equals the off take at each junction). These assumed flows are shown in Fig. 10.29. The two closed loops, *i.e. ABCD* and *DCFE* are then analysed by Hardy-Cross method.

The analysis of the pipe loops will require computation of head loss (H_L) in each pipe, which is to be computed by William Hazan's formula given by Eq. 6.12 (*a*) as :

$$H_L = \frac{1}{0.094} \cdot \left(\frac{Q}{C_H}\right)^{1.85} \cdot \frac{L}{d^{4.87}}$$

where $C_H = 100$ (assumed)

$$H_L = \frac{1}{0.094} \cdot \left(\frac{Q_a}{100}\right)^{1.85} \cdot \frac{L}{d^{4.87}}$$

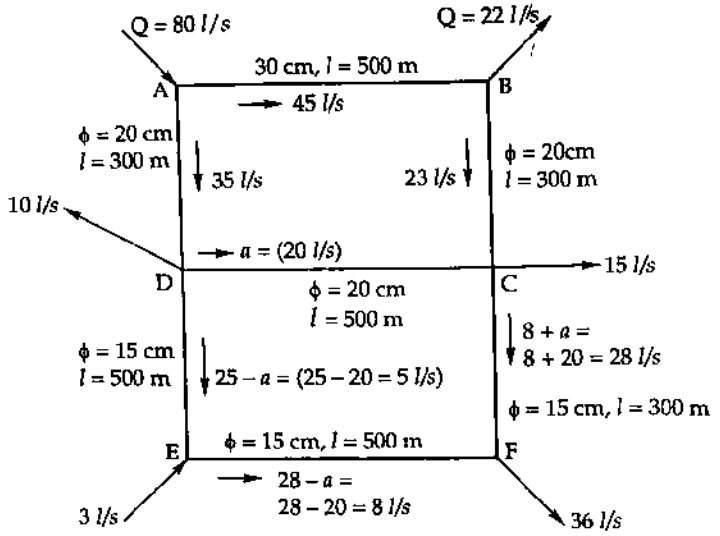


Fig. 10.29

$$= \frac{L}{470} \cdot \frac{Q_a^{1.85}}{d^{4.87}}$$

$$= K \cdot Q_a^{1.85}$$

$$\text{where } K = \frac{L}{470\, d^{4.87}}$$

For all the pipes of both the loops, we first compute K values, as given in table 10.20 below :

Table 10.20. Computation of K values for Pipes of Network of Fig. 10.28

Pipe	*L in m*	*d in m*	$K = \frac{L}{470 d^{4.87}}$
For Loop ABCD			
AB	500	0.30	373
BC	300	0.20	1,615
CD	500	0.20	2,690*
DA	300	0.20	1,615
For Loop DCFE			
DC	500	0.20	2,690*
CF	300	0.15	6,580
FE	500	0.15	10,940
ED	300	0.15	6,580
			*Common pipe

Now, both the loops are analysed for Hardy Cross procedure in table 10.21.

Table 10.21. Hardy Cross Procedure for First Correction

Pipe	*Assumed flows in*		*K*	$H_L = K.Q_a^{1.85}$	$\left\lvert \frac{H_L}{Q_a} \right\rvert$	*Corrected flows after 1st correction* Q_{a_1} = col. (2) + Δ_1 *l/s*
	l/s	m^3/s	*Value from table 10.21*		$= \left\lvert \frac{\text{col. (5)}}{\text{col. (3)}} \right\rvert$	
(1)	(2)	(3)	(4)	(5)	(6)	(7)
For Loop ABCD						
AB	45	(+) 0.045	373	(+) 1.12	26	49.9
BC	23	(+) 0.023	1615	(+) 1.52	66	27.9
CD (Common Pipe)	(–) 20	(–) 0.020	2690	(–) 1.94	97	(–) 7.9*
DA	(–) 35	(–) 0.035	1615	(–) 3.23	92	(–) 30.1
			Σ	(–) 2.53	281	

$$\Delta_1 = \frac{(-)\,\Sigma H_L}{x \,.\, \Sigma \left\lvert \frac{H_L}{Q_a} \right\rvert} = (-)\, \frac{-2.53}{1.85 \times 281} \text{ cumecs}$$

$$= \frac{+2.53 \times 1000}{1.85 \times 281} \text{ l/s} = \mathbf{4.9\ l/s}$$

For Loop DCFE						
DC	20	(+) 0.020	2690	(+)1.94	97	7.9*
CF	28	(–) 0.028	6580	(+) 8.80	314	20.8
FE	(–) 8	(+) 0.008	10940	(–) 1.47	184	(–) 15.2
ED	(–) 5	(–) 0.005	6580	(–) 0.37	74	(–) 12.2
			Σ	(+) 8.9	669	

$$\Delta_1' = (-)\, \frac{+8.9}{1.85 \times 669} \times 1000 \text{ l/s} = (-)\ \mathbf{7.2\ l/s}$$

*Discharge in pipe common to both loops, *i.e.* pipe *CD*

= assumed discharge + Δ_1 + Δ_1'

= (–) 20 + 4.9 + (+ 7.2)

[∵ Correction of – 7.2 *l/s* is in pipe *DC* ; and hence it will be equal to + 7.2 in pipe *CD*]

= – 20 + 4.9 + 7.2

= – 20 + 12.1 = **(–) 7.9 l/s.**

The corrected discharges after first corrections in various pipes are now given as :

Pipe	*Corrected discharges after first correction*
For Loop ABCD	
AB	49.9
BC	27.9
CD	(–) 7.9
DA	(–) 30.1
For Loop DCFE	
DC	7.9
CF	20.8
FE	(–) 15.2
ED	(–) 12.2

These discharges are now again used to re-analyse both the loops for the 2nd correction in Table 10.22, which is self explanatory.

Table 10.22. Hardy Cross Procedure for Second Correction

Pipe	*Assumed flows in*		*K*	$H_L = K.Q_a^{1.85}$	$\left\lvert \frac{H_L}{Q_a} \right\rvert$	*Corrected flows after 2st correction* Q_{a_1} = col. (2) + Δ_2 *l/s*
	l/s	*cumecs* $Q_a = Q_{a_1}$	*Value*		$= \left\lvert \frac{\text{col. (5)}}{\text{col. (3)}} \right\rvert$	
(1)	(2)	(3)	(4)	(5)	(6)	(7)
For Loop ABCD						
AB	49.9	(+) 0.0499	373	(+) 1.456	29	48.1
BC	27.9	(+) 0.0279	1615	(+) 2.150	77	26.1
CD (Common Pipe)	(–) 7.9	(–) 0.0079	2690	(–) 0.347	44	(–) 10.6*
DA	(–) 30.1	(–) 0.0301	1615	(–) 2.475	82	(–) 31.9
			Σ	(+) 0.784	232	

$$\Delta_2 = (-)\ \frac{0.784}{1.85 \times 232} \times 1000\ l/s = (-)\ 1.8\ l/s$$

Contd.

*Corrected flow in common pipe *CD*
= (–) 7.9 + (– 1.8) + (– 0.9)
= (–) 7.9 – 1.8 – 0.9 = (–) 10.6 l/s.

(1)	(2)	(3)	(4)	(5)	(6)	(7)
For Loop DCFE						
DC (Common Pipe)	7.9	(+) 0.0079	2690	(+) 0.347	44	10.6*
CF	20.8	(+) 0.0208	6580	(+) 5.089	245	21.7
FE	(–) 15.2	(–) 0.0152	10940	(–) 4.736	312	(–) 14.3
ED	(–) 12.2	(–) 0.0122	6580	(–) 1.897	155	(–) 11.3
			Σ	(–) 1.197	756	

$$\Delta_2' = (-)\ \frac{(-)\ 1.197 \times 1000}{1.85 \times 756}\ l/s = (+)\ 0.9\ l/s$$

The corrected flows after 2nd correction are finally worked out in col. (7) of Table 10.22, as below :

Corrected flows after 2nd correction

Pipe	*AB*	*AD*	*DC*	*DE*	*EF*	*BC*	*CF*
Corrected flow l/s	48.1	31.9	10.6	11.3	14.3	26.1	21.7

These flows are marked on the given pipe network in Fig. 10.30, it can be confirmed that all the points *A, B, C, D, E* and *F* are balanced.

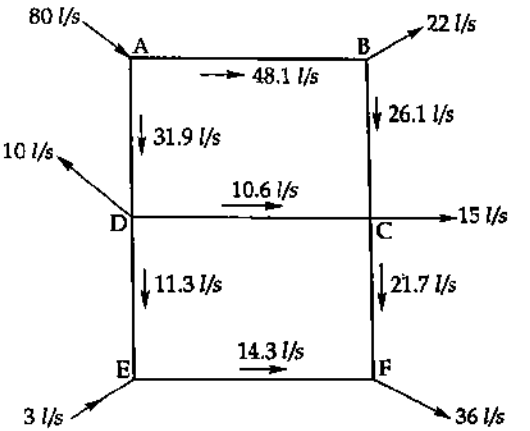


Fig. 10.30

Example 10.13. *Fig. 10.31 shows one of the circuits of a distribution system, along with the populations (P) to be served at different points. The pressure at the start point A is 35 m head of water and the min. pressure desired at the point D is 20 m head of water. Design the various pipes of this circuit by assuming the per capita demand to be 190 litres/person/day, and the peak rate of demand for the design of distribution system as 2.7 times the average.*

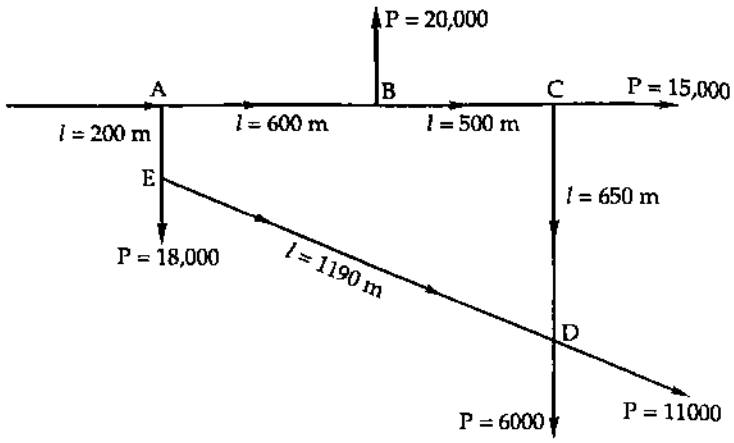


Fig. 10.31

Solution.

Average demand = 190 litres/person/day

Max. demand for which distribution pipes are to be designed

$= 2.7 \times 190 = 513$ litres/person/day.

The off-take discharges required at various points to serve the given populations are :

At point B $= \dfrac{513 \times 20,000}{24 \times 60 \times 60} = 119$ l/s

At point C $= \dfrac{513 \times 15,000}{24 \times 60 \times 60} = 89$ l/s

At point D along CD $= \dfrac{513 \times 6,000}{24 \times 60 \times 60} = 36$ l/s

At point D along ED $= \dfrac{513 \times 11,000}{24 \times 60 \times 60} = 65$ l/s

At point E $= \dfrac{513 \times 18,000}{24 \times 60 \times 60} = 107$ l/s

$\Sigma = 416$ l/s

$\therefore$ Total discharge input (inflow) required at $A = 416$ l/s.

The flow discharges as computed above are now written for all the pipes, as shown in Fig. 10.32. The flows have expressed in Fig. 10.32 on the assumption that the balancing point lies near the point D in the system (*i.e.* all the demand along the route $ABCD$ will be satisfied by the water flowing via AB, and all the demand along the route AED will be satisfied by the water flowing along AE.

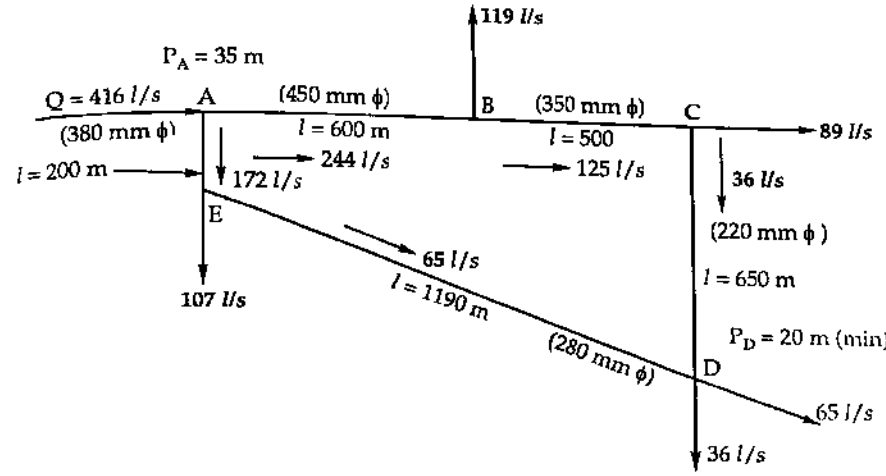


Fig. 10.32

The sizes of the pipes should now be such as the velocities produced in them are somewhere between optimum values (*i.e.* 0.9 to 1.5 m/sec) and the total head loss from A to D, via either of the two routes (*i.e.* $ABCD$ or AED) remains approximately equal, and less than the available head of $35 - 20 = 15$ m. The sizes of the pipes are assumed by common sense, as shown in Fig. 10.32 and in col. (4) of Table 10.23. The William-Hazen's nomogram is used to determine the head losses instead of using the formula, so as to facilitate calculations.

Table 10.23

Route	*Pipe*	*Discharge flowing in l/s* Q_a	*Proposed dia of pipe in mm*	*Length in metres L*	*Loss of head per 1000 m in metres (read from Fig. 10.22, based on Hazen William formula)*	H_L = *Loss of head in pipe of length L =* $\frac{\text{col. (5)} \times \text{col. (6)}}{1000}$	$\left\lvert \frac{H_L}{Q_a} \right\rvert$ *in metres/cumec* $\frac{\text{col. (7)}}{\text{col. (3)}/1000}$
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
ABCD	*AB*	244	450	600	8	4.8	$\frac{4.80}{0.244} = 19.7$
	BC	125	350	500	8	4.0	$\frac{4.0}{0.125} = 32.0$
	CD	36	220	650	8	5.2	$\frac{5.2}{0.036} = 144.2$
Σ						(+) 14.0	196.2

Contd...

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
AED	AE	172	380	200	10	2.0	$\frac{2.0}{0.172} = 11.6$
	ED	65	280	1190	10	11.9	$\frac{11.9}{0.065} = 183$
Σ						(–) 13.9*	194.6
Grand Total							390.8

Using Hardy-Cross equation, and Table 10.24, we have

$$\Delta_1 = \text{Correction to discharges} = \frac{\Sigma H_L}{x \cdot \Sigma \left| \frac{H_L}{Q_a} \right|} = \frac{14.0 - 13.9}{1.85 \times (390.8)} \text{ cumecs}$$

$$= \frac{0.1 \times 1000}{1.85 \times 390.8} \ l/s$$

$$= \frac{100}{724} = 0.14 \ l/s.$$

The correction to be applied is too small and hence the adopted diameters are O.K. **Ans.**

Note. The assumed pipe diameters and the discharges will be correct when the head loss between any two points worked out via different routes is found to be the same. This head loss should also be equal or less than the available head loss. If the head loss in one or both the routes is greater than the available head or there is a greater difference between the two ; it is necessary to change the values of the assumed diameters and adopt some other suitable values. Suitable adjustments have to be made by the intelligence of the designer till the conditions of nearly equal head loss in both the routes, and each less than the available head loss are obtained. This step is known as *adjustment of diameters*. Finally, the discharges should be adjusted by determining the Hardy-Cross correction (Δ). When the correction is very small, it may be ignored, otherwise new discharges should be tabulated and head losses along both the routes are worked out again. The process has to be repeated till the values of head losses along two different routes are nearly equal, and the Hardy-Cross correction value for flow (Δ) is quite less. When this is achieved, the balanced distribution conditions will be obtained.

10.15.2. Equivalent Pipe Method. This method is sometimes used as an aid in solving large networks of pipes, in which it becomes convenient to, first of all, replace the different small loops by single ***equivalent pipes*** having the same discharging capacities and causing the same head loss. This method is, therefore, also called the **head balanced method** of analysing water distribution pipe networks.

*The head loss *via* two routes (*i.e.* *ABCD* and *AED*), being 14.0 m and 13.19 m, are almost equal and less than the max. 15 m, and hence O.K.

In this method, pipe circuit can be reduced into a single equivalent pipe by using the following two principles of hydraulics :

(i) The loss of head caused by a given flow of water through the pipes connected in series is additive ;

(ii) The quantity of discharge flowing through the different pipes connected in parallel will be such as to cause equal head loss through each pipe.

The use of this method has been illustrated below :

Let *ABCD* represents a pipe circuit with the pipe sizes and lengths, as indicated in Fig. 10.33.

Let a flow of 60 *l/s* be assumed to be flowing through the routes *ABC* and *ADC*. Using William-Hazen's nomogram (Fig. 10.22), the head losses into the two circuits will be :

For circuit *ABC* $= \frac{4.1}{1000} \times 700 + \frac{9}{1000} \times 400$

$= 2.87 + 3.6 = \mathbf{6.47\ m}$

For circuit *ADC* $= \frac{9}{1000} \times 475 + \frac{5.2}{1000} \times 450$

$= 4.3 + 2.3 = \mathbf{6.6\ m}$

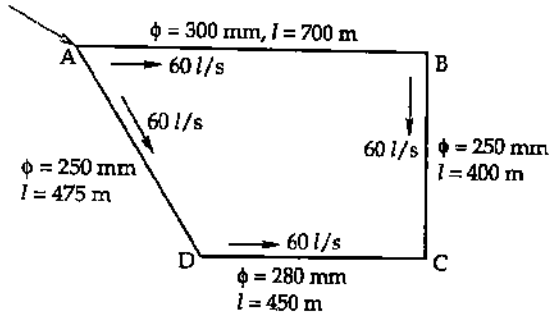


Fig. 10.33

Now, if the routes *ABC* and *ADC* are replaced by 300 mm dia pipes each, then the lengths of each pipe required can be obtained from the fact that a 300 mm dia pipe for a discharge of 60 *l/s* offers a head loss of 4.1 m per 1000 m length.

∴ Length of 300 mm ϕ pipe required for replacing route *ABC*

$$= \frac{6.47}{4.1} \times 1000 \text{ m} = 1578 \text{ m}.$$

∴ Length of 300 mm ϕ pipe required for replacing route *ADC*

$$= \frac{6.6}{4.1} \times 1000 \text{ m} = 1610 \text{ m}$$

Moreover, the loss of head via both the routes should be the same, provided the flows are properly divided. Hence, if a loss of head of 6.47 m is to occur via route *ADC* also (*i.e.* 300 mm ϕ pipe of 1610 m length), the rate of loss of head

per 1000 m will be

$$= \frac{6.47}{1610} \times 1000 = 4.02 \text{ m.}$$

In a 300 mm ϕ pipe, such a loss (*i.e.* 4.02 m per 1000 m) will occur only if the discharge is 61 *l*/s (from William-Hazen's monogram). Therefore, the circuits *ABC* and *ADC* can be replaced by an equivalent pipe carrying a total discharge of 60 + 61 = 121 *l*/s.

Now, for this equivalent pipe having a dia of 300 mm and a discharge of 121 *l*/s, the rate of loss of head per 1000 m is found from nomogram as 15 m. In order to ensure a head loss of 6.47 m in this equivalent pipe also, the length of this pipe should be

$$= \frac{1000}{15} \times 6.47 \text{ m} = \mathbf{431 \text{ m.}}$$

Hence the total loop *ABCD* can thus be replaced by a single equivalent pipe of 300 mm ϕ and 431 m long, irrespective of the flow discharges.

(3) **Other Methods.** Various others methods, such as the *electrical analyser method,* the *method of sections,* the *circle method,* the *pitometer distribution studies method,* etc. are also sometimes used for solving distribution pipe networks. However, these methods are not of much utility and are rarely used in our country, and hence left out for discussion in this volume.

APPURTENANCES IN THE DISTRIBUTION SYSTEM

Various appurtenances are required to be fitted in the pipe network or the distribution system for its efficient and controlled functioning. Besides the various valves, such as, sluice valves, check valves, air valves, drain or scour valves, etc. (already described in chapter 6); certain other appurtenances such as *fire hydrants, metres, water taps, stop cocks, pipe bends,* etc. are also required to be provided. Fire hydrants and water meters are described here, while the taps, pipe fittings, etc. shall be dealt in the next chapter.

10.16. Fire Hydrants

A hydrant is an outlet provided in a water distribution main or a sub-main (*i.e.* pipe of at least 15 cm dia) for tapping water, mainly during fires. They may sometimes, however, be used for withdrawing water for filling the municipal water tankers.

During a fire breakout, a nearby hydrant is connected to the fire hose, and the water obtained from the hydrant is used for extinguishing the fire. For fire fighting, as was mentioned earlier, the water is generally required at much higher pressure than that required for ordinary domestic uses, so as to obtain the water at large rates and also to make it reach several storeyed high buildings. Such high pressures are generally developed by attaching the fire

*The standard fire hose of 63 mm dia, 33 m long and 28 mm dia nozzle will deliver about 1100 litres per minute with a nozzle pressure of about 42 m head of water and a hydrant pressure of about 56 m head of water. More hydrant pressure is required in the case of longer hoses for developing the same nozzle pressure. Thus, for delivering the requisite standard discharge of 1100 litres/minute, a hydrant pressure of about 72 m head of water is required in a hose pipe of about 67 m length; 85 m head is required for 100 m length, and 100 m head is required for 133 m long hose pipes.

hydrant outlet to the fire engine. The fire engine will draw water from the hydrant, boost its pressure within the engine, and the high pressure water will come out from the outlet of the engine, to which the hose pipe will be connected. The other end of the hose pipe will finally carry the water to the building at a pressure of at least 32 m of water head, through the smooth nozzles (generally 28 mm in diameter). Sometimes, higher pressures upto about 100 m head of water* may have to be developed, and, therefore, they must be designed for this much of pressure.

The available water pressures at the fire hydrants should be as follows :

(*i*) 7 to 14 m head of water, when water has to be pumped with motor pumps.

(*ii*) 35 to 50 m head of water, when the direct flow from the hydrant is to be used.

In case of serious fires, water may be pumped from several fire hydrants, by means of pumps mounted on trucks.

The fire hydrants are generally provided at all the street crossings and turnings, and at 90 to 120 m intervals on straight runs in such a way, that each section of the community can be served simultaneously from two or more hydrants, each not away by more than 75 m or so.

The **requirements of a good hydrant** are :

(*a*) It should be such as to connect the hose or the motor pump easily to it.

(*b*) It should be cheap.

(*c*) It should be easily detectable during the panicky atmosphere of fire.

(*d*) If should not get out of order during operation.

(*e*) On being fully opened, it should allow undisturbed water flow.

The following two **types of fire hydrants** are commonly used :

(*i*) *Post fire hydrant ;* and

(*ii*) *Flush fire hydrant*

The essential difference between these two types of hydrants is that whereas the *post fire hydrant* remains standing above the ground like a post by about 0.9 to 1.2 m ; the *flush hydrant* is installed underground in a brick or a cast iron chamber with its top cover slightly above the street level. Obviously, the former (*i.e.* post hydrants) can be detected very easily as they project above the ground, but are liable to be damaged by children and mischief mongers. On the other hand, the latter types, *i.e.* flush hydrants, being underground, are a little difficult to be searched in the panicky circumstances during fire, but are less prone to damage by the mischievous people. However, the flush hydrants are made more easily recognisable by attaching a plate, bearing the letters **F.H.** (*i.e.* fire hydrant) on some nearby building or some other object. The same letters **F.H.** are also inscribed on the top cover of the hydrant chamber. Because of lesser civic sense prevailing in India, post hydrants are likely to be damaged here, and hence the flush hydrants are widely used here. Whereas, post hydrants are widely used in countries like America, where the chances of their being unnecessarily tempered or damaged by the general public are very less, because of the greater civic sense prevailing there.

The typical sections of these two types of hydrants are shown in Figs. 10.34 and 10.35. The method of functioning is the same for both the types.

The **post fire hydrant** (Fig. 10.34) consists of a barrel of cast iron, which is connected to the water main passing the street. There is a valve stem having a good leather valve at its lower end. The valve stem at the top is connected to a nut (or the handle) to regulate the flow. In a closed position, the leather valve rests against the valve seat. For opening the hydrant, the nut is operated (or the handle is rotated) so as to raise the leather valve up, and thereby admitting water into the hydrant barrel. The post fire hydrants may be provided with *one, two, three* or *four* outlet openings, depending upon which they may be classified as *one way, two way, three way,* or *four way* hydrants. These outlets are spaced around the periphery of the hydrant barrel. The advantage of having more number of outlets is that more number of fire tenders can be connected to the same hydrant. For usual cases, we generally have two outlet openings ; viz. (*a*) one 63 mm dia opening which is used and connected directly to the hose pipe when boosting of pressure is not required ; (*b*) Another 100 mm dia opening which is used when the boosting of pressure is required for connecting it to the fire engine or the pumps. When such a fire engine is connected to the fire hydrant, the hose pipe is taken out from the outlet of the fire engine for throwing water on the fire.

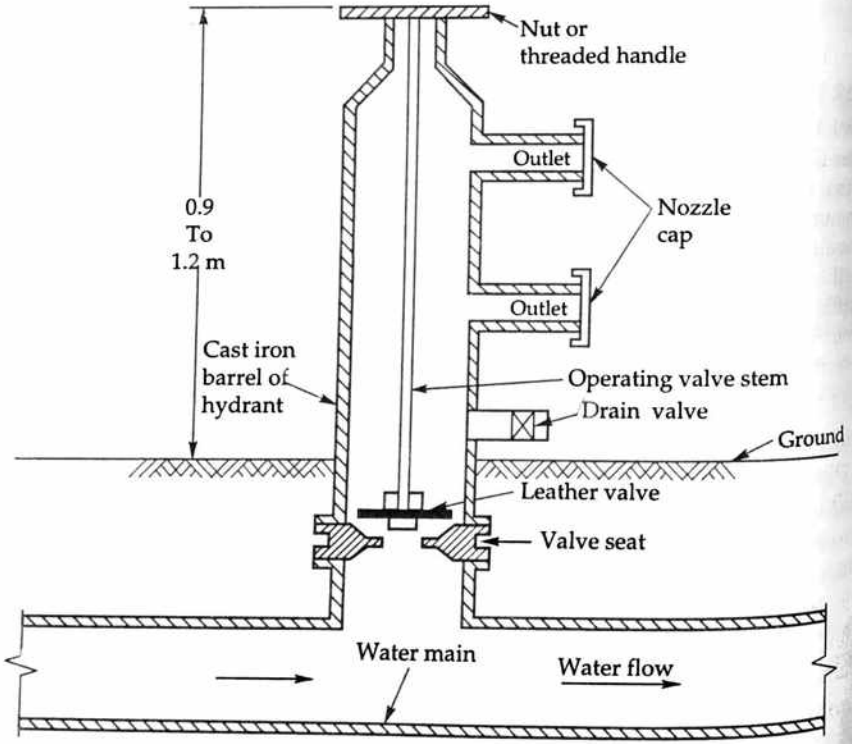


Fig. 10.34. Typical section of the post fire hydrant.

In case of **flush hydrants** (Fig. 10.35), there is generally only one outlet opening on the side of the moving stem. The same opening may be directly connected to the hose pipe (when no lift is required) or may be connected to the fire engine.

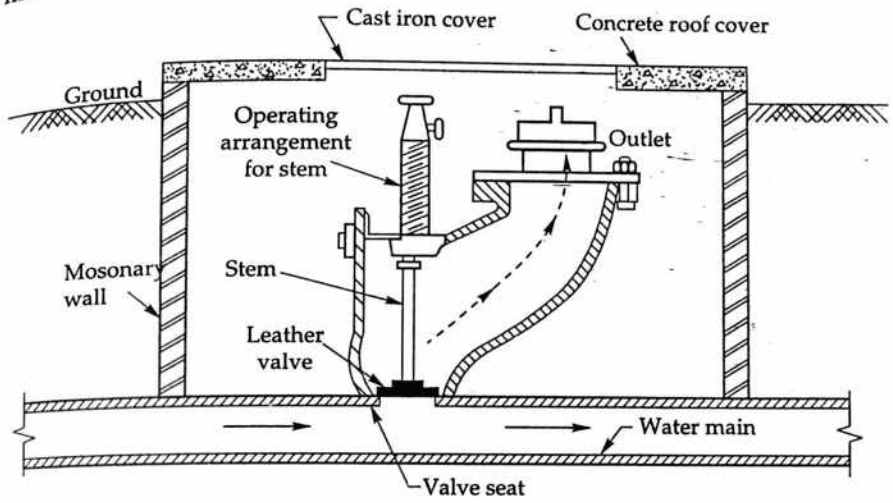


Fig. 10.35. Typical section of a flush fire hydrant.

10.17. Water Meters

Water meters are the devices which are used for measuring the quantity of water flowing under pressure through a pressure conduit. This measurement of the quantity of water supplied to the general public (for industrial, commercial or domestic use) is necessary, in order to charge the consumers according to the quantity of water supplied to them. The question as to whether the consumers should be charged as per the quantity of water supplied to them or at a flat rate, is highly debatable. There are points in favour of both, as discussed in article 2.4(10) under the heading "Policy of Metering and Methods of Charging".

10.17.1. Requirements of a Good Water Meter. These are given below :

(*i*) It must record the entire water passing through it, and should therefore, be capable of recording even slight discharges.

(*ii*) Its maintenance and repair should be easy.

(*iii*) It should measure the discharges within the maximum limit of 20% error.

(*iv*) It should be able to work efficiently at all the pressures in the mains.

(*v*) It should cause minimum hinderance to the flow and, therefore, cause minimum head loss in its working.

(*vi*) Its parts should not be affected easily by the chemicals present in the water passing through it.

(*vii*) It should prevent the back flow passing through it and should not be liable to clogging.

10.17.2. Types of Water Meters. Mainly, there are two types of meters, viz.

(*A*) The velocity meters or the *Inferential meters* ; and

(*B*) The positive meters or the *Displacement-meters.*

These are described below :

10.17.2.1 Velocity meters or Inferential meters. Such meters basically measure the horizontal velocity of water flowing through them. The measured velocity, when multiplied by the area of flow cross-section, will give the discharge through the meter. Automatic arrangements are then made to record the integrated discharge (or the volume of flow) over a period of time. Velocity meters can be successfully used for measuring high flows. They are therefore, widely used for measuring the supplies of industries and trades. Their additional advantage is the fact that they can be used for measuring raw and sedimented waters, and hence useful for measuring flows at water-works. *These meters are generally not used for measuring small domestic supplies although they are available in smaller sizes of upto 20 mm, and hence can be used for measuring such supplies also.* However at low flows, their accuracy is less, as compared to that of the "displacement type of meters".

The various examples of the inferential type (or velocity type) of meters are *rotary meters*, *turbine meters*, and *venturi-meters*.

A **rotary meter** consists of radial vanes attached to a shaft, and enclosed in a casing, as shown in Fig. 10.36. When the water enters and passes through the meter with a certain velocity, the radial vanes are rotated in the clockwise direction, which in turn, will revolve the shaft. The number of revolutions per unit time, made by the shaft, will depend upon the velocity of the flow. The greater is the velocity, the higher will be the speed of rotation, and *vice-versa*. The velocity of flow and, therefore, the discharge is thus, proportional to the speed of the shaft. The meter can hence be calibrated, as to directly read the discharge. This discharge can be further integrated by an integrating device of a system of gears, so as to read the total discharge volume, over a certain period of time.

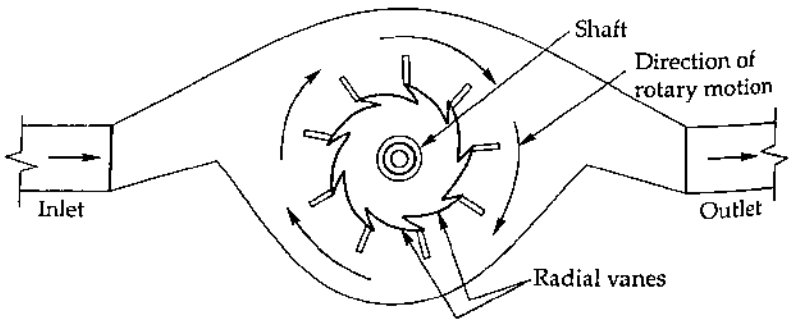


Fig. 10.36. Rotary meter (a type of velocity meter).

A **turbine meter** is similar to a rotary meter, and consists of a turbine wheel which is rotated by the moving water. The number of revolutions made by the turbine wheel will give the discharge as in rotary meters.

The well-known **venturi-meter** is again a velocity type of meter, and is based upon the principle of Bernoulli's equation. It can preferably be used for raw waters, and for measuring the high flows in large pipes with a nominal head loss. It is not suitable for measuring small flows. The well known venturi-meter consists of gradually contracting the normal pipe to the throat section, and then more gradually expanding to the normal size, as shown in Fig. 10.37.

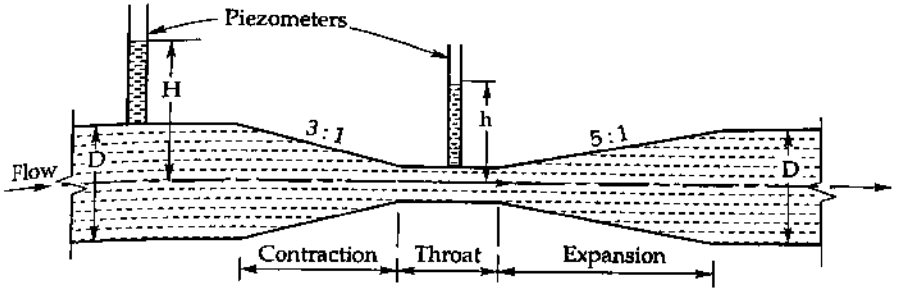


Fig. 10.37. Venturi-meter (a type of velocity meter).

The piezometers, one at the normal end, and another at the throat are inserted in the pipe. When the water passes through such a portion of the pipe, it will rise to the different levels in the two piezometer tubes on account of different velocities and pressures at the normal and contracted sections of the pipe. The discharge through the pipe is proportional to the difference of heads between the two piezometers.

The venturi-meter can be made automatic by first connecting the venturi-tubes to an apparatus which measures their head difference ($H - h$), and then integrates the discharge over any period of time.

Venturi-meters can be provided at the heads of the main water pipe lines, and also at the head of each district in the water distribution system. They are generally available to be fitted in the mains of sizes, 7.5 cm and above.

The comparative *advantages* and *disadvantages* of *inferential meters* over those of positive or displacement meters are given below :

(*i*) They are cheaper.

(*ii*) They are light and require lesser head.

(*iii*) They are less accurate.

(*iv*) If anything stops the rotation of rotary or turbine meters, the water will continue to flow without being recorded.

(*v*) They can be installed only on horizontal flows ; and hence on vertical pipes, the positive or strictly speaking semi-positive type of metres will have to be installed.

10.17.2.2. The Positive or Displacement Meters. These meters are more accurate because they measure the quantity of passing water by counting the number of times the meter chamber is filled and emptied. The capacity of the meter chamber when multiplied by the number of times it is filled and emptied, will directly give us the quantity of flow over the given period of time.

Various types of displacement metres, such as *reciprocating type, oscillating type, disc type,* etc. are available, depending upon the type of the motion of the moving part in the meter. The most commonly used type of such a meter is *a disc meter*, which is extensively used for measuring small flows in residential houses. A **disc meter**, such a shown in Fig. 10.38 (*a*) and (*b*),

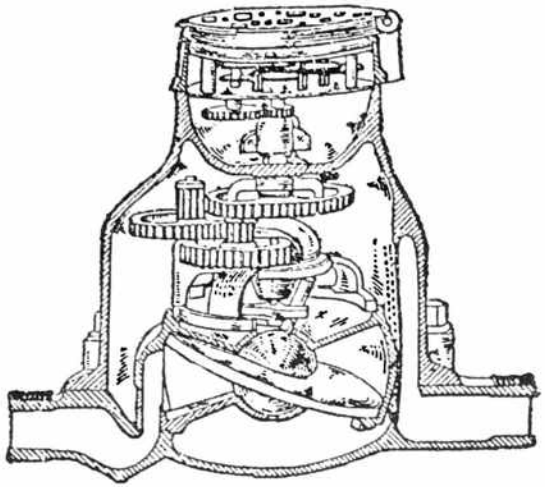


Fig. 10.38 (*a*) Perspective view of a Disc meter.

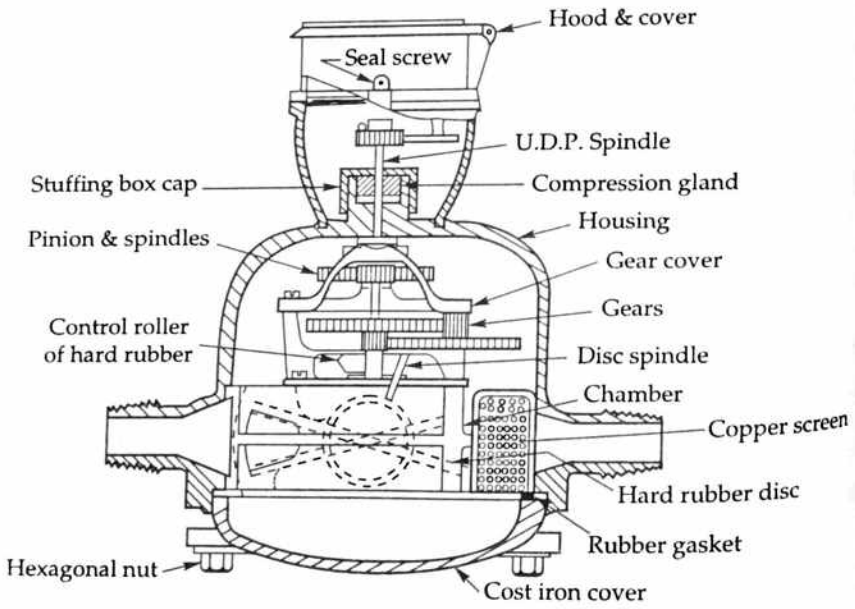


Fig. 10.38 (*b*) Section of a Disc meter.

consists of a disc placed inside a chamber, provided with an inlet and an outlet. The water when enters the chamber, oscillates the disc about its centre with a spiral motion.

Arrangements are made in such a way that one complete filling and emptying of the chamber gives one revolution to the train of gears, and the meter thus goes on recording the volume of the water passing through it. Such meters are available in different sizes, say from 16 to 150 mm in diameter with safe operating capacities of 90 to 4500 litres per minute, respectively.

In India, the manufacture of domestic water metres is guided by the provisions of I.S. 779—1994, and that of *bulk meters* by I.S. 2373–1981. Both the types of meters are now-a-days available in good quality. The installation and maintenance of *domestic water meters* is governed by I.S. 2401—1973, and their testing by I.S. 6784—1996. The biggest point which should be kept in mind while installing such meters, is that, they should not be located where a severe shock of water hammer may occur.

PROBLEMS

1. (*a*) Explain briefly the general methods of distribution of water employed in municipal water supply schemes.
   (*b*) Illustrate with sketches the different types of layouts of pipe systems in distributing water, and compare their comparative merits and demerits.
2. (*a*) Compare the merits and demerits of the 'continuous' and 'intermittent' systems of water supply. Under what conditions would you recommend the use of the latter ?
   (*b*) Write a detailed note on the detection and prevention of wastage of water in the distribution system.
3. Public water supply is conveyed from the source to the centre of supply by gravitational system or pumping system or combined system. Draw three separate longitudinal sections, indicating these systems and show the positions of various details in each.
4. Write short notes on the wastage of water in public water supplies.
   State various methods of detection and prevention of wastage. Enumerate the causes of such wastage.
5. State the factors you would take into consideration and the procedure you would follow in designing a distribution system for the water supply of a city.
6. State the functions of a Service Reservoir, and sketch the sectional elevation of the same, showing the various appurtenances.
7. Write short notes on the following :
   (*a*) Distribution of water to consumers.
   (*b*) Causes of water wastes, its detection and prevention.
8. The observed cumulative drafts by a town are stated below ; the drafts are at four hour interval :

Time	*4 a.m.*	*8 a.m.*	*Noon*	*4 p.m.*	*8 p.m.*	*Midnight*
Cumulative draft million litres	0.5	1.40	2.60	3.70	4.50	5.00

Determine the equalizing storage for the above drafts ; when the pumping is

(*i*) 24 hours, and

(*ii*) between 6 a.m. and 6 p.m. (*A.M.I.E. May 197*

9. (*a*) Explain the Hardy Cross method used for pipe net work analysis in wate distribution system.

(*b*) A pipe network consists of the following pipes :

Pipe	*Length (metres)*	*Diameter (cm)*	*Friction factor*
AB	400	30	0.014
BC	600	30	0.010
AD	500	40	0.012
DC	500	25	0.011

Inflow at *A* is 1.0 m^3/sec, while outflows at *B*, *C* and *D* are 0.3, 0.5 and 0.2 m^3 sec, respectively. Find the flow in each pipe taking only one trial. The pressure at *A* is 100 m of water.

10. (*a*) What factors do you consider in selecting a site for the location of a distribu tion reservoir, and how do you decide the full supply level in it ?

(*b*) Draw a neat sketch of an over-head distribution reservoir showing the various fittings and connections, and explain the function it can serve in public water supply scheme

11. (*a*) How will you estimate the quantity of water to be stored in the distribution reservoir ?

(*b*) Find out graphically or otherwise the balancing storage required in a distribution reservoir with the following data :

Population to be served = 50,000

Average per capita demand = 200 litres per day.

Pumping into reservoir is done for 15 hours a day (*i.e.* from 5 A.M. to 12.30 P.M. ; and from 1.30 P.M. to 9.00 P.M.).

Hourly variation factors between different hours are as under :

0–1	1–2	2–3	3–4	4–5	5–6	6–7	7–8	8–9	9–10	10–11	11–12
0.25	0.25	0.25	0.25	0.35	0.45	1.0	1.5	1.9	2.5	2.1	1.5
12–13	13–14	14–15	15–16	16–17	17–18	18–19	19–20	20–21	21–22	22–23	23–24
1.2	0.9	1.0	1.2	1.5	1.6	1.3	1.0	0.8	0.5	0.4	0.3

12. Write short notes on any four of the following :

(*i*) Hardy Cross method ;

(*ii*) Fire hydrants ;

(*iii*) Different types of distribution networks ;

(*iv*) Metering in distribution systems ;

(*v*) Stand pipes ;

(*vi*) Waste detection and prevention ; and

(*vii*) Distribution reservoirs.

13. (*a*) Discuss briefly the procedure commonly adopted for planning and designing of a distribution system in a water supply scheme. What precautions you would take to make the design economical ?

(*b*) Two 150 mm dia pipes of equal lengths form a loop in a distribution network. Calculate the diameter of an equivalent pipe of the same length to represent the loop (there are no intermediate draw offs).

—11

Water Supply Plumbing Systems in Buildings and Houses

11.1. Plumbing System in Water Supplies and Associated Terms

Plumbing is a general term which indicates the practice, materials and fixtures used in the installation or maintenance of all pipings, fixtures, appliances, and other appurtenances used in connection with both public or private water supply systems as well as sanitary or drainage systems, within or adjacent to any building or its connection with any point of public disposal.

The entire system of pipings, fixtures, appliances, etc. used in providing water supply and drainage facilities in a building is, therefore, called a plumbing system. Hence, a ***'plumbing water supply system'*** in a building consists of supply and distribution pipes (called *service pipes*), *connections*, *domestic storage tanks, flushing storage tanks*, etc., while a ***'plumbing sanitary or drainage system'*** includes water closets, urinals, and other such appliances like traps, soil waste and vent pipes and stacks, anti-siphonage pipes, house drains and house sewers, flushing cisterns, etc. Here in this book, we are concerned only with the water supply plumbing, as the 'drainage plumbing system' has been dealt in volume II.

11.2. The House Water Connection

In installing a 'water supply plumbing system' in a building, the first and main step, obviously is, to obtain a 'water connection' from the municipal water main, because the water supply to a house or a building can start only from this point. A typical water connection, connecting the service pipe with the municipal water main, is shown in Fig. 11.1. As is evident, the water connection consists of : (*i*) a ferrule ; (*ii*) a goose neck ; (*iii*) a service pipe ; (*iv*) a stop cock ; and (*v*) a water meter, as described below :

(*i*) **Ferrule.** A ferrule is a right angled sleeve made of brass or gun metal, and is joined to a hole drilled in the water main, to which it is screwed down with a plug. Its size usually varies between 10 to 50 mm dia. For all other connections of more than 50 mm dia, a tee branch connection, off the water main, is used.

(*ii*) **Goose neck.** Goose neck is a small sized curved pipe made of a flexible material (usually lead) and is about 75 cm in length forming a flexible connection between the water main and the service pipe.

(*iii*) **Service pipe.** Service pipe is a galvanised iron pipe of size less than 50 mm dia. It should be laid underground in a trench in which no sewer or drainage pipe is laid. The service pipe which supplies water to the building through the municipal main is thus connected to the main through the goose neck and ferrule.

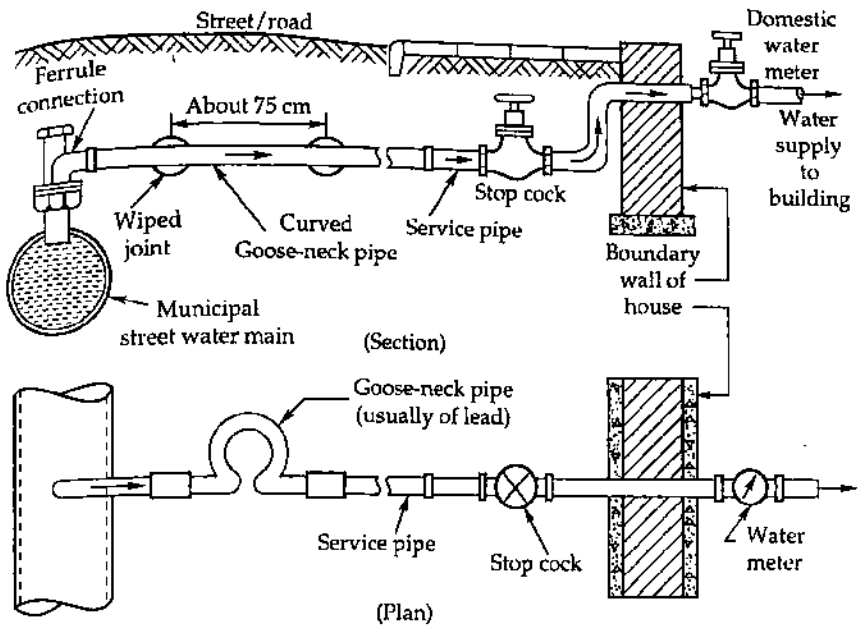


Fig. 11.1. The Water Connection.

(*iv*) **Stop cock.** The stop cock is provided before the water enters the water meter in the house. It is housed in a suitable masonry chamber with a removable cover, and is fixed in the street close to the boundary wall in an accessible position. Sometimes, it is provided just before the water meter inside the house, keeping both of them in one chamber. The details of stop cocks are given in the next article.

(*v*) **Water meter.** Water meter measures and records the quantity of water consumed in the house. The domestic type water meter generally employed for houses is fitted into the service pipe with *unions*, which enables the meter to be changed where necessary. The water meter is generally fixed in an iron box fitted in an opening or cavity made in the boundary wall of the house, and is covered with a movable iron cover. The details of the domestic water meters are given in the previous chapter in article 10.17.

11.3. Stop Cocks

A stop cock is a screw down type of sluice valve which is used in smaller sized pipes in service connections for stopping or opening the supply. They are generally provided at the water entrance of each building and also within the building. When provided just prior to the water meter in each house connection, they should be enclosed in a proper cast iron box having a hinged cover.

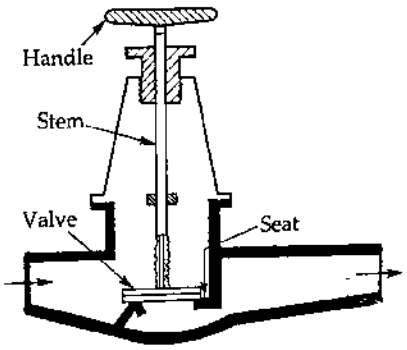


Fig. 11.2. A typical stop cock.

A typical stop cock is shown in Fig. 11.2. The body of the valve is so cast that the water passes through an orifice when the valve stem is raised. When the valve is closed, it rests against the seat, and thereby closing the orifice. They are extensively used in pipes upto 50 mm sizes.

11.4. Water Taps and Bib Cocks

Water taps are the types of valves provided at the end of house service pipes for withdrawing water at the consumer's house. Several varieties of water taps are available, and the most common of them is a *bib cock*. A typical bib cock is shown in Fig. 11.3. By rotating the handle of the bib cock, the orifice opening, through which the water passes, can be increased or decreased, thereby controlling the outflow through the spout. They are available to be fitted in different pipe sizes from 10 to 50 mm dia.

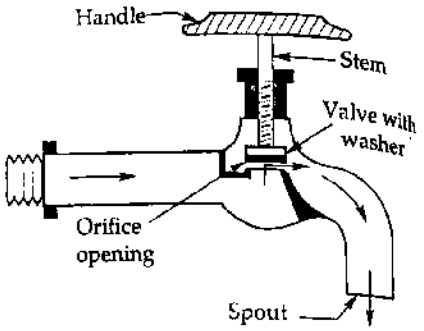


Fig. 11.3. A typical bib cock.

The bib cocks may also be of the *push type* which opens from a slight push given vertically upward, and closes down automatically due to self-weight when the push is removed. Such valves are used in public taps in order to avoid wastage of water from handle type bib cocks, which may be left open by the irresponsible people.

The bib cocks should be water-tight and should not leak, as it may lead to considerable wastage of water. The rubber or leather washers used in them may get damaged and lead to leakage, and hence should be replaced as and when needed.

11.5. Pipe Fittings

The various pipe fittings, such as bends, crosses, tees, elbows, wye unions, caps, plugs, flanges, nipples, etc. are frequently used in making service connection and also sometimes in bigger sized mains or sub-mains. Various types of bends and other important pipe fittings are shown in Fig. 11.4.

11.6. Storage of Water in Buildings

Storage of water within the premises of a building is usually necessary, because the municipal supplies are usually not available round the clock. Such storage may also help in meeting static fire demand ; and to provide uninterrupted water supply in the building in case the main municipal or ground water supply is shut off for repairs, or if there is a power failure. Storage may also help to supplement the direct municipal supply in case of excess or peak demand.

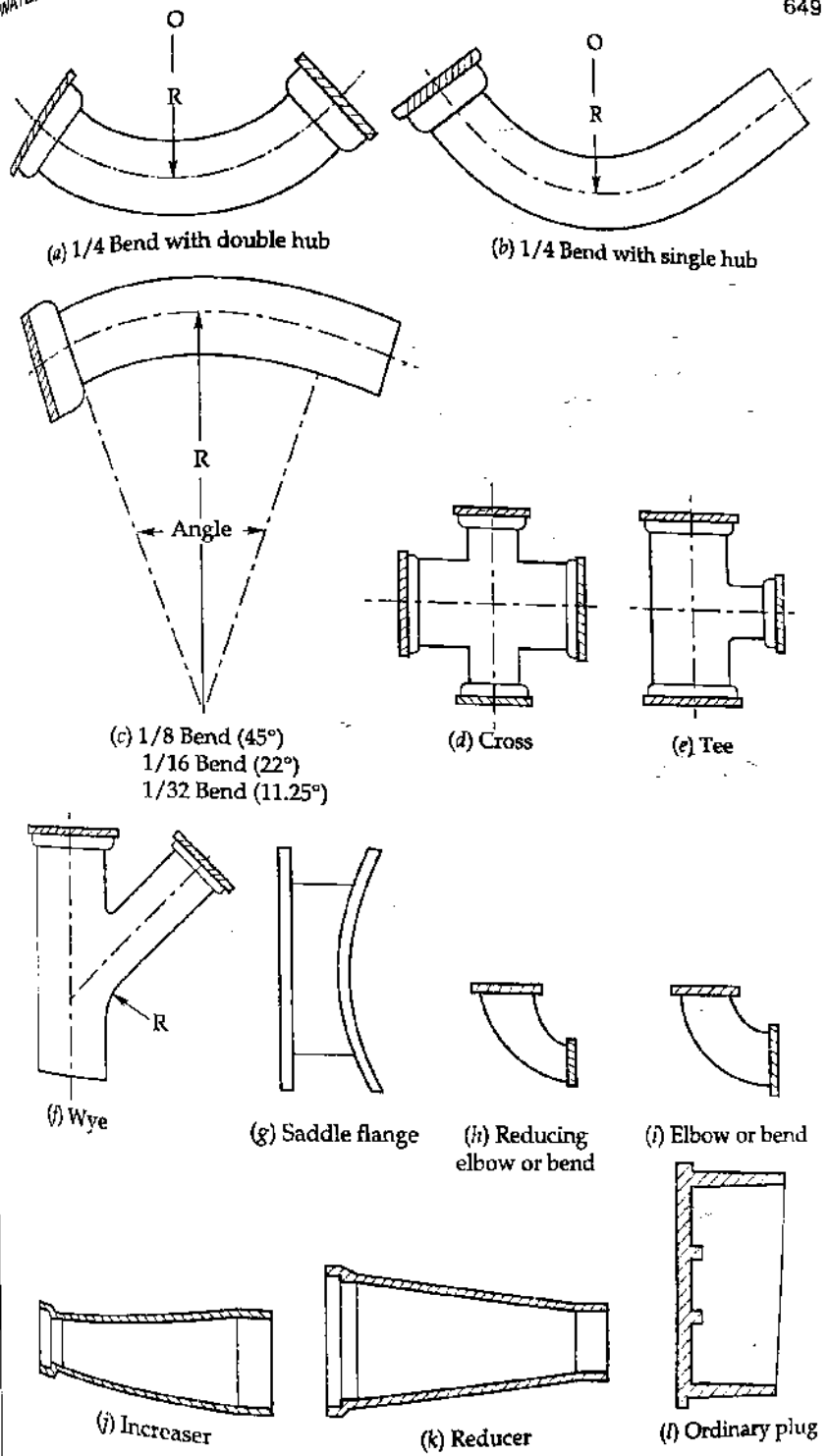


Fig. 11.4. Various pipe fittings.

11.6.1. Estimating Storage Capacity. The water storage capacity for a building will depend on the following factors :

(*i*) hours of supply ;

(*ii*) rate of supply, governed by the pressure in the mains ;

(*iii*) demand pattern in the given building ; and

(*iv*) fire storage required.

The following two types of water storages are reqd in the buildings :

(*i*) *Overhead storage* ; and

(*ii*) *Underground storage*

Both these types of storages are discussed below:

11.6.1.1. Overhead Storage. When the city water supply is available at sufficient pressure round the clock, as to rise to the highest floor, then no overhead storage is required, and all the water supply fixtures can be connected with the direct supply line. However, in India, most of the city water supplies are confined only to a few hours in the morning and evening. Due to this reason, it becomes imperative to store water in overhead tanks, particularly for its use in toilets and urinals. Most of the Indian municipalities require mandatory overhead water storage exclusively for flushing of toilets and urinals to ensure constant supply for flushing toilets. This requirement depends upon the number of sanitary fixtures in the building and may vary from city to city, depending upon the living standards of the occupants of the building. The required overhead storage capacity for flushing purposes per fixture for different types of buildings, stipulated by the Bureau of Indian Standards, vide its code: IS 1172, is given in table 11.1.

Table 11.1. Water Storage required for Flushing purposes in Buildings

S. No.	*Type of Building*	*Storage Capacity required*
1.	For tenants having common conveniences	900 litres per W.C. (Water Closet)
2.	For residential premises other than (1) above	270 litres for one W.C. seat, and 180 litres for each additional seat in the same flat
3.	For factories and workshops	900 litres per W.C. and 180 litres per urinal seat
4.	For cinema halls, public assembly halls, etc.	900 litres per W.C. seat and 350 litres per urinal seat.

In areas of chronic water shortage or intermittent supply, additional overhead storage is required for other domestic uses like washing and bathing, as indicated in table 11.2.

Table 11.2. Storage required for Domestic Uses

S. No.	*Type of Building*	*Unit (per)*	*Unit storage in litres per unit*
1.	Dwelling units	Resident	70
2.	Hostels	Resident	90
3.	Hotels	Resident	135
4.	Commercial buildings without canteens	Head	35
5.	Commercial buildings with canteens	Head	45
6.	Restaurants	Meal	7
7.	Day schools	Head	25
8.	Boarding schools	Resident	90
9.	Nursing homes and medical quarters	Resident	135

11.6.1.2. Underground Storage. Underground storage tanks (or tanks at ground level) are necessarily required to collect water from municipal supply lines, if the water pressure in the main is insufficient to reach the overhead tank. The water collected in the underground tank is then pumped up to the overhead tank. *Direct use of pumps on the municipal mains is always prohibited by the municipal authorities, as it reduces the water pressure in the adjoining houses or buildings.* Inspite of prohibition, many residential units do install their individual tullu pumps on the municipal mains, to avoid the construction of underground tanks.

The capacity of an underground storage tank should be the net difference between the daily peak demand and the flow during hours of supply. Provision should be added for non-supply due to power failures or other causes. *For normal buildings with a dependable water supply, underground storage capacity is taken at 12—24 hours of average daily demand.*

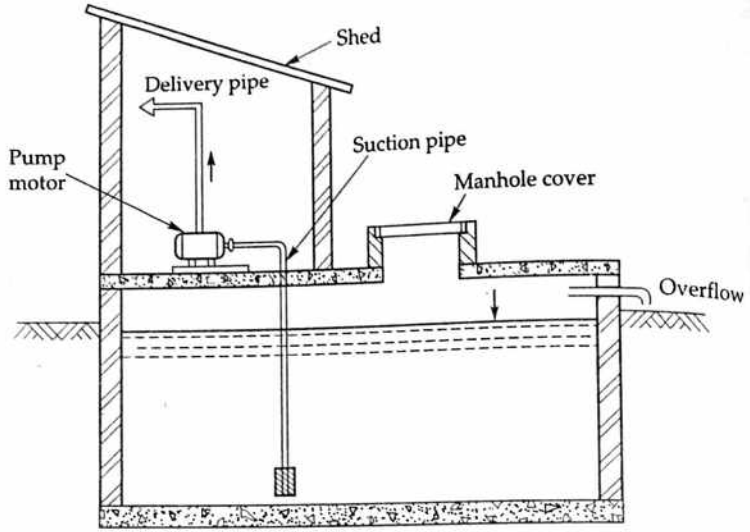
11.6.2. Domestic Under-ground Storage Tanks. Typical details of a rectangular under ground water tank are shown in Fig. 11.5. Such a tank is made of R.C.C. or brick masonry, and has to be structurally designed safe to withstand earth pressure (When the tank is empty). The tank is to be filled from the municipal supply inlet, and is covered from the top to avoid any contamination. The top cover may be provided with a manhole for inspection and cleaning purposes. Suitable pump is installed to lift the water from this tank and up to the overhead tank. Care should be taken to construct an underground tank so that :

(*i*) it is water-tight and does not leak when empty or full ;

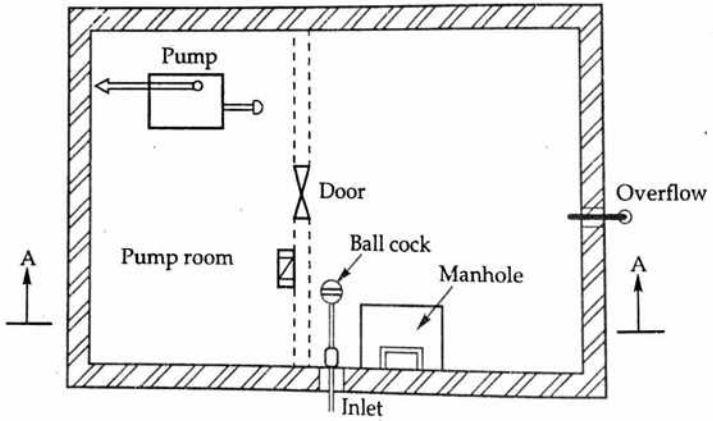
(*ii*) it is not located in low-lying areas, which may permit entry of surface water from the top ;

(*iii*) it should prevent ingress of subsoil water into the tank through leakage ;

(*iv*) it should not be located near sewers, septic tanks, soak pits, oil tanks, or under-ground car parking areas, to avoid seepage of surface waters ;



(*a*) Section A-A



(*b*) Plan

Fig. 11.5. A typical underground water storage tank.

(*v*) the overflow water level in the tank should be above the surrounding ground level, to prevent the surface water from entering the tank through overflows ;

(*vi*) the top slab is designed to carry the load for heavy traffic and fire tenders, which may be likely to move at the given site ;

(*vii*) the masonry or R.C.C. retaining walls of the tank should be structurally safe to withstand the earth pressure from behind, when the tank is empty.

11.6.3. Domestic Overhead Storage Tanks. Overhead tanks placed on the top of the buildings should be properly located as to safely transmit their loads to the beams and columns (or to the walls) in the building. Three types of tanks are usually being used :

(*i*) R.C.C. tanks ;
(*ii*) G.I. tanks ; and
(*iii*) HDPE tanks.

There three types of tanks are briefly described below :

(*i*) **R.C.C. or Masonry Tanks.** Tanks made from R.C.C. or brick masonry have traditionally been used, since they can be easily constructed in any shape and size to suit the site dimensions. Brick masonry tanks are generally used for smaller sizes, while R.C.C tanks are generally adopted in higher sizes. Such a tank has to be made water-tight by adding a water-proofing compound in the concrete and the plaster mix. All inlets, outlets etc. must be inserted at the time of construction. Precast R.C.C. tanks (Fig. 11.6) have also been used, but are becoming obsolete with the advent of HDPE tanks.

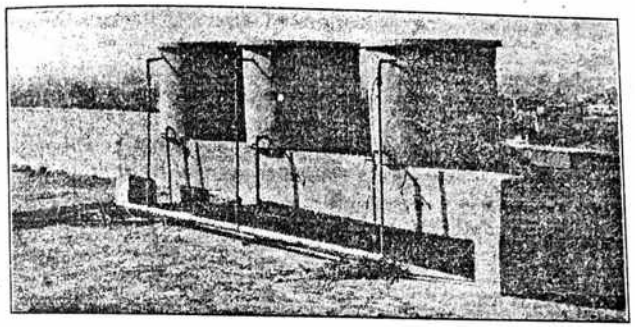


Fig. 11.6. Precast concrete tanks.

(*ii*) **Metal Tanks.** Metal tanks fabricated from *mild steel* or *galvanised iron sheets* have traditionally been used in our houses over the years, as they can be easily fabricated and are light, durable and easy to install. **Galvanised iron (G.I.) tanks** are made from G.I. sheets, usually 16 or 18 gauge, and fabricated by using galvanised angle iron for corner supports and the sheets riveted with G.I. rivets. The non-corrosive nature of galvanised tanks make them ideal for domestic and drinking water. Use of ungalvanised materials in the fabrication of G.I. tanks may, however, corrode them faster. G.I. tanks can be made only in smaller sizes (up to 1800 litres or 1.8 m^3 capacity) due to limitation of sheet sizes. A typical G.I. tank installation is shown in Fig. 11.7.

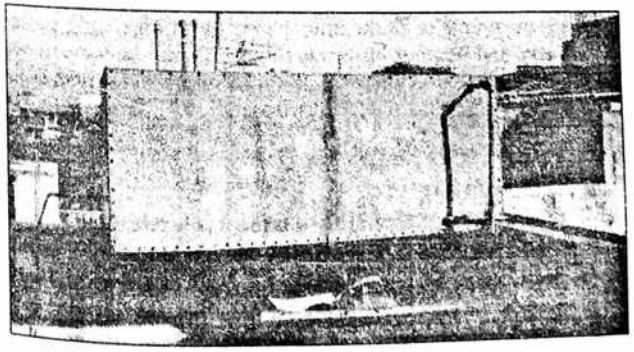


Fig. 11.7. Galvanised iron tank.

Metal tanks using **Mild steel (M.S.)** sheets can however be fabricated to any shape and size with suitable structural supports. The inside and outside surfaces of such tanks are protected against corrosion by suitable paints and coatings. When the tank water is used for drinking and domestic purposes, the paints used must be non-toxic.

(*iii*) **Plastic Tanks.** Plastic tanks are usually made of high density polyethylene (HDPE) or fibre glass reinforced plastic (FRP), or any such strong plastic material. They are usually available in a variety of shapes and sizes, and are light and corrosion resistant. Though, they are not immune to ultra violet radiation from sunlight, yet they can be protected by using appropriate resin coverings. ***Sintex brand HDPE tanks*** (Fig. 11.8) have nowadays become very popular in the country.

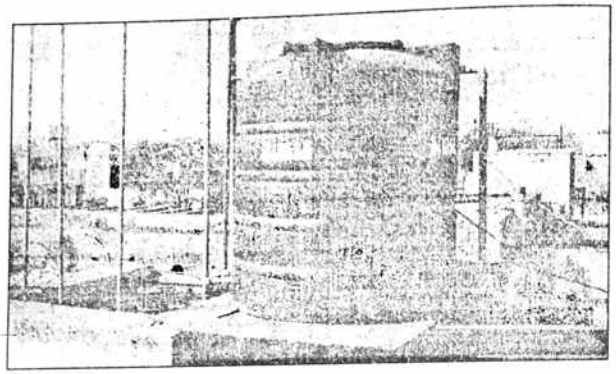


Fig. 11.8. HDPE tank.

11.6.4. General Requirements of Domestic Water Storage Tanks. The domestic water storage tanks should fulfil the following general requirements :

(1) Water Tanks should be water-tight. They should be constructed with non-corrosive and non-toxic materials. They should have smooth surface inside. They should be provided with sufficient number of manholes or accesses for repairs with locking arrangement and corrosion resistant steps, catch rings or ladders to reach the manhole bottom for maintenance.

(2) Water tanks should be provided with a vent pipe for ventilation as well as for prevention of negative pressures when water is drawn from the tank. Such negative pressures, may other wise trigger a collapse if the tank is weak, and retard flow from the tank.

(3) Water tanks should have an overflow pipe. Overflow pipes may also act as a 'warning pipe' to indicate overflow of water in the tank.

(4) Water tanks should have a scour pipe with a plug at the bottom so that it can be emptied easily. In case of under-ground tanks, emptying has to be achieved by using the pumping sets provided for boosting.

(5) Under no circumstances should any overflow or scour, be directly connected to any drain, gully trap or sewer. These connections should be discharged over properly designed drains with a minimum air gap of 75—100 mm.

(6) Vents and overflows must be protected by a suitable gauge or grating to prevent the entry of mosquitoes and insects. The top slab cover should be sloped away for proper drainage and to prevent stagnation of water on top.

11.7. Water Piping Systems in Buildings

The distribution of water within a building from the city mains or other sources such as ground water, can be obtained by several piping systems. Each method has its own merits and demerits depending upon the local conditions. Such piping systems must, however, be designed to provide uniform flow and pressure in all floors and places within certain practical limitations, of course. The piping systems to be adopted will differ when the main supply is used, or when the overhead tank supply is used, or when the underground-overhead tanks supply is used, as discussed below :

11.7.1. Piping System Using Direct Supply. In this system, water is supplied to all the fixtures at all floor levels of a building directly from the city main, through the house connection, as shown in Fig. 11.9. Such a system is, however, successful only when the water supply is available at adequate pressure round the clock. The minimum pressure available will, however, limit the number of floors (or storeyes) to which water can be supplied.

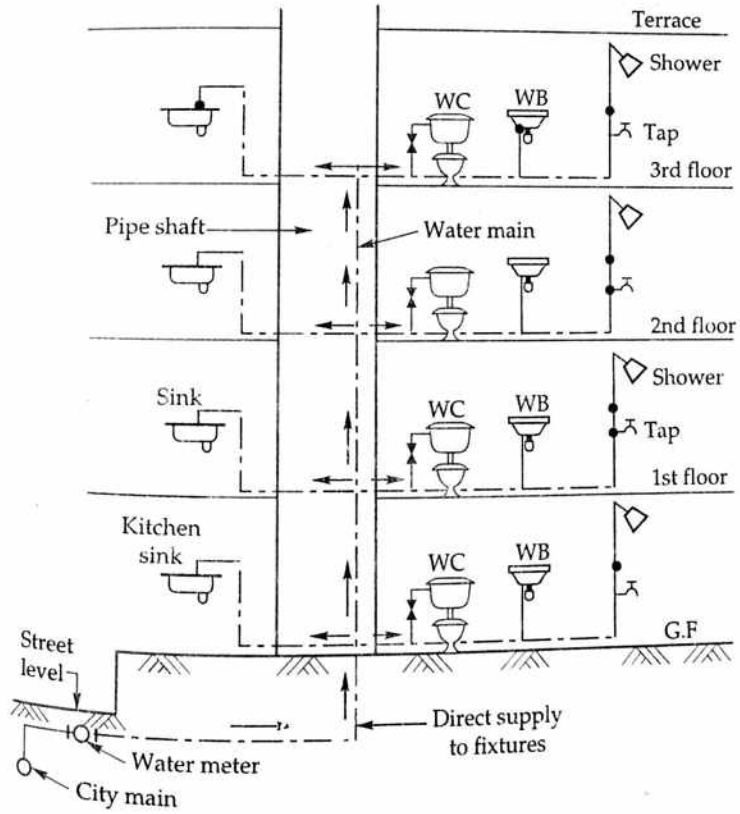


Fig. 11.9. Direct supply.

This system is suitable for countries that maintain high standards for water supply, as it eliminates the need for storage tanks, which may be a major source of contamination. This system is very convenient and economical to users, as it avoids the necessity of storage tanks, pumping sets, etc. Even in such a system, separate provision of supplying stored over-head water to closets and urinals is sometimes made to ensure supply to these fixtures in case of accidental low pressure or breakdown in the city supplies.

11.7.2. Piping System Using Overhead Tank. In many cities, the water pressure in the municipal mains is sufficient to make the water reach up to 3—4 storeyes or higher, but only for a limited period in the mornings and evenings. To meet the water requirement during non-supply hours, water is stored in overhead tanks placed on the terrace, which are filled with a direct connection from the mains. Supply to kitchens is taken from the direct main, while supply to taps for bathing and flushing etc. is taken from the overhead tank. Such a piping system is shown in Fig. 11.10.

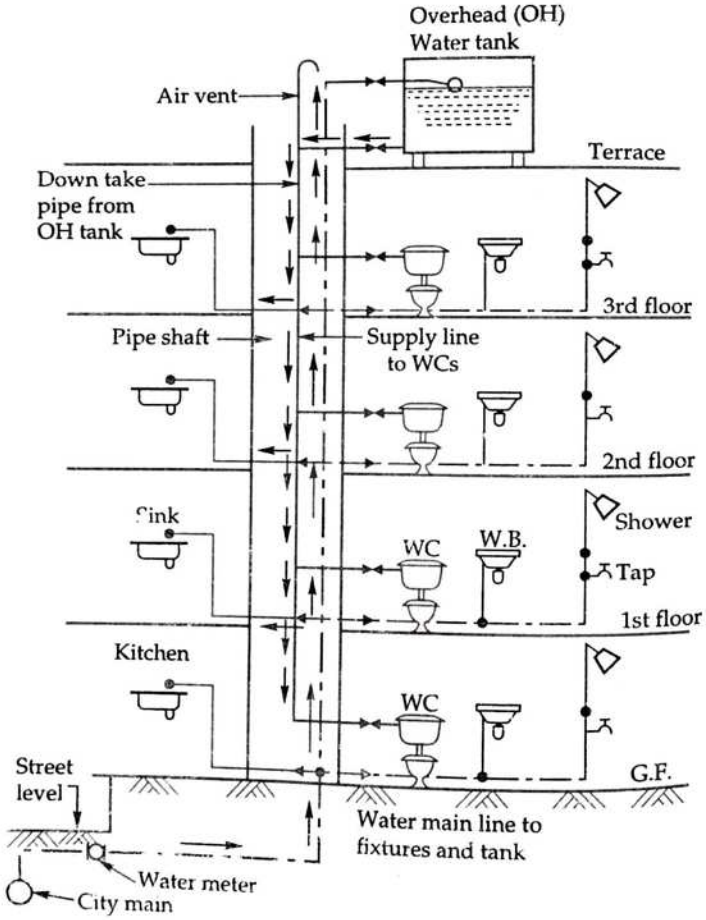


Fig. 11.10. Direct supply supplemented with an overhead tank supply.

In such a system, drinking water has to be stored by the consumer in his vessels everyday during hours of direct supply; or the tank water has to be used for drinking through a R.O. filter. This system is very common in Indian houses and even in smaller hotels and restaurants.

11.7.3. Piping System Using Under-ground—Overhead Tank Supply. When the municipal water supplies are at low pressure, whether intermittent or continuous, then it becomes necessary to store water in under-ground tanks (since direct connection of pumps to the mains is prohibited). The water collected in the underground tank is finally lifted to the overhead tank, as mentioned earlier.

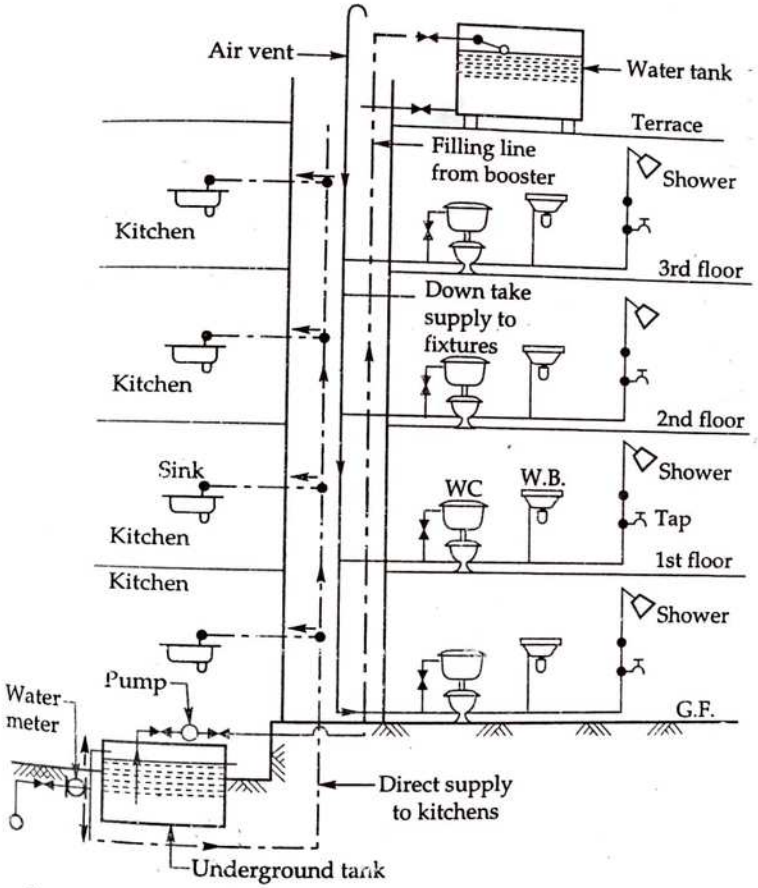


Fig. 11.11. Figure showing the piping system when the overhead storage is made through the underground tank by using a pump set, while the direct supply is used in kitchens and for drinking purposes.

Direct supply connections may also be made to the lower storeyes, as to obtain atleast some amount of direct supply for drinking purposes ; otherwise in all higher floors, the stored water will have to be used even for drinking purposes. Ordinary 2-3 storeyed houses will therefore install a

direct as well as an under-ground—overhead tank supply system, as shown in Fig. 11.11.

Multi-storeyed tall buildings, which cannot use low-pressured municipal supplies will of course have to pump up the entire requirement to the overhead tank by collecting it in the under-ground tank, and then providing all down-take connections from the overhead tank. Use of domestic filters and particularly of R.O. filters, etc. ; are quite essential in such cases to obtain safe drinking water. Such a piping system is shown in Fig. 11.12.

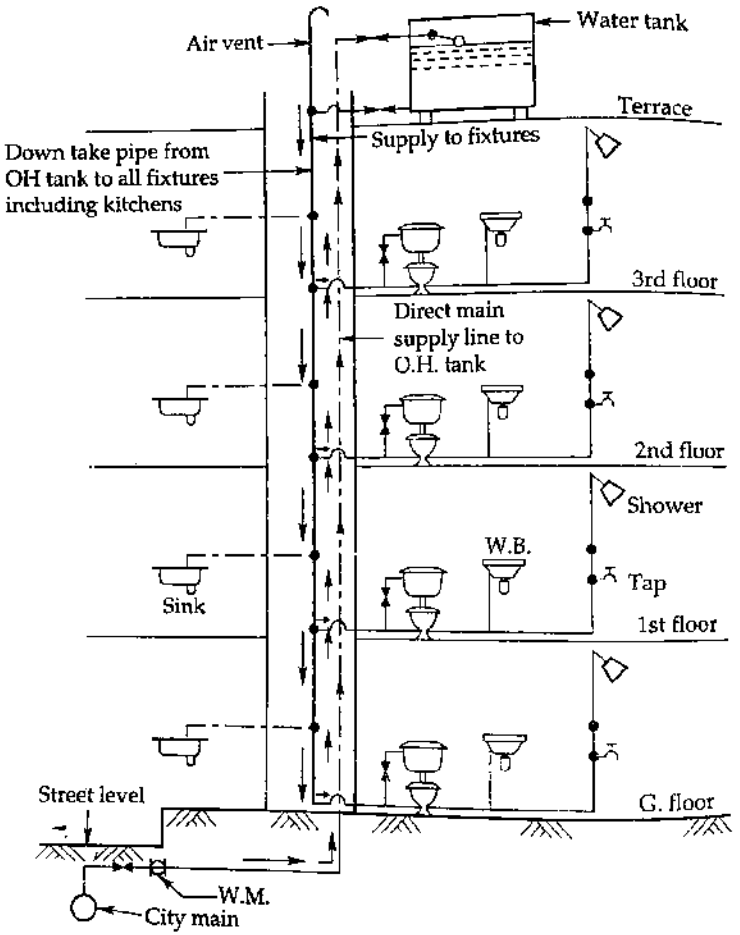


Fig. 11.12. Total overhead tank supply through underground tank.

11.7.4. Pumped Systems. Water can be distributed by an automatic pumping system (such as a hydro pneumatic system) directly to the supply-point, similar to that shown for direct supply. The pressure in the system is boosted by pumping sets that pump water from an under-ground/ground level tanks.

The main advantage of such a system lies in the fact that it eliminates the need for overhead tanks and is ideal for existing buildings when new

overhead tanks cannot be constructed, and also for low buildings when the tanks placed on roof cannot generate enough pressure at upper floors. The system is able to supply water at much higher pressures than it is possible from local overhead tanks. In hot climatic regions, the water supplied from overhead tanks becomes too warm, while in cold regions, it may freeze. A pumped system in such situations is more reliable. Fig. 11.13 shows such a piping system.

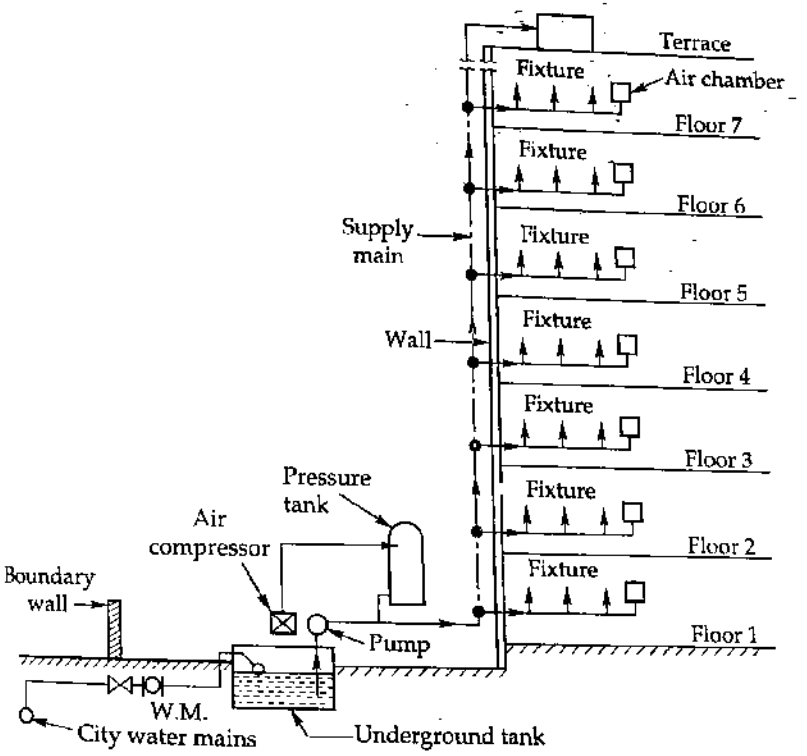


Fig. 11.13. Hydro-Pneumatic piping system.

11.7.5. Other Systems. Many new pumping systems have been developed for special applications and usages :

Continuous running system (Fig. 11.14) supply directly to the point of use where the demand for water is constant and matches with the pump capacity. Typical examples are : air conditioning cooling towers ; steam boilers, and industrial processes having a uniform water demand.

Modern systems include *variable-speed pumping systems, high-pressure systems*, etc.

11.8. Design Considerations for Water Piping Systems in Buildings

The water pipe network in a building will have to be designed as to be able to supply adequate quantity of water to all the floors of the building and with adequate pressure even in the remotest fixture. The following factors will affect this design :

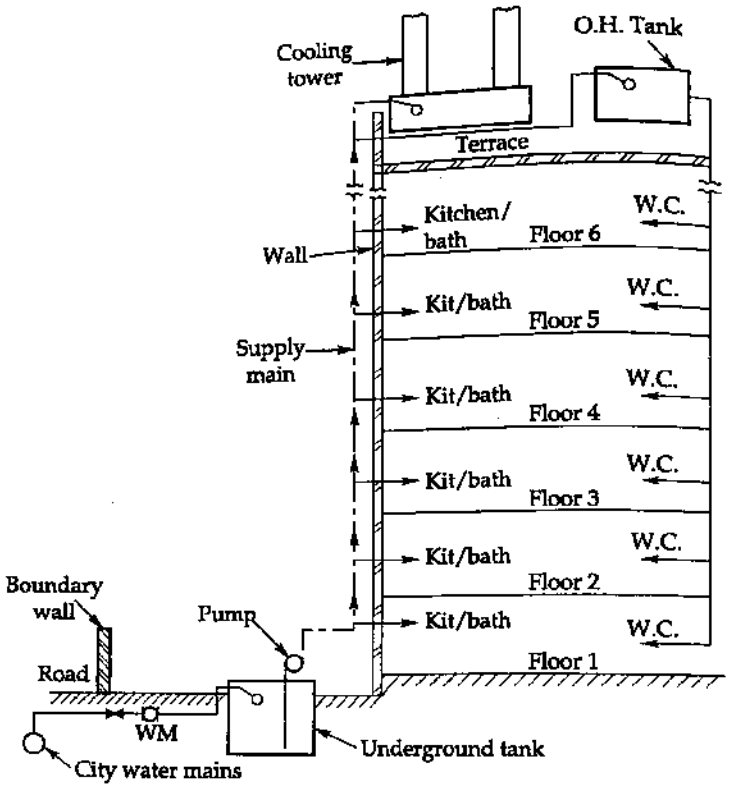


Fig. 11.14. Direct pumping-Continuous running system.

11.8.1. Water Demand for a Building. The daily water demand of a building will have to be worked out by multiplying the per capita demand with the population. The population for each type of building may be estimated on the basis of information obtained from the users. Alternatively, the population may be worked out on the following basis for different types of buildings :

Table 11.3. Projected Populations for Different Buildings

Type of use of the building	*Projected Population*
(*i*) Residence	5 persons per dwelling unit area
(*ii*) Offices	1 person per 10—15 m^2 of plinth area
(*iii*) Schools	Strength of the school plus teaching staff
(*iv*) Hostels	Number of beds + 4.5 × (warden's residence) + staff
(*v*) Hotels	Number of beds + staff + requirement of restaurant seats
(*vi*) Hospitals	Number of beds + staff (residential requirement, if any, should also be added.

Note. 5 to 15% additional population, depending on the usage of the building shall be added for visitors and floating population likely to use the building.

11.8.2. Required Flows for Various Water Supply Fixtures and Probable Simultaneous Demand. The water supply pipe network in a building will have to be designed as to be able to supply adequate quantity of water to all the floors of the building and with adequate pressure even in the remotest fixture. However, all the fixtures of one compartment having a W.C., wash basin, bath tub, ablution tap, etc., in a house or a hotel toilet will not be in operation simultaneously at the same time. *As the various fixtures will require different flow rates to work optimally, it would be sufficient to design the gross* **supply rate** *as to fulfil the required flow rate of the fixture having the highest rate of flow, as only one fixture can be used at a time in one toilet or unit.*

The required flow rates for different fixtures are given in col. (3) of table 11.4. These flow rates of various fixtures are further assigned a **Fixture Unit (F.U.)**, as shown in col. (4) of table 11.4. These units do not have an exact direct mathematical relation with the rate of flow, but depend upon factors like : the average working time of a fixture, etc.

For example, an ablution tap needs 0.12 litres/sec of flow and is assigned 1 fixture unit ; whereas a bath tub needs about 0.20 litres/sec of flow but is assigned 4 fixture units, which only shows that the use of a bath tub in place of ablution tap for bathing purposes is likely to put 4 times load on the water supply of the building, although the rate of flow required is not exactly 4 times.

Table 11.4. Flow Rates and Fixture Units of Various Types of Water Supply Fixtures

S. No.	*Type of Fixture*	*Required flow rate in litres / sec*	*Fixture Units (FU) assigned to each fixture*
(1)	(2)	(3)	(4)
1.	Ablution tap (occasionally used, *e.g.*, for W.C.s, baths, etc.	0.12	1
2.	Ablution tap in (public places of worship, common baths, etc.)	0.15	3
3.	Bath tubs (supply taps)	0.20	4
4.	Showers with spray heads, individual/ common	0.15	2
5.	Showers with 125 mm diameter, individual/common	0.20—0.30	4
6.	Wash basins	0.12	1
7.	Surgeons wash basins	0.15	2
8.	Kitchen sinks (domestic)	0.15	2
9.	Kitchen sinks (canteens, etc.)	0.15—0.20	4
10.	Drinking fountains (connection to water cooler)	0.10	1
11.	Bidet	0.15	3
12.	Water-closet with flushing tank	0.12	1
13.	Water-closet with flush valve	1—1.5	8
14.	Urinals automatic/manual flush tank	0.12	1
15.	Urinals, flush valve operated	0.5—1	6

The gross demand of a building will therefore be worked out by multiplying the number of flats (each having one toilet and one kitchen) in the entire building with the required flow rate of the fixture having the highest flow rate. Thus, if a flat in a building is provided with a water closet (flushing tank), a bath tub, a kitchen sink, a wash basin and an ablution tap, gross demand of the flat will be taken as 0.20 l/s, since this flow rate corresponds to a bath tub, which is the maximum rate among the flow rates of the 5 types of fixtures (see table 11.4).

Just as there is no probability of use of all the fixtures of a flat at any given time, in the same manner, the probability that even one fixture of all the flats of the same building or of a floor will be in use at the same moment is quite remote. If pipe diameters are calculated by assuming that all fixtures are open simultaneously, it would lead to prohibitively large sizes and unnecessary and uneconomical plumbing provisions. 100% simultaneous drew off in all the flats is however, feasible if the water supply hours are severely restricted in the building. It may also occur in buildings such as factories, wash-rooms, hostels, public toilets, showers in sports facilities, places of worship and the like, etc. In all such cases, all the fixtures are likely to be open at the same time during entry, exit and recess, and hence pipe sizes must be determined for 100% demand. However, in buildings with normal usage, the probability of simultaneous demand has to be calculated by statistical methods, which are quite cumbersome, and do involve complex statistical formulae to arrive at the probable flow.

A simple method known as ***'Fretwel's formula'*** gives the probable simultaneous demand as :

$$n = \sqrt{N} \qquad \text{...(11.1)}$$

where n = probable number of fixtures likely to be open at the same time

N = Total number of fixtures in the circuit

In order to avoid the use of such methods, simple tables and graphs have been developed to calculate the probable flow rate for a given number of fixtures to calculate the probable demand of a building or a floor.

The different types of water supply fixtures are assigned different Fixture Units (Table 11.4), and by considering the total *loading* or *fixture units* (*i.e. FU*), the probable water demand (l/s) can be read out by using the standard chart given in Fig. 11.15.

11.8.3. Principles of Distribution and Design Methods. As discussed earlier, the water supply can be supplied either directly from the city main, or from an overhead tank, or from a hydro-pneumatic pumping system. *The principles of calculating pipe sizes will however, remain the same.*

Pipe sizes must be calculated to permit flow at the most unfavourably placed fixture, *i.e.* the remotest fixture on the top most floor. A reasonably accurate estimate of the probable drew-of demand can be made from table 11.4 and Fig. 11.15. Pipe runs may be marked on a scale in plan. It is useful to draw an isometric drawing. The total developed length of the pipe line from its source to all supply points, and the number of fittings, *e.g.* bends, tees, and

elbows in the line, should be worked out. *Pressure required for the water to flow from a tap is its velocity head after overcoming the frictional losses in all fittings including the tap.*

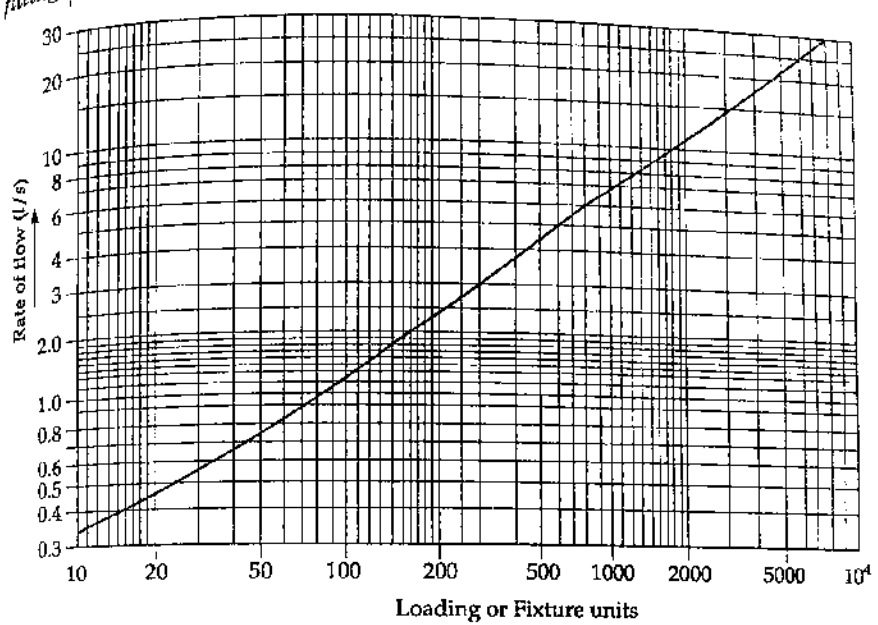


Fig. 11.15. Probable simultaneous demand (log-log scale).

Actual pressure loss through taps and supply fixtures will vary considerably with their types and designs. In western countries having high pressure supplies, the water taps are usually made to have a high pressure loss within the taps. If the same tap is used in a low pressure system, *i.e.* from an overhead tank with a low head, flow through the tap may be very low or not at all. Taps for such low pressure water supply systems should therefore have a low pressure loss within it. If such a low-loss tap, on the other hand, is used in a high pressure system, it would result in a high rate of flow and wastage of water.

Design Principles. Pipe sizes for any riser or vertical downtake can be calculated by using data on rate of flow, loading units, simultaneous demand, frictional loss in pipes, taps and fittings.

When water is supplied from an overhead tank or hydro-pneumatic system, the taps on the top floors will have the least pressure, and are likely to be deficient in water supply when lower taps are open. This problem may be overcome by *balancing the available pressure on all floors. For the upper floors this is achieved by adding the frictional loss of the branch lines to that of the main vertical line.* **The number of floors for which this resistance is to be added to the main line is the ratio between the probable simultaneous demand and the gross demand.**

As the pressure is higher in lower floors, pipe diameters are reduced successively to increase the frictional loss to balance the flow. However, due to the limitation of availability of only a few sizes of pipes in the market, it may

not always be possible to achieve balancing of the losses very accurately. In such cases, a combination of different pipe sizes may be used. Also, the higher residual pressures at lower floors can be reduced by using orifice flanges, or pressure reducing valves, or high-loss water taps. Balancing of pressures can also be achieved to a limited extent by using globe valves to throttle supply in high pressure areas or in high pressure zones.

In a water supply system, which has a very low pressure due to unfavourable location of overhead tanks, supply to the upper two or three floors may cease when taps on the lower floors are open. These floors should be isolated and supplied water by a separate down take.

In very tall buildings, it is always better to divide the building in vertical ***hydraulic zones***, and restricting the maximum pressure to 25—30 m on the lowest floors. The supply to each zone may be given from a separate tank installed over each zone. Say for example, in a 16 storeyed building in Fig. 11.16, the lower eight storeyes are being supplied water from tank No. 1, installed on 8th floor ; while the supply to upper 8 storeyes, is being made from tank No. 2 installed on terrace of 16 storeyed building.

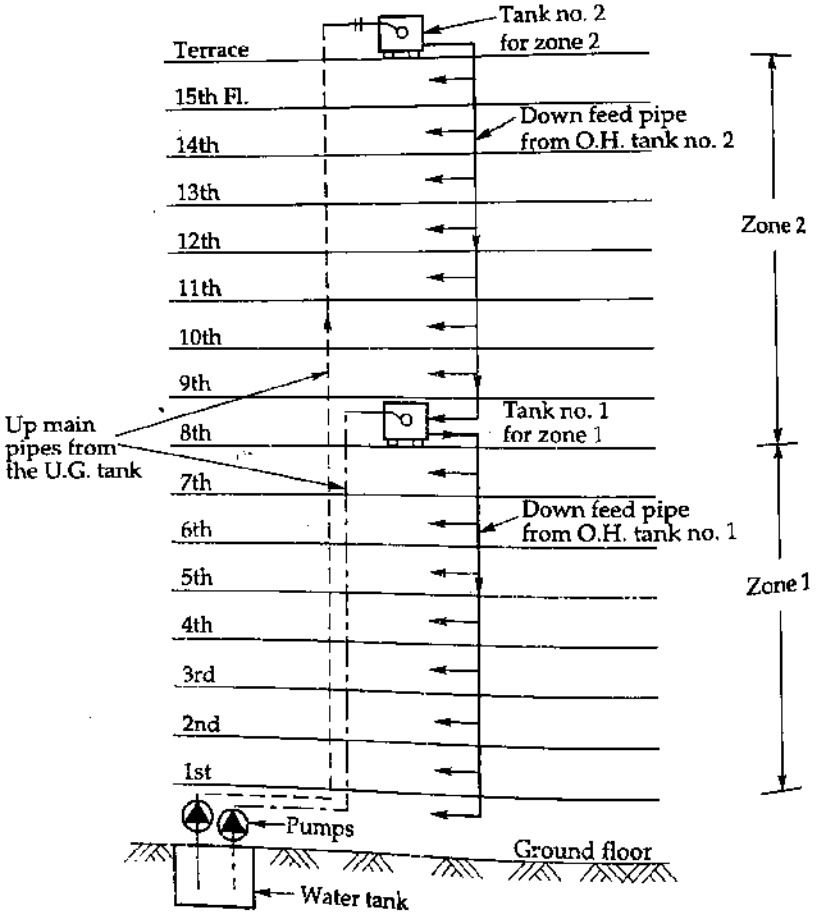


Fig. 11.16. Hydraulic zones for a tall multi-storeyed building.

If however, the supply is made up to the lowest floor in such tall buildings from the overhead tank located on the top most terrace, then pressure reducing valves must be used for supply to the lower floors. The left out static head must be restricted to the limits of the working pressure of the pipes used, since excessive pressure in the water pipes causes not only heavy draw-off and wastage of water, but also leakage in fittings, noises due to water hammer, and wear and tear of the taps. ***Pressure inhibitors*** fixed on each water tap may help in permitting a predetermined rate of flow irrespective of the pressure at each tap.

Example 11.1. *Water supply is being provided in a seven storey building through an overhead tank installed on the terrace at 1 m height. The ht. of each storey is 3 m and water taps are installed at 1 m ht. at each floor. One downtake pipe from the tank is serving two flats on each floor, and each flat is provided with the following water supply fixtures, @ mentioned distance from the downtake :*

1. *Water closet (W.C.)—flushing tank type @ 7.5 m*
2. *Wash basin (W.B.) @ 6 m*
3. *Bath taps with shower @ 4.5 m*
4. *Kitchen sink @ 1.5 m*

Design the pipe diameters for the main (downtake) as well as the branch lines. Suitable values of fixture units and discharge rates for the fixtures may be assumed. The probable simultaneous demand may be worked out by using the standard graph chart given in Fig. 11.15.

Solution. Let us assume the following fixture loads of various fixtures from Table 11.4:

Fixture	*F.U.*
W.C.	= 1
Wash basin (W.B.)	= 1
Bath taps with showers (*B*)	= 4
Kitchen sink	= 2
Total for each flat	= 8

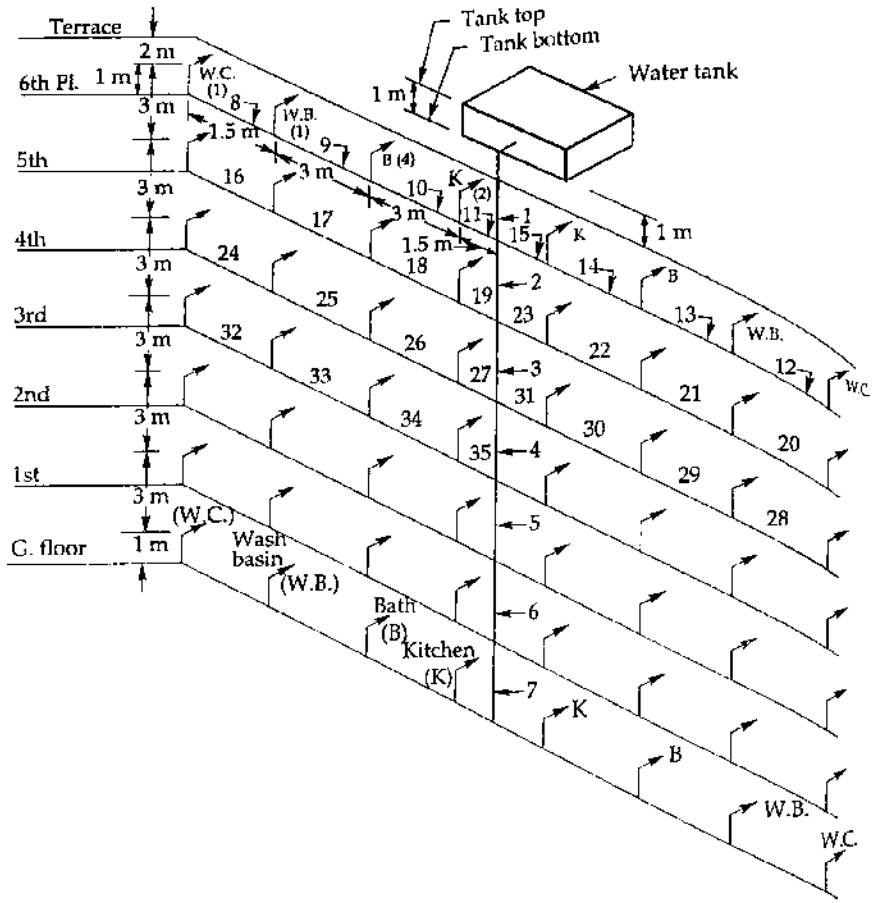
The maximum flow rate among the above mentioned fixtures pertain to Bath taps with showers (Table 11.4). Let it be 0.20 l/s (*i.e.* between 0.2 to 0.3 l/s)

Gross demand of the building

= 2 Flats on each floor × No. of floors × 0.2 l/s

= 2 × 7 × 0.2 = **2.8 l/s**.

An isometric diagramatic sketch of the pipe network is now drawn in Fig. 11.17, wherein the main downtake pipe portions between various floors are marked from 1 to 7. The branch lines from top most floor are numbered starting from 8 onward. Left side branch is marked first starting from extreme end, and continuing the numbers on right side branch from extreme end, as shown. The pipes are thus, numbered from 8 to 11 on left side, and from 12 to 15 on right side on 6th floor. The pipes on lower floors are similarly numbered.



Pipe No.	*Load in Fixture Units*	*Probable flow in l/s from Fig. 11.15*
1	112	1.3
2	96	1.25
3	80	1.00
4	64	0.90
5	48	0.50
6	32	0.50
7	16	0.40
8	1	0.12
9	2	0.24
10	6	0.30
11	8	0.30

Fig. 11.17. Water piping scheme. Load (F.U.) on main pipes (1 to 7) and branches 8 to 10 are shown alongwith flow (l/s) read from Fig. 11.15, in the above table

The probability of simultaneous use is worked out from Fig. 11.15. Thus, for pipe No 1, total load of 2 × 7 × 8 F.U. = **112 F.U.** is used to read out the ***probable demand*** as **1.3 l/s** (From Fig. 11.15). *As the head on the lower floors is high, it will tend to draw all the water ; and hence, larger sized pipes will have to be used in upper floors, and the design shall be done for upper floors to balance the available static head upto the remotest point in the branches by accounting their frictional resistance also. In other words, for upper floors, the frictional loss of the main shall be increased by adding the friction loss of the branch lines, and the available head shall not be less than this total loss.* The number of upper flats for which the frictional resistance of branch lines is to be added is given by the ratio of the probable demand and gross demand.

This ratio in this case is = $\frac{1.3}{2.8} = 46\%$

∴ Total No. of flats for which the branch frictional losses are to be added
$= 14 \times 0.46$
$= 6.44$; say **7**.

Hence, the friction loss of branch lines of both sides on top three floors (6th to 4th floors) and of one side on the 3rd floor shall be computed. Friction loss of branch lines on lower floors can be ignored.

While designing the pipes, we will have to work out the head loss for each pipe as to work out the residual pressure at each of the off-take points. The head loss in pipes can be computed by multiplying the pipe length with the loss of head per metre run—a value to be calculated by Hazen William's formula, which is given by Eq. (6.12 *a*) as :

$$H_L = \frac{1}{0.094}\left(\frac{Q}{C_H}\right)^{1.85} \frac{L}{d^{4.87}} \quad (i.e.\ \text{Eq. } 6.12a)$$

When loss in m per m pipe length is calculated, we have $L = 1$ m

$$\therefore \quad H_L/m = \frac{1}{0.094}\left(\frac{Q}{100}\right)^{1.85} \frac{L}{d^{4.87}} \quad (\text{Using } C_H = 100)$$

Here Q is in m^3/sec, and d is in m. If however, Q is in l/sec and d (pipe dia) is in cm, then the formula reduces to :

Loss of head in m/m

$$= \frac{1}{0.094}\left(\frac{Q \text{ in l/s}}{1000} \times \frac{1}{100}\right)^{1.85} \cdot \frac{1}{(d \text{ in cm}/100)^{4.87}}$$

or

$$\boxed{\text{Loss of head in m/m} = 32.875 \frac{(Q \text{ in l/s})^{1.85}}{(d \text{ in cm})^{4.87}}} \quad ...(11.2)$$

The various types of pipe fittings can be counted and converted into equivalent pipe lengths, by using the factors given in table 11.5.

Table 11.5. Equivalent Pipe Lengths for Fittings

Pipe fittings	*Equivalent length of pipe in pipe diameter*
90° Elbow	30
Tee	40
Gate Valve	20
Globe valve, bib taps	300

When counting of fittings is difficult, certain adhoc percentages as given in table 11.6, may be adopted to add to the pipe length.

Table 11.6 Percentage of pipe length to be added for pipe fittings

15 mm dia pipe	100%
20 mm dia pipe	100%
25 mm dia pipe	50%
50 mm dia pipe	25%

These percentages have been used in this example to compute head losses in pipe fittings, as shown in the calculation **Chart-Table 11.7.**

To work out the pipe diameters scientifically it is easier to tabulate the data, assume pipe diameters, and work out the residual head. The figure or isometric view, as drawn in Fig. 11.17, will help in completing the computation table (Chart Table 11.7).

The computing procedure is a trial and error method. If any pipe diameter assumed is over-sized or under-sized, the same has to be revised, and the calculations redone. It will be noticed that in the table, the head loss in each sector or floor is nearly equal to the available static head at that point. The pressure in the lower floors is however, higher than the static head available. This is to some extent unavoidable due to limited pipe sizes available in the market, and limitation of velocity of flow in pipes (maximum : 1.5—2 m/s). The computation table is otherwise self explanatory, and the finally computed pipe sizes are tabulated below :

Pipe	*Size*	
Main Down Take Pipe No. 1	50 mm dia.	Ans.
Main Down Take Pipe No. 2	40 mm dia.	
Main Down Take Pipe No. 3	40 mm dia.	
Main Down Take Pipe No. 4	32 mm dia.	
Main Down Take Pipe No. 5	25 mm dia.	
Main Down Take Pipe No. 6	20 mm dia.	
Main Down Take Pipe No. 7	20 mm dia.	
Branch pipes—all branches	20 mm dia.	

Example 11.2. *Water is to be supplied in a seven storey Low Income Group housing building having 4 flats on each floor. Each flat is provided with a toilet and a kitchen, and average number of persons living per flat is 5. The*

Chart Table 11.7. T

Pipe location	*Designed Pipe No.*	*Load on pipe in Fixture Units*	*Probable demand corresponding to Loading units of col. (3) Read from Fig. 11.15 in l/s (Q)*	*Pipe Length from Fig. 11.17 in m*	*Assumed Pipe dia in mm (d)*	i %a (1
(1)	(2)	(3)	(4)	(5)	(6)	
Top Floor						
Main	1	112	1.3	5.0*	50	
Branch L	8	1	0.12	1.5	20	
Branch L	9	2	0.24	1.5	20	
Branch L	10	6	0.30	3.0	20	
Branch L	11	8	0.30	1.5	20	
Branch R	12 to 15	← Same as in pi				
5th Floor						
Main	2	96	1.25	3.0	40	
Branch L	16 to 19	← Same as in pi				
Branch R	20 to 23	← Same as in pi				
4th Floor						
Main	3	80	1.0	3.0	40	
Branch L	24 to 27	← Same as in pi				
Branch R	28 to 31	← Same as in pi				
3rd Floor						
Main	4	64	0.9	3.0	32	
Branch L	32 to 35	← Same as in pi				
2nd Floor						
Main	5	48	0.8	3.0	25	
1st Floor						
Main	6	32	0.6	3.0	20	
Ground Floor						
Main	7	16	0.4	3.0	20	

*Water level in tank is 2 m above roof and 3 m is the height of the roof—giv

Table

Pipe fitt
90° Elbow
Tee
Gate Valve
Globe valve, bil

When counting of fi
table 11.6, may be ado

Table 11.6 Percei

15 mm dia pi
20 mm dia pi
25 mm dia pi
50 mm dia pi

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To work out the pip
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sector or floor is nearly
pressure in the lower flo
This is to some extent u
market, and limitation of
computation table is othe
sizes are tabulated below

Pipe
Main Down Take
Main Down Take
Main Down Take
Main Down Take
Main Down Take
Main Down Take
Main Down Take
Branch pipes—all

Example 11.2. *Water is t*
housing building having 4
toilet and a kitchen, and a

municipal water supply in the area is intermittent and irregular with supply restricted to 3 hours in the morning and 3 hours in the evening. Separate water meters are not to be provided in flats. Design the sizes of the various units that are to be installed to ensure continuous tank supply. The living standards do require average per capita daily demand of 160 L/d/p.

Solution. Av. daily water requirement

= Av. daily per capita demand × Population

= 160 × (7 × 4 × 5) = 22400 L = 22.4 m^3

Since there is no separate ground water or effluent supply, and the municipal supply is to be used for domestic as well as flushing purposes, separate storage for domestic and flushing purposes are not required*. An overhead tank is proposed to be installed for the building at the terrace.

However, due to municipal prohibitions, the pumps to lift water to the overhead tank shall not be attached to the municipal main ; rather an underground tank shall be provided to collect municipal supply in the morning as well as evening, during 3 hrs each. This water shall be lifted to the over-head tank, say by pumping for 2 hrs in the morning and evening each.

(*i*) ***Capacity of Under-ground Tank.*** Strictly speaking, the capacity of the under-ground tank would depend on the rate of inflow (supply) and rate of outflow (*i.e.* lifted water). The rate of inflow may, however, vary. The pattern of consumption may also change with seasons. A good deal of accuracy in calculating the tank size is, therefore, not required. An under-ground tank of 24 hrs consumption or daily requirement is sometimes recommended to cater for the prolonged shut down in municipal supply. ***Normally, however, in shift supplies, an under-ground tank of $\frac{2}{3}$ rd of one day requirement, and an over-head tank of $\frac{1}{3}$ rd of one day requirement may suffice.***

So, let us provide an under-ground tank of $\frac{2}{3}$ rd of one day's requirement of 22400 L.

∴ Tank capacity

$$= \frac{2}{3} \times 22400 \text{ L} = 15000 \text{ L} = 15 \text{ m}^3$$

Provide a tank of 4 m × 2.5 m × 1.5 m (water depth) size with a free-board of 0.2 m.

Over-all tank size = **4 m × 2.5 m × 1.7 m. Ans.**

(*ii*) ***Capacity of Over head Tank.*** Let us provide an over head tank storage equal to $\frac{1}{3}$ rd of one days requirement.

∴ Capacity of O.H. Tank

$$= \frac{1}{3} \text{ rd of one days requirement}$$

*When separate provisions are made for flushing and domestic uses, then the flushing requirement may be taken as $\frac{1}{3}$ rd and domestic requirement at $\frac{2}{3}$ rd.

$= \frac{1}{3} \times 224000$ L

$= 7500$ L. **Ans.**

The size of the overhead tank may be decided as per site availability.

(iii) ***Size of Rising Main.*** Average daily water requirement to be pumped

$= 22{,}400$ L

Pumping hours allowing power failures

$= 4$ hours. (assumed pumping for 2 hrs in the morning and evening each)

$\therefore$ Pumping rate $= \frac{22400}{4}$ L/h

$= 5{,}600$ L/h $= 1.56$ L/s

Now, assume that the length of the rising main from the underground tank to the over-head tank is measured from the building plan to be 30 m.

Let us use 50 mm dia rising main.

Let the equivalent length of all the pipe fittings like elbows, gate valve, etc. be 22 m (6 elbows × 3 m + 1 gate valve × 4 m = 22 m)

$\therefore$ Total pipe length causing head loss

$= 30 + 22$ m $= 52$ m

Let us use 40 mm dia pipe for the rising main, to carry 1.56 L/s. The velocity in the pipe shall be

$$= \frac{1.56}{1000 \times \frac{\pi}{4} \times (0.04)^2} = 1.24 \text{ m/s.}$$

Now, Head loss H_L/m

$$= 32.875 \frac{(Q \text{ in L/s})^{1.85}}{(d \text{ in cm})^{4.87}} \quad (i.e. \text{ Eq. } 11.2)$$

$$= 32.875 \frac{(1.56)^{1.85}}{(4)^{4.87}} = 0.0875 \text{ m/m}$$

Total head loss in 52 m length of pipe

$= 0.0875 \times 52$ m $= \mathbf{4.6}$ **m.**

Note. Use of smaller sized rising main will increase the velocity and head loss, causing water hammer noises, and additional burden on pump.

(iv) ***Design of Pump Capacity :***

Rate of pumping $= 1.56$ L/s

Head on the Pump.

Suction head $= 1.7$ m (depth of under ground tank)

Static head $= 23$ m

assuming the O.H. tank to be placed on roof with its W.L. 2 m above roof level, which is 21 m above the ground level

Frictional loss $= 4.6$ m (calculated)

Minimum Residual head required at inlet

$= 3$ m (assumed)

Total, delivery head $= \mathbf{32.3}$ **m**

H.P. of pump $= \frac{\gamma_w \cdot QH}{0.735 \cdot \eta}$

Assume η = efficiency of pump and motor

$= 0.6$

$r_w = 9.81$ kN/m^3

$Q = 1.56$ L/s $= \frac{1.56}{1000}$ m^3/s

$H = 32.3$ m

$$\text{H.P.} = \frac{9.81 \times \frac{1.56}{1000} \times 32.3}{0.735 \times 0.6}$$

$= \mathbf{1.13}$; say 1.5 H.P.

Let us use 1.5 H.P. Centrifugal Pump Set. **Ans.**

HOT WATER APPLIANCES AND INSTALLATIONS

Hot water is required in houses and buildings for bathing and washing of clothes and utensils, etc. Higher temperatures melt oil and grease from human body, pot and panes, making the cleaning work easier. Bathing with hot water opens the body pores, washing dirt and sweat easily, and giving a sense of freshness. Evidently, in winter season, the requirement of hot water becomes all the more necessary.

In order to meet the hot water requirement of houses, hostels, hotels, etc., hot water systems are designed and installed in accordance with the requirement of a particular building.

In its rudimentary form, water in Indian houses, particularly in rural areas, is heated in pots on local stoves and chulahas. Where electricity is available, water may be heated on electric heaters. Such appliances may, however, provide us with small quantity of hot water only, and are risky and dangerous, as the hot water may spill during transit, causing burns to elders or children. Open flame chulahas, and stoves may also cause burn injuries and harmful gases. *Hamams* and *electrical geysers* have however now been invented and are being largely used to fulfil domestic demand of hot water.

11.9. Hot Water Requirement

Hot water is used in kitchens and bathrooms in residential areas. The quantum of consumption of hot water widely depends upon the weather conditions of the area, as well as the habits and living standards of the people. In India, natural climatic temperature widely varies from near Arctic conditions in Leh (J & K State)—where temperatures may get as low as (–) 20°C in winter, to as high as 40—45°C in southern India. Hot water requirement therefore varies widely, and will be much high in areas of cold climates as compared to the hotter regions.

In addition to its requirement for domestic uses, large quantity of hot water may be required for laundries, and industrial kitchens and in buildings like hotels, hospitals, schools, hostels, industrial canteens, etc. Hot water may also sometimes be required as a heating medium for space heating and air-conditioning. Although the water is not consumed in such room heating uses, yet the boiler capacity has to be increased to provide for the heat load. Average daily hot water requirement in residences and other types of buildings in cold climates are given in table 11.8.

Table 11.8. Average Daily Hot Water Requirements in Buildings in Cold Climates

S. No.	*Type of Building / Use*	*Average daily hot water demand in L / d / head*
(1)	(2)	(3)
1.	*Residences*	
	(*a*) Residential (with showers/taps)	45
	(*b*) Residential (with bath tubs)	135
2.	*Factories*	
	(*a*) Factories (with showers)	90
	(*b*) Factories (ablution taps only)	30—45
3.	*Hospitals*	
	(*a*) Beds (patients, clean-up nurses station, etc.)	180
	(*b*) Staff, doctors and nurses	90
	(*c*) Visitors	10
4.	*Hostels*	
	(*a*) Colleges, schools, nurses hostels, etc.	135
	(*b*) Offices, schools, colleges (day-use where required)	145
5.	*Hostels*	
	(*a*) Rooms with showers	70—90
	(*b*) Rooms with tubs	135
	(*c*) Hotel staff	25—45
	(*d*) Visitors (Av. 2/room daily)	15
	(*e*) Swimming pools, changing rooms	45
	(*f*) Kitchens/restaurants	5/meal served
6.	*Laundries* (3—5 kg of linen per room (includes staff uniforms, restaurant linen)	20/kg of laundry

11.10. Domestic Hot Water Appliances

The various appliances, which are usually used for generating hot waters for domestic uses, are :

(*i*) *Hamams* ; and

(*ii*) *Electrical geysers.*

These appliances are discussed below :

11.10.1. Hamams. Hamam is an old displacement type hot-water heater, using solid fuels like coal, wood, etc. It is found not only in India, but also in other countries. This appliance (Fig. 11.18) had been in use for many centuries, and still serves its purpose safely, if installed in a proper manner.

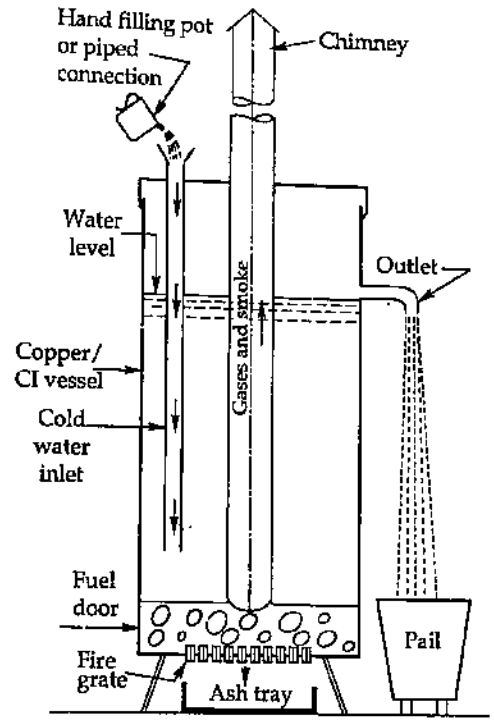


Fig. 11.18. Traditional Hamam.

11.10.2. Electrical Geysers or Hot Water Heaters. The natural hot water springs found in many parts of the world (such as the one in Tatapani near Shimla, one near Sohna, one near Manali—all in India) are known as 'Geysers'. Modern appliances generating hot water artificially have also acquired the same name. Electrical Geysers generate hot water using electrical *heating elements*. LPG may be used in **gas water heaters.**

Electrical geysers have now a days become a very common appliance of our modern life, and even in our developing country, geysers are now-a-days being used in houses even in smaller towns and some advanced villages, where electricity is available.

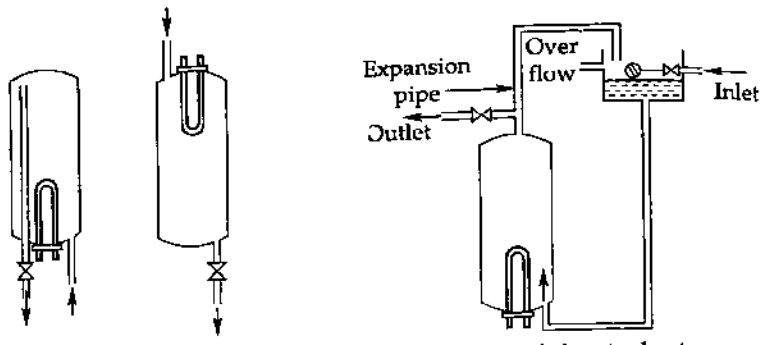
Types of Electrical Geysers

Instantaneous and *Storage* types of geysers are usually available in the market these days, and are discussed below :

11.10.2.1. Instantaneous electrical geysers. These geysers are designed for direct connection to the water-taps, and the electric heating element is switched on after the flow of water is established. Such an appliance requires a heating element bound round a pipe, which heats the water to a specified temperature matching the rate of flow. The heating element has to be of a much higher capacity (wattage) as compared to the one required in a storage heater. An electrical load of 6 kW is found to give warm water at a rate of about 3 litres/minute, while an electrical load of 3 kW will give warm water at a rate of about 1.4 litres/min. These heaters not only require heavy wiring switch board connections but also require very efficient earthing to avoid shocks. *Sub-standard instantaneous heaters can lead to fatal electrical shocks.*

It would therefore be advisable to install a storage type of a geyser rather than using an instantaneous geyser. Storage geyser shall even be preferred to an **electrical immersion rod**—which is sometimes used to heat up a bucket of water, by direct immersion in water.

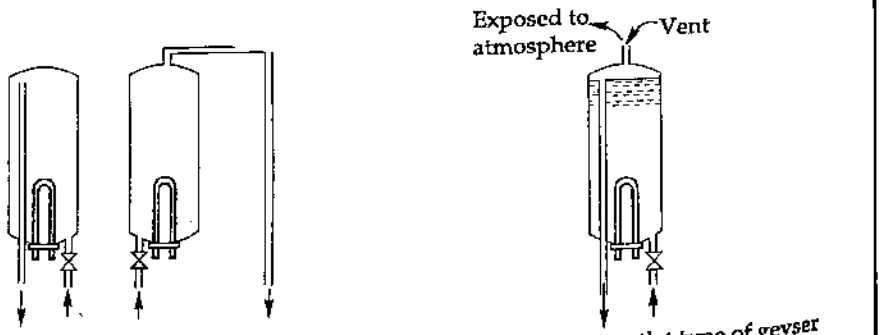
11.10.2.2. Storage geysers or Storage water heaters. These geysers are usually of **pressure type** (Fig. 11.19), which are connected with an assured water supply from a cold overhead storage tank. Such geysers should under no circumstances be connected to the direct main water supply, which may stop and suck back the water from the geyser (inlet may become the outlet) if the check valve fails to function, which is quite a normal occurrence.



(*a*) Closed water heaters (*b*) Cistern fed water heater

Fig. 11.19. Pressure geysers.

Direct main water supply can however, be connected to the sparcely used **non pressure type** or **open outlet type** of a geyser (Fig. 11.20), in which the outlet is kept open and not provided with any bib cock or stop cock. In such a geyser the inlet stop cock is opened to draw water from the open outlet.



(*a*) Closed unvented open outlet type water heaters

(*b*) Vented open outlet type of geyser (the geyser is open to atmosphere so that under no conditions of use can the pressure at the surface of water be other than the atmospheric

Fig. 11.20. Non-pressure geysers (open outlet type water heaters).
[**Note.** There is no bib cock or stop cock at the outlet.]

Thus, when hot water is not being drawn, the supply to such a geyser (either direct or from over-head tank) will be closed by closing the inlet stop cock, and hence there will act no water pressure on the geyser. When supply is open, equal amount of hot water will get out of the geyser. There will, thus, be no water pressure ever acting on the geyser ; and hence such a geyser is called a *non-pressure type of a geyser*. Such geysers are manufactured in smaller capacities, and are rarely used in modern day life, since it gives only hot water in a bucket and won't allow automatic mixing of cold and hot water.

In the ***pressure type of storage geyser***, however, the inlet is not blocked and its stop cock even if provided for convenience of connections, is kept always open. The outlet is connected to a bib cock (water tap) through a stop cock. The outlet stop cock is usually kept open, and hot water can be drawn by just opening the hot water tap. In such a geyser, therefore, the over-head tank supply is always open, which keeps the geyser always under the pressure of water. This pressure allows hot water to be distributed to a number of points, and permits mixing of hot and cold supply through a mixing fitting (mixer). The non-pressure type of geyser though provided with a *thermostat*, does not, however, require any ***expansion pipe*** or ***vacuum breaker***, as are required by the pressure type of geysers, as shall be discussed a little later. The difference between pressure and non-pressure types of geysers is illustrated in Fig. 11.19 and 11.20.

A storage geyser consists of a thermally insulated cylindrical or rectangular vessel, one or more electric heating elements*, *thermostat(s)** to automatically cut off and cut on the electric current, and *pipe connections* ; and is assembled by the manufacturers ready for installation on site. The rated capacities and power input (kW) requirements of storage heaters of various sizes, as per recommendations of IS 2082—1965, are shown in table 11.9.

Table 11.9 Rated Capacities and Power Input Requirements of Storage heaters of various capacities

Rated capacity in litres	6	15	25	35	50	70	100	140	200
Recommended input in kW	1	1	1	1.5	2	2	3	4	5

When operating continuously under thermostatic control, a 2 kW geyser is adequate for small dwellings, requiring 90 litres of hot water storage. Smaller sized geysers of 1 kW (15 to 25 litres) can also serve the purpose in small households, but there would have to be some time gap between the use of hot water by one person for bath and the next bather.

A photoview and section of a closed automatic geyser (pressure type ofcourse) provided with a pressure release safety valve near the inlet is shown in Fig. 11.21 (*a*) and (*b*).

*Two electrical heating elements are usually provided in *dual heater type* water heater having two heating units-one towards the top container and the other near the bottom, the unit being independently controlled. The shape of this geyser is generally cylindrical.

Fig. 11.21 (a) Photoview of an automatic pressure geyser (storage water heater).

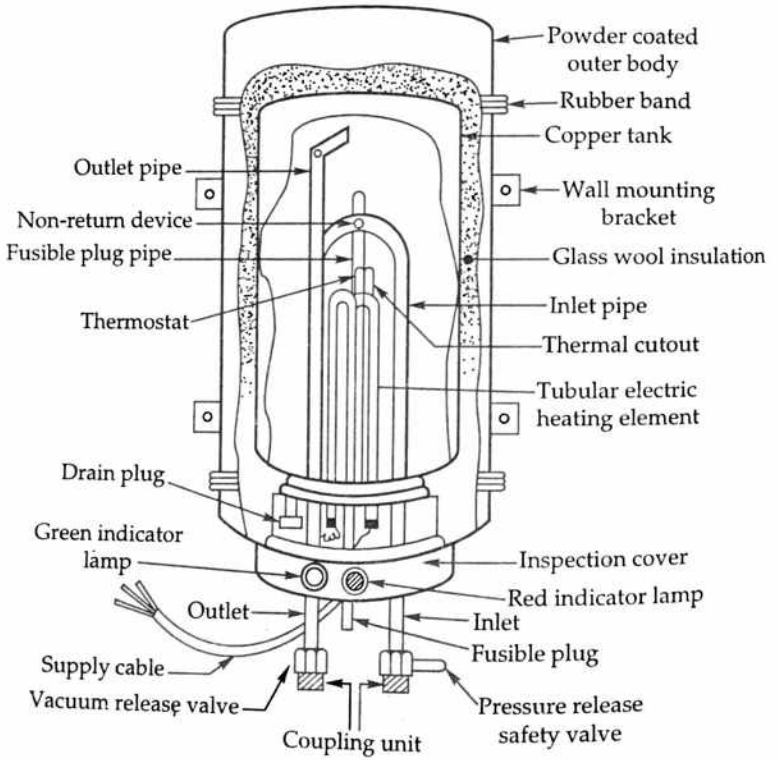


Fig. 11.21. (b) Sectional view of a closed automatic pressure geyser (Storage type water heater).

In all storage geysers, the cold water enters at the bottom of the geyser tank, while the hot water from the top is withdrawn. The withdrawn hot water is infact displaced by the equivalent cold feed entering through the inlet. This keeps the geyser always filled up with water, thereby eliminating the danger of the geyser becoming empty and burning of the heating element.

Now, we will discuss the dangers posed by the pressure geysers and their remedial measures.

Geyser Implosions. All pressure type automatic geysers are controlled by an automatic thermostatic switch, which cuts on and off the electric supply to the geyser at a pre-set temperature. They operate at a differential temperature of about 10—12°C and cut out say at 70°C (The temperature setting is adjustable).

If the water supply in such a geyser is from an over-head tank installed at a height of 10 m, the pressure in the heater = gauge pressure + local atmospheric pressure = (98.1 + 101) kN/m^2 = 199.1 kN/m^2, and the boiling point of water corresponding to this pressure will be 119.5°C (Appendix Table A-5)*. **If the thermostat of the geyser fails to cut off** at set temperature of 65—70°C, *the geyser will continue to heat the water up to its boiling point at the given pressure* (119.5°C in the case of 10 m high storage tank). During such an over-heating, if no pressure relief valve is provided in the geyser, or the provided one fails to operate, then eventually the pressure over the water in the geyser increases due to steam, whereby although the water stops boiling but the steam continues to form and the temperature of water rises further, up to the value of increased boiling temperature. At sufficiently elevated temperature, the weak places in the geyser body like solder at seams of the vessel etc. may melt and leak, and the super-heated water (at high pressure) with release of pressure may change into flash steam with **explosive violence**. Many a times, when hot water taps are opened, the over-heated water coming out from the tap, immediately changes into steam (*flash steam*) within a few seconds, sometimes with explosive violence, like a bomb blast, causing heavy damage to life and property. To avoid such accidents, all pressure type automatic geysers must be constructed from enamelled metal sheets capable of sustaining high pressures, and each geyser must be provided with a good quality *temperature-pressure relief valve*, set to release at preset settings. This should be in addition to installing **fusable link**, similar to the one provided in domestic pressure cookers.

Vacuum Formation in Geysers. Sometimes hot water geysers are installed on different floors of a building on a common down-take pipe, as shown in Fig. 11.22. In such installations, if the outlet from the over-head tank (*i.e.* the feed valve or stop cock is closed and no check valves are provided on the inlets of the geysers, then the water from the geysers of upper floors

*Water boils at 100°C at local atmospheric pressure, *i.e.* when the toal absolute pressure is about 101 kPa (101 kN/m^2) with zero gauge pressure due to water head. If there is some gauge pressure over the local atmospheric pressure, such as 10 m or 20 m of water head (*i.e.* 98.1 kPa or 196.2 kPa), then the boiling temperature of water increases, as shown in Appendix table A-5.

will be drawn, if the hot water is withdrawn from a geyser in the lower floor, and if all the taps on the upper floors are shut. *This will generate a negative pressure in the upper level geysers and the created vaccum will cause inward collapse of the geysers if they are not constructed strongly.* Such possibilities can be avoided by installing **an expansion pipe** from the top of the geyser, extending above the water level in the over head tank, as shown in Fig. 11.23. This pipe will also serve the function of a vent pipe to help remove *air-locks* from the line. During initial filling of the geyser, it expels the air from within the system ; and also acts as an expansion pipe for the heated water. This pipe also allows the exit of any steam formed due to over-heating. The cold feed pipe of the geyser and the vent pipe act as a 'U' tube, providing hydraulic balance in the system. The expansion pipe has to be separate and independent for each geyser. It should only rise up, without dipping below the level of its origin and should not have any kind of valve on it.

A check valve or a vacuum breaker installed on the geyser inlet also eliminates the problem of negative pressures and collapse of upper floor geysers.

Independent expansion pipes are difficult to be installed in tall buildings and one has to depend on other safety devices like pressure relief valves, vacuum breakers, check valves, etc.

Standing Energy Loss in Automatic Geysers. The automatic geysers are usually kept switched on in modern days, as to ensure availability of hot water round the clock. In such a case, a geyser will go

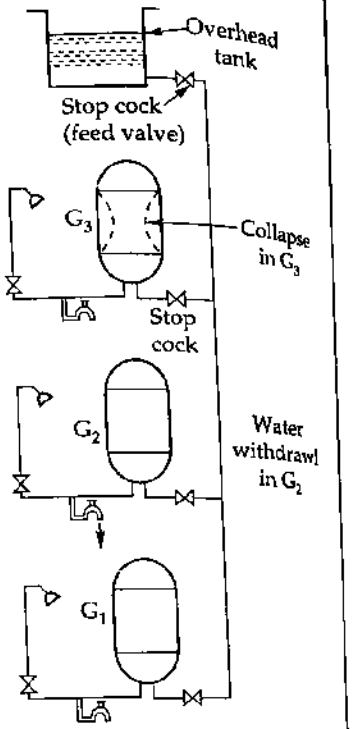


Fig. 11.22. Collapse of geysers due of negative pressure. (*i.e.* vacuum formation in geysers)

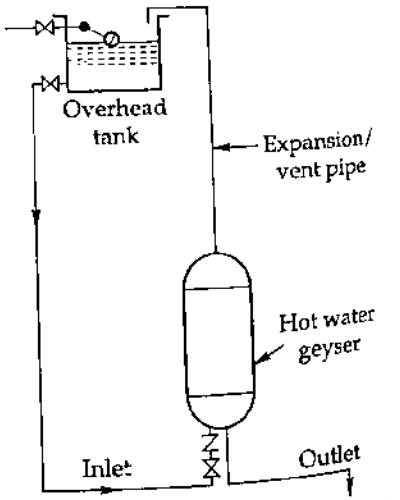


Fig. 11.23. Provision of Expansion pipe in automatic geysers to avoid negative pressures and collapse of geysers of upper floors when installed on a common down-take pipe.

on loosing heat inspite of thermal insulation provided on its body. The loss of heat will eventually depend on the cut off temperature of the geyser (*i.e.* the maximum temperature at which the thermostat will automatically switch off the electric current—kept at about 65—70°C) and the ambient air temperature. All the geysers manufactured in India are required to ensure some maximum energy loss, when tested over 24 hrs at a temperature difference of 45°C between the ambient air temperature and thermostat cut out temperature. BIS Code IS 2082 : 1993 has specified the maximum energy loss over 24 hours to be as shown in table 11.10.

Table 11.10. Limiting Values of Standing Loss In Automatic Geysers

Rate capacity of geyser in litres	*Max. Permissible Energy Loss in kilowatt hour per day for 45°C difference*
6	0.880
10	1.100
15	1.265
25	1.540
35	1.760
50	2.035
70	2.310
100	2.640
140	2.970
200	3.300

11.11. Solar Water Heaters

11.11.1. Introduction. Solar water heating, now a days, has become quite cost effective and saves on fossil fuels or electricity, which are usually scarce. Solar water heaters have therefore, gained popularity in countries notably Japan, Israel, Australia, and U.S.A., where commercial solar water heaters are readily available. Solar water heaters have also been designed in India to suit the indigenous raw materials, tropical climate, portability and ease of manufacture etc., so as to produce efficient and economical solar water heaters. National Physical Laboratory (NPL) New Delhi, and C.B.R.I. (Central Building Research Institute) Roorkee have remained associated with the design work of solar water heaters favourable to Indian conditions.

11.11.2. Solar Water Energy. The design of a solar water heater and particularly that of the area of the absorber unit, is primarily governed by the solar energy available at the place of installation of the solar heater. In order to help designers, the solar energy available during different months of the year at four widely separated cities of india ; *i.e.* Poona, Delhi, Kolkata and Chennai, are given in table 11.11.

Table 11.11. Daily Total Solar Radiation on a Horizontal Surface in Calories/cm²/day

Months → / *Cities ↓*	*Jan.*	*Feb.*	*Mar.*	*April*	*May*	*June*	*July*	*Aug.*	*Sept.*	*Oct.*	*Nov.*	*Dec.*
Poona	460	562	598	617	664	589	395	402	496	518	474	447
Delhi	350	447	556	606	630	614	485	452	491	501	413	345
Kolkata	339	394	489	506	542	472	454	481	427	396	393	363
Chennai	472	559	591	595	546	523	423	422	503	369	373	397

11.11.3. Construction and Design Details of a Solar Water Heater. A solar water heater is a coupled unit of :

(1) *a solar energy collector* ; and

(2) *a storage tank* with a suitable device to minimise heat lost.

These units of a solar water heater are discussed below :

(1) The Collector Unit. The main function of the collector unit is to absorb solar radiation and to transfer this heat energy to the cold water for heating the same. It usually is a blackened plate, constructed in such a way that channels are formed, through which water can circulate. Copper in the form of tubes and sheets proves to be the best material for this unit due to its high thermal conductivity, long life, and high corrosion resistant. This metal is however, quite costly in India, and hence G.I. (galvanised iron) sheets are generally used to make this unit. The closed or covered channels are formed by welding inverted corrugated G.I. sheet over a plane G.I. sheet. The side water channels are fabricated from plane G.I. sheets.

Heat loss from the absorber is reduced by providing insulation at the back and sides. *The convection heat losses from the blackened surface of the plate is reduced by covering it with a glass plate which acts as a heat trap.* The insulation is protected with cover boxes, and the whole assembly is made air tight.

Area of the heat absorber depends on the available solar radiation, mean dry bulb temperature, the quantity of hot water, required. In India, owing to large variation in the living standards, no definite figure for daily demand of hot water can be given, but the consumption by a middle class family of 5 persons, is estimated to be about 140 litres/day. For solar heating of 140 litres of water up to a temperature of about 50°C on a winter day, the required collector area is estimated to be 2.1 m^2 for Delhi, 1.8 m^2 for Chennai, 1.5 m^2 for Poona, and 2.8 m^2 for Kolkata. Thus, a unit of 2.1 m^2 area, described here after, will serve for many places in India.

An absorber unit of a solar heater is made of smaller typical units, each of 1.05 m^2 area. Each smaller unit can, thus, heat up 70 litres of water per day. Two or three such typical units can be coupled together to produce larger capacity heater.

A typical absorber unit, shown in Fig. 11.24, consists of a corrugated and a plane G.I. sheet 122 cm × 79 cm of 24 and 20 gauge respectively, which are welded together. Five rivets are fixed, four at equal distance from the sides,

and one at the centre in the depressions in the G.I. sheet. To this unit is welded a top and a bottom water channel made out of 20 gauge G.I. sheet. The water channels are in the shape of rectangular boxes of size 40 mm × 25 mm (ht), as shown. These are closed at one end ; while at the other end, a 10 cm long 25 mm dia G.I. pipe nipples are welded, as shown.

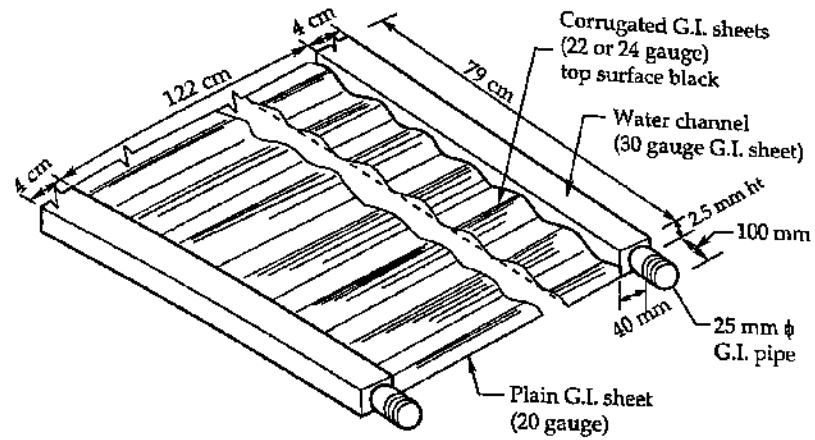


Fig. 11.24. A typical isolated absorber unit of 1.05 m^2 area (122 cm × 79 cm).

A cover box of size 137 cm × 90 cm × 11.5 cm is made of 24 gauge G.I. sheet. A wooden frame of size 137 cm × 90 cm × 4 cm made of 2.5 cm thick deodar wood is fixed inside this cover box, flush with top edges. 5 cm thick glass fibre or mineral wool mattress with one side covered with craft paper is placed inside this cover box, and the absorber unit is finally placed over the wool mattress in the cover box, as shown in Fig. 11.25. The side pipes of the absorber unit are brought out through holes at the side of the cover box. Insulation is pushed around the absorber. 2 to 3 coats of a dull black paint (lamp block dispersed in shellac-spirit solution) are applied over the corrugated surface and nearby sides. 3 mm thick transparent window glass sheet of size 137 cm × 90 cm, with rubber beading all around, is placed over the wooden frame in the cover box. The ends are pressed by folded G.I. strips.

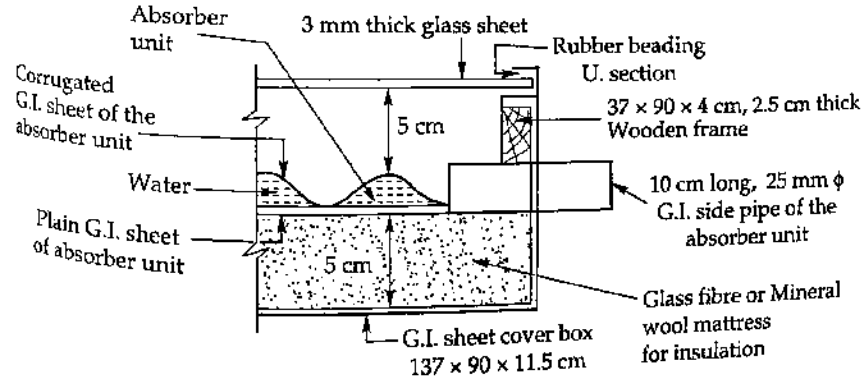


Fig. 11.25. Details of encased collector unit, *i.e.* the absorber unit placed in the cover box etc. complete.

Orientation of the Collector Unit. In order to absorb maximum solar energy all the year round, the collector should be placed with its face towards the equator ; *i.e.* facing towards the south at a place located in the northern hemisphere, and facing towards the north when placed in the southern hemisphere, and is tilted so that its angle with the horizontal is 15° in excess of the latitude of the place. [Please see photo Fig. 11.27].

(2) **The Water Storage Tank with Stand.** Since the hot water may have to be stored over night for use in the next morning, the tank must be well insulated and weather proof. It should have double walls, having a sandwich of at least 10 cm of fibre glass or mineral wool or similar insulation. The hot water from the collector unit is usually allowed to enter the tank at a depth of about 30 cm below the water level in the tank. The cold water to the collector unit comes from the bottom of the tank.

Fig. 11.26 shows a typical 140 litres storage tank for a family of 5 persons. It is a two walled box filled with glass fibre insulation in between the walls. The inner tank is rectangular 40 cm × 40 cm and 91.5 cm high. It is fabricated of a 18 or 16 gauge G.I. sheet. G.I. sheet nipples and sockets are welded at places as shown. The outer tank is made of 24 gauge G.I. sheet with bottom made of 16 or 18 gauge sheet. Its dimensions are 60 cm × 60 cm × 112 cm.

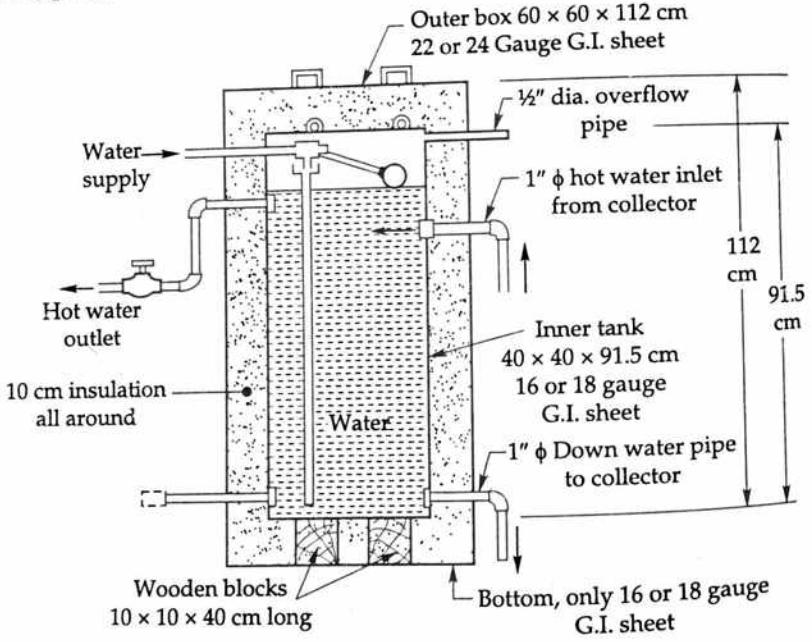


Fig. 11.26. Typical 140 litres water storage tank.

Cold water is allowed to enter the inner tank through a float system. Beneath the float outlet, is fitted a G.I. tray having a 25 mm dia G.I. pipe welded at its centre. The pipe runs to the bottom of the tank. By this arrangement, cold water from the float does not mix with the hot water at the top, but is directed by the G.I. pipe to the bottom of the tank.

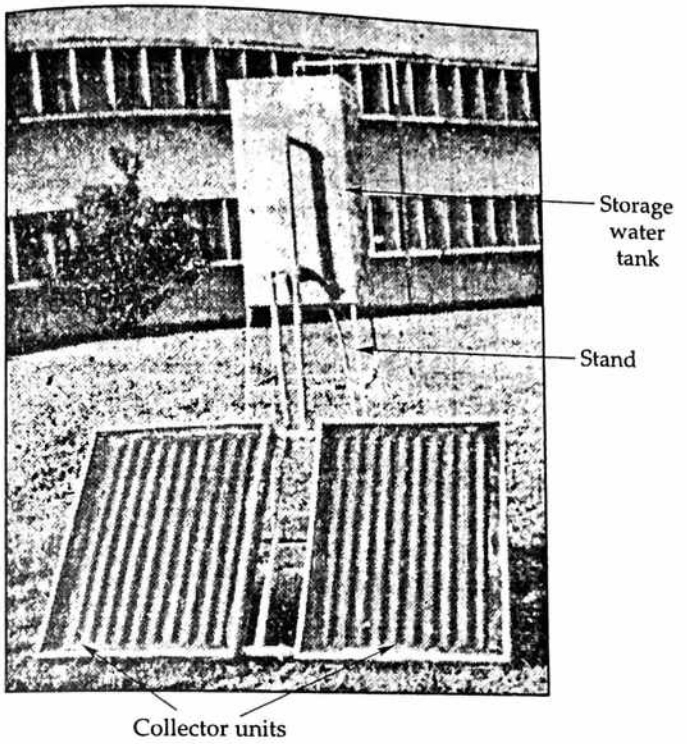


Fig. 11.27. Photoview of an installed solar water heater.

The tank is placed on a stand as shown in the photo Fig. 11.27. The stand is made of 25 mm × 25 mm × 3 mm angle iron frame. At the top, it is 61 cm × 61 cm ; and at the bottom, its size is 91.5 cm × 91.5 cm. Cross ties are provided to reinforce the frame. The height of the stand is 140 cm. The stand can also be made of masonry.

The storage tank is finally connected with the collector unit through G.I. pipes and bend of approximate 120° angle. The connection pipes are insulated with 4 cm thick glass fibre or mineral wool. For weather proofing, alkathene or polyethylene sheets are wrapped over the insulation. The collector is finally oriented as indicated earlier. Cold water connection to the tank is made through a float and hot water is taken out of the tank through a $\frac{1}{2}''$ (12.5 mm) insulated G.I. pipe, as shown in Fig. 11.26.

11.12. Centralised Hot Water Systems

Individual geysers, described in the previous article, prove useful in small installations, such as houses, hostels, etc. However, in large installations like hotels, the demand of hot water is usually large involving—large number of supply points ; and hence installing individual geysers in such buildings may prove to be uneconomical and troublesome, involving heavy consumption of electricity and maintenance problems. In such buildings, therefore, it is usually advantageous to provide a central hot water system, in which the water is heated at one particular place in a boiler (of course water is heated

and not boiled), and then circulated in the entire building through a properly designed network of hot water piping system, as to make the hot water reach at each distribution point. The boiler to be used in such a system can use either electricity or any other fossil fuel like coal, wood, kerosene, diesel, furnace oil, natural gas, L.P.G. (liquefied petroleum gas), gobar gas, solar energy or a combination of these fuels. The various factors which are considered in the design of such a system are discussed below :

11.12.1. Ambient Temperatures. The temperature of the hot water to be supplied is usually kept between 55°C to 80°C, depending upon the required uses and climatic conditions of the place. The heat required in the boiler to achieve this desired temperature will of course depend upon the temperature of the incoming cold water, which may usually vary between 10 to 25°C, although of course it may be lower in sub-zero ambient conditions. The furnace or boiler requirements should be designed for the lowest temperature conditions, if the same is sustained over long periods, one weak or more, Lowest ambient temperature to be considered in design should be established after a study of the area's temperature conditions. Air temperatures determine the viscosity of liquid fuels (furnace oil and light diesel oils), which may need pre-heating in very cold climates.

11.12.2. Water Pressure in the System. Pressure in hot water taps should be the same as that in the cold water taps. Unequal pressure may result in back flow of water from one system to another, when hot and cold mixers are used and may cause scaling.

11.12.3. Hot Water Storage and Generation. If a boiler is able to generate hot water at the same rate as the peak demand in the system, then evidently no storage of hot water is required. However, by providing a storage tank for hot water, the boiler capacity can be reduced. Large storage tanks for hot water are, however, usually avoided due to availability of limited space. But some storage may become necessary in certain specific cases, to take care of emergent requirements. The system to be adopted will finally depend on the capital and operational costs and the availability of space.

11.12.4. Hot Water Piping Systems. The hot water generated in the central boiler should be supplied to the various supply points efficiently, without excessive loss of temperature and pressure. If the length of the supply pipes from the boiler to the supply point is long, a lot of heat from the water will dissipate, even if the pipes are insulated.

Secondly, if there is no **recirculation of hot water** in the distribution system, then hot water from a tap will come out only after initial delivery of cold water for 1 to 5 minutes (since the stagnant water in the pipe length from boiler to the off take point will loose heat by convection and become cold). If the rate of flow at the tap is about 8—10 l/min, then the user will be wasting about 8 to 50 l of water before getting the hot water. In such a non-circulating system, the energy used in heating the water initially, also gets wasted. *To avoid this wastage of heat, a return pipe is installed from the remotest section of the hot water main, which is connected back to the vessel supplying hot water to the building, as shown in Fig. 11.28. In case, there are a number of risers, each one is provided with a return line, connected to a common return header to*

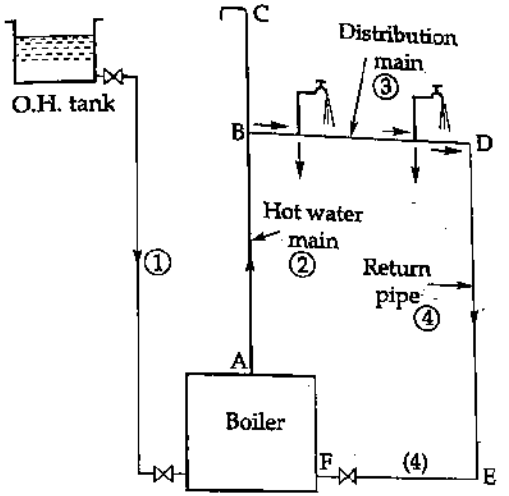


Fig. 11.28. Typical hot water distribution network supporting recirculation by thermo-syphonic action.

the boiler. The recirculation is automatically achieved due to *thermo-syphonic action* (Fig. 11.28) ; or is sometimes forced by using a *recirculation pump* (Fig 11.29), as explained below :

11.12.4.1. Piping System using Natural Thermo-syphonic action. A simple typical installation of hot water system, using an overhead cold storage tank, a centralised boiler at ground level, one hot water main (2) rising vertically up from the boiler, one distribution main (3) is shown in Fig. 11.28. The remotest point *D* of the distribution main is connected back to the boiler by a return pipe *DEF* (4).

In such an installation, the circulation of hot water from the main pipe connected to the boiler and the return line can occur without the aid of pump by the natural thermo-syphonic action, which occurs due to the difference in the density of water at different temperatures ; density being more at lower temperatures, as shown in Appendix table A-4. **Thermo-syphonic circulation** will take place only when the boiler is placed below the point of supply (3) so that the hottest water from the boiler rises in the main line (2) to the top of the system. Hot water in the rest of the system will have a lower temperature due to loss of heat in the system, and eventually will be heavier (more density), tending to settle at the bottom of the system through the return pipe. This water will, thus, move towards the boiler, from where it again rises after getting re-heated. *Convective Current is thus generated in the system, which is known as thermo-syphonic action.* The over-head tank maintains pressure in the system through a cold feed line (1). The hot water pipe is extended up, as an expansion or vent pipe above the over-head tank.

The circulating pressure developed in the system is calculated by the equation :

$$H = \frac{h}{\rho_w}(\rho_{w_2} - \rho_{w_1}) \quad \ldots(11.3)$$

where H = Circulating pressure in metre head of water

h = mean height in m between the boiler and the supply point

ρ_{w_1} = density of hot water at flow temperature t_1 in pipe AB

ρ_{w_2} = density of less hot water at return temperature in pipe DEF

ρ_w = density of water at 0°C = 998.8 kg/m³

For example, let us assume that :

h = 12 m

t_1 = flow temperature at boiler = 60°C

t_2 = return temperature = 35°C

$\therefore$ ρ_{w_1} at 70°C = 978 kg/m³

ρ_{w_2} at 45°C = 990 kg/m³ [See Appendix Table A-4]

$\therefore$ H = Circulating pressure in the system

$$= 12 \text{ m} \times \frac{990 - 978}{998.8}$$

$$= 0.14 \text{ m of water head}$$

$$\approx 1.4 \text{ kN/m}^2.$$

The pipe diameter is calculated from the pressure loss formula after calculating the probable flow and length of the line to the remotest point. The size of the return pipe is calculated by estimating the heat loss in the line from a designed temperature drop.

Circulation by natural thermo-syphonic action usually succeeds only in smaller installations, where the boiler is installed in the basement below the lowest supply-line level, and the usage of hot water is uniform. When the required flows are larger, the limited circulating pressure makes the pipe sizes very large and uneconomical.

11.12.4.2. Piping System with Forced Circulation. Since the natural thermo-syphonic circulating pressure is usually low, it becomes necessary in larger installations to generate additional pressure within the system to force the circulation by means of a pump. This helps in using reduced pipe sizes and allows hot water to circulate in all parts of the distribution system for quick draw off of the hot water.

The circulating pumps should however, never be used as booster pumps for increasing the pressure in the hot water lines, which infact is provided by the cold water over-head tank. When the source of cold water as well as hot water remains the same, the pressure in the cold water taps as well as in the hot water taps will remain almost the same, which is very much desirable.

11.12.5. Piping Layouts. The hot water piping network can be arranged in different ways depending upon the architectural and structural considerations of a building. The most common systems are :

(i) Downfeed system ;

(ii) Upfeed system ; and

(iii) Reverse circulation system.

These systems are discussed below :

11.12.5.1. Downfeed System. In this system (Fig. 11.29), the main vertical hot water line AB (pipe marked as 1) is lifted up to the top floor or to terrace level, where the distribution main BC [*i.e.* pipe (2)] will run over the floor. Single downtake pipes (3) are dropped in each shaft to join a common return

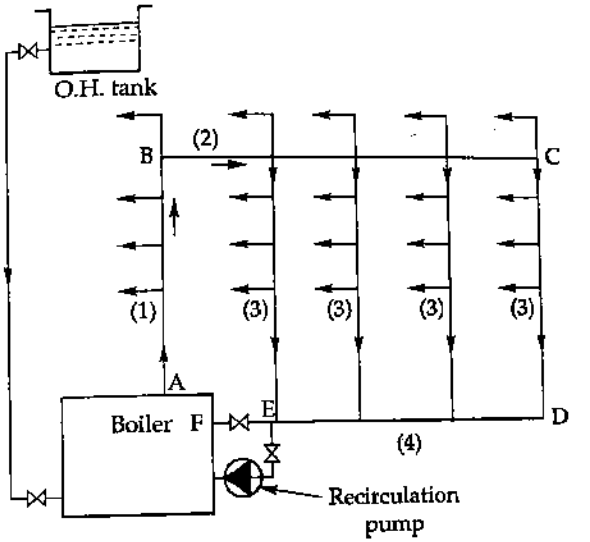


Fig. 11.29. Downfeed piping system.

line DE (4) at the lowest level, which is connected to the boiler. A recirculation pump may be attached as shown, to improve circulation pressure, as discussed earlier.

For efficient operation, the flow pipe on the top floor must run horizontally at the same level. Any change in its direction in vertical plane should be avoided to prevent air-traps. Accessible space is therefore necessary to lay this main pipe (2) on the top floor. When such space on the top floor is not available, this pipe may be laid on the terrace.

This system is simple, efficient and economical.

11.12.5.2. Upfeed System. In this system (Fig. 11.30), the main flow line (1) and the return pipe (4) are both laid at the ceiling of the lowest floor like basement roof or on ground floor. Vertical upflow pipes (2) and their corresponding return pipes (3) supply the water to the rooms above. Circulation may occur either under natural thermo-syphonic action or by forced circulation, as discussed earlier.

11.12.5.3. Reverse Circulation System. This system (Fig. 11.31) is similar to the upfeed system except that the return flow occurs in the same direction as that of the main flow pipe (1) till the end of the remotest circuit, from where it returns to the boiler through a separate reverse

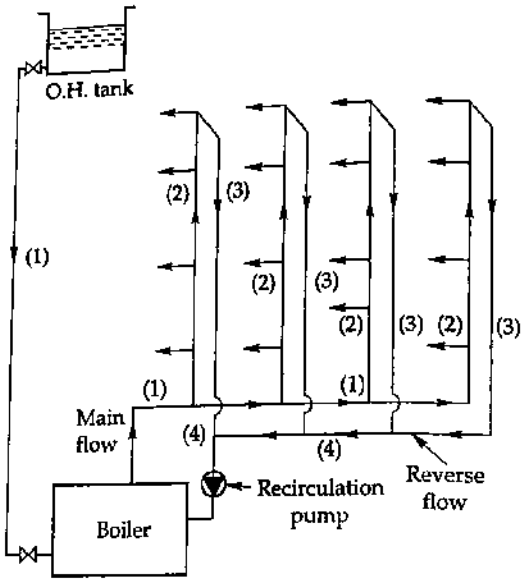


Fig. 11.30. Upfeed piping system.

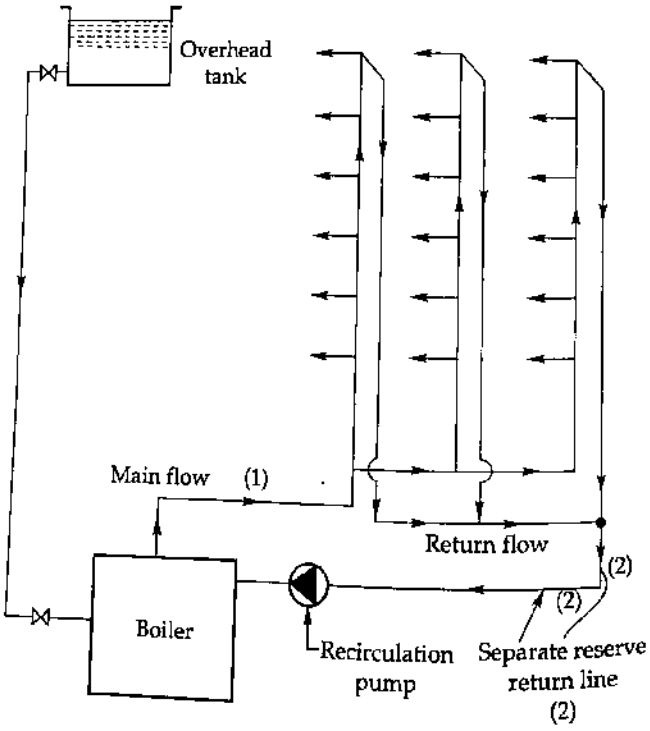


Fig. 11.31. Reverse circulation system.

return line (2). This system is suitable where the circuits are long and where there is a possibility of the least favourably placed fixtures not getting adequate flow or pressure. A reverse return line enables closer balancing of pressure in the system.

PROBLEMS

1. (*a*) Explain with a neat sketch as to how municipal water mains are connected to private buildings and houses for giving water supply connections.
   (*b*) What factors will you keep in mind while designing plumbing system for water supply to a house.
2. Discuss the water storage requirement of a house or a residential building, and how is it accomplished ? In this connection, discuss the underground and over head storage tanks indicating the need of underground tanks, and indicating the types of overhead tanks.
3. Discuss the various types of water piping systems that may be employed in buildings for fulfilling the water demands of its residents, giving merits and demerits of each system.
4. (*a*) How would you assess the water demand of the buildings to be used for different purposes.
   (*b*) Discuss the 'flow rates' and 'fixture units' in relation to computation of gross water demand of a building. How is probable simultaneous demand computed ? What is probable demand ?
5. Briefly discuss the design principles involved in the design of a water supply network to be laid in a multi-storeyed residential building.
6. (*a*) Enumerate and discuss the various types of domestic appliances that are available for obtaining hot water supply in houses. In this context, explain and differentiate between the pressure type and non-pressure type of electric storage water geysers with neat sketches.
   (*b*) How is the hot water requirement of a building computed ?
7. Discuss in details the working of a pressure type of a storage water heater, giving its benefits over the non-pressure type. Also discuss the safety requirements of such an appliance, explaining the 'explosion' and 'negative pressure', against which such safety measures are provided.
8. Explain as to what do you understand by :
   (*a*) Pressure release valve ; and
   (*b*) Check valve.
   Also discuss their uses in a domestic water geyser, being used in modern bathrooms for giving hot and cold (mixed) supplies. In this context, also discuss the use of an expansion pipe, and explain as to how it can be avoided.
9. Discuss the use of a centralised boiler system for providing centralised hot water supply in a hotel building, clearly bringing out the various piping systems that can be provided to ensure assured hot water and cold water supply in all the bathrooms of the building.
10. What do you understand by 'recirculation of hot water'. How is it achieved in centralised hot water systems ?
11. Explain the use of thermo-syphonic action in a hot water piping layout, indicating its limitation which necessitates the use of a forced system.
12. Differentiate with neat sketches the 'downfeed' and 'upfeed' piping systems in centralised hot water layouts. Also indicate the layout of a reverse circulation system in this context.

13. Write short notes on :
   (*i*) Storage geysers for individual bath rooms
   (*ii*) Centralised hot water systems for hotel buildings
   (*iii*) Non-pressure storage geysers *vs* Pressure type of storage geysers
   (*iv*) Instantaneous geysers
   (*v*) Thermo-syphonic circulation of hot water in a building
   (*vi*) Use of expansion pipes, check valves and pressure release valves in modern hot water heaters (geysers)
   (*vii*) Geyser implosions and collapse of geysers
   (*viii*) Solar water heaters.

—12

Water Supplies of Small Communities in Rural Areas

12.1. Introduction

More than 80% of the Indian population live in rural areas but only about 60% of them have some form of potable water supply, with only about 20% having piped water supply.

It has not been feasible to cover all our villages with potable supplies because of various constraints, such as :

(*i*) scattered and inaccessible nature of villages ;

(*ii*) non-availability of nearby water sources ;

(*iii*) non-availability of adequate funds to take up those projects which may serve only a few people, and hence may not become financially viable.

In view of these constraints, it becomes very difficult to supply piped potable water to the villages through a full fledged conventional water-treatment plant ; and hence, special simple methods are devised and adopted to treat the available water supplies in rural areas.

12.2. Sources of Water for Small Communities in Rural Areas

Since most of the Indian rivers are non-perennial with meagre storages, and generally located at far off distances from the rural villages, it has generally not been feasible to supply surface waters to the scattered and isolated rural areas except in certain cases. Most of our villages, therefore, do depend upon ground water supplies, to be extracted through *wells* and *tubewells*. In hilly regions, sometimes, natural *springs* also become available in exceptional cases, which avoid lifting of ground water. *Infiltration galleries* have also been constructed in a few cases to supply water.

The degree and type of treatment to be given will evidently depend upon the quality of the water source available. The characteristic features of the various water sources are therefore, summarised here :

12.2.1. Rivers and Streams. In tropical countries like India, the waters of rivers and streams are almost invariably soft, containing relatively low concentrations of dissolved salts, although this does not necessarily apply elsewhere. They are, however, often affected by fecal pollution, usually highest in small streams near settlements. The quality of the water can vary considerably with rainfall, but turbidity can be expected at most times. Slow flowing rivers can contain considerable amounts of organic colouring matter.

12.2.2. Lakes. Large lakes form obvious ample reservoirs of water. In general, the water quality is fairly good and consistent but may well be polluted, and in

particular contain *bilharzia contamination* near the shores. Near the areas of industrial and agricultural development, and sewage and industrial effluent discharges, however, there is an increasing danger of *algal growth forming*, which may affect the water to such an extent that it becomes impracticable to consider it as a source for public water supply.

12.2.3. Swamp Waters. Swamp waters, which can include very slow flowing rivers, generally contain *fecal pollution* and other *organic matter* in comparatively high concentrations, giving rise to an *unacceptable colour*. These waters are usually *acidic*.

12.2.4. Springs and Boreholes. Springs are natural sources of ground water, while the boreholes are dug or drilled to lift ground water. These sources can provide reliable water supply. Moreover, if properly maintained and kept sufficiently away from pit privies and soakpits, they can provide us water almost free from fecal pollution.

Proper protection of the site of spring from defecation and drainage is, however, very important to avoid its contamination, as shown in Fig. 12.1. Quite often, a source like this can be utilised to provide a supply to a community without pumping, at minimum cost and maintenance (Fig. 12.1).

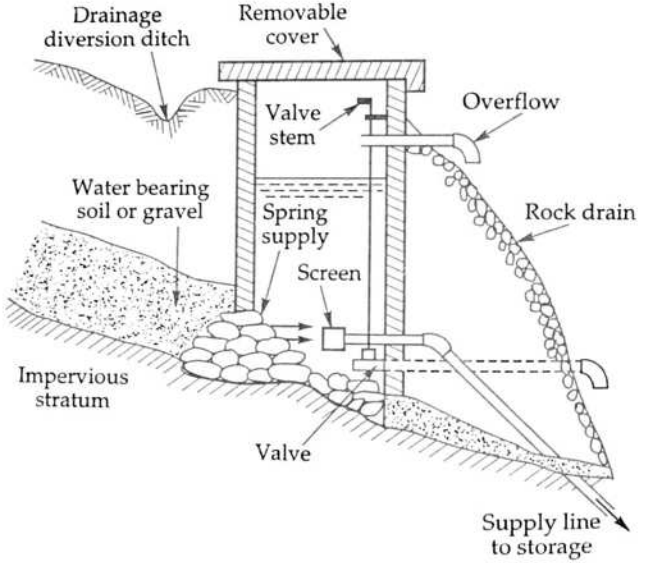


Fig. 12.1. Protection of spring supply to prevent pollution from surface runoff.

A well or a bore hole must be dug or drilled into the permeable strata, and the water lifted manually or by mechanical pumps. Again it is essential that careful control is kept on the land adjacent to the well head. It is usually advisable to prevent surface water entering the well by sealing the top section with brick or concrete, as shown in Figs. 12.2 and 12.3 for dug and driven wells, respectively.

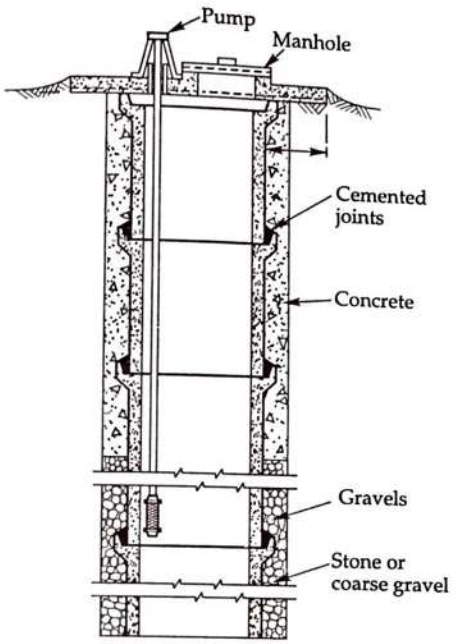


Fig. 12.2. Protection of a dug well to obtain uncontaminated community water supply in rural areas.

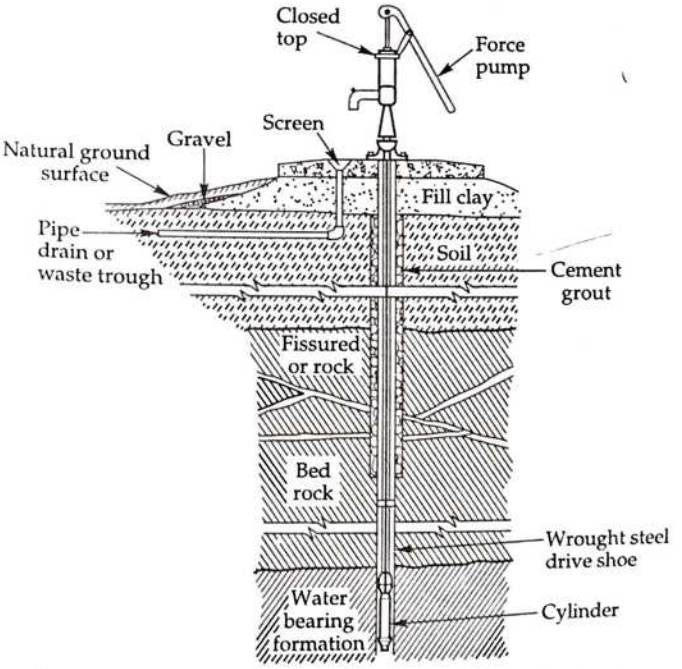


Fig. 12.3. Protection of a shallow driven tubewell provided with a hand pump for obtaining small individual supplies.

In order to obtain larger centralised discharges for distribution among the villagers through piped supply after treatment, deep tubewells may be drilled. Water from such tubewells shall be lifted by means of pumps, which may be run on electrical, diesel or wind power. The chances of contamination of such deep supplies are very meagre, although however, they may contain large scale minerals and dissolved salts.

Electrical and *diesel pumps* are more often used to lift tubewell supplies and have already been discussed in details in chapter 4 on Ground Water Development. *Wind* has also proved to be an economical source of energy for lifting ground water from wells and tubewells, in areas where wind velocities of more than 15 metres per second are available for more than 75% of the time. **Wind mills** (Fig. 12.4) are installed for this purpose on towers to rise atleast 5 m above the surrounding obstacles to provide a clear sweep of wind to the mill. Windmills usually drive reciprocating pumps through a connection of the pump rod from the mill to the piston rod of the pump. Provision is also made for pumping by hand during long periods of relative calm (absence of winds). It is a good practice to provide adequate elevated water storage to maintain the water supply during periods when there is insufficient wind. Wind mills are normally manufactured in sizes expressed in terms of diameter of their wheels. To obtain good results, a windmill should be lightly loaded, as it would work faster on a lighter load. The dia of wind wheel may usually vary between 2 to 3 m. The estimated pumped discharges and involved heads at maximum efficiency with different dia wind mills are indicated in table 12.1.

Fig. 12.4. Wind mill for water pumping.

Table 12.1. Estimated Discharges and Heads at Maximum Efficiency from a Windmill of a given Diameter

Dia of windmill in m	*Lift head and discharge*	*Internal dia of pump in mm*						
		50	*60*	*65*	*70*	*75*	*90*	*100*
2.0	(*i*) Total head (*H*) in m	20	15	12	10	9		
	(*ii*) Average *Q* in m^3/d	5.0	7.4	8.8	10.0	12.5		
2.5	(*i*) Total head (*H*) in m	33.75	28.5	22.5	19.5	16.5	13.0	11.0
	(*ii*) Average *Q* in m^3/d	4.0	7.0	8.5	9.7	12.7	14.5	17.8
3.0	(*i*) Total head (*H*) in m	64	50	36.5	31	25	17	14
	(*ii*) Average *Q* in m^3/d	4.0	5.5	7.2	8.2	10.0	14.7	17.5

Animal power has also been used traditionally in our villages to lift water from open wells through the use of a **mote*** and a **persian wheel***, which are respectively shown in Figs. 12.5 and 12.6.

The simple *manual arrangement*, which is often used in villages to lift water from open wells, called Denkli, is also shown in Fig. 12.7*.

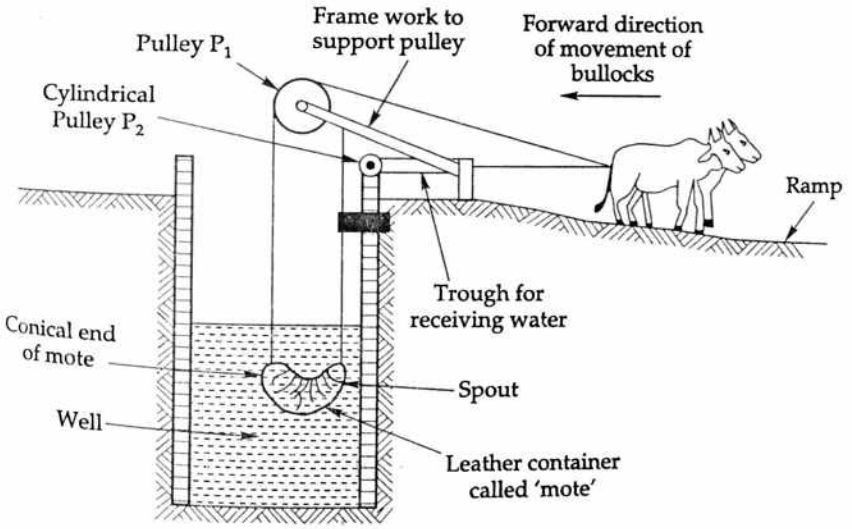


Fig. 12.5. Mote.

*Detailed description is available in *Elementary Irrigation Engg.* by the same author.

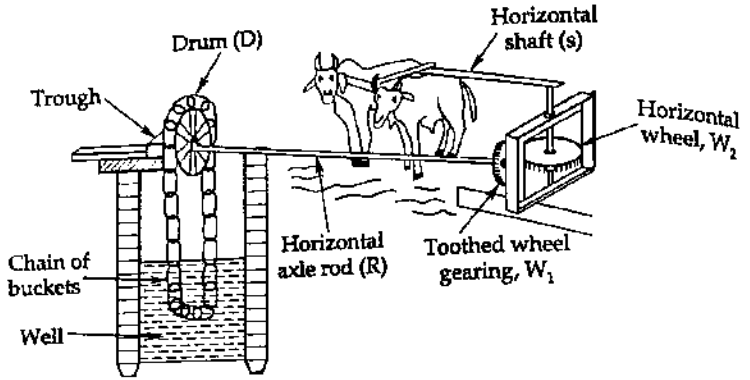


Fig. 12.6. Persian wheel.

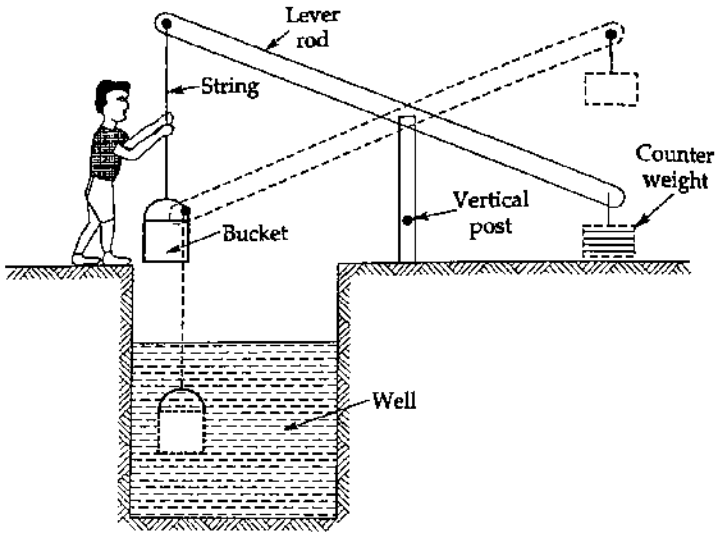
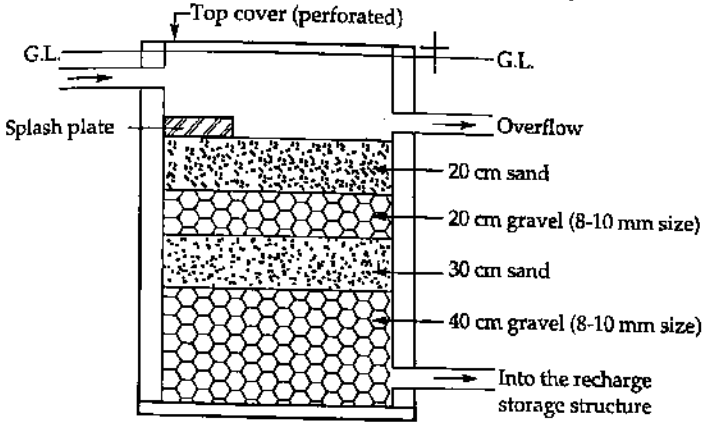


Fig. 12.7. Denkli.

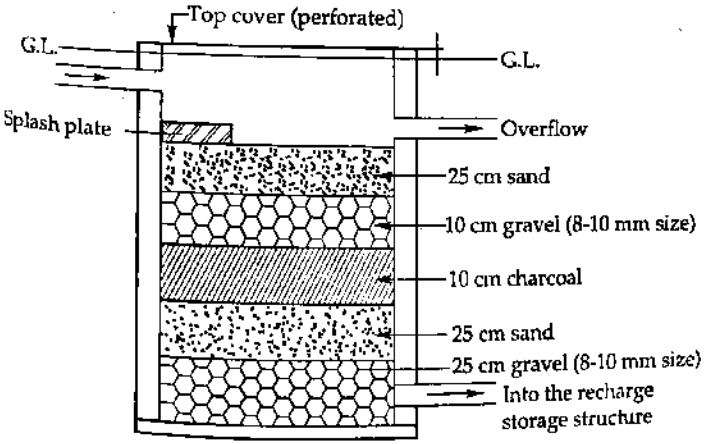
12.2.5. Rain Water Harvesting or Roof Top Water Harvesting. Collection of rain water from paved or G.I. corrugated roofs and paved court yards of houses either in storage tanks or in the ground water reservoir is known as *rain water harvesting*. This collected water serves as a good source of water in rural and water scarce areas. This practice has been adopted since olden times, particularly in rural areas in places having high rainfall intensity, well distributed in the year. Such areas in India include : Himalayan areas, North eastern states, Andaman and Nicobar islands, Lakshadweep islands, Rajasthan, and Southern parts of Kerala and Tamil Nadu. *This technique is highly promising even for urban areas and places where the rainfall occurs only for a few months in a year, and where other sources of water are scarce and ground water levels have gone down, such as in Gujarat, Haryana, Delhi, Madhya Pradesh, etc.* So much so that roof top water harvesting in society buildings and other large sized complexes have already

been made compulsory in Delhi and Tamil Nadu, and this movement is gathering momentum. This practice is already quite prevalent in water scarce Gujarat, where even the old house of Mahatma Gandhi at Porbander can be seen to contain an arrangement for collection of rain water in a storage tank at ground level for its direct use.

In its simplest form, roof top water harvesting involves taking down a PVC or M.S. pipe of 90-120 mm dia from the roof's outlet to the ground floor, which can be connected to a water tank (placed either above the ground level or below the ground level) or to the underground water-table. The rain water before collection should however generally be passed through simple sand or charcoal filters [Fig. 12.8 (*a*) and (*b*)] for the removal of suspended particles and micro-organisms from the rain run-off being collected. The roofs or court yards should also be kept as clean as possible at the time of rains. These filters can be vertical or horizontal and can be easily constructed. Their study interface with the water, however, needs to be periodically cleaned to prevent clogging of its pores.



(*a*) Sand Filter



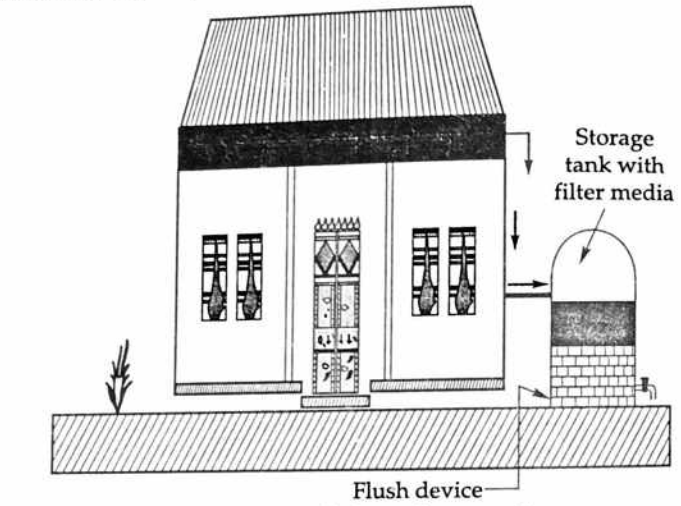
(*b*) Charcoal filter

Fig. 12.8. Filters to be used for filtering rain water.

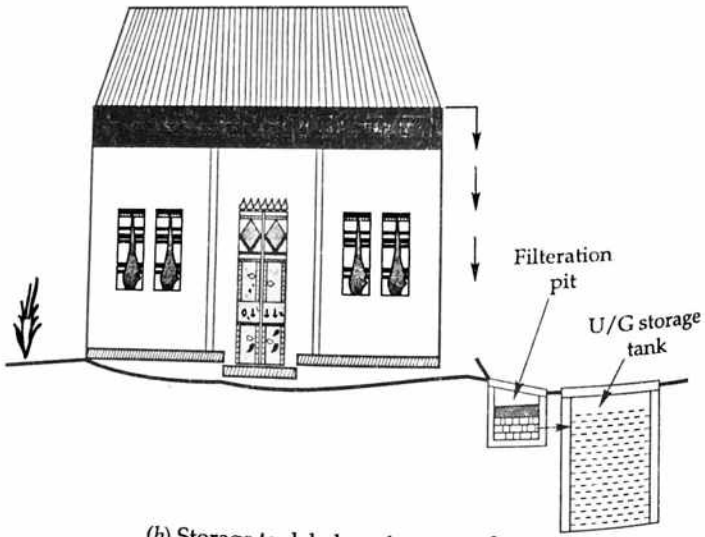
Various rain water collection techniques employed for collection of water or for recharging the ground water are discussed below :

12.2.5.1. Rain water collection for direct use. The rain water collection for direct use can be practised by collecting the water coming down from the roof into a storage tank of plastic, R.C.C. or masonry. The tank can be placed either above the ground (Fig. 12.9 *a*) or below the ground (Fig. 12.9 *b*) depending on the availability of space and cost considerations. Cleaning and disinfecting the tank at regular intervals and raking of filters is a must.

In a campus, where sufficient space is available, the water can be stored in an open excavated pond, which also acts as a settling pond for impurities.



(*a*) Storage tank above the ground



(*b*) Storage tank below the ground

Fig. 12.9. Collection of rain water for direct use in a storage tank.

12.2.5.2. Rain water collection for ground water recharge. When rain water collection for direct use is difficult or costly or impracticable, then ground water recharge option can be easily practised to supplement the falling ground water-table of the area. The ground water recharging can be practised by directing the rain water to infiltrate into the ground to join the water-table either through a *recharge pit* (Fig. 12.10) ; or through a *recharge well* (Fig. 12.11) ; or through a *recharge well with a pressure filter* (Fig. 12.12), depending upon the available circumstances, as discussed below :

(*i*) ***Recharge through a recharge pit.*** This method (Fig. 12.10) is suitable in areas having shallow ground water-table/aquifer and for smaller buildings with roof area of 100-150 sqm. A pit of 1-2 m deep is excavated at a suitable location in the plot, and the run-off water from the catchment area (roof, etc.) is diverted into this recharge pit. The pit is filled with crushed gravel and coarse sand to filter out the rain water, before it infiltrates through the aquifer to join the water-table, as shown in Fig. 12.10.

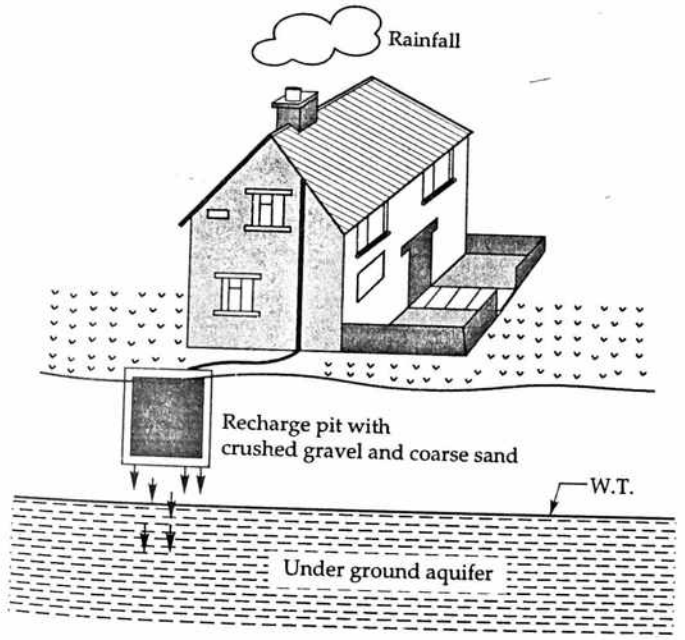


Fig. 12.10. Percolation of rain runoff to the underground reservoir through a recharge pit.

(*ii*) ***Recharge through a recharge well.*** This method (Fig. 12.11) is suitable in areas having low ground water-tables. In this method, a bore hole is dug or drilled upto or near to the ground water level, or upto the porous strata or rock fracture for effective recharge.

The diameter of the bore hole may vary from 0.2 to 0.3 m depending upon the porosity of the receiving strata. Slotted pipes are then installed into this bore hole to enable permeability in the ground water-table. The slots should be located to coincide with favourable pervious strata. Such a **recharge well** should preferably be constructed 10-15 m away from the built area.

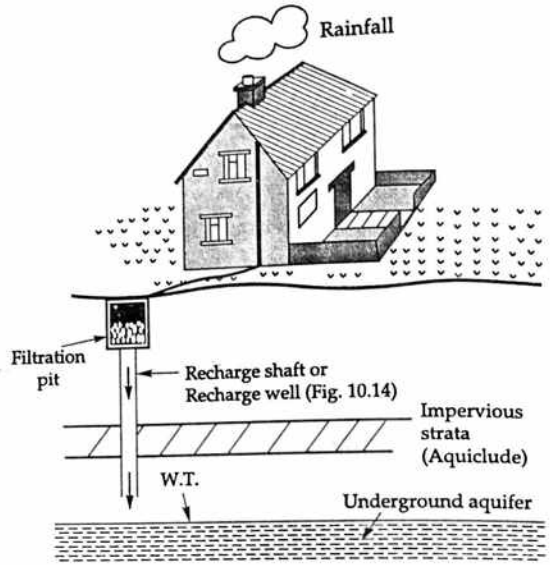


Fig. 12.11. Percolation of rain runoff to the underground reservoir through a recharge well.

This is the most common type of roof top rain harvesting which can be easily adopted in almost all buildings. A perspective view of the roof-top rain water harvesting through recharge wells in a building complex is shown in Fig. 12.12.

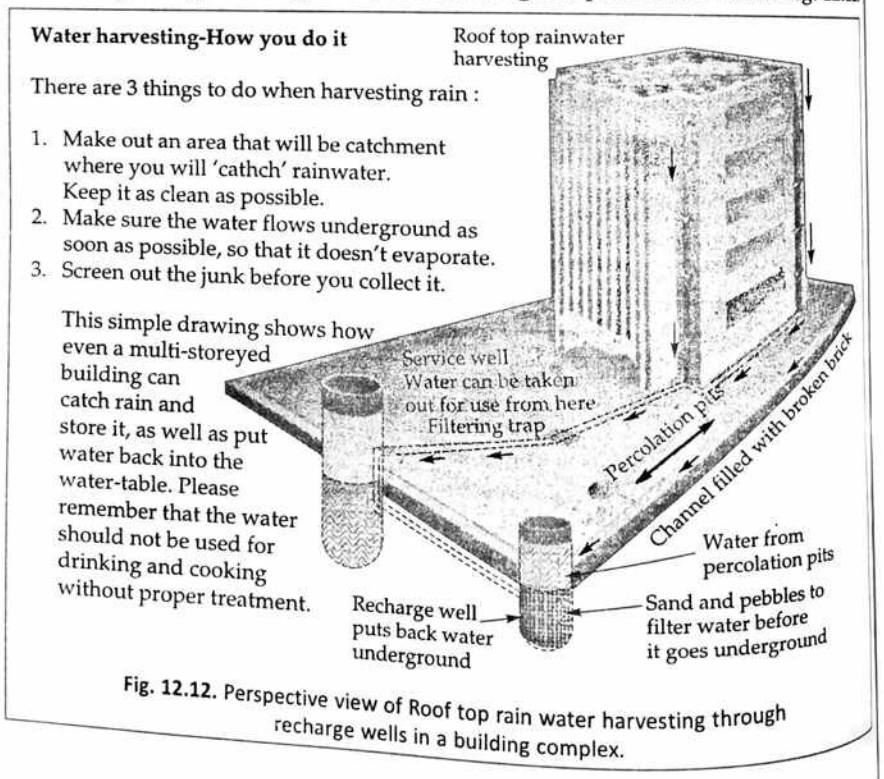


Fig. 12.12. Perspective view of Roof top rain water harvesting through recharge wells in a building complex.

(iii) ***Recharge through a recharge well with pressure filter.*** This method is suitable for areas with low ground water-tables, when ground profiles involve large depths of impervious strata, making ground water infiltration difficult under simple force of gravity, at the rate of rainfall.

In this method, the rain water is first of all collected into an underground tank, from where it is lifted up by a pump (through a filter) and pressured down into a recharge well, as shown in Fig. 12.13. The shaft or recharge well is drilled upto the ground water level or just above the normal tubewell withdrawal level, and may be made by lowering G.I. tubewell type pipe into a bore hole of 0.15 to 0.3 m dia. (depending upon the porosity of the receiving strata). The pipe to be lowered in the bore must be slotted in the lower part of the shaft to enable permeability in the ground water-table. The slots in the upper length may also be located to coincide with favourable permeable strata, if any. The details of a recharge well are shown in Fig. 12.14.

The settlement tank of size varying from 1.5 to 5 cu. m is usually constructed for collecting the rain run off. A dual media pressure filter is utilised to filter the collected water and to pump it into the ground.

Note. Such a dual media pressure filter is a sophisticated device and hence recourse can be made to use a simple sand filter tank before collection of water into the storage tank.

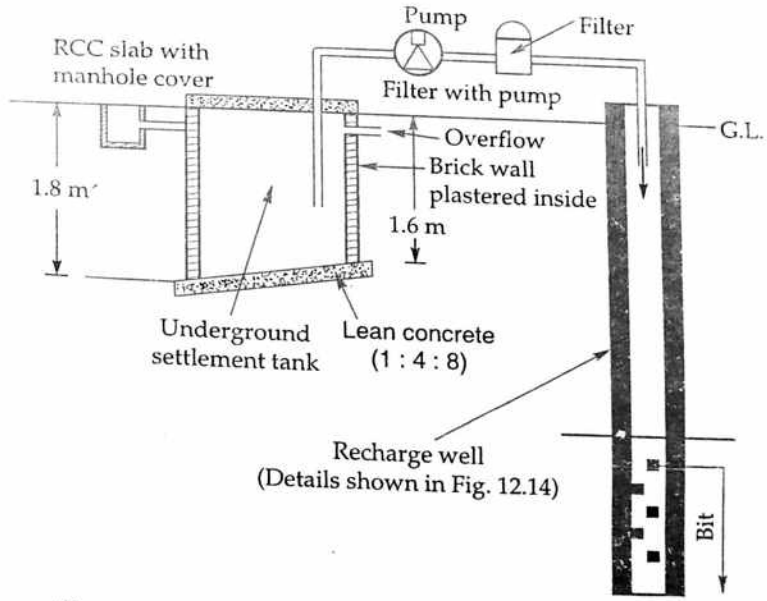


Fig. 12.13. Percolation of rain runoff to the underground reservoir through a recharge well with pressure filter.

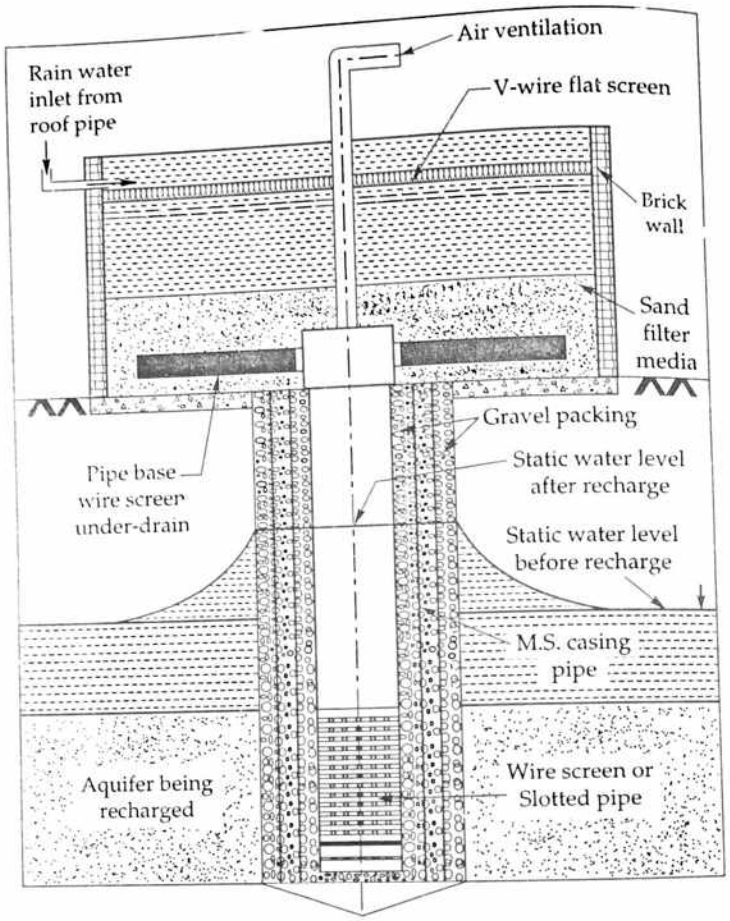


Fig. 12.14. Section of a recharge well to be adopted for percolation arrangements shown in Fig. 12.11 and 12.13.

12.2.5.3. Annual rainwater harvesting potential. The annual rainwater harvesting potential (ARHP) can be easily computed for a given area by using the simple equation

$$ARHP = C.I.A.$$

where ARHP = Annual rainwater harvesting potential in cu. m.

C = Run off coefficient [% age]

A = Area in sq. m.

I = Annual rainfall in m

The value of C (run-off coefficient) may be taken as below :

(*i*) *For roofs*

Tiles	:	0.8 to 0.9
Corrugated	:	0.7 to 0.9

(*ii*) *For ground surfaces*

Concrete/Paved	:	0.6 to 0.8
Plastic sheeting or Gravel covered	:	0.7 to 0.8
Butyl rubber	:	0.8 to 0.9
Brick pavement	:	0.5 to 0.6

(*iii*) *For treated ground surfaces*

Compacted and smoothened soil	:	0.3 to 0.5
Clay/cow-dung threshing floors	:	0.5 to 0.6
Silicone treated soil	:	0.5 to 0.8

(*iv*) *For untreated ground catchment*

Soil on slope less than 10%	:	0.0 to 0.3
Rocky	:	0.2 to 0.5

Note. The above average values of C may be taken for the sake of simplicity. As a matter of fact, an intense rainfall, generates more run-off than a light but prolonged rain. Also, the frequency, slope, surface hardness and lack of vegetation cover of the catchment contributes to the increased run-off. Deliberate increase in slope and hardness of the catchment area will yield greater run-off.

Example. *Calculate annual rain water harvesting potential from a house having the following data :*

Total catchment area	= 500 sq. m.
Roof area	= 140 sq. m.
Paved area	= 160 sq. m.
Kutchcha area	= 200 sq. m.
Average annual rainfall	= 60 cm.

Using : $ARH = C.I.A. = (C_1A_1 + C_2A_2 + C_3A_3)I$

$= [0.8 \times 140 + 0.7 \times 160 + 0.1 \times 200]\,0.6$

$= 146.4\ m^3 = 1{,}46{,}000$ litres. Ans.

[**Note.** An average monthly water need of a family of 5 person is about 10,000 litres ; and hence the above storage can suffice for 14.6 months. In other words the annual requirement of the family can be fulfilled by the generated rain water yield from their house itself.]

12.3. Selection of the Suitable Source of Water

Where more than one possible source is available, the choice should be made in order of the following priorities :

(*i*) the *reliability* of the source ;

(*ii*) the *purity* of the water available in the source ; and

(*iii*) the *ease* with which the water from the source can be supplied to the consumers.

If large quantity of supply is planned from an unexploited source, the services of on analytical laboratory should be employed to assess the quality of the available water as well as to obtain advise on necessary treatment. Very small communities, however, may not be able to call upon these services, but the assessment of purity should not be left entirely to personal judgement.

Samples of water should, therefore, invariably be collected for testing. Since the natural bodies of water of most kinds generally do not remain uniform and may vary from season to reason, it becomes necessary to collect as much representative samples as possible. Multiple samples taken over a period of time will provide much more information than a single sample. Samples should also be analysed soon after collection as they are liable to undergo certain biological and chemical changes if kept for long periods at normal temperatures.

Certain quality checks on the water source can now a days be made easily with the help of some mobile *analytical water testing kits*, which are easily available these days. Central Water and Air Pollution Control Board has devised one such kit. Imported kits are available in different varieties from E-Merck India Ltd, Shiv Sagar Estate, 'A', Dr. Annie Besant Road, Worli, Bombay 400018. The three standard kits of E-Merck Co., being sold under the brand names *Merckoquant, Aquamerck* and *Aquaquant* can be used for testing various parameters, as shown in **colour photo Figs. 12.1, 12.2, and 12.3** respectively.

Merckoquant analytical strips can be easily used to test the presence of metals like arsenic, cobalt, lead, chromate, peroxide, potassium, and total hardness. *Aquamerck* kits help us to measure acidity, alkalinity, ammonium, carbonate hardness, chlorides, chlorine and pH, magnesium, nitrates, nitrites, total hardness, phosphates, sulphites, etc. *Aquaquant* test kit helps us to measure small concentrations of parameters like aluminium, calcium, chromium, copper, colour cyanide, manganese, sulphate zinc, etc.

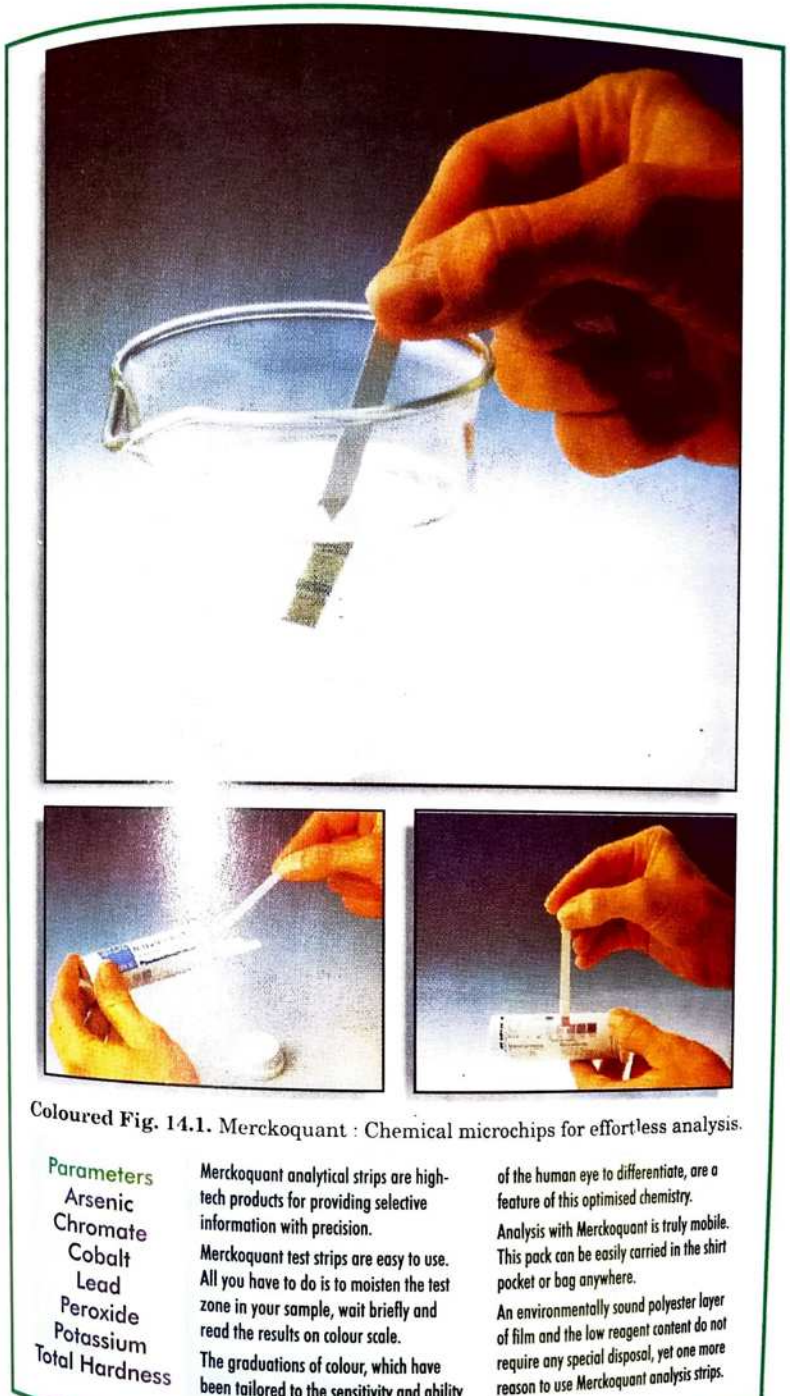
The parameter to be tested for a given water source will certainly depend upon the type of the water source. Say for example in the case of surface sources and shallow wells, fecal pollution may be widely present, thus necessitating testing of parameters like *Chlorine demand* or *BOD*, etc. Testing the presence of *Ammonia* by using Nesslers solution can help in detecting recent fecal pollution. In case of deep bore waters, however, parameters like *hardness*, *pH value*, dissolved metals like lead, *iron, fluorides, arsenic, nitrates*, etc. will become more important. The permissible and absolute maximum concentrations of these parameters, as per IS code, are given in Table 8.10, and should be adhered to, while selecting the water source to be used for the water supply of the village.

12.4. Quantitative Requirement of Water in a Rural Water Supply Schemes

The hand book on Water Supply and Drainage, Bureau of Indian Standards, New Delhi has laid down the following minimum per capita domestic consumption for rural communities with population upto 20,000:

(*i*) where piped supply with service connections is proposed	Not less than 70—100 l/c/d
(*ii*) where supply is proposed through hand pumps or central stand posts	Not less than 40 l/c/d

The estimated water requirement and other factors relevant to village water supply can also be computed by using more detailed information on number of taps, etc. as given below :



Coloured Fig. 14.1. Merckoquant : Chemical microchips for effortless analysis.

(1) The capacity of each water tap is 10 l/min ; and hence the total quantity in litres can be worked out by multiplying the No. of taps with minutes of supply and rate factor of 10 l/min.

(2) If houses are not provided with taps and only street taps are provided, then the general average per capita demand may be taken as 18 l/person/day. If however, the houses are provided with taps, then the average per capita demand of 45 l/c/d may be considered.

(3) If the entire house is piped and provided with flush system, than the per capita water demand may be taken as 225 l/c/d (with min. figure of 200 l/c/d) which can further be reduced upto 135 l/c/d.

(4) The demand for a school or a hospital may be taken as 45 l/c/d.

(5) Loss through faulty joints and careless handling of taps may be taken as 25%.

(6) The per capita demand for human and animal consumption may be considered on the basis of factors indicated in table 12.2.

Table 12.2. Human and Animal Water Demand In Rural Areas

S. No.	*Item*	*Demand*
1.	Hand pumps are installed	45 l/c/d
2.	Piped supply with kitchens	75 l/c/d
3.	Schools/Camps	45 l/c/d
4.	Schools with flush toilets	100 l/c/d
5.	Schools (day only)	75 l/c/d
6.	Schools without piping	25 l/c/d
7.	**Livestock**	
	Horse	50 l/animal/d
	Dairy cow	70 l/animal/d
	Sheep	10 l/animal/d
	Chicken (100 Nos)	20 l/100 No./d

(7) For every 200 people, at least one tap must be provided.

(8) Water carrying distance should not exceed 100 metres.

(9) The design population for computing water demand should be taken as 1.5 times the present population.

WATER TREATMENT METHODS FOR RURAL SUPPLIES

12.5. Assessing the Required Water Treatment for a Given Source

The treatment of water for rural and small communities will largely depend upon the quality of available raw water, and also upon the availability of funds to meat the cost of technical personnel as well as the capital and *O.M* cost of the treatment plant. In less developed countries, the availability of funds are generally scarce and usually insufficient to meet the cost of a full fledged treatment plant. Most of the Indian rural supplies are therefore being derived from wells and tubewells (waters of which usually does not require much

treatment) and are usually supplied after simple **chlorination**, which ensures elimination of bacterial contamination present in raw water, or of enroute recontamination in piped supply, which otherwise may cause spread of large scale water borne diseases.

When deep bore waters are to be used for drinking purposes, testing for the dissolved metals like *fluorides, iron, manganese, nitrates, arsenic*, etc. becomes a virtual necessity. Many of the Indian deep well waters are found to contain excess concentrations of *fluorides* and *iron*. Simple methods have, therefore, been devised to treat rural supplies for removing fluorides and iron, as discussed in the next article.

Excess concentration of *nitrates*, if present in the raw water, cannot be easily removed. In such a case, we will have to look out for an alternate source. The excess nitrates usually occur in water as an end product of biodegradation of animal excreta, as may be found close to pit privies, manure heaps, or yards containing live stocks. All such areas should not be situated near the water source. Tests for the nitrate concentrations can be made, and if no alternate source is available, then even higher nitrate concentrations can be allowed. The testing for nitrates can altogether be avoided if the water source is away from the animal excreta and no evidence of methaemoglobinaemia (blue baby disease) is observed in small children locally.

Similarly, higher concentrations of dissolved salts may sometimes be ignored, if the local people have been able to tolerate the raw water without any harmful effects in the past.

Acidic and *alkaline* waters can be easily tested with pH papers. pH values below 6.5 indicate a *corrosive water* (acidic water) requiring **treatment with limestone**. pH values above 9.0 or so, may indicate an alkaline water, requiring treatment outside the scope of small rural projects, and such a source should be rejected, if possible.

The *suspended matter*, if present in raw water, may not always be injurious to health, but is always better removed. The usual methods of simple treatment are **settlement** and **filtration**. Most useful information can be obtained by settling a sample, say 1 litre in a tall clear glass bottle, for a period of an hour or more. Visual observation will indicate how much improvement of clarity can be achieved by settlement in a given time. Observation of the solid particles may indicate sand or coarse mineral solids, vegetable particles, algae, protozoa, bilharzia, carcaria and other forms of life.

A magnifier is useful for this purpose. If a second sample is filtered through a filter paper or clean blotting paper, it will be a guide to the likely improvement possible with sand filters. If the filtrate is coloured, it may indicate dissolved organic matter probably derived from swamps having algal growths.

Organic matter derived from fecal pollution, may contain pathogenic bacteria, and hence will have to be necessarily removed by disinfection or **chlorination**. Specific bacteriological tests are not usually practicable, but the general health of the local people is a good indication of serious risks. In general, most surface waters should be regarded as polluted and every effort made to **filter and disinfect** them. Some bore hole waters are found that are not polluted, and hence may be the preferable sources, if available. The simple

chlorine demand test can be performed to measure organic pollution and can be used for comparing different waters.

To perform **chlorine demand test**, a sample of raw water of 500 ml may be collected in a clean glass vessel, to which 3 drops (0.15 ml) of *stock chlorine solution** is added and thoroughly mixed and allowed to stand for 30 minutes in a shady place to enable the chlorine to react with the impurities present in water. The residual chlorine left in the sample is then determined by a simple chlorine residual test to be performed as given below :

Take 50 ml of sample in a clean glass vessel and add 5 ml of chlorotex reagent. A colour develops immediately, if chlorine is present. The developed colour will indicate the quantum of residual chlorine, as indicated in table 12.3.

Table 12.3

Colour developed	*Quantum of residual chlorine in mg/l*
White milky fluorescene	Nil
Faintly pink and milky	0.1
Pink	0.2
Red	0.5
Purple	0.6
Violet	0.8
Blue	1.0 or more[a]

The chlorine demand of raw water can be computed by subtracting the residual chlorine from the initial chlorine dose in the test performed with 3 drops (0—15 ml) of 1% stock chlorine solution in 500 ml of raw water. The total chlorine added in such a case in mg/l will be calculated as :

= 0.15 g (*i.e.* 0.15 ml) of chlorinated water × Chlorine strength (1%)

$$= (0.15 \times 1000 \text{ mg}) \times \frac{1}{100} \text{ of chlorine in 500 ml of water}$$

= 1.5 mg of chlorine in 500 ml of water

≈ 3 mg of chlorine per litre of water.

Hence, the initial chlorine dose with 3 drops of 1% chlorine solution will be equal to 3 mgl. The residual chlorine in mg/l, computed from table 12.3, when subtracted from initial dose, will easily give us the chlorine demand.

12.6. Treatment Methods for High Fluoride Waters

Ground waters in many parts of India are found to contain very high concentrations of fluorides, which cause mottling of teeth initially, and

*The **stock chlorine solution** can be prepared from dry bleaching powder (chloride of lime), which usually contains 35% of available chlorine, or by HTH (high test hypochlorite) which contains about 70% available chlorine. The bleaching powder or HTH powder is dissolved in clean water and the solution is kept in a dark bottle with a glass stopper. 1% chlorine solution can thus be made by mixing x gm of bleaching powder in 500 ml (500 gm) of clean water, where $x = 500 \text{ gm} \times \frac{1}{100} \times \frac{100}{35} = 15$ gm. Similarly 7.5 gm of HTH powder when mixed in 500 ml of clean water will give 1% chlorine solution.

fluorosis disease with continued use of fluoride rich waters. The disease and particularly its initial effects (mottling of teeth) is quite common in areas of States like Andhra Pradesh, Tamil Nadu, Kerala, Karnataka, Maharashtra, Punjab, Haryana and Rajasthan. Removal of excess fluorides from water, called **defluoridation**, therefore becomes very important in the context of Indian water supplies in rural areas, since most of our rural supplies are made from the ground waters.

A simple method, called the **Nalgonda method**, has therefore, been designed to defluoride waters in our country. *This technique, not only helps in removal of fluorides, but also helps in removing colour, odour, turbidity, bacteria and organic contaminations from raw waters. The method is simple and uses chemicals which are readily available.*

The Nalgonda technique uses aluminium salt (alum) for removing fluoride. The raw water is firstly mixed with adequate amount of lime (CaO) or Sodium carbonate (Na_2CO_3) and thoroughly mixed. Alum solution is then added, and water is stirred slowly for about 10 minutes, and allowed to settle for nearly one hour. The precipitated sludge is discarded and supernatant containing permissible amount of fluoride is withdrawn. *The added lime or sodium carbonate helps to ensure adequate alkalinity required for effective hydrolysis of aluminium salts, so that the residual aluminium does not remain in the treated water.* Bleaching powder is also generally added with lime prior to addition of lime, to achieve simultaneous disinfection of treated water and also to keep the system free from undesirable biological growth.

The Nalgonda method, thus involves *precipitation, settling* and *filtration.*

For small domestic individual supplies, the method can be practised in a container (bucket) of say 40 litres capacity, as shown in Fig. 12.15. The amount of alum to be added in mg/l in 40 litres of raw water, at various alkalinity and fluoride levels is given in table 12.4.

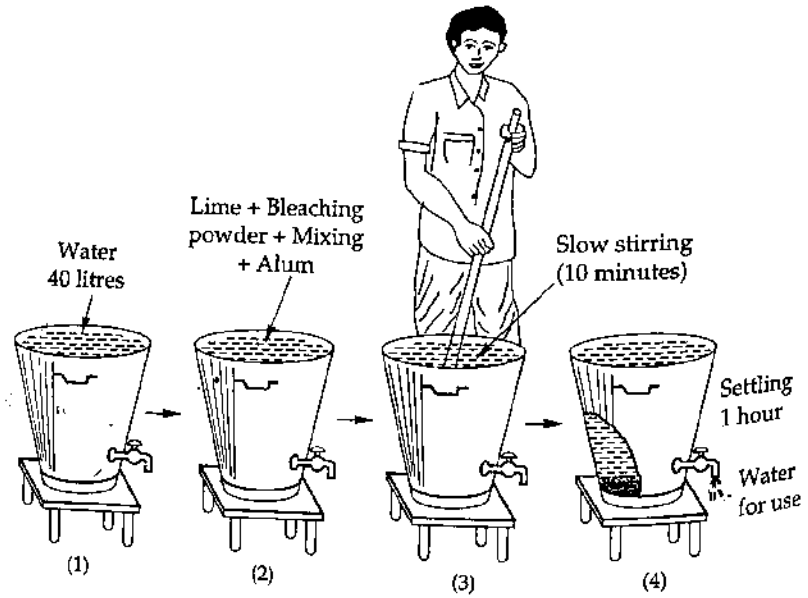


Fig. 12.15. Defluoridation (Nalgonda technique) at domestic level.

Table 12.4. Alum dose (mg/l) reqd. for one bucket of raw water having different Fluoride and Alkalinity levels

Test water Fluoride (F) mg/L	*Test Water Alkalinity ($CaCO_3$) in mg/L*							
	125	*200*	*300*	*400*	*500*	*600*	*800*	*1000*
2	60	90	110	120	140	160	190	210
3	90	120	140	160	205	210	235	310
4		160	165	190	225	240	275	375
5			205	240	275	290	355	405
6			245	285	315	375	425	485
8					395	450	520	570
10							605	675

For community use, the technique has also been used and different types of plants have been devised for different capacities. Fig. 12.16 shows a typical defluoridation plant (fill and draw type) which may be used up to about 8000 litres/day capacity for a very thinly populated village of about 200 population.

Fig. 12.17 shows a bigger sized typical installation, which is operated on electricity, and can serve a population of about 1500 @ 40 L/c/d.

The **smallest defluoridation plant** (Fig. 12.16) comprises of a hopper bottomed cylindrical tank with a depth of 2 m, equipped with a *hand-operated*

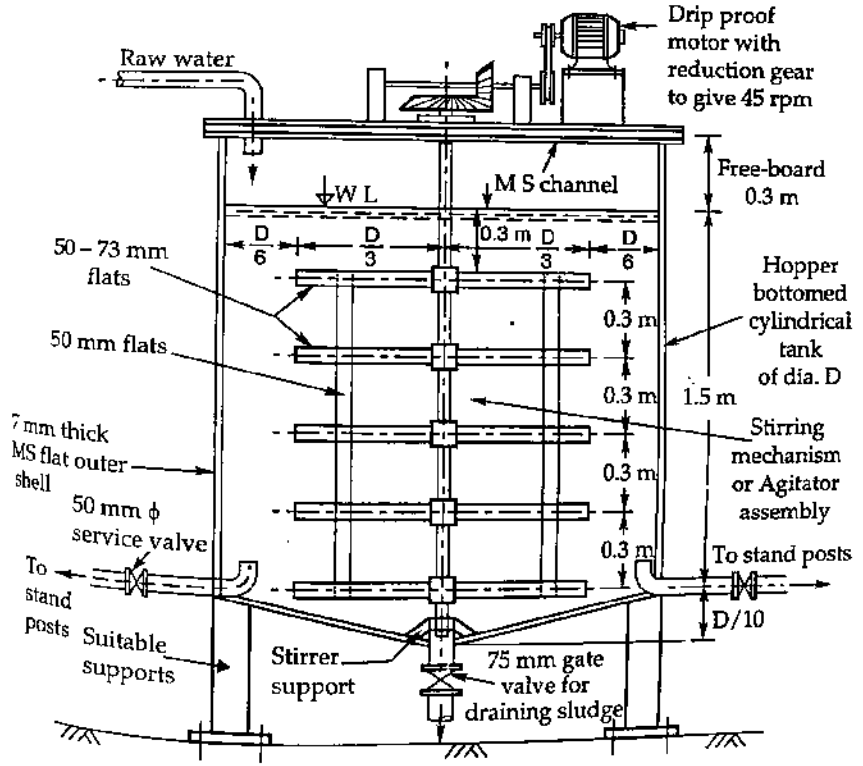


Fig. 12.16. Fill and draw type of Non-electric Defluoridation plant to serve upto 200 persons @ 40 Litres per capita per day.

or power-driven stirring mechanism as shown. Raw water is pumped or filled into this tank, to which is added the requisite quantities of lime (or sodium carbonate), bleaching powder and alum with steering. The contents are stirred for 10 minutes and allowed to settle for two hours. The defluoridated supernatant water is withdrawn to be supplied through stand posts and the settled sludge is discarded.

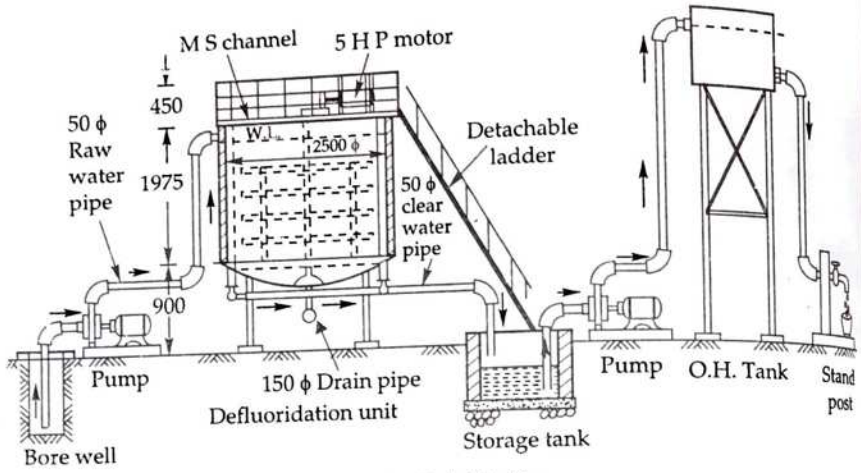
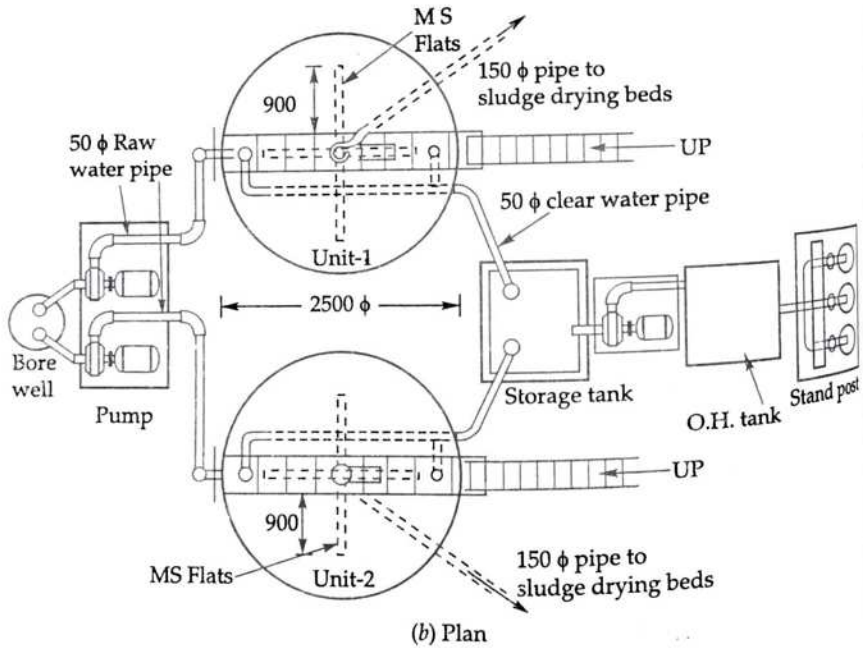


Fig. 12.17. (*a*) Section



(*b*) Plan

Fig. 12.17. Fill and Draw type of Electrically operated Defluoridation Plant to serve 1500 persons @ 40 l/c/d. (Dimensions in mm)

With a pump of adequate capacity used for filling the tank, the entire operation can be completed in 2-3 hours, and the operation of the plant can be done a number of times, to obtain treated water in a number of batches, say 5 to 10 per day. The accessories required in this plant are very few and easily available. These include 16 litres buckets for dissolving alum, preparation of lime slurry (or sodium carbonate solution), bleaching powder and a weighing balance. The plant can be located even in a open space with precautions to cover the motor. Semi-skilled labour can easily operate this plant.

A typical bigger sized **electrically operated batch type plant** with 2 units in parallel, sufficient to serve 1500 people @ 40 L/c/d is shown in Fig. 12.17. It consists of two cylindrical tanks, each of 10 m^3 capacity, a sump well, and an overhead reservoir, as shown.

Both the cylindrical tanks are having dished bottom and provided with inlet, outlet and sludge drain. The tanks will have sturdy railings, etc. Each tank is fitted with an ***agitator assembly*** consisting of : (*i*) a 5 H.P. drip proof electric motor (3 phase, 50 Hz, 1440 RPM, 415 V ± 6% voltage fluctuation) ; and (*ii*) a gear box for 1440 RPM input speed with reduction ratio 60 : 1 to attain an output speed of 24 RPM, complete with downward shift to hold the agitator paddles. The agitator is fixed to the bottom of the vessel by sturdy suitable stainless steel supporting bushings. The raw water is pumped into the units and treated by Nalgonda technique. The treated water collected in a sump is pumped to an over-head tank, from where it is supplied through stand posts. The approximate alum doses required to obtain permissible fluoride limit of 1 mg/l (as *F*) at various alkalinites and fluoride levels are given in table 12.5.

Table 12.5. Alum Dose in mg/l for different Fluorides and Alkalinity Levels for an Electrically Operated Plant (Batch Type—Fig. 12.17)

Test water Fluoride (F) mgF/l	*Test Water Alkalinity, mg $CaCO_3$/l*							
	125	*200*	*300*	*400*	*500*	*600*	*800*	*1000*
2	143	221	273	312	351	403	468	520
3	221	229	351	403	507	520	585	767
4	*	403	416	468	559	598	689	936
5	*	*	507	598	689	715	884	1010
6	*	*	611	715	780	936	1066	1209
8	*	*	*	*	988	1118	1300	1430
10	*	*	*	*	*	*	1508	1690

Note. *To be treated after increasing the alkalinity with lime or sodium carbonate.

A **continuous type of large sized plant** (Fig. 12.18) includes a *channel mixer, pebble bed flocculation, sedimentation tank,* and *constant rate sand filters.* The designs for such a plant are available for 500, 1000, 2000 and 5000 populations @ 70 lpcd (litres per capita per day). The plant is gravity operated

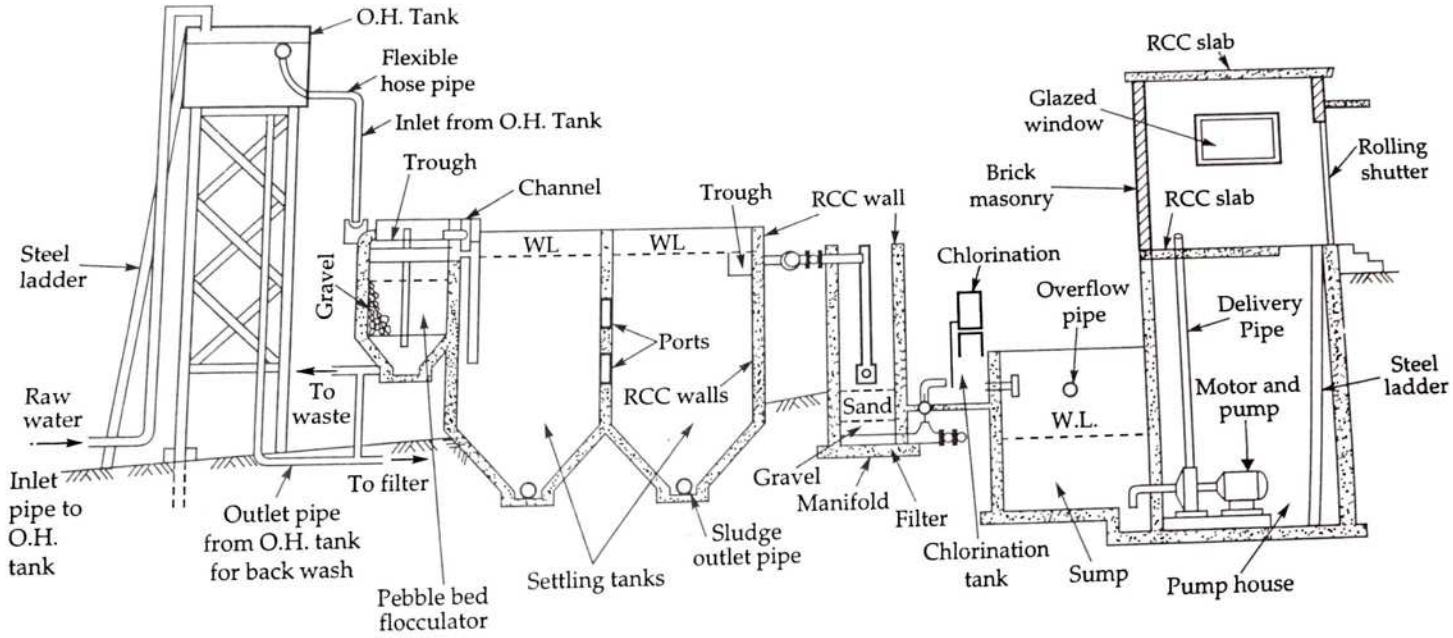


Fig. 12.18. (*a*) Sectional Elevation of a Continuous Defluoridation Plant.

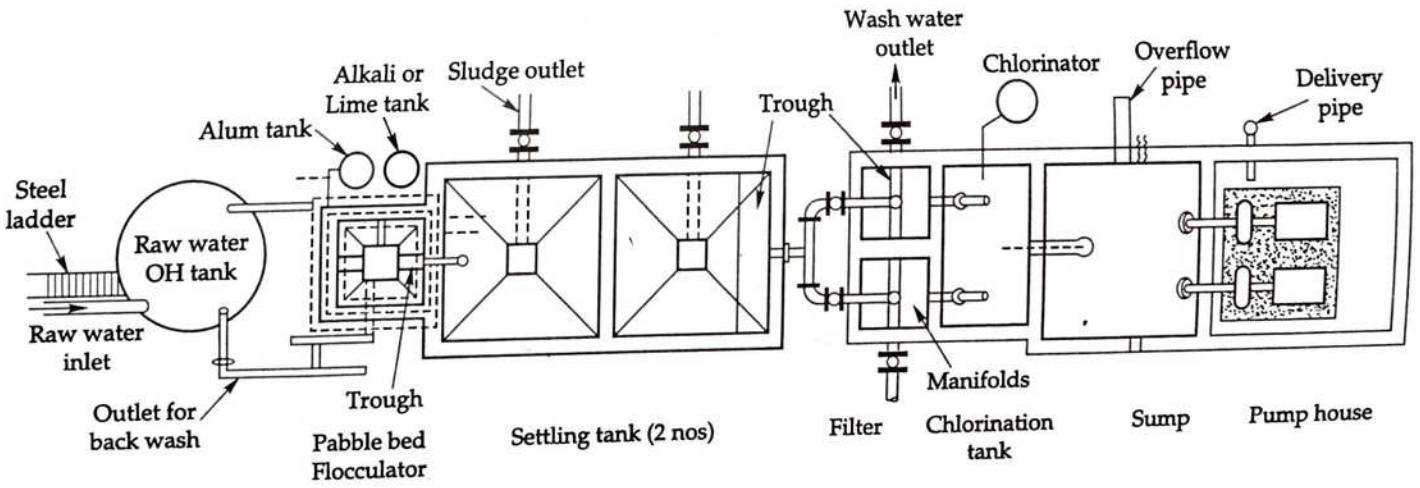


Fig. 12.18. (b) Plan of a Continuous Defluoridation Plant

Fig. 12.18. Details of a Typical Continuous type of Defluoridation Plant.

except for filling of the over-head raw water tank and for lifting the treated water from treated water sump for distribution. ***Channel mixer*** is provided for mixing lime slurry (or sodium carbonate solution) and alum with the raw water. ***Pebble bed flocculation*** is used in place of conventional flocculation, to avoid the dependence on electric power supply. The plant requires power supply for 2 hrs each during morning and evening for filling the overhead tank and for supply of treated water. The design of various units is made on the following basis :

(*i*) Water consumption	70 lpcd
(*ii*) Flash mixing detention period, velocity to be maintained	30 secs
	30 minutes
(*iii*) *Pebble bed flocculator*	
(*a*) detention period (considering 50% voids)	
(*b*) size of media	20—40 mm
(*c*) depth of media	1.2 m
(*d*) rate of back wash	0.5 m/min
(*iv*) *Sedimentation tank*	
(*a*) liquid depth	3 m
(*b*) weir loading rate	< 300 $m^3/m/d$
(*c*) surface over loading rate	< 20 $m^3/m^2/d$
(*v*) *Sand gravity filter*	
(*a*) depth of water over sand	2 m
(*b*) rate of filtration	5 $m^3/m^2/h$
(*c*) head required for back washing	12 m
(*d*) filter backwash rate (minimum)	1.0 m
(*e*) gravel depth	0.45 m
(*f*) effective size of sand	0.6 mm to 0.8 mm

The size of all the units, *viz.* overhead tank, channel mixer, pebble bed flocculator, sedimentation tank, sand filter, and underground storage tank are based on the above parameters for 500, 1000, 2000 and 5000 persons.

The Nalgonda technique used in the above described plants (Fig. 12.16 to Fig. 12.18) do possess several advantages over the ion-exchange process (other method of defluoridation) as indicated in article 9.25.2.

12.7. Treatment Methods for Removing Iron

Many of the Indian deep bore hole waters are found to contain excess concentrations of iron than the permissible value of 0.3 mg/l. The concentrations may sometimes even exceed the absolute max. value of 1.0 mg/l. The excess concentration of iron does impart bitter characteristic metallic taste to the water. The oxidised precipitates of iron, if present, can cause colouration of water, which may be yellowish brown to black, thereby rendering the water unsuitable and objectionable for domestic purposes. The iron rich waters also cause staining of the plumbing fixtures and clothes washed in laundries. Deposition of iron oxide in the pipe lines of the distribution system may not only lead to reduced carrying capacity, but also promote the growth of ***iron bacteria*** (micro-organisms) causing **bacterial slimes.**

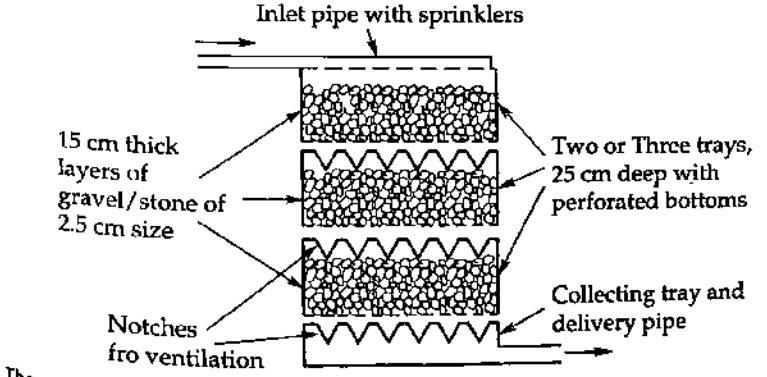
Usually, iron concentrations in excess of 0.2—0.3 mg/l may cause nuisance, even though its presence does not affect the hygienic quality of water.

Iron, in ground waters, is usually contributed by the dissolution of rocks and minerals, chiefly oxides, sulphides, carbonates and silicates of iron. This dissociation is enhanced by the presence of carbon dioxide present in ground water. *The most usual soluble iron compounds found in waters are ferrous sulphate and ferrous bicarbonate.* Therefore, in clear ground waters, iron present in dissolved state, is generally found as ferrous ions (bivalent)—Fe^{++}. The trivalent ferric ions (Fe^{+++}) usually occur in precipitated (insoluble) form, which can be easily settled out.

The iron may be present in water either in combination with organic matter or without such combination. When present without combination with organic matter, the iron can be easily precipitated by ***aeration***, whereby the soluble ferrous compounds get oxidised into insoluble ferric compounds, which can be sedimented out. *Oxidation, followed by filtration alone, or by settling and filtration, can, thus bring about the removal of iron from water.*

On the other hand, when iron is present in combination with organic matter, it becomes difficult to break the bond between them and to cause their removal. However, when once this bond is broken, it can be removed as above. This bond may be removed either by adding lime and thereby increasing the pH value of water to about 8.5 or 9, or by adding chlorine or potassium permanganate during oxidation, which when followed by settling and filtration can remove iron.

12.7.1. Simple Technique for Removal of Iron from Raw Supplies in Rural Areas and Small Communities. Efficient iron removal can be accomplished by aerating the water. This can be easily carried out by allowing the water, after treatment with lime to raise the pH value if necessary, to trickle through two to three layers of broken stone or gravel of about 2.5 cm



The approximate rate of flow through this type of filter is up to 1600 litres per hour per m^2 area. The number of trays should be found by trial.

Fig. 12.19. Trickling filter for aeration and iron removal.

size and 15 cm deep. Two or more trays containing such gravel or stone, may be used, separated by air spaces (notches for ventilation), as shown in Fig. 12.19. If coke can be obtained, it often performs more efficiently than gravel or rock. It may be worth experimenting with hard charcoal, broken into suitable sizes. The rate of application of water to this kind of filter is approximately 1600 L/m²/h. Iron compounds may also be precipitated by chlorine and some other chemicals, but these processes require more control than is likely in rural areas. The trickling device, described above for iron precipitation, aerates the water and may have beneficial effects on the general taste and odour.

For small communities in rural areas, piped distribution of water is generally not used, being costly. A hand pump or a forced hand pump to lift ground water is, therefore, widely used in such areas. If the ground water contains iron, the same can be removed by connecting a ***small iron removal plant*** with the hand pump. Such an iron removal plant (Fig. 12.20) contains

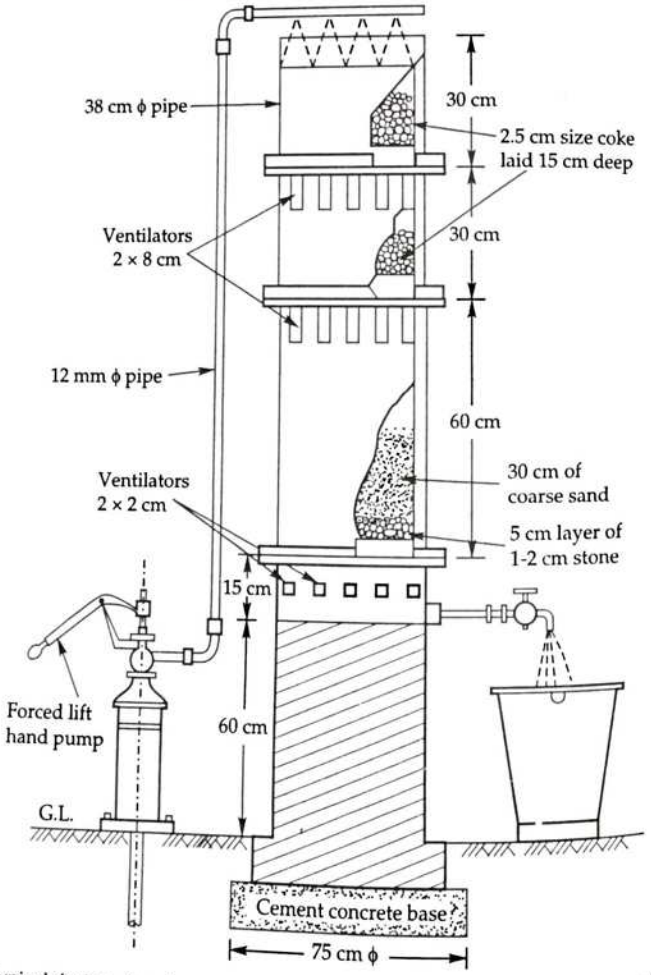


Fig. 12.20. Typical design of an iron removal plant for individual houses up to 200 L/hr capacity.
Courtsey : GOI Manual on Water Supply and Treatment.

two tray type aerators, operating at a rate of about 1260 L/m²/h to aerate the water. The water is they settled in a sedimentation basin having a detention period of about 3 hours, and the clarified water finally passes through a rapid sand filter. Provision could be made for adding sodium carbonate to the raw water wherever essential. A typical design of such an iron removal plant in shown in Fig. 12.20.

Based upon the above principles, NEERI has designed package iron removal plants having different capacities, such as 500 L/hr, 1000 L/hr, 1500 L/hr, 2000 L/hr depending upon the requirement of treated water and discharge of hand pump. Typical details of such a plant of 1000 L/hr capacity are shown in Fig. 12.21.

Such a plant is designed in a circular (or a rectangular) shape, having *aeration chamber, collection chamber, settling chamber*, and *filter*. The *settling chamber* is provided with a *plate settling device* to enhance settling and reduce detention time, thereby reducing the dimensions of the settling chamber.

The *aeration chamber* contains media of size 2.0—5.0 cm gravel or stone chips to increase the surface area of air-water interface. The iron contaminated hand pump water trickles over the aeration media through spraying device. The aerated water flows through pores over baffle plate to *collection chamber*, to *settling chamber*, and to *filter*.

The filter bed of 20 cm depth contains sand media of size 0.8—1.4 mm, supported by 5 cm depth of gravel of size 0.8—1.0 cm. The treated water is taken out from the water tap attached to it.

The sedimentation chamber, having a detention period of one hour, is provided with two plate *settlers* at 45° angles, as shown (Fig. 12.21).

The filter is cleaned by making back-wash connection with hand pump, scrapping sand bed manually and opening the *sludge scouring valve*. The hand pump is operated with possible high speed till clean water gets accumulated over the sand bed.

Continuous iron removal plants in capacities of 5000 L/hr, 10000 L/hr and 20000 L/hr, which can be used for larger communities, have also been designed by NEERI, and are shown in Figs. 12.22, 12.23 and 12.24, respectively.

Such a continuous community plant not only removes iron from the raw water, but also helps in removing the free carbon dioxide and toxic substances, which are usually present in ferruginous waters. Moreover, since simple aeration may not remove the iron which is associated with organic matter, lime addition to raise the pH value to about 8.5—9, may also become necessary to treat community water supplies. Hence, in such plants, provision is also made for chemical dosing. All in all, such a plant comprises of : *raw water overhead storage tank, cascade tray aerators, chemical dosing cum sedimentation tank, sand filters, and clear water tank* with provision of disinfection (chlorination).

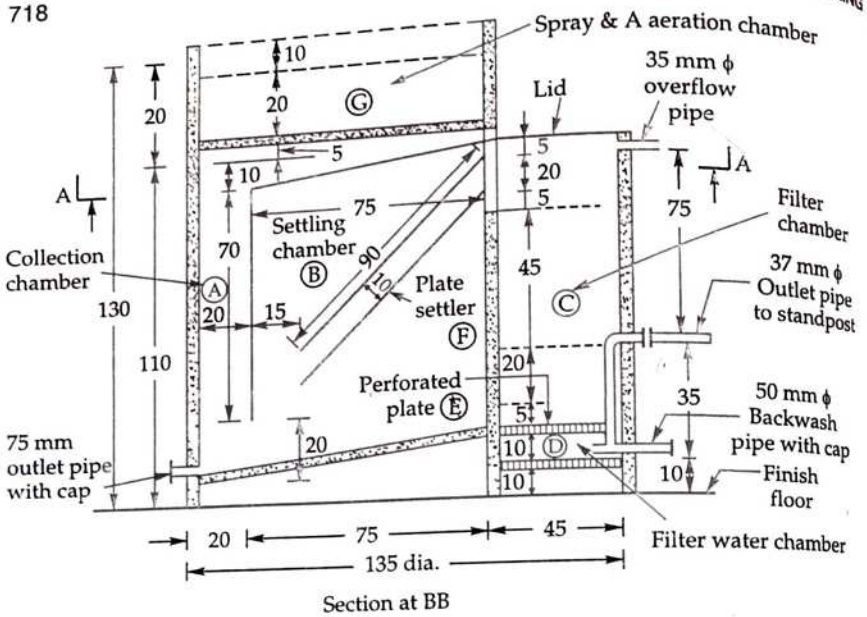


Fig. 12.21. (*a*) Section at B-B

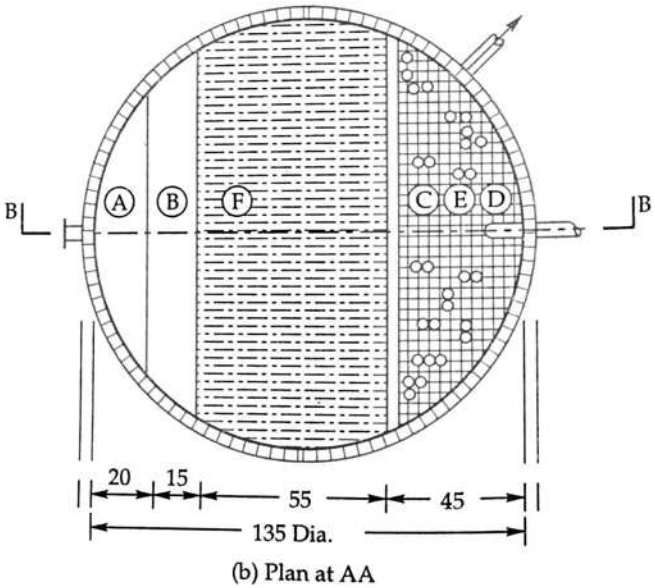


Fig. 12.21. (*b*) Plan at A-A (dimensions in cm).

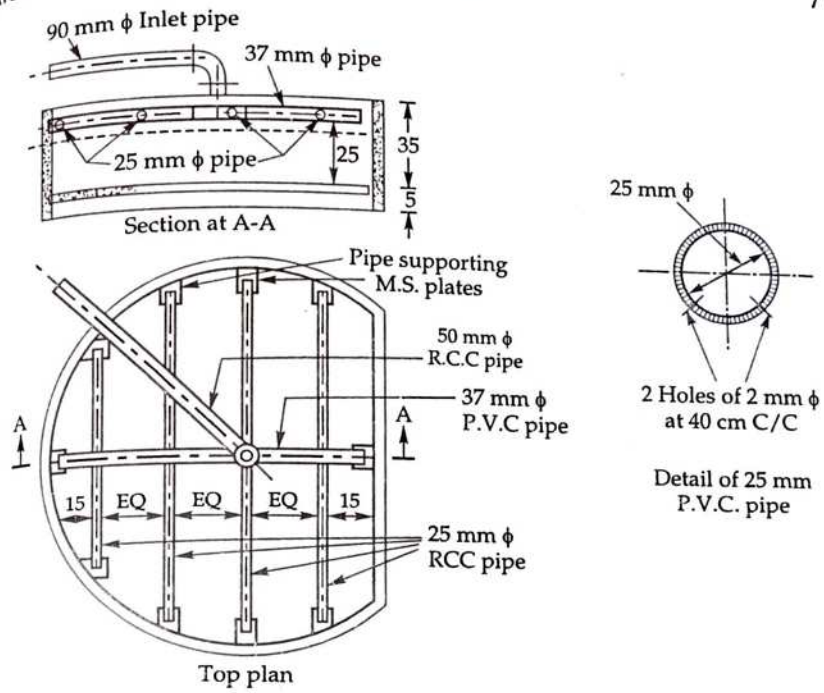


Fig. 12.21. (*c*) Top plan.

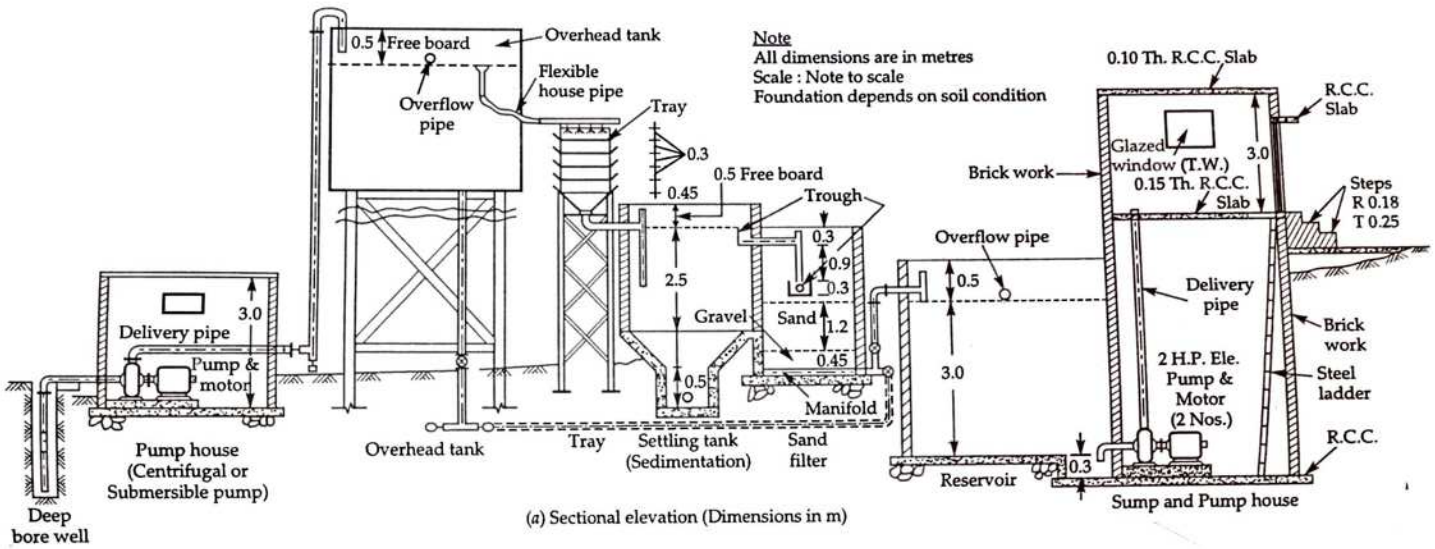
Detail Specifications		
A. Collection chamber	20 × 70	semicircular
B. Settling chamber	90 × 80	semicircular
C. Filter chamber	45 × 70	semicircular
D. Filter water chamber	45 × 10	semicircular
E. Perporated plate	45 × 5	semicircular
F. Plate settler	55 × 90	semicircular
G. Spray and aeration chamber	85 × 30	semicircular

Notes:

1. All Dimensions are in cms
2. Piping work as specified
3. Main body—Ferrocement pipe 135 cm I.D. with 2 cm thick
4. Frame work—Ferrocement
5. Joints—Wherever necessary welding should be adopted
6. Paints—Primer and anticorrosive paint shall be given both sides of the unit.

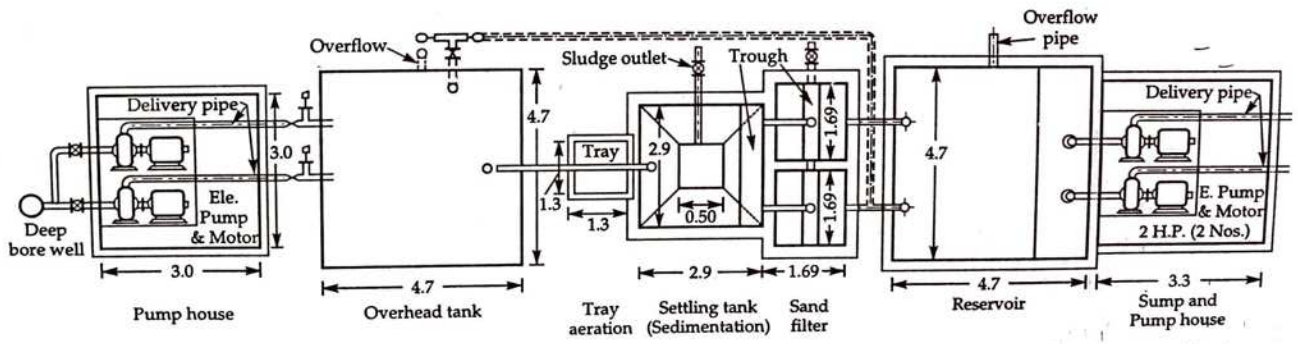
Fig. 12.21. Iron removal plant of 1000 L/hr capacity.
Courtsey : GOI Manual on Water Supply and Treatment.

Tray aerators are commonly used in such a plant for aerating water. The trays are designed for an aeration rate of 1260 L/m²/hr and spaced at intervals of 1 m. The aerated water is settled in *sedimentation tank* having a detention



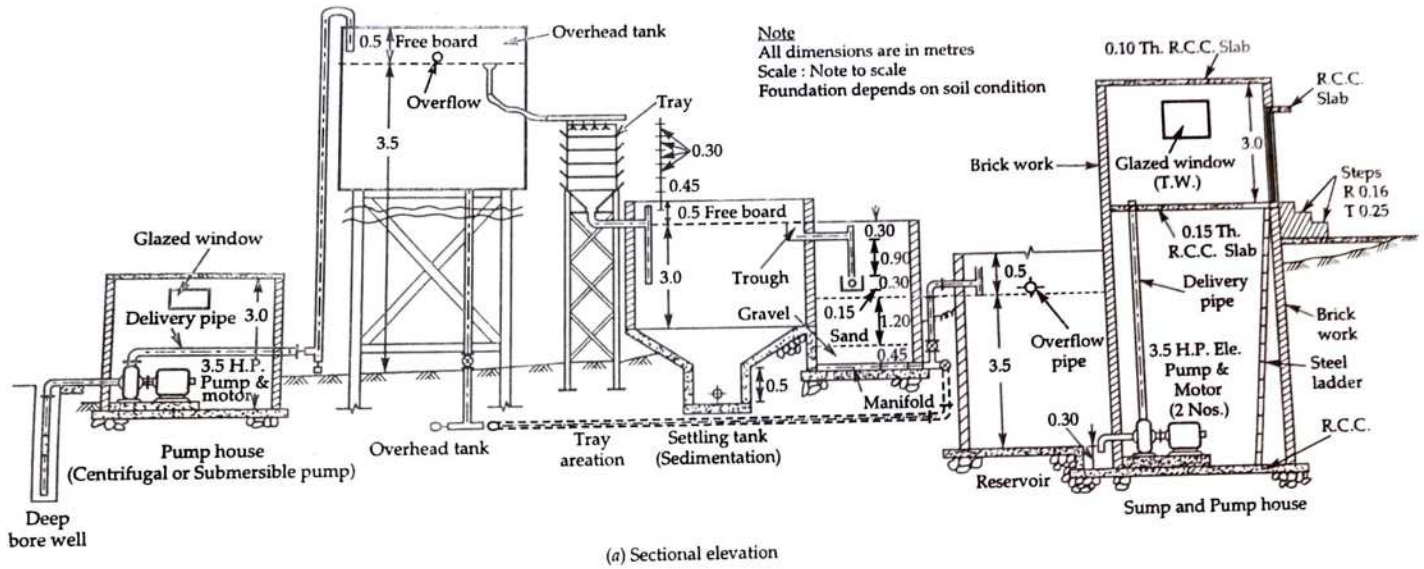
(*a*) Sectional elevation (Dimensions in m)

Fig. 12.22. (*a*) Sectional Elevation of a continuous iron removal plant-capacity = 5000 L/hr.



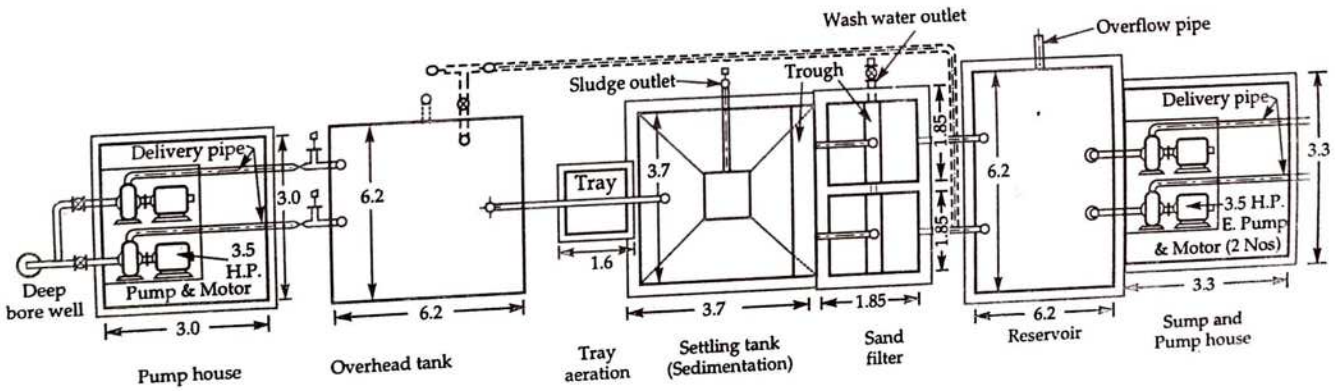
(*b*) Layout plan

Fig. 12.22. Continuous iron removal plant for community water supply—capacity 5000 L/hr.
Courtsey : GOI Manual on Water Supply and Treatment.



(*a*) Sectional elevation

Fig. 12.23. (*a*) Section elevation of a continuous iron removal plant-capacity = 10,000 L/hr.



(*b*) Layout plan

Fig. 12.23. Continuous iron removal plant for community water supply—capacity 10,000 L/hr.
Courtsey : GOI Manual on Water Supply and Treatment.

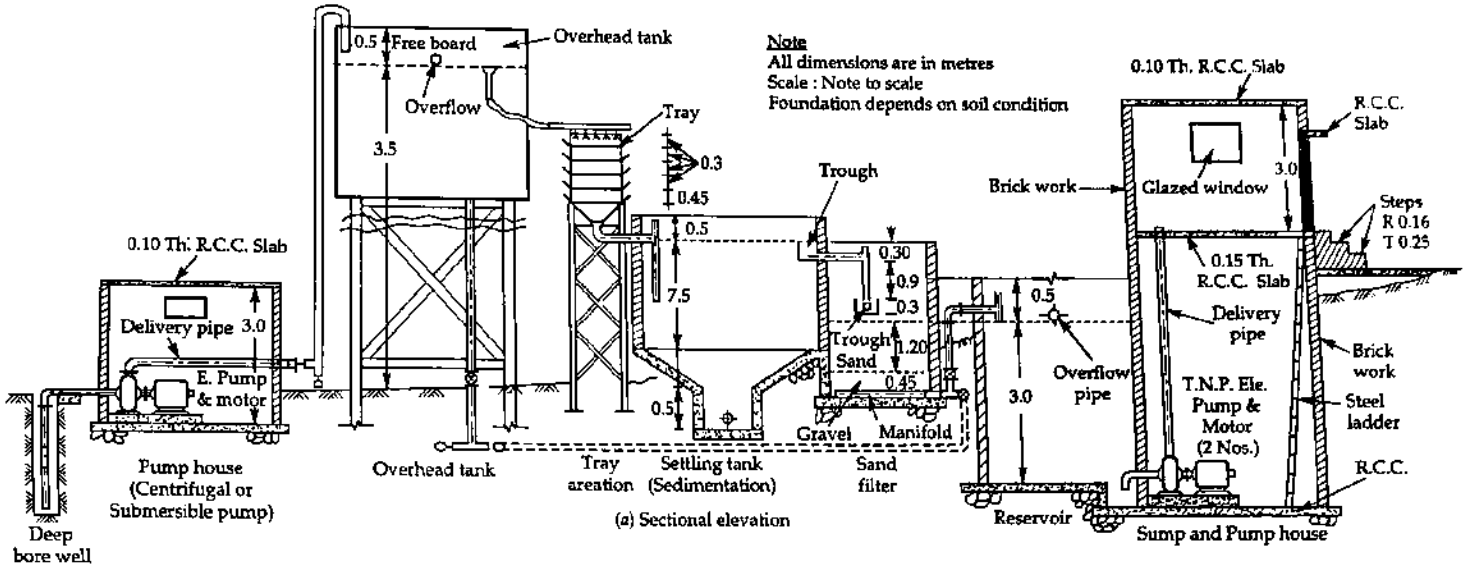
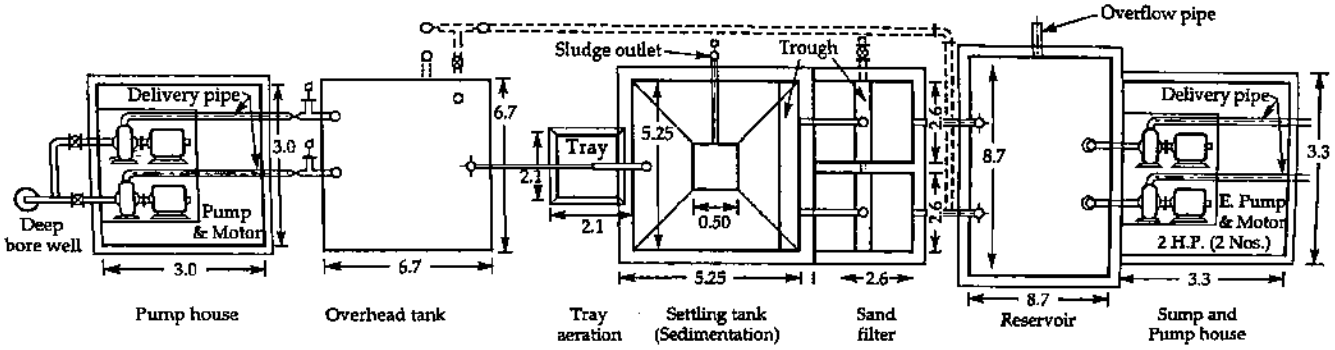


Fig. 12.24. (*a*) Sectional elevation of a continuous iron removal plant-capacity 20,000 L/hr.



(b) Layout plan

Fig. 12.24. Continuous iron removal plant for community water supply—capacity 20,000 L/hr.
Courtsey : GOI Manual on Water Supply and Treatment.

period of 2.5 hours. The clarified water is filtered through a *rapid sand filter* having sand of effective size 0.6—0.8 mm and uniformity coefficient 1.3, with an effective depth of 1.2 m. The head of water above sand is 1.35 m and the rate of filtration 5000 L/m²/h, and the total head required for filter wash is 12 m. The sand is supported over a gravel layer of depth 0.4—0.6 m, and it is arranged as follows :

gravel size	*gravel depth*
68—38 mm	13—20 cm
38—20 mm	8—13 cm
20—12 mm	8—13 cm
12—5 mm	5—8 cm
5—2 mm	5—8 cm

Since power shut downs are very heavy in rural areas, and rarely more than 2—3 hours supply is available in the morning and evening, the raw water supply hours can be assumed to be 2 hours in the morning and 2 hours in the evening. During these 4 hours of pumping, the total daily requirement of water is to be pumped into the raw water overhead tank. The treatment plant has therefore to be designed to operate under gravity from the raw water elevated storage tank taking these facts into account.

To avoid extra cost for additional overhead distribution tank for filtered water, the filtered water from the pump well could be directly pumped for distribution. In such a case, the distribution of treated water would follow the same time schedule as would be there for pumping raw water ; and back washing of the sand filter would be carried out by using raw water from the overhead raw water tank.

12.8. Chlorinating Small Community Supplies

It is essential that public water supplies are bacteriologically safe. Some deep well waters are naturally safe, while most surface waters are polluted, and must be sterilized. This sterilization is usually carried out by chlorination.

Chlorination is usually carried out by chlorine gas in large treatment plants, producing public supplies exceeding 25,000 L/hr. Small community supplies, however, find it more convenient to use solutions of chemicals containing free chlorine, such as bleaching powder, chlorinated lime, sodium hypochlorite or HTH, etc.

The bleaching powder naturally contains 30—35% of available chlorine. Thus, if 1 kg (10^6 mg) of bleaching powder of 30% strength is added to one million litre of water, the chlorine dose will be equivalent to 0.3 mg/l.

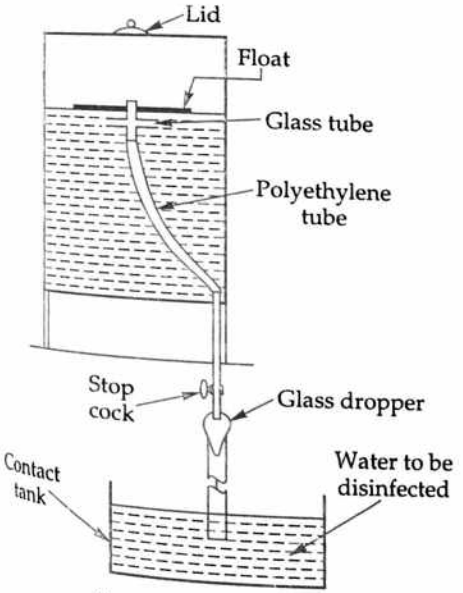
The chlorine dosing equipment should be sufficient firstly to allow not less than 30 minutes of contact with the water before use, and secondly to maintain a chlorine residual of atleast 0.3 mg/l after that time. The methods of estimating the required chlorine dose and measuring the residual have been discussed earlier in chapter 9. In case, chlorine dose could not be computed by testing, a general dose of 5 mg/l may be considered sufficient and applied by preparing appropriate stock solution of chlorine, which can be dozed at a

constant rate into the well water contained in the *contact tank* (which provides 30 minutes contact).

Say for example, by dissolving 100 gm of bleaching powder of 35% strength in 70 litres of water, we will get a stock solution of $\frac{(100 \times 1000)\text{ mg} \times 0.35}{70\text{ L}} =$ 500 mg of available chlorine/L.

If one litre of this stock solution is added to 100 litres of water, the chlorine dosage will be $= \frac{1}{100} \times 500 = 5$ mg/L. Hence, if the dosage of stock solution (kept in the dozer or dozing equipment) is regulated @ one part of stock solution to 100 parts of water, then the applied dosage will become 5 mg/L. The dosage can, thus be controlled to any desired value.

An economical and simple type of a **dosing equipment** or **doser** is shown in Fig. 12.25. It consists of a metal drum which is painted inside with bituminous paint to protect the metal from corrosion. A rubber delivery tube is fitted to the bottom of the drum, the other end of which will supply bleaching powder solution to the contact tank through a stop cock device to control the flow rate. Another flexible tube is fitted with one end connected to a float and the other



(*a*) Section of a doser

(*b*) Photoview of a doser
[Available in 35 L, 70 L, 110 L, 180 L and 220 L capacity from MAIC (INDIA) Laboratories of 0/11A, Jangpura Ext. New Delhi-110014]

Fig. 12.25. Displacement type of a dosers.

connected to the inlet of the delivery tube. This arrangement will maintain a constant head of feed solution over the mouth of the inner flexible tube.

This type of a doser can also be used for feeding alum or soda in raw water, if required for flocculation proposes.

All new water mains as well as water-works plant should be sterilized with a chlorine dose of at least 25 mg/L and thoroughly flushed out before use.

PROBLEMS

1. Enumerate and discuss the comparative merits and demerits of various sources of water, especially in the context of rural water supply schemes. In this connection, draw neat sections showing the protection works that you will suggest for obtaining unpolluted supply from (*a*) a spring ; (*b*) a tubewell ; and (*c*) a shallow driven well.
2. Enumerate and discuss the various methods that are employed to lift water from wells and tubewells in rural areas in India. Also discuss the method employed for rain water harvesting for drinking purposes.
3. Discuss the various factors that you will consider in order of priority for selection of a source of water for a rural water supply scheme. Also discuss the quality checks that you will like to perform on the available water sources to assess their usefulness before making a final choice.
4. How will you estimate the water requirement of a village, while planning a water supply scheme for rural areas. Also discuss as to how you will assess the necessary treatment which should be given to the available water source in the area ?
5. The ground water proposed to be used for a rural water supply is found to contain high concentration of fluoride. Discuss the suggested treatment method that is generally used in India in such cases. Also draw neat sketches of the flow diagram for the treatment plant that you will recommend if the supply is to be made to : (*i*) 5,000 persons @ 100 litres per capita per day ; (*ii*) 800 villagers @ 70 l/c/d.
6. (*a*) Explain the Nalgonda technique of defluoridation by exhibiting its use for an individual rural household.

   (*b*) How does excess concentration of iron in water affect the consumers ?
7. Discuss in details the necessity of iron removal from bore-hole waters and the techniques to be used for doing so. Sketch the type design of an iron removal plant, capable of giving 200 litres per hour of treated water.
8. Discuss the working and construction of a continuous iron removal plant that you will recommend for a water supply scheme to be designed for meeting the water demand of a town estimated to be about 40,000 L/d, when electricity is assured only for 2 hours in the morning and 2 hours in the evening.
9. Explain as to how you will chlorinate rural water supplies by using bleaching powder and a doser. Assuming that a total chlorine dose of 3 mg/l is required to treat the raw bore water and bleaching powder contains 30% chlorine, compute the quantity of bleaching powder that you will dissolve in the doser tank of 70 litres capacity, which supplies chlorine stock solution to raw water in the ratio of 1 L to 100 L in the contact tank.

—13

Planning and Preparing Water Supply Projects

13.1. General Introduction

A general layout of a water supply project from the source (river) to the distribution of the treated water to the consumers, is shown in Fig. 13.1. Such a project essentially consists of (*i*) works for collection of water ; (*ii*) works for conveyance of water ; (*iii*) works for treatment and purification of water ; and (*iv*) works for distribution of water to the consumers. Before the actual construction of these works is taken up, it is necessary to properly plan and prepare the full scheme, and also to design the various components of the project. *The proper planning will ensure an economical and efficiently functioning water supply scheme, which will serve the various objectives in view, most efficiently and with the minimum expenditure and recurring operational troubles.* The prepared scheme should also be such as to be accommodated only within the available funds or the funds likely to be received in the near future, as and when extensions are desired and suggested.

13.2. Data to be Collected

In general, the following data must be collected for planning water supply projects :

(*i*) *Hydrological data* and data on available surface water sources existing in the vicinity of the area. (This will help in estimating the availability of surface water).

(*ii*) The *geological data* and data on available ground water in the vicinity of the area. (This will help in estimating the availability of ground water).

(*iii*) *Data on water requirements of the city* including the data on population and their water needs, and the data on industrial establishments and their requirements. (This will help in estimating the quantity of the water required at present as well as in the future).

(*iv*) *Data on the existing water supply projects.* (This will help in assessing the net additional availability of water, and also the net additional requirements of water).

(*v*) *Data on the sanitary conditions of the area* and the possible sources of pollution. (This will help in estimating the possible pollution or contamination of the water from such sources, and thereby helping in assessing the remedial measures).

(*vi*) *Data on the topography of the area.* (This will indicate low lying areas and high ridges in relation to the available sources, the density of population

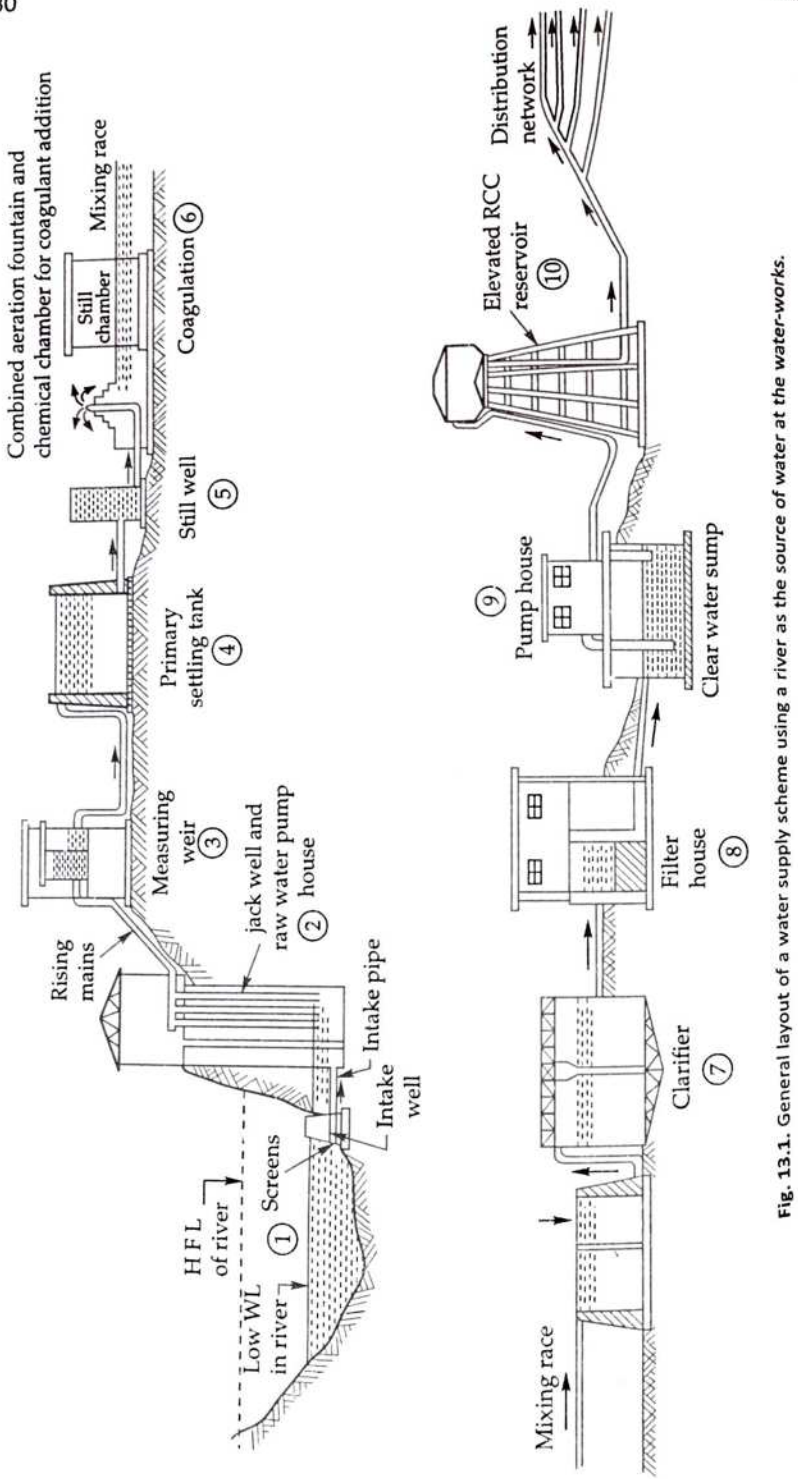


Fig. 13.1. General layout of a water supply scheme using a river as the source of water at the water-works.

in different localities, etc. It will help in properly positioning the intakes, treatment works, and above all, distribution reservoirs, etc. ; thereby evolving simple and a cheap conveyance and distribution of water).

(*vii*) ***Legal data*** on various legal laws on water rights, land zoning, land ownership, administrative pattern, etc. may also be collected. (This will help in using such waters, and positioning the various works at such places, that will not involve much legal troubles).

(*viii*) ***Data on the public opinion*** may also be collected. (This may help in counteracting the false claims of the ambitious politicians during the justification debates, while seeking administrative approval and expenditure sanction for the project from the Government).

13.3. Analysis of Data and Project Formulation

After determining the basic data, the future needs are evaluated. This will help in designing the project in such a way as to give useful service till the end of the design period. After evaluating the future needs, the project is planned, and proposals framed for : using a particular source or sources ; locating the positions and types of intakes to be used ; determining the extent of the treatments required, and locating the various units of the treatment plant ; planning the positions and types of different units of the distribution system ; determining the positions and types of pumping machinery if needed, etc.

A flow diagram indicating the sequence of operations and positioning of various components should be drawn and finalised, in addition to drawing certain other drawings*. The various components and units of be waterworks should then be designed, as to be structurally and hydraulically safe. In general, the following things must be done while planning and designing a municipal water supply scheme :

(*i*) Estimate the future population of the community and study the local conditions, as to determine the quantity of water which is required to be provided by the project.

(*ii*) Locate a reliable source or sources of adequate quality water, so as to fulfil the needs and requirements of the community.

(*iii*) Make provisions for the necessary storage of water, if needed, and design the various works required to deliver the water from its source to the consumers.

(*iv*) Determine the physical, chemical and biological characteristics of the available water.

(*v*) Design the water treatment facilities required, if any, for bringing the quality of the water to the required potable standards.

(*vi*) Design the distribution system including the storage facilities, pumping stations, layout and sizing of pipe networks, location of fire hydrants and other valves, etc.

(*vii*) Make provision for establishing an organisation which will maintain and operate the various units of the waterworks.

*Described in the next article.

13.4. Project Drawings

The entire planning and design of the various components of the water supply scheme has to be expressed through drawings. The following drawings are, therefore, generally prepared and used for estimating and executing the project :

(*i*) ***Topographical map.*** This will show the topographical features of the area, such as, roads, sources of supply, limits of the community to be served, etc. It may also show the layout of the pipes conveying water from the source to the city. This map may be prepared on a scale of 1 : 500 or so.

(*ii*) ***Site plan.*** A site plan of the town showing the location of the scheme and the area to be served by the scheme, may be prepared. A scale of 1 : 500 or so, may be used.

(*iii*) ***Contour plan.*** A contour plan of the entire area, on a scale of 1 : 100 or so, may be prepared, which will show the locations of water mains, sub-mains, branches, valves, hydrants, service reservoirs, pumping stations, boosters, roads and streets, etc. This map is the most important. Sometimes, the site plan and the contour plan are combined together (*i.e.* contours are shown on the site plan itself) and no separate site plan is prepared.

(*iv*) ***Flow diagrams.*** The line plan and section of the entire scheme showing the sequence of operations and all other aspects of the scheme should be prepared.

(*v*) ***Detailed drawings.*** The detailed drawings of the various components and units of the scheme are prepared giving full dimensions as well as hydraulic and structural details. These drawings will include the L-sections of the mains and the branches, the details of the treatment units, etc.

13.5. Project Estimates

After the preliminary drawings have been prepared (excluding the detailed drawings), a ***preliminary estimate*** indicating the tentative cost of the scheme is prepared and sent to the competent authority for seeking administrative approval and expenditure section. The preliminary estimate is a rough estimate and may also include the cost of the staff quarters, pump houses, etc., in addition to the cost of various units and components of the scheme. The preliminary estimate may be prepared on the basis of the tentative costs of the different units on per sq m or per cum basis, computed on the basis of actual incurred costs on such a work in the near past, with suitable provision for inflation and cost indices. Detailed cost estimates may be prepared, if such unit rates are not available.

The cost of different components of a water supply scheme in relation to the total cost are roughly computed as shown in table 13.1.

During the time, the formalities for obtaining the administrative approval are completed, the detailed drawings and *detailed estimates* for the execution of the project are prepared. After the administrative approval is obtained, the competent authority of the concerned Engineering Deptt. will technically sanction the estimate. This detailed estimate, after being technically sanctioned, becomes a kind of guarantee that the public money will be safely and soundly used on the project, and is hence ready for call of tenders and awarding the execution of works by suitable agencies.

Table 13.1. Tentative Costs of Components of a Water Supply Scheme

S.No. (1)	*Component item* (2)	*Cost of the item expressed as percentage of the total* (3)
(*i*)	Pumping stations	18%
(*ii*)	Reservoirs	6%
(*iii*)	Treatment plant	10%
(*iv*)	Supply penstocks	9%
(*v*)	Distribution system	50%
(*vi*)	Buildings for housing operational staff, etc.	2%
(*vii*)	Meters and other contingencies	5%
	Total	100%

13.6. Project Reports

The project drawings and estimates should always be accompanied by a report on the scheme. This report should include the justification for the necessity of the scheme, the economical justification for carrying out the scheme including the cost-benefit considerations*, the pros and cons of the alternative proposals if any, extent of land acquisition involved, the design principles and actual designs** involved in the design of the various units, water rates proposed to be levied on the consumers, etc.

A general outline of a typical detailed report on a water supply project is given in Table 13.2.

Table 13.2. General Outlines of a Detailed Report on a Water Supply Project

S. No. (1)	*Topic and Item* (2)	*Points to be discussed* (3)
1.	Necessity of the scheme	The reasons for the necessity and urgency of the scheme.
2.	General details of the area	The present area to be served by the scheme and the future area to be included ; the topographical and hydrological features of the area and its surroundings ; present and designed populations ; the facilities for transport ; etc.
3.	Water requirements	Assumptions made in deriving the average daily demand, the domestic, industrial, and fire demand ; the variations in consumption.

Contd...

*Please refer chapter on "Engineering Economy in Water Resources Planning" in "*Water Resources Engg. (Vol. I)–Hydrology and Water Resources Engineering*" by the same author.

**may be avoided in preliminary reports.

(1)	(2)	(3)
4.	Sources of supply	Description of available nearby sources ; the quantities available ; their general water qualities ; reliability of supplies, ; sanitary conditions surrounding these sources and the chances of their getting contaminated ; suitability and choice of particular source or sources.
5.	Purifications suggested	The physical, chemical and bacteriological characteristics of water to be used ; the standards to be obtained after purification ; the recommended treatment processes, the designs of various units of the treatment plant, the expected results, the special treatments reqd. if any.
6.	Pumping Installations	The type and horse power of pumps reqd., the stages and places where required ; the standby units, their operation and operational staff required ; the locations of pumping stations and details of buildings, required.
7.	Distribution system	The locations and necessity of reservoirs and overhead tanks, materials for their constructions, storage capacities ; dimensions and sizes, operation and control methods ; the details of distribution networks, their layouts, location of valves, hydrants and other service connections, etc.
8.	Water pressures	The pressure with which water can be supplied, loss of head in the distribution system, minimum head to be maintained ; increased head due to pumping.
9.	Provisions for fire fighting	General locations of hydrants ; their numbers per km length of pipe ; their types ; the pressure available at the hydrants ; the size and number of fire streams to be used ; etc.
10.	Cost expenditures	The first cost of installing various units of the waterworks ; the RMO costs ; the methods of raising funds ; the revenue to be recovered and rates of water charges.
11.	Economics of the scheme	Cost of the entire scheme per capita and per kilolitre of treated water ; justification of unit costs ; further measures required if any, to economise the scheme.
12.	Engineering economy of the project	Cost-benefit considerations ; cost-benefit ratio and other considerations of optimisation in planning.
13.	Summary and conclusions	Review of the whole scheme in brief with reference to overall expected performance and achievements in relation to the aims and goals.

PROCESS DESIGN OF A COMPLETE WATER TREATMENT PLANT PROJECT AS A WHOLE

Example 13.1. *A water treatment plant is proposed to be constructed at 'Rasso' to supply water to a town 'Awahdevi' and its nearby villages in Himachal Pradesh State of India, having the designed daily water demand for the year 2017 to be 8 MLD. The corresponding projected water demand for the year 2002 i.e. when the present project starts operating, is 5.9 MLD.*

The water for supply is proposed to be lifted from River Beas, where jack well intake with provision to fix pump sets of required capacity, have been constructed.

The river water has been found to be highly turbid with turbidity as high as 4500 NTU. Inorder to remove the large sized and rapidly settleable solids to reduce turbidity, it is proposed to construct a pre-sedimentation tank and a ***raw water storage tank with sump*** *of $\frac{1}{2}$ hr capacity near the intake. From here, the water is proposed to be lifted in* ***the 1st stage pump house*** *and taken through the* ***1st stage rising main*** *to the* ***main raw water tank at Bheri.*** *The pumping of water is to be done for 16 hours a day. The main Bheri raw water tank will store this pumped water to supply it to the treatment plant by gravity as to maintain continuous flow in the water-treatment plant for 24 hours, as shown in Fig. 13.2.*

The proposed water treatment plant is planned to be designed to have the following units :

(i) ***Cascade type aerator.*** *Since the raw water does not contain too much colour and odour, only nominal aeration is proposed by constructing an artistic cascade type aerator. This unit will help in maintaining the wanted oxygen levels in water, remove dissolved iron and manganese, remove CO_2 and H_2S gases as well as the colour and tastes caused by volatile oils etc. released by algae and other micro-organisms. The raw water reaching the plant will be pumped into the aerator tank through a 450 mm dia pipe and will outflow through 4 No. 250 mm dia pipes.*

(ii) ***Alum dosers and Wash water tank.*** *Alum doser is the appurtenance through which measured quantity of alum is added to the water supply system before sedimentation of water. The requisite alum dose shall be mixed in two polyethylene tanks of 3000 L each, which shall be inter-connected. The raw water shall first be added in one tank where the alum dose of the day shall be mixed manually. The alum mixed solution shall flow into the other tank, where a float control will be provided to ensure constant depth of water. The water mixed with alum from the second tank shall flow under constant head to the* ***alum mixing chamber*** *to be provided in the aerator.*

When once the system becomes operational and clear water starts coming from the filtration plant to the clear water reservoir, a 8 H.P. motor shall be operated to fill the ***wash water tank*** *of 2.5 lakh litre capacity located at higher altitude. A pipe connection is proposed to be given from this wash water tank to the first tank of the alum dosing equipment. This arrangement is proposed to avoid any significant co-augulation in the alum dosing tank itself before the mixture enters the sedimentation cum coagulation tank. The raw water will*

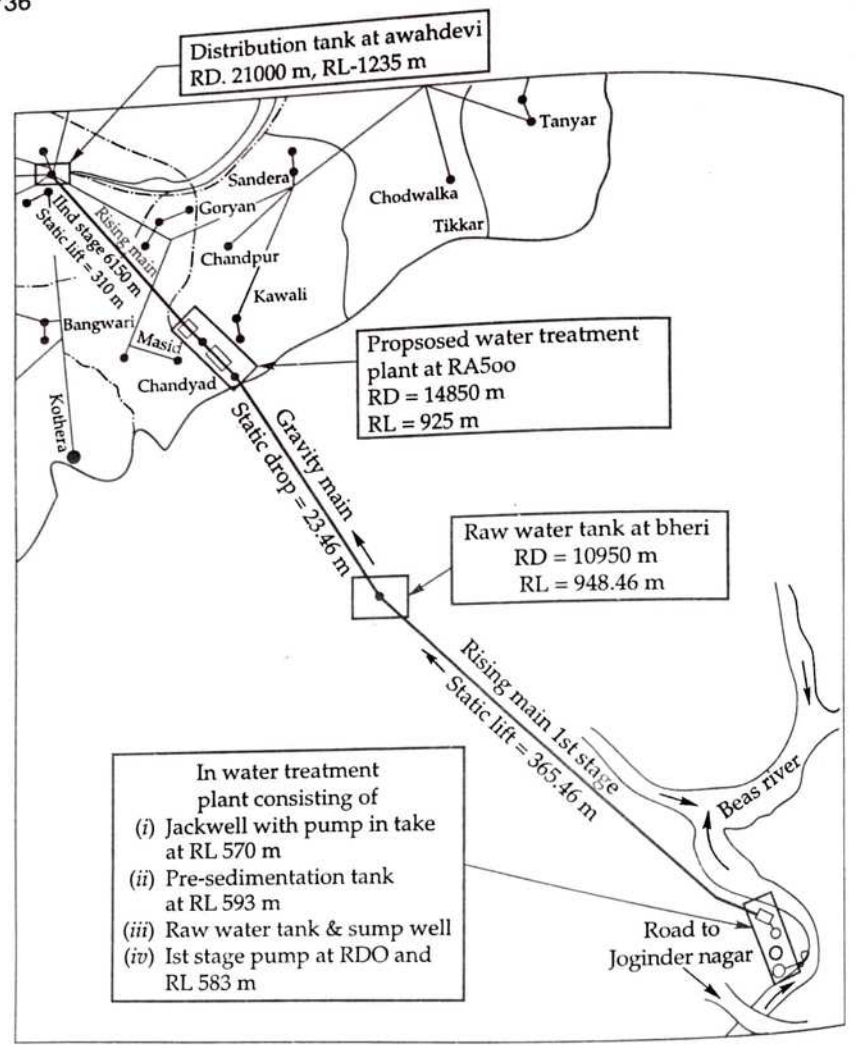


Fig. 13.2. General layout of the proposed water supply scheme.

therefore be used in Alum doser tank only in the first week of the operation of the water treatment plant. After one week, the filtered water from wash water tank shall only be used.

(iii) ***Combined Coagulation cum Sedimentation Tank.*** *It shall be constructed to allow formation of flocs and settlement of particles.*

(iv) ***Rapid Gravity Filter.*** *Filter units shall be constructed to filter the sedimented water, as usual, with provision of wash water tank as stated in (ii) above.*

(v) ***Chlorinator.*** *Considering the remote area and difficulty in transporting and storing the chlorine gas cylinders, it has been decided to use bleaching powder for disinfection by providing gravity type of chlorinator. Bleaching powder mixed clear water will be fed by gravity into clear water tank where contact period shall be more than $\frac{1}{2}$ hr, which is the acceptable limit.*

(vi) ***Clear Water Reservoir.*** *It shall be constructed to store clear treated water, from where it will be lifted by 2nd stage pump house to deliver it to the main distribution reservoir at Awahdevi. This 2nd stage* ***pumping*** *is also to be done for 16 hrs a day.*

(vii) ***Main Distribution Tank at Awahdevi.*** *The main distribution reservoir will obtain pumped water from 2nd stage pump house and deliver it by gravity to service reservoirs of towns and villages for 18 hours per day.*

The schematic layout (block diagram) of the various treatment units is shown in Fig. 13.3.

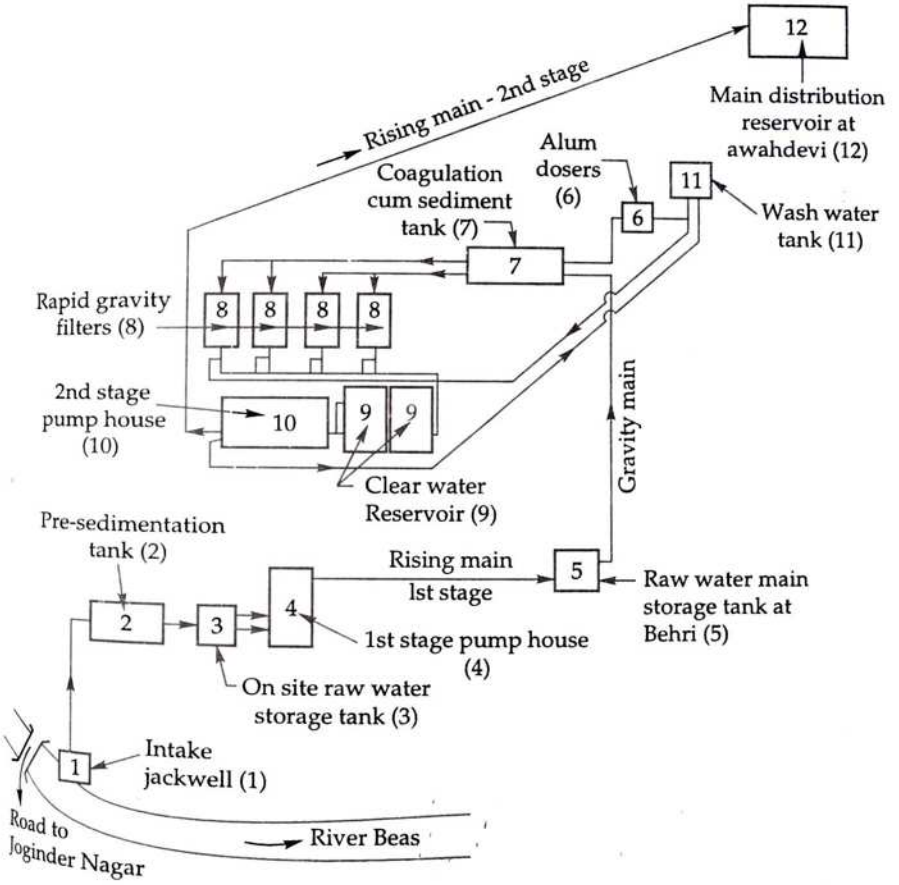


Fig. 13.3. Block diagram showing schematic layout of the various units of the proposed water supply project.

You have to carry out the hydraulic design (process design) of the above stated units of the water treatment plant, the layout of which is shown in **Chart Fig. 13.4.**

You have also to suggest the size of the pre-sedimentation tank and that of the main raw water storage tank at Bheri. Also design the 1st stage and 2nd stage rising mains and their pumping requirements, along with designing the gravity main between Bheri raw water storage tank and the Water Treatment Plant (W.T.P.).

Solution. The hydraulic design of the various units is carried out below by following the general principles explained in the Text. We shall start our design from pre-sedimentation tank.

(1) **Design of Pre-sedimentation Tank.** We shall design the tank to remove particles upto 0.1 mm size, and adopt the following general parameters/provision3.

General Parameters

1. Overflow rate	= 20 to 80 $m^3/m^2/day$
2. Minimum side water depth	= 2.5 m
3. Detention time	= 0.5 to 3 hrs
4. Side slopes (for non mechanical cleaning)	= 10% from sides towards center line
5. Longitudinal slope	= 1% in rectangular tank
6. Ratio of length and breadth	= 3 : 1 to 5 : 1
7. Settling velocity	= To ensure removal of minimum size of particles of 0.1 mm.

Hydraulic Design :

(i) Dimensions of tank

Water required by the year 2017

$$= 8.00 \text{ MLD} = \frac{8.0 \times 10^3}{24} = 333.33 \text{ m}^3/\text{hr}$$

Since pumping is for 16 hrs., the discharge coming into the pre-sedimentation tank

$$= \frac{333.33 \times 24}{16} = 500 \text{ m}^3/\text{hr}$$

Water loss in desludging = 2%

Design average flow

$$= \frac{500 \times 100}{100 - 2} = 510.205 \text{ m}^3/\text{hr}$$

Assume detention period = 1.5 hrs

Effective storage of sedimentation tank

$$= 510.205 \times 1.5 = 765.31 \text{ m}^3$$

Assuming effective depth = 3.0 m

Area of tank required $= \frac{765.31}{3.0} = 255.10 \text{ m}^2$

Assume $L : B$ = 3 : 1

$$= \frac{0.142}{9.3 \times 3} = \mathbf{0.0051 \text{ m/sec}} = 0.310 \text{ m/min}$$

Settling velocity $v_s = \frac{v \times H}{L}$

Assuming ...

Area of tank required $= \frac{765.31}{3.0} = 255.10 \text{ m}^2$

Assume $L : B = 3 : 1$

$3B \times B = 255.10$

$B = 9.22$ m ; say 9.3 m

$L = 3 \times 9.22 = 27.66$ m ; say 27.8 m

Provide a tank of size 27.8 m × 9.3 m × 3.0 m (water depth)

Provide a free-board of 0.5 m

Hence, use **a tank of size 27.8 m × 9.3 m × 3.5 m.**

Loading on the tank/overflow rate

$$\frac{Q}{A} = \frac{510.205 \times 24}{27.8 \times 9.30}$$

$= 47.36 \text{ m}^3/\text{m}^2/\text{day}$ (in the range of 2 to 80 $\text{m}^3/\text{m}^2/\text{day}$, O.K.)

(ii) Settlement Velocity (v_s) by the method of Stoke's Law

Here

(*a*) Size of particles (d) $= 0.10 \text{ mm} = 0.1 \times 10^{-3} \text{ m}$

(*b*) Specific gravity (G) = 2.65

(*c*) Kinematic Viscosity at 20°C

$\nu = 1.01 \times 10^{-6} \text{ m}^2/\text{sec}$

(*d*) Value of g $= 9.81 \text{ m/sec}^2$

$$\therefore \quad v_s = \frac{g}{18} (G_s - 1) \frac{d^2}{\nu}$$

$$= \frac{9.81}{18} \frac{(2.65 - 1) \times (0.1 \times 10^{-3})^2}{1.01 \times 10^{-6}}$$

$= 0.0089 \text{ m/s} = \mathbf{8.9 \times 10^{-3} \text{ m/s}}$

Reynolds Number, $R_e = \frac{v_s d}{\nu}$

$$= \frac{8.90 \times 10^{-3} \times 0.1 \times 10^{-3}}{1.01 \times 10^{-6}} = 0.88 < 1$$

Hence, the flow is laminar and Stoke's Law is applicable.

(iii) Settlement Velocity (v_s) by the method of Discharge entering the tank and Tank Dimensions

(*a*) Discharge entering the basin (Q)

$= 510.205 \text{ m}^3/\text{hr} = 0.142 \text{ m}^3/\text{sec}$

(*b*) Depth of water in tank $H = 3.0$ m

(*c*) Width of basin $B = 9.30$ m

(*d*) Length of the tank $L = 27.80$ m

Horizontal flow velocity is given as :

$$v = \frac{Q}{BH}$$

$$= \frac{0.142}{9.3 \times 3} = \mathbf{0.0051 \text{ m/sec}} = 0.310 \text{ m/min}$$

Settling velocity $v_s = \frac{v \times H}{L}$

$$= \frac{0.130 \times 3}{27.80} = 0.033 \text{ m/min} = 5.5 \times 10^{-4} \text{ m/sec}$$

Thus, particles having settling velocity greater than or equal to 5.5×10^{-4} m/sec shall settle in the tank of dimensions assumed above.

(iv) Time period required for particles to settle in the tank

As given in para *(ii)* above, a particle of size 0.1 mm shall settle in the tank if its settling velocity is greater than equal to 8.9×10^{-3} m/sec

Since we have assumed the water depth = 3.00 m

Time required to settle down this depth

$$= t_1 = \frac{3.0}{8.9 \times 10^{-3}} \text{ secs}$$

$$= 337.078 \text{ sec} = 0.0936 \text{ hr.}$$

Since the horizontal velocity of flow is 0.0051 m/s and the length of the tank is 27.80 m, the time taken by water to reach the outlet t_2 is given as :

$$t_2 = \frac{27.80}{0.0051} = 5450.98 \text{ sec} = 1.51 \text{ hrs}$$

Thus, the time required by the water to flow from the inlet to the outlet is more than the time required by the particle size 0.1 mm to settle down. Hence all the particles of size 0.1 mm having a settling velocity greater than or equal to 8.9×10^{-3} m/sec shall be retained in the sedimentation tank.

Note. *Longitudinal slope of 1 in 50 is being provided ; and side slope of 1 in 30 is being provided towards the longitudinal central line.*

Actual detention time in the sedimentation tank

$$= \frac{27.80 \times 9.30 \times 3}{510.205} = 1.52 \text{ hrs.}$$

(v) Check Against Scour of deposited particles

Scour velocity (v_d)

$$v_d = \sqrt{\frac{8\beta}{f} g\,(G-1)d}$$

where $\beta = 0.04$ (for unigranular particles)

$f = 0.03$ (Darcy-Weisbach friction factor)

G = Sp. Gravity of particles = 2.65

g = Acceleration due to gravity = 9.81 m/sec^2

d = Size of the settling particles = 0.1 mm

$= 0.1 \times 10^{-3}$ m.

$$\therefore \quad v_d = \sqrt{\frac{8 \times 0.04 \times 9.81 \times (2.65-1) \times 0.1 \times 10^{-3}}{0.03}} = 0.13 \text{ m/sec}$$

Thus to avoid scour, the flow velocity should not exceed v_d. Here the horizontal flow velocity (v) is 0.0051 m/sec, which is less than the scour velocity (v_d) ; and hence the design is O.K.

The details of the above designed pre-sedimentation tank are shown in Fig. 13.5.

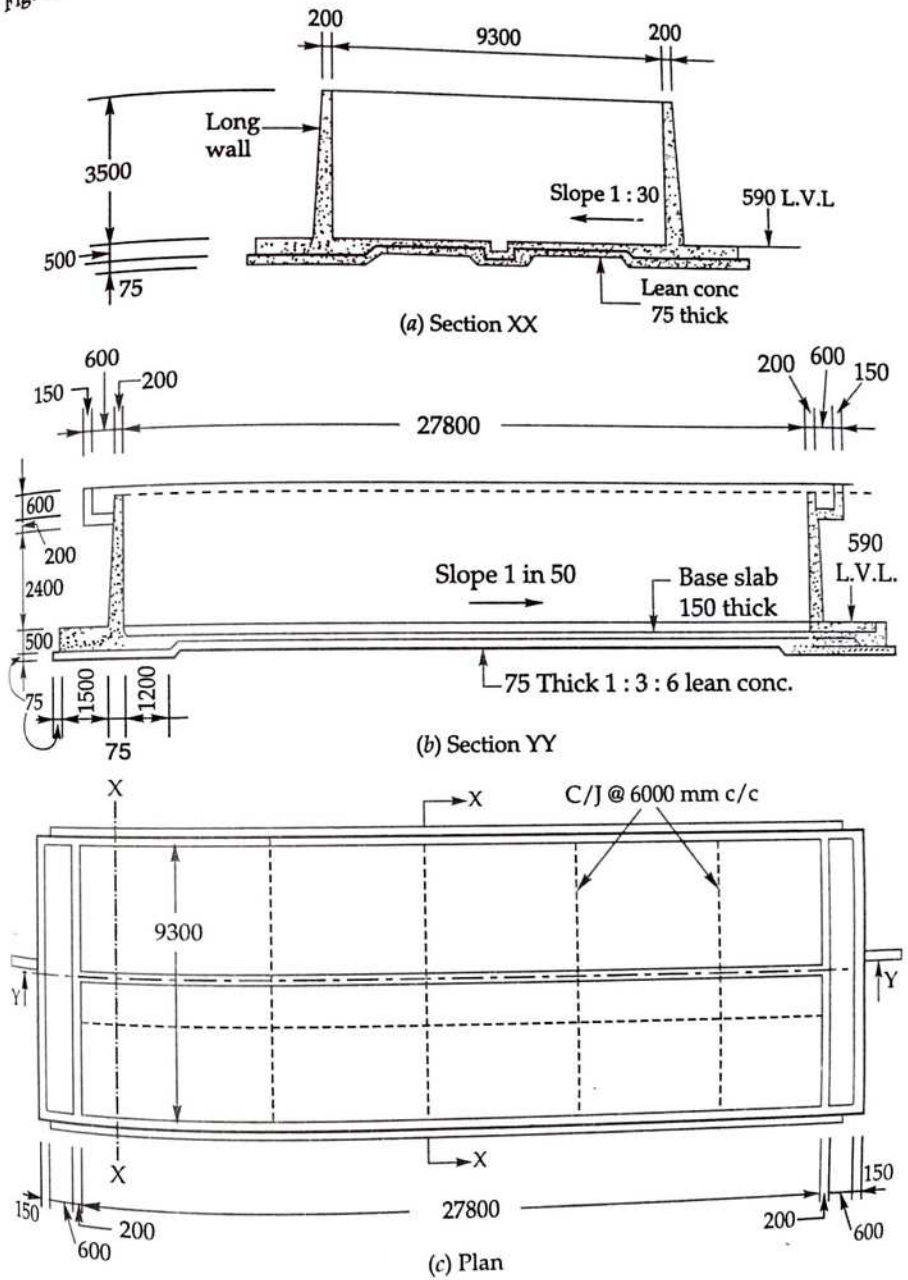


Fig. 13.5. Details of pre-sedimentation tank.

(2) Design of Raw Water Tank at Bheri

A raw water storage tank of 8.0 hrs detention period is proposed to be provided so that the water coming from the rising main at the rate

1.5 × Average discharge due to 16 hrs pumping can be stored in it for 8.0 hrs. The outflows from the Bheri Tank will be at the rate of average discharge and the water stored in the tank shall maintain the continuous flow in the filtration plant for 24 hrs.

Water requirement for the year 2017 = 8.00 MLD

Taking 8.05 hrs detention period,

capacity of tank = (8.00 × 8.0)/24 = 2.67 ML = 2670 m^3

Provide water depth = 3.0 m

Plan area of tank required = 2670/3 = 890 m^2

Assume a ratio of $L : B$ = 2.5 : 1

$\therefore$ $2.5\,B \times B = 890$; or $B = 20$ m and $L = 50$ m

Provide a tank of size 50 m × 20 m × 3.0 m (water depth); Provide a free-board = 0.5 m

Hence, provide **a tank of size 50 m × 20 m × 3.5 m.** Details are shown in Fig. 13.6.

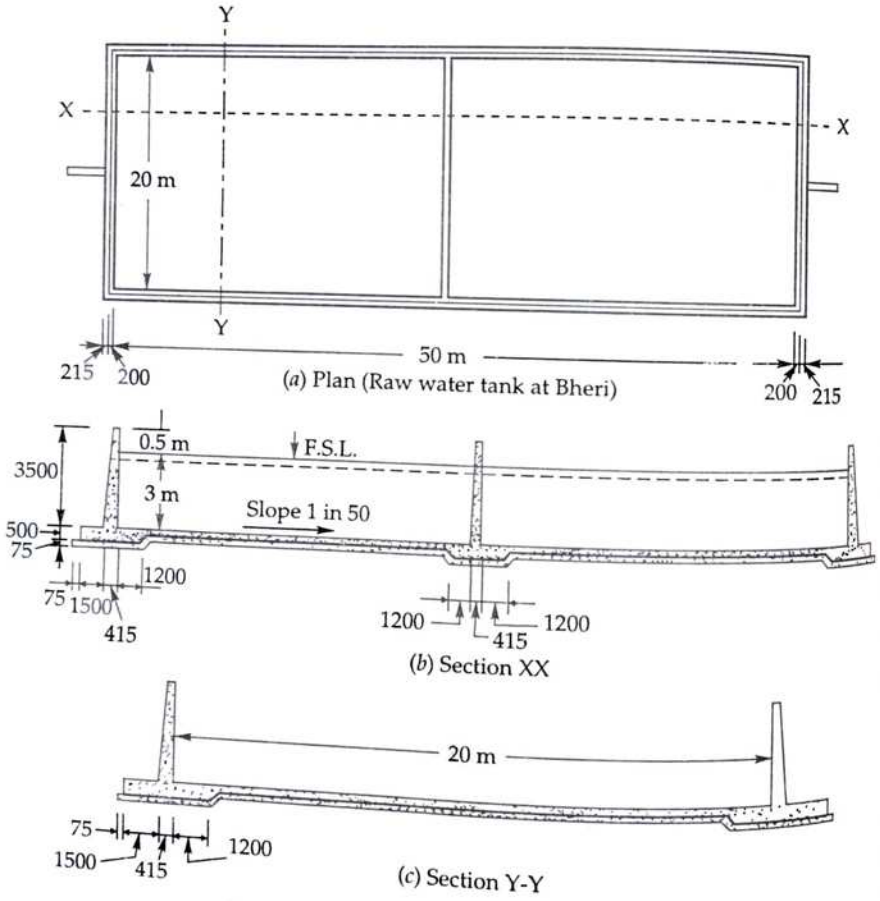


Fig. 13.6. Details of Raw Water Tank at Bheri.

(3) Design of Cascade Aerator

Water requirement for the year 2017 = 8 MLD

$\therefore$ Average water requirement (discharge)

$= \frac{8 \times 10^6}{24 \times 60 \times 60} = 0.093\ m^3/s$

1.5 × A ... discharge due to 16 hrs pumping can be stored in it for 8.0 hrs.
The ou ... will be at the rate of average discharge and
the w ... continuous flow in the
filtrat

W:

Ta

P

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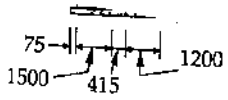


(c) Section Y-Y

Fig. 13.6. Details of Raw Water Tank at Bheri.

(3) Design of Cascade Aerator

Water requirement for the year 2017 = 8 MLD

∴ Average water requirement (discharge)

$$Q = \frac{8 \times 10^6}{24 \times 3600 \times 1000} \text{ m}^3/\text{s} = 0.092 \text{ m}^3/\text{s}.$$

Width of the weir proposed = 4.0 m

For Broad crested weir, $Q = 1.65\ B\ .\ H^{3/2}$

$$\therefore \quad H = \left(\frac{Q}{1.65\ B}\right)^{2/3}$$

$$= \left(\frac{0.092}{1.65 \times 4}\right)^{2/3} = 0.026 \text{ m ; say 3 cm.}$$

So a total depth of 3 cm shall be flowing over the weir.

Steps. **Six steps** of 0.30 m size shall be provided, on which the water shall be flowing down to the outlet chamber.

At the outlet chamber one side wall has been kept 200 mm below the top surface of the tank so as to prevent the excess flow of water towards alum mixing chamber.

Four pipes of 250 mm dia have been provided at the outlet of Aerator to the Alum mixing chamber. Enough discharge of water will flow through these pipes. The over-flow shall take place only when the outlet of alum mixing chamber are clogged. All the masonry walls shall be plastered. The wall thickness has been proposed to be kept as 0.30 m.

The details are shown in **Chart Fig. 13.7.**

Alum dose. The dose of alum varies from 15 to 20 mg/L of water but actual dose is to be ascertained by experiments in the laboratory from time to time. The optimum coagulation occurs when pH value of water is between 6 to 8. Since Alum makes the water acidic, the alum dosing tank will be lined with rubber or preferably polyethylene tanks shall be used. Here, we shall use 2 such tanks each of 3000 L capacity.

Now, assuming the peak alum dose to be 20 mg/L and normal dose to be 20% of that, we get

$$\text{Alum required @ 20 mg/L} = \frac{20}{1000} \times 8.00 \times 10^6 \times \frac{1}{1000}$$

$$= 160 \text{ kg/d}$$

Only 20% of 160 kg shall be used in normal conditions. Thus, for most of the year, the alum to be used will be 32 kg/day.

(4) Design of Sedimentation Tank

General Provisions

Overflow rate = 15 to 30 $m^3/d/m^2$

Minimum side water depth = 2.5 m

Detention Period for coagulated water = 2.0 to 4.0 hours

Weir loading= 300 $m^3/d/m^2$

Side slopes for non mechanical cleaning
= 10% from sides towards the longitudinal central line

Longitudinal slope = 1% in case of rectangular tank

Ratio of length and width = 3 : 1 to 5 : 1

Settling velocity = To ensure removal of minimum size of particle of 0.02 mm

Detention time to flocculation chamber = 10 to 30 minutes

Skimming weir = To cheek the froth on the surface of outlet to reduce load on the filter

Horizontal flow velocity = 0.15 to 0.9 m/min

Hydraulic Design of Proposed Sedimentation cum Flocculation Tank

(i) Dimensions of the Tank

Water required by the year 2017

$= 8.00 \text{ MLD} = 333.33 \text{ m}^3/\text{hr}$

Water lost in desludging

$= 2\%$

Design average flow

$$= \frac{333.33 \times 100}{(100 - 2)} = 340.13 \text{ m}^3/\text{hr}$$

Assume a total detention period of 2.6 hrs

Effective storage of sedimentation tank

$= 340.13 \times 2.6 = 850.325 \text{ m}^3$

Assume effective depth H

$= 3.0$ m

Area of the tank required

$$A = \frac{850.325}{3} = 283.44 \text{ m}^2$$

Assume $L/B = 3 : 1$

Thus $3B \times B = 283.44$ m

or $B = 9.7$ m ; say 9.8 m

and $L = 29.16$ m ; say 29.2 m

Provide a tank of size 29.2 m × 9.8 m × 3.0 m (water depth)

Provide a free board of 0.5 m

Loading on the tank/overflow rate

$= Q/\text{Area}$

$$= \frac{340.13 \times 24}{29.2 \times 9.8} = 28.52 \text{ m}^3/\text{m}^2/\text{day}$$ (which is within the prescribed range of 15 to 30 $\text{m}^3/\text{m}^2/\text{day}$, O.K.)

Hence, use **a tank of size 29.20 m × 9.8 m × 3.5 m**. Provide extra depth for sludge storage.

Provide longitudinal slope of 1 in 50. Also provide side slope of 1 in 30 towards the longitudinal central line.

(ii) Settlement Velocity (v_s) by the method of Stoke's Law

The following parameters have been adopted.

Size of particle (d) = 0.02 mm = 0.02×10^{-3} m

Specific gravity of particles (G) = 2.65

Kinematic viscosity of particles at 20°C = $\nu = 1.01 \times 10^{-6}$ m^2/s

Using $v_s = \frac{g}{18}(G - 1)\frac{d^2}{\nu}$, we get

$$v_s = \frac{9.81}{18}(2.65 - 1)\frac{(0.02 \times 10^{-3})^2}{1.01 \times 10^{-6}}$$

$$= \mathbf{3.56 \times 10^{-4} \text{ m/s}}$$

Reynolds Number

$$= \frac{v_s \times d}{\nu} = \frac{(3.56 \times 10^{-4})(0.02 \times 10^{-3})}{1.1 \times 10^{-6}}$$

$$= 7.04 \times 10^{-2} < 1$$

Hence, the flow is laminar and Stoke's Law is applicable.

(iii) Settlement Velocity (V_s) by the method of Discharge entering the tank and Tank dimensions

Parameters adopted

Discharge entering the basin $Q = 340.13 \text{ m}^3/\text{sec} = 0.094 \text{ m}^3/\text{hr}$

Depth of water in the tank, $H = 3.0$ m

Width of tank, $B = 9.8$ m

Length of tank, $L = 29.2$ m

Horizontal flow velocity

$$v = \frac{Q}{BH} = \frac{0.094}{9.8 \times 3} = 0.0032 \text{ m/sec} = 0.19 \text{ m/min}$$

Settling velocity

$$v_s = \frac{v \times H}{L} = \frac{0.0032 \times 3}{29.20}$$

$$= 0.000328 \text{ m/sec} = 3.28 \times 10^{-4} \text{ m/sec}$$

Thus, the particles having settling velocity greater than or equal to 3.28×10^{-4} m/sec shall settle in the tank of the dimensions assumed above.

(iv) Time period required for particles to settle in the tank

As given in para *(ii)* above, a particle of the size of 0.02 mm shall settle in the tank if its settling velocity is greater than or equal to 3.56×10^{-4} m/s. Since we have assumed the water depth as 3.0 m, the time required by the particles to settle down is equal to

$$t_1 = \frac{3.0}{3.56 \times 10^{-4}} = 8426.965 \text{ sec} = 2.34 \text{ hrs.}$$

Now the horizontal velocity of flow is 0.0032 m/sec and the length of tank is 29.2 m, the time taken by water to reach the outlet :

$$t_2 = \frac{29.20}{0.0032} = 9125 \text{ sec} = 2.53 \text{ hrs} < t_1$$

Thus, the time required by water to flow from the inlet to the outlet is more than the time required by the particle size 0.02 mm to settle down. Hence, all the particles of size 0.02 mm having a settling velocity greater than or equal to 3.56×10^{-4} m/s shall be retained in the Sedimentation Tank.

Actual detention time in the Sedimentation Tank

Tank size $= 29.2 \times 9.8 \times 3 = 858.48 \text{ m}^3$

Discharge $= 0.094 \text{ m}^3/\text{sec}$

Detention time $= \dfrac{858.48}{0.094 \times 3600} = 2.54 \text{ hrs} > 2.5 \text{ hrs.}$

Check Against Scouring of deposited particles

Scour velocity (v_d) is given as :

$$= v_d = \sqrt{\frac{8\beta}{f'} g(G-1)d}$$

Assuming $\beta = 0.04$ (for unigranular particle)

$f' = 0.03$ (Darcy-Weisbach friction factor)

G = Sp. gr. of particles = 2.65

g = Acceleration due to gravity = 9.81 m/sec^2

d = Size of the settling particles = 0.02 mm

$$\therefore \quad v_d = \left[\frac{8 \times 0.04 \times 9.81 (2.65-1) \times 0.02 \times 10^{-3}}{0.03}\right]^{1/2} = 0.058 \text{ m/sec.}$$

Since the horizontal flow velocity (v) of 0.0032 m/s is less than the displacement velocity of 0.058 m/s, the design is O.K.

Design of Floc Chamber

In addition to 29.2 m length of settling tank, the floc chamber at the entry has to be provided. Assume the depth of the floc chamber as half the depth of the sedimentation tank *i.e.* 3.5/2 = 1.75 m ; say 1.8 m.

Detention period = 20 minutes

Capacity of the chamber = flow required in 20 minutes

$= 0.094 \times 20 \times 60 = 112.8 \text{ m}^3$

Width of the chamber = 9.8 m

Length of the chamber $= \dfrac{112.8}{9.8 \times 1.8} = 6.4 \text{ m.}$

Thus, the *floc chamber* shall have the effective dimensions of 6.4 m × 9.8 m × 1.8 m.

Design of Inlet-Outlet

Inlet structure

The inlet structure is designed to minimize turbulence ; distribute the water and suspended solids throughout the depth of basin ; and avoid the deposition of solids.

Provide a 0.6 m wide and 0.6 m deep influent channel that runs across the width of the tank. The design that follows provides overflow weir in the inlet structure for whole width of the tank. The depth of flow over the weir on full discharge shall be :

$$Q = 1.65\ B\ (H)^{3/2} \text{ (for broad crested weirs)}$$

where $B = 9.8$ m

$Q = 0.094$ m^3/sec

$$\therefore \quad H = \left[\frac{0.094}{1.65 \times 9.8}\right]^{2/3} = 0.033 \text{ m}$$

A free-board of 0.5 mm has been provided at the inlet.

Velocity of flow over the weir

$$Q = v \times A$$

$$v = \frac{0.094}{0.033 \times 9.8} = 0.29 \text{ m/sec.}$$

Outlet structure

The outlet structure shall consist of skimming weir plate, outlet box, outlet pipe and one overflow pipe to provide overflow when water to pass through the filter is reduced. The concrete weir on the outlet shall be 500 mm below the inlet weir crest. A skimming weir consisting of 8 mm thick and 10 mm depth steel plate with 90° *V*-Notches having depth of 5 cm is provided along the total width of the tank.

There are 4 outlet pipes of 250 mm dia which take the water to the filter tank. One overflow pipe of 300 mm dia has been provided on the side of the outlet box to prevent surplus water from flowing towards the filter. The details are shown in Fig. 13.8.

(5) Design of Rapid Gravity Filters

General provisions

(*a*) Required flow of filtered water = 8.00 MLD

(*b*) Quantity of Back wash water used = 3% of filter output

(*c*) Time lost during back washing = 30 minutes

(*d*) Design rate of filtration = 5.4 m^3/m^2/hour

(*e*) Length width ratio = 1.25 to 1.33 : 1

(*f*) Under drainage system = central manifold

(*g*) Size of perforations = 9 mm

Design :

Filtered water required = 8.00 MLD

Filtered water required per hour $= \dfrac{8.00 \times 10^6}{24 \times 1000} = 333.33 \text{ m}^3$

Design flow for filter after accounting for backwash water (3%) and washing time ($\frac{1}{2}$ hr)

$$= \frac{333.33 \times (1 + 0.03) \times 24}{23.5} = 350.63 \text{ m}^3/\text{hour}$$

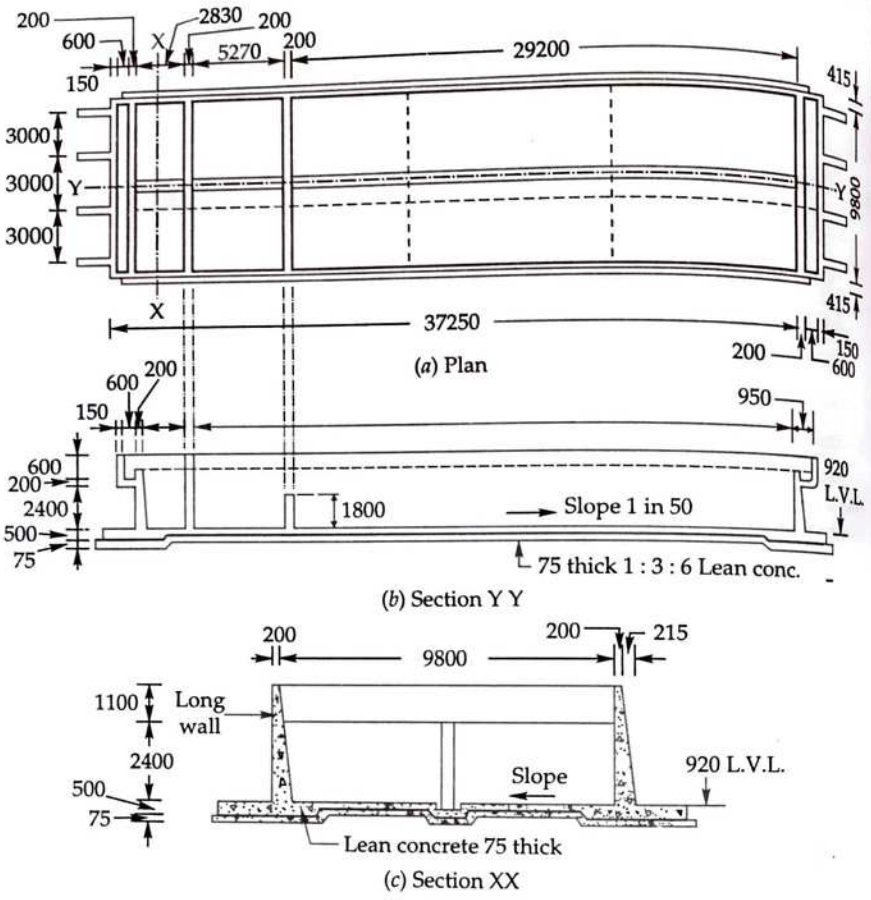


Fig. 13.8. Coagulation (flocculation) cum sedimentation tank.

Plan area of filter required

$$= \frac{350.63}{5.4} = 64.9 \text{ m}^2.$$

Generally, the plan area of the filter unit is kept as 10 m^2 to 80 m^2.

Provide two filter units each of 33 m^2.

Provide 2 units, each 6.6 m × 5 m size

Design of sand depth

Assume a depth of sand as 60 cm and an effective size of sand as 0.6 mm. The depth can be checked against break-through of flock, through the sand bed, by calculating minimum depth required by **Hudson's formula,** given as :

$$\frac{Qd^3h}{L} = B \times 29323 \qquad ...(13.1)$$

Here $Q = 5.4 \times 2 = 10.8$ m^3/m^2/hour
(assuming 100% overloading of filter under emergencies)
d = size of sand in mm = 0.6 mm
h = Terminal head loss in metres = 2.25 m
L = depth of bed in m

Assume $B = 4 \times 10^{-4}$ per poor response to filtration and average degree of pre treatment.

$$\therefore \quad \frac{10.8 \times (0.6)^3 \times 2.25}{L} = (4 \times 10^{-4})\ 29323$$

or $$L = 0.45 \text{ m}$$

Hence depth assumed is O.K.

Estimate of gravel and size gradation

Assume a size gradation of 2 mm at top and to 50 mm at bottom. The requisite depth in cm of a component gravel layer of size d in mm can be computed from Empirical Formula.

$$\boxed{L = 2.54K (\log d)} \qquad ...(13.2)$$

where K varies from 10 to 14

Let us assume $K = 12$

The depth of various layers of gravel are

Size, mm	=	2	5	10	20	40
Depth, cm	=	9.2	21.3	30.5	40	49
Increment, cm	=	9.2	12.1	9.2	9.5	9

Provide a gravel depth of 50 cm.

Design of under Drainage system

Plan area of each filter unit = 6.65 × 5.0 = 33.25 m^2

Total area of perforation = 3×10^{-3} × Area of filter
= 0.0997 m^2 = 997 cm^2

Total number of perforations of dia 9 mm

$$= \frac{997}{1/4 \times (0.9)^2} = 1567.18 \text{ Nos ; say 1569 Nos.}$$

Total cross sectional area of laterals
= 3 × Area of perforations = 3 × 997 = 2991 cm^2

Area of central manifold
= 2 × Area of laterals = 2 × 2991 = 5982 cm^2

Diameter of central manifold
$= (5982 \times 4/\pi)^{1/2} = 87.27$ cm

Provide a commercially available diameter of 900 mm.

Assume a spacing of 15 cm for laterals.

No. of laterals on both sides of manifold

$= 2 \times 6.65 \times 100/15 = 88.67 =$ say **89**

Cross sectional area of each lateral

$$= \left(\frac{2991}{89}\right)^{1/2} = 33.61 \text{ cm}^2$$

Dia of lateral $= [33.61 \times 4/\pi]^{1/2} = 6.54$ cm

Provide laterals of diameter **80 mm**. No. of perforations per lateral

$$= \frac{1568}{80} = 19.6 \text{ ; say } 20$$

Length of lateral

$= \frac{1}{2}$ [width of filter – dia of manifold]

$= \frac{1}{2}$ [5 – 0.9] = 2.05 m

$$\text{Spacing of perforations} = \frac{2.05 \times 100}{20} = 10.25 \text{ cm}$$

provide 20 perforations of 9 mm dia at c/c spacing of 102.5 mm.

Check

$$\frac{\text{Length of each lateral}}{\text{Dia. of lateral}} = \frac{2.05 \times 100}{80}$$

$= 25.63$ (which is less than 60)

Now let us assume that the rate of washing of filter be 45 cm/min or 0.45 m/min.

$$\text{The wash water discharge} = 0.45 \times \frac{(6.65 \times 5)}{60} = \mathbf{0.25} \text{ m}^3\text{/sec}$$

Velocity of flow in lateral for wash water

$$= \frac{0.25}{89\left[\frac{\pi}{4} \times \frac{(8.0)^2}{100}\right]} = 0.62 \text{ m/sec}$$

Similarly, velocity of flow in manifold

$$= \frac{\text{Discharge}}{\text{Area}} = \frac{0.25 \times 4}{\pi \times 0.9^2} = 0.39 \text{ m/sec}$$

which is less than 1.8 to 2.4 m/sec (maximum permissible)

The filter details are given in **Chart Fig. 13.9.**

Design of wash water troughs

Assume a wash water rate of 36 $m^3/m^2/hr$

Wash water discharge for one filter

$= 36 \times 6.65 \times 5 = 1197 \text{ m}^3$ per hr

$= 0.33 \text{ m}^3\text{/sec}$

Assume a spacing of 1.5 m for wash water troughs which will be parallel to the longer dimension of the filter unit.

$$\text{Number of troughs} = \frac{5.0}{1.5} = 3.3 \text{ ; say } 4$$

$$\text{Discharge per unit trough} = \frac{0.33}{4} = 0.0842 \text{ m}^3\text{/sec}$$

For a width of 0.4 m, the water depth in the trough is given as :

$Q = 1.376\ by^{3/2}$ (*i.e.* Eq. 9.21)

or $0.0824 = 1.376 \times 0.4 \times y^{3/2}$

or $y = 0.281$ m ; say 0.31 m

Assume a free-board of 0.1 m, provide a depth of 0.40 m.

Provide **4 troughs of 0.4 m width × 0.4 m depth in each filter.**

Bottom of trough from sand bed = 0.05 m.

Computation of total depth of filter box

Depth of filter box = Sum of depths for (*i*) Under drains ; (*ii*) Gravel ; (*iii*) Sand ; (*iv*) Water depth ; (*v*) Free board

$= 1.0 + 0.40 + 0.60 + 1.20 + 0.50 =$ **3.7 m.**

Design of wash water pump

Water required for washing = 500 L/min/m^2

Area of filter $= 6.65 \times 5.0 = 33.25$ sq. m.

$$\text{Water required} = \frac{500 \times 33.25}{60} = 277.08 \text{ L/sec}$$

Assuming that the filter shall be washed in 15 minutes, the size of the tank required for wash water shall be

$= 277.08 \times 60 \times 15 = 249375$ L

= say 2.5 lacs litres. At 10 m head difference from the filter.

The pump is designed to fill the wash water tank in 3 hrs.

The rate of pump output shall be

$$= \frac{2,50,000}{3 \times 60 \times 60} = 23.148 \text{ L/sec} \quad \text{or} \quad 0.023 \text{ m}^3\text{/sec}$$

Water power required

$= \gamma_w . Q . H = 9.81 \times 0.023 \times 20 \text{ kW} = 4.51 \text{ kW}$

Brake Horse Power required

$$= \frac{4.51}{0.8 \times 0.735} \text{ HP} = 7.67 \text{ HP ; say } 8 \text{ HP}$$

Size of wash water rising main

The rising main from the pump shall take the water from the clear water reservoir to the wash water tank, to fill the tank in 3 hours @ 0.023 m^3/sec.

Let, velocity in rising main = 2 m/sec and

Dia of rising main = d

$$Q = A \cdot v$$

$$\therefore \quad 0.023 = \frac{\pi}{4} \times d^2 \times 2$$

$$\Rightarrow \quad d = 0.12 \text{ m ; say 150 mm}$$

The actual velocity in the rising main

$$= \frac{0.023}{\frac{\pi}{4} \times (0.15)^2} = 1.30 \text{ m/sec}$$

Hence, a pipe of 150 mm dia shall be used as rising main.

Let D be the dia of pipe to supply wash water to filter at 0.25 m^3/sec on 2 m/sec velocity.

$$\therefore \quad Q = \frac{\pi}{4} \times D^2 \times 2$$

or $$0.25 = \frac{\pi}{4} \times D^2 \times 2$$

or $$D = 0.399 \text{ m ; say 400 mm.}$$

Thus, the rising main shall have a dia of 150 mm and the pipe to wash the filter shall have the dia of 400 mm.

Size of wash water tank

Capacity of the proposed tank = 2,50,000 L = 250 m^3

Assuming depth of water in the tank = 3.0 m

$$\text{Area of tank required} = \frac{250}{3} = 84 \text{ m}^2$$

Size of square tank provided = $(84)^{1/2}$ = 9.16 m ; say 9.2 m

Hence, *provide a square tank of 9.2 m × 9.2 m × 3 m (water depth).*

Provide a free-board of 0.5 m. Details of wash water tank are shown in Fig. 13.10.

(6) **Design of Chlorinator**

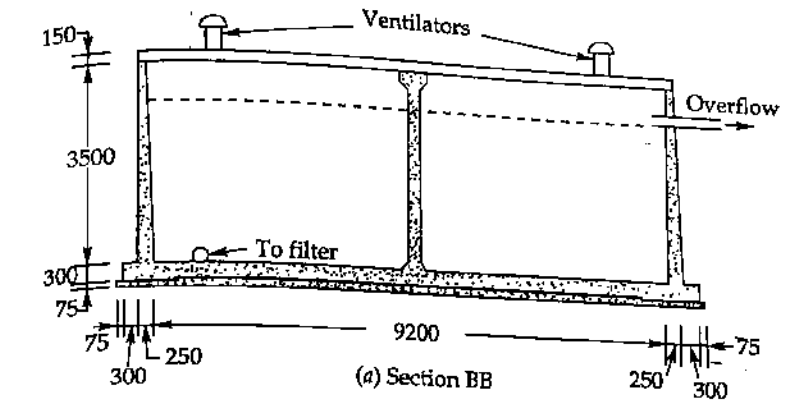
Two polyethylene tanks, each of 3000 L capacity, shall be installed to mix clear water with the requisite amount of bleaching powder, which water will flow by gravity into the clear water tank to have contact period of more than $\frac{1}{2}$ hr, as to cause disinfection. Qty of bleaching powder is worked out as below:

Normal dose of chlorine to be taken

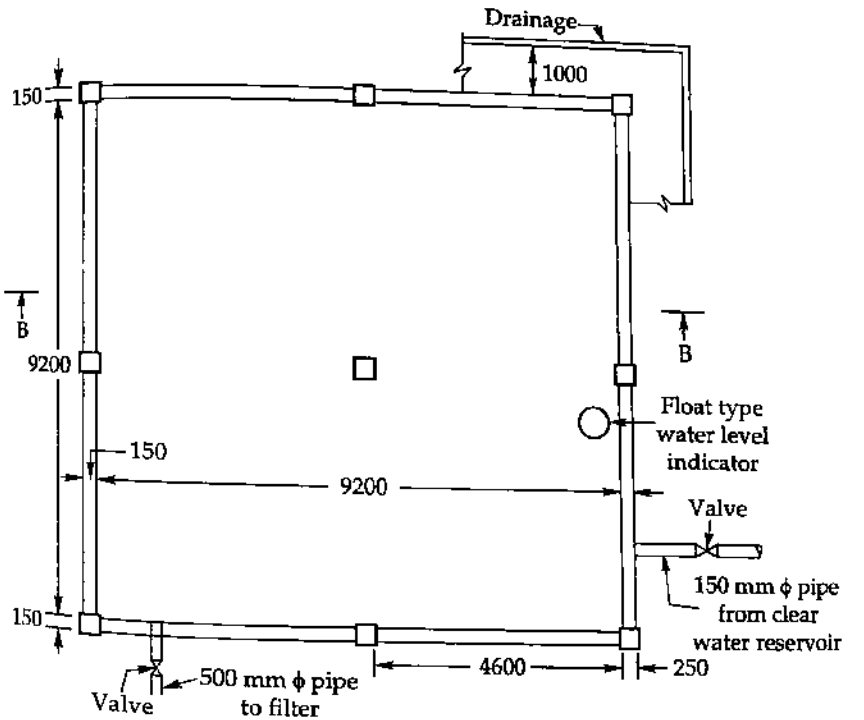
= 0.3 ppm for a contact period of 30 minutes

Average daily demand of water of the year 2017

= 8.00 MLD



(*a*) Section BB



(*b*) Plan

Fig. 13.10. Wash Water Tank Details.

Chlorine required per day

$$= \frac{0.3 \times 8.00 \times 10^6}{10^6} \text{ kg} = 2.4 \text{ kg ; say 2.5 kg}$$

Since chlorine content in bleaching powder is 30%, it means that 30 kg of chlorine is contained in 100 kg of bleaching powder.

Bleaching powder required per day

$$= (2.5 \times 100)/30 = 8.33 \text{ kg}$$

Annual consumption of bleaching powder

$= 8.33 \times 365 = 3041.67$ kg ; say 3 tonnes.

(7) Design of Clear Water Reservoir at W.T.P.

Treated water stored in clear water reservoir = 8.00 MLD

Since pumping from clear water reservoir to Awahdevi distribution tank is also for 16 hrs, detention period of 8 hrs is provided in clear water reservoir to ensure 24 hrs supply capacity of clear water reservoir

$$\therefore \text{ Capacity of clear water reservoir} = \frac{8.00 \times 8.0}{24} = 2.67 \text{ ML}$$

$$= 2670 \text{ m}^3$$

Assume water depth as 3.0 m

$$\text{Plan area required} = \frac{2670}{3} = 890 \text{ sq. m.}$$

Assume a ratio of $L : B$ of 2.5 : 1

$$\therefore \quad 890 = 2.5\, B \times B$$

or $\quad B = 18.86$ m ; say 20 m

$L = 47.17$ m ; say 50 m

Thus, provide a tank of size 50 m × 20 m × 3.0 m (water depth).

A free board of 0.5 m is also provided. Hence, provide **a covered tank of 50 m × 20 m × 3.5 m size of 8 hr detention period**. Details are shown in **Chart Fig. 13.11.**

(8) Design of Main Distribution Tank at Awahdevi

Water requirement for the year 2017

= 8.00 MLD

Since the water will come in the tank in 16 hrs pumping from clear water reservoir and it will go to the service reservoirs in 18 hrs, a storage of 2 hrs is required on the tank at Awahdevi.

A provision of 3 hrs storage is however being made in the design.*

$$\text{Capacity of tank required} = \frac{8.00 \times 3 \times 10^3}{24} = 1000 \text{ m}^3$$

**JUSTIFICATION FOR KEEPING 3 hrs. STORAGE CAPACITY AT AWAHDEVI*

The storage tank at Awahdevi is basically a distribution point at the highest level between the treatment plant at Rosso and the service reservoirs at various distribution points. The water being pumped into the tank at Awahdevi is being brought in a pumping time of 16 hrs per day. The water from the storage tank at Awahdevi is flowing out to the service reservoirs by gravity flow in a time period of 18 hrs. Thus, we require a storage capacity of 2 hrs at Awahdevi to balance the transmission of the water from the water treatment plant to the service reservoirs which have a storage capacity of 12 hrs. The Consultants have proposed a storage capacity of 3 hrs at Awahdevi so that the scheme can be put to operation at 0 hrs both at the water treatment plant and the onward transmission of water to the service reservoirs. The 1 hr overlap of storage capacity is sufficient at Awahdevi.

Provide depth of water = 3.0 m

Plan area of tank required

$$\frac{1000}{3} = 333.33 \text{ m}^2$$

Assume $L : B = 2.5 : 1$

$$2.5\, B \times B = 333.33$$

$$\therefore \quad B = 11.55 \text{ m ; say } 12.0 \text{ m}$$

or $\quad L = 28.86$ m ; say 29.0 m

Provide a tank of size 29.0 m × 12.0 m × 3.0 m (water depth)

A free board of 0.5 m is also provided.

Hence, provide **a covered tank of size 29 m × 12 m × 3.5 m, as per details** shown in Fig. 13.12.

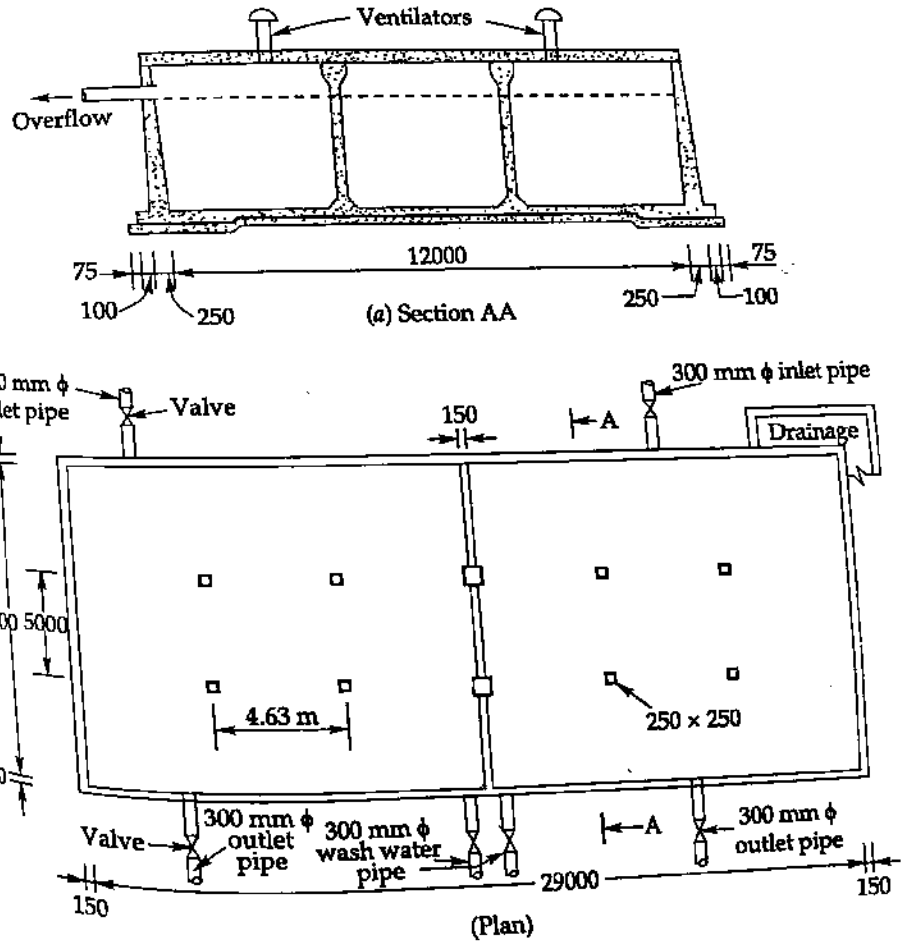


Fig. 13.12. Main Distribution Tank at Awahdevi.

(9) Design of 1st Stage Rising Main and its Pump House

The 1st stage rising main is to be operated between Pre-sedimentation tank/sump and Bheri raw water storage tank. The static lift between these two places, as marked in Fig. 13.2, is 365.46 m. The design discharge shall be taken as 8 MLD to be pumped in 16 hrs.

$\therefore$ Q = 8 MLD to be pumped in 16 hrs

$$= 8 \times \frac{24}{16} = 12 \text{ MLD}$$

$$= \frac{12 \times 10^6}{24 \times 3600} \text{ L/s} = \frac{12 \times 10^6}{24 \times 3600 \times 1000} \text{ m}^3/\text{s} = 0.139 \text{ m}^3/\text{s}.$$

Let us use 350 mm dia* M.S. ERW pipe for rising main. The head loss in the pipe is calculated by using Modified Hazen-William's formula as :

$$H_L = \frac{L \cdot \left(\frac{Q}{C_R}\right)^{1.81}}{994.62\, d^{4.81}} = \frac{10{,}950 \times \left(\frac{0.139}{1}\right)^{1.81}}{994.62 \times (0.35)^{4.81}} = 48.3 \text{ m}.$$

Assume other losses in fittings @ 10% = 4.83 m

Total frictional losses = 48.3 × 4.83 = 53.13 m

Static lift = 365.46 m

$\therefore$ Total head required for pumping = 365.46 + 53.13 = **418.59 m.**

Power required to run the pumps giving 0.139 m^3/s at a head of 418.59 m

$$= \frac{\gamma_w \cdot Q \cdot H}{\eta} = \frac{9.81 \times 0.139 \times 418.59}{0.6} \frac{\text{kN}}{\text{m}^3} \cdot \frac{\text{m}^3}{\text{s}} \cdot \text{m} = \text{kN} \cdot \text{m/s}$$

= kJ/s = k Watt = 951.3 kW

1 HP = 735 Watts = 0.735 kW

$\therefore$ H.P. of pump required = $\frac{951.3}{0.735}$ = 1294 HP ; say 1300 HP.

Hence, **provide 2 pumps each of 650 HP, with one additional pump of equal capacity as standby for repairs etc. Ans.**

***Note.** The pipe size of 350 mm dia chosen above is infact to be decided after conducting economy calculations on different sized pipes like 2 pipes each of 300 mm ϕ, 2 pipes each of 350 mm ϕ, one pipe of 350 mm ϕ, one pipe of 400 mm ϕ, one pipe of 450 mm ϕ, one pipe of 500 mm ϕ etc. and then computing the cost of pump (original and running) and the cost of pipe together. *The pipe dia corresponding to which the cost of pipe and pumping is found to be minimum will be the most economical dia, and hence chosen.*

For the present project, such calculations were infact conducted and shown in tables 13.2 to 13.4, and *the pipe dia of 350 mm was found to be the most economical one*, and hence adopted. Similar exercise was done for 2nd stage pumping. The details of provided Pump House are shown in Chart Fig. 13.13.

(10) Design of 2nd Stage Rising Main and its Pump House

The 2nd stage rising main is to be operated between WTP at Rasoo and Main Distribution reservoir at Awahdevi for a static lift of 310 m in a pipe length of (*RD* 21000 − 14850 = 6150 m).

Let us use 400 mm dia M.S. ERW pipes, being the most economical dia, as shown in Table 13.2 to 13.4. The head loss in this pipe is computed as :

$$H_L = \frac{L \cdot \left(\frac{Q}{C_R}\right)^{1.81}}{994.62\, d^{4.81}}$$

Here Q = 12 MLD, since pumping is for 16 hours = 0.139 m^3/s

$C_R = 1$

L = 6150 m

$\therefore$ $H_L = \frac{6150(0.139)^{1.81}}{994.62(0.4)^{4.81}} = 14.26$ m

Other losses @ 10%	= 1.426 m
Total frictional losses	= 15.686 m
Static lift	= 310 m

Total head against which pumping is to be done

= 310 + 15.686 = 325.686 m

Power required to run the pump giving 0.139 m^3/s at a head of 325.686 m

$$= \frac{\gamma_w \cdot Q \cdot H}{\eta} \left(\frac{\text{kN}}{\text{m}^3} \cdot \frac{\text{m}^3}{\text{s}} \cdot \text{m} = \text{kN m/sec} = \text{k Watt} \right)$$

$$= \frac{9.81 \times 0.139 \times 325.686}{0.6} = 740 \text{ kW}.$$

1 HP = 735 Watt = 0.735 kW

$\therefore$ H.P. of Pump required = $\frac{740}{0.735}$ = **1006 HP.**

Provide 2 pumps each of 503 HP with one additional pump as stand by unit. Details of typical pump house for stage pumping are shown in **Chart Fig. 13.13.**

(11) Design of Gravity Main between Raw Water Tank at Bheri and Water Treatment Plant at Rasso.

The length of this pipe is 3900 m and drop or head available for flow is 23.46 m. Since discharge occurs continuously for 24 hrs a day, the discharge through the pipe = $\frac{8 \times 24}{24}$ = 8 MLD = $\frac{8 \times 10^6}{10^3 \times 24 \times 3600}$ m^3/s = 0.0926 m^3/s. The computations of frictional head loss and residual head are carried out for this gravity main in Table 13.5, for different sizes, from which the most efficient dia (giving maximum velocity and minimum residual head) of 0.35 m (350 mm) is chosen. Hence, we will use one pipe of 350 dia for gravity main between raw water tank and W.T.P.

Table 13.2. Table Showing Velocity and Head Loss Computations for Different Dia Rising Mains (1st Stage as well as 2nd Stage Rising Mains)

Stretch	Length (m)	Dia. of pipe (mm)	Static head (m)	Velocity 1st 15 years (m/sec)	Flow through the pipe in kilo litre per day (kLD)	Fractional losses (m)	Other losses 10% (m)	Total head required (m)
First	10950×2	300	365.46	0.985	6000×2	28.89	2.889	397.24
Stage	10950×2	350	365.46	0.722	6000×2	13.76	1.376	380.60
Pumping	**10950**	**350**	**365.46**	**1.44**	**12000**	**48.3**	**4.83**	**418.59**
	10950	400	365.46	1.108	12000	25.39	2.539	393.39
	10950	450	365.46	0.87	12000	14.41	1.441	381.33
	10950	500	365.46	0.71	12000	8.68	0.868	375.00
Second	6150×2	300	310	0.985	6000×2	16.23	1.623	327.85
Stage	6150×2	350	310	0.722	6000×2	7.73	0.773	318.50
Pumping	6150	350	310	1.44	12000	25.22	2.522	337.74
	6150	**400**	**310**	**1.108**	**12000**	**14.26**	**1.426**	**326.69**
	6150	450	310	0.87	12000	8.09	0.809	318.90
	6150	500	310	0.71	12000	4.87	0.487	315.36

Table 13.3. Calculation Table Showing kilowatt Required and Cost of Pump Sets in Stage pumping (1st and 2nd Stage Pumping)

S. No.	Stretch	Dia of pipe mm	Total head required for lifting (H) in m	kW required for pump $= \gamma_w . Q. H = 9.81 \times 0.139\ H$	kW required including 50% stand by $= 1.5 \times (5)$	H.P. of pumps required including 50% stand-by units $= \frac{\text{col.(6)}}{0.735}$	Cost of pump sets @ Rs. 3000/- per HP i.e. col. (7) × 3000 Rs. ('000)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1	First	300	397.24	897.76	1346.64	1804.50	5413.49
2	Stage	350 × 2	380.60	860.43	1290.22	1728.90	5186.70
3	Pumping	**350**	**418.59**	**951.31**	**1426.96**	**1941.45**	**5824.34**
4		400	393.39	889.06	1333.59	1787.00	5361.00
5		450	381.33	861.81	1292.71	1732.23	5196.69
6		500	375.00	847.50	1271.25	1703.48	5110.43
1	Second	300	327.85	740.95	1111.42	1489.30	4467.91
2	Stage	350 × 2	318.50	719.82	1079.73	1446.83	4340.50
3	Pumping	350	337.74	763.29	1144.94	1534.77	4604.31
4		**400**	**325.69**	736.05	**1144.08**	**1479.46**	**4438.38**
5		450	318.90	720.71	1081.07	1448.63	4345.89
6		500	315.36	712.70	1069.06	1432.54	4297.62

Table 13.4. Comparative Statement of Overall Cost of Pumping and Pipe Required to Determine the Most Economical Pipe Dia, in Ist stage as well as 2nd Stage Pumping

S. No.	Pipe size (mm)	Total head in m	Class of pipe	Rate/m	Cost of pipeline Rs. ('000)	Cost of Pumping			Total cost of pipe and pumping (col.(6) + col.(7) + col.(9) Rs. ('000)
						Cost of pump sets Rs. ('000)	Annual energy charges (C_R) Rs ('000)	Capitalised energy charges (C_C). Rs ('000)	
(1)	(2)	(3)	(4)	(5)	(6)	(7)*	(8)*	(9)*	(10)
1st Stage	300 × 2	397.24	M.S.— E.R.W. Pipe	3200	70080	5414	5934	45131	120625
	350 × 2	380.60		3628.15	79457.1	5187	5685	43240	127884
	350	**418.59**		**3628.2**	**39729**	**5824**	**6288**	**47823**	**93376**
	400	393.39		5005	54805	5361	5876	44694	104860
	450	381.33		7132.17	78097	5197	5696	43324	126618
	500	375.00		8644.8	94660	5110	5601	42605	142375
2nd Stage	300 × 2	327.85	M.S.— E.R.W. Pipe	3200	39360	4468	4897	37248	81076
	250 × 2	318.50		3628.15	44626	4340	4758	36186	85152
	350	337.74		3628.2	39729	4604	5045	38372	82705
	400	**325.69**		**5005**	**61562**	**4438**	**4865**	**37002**	**72221**
	450	318.9		7132.17	43863	4346	4763	36231	84440
	500	315.36		8644.8	53163	4298	4710	35828	93289

(7)* Cost of Pump sets = From col. (8) of Table 13.3.

(8)* *Annual cost of Energy Charges :*

Q in yr 2002 (Start of project) = 5.9 MLD

Q in yr 2017 (Design discharge after 15 yrs) = 8 MLD

$$\text{Average } Q = \frac{5.9 + 8.0}{2} = \frac{13.9}{2} = 6.95 \text{ MLD}$$

Hrs of pumping = 16 hrs

$$\text{Average hrs of pumping of 15 yrs} = \frac{6.95}{8.0} \times 16 = 13.9 \text{ h/d}$$

Annual energy cost @ Rs. 1.30 per k.w. hr (unit) of electricity consumed

= Rs. 1.30 (kW × 13.9 h/d) × 365.24 d

= **6609.4 kW**

C_R = 6609.4 × col. (5) of Table 13.3.

(9)* Energy charges capitalised over 15 yrs @ 10% annual interest rate

$$= C_c = C_R\left[1 - \frac{(1+r)^n}{r}\right] \qquad \text{where } n = 15,\ r = 10\%$$

$$\therefore \quad C_c = C_R\,[7.606] = 7.606\ C_R$$

= 7.606 × col. (8) of Table 13.4.

Note. From this table, it is seen that the most economical size of rising main for 1st stage is one number MS-ERW pipe of 350 mm dia pipe ; and for 2nd stage, it is one number MS-ERW pipe of 400 mm dia.

Table 13.5. Table Showing Computation of Velocity and Loss of Head for Different Dia Pipes for Gravity Main between Raw Water Tank and W.T.P.

S. No.	Length (m)	Dia. of pipe (d) × No. of pipes in parallel (m)	Discharge through the pipe $= Q$ (m^3/s)	Vel. through the pipe $= \frac{Q}{A} = \frac{Q}{\frac{\pi}{4} \cdot d^2}$ (m/s)	Frictional head loss $H_L = \frac{3900Q^{1.81}}{994.62\, d^{4.81}} = \frac{3.92\, Q^{1.81}}{d^{4.81}}$ (m)	Other losses (H_L') @ 10% × col. (6) (m)	Residual head = Total available drop $- H_L - H_L' = 23.46$ − col. (6) − col. (7) (m)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1	3900 × 2	0.3 m × 2	0.0463 × 2	0.66	4.93	0.49	18.04
2	3900 × 2	0.35 m × 2	0.0463 × 2	0.48	2.35	0.24	20.87
3	**3900 × 1**	**0.35 m × 1**	**0.0926 × 1**	**0.96**	**8.24**	**0.82**	**14.40 (Min.)**
4	3900 × 1	0.4 m × 1	0.0926 × 1	0.74	4.33	0.43	18.70
5	3900 × 1	0.45 m × 1	0.0926 × 1	0.58	2.46	0.25	20.75
6	3900 × 1	0.50 m × 1	0.0926 × 1	0.47	1.48	0.15	21.83

PROBLEMS

1. It is proposed to provide protected water supply to a metropolitan town with a population of 8,00,000. Write out briefly the investigations to be carried out, data to be collected, and other salient features to be considered in formulating a suitable scheme for this town.
2. (*a*) State the investigations you would conduct and the data you collect for preparing a water supply scheme for a city.
   (*b*) Mention and explain briefly the points which you will discuss in the project report for the above water supply scheme.
3. A town with an ultimate population of 2,00,000 is to be supplied with surface water from a perennial river, which has fairly high turbidity during the rainy season. Design the dimensions of the main units of the water treatment plant and draw the flow diagram. Rate of water supply is 180 litres per head per day. Make your own assumptions for the design.

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14.2. Essential Re Surroundings

The essential requiremei

(*i*) The water should
appearance.

(*ii*) It should be safe to bath

(*iii*) It should have approved
immediately remove the bacteria
from their bodies.

(*iv*) It should have a comfortable te

(*v*) It should be *chemically balanced*,
effects on the eyes, nose, throat and skin of
bath in the pool.

(*vi*) Minimum amount of chemicals should be added in treating the water of the swimming pool.

(*vii*) The water of the pool should be transparent, as to make the pool bottom clearly visible at maximum depth.

(*viii*) The pool bottom, side walls and surroundings must be maintained clean and hygienic to a very high standard.

(*ix*) The pool must have an efficient surface water removal system and suitable turn-over period for changing the water of the pool to suit variable *loading conditions** in the pool.

*The amount of pollution added to the pool depends upon the number of swimmers using the pool. The heaviest bathing load is expected during warm weather and on weekend holidays, etc., and smaller attendance during winter months and week days. Bathing loads will also vary during the course of a day, and may be heavier during summer evenings.

			23.46
			(6)
			Col. (7)
			(m)
			(8)
		49	18.04
		0.24	20.87
		0.82	**14.40 (Min.)**
		0.43	18.70
		0.25	20.75
	1.48	0.15	21.83

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...water supply to a metropolitan town with a ...iefly the investigations to be carried out, data to ...atures to be considered in formulating a suitable ...s.

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3. A town with an ultimate population of 2,00,000 is to be supplied with surface water from a perennial river, which has fairly high turbidity during the rainy season. Design the dimensions of the main units of the water treatment plant and draw the flow diagram. Rate of water supply is 180 litres per head per day. Make your own assumptions for the design.

—14

Water Treatment in Swimming Pools

14.1. Layout, Shape and Structural Design of Swimming Pools

Swimming pools are individual projects, constructed in big hotels, schools, farm houses or in other community development projects. The general layout of a small swimming pool provided with bath houses is shown in Fig. 14.1 ; while the typical structural details of a 21 m × 9 m pool are given in Fig. 14.2.

The quality of water required in a swimming pool must be properly controlled, as to provide safe and pleasant bathing conditions in the pool. The water of a pool should always be crystal clear, of good colour and attractive appearance.

14.2. Essential Requirements of Swimming Pool Waters and Its Surroundings

The essential requirements of swimming pool waters are summarised below :

(*i*) The water should be clear, of good colour, clarity and attractive appearance.

(*ii*) It should be safe to bath in it.

(*iii*) It should have approved quantities of disinfectant dissolved in it, to immediately remove the bacteria and pollution being added by the bathers from their bodies.

(*iv*) It should have a comfortable temperature of about 27 to 30°C.

(*v*) It should be *chemically balanced*, as not to cause discomfort and adverse effects on the eyes, nose, throat and skin of men, women and children taking bath in the pool.

(*vi*) Minimum amount of chemicals should be added in treating the water of the swimming pool.

(*vii*) The water of the pool should be transparent, as to make the pool bottom clearly visible at maximum depth.

(*viii*) The pool bottom, side walls and surroundings must be maintained clean and hygienic to a very high standard.

(*ix*) The pool must have an efficient surface water removal system and suitable turn-over period for changing the water of the pool to suit variable *loading conditions** in the pool.

*The amount of pollution added to the pool depends upon the number of swimmers using the pool. The heaviest bathing load is expected during warm weather and on weekend holidays, etc., and smaller attendance during winter months and week days. Bathing loads will also vary during the course of a day, and may be heavier during summer evenings.

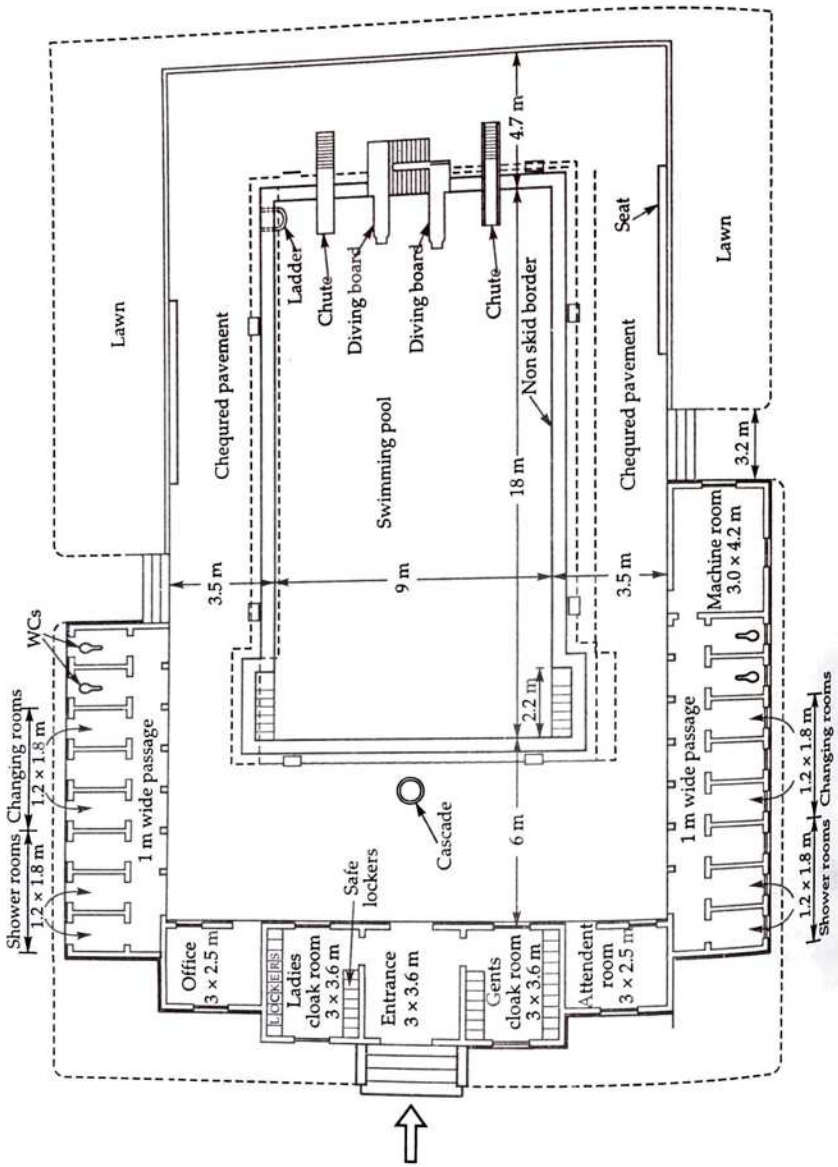


Fig. 14.1. Typical layout for an open swimming pool with bath houses.

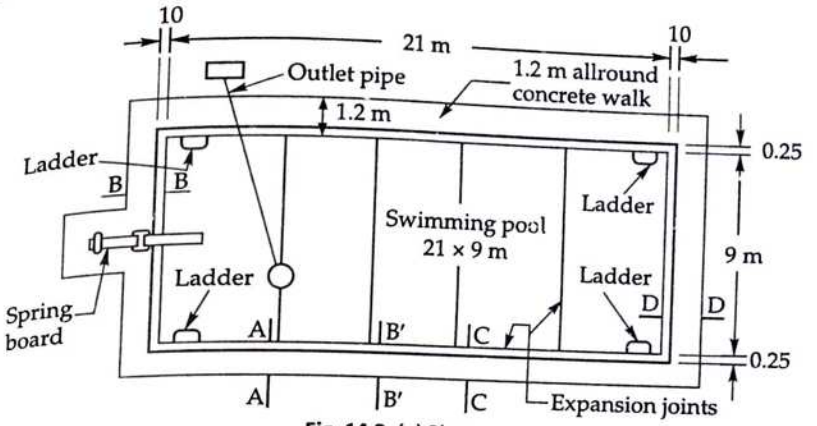


Fig. 14.2. (*a*) Plan

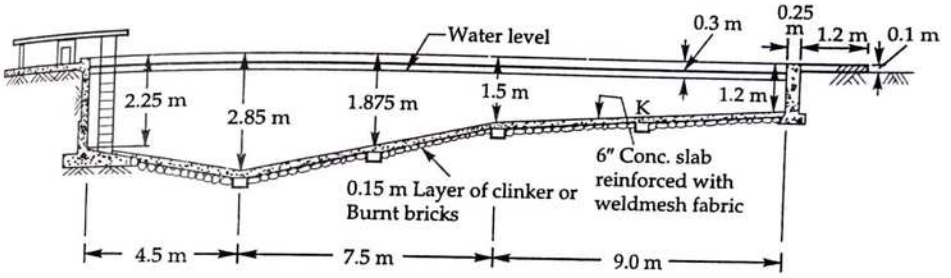


Fig. 14.2. (*b*) Longitudinal section

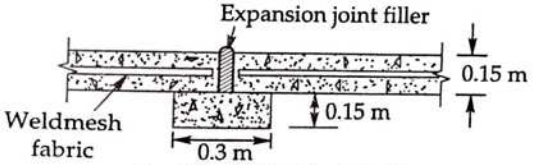


Fig. 14.2. (*c*) Detail at joint K

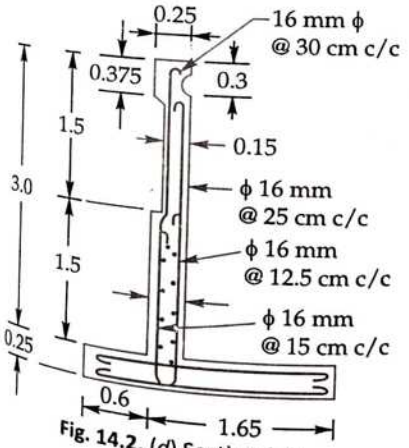


Fig. 14.2. (*d*) Section A-A

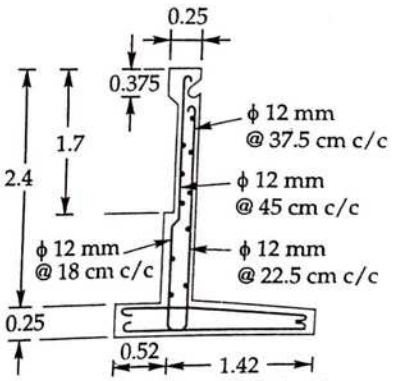
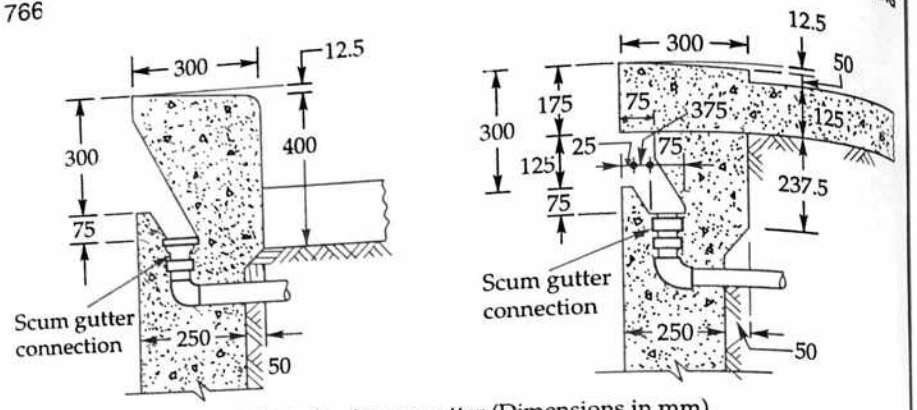
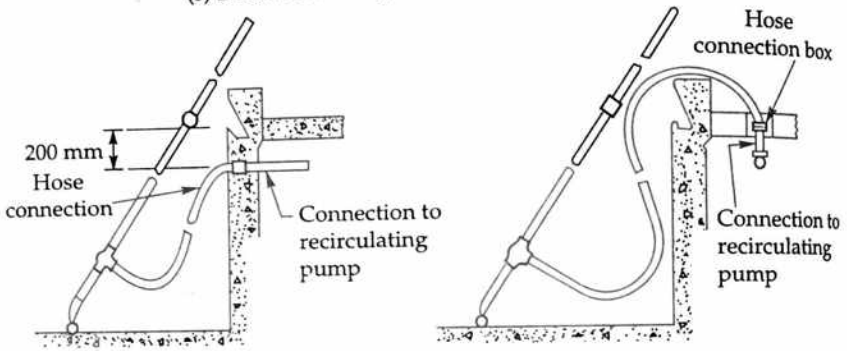


Fig. 14.2. (*e*) Section B-B & B′ - B′



(*c*) Details of scum gutter (Dimensions in mm)



(*d*) Details of vacuum cleaner connection

Fig. 14.3. Typical plumbing layout for a Recycling type of Swimming Pool.

gases, such as hydrogen sulphide, etc. Aeration also gives a sparkling effect to the pool water.

Depending upon the capacity of the pool and the availability of water, an initial filling time of 24—48 hours is considered reasonable. A large dia connection will be required for initial filling of the pool.

Inspite of the reuse of old water, certain fresh water will be required in the pool to compensate the water lost in *evaporation, filter back washing, surface skimming* and *vacuum cleaning of the floors*. This quantity may amount to 2—5% of the pool volume. This lost water will have to be compensated almost daily by bringing in fresh supply in the pool. The typical plumbing layout details of such a swimming pool, using recycling of pool water, are shown in Fig. 14.3.

14.4. Swimming Pool Water Treatment System for Recycling of Pool Water

Fig. 14.4 shows a typical modern arrangement of a swimming pool water treatment system. It primarily uses a ***pressure filter*** (see Fig. 14.5 for Photoview) for filtering the waste water of the pool for its recirculation. The pool water is also chlorinated, by using chlorine gas piping system, as shown in Fig. 14.4. A sketch of direct main chlorination system is also shown in Fig. 14.6.

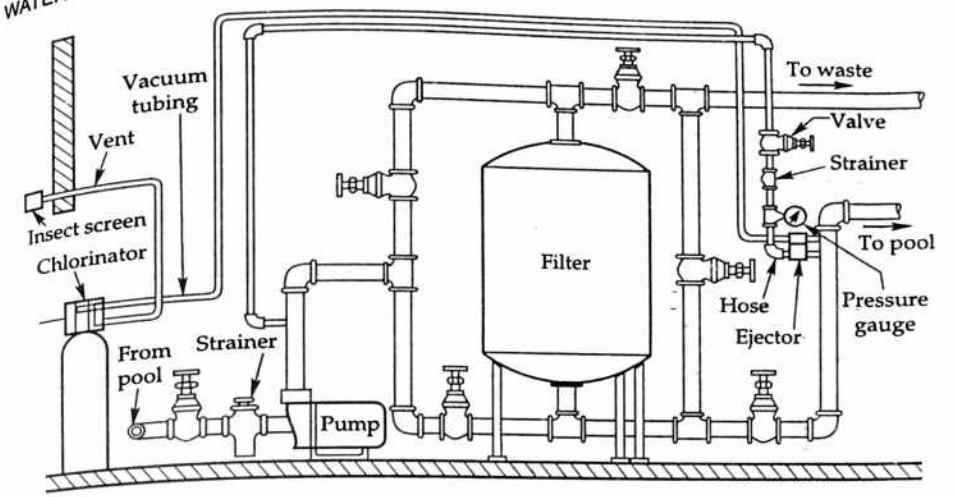


Fig. 14.4. A typical swimming pool water treatment system (Recycling of pool water).

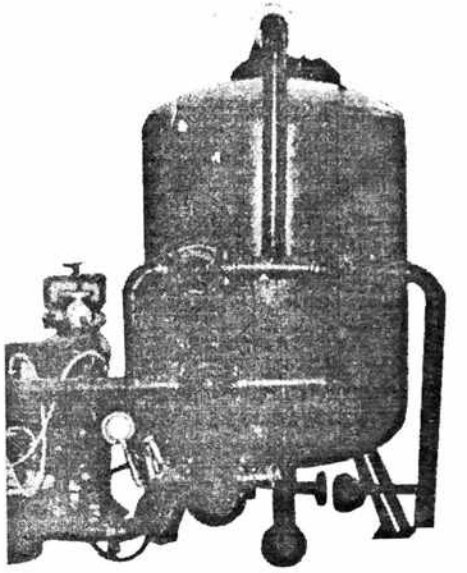


Fig. 14.5. Photoview of the pressure filter (vertical type).

Pressure filters in different sizes are available to suit the specific requirements of water filtration of a given pool. The capacity of the pressure filter will depend on the rate of filtration and area of bed to purify the entire pool water in not more than 8 hours. A rate of filtration of more than 13.5 litres per minute should be avoided where feasible, as less satisfactory water is produced at faster filtration rate.

MAIC (INDIA) LABORATORIES, 0/11-A, Jangpura Extension, New Delhi-14, India, is manufacturing all types of equipment which are required for recirculation

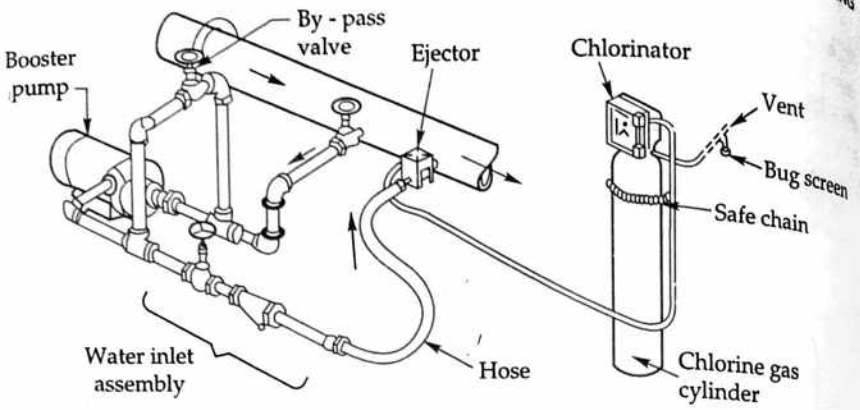


Fig. 14.6. Direct mains chlorination system.

cum disinfection plants for swimming pool waters. The firm is also manufacturing **suction sweepers** or ***vacuum cleaners*** (Fig. 14.7), which are used for vacuum cleaning of the floors of the pools for removing the settled insoluble solid wastes like dust (soil and sand), precipitated chemicals, fluff, etc., and are connected to the recirculating pumps, as shown in Fig. 14.3 (*d*).

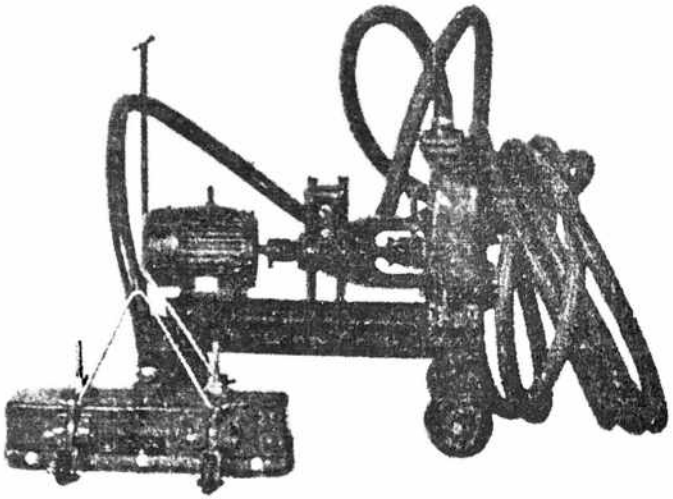


Fig. 14.7. Photoview of a Suction Sweeper used for cleaning the pool bottom daily.

14.5. Pollution Zones in a Swimming Pool

The **physical and chemical pollution** of swimming pools can be divided into the following 4 zones :

(*i*) *Surface pollution* ;
(*ii*) *Dissolved pollution* ;
(*iii*) *Suspended pollution* ; and
(*iv*) *Deposited insolubles*.

These four pollution zones are shown in Fig. 14.8, and the possible constituents of each zone are indicated below :

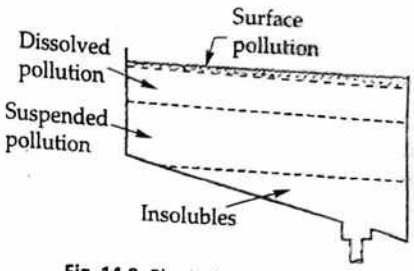


Fig. 14.8. Physical pollutants in a swimming pool.

(*i*) ***Surface Pollution*** may contain impurities and wastes like :
- (*a*) hair
- (*b*) dust
- (*c*) body grease, cosmetics
- (*d*) excreta from human nose and throat
- (*e*) floating debris ; and
- (*f*) grass and leaves.

(*ii*) ***Dissolved pollution*** may contain impurities like :
- (*a*) urine ;
- (*b*) perspiration ; and
- (*c*) suntan lotions, etc.

(*iii*) ***Suspended pollution*** consists of :
- (*a*) particles produced by chemical reactions in water treatment, etc.

(*iv*) ***Deposited insolubles*** settling at the tank bottom may include solid wastes like :
- (*a*) dirt (soil, stones) ;
- (*b*) precipitated chemicals ;
- (*c*) sand from filters ; and
- (*d*) fluff, etc.

The **methods of removal of the above mentioned pollutions** from the pools are summarised below :

(*i*) ***Removal of surface pollution***
- (*a*) remove surface water as soon as possible. Deck level pools are most efficient for this purpose, but other systems are also effective.
- (*b*) Add sufficient chlorine to maintain free chlorine residual of 1—2 mg/L at all times.

(*ii*) ***Removal of dissolved pollution***
- (*a*) Provide adequate filtration cycle
- (*b*) Add sufficient chlorine to breakdown, nitrogenous matter and render it harmless.

(*iii*) ***Removal of suspended pollution***
- (*a*) use minimum quantity of chemicals.
- (*b*) maintain balanced water in the pool.
- (*c*) maintain careful control of pH and alkalinity.
- (*d*) use chemicals which do not produce suspended solids.

(*iv*) ***Removal of deposited insoluble pollution***

Adopt vacuum cleaning of the pool bottom daily.

The biological pollution in a swimming pool is caused by the bacteria and living micro-organisms, which multiply rapidly in a pool water. The bacteria are usually added to the pool water by the bathers and swimmers from their noses, throats, urines, sores & bounds, and may be very harmful and pathogenic. These bacteria may cause the water to turn green, cloudy and translucent, soon making the pool to be unsafe for public bathing and swimming, if not treated effectively. From biological pollution point of view, the top 16 cm depth of the pool generally contains 75% of the bacterial load, as shown in Fig. 14.9. Algae may grow in lower portion of the pool, and may be caused by the spores present in the supplied water or collected from the atmosphere.

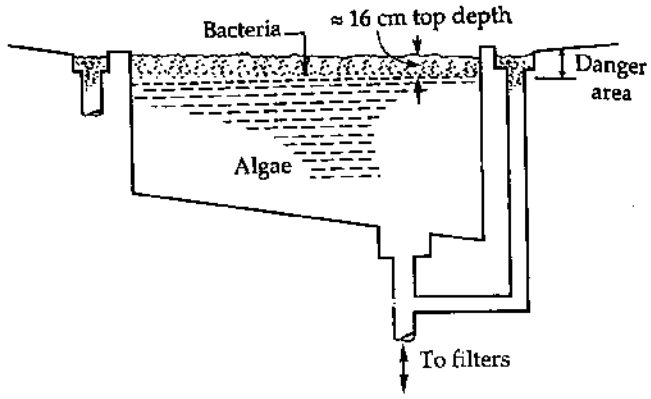


Fig. 14.9. Biological pollutants in a swimming pool.

The biological pollution of the pools can be removed or minimised by :

(*a*) maintaining free chlorine residual of 1—2 mg/L in the pool water at all times. Chlorine will kill-off all the bacteria immediately and render the pool water safe for public bathing ;

(*b*) providing an efficient top water drew-off system ; and

(*c*) providing an efficient filtration system-operated in a proper manner.

14.6. Controlling the Water Quality of a Swimming Pool

14.6.1. Hardness. The quality of water supplied to the swimming pools may vary depending upon the source of supply. When surface supplies, such as municipal waters, are used, the water will generally be soft. However, when tube-well waters are used, the water may usually be hard. The comparative merits and demerits of soft and hard waters, when used for swimming pools, are given below :

(A) **Hard Waters**

Merits

(*i*) can be helpful as an alkali in neutralising acidity in pool water (*acidity develops in pools due to the use of chlorine gas as a disinfectant*)

(*ii*) in controlled quantities, it can improve colour, and appearance of the pool water.

Demerits

(*i*) causes water to form scale on the walls and floor of the pool as well as in hot water taps, showers and pipe fittings.

(*ii*) causes water to feel hard.

(B) **Soft Waters**

Merits. Nil.

Demerits

(*i*) contains humic and fulvic acid

(*ii*) causes corrosion in filters, water pipe lines, filter shells, pool water heaters, etc.

(*iii*) attacks the tiles and grouting on pool bottom and side walls.

(*iv*) products of iron corrosion may get deposited on the floor tiles as dirty brown rust marks.

(*v*) if chlorine is added to freshly filled pool, the green colour is intensified due to the presence of products of iron corrosion, and becomes difficult to remove.

From the above, it can easily be concluded that an ideal water for a swimming pool will be the one, which contains sufficient hardness (particularly the calcium salts) to combat corrosive tendencies.

For this reason, the control on the hardness content of the swimming pool water is now strongly recommended, and **calcium hardness tests** should regularly be performed on pool waters. It is recommended that the pool water **must have a minimum hardness of 50 mg/L (as $CaCO_3$ of course) and an ideal hardness of 125 mg/L (as $CaCO_3$).** If the hardness of pool water is *too low* (less than 50 mg/L), it will make the water aggressive, causing erosion and pitting of the tiles and grouting existing on the pool bottom and sides. On the other hand, if the hardness is *too high* (much higher than the ideal value of 125 mg/L), then scaling will occur and iron present in water may also cause corrosion in filter plant.

14.6.2. pH Value. A very strict and regular control on the pH value of a swimming pool water is generally required to keep the pool water in a balanced condition ; and to ensure efficient chlorination. The permissible pH range in a swimming pool water is between 7.2 to 8.0, and the **preferred range is between 7.4 to 7.6.** Inorder to maintain the pH value to be between 7.4 to 7.6, *it usually becomes necessary to add alkalis like sodium carbonate and sodium bicarbonate to increase the pH value,* since the pH value in a pool usually decreases due to the general tendency of the pool water to become acidic due to the use of chlorine gas as disinfectant. Very soft water when used in a pool may therefore, require large additions of these chemicals, as otherwise they may prove to be very corrosive. When hard water is used in a pool, it is naturally alkaline and is usually having a high pH value. Such a hard water contains dissolved alkali, which can give a buffering effect with less chemical being required to maintain chemical balance.

In a pool water, therefore, sufficient alkali should be present (or added) to maintain the pH at 7.4—7.6, and there should also be sufficient bicarbonate alkalinity present to give a buffering effect and prevent rapid fluctuations in the pH value. Should insufficient alkali be present in the pool water, the pH

will fall and the colour, clarity and the appearance of the water will get adversely affected, causing serious adverse effects on the comfort level of the swimmers. With a low bicarbonate alkalinity in water, the effect will be more rapid, resulting in a corrosive and aggressive water, which will attack the grouting in the tiles in the pool and the metal of the filter and pipe lines.

If the pH value of a pool water tends to increase beyond the required range, then also the bather's comfort level will reduce. The following steps should then be taken to reduce the pH to bring it to the level of 7.4—7.6 :

(*i*) adjust and reduce the amounts of chemicals if being added

(*ii*) dilute the pool supply with fresh soft water supply or by backwashing the filters

(*iii*) in extreme case or in very hard water areas, acid may have to be added, or water softening installation may be required.

14.6.3. Alkalinity. The alkalinity to be maintained in a swimming pool water will be chiefly governed by the chlorination method being adopted in the pool, as indicated in table 14.1.

Table 14.1

Form in which chlorine is used for disinfection	*Preferred range of alkalinity in mg/l*
as chlorine gas	180—200
as sodium hypochlorite	150—180
as calcium hypochlorite (HTH)	100

Alkalinity should not be confused with pH, since '*alkalinity*' represents the quantitative analysis of the alkali present in water as *bicarbonate*, which acts as a 'buffer' to be readily available with fluctuating pH conditions and maintains the clarity and comfort conditions in the pool. pH on the other hand is the qualitative analysis of the acidity or alkalinity of the pool water, as registered on the pH scale. One function of bicarbonate alkalinity is to maintain hardness salts in solution.

Low alkalinity allows rapid fluctuations in pH to occur. Corrosive conditions soon develop leading to problems in maintaining comfortable bathing conditions in the pool.

High alkalinity contains a high degree of 'buffering', and the pH cannot be readily adjusted to the desired level of 7.4—7.6. In hard water, this may cause scale formation on the walls and floor of the pool.

14.6.4. Total Dissolved Solids (TDS). When chemicals are added to the natural waters to increase the alkalinity etc., the amount of total solids dissolved in water may increase to an undesirable level. The presence of excess TDS may make the water to appear dull and lifeless, and possibly turbid due to suspended solids, with soapy and salty taste, giving unpleasant bathing conditions. Should these solids include a high chloride content, the corrosive action of water may further increase.

For this reason, **the TDS should not be allowed to exceed 1500 mg/L** in pool water.

Control over TDS can be obtained as follows :

(*i*) Add minimum quantities of chemicals to produce efficient disinfection of the pool water.

(*ii*) Back-wash the filters regularly with sufficient water.

(*iii*) Dilute the pool water by adding fresh water from the supply mains to replace some quantity of recycled water. This should particularly be resorted to in shallow learning and leisure pools, where heavy bathing loads are experienced, and the water/bather ratio is low, particularly on the weekend holidays. Dilution with fresh water will generally alleviate the problems of high TDS even in public swimming pools. *It is infact, a good operational practice for all public swimming pools to be emptied and refilled at least once a year.*

14.7. Chlorination of Swimming Pools

Chlorine is usually added to the swimming pool water to cause disinfection, and helps in removing the bacterial load by the action of hypochlorous acid (HOCl). Besides killing the germs, it also helps in removing the urine and other nitrogenous pollutants, breaking them down into ammonium compounds, which react with chlorine to form chloramines which are soluble in water. These chloramines may again react and breakdown, if the pool water is not in a balanced condition. If this happens so, then pungent nitrogen trichloride gas will get liberated from the pool surface. In this process, hydrochloric acid (HCl) will also form, which will further reduce the pH value of the water, thereby causing further imbalance in the pool.

Since the bathers continuously add bacteria and nitrogenous matter (urine) in the pool water, it becomes necessary to continuously add chlorine to the pool in proportionate amounts to ensure free residual chlorine of about 1—2 mg/L in the pool water, all the time. *The combined chlorine in water should, however, not exceed $\frac{1}{3}$rd of the total chlorine present.* Should insufficient free chlorine is added, the combined chlorine level will rise, and the water looses colour and clarity, liberating nitrogen trichloride gas in the pool hall, causing skin irritation and smarting eyes. In such a situation, more direct chlorine must be added to restore the balance with *shock treatment* to raise the free chlorine level quickly and 'burn up' chloramines.

Under such imbalances in the pond, the pH will fall, if alkali is not sufficiently present or added, which will further adversely affect the colour, clarity and appearance of the water. With low bicarbonate alkalinity in water, the effect will be more rapid, resulting in corrosive aggressive water, which will attack the grouting in the tiles in the pool and the metal of the filters and pipe lines, as stated earlier.

Since the bathing loads can fluctuate considerably, the amount of pollution load cannot be accurately estimated. But the amount of chlorine added must be reasonably accurate to maintain good quality water and comfort conditions in the pool. *Automatic control*, therefore, becomes desirable to obtain accurate adjustments of the chemicals added in proportion to the chlorine demand, as to maintain balanced water under all conditions.

14.7.1. Manual Application of Chlorine. Adding **chlorine granules** direct to the pool water may be used for **shock treatment** to temporarily raise the

free chlorine levels after peak bathing loads, or where other methods of chlorine application are out of order. Should this become the routine and regular method of chlorine application in the pond, it would lead to fluctuating free chlorine levels, high combined chlorine levels, pool water of poor colour, clarity and appearance, and chlorinous pungent smells in the pool hall, with a deterioration in the comfort conditions for the bathers.

Chlorine solutions of controlled strength by mixing chlorine gas (sometimes HTH) with water may be produced manually and the same can be pumped into the pool in controlled quantities to suit bathing conditions. In this process, there remains a manual control in preparation of the solution, and also on deciding as to when and how the solutions are to be pumped into the pool. With an experienced and vigilant plant operator, the pool can probably be maintained in good condition, but otherwise similar problems can occur as in manual application of chlorine granules.

14.7.2. Automatic Application of Chlorine. This is achieved by automatic pumping of controlled strength chlorine solutions in controlled quantities to suit automatically recorded chlorine levels in the pool. The use of such automatically controlled chlorine pumping is always recommended where feasible, since by this method, more accurate control of the levels of free and combined chlorine will be maintained, and many of the problems of high combined chlorine levels, emission of pungent chlorinous odours, smarting eyes and irritation of skin, will be eliminated. Automatic control can also reduce the amount and cost of chemicals used in the plant.

14.8. Testing the Pool Water

Whether the chlorination is done manually or automatically, testing of pool water must be carried out at regular intervals, and adjustments made to ensure the correct levels of free and combined chlorine. Tests should, at the same time, be carried out for pH and alkalinity ; and occasional tests conducted for calcium hardness, thereby ensuring pool water to be balanced chemically, as per the limits stated in the text, and again summarised in table 14.2.

Table 14.2. Acceptable Values of Various Water Quality Parameters to be Tested in Swimming Pools

S. No.	*Parameter*	*Acceptable value*
1.	Free chlorine	1—2 mg/L
2.	Combined chlorine	Not more than 1/3rd of total chlorine (*i.e.* free and combined chlorines)
3.	pH	7.4—7.6
4.	Alkalinity (*a*) when using chlorine gas as disinfectant (*b*) when using calcium hypochlorite as disinfectant (*c*) when using sodium hypochlorite as disinfectant,	 180—200 mg/L 100 mg/L 150—180 mg/L
5.	Calcium hardness	100 mg/L

Inorder to maintain pool water within these limitation ranges, the pH and alkalinity will need continuous readjustments along with the addition of disinfectant in accordance with the varying pollution load on the pool.

The addition of disinfectant will, thus, be regulated in steps with the added pollution. The added disinfectant will change the pH of water (chlorine gas produces acidic conditions ; while hypochlorites, if used as disinfectant, will produce alkaline conditions). Inorder to readjust the changed pH, chemicals may have to be added to the pool water (such as an alkali—when using chlorine gas as disinfectant). With an experienced and efficient plant operator, the amount of chlorine gas and alkali addition can be manually or semi-automatically controlled within the acceptable limits (Table 14.2) during normal bathing conditions ; but with heavy bathing loads, problems may arise which may adversely affect the working of the pool, causing uncomfortable bathing conditions.

With an inefficient and inexperienced plant operator, serious problems can very quickly arise and cause discomfort to the bathers using the pool. In extreme cases, the pool may have to be closed because of unhealthy water conditions.

For these reasons, automatic controlled techniques are nowadays exclusively used in all modern swimming pools. With the automatic control of free chlorine, most of these problems get solved, and the levels of free chlorine and pH adjusted to suit all bathing conditions. With such a control, combined chlorine will not be allowed to build up to unpleasant levels, the quality of pool water will be improved, and safe and comfortable bathing conditions will prevail. There will also occur a saving in the amount of chemicals used, and the time the operator spends in controlling water quality, will be reduced.

PROBLEMS

1. Explain and discuss the inter-connection of chlorination, pH value, and Alkalinity in reference to a swimming pool water, bringing out clearly the importance of these parameters in safe operation of a pool.
2. (*a*) Differentiate between 'fill and empty type' and 'flow through type' of swimming pools.
   (*b*) What is meant by recycling of water in a swimming pool, and how is it achieved in a 'flow through type' of a pool ?
3. Discuss the water treatment techniques that may be adopted to keep the water of a swimming pool in safe and healthy condition. What are the essential requirements of a good swimming pool.
4. (*a*) Enumerate the essential requirements of a swimming pool water.
   (*b*) Compare the merits and demerits of soft and hard waters for their use in swimming pools.
   (*c*) Draw a neat sketch of a typical swimming pool water treatment system using a pressure filter for keeping the water safe and healthy in a flow through type of a swimming pool.
5. Explain with neat sketches the pollution zones of a swimming pool with reference to the physical and biological pollution loads. Also point out the methods of removal of different types of pollutions from these zones.
6. (*a*) List out the permissible values of pH, alkalinity, total dissolved solids, calcium hardness, free chlorine and combined chlorine for a swimming pool water.

(*b*) Discuss the impact of chlorination of a swimming pool water on the pH and Alkalinity levels of the pool. How do you propose to remove the imbalance created by the chlorination of the pool ?

7. (*a*) List out the various tests that are regularly performed on the water of a swimming pool to check the presence of balanced water in the pool, indicating the permissible values of the tested parameters.

(*b*) How will the detected imbalances be removed from the pool water ?

8. Write brief but explanatory notes on the following :

(*a*) Recycling and Treatment of swimming pool waters.

(*b*) Use of soft and hard waters in swimming pools.

(*c*) Pollution zones in a swimming pool.

(*d*) Suction sweepers for swimming pools.

(*e*) Direct mains chlorination system of a swimming pool.

(*f*) Chlorination of a swimming pool and its likely impact on pH and Alkalinity.

(*g*) Balanced water in a swimming pool.

—15

Laboratory Experiments in Water Supply Engineering

Several laboratory tests are conducted on water (raw as well as treated) to determine its characteristics and to determine the degree of treatment required to make it potable, as stated in the text. Usually, at under-graduate level, the following tests will be required to be performed in the water testing laboratory :

1. To find the *turbidity* of a given sample of water.
2. To find the *odour* of a given sample of water.
3. To find the *colour* of a given sample of water.
4. To determine the *pH value* of a given sample of water.
5. To determine the *Carbonate, Bicarbonate* and *Hydroxide Alkalinity* in a given sample of water.
6. To find out the *Concentration of chlorides* in the given sample of water.
7. To estimate the *hardness* of the given sample of water by EDTA method.
8. To determine *Residual chlorine* in a given sample of water.
9. To determine the *Chlorine demand* of a given sample of water.
10. To determine the *Available chlorine percentage in a given Bleaching Powder*.
11. To determine the amount of *Dissolved oxygen (D.O.)* in the given sample of water by *Winkler method.*
12. To determine the amount of *Total Solids*, *Suspended Solids* and *Dissolved Solids* in a given sample of water.
13. To determine the *Quantity of alum* required to coagulate a given sample of water.

These experiments have generally been explained in the text, yet, however, all the above experiments are described here in details, in the form, you will need them to record in your 'Experiments Notebook', after performing these tests in the laboratory.

EXPERIMENT NO. 1

15.1. Objective : To find the Turbidity of a Given Sample of Water by :

(1) Jackson's turbidimeter ;

(2) Bayli's turbidimeter ; and

(3) Photometers.

Theory. Turbidity is caused in natural waters by finally divided suspended particles of clay, silt, sand or by some organic materials, and by microscopic organisms. It is usually expressed in mg/l (or ppm), and may be determined by optical observations, *as the interference to the passage of light through the given water.* The *standard unit* of *turbidity* is that turbidity which is produced by mixing 1 mg of *finally divided silica* SiO_2 (called **Fuller's earth**) in one litre of distilled water. *In excess of 5 units, it is noticeable to the average water consumer, and hence represents an unsatisfactory condition for water. Turbidity in a typical clear lake water is about 25 units and for muddy waters, exceeds 100 units.*

Turbidimeters are generally used to measure the turbidity of the given water samples. A **turbidimeter** generally works on the principle of measuring the interference caused by the water sample to the passage of light rays. *Jackson turbidimeter* and *Baylis turbidimeter* are the two types of turbidimeters, which have largely been used from the past to measure turbidities, and work on the principle of *absorption.*

Turbidity is considered to be a very important factor in drinking waters due to aesthetic and psychological reasons, besides being a sign of pollution in some particular cases. The turbidity may also affect the filteration and disinfection processes in water treatment processes.

The permissible turbidity for drinking water is usually kept between 5 to 10 units, preferably below 5 units.

15.1.1. Turbidity Test by Jackson's Turbidimeter on Raw Waters (*For Turbidity Range between 25—1000 mg/l*)

Apparatus Required

Jackson's turbidimeter (Fig. 8.2) in a portable wooden box containing :

(*i*) Cylindrical base stand fitted with a candle, or a switch and bulb with dry battery set (torch cells).

(*ii*) Metallic cylinder (container) to be fitted on to the base stand.

(*iii*) Calibrated glass tube to be inserted into the upper metallic cylinder.

(*iv*) A table on a white regular plate fixed to the base stand, giving the turbidity in ppm, corresponding to different readings of the graduated glass cylinder.

Procedure

(*i*) Check up the working of the bulb fitted in the turbidimeter.

(*ii*) Fix up the upper metallic cylinder (container) on the base stand.

(*iii*) Insert the calibrated glass tube into the upper metallic cylinder (container). Switch on the bulb of the turbidimeter.

(*iv*) Shake the water sample thoroughly ; pour the sample into the calibrated glass tube, little by little with the help of a funnel, taking care as not to spill the same into the metallic container.

View the lighted bulb from vertically above, in between each addition of the water sample, and *continue the pouring of the water in the calibrated glass tube till the light becomes just extinguished.*

(*v*) Stop pouring the water sample in the tube, and remove the calibrated glass tube from the metallic container.

(*vi*) Record the depth of water in the calibrated tube. Take at least three such readings on each sample, and obtain the mean value.

(*vii*) Find the turbidity of the given sample of raw water in *JTU* (Jackson's turbidity unit) or in mg/l, from the standard tables supplied with the turbidimeter (instrument).

Note. *The longer is the light path i.e. depth of water in the tube, the lesser will be the turbidity.* For example, a light path of 10.8 cm is 200 JTU, 21.5 cm is 100 JTU, and 72.9 cm is 25 JTU.

Observations Table

S. No.	*Depth of water in the calibrated glass tube in cm*	*Turbidity in mg/l or JTU (from Std. Tables)*
1. 2. 3.		
Average		

Result. The turbidity of the given water sample is ..... mg/l. **Ans.**

15.1.2. Turbidity Test by Bayli's Turbidimeter on Treated Waters (*For Turbidity Range between 0—10 mg/l*)

Apparatus Required

Bayli's Turbidimeter (Fig. 8.3) in a portable galvanised iron box containing :

(*i*) 250 watts electric bulb with reflector.

(*ii*) Two glass tubes to be placed vertically side by side in the instrument at the fixed place. One tube is to be filled with the given water sample ; and the other to be filled with the prepared standard water of known turbidity. Standard water shall be replaced till matching is obtained.

(*iii*) White opal glass plate at the bottom of the tubes.

(*iv*) Blue cobalt plates surrounding the tubes.

Procedure

(*i*) Check up the working of the electric bulb.

(*ii*) Shake the given water sample of the given water (containing turbidity in the range of 0—10 mg/l) and pour it in one of the two glass tubes, and place it in position.

(*iii*) Prepare standard water samples of known turbidities, approximately nearing the turbidity of the given water sample, containing unknown turbidity.

(*iv*) Fill the above standard waters one by one in the matching glass tube, and place it adjoining the tube containing the given water sample of unknown turbidity, in the instrument, over the white opal glass plate, surrounded with blue cobalt plates.

(*v*) Light the electric bulb, and observe the blue colour in both the tubes from the top of the instrument.

(*vi*) If the colour of both the tubes differ, replace the standard water tube, and again compare the colours.

(*vii*) Repeat the process till the matching is obtained.

(*viii*) The known turbidity of the standard solution, matching with the given water sample, may then be noted as the turbidity of the given water.

Result. The turbidity of the given sample of water is ..... mg/l. **Ans.**

15.1.3. Turbidity Test by Photometers and Nephelometers. In modern days, the turbidity measurements are made conveniently and easily through the use of ***photometers***. A beam of light from a source, produced by a standardized electric bulb, is passed through a sample vial. The light that emerges from the sample is then directed to a photometer that measures the light absorbed. The read out is calibrated in terms of turbidity, usually measured in ***formazin turbidity unit (FTU)***. The FTU is based on a chemical, formazin, that provides a more reproducible result, and hence has replaced silica standard unit. ***One FTU*** *may be defined as the turbidity produced by 1 mg of formazine polymer in 1 litre of water.*

A second method of measuring turbidity is by using the principle of *scattering of light* rather than the principle of absorption of light. In such an instrument, the sample "scatters" the light that impinges on it. The scattered light is then measured by putting the photometer at right angles to the original direction of the light generated by the light source. This measurement of light scattered at 90° angle is called *nephelometry* ; and the unit of turbidity in such an instrument, called a **nephelometer** (Fig. 9.3), is known as the ***nephelometric turbidity unit (NTU).***

These modern compact instruments are all calibrated and digitised, and can be easily used to measure turbidities of the given water sample in a short time in the laboratory.

EXPERIMENT NO. 2

15.2. Objective : To Find The Odour of the Given Sample of Water

Theory. Odours in water are caused by the presence of algae, leaves, decaying vegetables, or contamination from sewage and industrial waste. The odour in a given water, is determined at a temperature of 24 to 25°C.

Apparatus. The instrument used for measuring odour is known as ***Osmoscope.*** It consists of two open ended glass tubes, each of 10 mm dia, 30 cm long, and kept at about 8 mm apart by a spacer of cork and adhesive tape strip. The loose plain ends of the tubes are held slightly above the water surface, and the upper bevelled ends are inserted in the nostrils of your nose, to smell the odour.

Procedure. The given water is mixed with odour free water in different proportions, such as 1 in 2, 1 in 4, 1 in 6, 1 in 8 and 1 in 10, etc. to prepare different mixture samples (each of 200 ml) to be tested for odour. Say for example 1 in 2 mix can be prepared by mixing 100 ml of given water in 100 ml of odour free water to make 200 ml of mixture, thereby giving ratio of 1 in 2 mix. Similarly, a mix of 1 in 4 can be prepared by mixing 50 ml of given water in 150 ml of odour free water to make 200 ml of mix.

The mixture of the greatest dilution (such as 1 in 10) is first tested by the instrument for detecting odour ; and then the other samples are tested one by one in the order of their decreasing dilutions. The mixture giving first detectable odour will be taken as having the threshold odour. Say for example, the mixture of 1 in 4 (*i.e.* one part of given water mixed in 3 parts of odour free water to make mixture volume of 4 parts), if found to give the first detectable odour, then the threshold odour number will be 4.

For public supplies, the water should be free from odour. *i.e.* its threshold number should be 1 and should never exceed 3.

Note. Depending upon the smell, the odour is sometimes described as vegetative, aromatic (spicy), greasy, fishy, earthy, musty, disagreeable, etc.

Result. The odour intensity of the given sample of water is ..... Threshold odour number (TON).

EXPERIMENT NO. 3

15.3. Objective : To Find the Colour of a Given Sample of Water

Theory. Dissolved organic matter from decaying vegetation, or some inorganic matter such as coloured soils etc., may impart colour to the water. The excessive growth of algae and aquatic micro organisms may also sometimes impart colour to the water.

The presence of colour in water is not objectionable from health point of view, but may spoil the colour of the clothes if washed in such waters. Colours in water are also objectionable from aesthetic and psychological point of view, since people may not like to drink coloured waters.

The colour of a given sample of water can be measured by comparing its colour with the colour of standard glass tubes, containing solutions of different standard colour intensities. These glass tubes are known as **Nessler tubes.**

One standard unit of colour is that which is produced by dissolving one mg of platinum cobalt in one litre of distilled water. Different standard colour intensities of 1, 2, 3 ..... mg/l, can be easily prepared by dissolving 1 mg, 2 mg, 3 mg ..... respectively of platinum cobalt in 1 litre of distilled water.

The maximum permissible colour for domestic supplies is 20 mg/l (on cobalt scale described above).

Procedure

(*i*) Fill a 100 c.c. (ml) Nessler tube to the mark with the water to be tested. If the turbidity is high, it should be removed by centrifuging or by filtering.

(*ii*) Compare the colour of the sample with that of the standard coloured Nessler tube (containing known value of standard colour intensity of platinum cobalt) by looking vertically down through the two tubes at a white surface in good light.

(*iii*) If the colour of the sample is greater than that of the chosen standard, the standard water should be diluted with distilled water, until the required matching is obtained.

(*iv*) The colour intensity of the standard colour, matching with the sample, may be noted down to represent the colour of the given water sample.

Result. The colour number of the given water is ..... (on cobalt scale). **Ans.**

EXPERIMENT NO. 4

15.4. Objective : To Determine the pH Value of a Given Sample of Water

Theory. The pH value of a water (or of a solution) is defined as the log of reciprocal of hydrogen ions present in that water ; *i.e.*

$$pH = \log_{10} \frac{1}{H^+} \quad \text{...(15.1)}$$

A pure water is, infact, a balanced combination of + vely charged hydrogen ions (H^+) and negatively charged hyroxyl ions (OH^-) ; both ions being equal. Moreover, the product of H^+ and OH^- has been found to be 10^{-14} moles/litre.

Hence, in a pure or a neutral water, the quantum of H^+ and OH^- will each be equal to $\sqrt{10^{-14}} = 10^{-7}$ moles/litre. The pH value of such a pure water will be equal to $\log_{10} \frac{1}{10^{-7}} + = \log_{10} 10^7 = 7$.

If in a given water, OH^- exceeds 10^{-7} moles/litre (say, it becomes 10^{-6}), then its pH value will become less than 7 (say 6 in this case) ; and *vice versa. It therefore, follows that if the pH value of the water is found to be less than 7, it will become acidic in nature ; and if its pH is found to be more than 7, it will become alkaline in nature.*

The pH value of water, thus, indicates the acidity or the alkalinity of water. The maximum acidity will be at 0 value of pH, and the maximum alkalinity will be at pH value of 14.

The pH value is having a large scale importance in water supply system, since it controls water as well as sewage treatment processes. The pH value of raw water infact, must be taken into account, while deciding the various treatment processes like Coagulation, Disinfection, Water-softening, etc.

The pH value also becomes important in corrosion control, since lower pH values (acidic waters) may cause *tuberculation* and *corrosion* of the pipes and treatment tanks, etc. Higher pH values (alkaline waters) may on the other hand, produce *incrustation, sediment deposit, difficulty in chlorination,* besides producing certain psycological effects on human system, if such alkaline waters are consumed.

The permissible pH values for public supplies may range between 6.6 to 8.5.

15.4.1. Measuring pH by a pH-meter. The pH value of a given water can be measured directly in modern days by using a **pH-meter**, such as shown in Fig. 15.1. A beaker or a *glass (electrode)*, in association with a *calomel electrode*, dipped into the water (solution), detects hydrogen ions. The meter is pre-calibrated by using standard solutions of known pH values, and can be used to directly read the pH value of the given sample of water, kept in the glass (or the beaker).

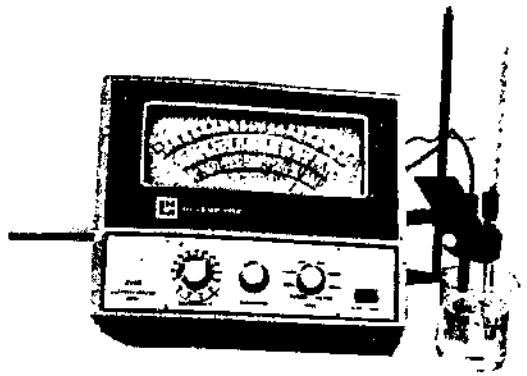


Fig. 15.1. A pH meter provided with glass and Calomel electrodes for measurements of Hydrogen ions (H^+).

15.4.2. Measuring pH by Using Indicator Dyes. Universal indicator *(reagent)* is largely used to measure pH value of water in the range between 4 to 11 ; while **Bromothymol blue** indicator is used for pH range of 6 to 7.6.

A given water sample is, hence, first of all tested with a *universal indicator (reagent)* ; and if its pH comes between 6 to 7.6, it is again tested with a Bromothymol blue indicator, to get a precise value of its pH. The instrument which is used while testing with Bromothymol blue is known as **aquascope.**

Procedure

(*i*) Take four test tubes, and fill them one-fourth each with sample water to be tested. Let these tubes be marked as A, B, C and D, respectively.

(*ii*) Add 15 drops of the *universal indicator* reagent to each of the 4 test tubes.

(*iii*) Mix the solution in all the 4 test tubes thoroughly by turning them up and down.

(*iv*) *Observe the tinge of colours developed in the test tubes, and match them with the colour scale given on the indicator bottle itself.*

(*v*) The colour scale given on the bottle will give the pH value directly. *This is the rough estimation of pH value.*

(*vi*) The water samples found to have pH in the range of 6 – 7.6 in the above test are further subjected to testing by using *Bromothymol blue* indicator in an *Aquascope* (complete with cells and slides).

(*vii*) Fill the sample to be tested in the cell up to block line mark. Put 15 drops of Bromothymol blue indicator in the middle compartment of the cell, and stir it with a stirrer. Wait for 5 minutes.

(*viii*) A colour will develop (ranging from yellow to blue).

(*ix*) The developed colour may be matched with the standard colour slides available in the Aquascope itself.

(*x*) The two slides are put in the slots in front of the slide compartments of the cell, and then the colour developed in the middle compartment is matched with the slides on the sides.

Observations

S.No.	Sample	Colour and pH developed on using			
		Universal indicator		Bromothymol blue	
		Colour	pH	Colour	pH
1.	A				
2.	B				
3.	C				
4.	D				

Result. The pH value of the 4 samples are as follows :

A = ...
B = ...
C = ...
D = ...

Average value = ... **Ans.**

EXPERIMENT NO. 5

15.5. Objective : To Determine the Carbonate, Bicarbonate and Hydroxide Alkalinity in a Given Sample of Water

Theory. Alkalinity of a water is a measure of its capacity to neutralize acids, *i.e.* to absorb hydrogen ions, without significant pH change. It is caused by the presence of *hydroxides* (OH^-), *carbonates* (CO_3^{--}) and bicarbonates (HCO_3^-). A negligible amount of **negative alkalinity** is also caused by hydrogen ions (H^+), which is very small, and hence ignored [Please see article 8.2.1 (*iii*)].

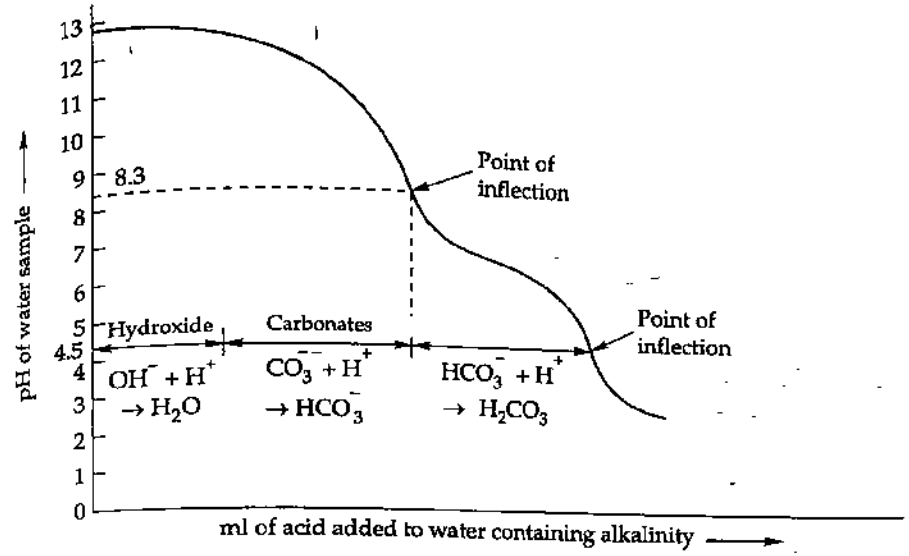
Presence of carbonate and bicarbonate in water is, infact, an interchangeable process, and depends upon the pH value of water. *At lower values of pH, the carbonates change to bicarbonates. So much so, that at pH less than 8.3, only bicarbonates are found to occur*, since all the carbonates are converted into bicarbonates by the action of CO_2 and H_2O, as indicated below :

$$CO_2 + H_2O + CaCO_3 \xrightarrow{pH < 8.3} Ca(HCO_3)_2$$

Even bicarbonates cease to exist in water, if its pH falls below 4.5, since they are all converted into carbonic acid.

Natural waters blooming with algae are, however, usually found to be rich in carbonates, as it makes the availability of CO_2 short for conversion of carbonates into bicarbonates by using it for its own photosynthetic activity. Consequently, algae raises the pH value of water to above 8.3.

When the pH value of water is more than 8.3, carbonates as well as bicarbonates, both may exist. *Hydroxides are, however, found to exist at much higher values of* pH, say well above 10 or 11, as shown in Fig. 15.2. The total alkalinity in water may consists of : (*i*) *only hydroxides* ; (*ii*) *only carbonates* ; (*iii*) *hydroxides and carbonates* ; (*iv*) *carbonates and bicarbonates* ; and (*v*) *only bicarbonates*. The first four possibilities will exist when pH of water is more than 8.3 ; while the fifth and only possibility will exist when pH of water is less than 8.3 and more than 4.5, as shown in Fig. 15.3. This is based on the assumption *that hydroxide and bicarbonate alkalinity cannot be present together in the same sample*, although this may not be 100% true always.



Note. Negative alkalinity caused by H^+ ions is very small, and hence ignored.

Fig. 15.2. Titration curve for a hydroxide carbonate mixture.

Alkalinity of a water sample can be measured by titrating the given water sample with a *standard solution of sulphuric acid*, which when slowly added to the given water will go on increasing the H^+ ions, and reducing its pH value. The amount of sulphuric acid added till the pH of the mixture (water + titrant) is reduced to 8.3, as detected by the use of an indicator dye-*phenolphthalein*, will help in computing the alkalinity existing in the water above pH of 8.3. Further addition of sulphuric acid, till the pH is reduced to about 4.5 will help in determining the total alkalinity in the water (since no alkalinity can exist below pH 4.5).

The standard solution of titrant (H_2SO_4 in this case) is the one whose strength or reacting value per unit volume, is known. Volumetric analysis could be practised solely with such solutions, but it is much more convenient to prepare and use solutions which are equivalent to one another in strength, so that 1 ml of solution A will react with exactly 1 ml of solution B, and so on. *Normal solutions of titrants* are hence used in volumetric analysis. **A normal solution** is *defined as the one that contains one equivalent of a substance per litre of solution.* The *normality of a solution is* its relation to the normal solution, and the symbol N is used as the abbreviation for "normal".

The advantage of using N or a part of N solution of H_2SO_4 as a titrant, is obtained since an N solution will contain 1 mole per litre of H_2SO_4, and will react with 1 mole per litre of alkalinity in the water. Since alkalinity is measured as $CaCO_3$ (having equivalent wt. = 50), its one mole will weigh 50 gm/l, or 50 mg/ml. In other words, 1 ml of alkalinity consumed by the use of 1 ml of N.H_2SO_4 will weigh 50 mg. Hence,

Alkalinity (as $CaCO_3$) in water sample in mg/l

$$= \frac{\text{ml of N } H_2SO_4 \text{ used} \times 50 (i.e. \text{ Eq. wt. of } CaCO_3)}{\text{Vol. of water sample in litres.}} \quad ...(15.1)$$

or Alkalinity in mg/l (as $CaCO_3$)

$$= \frac{[\text{ml of } H_2SO_4 \text{ used} \times N \times 50] \times 1000}{\text{ml of water sample}} \quad ...(15.2)$$

If instead of using N solution, we use $\frac{N}{50}$ solution, then

We have

$$\frac{\text{ml of } H_2SO_4 \text{ used} \times \frac{N}{50} \times 50 \times 1000}{\text{ml of water sample}} \quad ...(15.3)$$

or Alkalinity in mg/l (as $CaCO_3$)

$$= \frac{\text{ml of } \frac{N}{50} H_2SO_4 \times 1000}{\text{ml of water sample}} \quad ...(15.4)$$

On generalisation for any titrant and material in the water sample, we can hence write :

mg/l of active material in the sample

$$= \frac{\text{ml of N titrant} \times (\text{E.W.}) \times 1000}{\text{ml of water sample}} \quad ...(15.5)$$

$$= \frac{\text{ml of } \frac{N}{\text{E.W.}} \text{ titrant} \times 1000}{\text{ml of water sample}} \quad ...(15.6)$$

where E.W. is the Eq. wt. of material in the water sample.

N is the Normality of the titrant, such as $\frac{N}{50}$ or $\frac{N}{EW}$ representing its strength (usually to be adopted equal to N/EW). Normality N representing one mole of titrant powder dissolved in one litre of water

In the titration to be carried out in the given case to measure alkalinity as $CaCO_3$ (E.W. = 50), we shall use $\frac{N}{50}$ solution of H_2SO_4, which can easily be prepared by dissolving $\frac{1}{50} \times 49$ gm = 0.98 gm of H_2SO_4 powder in 1 litre solution of distilled water (since Eq. wt. of H_2SO_4 is 49).

Titration of water containing alkalinity with $\frac{N}{50}$ H_2SO_4 will help in slowly adding an acid solution to the water sample, and reading the resultant

changes in its pH value ; which will help in knowing the existence of ***bicarbonate-carbonate character*** of the given water.

For highly alkaline waters, the first step is to titrate it by adding acid to reduce its pH up to 8.3. The second step is to titrate it to reduce its pH to an indicated value of about of 4.5. If the initial pH of the given water is less than 8.3, the second step itself will become the first step in the method of titration to be adopted, and discussed below. Indicator dyes which change the colour of water at specific pH values can be used to determine the end points of these titrations, if a pH-meter is not used.

Phenolphthalein, as you know, is an indicator chemical which turns water into pink, if the pH is more than 8.3. At pH = 8.3, it changes the water from pink to colourless. In the first stage of the above mentioned titration, phenolphthalein is therefore used to compute the amount of acid required to reduce the pH of water upto 8.3. In the second stage of titration, **methyl Orange** solution is used as an indicator, since it changes from red/orange to yellow in orange at pH 4.6.

The ml of H_2SO_4 used in the first titration, say x, will represent the alkalinity existing above the pH of 8.3 ; while the ml of H_2SO_4 used up the end of second titration, say y, will represent the total alkalinity (since no alkalinity can exist below pH 4.5).

The alkalinity is conventionally expressed in terms of mg of $CaCO_3$ per litre. If $\frac{N}{50}$ H_2SO_4 is used in the titration, then 1 ml of acid will neutralise 1 mg of alkalinity as $CaCO_3$ (having equivalent mass = 50). Hence, alkalinity is given by the equation :

Alkalinity as mg/l of $CaCO_3$

$$= \frac{\text{ml of } \frac{N}{50} H_2SO_4 \text{ used} \times 1000}{\text{ml of water sample}} \quad ...(15.7)$$

Thus, if the titration is done by using 0.02 N.H_2SO_4 in the Burette, and 100 ml of water sample is taken in the flask for titration, as is usually done, then

Alkalinity as mg/l of $CaCO_3$

$$= \frac{x \text{ (or } y) \times 1000}{100} = 10x \text{ ; or } 10y \quad ...(15.8)$$

The alkalinity determined upto pH = 8.3 by phenolphthalein ; *i.e.* $10x$, is usually called the **phenolphthalein alkalinity** and is represented by P. The total alkalinity determined upto pH = 4.5, *i.e.* $10y$, is evidently called the **total alkalinity**, and is represented by T.

Fig. 15.3, shows the graphical representation of the various forms of alkalinity found in water samples. If pH of a sample is less than 8.3, the entire alkalinity (T) will be in the form of bicarbonates [Fig. 15.3 (*e*)], and in such a water sample, addition of phenolphthalein will not cause any pink colour ; and hence $P = 0$. Samples having pH > 8.3, may have either (*i*) hydroxides ; or (*ii*) carbonates ; or (*iii*) hydroxides and carbonates ; or (*iv*) carbonates and bicarbonates, as discussed below.

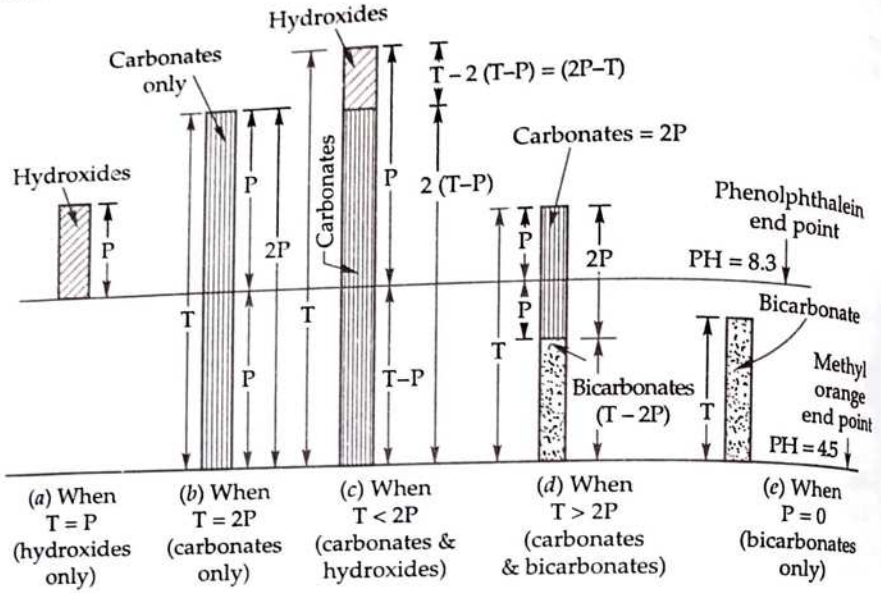


Fig. 15.3. Graphical representation of various forms of alkalinity in water w.r. to its pH values.

(*i*) **Hydroxides only.** Samples containing only hydroxide alkalinity have a high pH, usually above 10. Titration is essentially complete at the phenolphthalein end point. In this case, the hydroxide alkalinity is equal to P, *i.e.* Phenolphthalein alkalinity, as shown in Fig. 15.3 (*a*).

(*ii*) **Carbonates only.** Samples containing only carbonate alkalinity have a pH of 8.3 or more. *The titration to phenolphthalein end point is exactly equal to one half of the total titration.* In this case, carbonate alkalinity is equal to T *i.e.* total alkalinity, which equals to $2P$, as shown in Fig. 15.3 (*b*).

(*iii*) **Hydroxide and carbonate alkalinities both.** Samples containing hydroxide as well as carbonate alkalinity have a high pH, usually above 10. The titration from the phenolphthalein end point to the methyl orange end point represents one-half of the carbonate alkalinity.

$$\therefore \text{ Carbonate alkalinity} = 2(T - P)$$

$$\text{and Hydroxide alkalinity} = T - 2(T - P)$$
$$= T - 2T + 2P$$
$$= (2P - T)$$

This is shown in Fig. 15.3 (*c*).

(*iv*) **Carbonate-bicarbonate alkalinities both.** Samples containing carbonate and bicarbonate alkalinities have a pH > 8.3, and usually less than 11. The titration to the phenolphthalein end point represents one half of the carbonate alkalinity. The carbonate alkalinity is, thus, calculated as equal to $2P$; and bicarbonate alkalinity as equal to $T - 2P$, as shown in Fig. 15.3 (*d*).

(*v*) **Bicarbonate only.** As already discussed, samples having a pH of 8.3 or less, usually less, will have only bicarbonate alkalinity, which will be equal to total alkalinity (T), since in this case, $P = 0$, as shown in Fig. 15.3 (*e*). In this

case, phenolphthalein will not produce any pink colour when added to the water.

Apparatus Required. Titration equipment like a *burette with stand, pipette, conical flasks* (two nos.), *measuring cylinder* (100 ml) ; *white tile* to place the flask on, etc., as shown in Fig. 15.4.

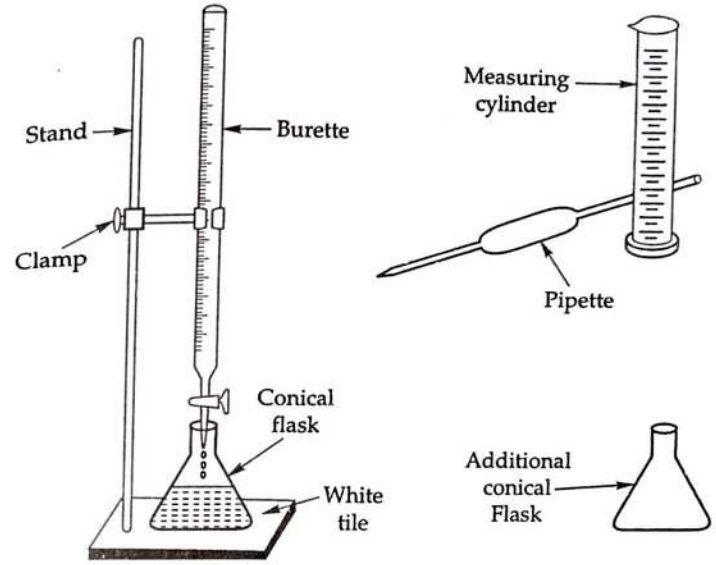


Fig. 15.4. Titration equipment.

Reagents required. Phenolphthalein indicator ; Methyl orange indicator (or Mixed Bromocresol green methyl red indicator) ; and $\frac{N}{50}$ standard sulphuric acid solution for titration (to be filled in the burette).

Procedure

(*i*) If the water sample is turbid, filter the same and proceed.

(*ii*) Take 100 ml water sample in a conical flask. Add 3 to 4 drops of phenolphthalein indicator. If the sample becomes pink, presume that the pH of water is more than 8.3, and it contains phenolphthalein alkalinity (P), and proceed as in (*iii*) below. If pink colour does not appear, note down $P = 0$.

(*iii*) Titrate the pink water in the flask by slowly adding $\frac{N}{50} H_2SO_4$ from the burette, until the pink colour just disappears. Difference of second burette reading and the first burette reading ; *i.e.* say x, will indicate the ml of H_2SO_4 used in neutralising phenolphthalein alkalinity ; and hence $P = 10x$ [Pl. see Eq. 15.7 and 15.8]

(*iv*) Now, add 3 drops of methyl orange indicator to the same flask, and titrate it again with the same standard $\frac{N}{50}$. H_2SO_4. Note the first change in colour from red to 'yellow in orange'. Record this third Burette reading.

Difference between the third Burette reading and the first (initial) burette reading will indicate the total ml of H_2SO_4 consumed ; *i.e.* say y, where $T = 10y$. To detect the change in colour to find out the end point of this titration, one more flask with 100 ml of water sample with 3 drops of methyl orange may be kept aside, as to know the original red colour of the solution.

Mixed Bromocresol green methyl red indicator is also preferably used as a substitute to *methyl orange* indicator, as it changes its colour from bluish-grey at pH 4.8 to light pink at pH 4.6 ; and hence the end point of titration can be better appreciated with this indicator.

Observations. The observations table to be drawn is given below :

Sample No.	*First/ initial burette reading*	*Second burette reading for end pt. of phenol-phthalein*	*Third burette reading for end pt. of methyl orange*	*x in ml (3) – (1)*	*y in ml (4) – (1)*	*P = 10x mg/l 10(5)*	*T = 10y mg/l 10(6)*
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)

Computations and Results. Depending upon the values of T and P computed in col. (8) and (7) above, alkalinities may be calculated as below :

(*i*) When $T = P$, $OH^- = P$, $CO_3^{--} = Nil$, $HCO_3^- = Nil$ ...(15.9)

(*ii*) When $T = 2P$, $OH^- = Nil$, $CO_3^{--} = 2P$, $HCO_3^- = Nil$ ...(15.10)

(*iii*) When $T < 2P$, $OH^- = (2P - T)$, $CO_3^{--} = 2(T - P)$, $HCO_3^- = Nil$ ...(15.11)

(*iv*) When $T > 2P$, $OH^- = Nil$, $CO_3^- = 2P$, $HCO_3^- = T - 2P$ ...(15.12)

(*v*) When $P = 0$, $OH^- = Nil$, $CO_3^{--} = Nil$, $HCO_3^- = T$. ...(15.13)

EXPERIMENT NO. 6

15.6. Objective : To Find out the Concentration of Chlorides in the Given Sample of Water

Theory. Chlorides are found to occur in all the natural waters, and their quantity may vary widely, depending upon various factors. For example, upstream hilly reaches of rivers may have lesser chlorides while the downstream reaches may have more chlorides, since the solvent action of water may dissolve a lot of chloride from the top soil, over which the water travels during its journey. The sewage and industrial wastewaters finding

their way into the river may also increase the chloride content of a raw water, since the human excreta and certain specific industrial wastes may usually contain a large amount of chloride, particularly sodium chloride (common salt). Sea water intrusions and sprays caused during tides into natural surface water sources, may also increase their chloride content.

Chlorides in reasonable concentrations are not harmful to humans. At concentrations above 250 mg/l, they give a salty taste to the water, which is objectionable to most of the people. For this reason, chlorides are generally limited to 250 mg/l in supplies intended for public use. However, in many areas of the world, where water supplies are scarce, sources containing as much as 2000 mg/l of chloride are used for domestic purposes without the development of adverse effects on human health, once the human system becomes adapted to such a water.

The chloride content of water to be used for irrigation is also generally controlled along with the total salinity of the water. Salt sensitive crops may usually require water having chloride concentrations at par or even below the drinking water standards.

The sudden increase of chloride in waters of low chlorides is also an indication of organic pollution. Before, the development of bacteriological testing procedures, chemical tests for chlorides and for nitrogen (in its various forms) were largely used to detect contamination of ground waters by wastewaters. However, this use of chloride testing has now a days been replaced by the more sensitive bacteriological tests.

Chlorides in a given water can be easily measured by volumetric procedures employing internal indicators. Mohr's method, employing *silver nitrate* solution as a titrant, and *potassium chromate* as the indicator is satisfactory. However, the procedure using mercuric nitrate as the titrant and diphenyl carbazone as the indicator, has certain inherent advantages over the Mohr's or Argentometric method, as it is usually called.

The Mohr's method or the *Argentometric method*, which is largely used to measure chlorides in a given water sample is explained below :

The **Mohr's method (*Argentometric method*)** uses a solution of silver nitrate ($AgNO_3$). It is most convenient to prepare this solution with normality $\frac{N}{35.5}$ (*i.e.* 0.0282 N), since its each ml is equivalent to 1 mg of chloride ion (having equivalent wt. equal to 35.5). The silver nitrate solution can be standardized against standard chloride solutions prepared from pure sodium chloride. In the titration, the chloride ion present in water is precipitated as white silver chloride, as :

$$Ag^+ + Cl^- \rightleftharpoons \underset{\text{White ppt.}}{AgCl \downarrow} \qquad ...(15.14)$$

The end point cannot be detected visually, unless an indicator capable of demonstrating the presence of Ag^+ is present. The indicator, normally used is *potassium chromate* (K_2CrO_4), which supplies chromate ions (CrO_4^{--}). As the concentration of chloride ions approaches extinction, the silver-ion concentration increases to a level at which the solubility product of silve chromate is exceeded, and it begins to form a ***reddish-brown precipitate***, a:

$$2Ag^+ + CrO_4^{--} \rightleftharpoons \underset{\text{Reddish brown ppt.}}{Ag_2CrO_4} \downarrow \quad \text{...(15.16)}$$

This is taken as the evidence of all the chlorides having been precipitated. Since an excess of Ag^+ is needed to produce a visible amount of Ag_2CrO_4, the **indicator error** or the **blank** must be determined on distilled water, and subtracted from the titrations performed on the given water sample.

The following **precautions** must be observed in this test, if accurate results are to be obtained :

(*i*) A uniform sample size must be used, so that ionic concentration needed to indicate the end point will be constant. 25 ml samples or 100 ml samples are quite often used.

(*ii*) *The pH of water sample must be in the range of 7 to 8*, because Ag^+ may get precipitated as AgOH at high pH values, and the CrO_4^{--} may get converted to $Cr_2O_7^-$ - at low pH levels.

(*iii*) A definite amount of indicator must be used to provide a certain concentration of CrO_4^{--} ; otherwise Ag_2CrO_4 may either form too soon, or not soon enough.

The *indicator error* or the **blank error** may vary somewhat with the ability of individuals to detect a noticeable colour change. The usual range is 0.2 to 0.4 ml of titrant.

If the $AgNO_3$ solution used for titration is $\frac{N}{35.5}$ (*i.e.* 0.0282 N), the calculation of chlorides can be made by using the following equation :

Chloride ions (Cl^-) in mg/l

$$= \frac{(\text{ml } AgNO_3 - \text{Blank}) \times 1000}{\text{ml of sample}} \quad \text{...(15.16)}$$

Apparatus Required. Burette with stand, pipette, measuring cylinder to measure upto 100 ml. Two conical flasks, white tile to place the flask on, potassium chromate (K_2CrO_4) indicator, $\frac{N}{35.5}$ silver nitrate ($AgNO_3$) solution for titration (to be filled in the burette).

Procedure

(*i*) Take 25 ml* of water sample in a conical flask.

(*ii*) Place the same quantity of chloride free distilled water in another flask, to serve as a blank.

(*iii*) Add to both the flasks, 10 drops of potassium chromate indicator, each.

(*iv*) Titrate the sample as well the distilled water (**blank**) in both the flasks with $\frac{N}{35.5}$ silver nitrate solution.

(*v*) Note the amount of titrant used till *yellow* colour in the flask is turned into *reddish* colour.

*Smaller sample volume is taken in this test to limit the consumption of titrant $AgNO_3$ which is a very costly substance.

Observations

The observations table to be drawn is given below :

S.No.	*Quantity of sample used in ml*	*Test with blank (Distilled water)*			*Test with water sample*		
		Initial burette reading ml	*Final burette reading ml*	*Amount of $AgNO_3$ used in ml (4) – (3)*	*Initial burette reading ml*	*Final burette reading ml*	*Amount of $AgNO_3$ used in ml (7) – (6)*
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)

Calculations. Chloride concentration is determined by the following eqn. :

Chlorides as Cl^-

$$= 1000 \times \frac{\left[\begin{array}{l}\text{ml of } AgNO_3 \text{ used for sample } i.e. \text{ col. (8)} \\ - \text{ml of } AgNO_3 \text{ used for Blank } i.e. \text{ col. (5)}\end{array}\right]}{\text{ml of sample taken } i.e. \text{ col. (2)}} \quad \text{...(15.17)}$$

Result. The concentration of chlorides is found to be ..... mg/l. **Ans.**

EXPERIMENT NO. 7

15.7. Objective : To Find out the Hardness of the Given Sample of Water by Standard E.D.T.A. Method

Theory. Hardness in water is that characteristic which prevents the formation of sufficient foam (or leather), when such hard waters are mixed with soap. The hardness is usually caused by the presence of calcium and magnesium salts* present in water, which form scum by reaction with soap.

Hard waters are undesirable because they may lead to greater soap consumption, scaling of boilers, erosion and incrustation of pipes, and making food tasteless, etc.

*In addition to Ca^{++} and Mg^{++}, other divalent metallic ions like Fe^{++}, Mn^{++}, Sr^{++} etc. may cause hardness, but their presence in most of the waters is generally negligible, and hence hardness for all practical purposes is attributed to Ca^{++} and Mg^{++} only.

The *bicarbonates* and *carbonates* of the divalent metallic ions, principally of Ca^{++} and Mg^{++}, cause **carbonate hardness** (earlier called ***temporary hardness***), while their *sulphates, chlorides* and *nitrates* cause **non-carbonate hardness** (earlier called ***permanent hardness***).

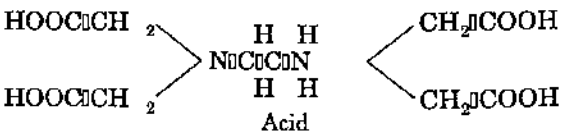
Hardness of water is usually measured in mg/l of $CaCO_3$, and depending upon their hardness, the waters are usually classified as follows :

S. No.	*Category of hardness*	*Hardness in mg/l*
1.	Soft	0—75
2.	Moderately hard	75—150
3.	Hard	150—300
4.	Very hard	> 300

The hardness of a given water can, most precisely, be computed by carrying out a ***complete cation analysis***, as to determine the quantum of each divalent cation in the water. The hardness caused by each cation can then be computed and totalled up to determine the total hardness as $CaCO_3$, as explained under Eq. 8.1 and 8.1 (*a*). Such a complete cation analysis, however, involves complicated methods, such as *standard atomic absorption* ; or the *inductively coupled plasma procedures* ; or *ion chromatography* ; or *ion-specific electrodes procedure*, etc. It has also been shown that computation of Sr^{++} is also vitally important, to compute total hardness, since many waters are found to contain Sr salts, and its non inclusion in computing total hardness may lead to errors.

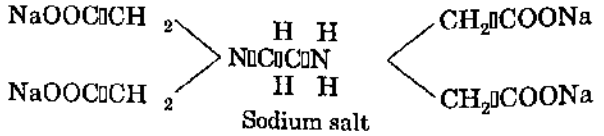
The total hardness of a given sample of water can however, be easily computed by titration method, by using the *E.D.T.A. (Ethylene-diamine-tetra-acetic acid)* solution, or its sodium salt [*i.e.* Disodium ethylene diamine tetra acetate, [(Na_2 EDTA)] as the titrating agent. The chemical formulas of these ***titrating agents*** are given as under :

(*i*) *EDTA acid*



Acid

(*ii*) *EDTA salt*



Sodium salt

These compounds, usually represented by EDTA, are chelating agents and form extremely stable complex ions with Ca^{++}, Mg^{++} and other divalent metallic ions causing hardness, as shown in the equation :

$$M^{++} + EDTA \longrightarrow [M \,.\, EDTA]_{complex} \quad ...(15.18)$$

The successful use of EDTA for determining hardness depends upon having an indicator present to show when EDTA becomes in excess, or when all the

ions causing hardness have been used up to form the complex compounds of the divalent ions.

The *indicator dye* which is found to serve as an excellent indicator to show when all the hardness ions have been complexed, is known as the **Eriochrome Black T.**

When a small amount of Eriochrome Black *T*, having a blue colour, is added to a hard water with a pH of about 10.0, it combines with divalent metallic ions to form a weak complex ion, which is *wine red* in colour, as shown in the following equation :

$$M^{++} + \text{Eriochrome Black } T \longrightarrow \underset{\text{Wine red}}{(\text{M. Eriochrome Black } T)_{complex}} \quad ...(15.19)$$

During the titration with EDTA, all free hardness causing ions are complexed according to the equation (15.18). Finally, the EDTA disrupts the wine red (M. Eriochrome Black $T)_{complex}$, because it is capable of forming a more stable complex with the hardness ions. This action frees the Eriochrome Black *T* indicator, *and the wine red colour changes to a distinct blue colour, showing the end point of titration.*

If $\frac{N}{50}$ EDTA is used, then 1 ml of the titrant used measures 1 mg of hardness as $CaCO_3$.

∴ Hardness in mg/l as $CaCO_3$

$$= \frac{\text{ml of } \frac{N}{50} \text{ EDTA Titrant}}{\text{ml of water sample}} \times 1000 \quad ...(15.20)$$

Although the EDTA method is subject to certain interferences, but most of them can be overcome by proper modifications. The method yields very precise and accurate results. It, hence, is the most preferred method in most of the laboratories, at the present time.

Apparatus Required. The *titration apparatus* including the burette with stand, pipette, two conical flasks, measuring cylinder, white tile to place the flask on. *Standard* $\frac{N}{50}$ *EDTA solution, Eriochrome Black T indicator, Ammonia buffer* (to raise pH of water to about 10).

Procedure

(*i*) Take 100 ml of water sample in a conical flask.

(*ii*) Add one ml of Ammonia buffer to the flask.

(*iii*) Add 6 drops of Eriochrome Black *T* indicator to the flask. Wine red colour will be developed.

(*iv*) Titrate it with standard EDTA solution (to be filled in burette) till the colour changes from *wine red* to *blue*.

Calculations. The total hardness of water in mg/l is given by the Eq. :

Hardness in mg/l as $CaCO_3$

$$= \frac{\text{ml of EDTA used} \times \frac{N}{50} \times 50 \times 1000}{\text{ml of water sample}} \quad ...(15.21)$$

or Hardness in mg/l as $CaCO_3$

$$= \frac{\text{ml of } \frac{N}{50} \text{ EDTA used} \times 1000}{\text{ml of water sample}} \quad (i.e.\ \text{Eq. } 15.20)$$

Observations. The observation table to be drawn is given below :

Sample S. No.	*ml of water sample used in flask*	*Initial Burette reading ml*	*Final Burette reading ml*	*ml of $\frac{N}{50}$ EDTA used in ml (4) – (3)*	*Hardness (mg/l) as $CaCO_3 = \frac{(5)}{(2)} \times 1000$*
(1)	(2)	(3)	(4)	(5)	(6)

Result. The hardness of the given sample of water is found to be ..... mg/l. Ans.

EXPERIMENT NO. 8

15.8. Objective : To Determine the Residual Chlorine in a Given Sample of Water by

(1) Orthotolidine test ;

(2) Starch Iodide test ; and

(3) DPD test

Theory. When a filtered water is chlorinated by using *chlorine* (as liquid chlorine or chlorine gas), or *bleaching powder*, or *hypochlorites*, etc., as discussed in article 9.24, the chlorine shall initially be consumed in killing bacteria etc. and then in oxidising the organic matter. When this oxidation is completed ; *i.e.* when the break point is reached (Fig. 9.42), then whatever chlorine is added to the water, it appears as *residual chlorine*.

This residual chlorine is measured as the chlorine left in water after the required *contact period*, which will ensure complete killing of bacteria and oxidation of the organic matter. Usually, a free chlorine residue of 0.2 to 0.3 mg/l after a contact period of 10-20 minutes is considered to be sufficient and satisfactory to take care of the future contamination of water to be supplied through the distribution system. Hence, for purified waters, residual chlorine is tested, usually by *orthotolodine test* or by *starch iodide test or* by *DPD test*, as discussed below :

15.8.1. Orthotolidine Test. Orthotolidine test generally uses a **comparator** instrument, called a **chloroscope**, which helps in comparing the colour developed in water with orthotolidine reagent, with the standard colours developing at different values of free chlorine, available in the form of slides.

Hence, in an orthotolidine test, the chlorinated water, after the given contact period of say 10 minutes, is mixed with orthotolidine solution. The colour formed after 5 minutes is noted. The formation of yellow colour confirms the presence of free chlorine. The more yellow is the colour, the greater is the chlorine residual. Hence, the developed colour in the given sample is compared with the standard colours to estimate the free residual chlorine in the given water sample, by colour matching technique.

Apparatus Required

(*i*) Comparator box *i.e.* chloroscope ; and

(*ii*) Orthotolidine solution (standard).

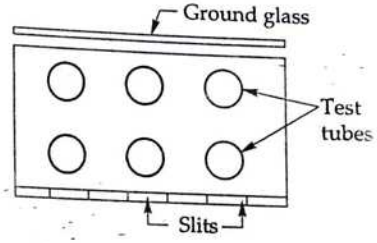


Fig. 15.5. Plan of a comparator box or chloroscope.

Procedure

(*i*) Fill the three test tubes in the front row of the comparator box with the given water sample up to the given markings.

(*ii*) Add 6 drops (0.5 ml) of standard orthotolidine standard solution in the middle test tube and mix with a stirrer.

(*iii*) A yellow colour will be developed indicating the presence of free chlorine.

(*iv*) Fill distilled water in the central tube of the back row.

(*v*) Take two standard colour samples in side tubes of the back row.

(*vi*) View from front side against sufficient white or natural light.

(*vii*) Compare the colour developed in the water sample (after 5 minutes of addition of orthotolidine) with the standard colours. The standard colours in two side tubes of back row may be changed till a good matching of colour is obtained. The known value of chlorine in mg/l contained in the standard colour, which matches with the colour developed in the water sample, will indicate the free residual chlorine in the water sample.

Observations

S. No.	*Sample*	*Residual chlorine in mg/l*
1.		
2.		
3.		
Average		

Result. The residual chlorine is found in the given water sample to be ..... mg/l. **Ans.**

Note. The orthotolidine colourimetric indicator method described above was developed by Mr. Phelps in 1908. Due to its simplicity, this method became quite popular, but is found not to be giving precise results. Moreover, orthotolidine is now known to be a toxic compound with corcinogenic potential. Hence, the use of this method as a standard procedure has been eliminated from the "Standard Methods".

15.8.2. Starch Iodide Test. The starch iodide test is an age old method for testing the total chlorine residual in a given water sample, and is still being used, depending upon the oxidising power of free and combined chlorine residuals to convert iodide ion to free iodide. This reaction is represented as follows:

$$\underset{\text{Chlorine}}{Cl_2} + \underset{\text{Iodide ions}}{2I^-} \longrightarrow \underset{\text{Iodine (yellow colour)}}{I_2} + 2Cl^- \qquad ...(15.22)$$

In the presence of starch, the iodine produces a blue colour, which is taken as the evidence of residual chlorine.

$$I_2 + \text{Starch} \longrightarrow \text{Blue colour} \qquad ...(15.23)$$

The intensity of blue colour produced can indicate the amount of chlorine residual. Iodine ions can be added by adding potassium iodide and acetic acid to the given water.

The total amount of chlorine residual present in the given water can also quantitatively be measured by titrating the iodine released with a standard solution of a reducing agent, such as *sodium thiosulphate*. The end point of titration is indicated by the disappearance of the blue colour. This reaction is indicated below :

$$\underset{\text{Free iodine}}{I_2} + \underset{\text{Sod. thiosulphate}}{Na_2S_2O_3} \longrightarrow Na_2S_4O_6 + \underset{\substack{\text{Sod. iodide to liberate iodine ions}}}{2NaI} \qquad ...(15.24)$$

or

$$\underset{\text{Free iodine}}{I_2} + 2S_2O_3^{--} \longrightarrow S_4O_6 + 2I^- \qquad ...(15.25)$$

Procedure

(*i*) Take 25 ml of given water sample in a conical flask.

(*ii*) Add a small crystal of *Potassium Iodide* and distilled water to the above flask containing water sample, to make 100 ml. The potassium iodide will produce iodine ions in solution, which will react with Cl_2 to form I_2.

(*iii*) Add about 0.5 ml of concentrated hydrochloric acid, or about 10 ml of acetic acid to act as buffer to reduce the pH to a low value between 3.5 to 4.2 to avoid conversion of Cl_2 into HOCl and OCl^-.

(*iv*) Allow the reaction to complete. It will turn yellow. The yellow coloured iodine solution is now to be titrated against the standard sodium thiosulphate (in burette), as to obtain the quantity of iodine contained in the above solution. When sodium thiosulphate titrant is slowly added to the above iodine solution, the yellow colour will slowly disappear, and liquid will change to colourless. However, the change from yellow colour to colourless can not be precisely determined ; and hence an ***end point indicator*** in the form of ***soluble starch solution*** is used near the end of titration, which initially is carried out without the starch solution indicator till the original yellow colour changes to light yellow. The end point indicator will change the light yellow solution to blue, and titration can be continued till blue colour just disappears. Hence, the test is further proceeded as below :

(*v*) Titrate the above yellow coloured iodine solution against the standard $\frac{N}{40}$ sodium thiosulphate solution (taken in the burette) till yellow colour becomes light yellow or faintly yellow.

(*vi*) Add 1 ml of starch (end point indicator) to the conical flask to produce blue colour.

(*vii*) Continue the titration with $\frac{N}{40}$ sodium thiosulphate till the blue colour just disappears.

(*viii*) Note down the total ml of titrant used in the entire titration. Let it be x ml.

(*ix*) Using distilled water in place of water sample, repeat the entire exercise given above in step (*i*) to (*viii*), and determine the amount of sodium thiosulphate used in titration. Let it be y ml.

Residual chlorine in 25 ml of water sample, originally used in the test after being diluted to 100 ml, can finally be computed as under :

Residual chlorine in diluted sample

$$= \frac{(x-y)\times 1000}{\text{ml of diluted sample}} \times \left[\frac{N}{40}\times 35.5\right] \quad \because \text{ EW of } Cl_2 = 35.5 \qquad ...(15.26)$$

Residual chlorine in the given water will be 4 times the above value, since the water sample of 25 ml was diluted to make 100 ml.

∴ Residual chlorine in original water sample

$$= \frac{(x-y)\times 1000\times \frac{N}{40}\times 35.5}{\text{ml of diluted sample, } i.e.\ 100 \text{ ml}} \times \frac{\text{ml of diluted sample}}{\text{ml of water sample}}$$

$$= \frac{(x-y)\times 1000\times 0.8875}{\text{ml of water sample, } i.e.\ 25 \text{ ml}} \qquad ...(15.27)$$

$$= (x-y)\ 35.5 \text{ mg/l} \qquad ...(15.28)$$

Observations

Strength of Sodium Thiosulphate used in burette = N/40

S. No.	*Quantity of water sample used in ml*	*Test on water sample*			*Test on distilled water*			*Total amt. of residual chlorine* $\frac{(x-y)\ 887.5}{Col\ (2)} = (x-y)\ 35.5$ *for 25 ml of water sample mg/l*
		Initial burette reading ml	*Final burette reading ml*	*Amt. of N/40 Sod. Thio-sulphate used in titration in ml ; say x (4) – (3)*	*Initial burette reading in ml*	*Final burette reading in ml*	*Amt. of N/40 Sod. Thio-sulphate used in titration in ml ; say y (7) – (6)*	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)

Result. The amount of residual chlorine is ..... mg/l. **Ans.**

15.8.3. DPD Test. The DPD method is one of the modern methods available to compute the residual chlorine in water. It can, infact, determine not only the total chlorine residual in water, but the individual chlorine residues of free chlorine, and that of the combined chlorine. Since the combined chlorine may exist either as mono-chloramines or as dichloramines, the test can even determine their individual residuals. This test is based and conducted on the following principles :

When N-diethyl-*p*-phenylene diamine (DPD) reagent* is added to a water sample containing *free chlorine residual*, an instantaneous reaction occurs, producing a *red colour*. The intensity of the red colour can be measured in the neutral pH range of 6.2 to 6.5, as to tell us the *amount of free chlorine* present in water. This optimum pH range of 6.2 to 6.5 is made by adding a *phosphate buffer solution* to the sample water, while mixing with DPD reagent-solution. This estimation of red colour, measuring free chlorine, say x, can be made either by colour matching techniques, or by titrating the water (having red colour) with *ferrous ions*, by using *Ferrous Ammonium Sulphate (FAS)*, until the red colour disappears. This will constitute the first part of the test.

When a small amount of iodide is added to the above red coloured solution (obtained in 1st part of the test), the iodine ion acts catalytically with monochloramines existing in the water sample, to produce iodine, which inturn, oxidises more DPD, to form additional red colour. The intensity of red colour obtained above in the second part of the test, say y, will evidently measure the free chlorine plus mono-chloramines. If this measurement is y, then mono-chloramines will be equal to $y - x$.

Now, if a large quantity of iodide is further added to the above solution (obtained in second part of the test), then *dichloramines* will also react to form still more red colour. The intensity of this red colour, say z, will evidently give the sum total of free chlorine, chloramines and dichloramines. The dichloramines can then be easily worked out as $z - y$.

Hence,

free chlorine	$= x$
monochloramines	$= y - x$
dichloramines	$= z - y$
combined chlorine	$= z - x$
total chlorine	$= z$

Ans.

The above test is relatively an easier one, and can help in estimating chlorine residuals of different forms of chlorine.

In modern days, comparator instruments such as a **BDH Comparator****, has also been manufactured to help compare the red colours developed in the above test (at three stages), as to easily and directly read out the colour intensities in terms of chlorine residuals in mg/l. The use of such a comparator will help avoid the titration process.

To perform this test with a BDH comparator, **10 ml** of water sample is taken in a tube, and is placed in the left compartment of the comparator. A reagent tablet (DPD chlorine tablet) is placed in another tube, to which is added 1 cm

*It is usually available in the form of water soluble DPD chlorine tablets.

**Comparator manufactured by British Drug House.

depth of sample water, and the tablet is allowed to dissolve. More sample water is now added to this, until the tube contains 10 ml. *Phosphate buffer solution* is also added to keep the pH of the water solution to optimum value of 6.2 to 6.5. This tube is placed in the right hand compartment of the comparator. The red colour developed instantaneously in the second tube is viewed against the sample water, and difference in colour intensity is viewed through the viewing window, as to compare the developed colour with the standard colours available on the disc of the comparator. The disc is infact rotated till the colours match. The residual free chlorine amount (x) can then be directly read in the window in the lower right hand corner of the instrument.

This comparator is usually provided with a special dulling screen provided with a disc, which must be fitted to the right hand viewing window of the comparator. The BDH discs will be available in two alternatives, for two different chlorine concentration ranges, *i.e.*

(*a*) 3/40/A 0.1 to 10 mg/l of chlorine

(*b*) 3/40/B 0.2 to 4.0 mg/l of chlorine.

The second part of the test can be conducted as described earlier by adding a small amount of iodide (potassium iodide crystal) to the red coloured solution (tube in right hand compartment) and again determining the chlorine concentration of darker red colour as y. The third part of the test can finally be conducted by adding a large amount of iodide to the red solution tube, to further increase the intensity of colour, which can again be measured in the instrument by colour matching, as to give the value of z. The residuals of free chlorine, chloramines, and dichloramines will thus be individually determined as x, $y - x$ and $z - x$; the total chlorine being equal to z. **Ans.**

Note. Proper observation table may be drawn as required for noting down the above observations.

EXPERIMENT NO. 9

15.9. Objective : To Determine the Chlorine Demand of a Given Sample of Water

Theory. The chlorine demand of a water sample will be equal to the chlorine required to be added to the water sample, as to just make the free chlorine available in the water sample, after a contact period of say 30 minutes.

Procedure

(*i*) Place 200 ml of the well mixed water samples in each of the 10 No.—250 ml bottles.

(*ii*) Add 0.5 ml of standard $\frac{N}{35.5}$ *chlorine water* to the first bottle, 1.0 ml to the second, 1.5 ml to the third, and so on, as to add 5 ml to the tenth bottle.

(*iii*) Shake each bottle gently and allow to stand for 30 minutes of contact period.

(*iv*) Add a crystal of potassium iodide and 1 ml of concentrated hydrochloric acid to each bottle.

(*v*) Add 1 ml of starch solution to each bottle.

(*vi*) Identify that bottle (out of 10 bottles), which contains the least blue colour (rather just appearing blue colour). The chlorine water added in this particular bottle may be recorded. Let it be x ml (it will be one of the figs between 0.5 ml, 1.0 ml, 1.5 ml ..... 5 ml). This figure x multiplied by the sample volume (200 ml) divided by 1000 ml (1 litre) will give the chlorine demand of water in mg/l, as :

Chlorine demand of water in mg/l

$$= \frac{\text{ml of } \frac{N}{35.5} \text{ chlorine water in the identified bottle} \times 1000}{\text{ml of water sample, } i.e.\ 200 \text{ ml}} \quad ...(15.29)$$

$$= \text{ml of chlorine water} \times 5 \quad ...(15.30)$$

The above computed value of chlorine demand upto break point chlorination, will be further increased by 0.2 to 0.3 mg/l for ensuring free chlorine to take care of subsequent contamination in the distribution system, as to compute the total chlorine demand of the water. **Ans.**

EXPERIMENT NO. 10

15.10. Objective : To Determine the Available Chlorine Percentage in a Given Sample of Bleaching Powder

Theory. In order to work out the disinfecting power of bleaching powder and its requirement for treating a given water having a particular chlorine demand, we have to work out the chlorine content in the given bleaching powder. The procedure adopted to perform this test is explained below :

Procedure

(*i*) Fill a clean and dry weighing bottle, about half-full, with the given bleaching powder. Insert the stopper and weigh the bottle. Let its mass be M_1.

(*ii*) Place about 100 ml of distilled water in a beaker.

(*iii*) Carefully remove the stopper from the weighing bottle, and pour from the bottle, about 0.7 gm of the powder into the beaker.

(*iv*) Insert the stopper and weigh the bottle again. Let its mass be M_2. Note down the difference in mass M as equal to $M_1 - M_2$, as the gm of bleaching powder placed in the beaker.

(*v*) Thoroughly mix the powder and distilled water in the beaker, and pour the solution with several rinsings into a 200 ml *volumetric flask*.

(*vi*) Fill the 200 ml flask with distilled water upto the mark, as to make the *chlorine solution* equal to 200 ml. Mix thoroughly.

(*vii*) Dissolve about 2 gm of potassium iodide and 2 ml of glacial acetic acid in 2 ml of distilled water in a *conical* (Erlenmeyer) *flask*.

(*viii*) Pipette out 25 ml of the chlorine solution from the volumetric flask, and add it into the above flask, containing iodide acetic acid mix.

(*ix*) Add a few drops of starch indicator to the conical flask to develop blue colour.

(*ix*) Titrate this blue solution in the conical flask against the $\frac{N}{40}$ sodium thiosulphate, filled in the burette on stand.

(*x*) Go on adding standard sodium thiosulphate solution slowly to the flask, till the blue colour in the flask just disappears.

(*xi*) Note down the ml of sodium thiosulphate solution used in the above titration. It may be averaged out by performing the above test twice or thrice. Let this value works out to x ml.

(*xii*) Chlorine in chlorine solution is then computed as follows :

Calculations

Chlorine in mg/l in chlorine solution

$$= \frac{\text{ml of } \frac{N}{40} \text{ sodium thiosulphate used } (i.e.\ x) \times 35.5 \times 1000 \times \frac{N}{40}}{(\text{ml of chlorine solution} = 25 \text{ ml})} \quad ...(15.31)$$

$$= \left[\text{ml of } \frac{N}{40} \text{ sodium thiosulphate, } (i.e.\ x) \times 35.5\right] \quad ...(15.32)$$

The chlorine solution has been prepared by us by mixing M gm of bleaching powder in 200 ml of chlorine solution, which means 5 M gm of powder per litre, or 5000 M mg/l of powder.

$\therefore$ % age chlorine in sample

$$= \frac{\text{mg/l of chlorine}}{\text{mg/l of powder}} \times 100$$

$$= \frac{\text{ml of sodium thiosulphate, } (i.e.\ x) \times 35.5}{5000\ \text{M}} \times 100 \quad ...(15.33)$$

$$= \frac{\text{ml of thiosulphate, } (i.e.\ x) \times 0.71}{\text{gm of sample used (M)}} \% = \frac{0.71x}{\text{M}}. \quad \textbf{Ans.} \quad ...(15.34)$$

Observations

Initial mass of weighing bottle containing bleaching powder	$= M_1$ gm
Final mass of weighing bottle containing bleaching powder (after its removal for preparing chlorine solution)	$= M_2$ gm
Mass of powder used in preparing chlorine solution in beaker and then into volumetric flask	$= M$ gm $= (M_1 - M_2)$
Volume of chlorine solution in volumetric flask	= 200 ml
Volume of chlorine solution pipetted out into the conical flask for titration	= 25 ml

Titration Readings

S. No.	*Initial burette reading at start of titration ml*	*Final burette reading at end of titration ml*	*Vol. of titrant i.e. M/40 sodium thiosulphate used in titration upto end point (ml) (3) – (2) ; say x*	*%age of chlorine in powder sample* $= \frac{0.71x}{M}\%$
(1)	(2)	(3)	(4)	(5)

The final computation is done by using the Eq. derived upon ; *viz.*

% age of chlorine in powder sample = $\frac{0.71x}{M}$ % (*i.e.* Eq. 15.34) as shown in col. (5) above

Result. The percentage chlorine content of the given bleaching powder works out to be .....%. **Ans.**

EXPERIMENT NO. 11

15.11. Objective : To Determine the Amount of Dissolved Oxygen (D.O.) in the Given Sample of Water by Winkler Method

Theory. All the gases of the atmosphere are soluble in water to some degree. Both nitrogen and oxygen are classed as poorly soluble, and since they donot react with water chemically, their solubility is directly proportional to their partial pressures. Hence, Henry's law may be used to calculate their amounts present at saturation at any given temperature. The amount of oxygen found dissolved in a given water at a given temperature and pressure is known as the D.O. value.

The saturation D.O. value is the maximum dissolved oxygen which a given water can contain at the given temperature and pressure. *The saturation D.O. decreases with an increase in temperature*, as can be seen from Appendix Table A-6 at the end of the book. It can be seen that whereas the saturation D.O. value at 0°C is 14.62 mg/l ; its value at 20°C is 9.17, and at 30°C its value is 7.63 mg/l. These values are at one atmospheric pressure (760 mm of Hg or 101.325 kPa) and for non-salty water. *The D.O. value (or solubility of O_2) increases with increase in pressure ; and reduces with increase of chloride concentration in water.* Thus, for sea water, which normally contains about 19,000 mg/l of salt (NaCl), the maximum D.O. (saturation D.O.) at 0°C is about 11.2 mg/l as against the value of 14.62 mg/l for non salty water ; and at 30°C, its saturation D.O. is around 6.0 mg/l as against the value of 7.63 mg/l for non-salty water.

Unpolluted surface waters are usually found to contain large amounts of dissolved oxygen (upto saturation value or so), because they absorb it from the atmosphere. Algae and other tiny plants growing in water also give oxygen to the water. The presence of dissolved oxygen in water is necessary to keep it fresh and sparkling. Addition of organic pollution to the water often reduces its D.O., since oxygen is consumed by the bacteria to decompose and stabilize the organic matter. *Fish and other forms of aquatic life in natural surface waters usually need a minimum D.O. of about 4 mg/l*, as otherwise they may be stifled by the lack of oxygen, to die down. Much higher values of D.O. are also not good, since *water containing high D.O. may cause corrosion of the pipes* supplying such waters. For domestic supplies, D.O. value of about 4—8 mg/l may hence suffice.

The D.O. testing is very necessary in environmental engineering since D.O. tests will be required to compute the pollution level of a given water source. D.O. tests are also required to determine BOD of wastewaters. The rate of air supply to aerobic treatment processes is also monitored by conducting D.O. tests, as to maintain aerobic conditions and to prevent wastage of power by excessive aeration. To avoid corrosion of boilers, sometimes, excess oxygen is removed from the available water ; and this can be done only by D.O. testing.

The **Winkler's method** is generally adopted to determine D.O. of a given water. The original test given by Winkler has, however, slightly been modified, since the original test fails to give reliable results in case of waters containing substances like nitrite ions, ferric ions, etc. Such oxidising agents are capable of oxidising Iodine ions (I^-) to iodine (I_2), which in the original test, is to be done solely by MnO_2 formed by dissolved oxygen ($\frac{1}{2}$ O_2), and the I_2 formed in the test has to represent the D.O. value of water. Such interfering oxidising ions like Nitrite ions may usually be found in the effluents of wastewater treatment plants employing biological treatments. The original test is, thus, applicable only to relatively pure waters.

(*a*) In the **original Winkler method**, the given water is mixed with Manganese Sulphate ($MnSO_4$) and an *alkali-iodide reagent* (NaOH + KI). The Mn^{++} ions released from $MnSO_4$ react with OH^- ions of water, to form white ppt, if no D.O. is present in water ; or to form red ppt. if D.O. is present in water, as shown below :

$$Mn^{++} + 2OH^- \longrightarrow \underset{\text{White ppt.}}{Mn(OH)_2} \downarrow \text{(in the absence of D.O.)} \quad ...(15.35)$$

$$Mn^{++} + 2OH^- + \underset{\text{D.O.}}{\tfrac{1}{2}O_2} \longrightarrow \underset{\text{Red ppt.}}{MnO_2} \downarrow + H_2O \text{ (in the presence of D.O.)} \quad ...(15.36)$$

The oxidation of Eq. (15.36) which occurs in the presence of D.O. is known as ***fixation of oxygen***, and occurs slowly, particularly at low temperatures. Additionally, it is necessary to move the flocculated matter throughout the solution to enable all the oxygen to react. *Vigorous shaking of the sample for atleast 20 seconds is hence needed in the test. In the case of brackish or sea waters, much longer contact period is required.*

The MnO_2 (red ppt.) formed above with the help of available D.O. in the water sample under low pH conditions, and H^+ ions (created in the test by the addition of concentrated H_2SO_4) oxidise Iodine ions (I^-) released by the alkali-iodide reagent (NaOH + KI) to Iodine (I_2) as indicated below :

$$MnO_2 + \underset{\substack{\text{Iodine ions}\\ \text{from the alkali}\\ \text{iodide reagent}}}{2I^-} + \underset{\substack{\text{Hydrogen}\\ \text{ions released}\\ \text{by } H_2SO_4}}{4H^+} \xrightarrow{\text{Low pH}} Mn^{++} + \underset{\text{Iodine}}{I_2} + 2H_2O \quad ...(15.37)$$

Iodine (I_2) is rather insoluble in water, but forms a complex with the excess iodide present to reversibly form the more soluble tri-iodate, thus preventing escape of I_2 from solution, as :

$$I_2 + I^- \rightleftharpoons I_3^- \quad ...(15.38)$$

The amount of I_2 formed in the test solution can finally be computed by titrating the solution against the standard $\frac{N}{40}$ sodium thiosulphate, as was done in Starch Iodide Test of Experiment No. 8 (Article 15.8.2). The quantity of I_2 in the test is equivalent to the dissolved oxygen ($\frac{1}{2}$ O_2, having equivalent wt. = 8) in the original sample. The D.O. value is finally computed by using the equation given below :

$$\text{D.O. in mg/l} = \frac{\text{Ml of } \frac{N}{40} \text{ Sod. thiosulphate used} \times \frac{N}{40} \times 8 \times 1000}{\text{Ml of water sample}}$$

$$= \frac{\text{Ml of } \frac{N}{40} \text{ Sod. thiosulphate used } (x) \times 200}{\text{Ml of water sample}} = \frac{x \times 200}{100} = 2x \quad ...(15.39)$$

(*b*) In the **Modified Winkler Test**, interference from Nitrite ions, etc. is prevented by adding **Sodium Azide (NaN_3)** to the given water, which helps in destroying the harmful NO_2^- ions, which, otherwise, would have oxidised I^- (ions) to I_2 and consumed some D.O., as shown below :

$$2NO_2^- + 2I^- + 4H^+ \longrightarrow I_2 + N_2O_3 + 2H_2O \quad ...(15.40)$$

$$N_2O_2 + \tfrac{1}{2}O_2 + H_2O \longrightarrow 2NO_2^- + 2H^+ \quad ...(15.41)$$

Destruction of harmful NO_2^- ions by Azide is also shown below :

$$NaN_3 + H^+ \longrightarrow HN_3 + Na^+ \quad ...(15.42)$$

$$HN_3 + NO_2^- + H^+ \longrightarrow N_2 + N_2O + H_2O \quad ...(15.43)$$

Sodium azide is most conveniently incorporated in the *alkali-KI reagent*; and the produced reagent is known as **Azide-Alkali iodide reagent**. This reagent is hence used in the test for all practical purposes to obtain precise results.

Apparatus Required. 300 mm BOD bottle ; conical flasks ; 25 cc burettes ; 400 cc measuring cylinder ; white tile ; five pipettes ; etc.

Reagents required. (*i*) Standard manganese sulphate ($MnSO_4$) ; (*ii*) Azide alkali iodide reagent (NaN_3 + NaOH + KI) ; (*iii*) $\frac{N}{40}$ Sodium Thiosulphate ; (*iv*) Starch end point indicator solution ; (*v*) Concentrated sulphuric acid ; etc.

Procedure

(1) Fill the given water sample in a glass stoppered 300 ml BOD bottle. Be careful to avoid contact of the sample with air. The bottle should be completely filled.

(2) Immediately after filling the BOD bottle with the given water, add 2 ml of manganese sulphate solution by means of a pipette, dipping the end of the pipette just below the surface of water. Since the bottle was already full, 2 ml of water will be spilled out of the bottle, for which correction will have to be made while taking out known volume of water for testing.

(3) Add 2 ml of *Azide Alkali Potassium Iodide* in a similar manner. The addition of this reagent will also displace 2 ml of water from the bottle, for which correction will have to be made while taking out known volume of water for testing.

(4) Insert the stopper with care to exclude air bubbles, and mix by repeatedly inverting and shaking the bottle vigorously.

(5) Red precipitate will form if D.O. is present in water. Allow the precipitate to settle half way, and mix again.

(6) Again allow the precipitate to settle half way.

(7) Add 2 ml of concentrated sulphuric acid to the bottle, in the same manner as done in step 2 and 3. This will not displace any oxidised floc and, thus, no correction need be made for its addition. Insert the stopper at once, and mix up thoroughly as before.

(8) Allow the solution to stand for atleast 5 minutes to ensure formation of I_2, which is to be titrated against sodium thiosulphate.

(9) Withdraw 101.5 ml* of this solution in the conical titration flask, and immediately add $\frac{N}{40}$ sodium thiosulphate solution drop by drop from the burette, alter recording the initial burette reading, until the yellow colour becomes very light.

(10) Add 1 ml of starch solution (end point indicator). This will give blue colour. Now, continue the titration by adding sodium thiosulphate drop by drop, till the blue colour just disappears. Record the final burette reading, as to record the ml of thiosulphate used in the titration.

Observations and calculations

Sample size = 100 ml of original sample.

S. No.	*Initial burette reading (ml)*	*Final burette reading (ml)*	*ml of thiosulphate used ; (3) – (2) ; say x*
(1)	(2)	(3)	(4)
			Concordant reading = *x* ml

*Since 4 ml of water has been lost from 300 ml BOD bottle due to addition of reagents, the original water of **100 ml** will be found to be contained in **101.5 ml** of solution in the BOD bottle.

The MnO_2 (red ppt.) formed above with the help of available D.O. in the water sample under low pH conditions, and H^+ ions (created in the test by the addition of concentrated H_2SO_4) oxidise Iodine ions (I^-) released by the alkali-iodide reagent (NaOH + KI) to Iodine (I_2) as indicated below :

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$$\text{D.O. in mg/l} = \frac{\text{Ml of } \frac{N}{40} \text{ Sod. thiosulphate used} \times \frac{N}{40} \times 8 \times 1000}{\text{Ml of water sample}}$$

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Sodium azide is most conveniently incorporated in the *alkali-KI reagent*; and the produced reagent is known as **Azide-Alkali iodide reagent.** This reagent is hence used in the test for all practical purposes to obtain precise results.

Apparatus Required. 300 mm BOD bottle ; conical flasks ; 25 cc burettes ; 400 cc measuring cylinder ; white tile ; five pipettes ; etc.

Reagents required. (*i*) Standard manganese sulphate ($MnSO_4$) ; (*ii*) Azide alkali iodide reagent (NaN_3 + NaOH + KI) ; (*iii*) $\frac{N}{40}$ Sodium Thiosulphate ; (*iv*) Starch end point indicator solution ; (*v*) Concentrated sulphuric acid ; etc.

Procedure

(1) Fill the given water sample in a glass stoppered 300 ml BOD bottle. Be careful to avoid contact of the sample with air. The bottle should be completely filled.

(2) Immediately after filling the BOD bottle with the given water, add 2 ml of manganese sulphate solution by means of a pipette, dipping the end of the pipette just below the surface of water. Since the bottle was already full, 2 ml of water will be spilled out of the bottle, for which correction will have to be made while taking out known volume of water for testing.

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(4) Insert the stopper with care to exclude air bubbles, and mix by repeatedly inverting and shaking the bottle vigorously.

(5) Red precipitate will form if D.O. is present in water. Allow the precipitate to settle half way, and mix again.

(6) Again allow the precipitate to settle half way.

(7) Add 2 ml of concentrated sulphuric acid to the bottle, in the same manner as done in step 2 and 3. This will not displace any oxidised floc and, thus, no correction need be made for its addition. Insert the stopper at once, and mix up thoroughly as before.

(8) Allow the solution to stand for atleast 5 minutes to ensure formation of I_2, which is to be titrated against sodium thiosulphate.

(9) Withdraw 101.5 ml* of this solution in the conical titration flask, and immediately add $\frac{N}{40}$ sodium thiosulphate solution drop by drop from the burette, alter recording the initial burette reading, until the yellow colour becomes very light.

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Observations and calculations

Sample size = 100 ml of original sample.

S. No.	*Initial burette reading (ml)*	*Final burette reading (ml)*	*ml of thiosulphate used ; (3) – (2) ; say x*
(1)	(2)	(3)	(4)
			Concordant reading = x ml

*Since 4 ml of water has been lost from 300 ml BOD bottle due to addition of reagents, the original water of **100 ml** will be found to be contained in **101.5 ml** of solution in the BOD bottle.

The D.O. in water is finally computed from Eq. 15.39 as :

$$\text{D.O. in mg/l} = \frac{x \times 200}{100} = 2x. \textbf{ Ans.}$$

Result. The D.O. of the given water sample is ..... mg/l. **Ans.**

EXPERIMENT NO. 12

15.12. Objective : Find Out the Suspended Solids (SS) ; Dissolved Solids (DS) and Total Solids (TS) in a Given Sample of Water

Theory. The total amount of solids (TS) includes the suspended solids (SS) as well as the dissolved solids (DS). The total solids in a water sample can be determined by evaporating the water sample and weighing the dry residue left. The suspended solids can be found by filtering the water sample and weighing the residue left on the filter paper. The difference between total solids and suspended solids will represent nothing but the dissolved solids.

The dissolved solids, which usually predominate in waters, consist *mainly of inorganic salts, small amount of organic matter*, and *dissolved gases. The suspended solids contain much of the organic matter, and any increase thereof tends to increase the degree of pollution in water.*

The amount of total solids (T.S.) in a given water up to 500 mg/l generally makes it suitable for domestic uses. Waters with higher T.S. contents up to about 1000 mg/l, may also be acceptable, although may cause some psychological effects on some human beings, consuming such a water. High concentrations of dissolved solids in waters when used in boilers, may lead to boiler troubles like priming and foaming.

Apparatus Required. Table, porcelain dish, measuring cylinder, heater, weighing machine with weights, beaker, filter paper, funnel, oven, etc. The arrangement used in this test is shown in Fig. 15.6.

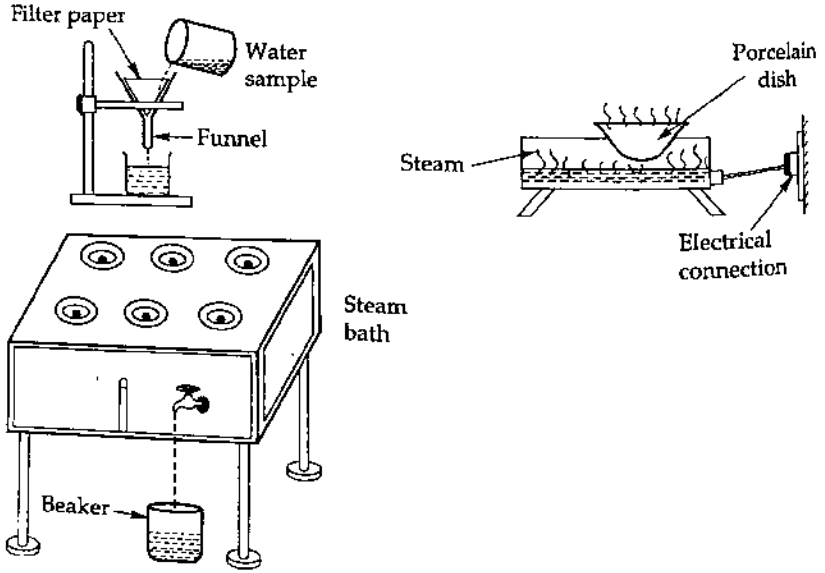


Fig. 15.6. Apparatus reqd. for performing test for TS and SS.

Procedure

(1) Weigh the given porcelain dish (clean and dry) and record its mass as M_1 gm.

(2) Fill 50 ml of water sample in the porcelain dish.

(3) Place the dish on steam bath and evaporate the sample to dryness.

(4) After cooling the dish, record the mass of the dish with dry residue in gm. Let it be M_2 gm.

(5) Take another 50 ml of water sample, and the given standard filter paper.

(6) Record the mass of the filter paper. Let it be M_3 gm.

(7) Filter the water through the filter paper in a funnel.

(8) Place the filter paper with residue in an oven, and evaporate it to dryness.

(9) Record the mass of the filter paper and residue as M_4 gm.

Observations

Volume of water taken in each test	= 50 ml
Mass of empty dish	= M_1 gm
Mass of dish + dry residue	= M_2 gm
Mass of filter paper	= M_3 gm
Mass of filter paper + dry residue	= M_4 gm.

Computations

(*i*) Suspended solids in 50 ml sample = $(M_4 - M_3)$ gm

Concentration of *suspended solids*

$$= \left(\frac{M_4 - M_1}{50} \times 1000\right) \text{ mg/l} = x \text{ mg/l.} \quad \textbf{Ans.}$$

(*ii*) Total solids in 50 ml sample = $(M_2 - M_1)$ gm

Concentration of *total solids*

$$= \left(\frac{M_2 - M_1}{50} \times 1000\right) \text{ mg/l} = y \text{ mg/l.} \quad \textbf{Ans.}$$

(*iii*) Concentration of *dissolved solids* = $(y - x)$ mg/l. **Ans.**

EXPERIMENT NO. 13

15.13. Objective : To Determine the Quantity of Alum Required to Coagulate a Given Sample of Raw Water

Theory. Alum is the name given to the *Aluminium sulphate* with its chemical formula as $Al_2(SO_4)_3 \cdot 18H_2O$. The alum when added to raw water, *reacts with the bicarbonate alkalinities, which* are generally present in raw waters, so as to form a *gelatinous precipitate (floc) of aluminium hydroxide.* This floc attracts pure suspended matter and colloids present in raw water, thereby growing in size. The floc formation is assisted by slow mixing, called **floatation.** The flocs finally settle down to the bottom of the tank, for being removed in the sedimentation tank. The above process/technique is known as **coagulation.**

The chemical equation involved in coagulation is given below :

$$Al_2(SO_4)_3.18H_2O + 3Ca(HCO_3)_2 \xrightarrow{\text{pH 5.5 to 8.0}} \underset{\text{Al. hydroxide ppt.}}{3CaSO_4 + 2Al(OH)_3 \downarrow} + 6C_2O \uparrow \quad ...(15.44)$$

The amount of alum required for coagulation depends on the turbidity and colour of raw water. The use of ***optimum amount of coagulant*** is indicated by the formation of large feathery flakes ; and can be approximately determined in the laboratory by *jar test*, which can finally be adjusted with actual results obtained at the treatment plant. The optimum dose of alum is usually found to vary from 5 mg/l for relatively clear waters to about 85 mg/l for highly turbid waters. *The average normal dose is about 17 mg/l.*

The **jar test** involves the use of a stirring device shown in Fig. 15.7. The stirrer consists of six paddles capable of rotation with variable speed between 0 to 100 rpm. In the test, 1 litre water is placed in each of the jars or beakers, which are dosed with different amounts of coagulant (alum). After rapid mixing to *disperse* the chemicals, the samples are stirred slowly for *floc formation* ; and then allowed to *settle* under quiescent conditions. For this purpose, the jars are initially mixed at a speed of 60 to 80 rpm for 1 minute, and then stirred at a speed of 30 rpm for 15 minutes. After the stirrer is stopped, the nature and settling characteristics of the floc are observed and recorded in quantitative terms, such as poor, fair, good, very good, or excellent. A hazy sample indicates poor coagulation, while proper coagulated water contains floc that is well formed, with clear liquid between particles. The lowest alum dosage that provides good turbidity removal during the jar test is considered for the first trial dosage in plant operation, and the final optimum quantity is adjusted by actual observations at the WTP (Water Treatment Plant).

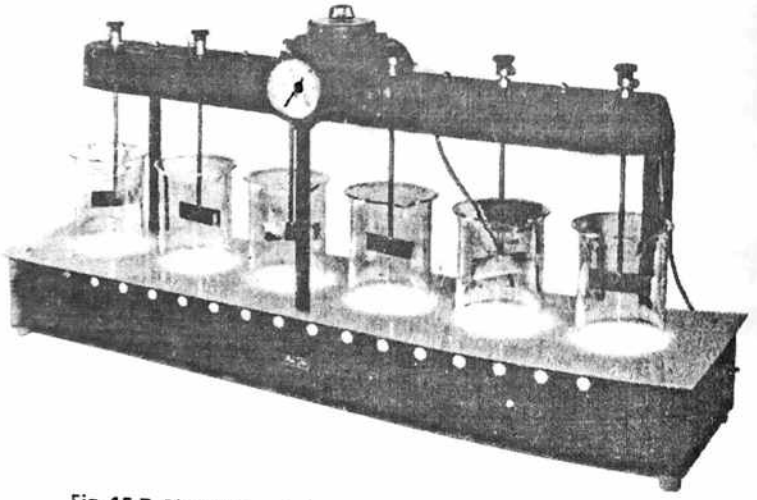


Fig. 15.7. Photoview of the stirring device used in the jar test.

Apparatus Required. The *laboratory flocculator* complete with stirring (paddles mechanically operated) device ; six glass jars or beakers.

Reagents Required. Alum solution (made by dissolving 14.28 gm of alum in one litre distilled water).

Procedure

(1) Fill 1 litre water sample in each of the six jars.

(2) Attach the sample jars to the stirring device by lifting the paddles in the right upward direction.

(3) Add coagulant solution in progressive volumes of 0.25 ml, 0.5 ml, 0.75 ml, 1.0 ml, 1.25 ml and 1.5 ml into the series of the six sample jars.

(4) Flocculate the samples rapidly for about one minute with mechanically operated paddles, @ 60-80 rpm ; followed by gentle stirring @ about 30 rpm for 15 minutes.

(5) Remove the jars from the stirring device after stirring is complete.

(6) Let the samples (jars) stand for 30 minutes for settling of flocs.

(7) Select the minimum dosage giving the best floc and settling characteristics.

Observations

Quantity of water filled in each jar = 1000 ml

Jar No.	*Amount of coagulant added in ml*	*Observation in regard to quality of floc formed*
1	0.25	
2	0.50	
3	0.75	
4	1.0	
5	1.25	...Excellent...
6	1.50	
Selected optimum dosage = 1.25 ml (say x ml)		

Calculation

Selected optimum dosage = x ml per litre of water sample.

Strength of alum solution prepared

= 14.28 gm/l = 14.28 mg/ml

∴ Optimum dosage in mg/l

$$= 14.28 \frac{\text{mg}}{\text{ml}} \times x \text{ ml per litre of water}$$

$$= \mathbf{14.28}\, x. \quad \textbf{Ans.}$$

16

Chapterwise Multichoice Objective Questions

CHAPTER 1-2

Q. 1. Per capita water demand is defined as the litres of water consumed daily by each person. Naturally it has to be some average value, over a period of time. Over how much period, the averaging is done here :

(*a*) 24 hours (*b*) one year
(*c*) 10 years (*d*) 35 years.

Q. 2. Which one of the following practices, causes reduction in the per capita water consumption ?

(*a*) good quality water (*b*) hotter climate
(*c*) modern living (*d*) metering system.

Q. 3. The multiplying factor, as applied to obtain the maximum daily water demand, in relation to the average *i.e.* per capita daily demand, is :

(*a*) 1.5 (*b*) 1.8
(*c*) 2.0 (*d*) 2.7.

Q. 4. The multiplying factor, as applied to obtain the peak hourly demand, in relation to the maximum daily demand (per hour of course) is :

(*a*) 1.5 (*b*) 1.8
(*c*) 2.0 (*d*) 2.7.

Q. 5. The multiplying factor, as applied to obtain peak hourly demand, in relation to the average daily demand (per hour of course), is :

(*a*) 1.5 (*b*) 1.8
(*c*) 2.0 (*d*) 2.7.

Q. 6. If the average daily water consumption of a city is 24000 cum, the peak hourly demand (of the maximum day of course) will be :

(*a*) 1000 cu m/hr (*b*) 1500 cu m/hr
(*c*) 1800 cu m/hr (*d*) 2700 cu m/hr.

Q. 7. The total water requirement of a city is generally assessed on the basis of :

(*a*) maximum hourly demand
(*b*) maximum daily demand + fire demand

(*c*) average daily demand + fire demand
(*d*) greater of (*a*) and (*b*).

Q. 8. The water treatment units may be designed, including 100% reserves, for water demand equal to :

(*a*) average daily (*b*) twice of (*a*)
(*b*) maximum daily (*d*) twice of (*c*).

Q. 9. Coincident draft in relation to water demand, is based on :

(*a*) peak hourly demand
(*b*) maximum daily demand
(*c*) maximum daily + fire demand
(*d*) greater of (*a*) and (*c*).

Q. 10. The distribution system in water supplies, is designed on the basis of :

(*a*) average daily demand (*b*) peak hourly demand
(*c*) coincident draft (*d*) greater of (*b*) and (*c*).

Q. 11. As compared to the geometrical increase method of forecasting population, the arithmetical increase method gives :

(*a*) lesser value
(*b*) higher value
(*c*) equal value
(*d*) may vary, as it may depend on the population figures.

Q. 12. The population of a town in three consecutive decades are : 1 lakh, 1.4 lakh, 1.68 lakh, respectively. The population of this town in the fourth consecutive decade, according to geometric method, would be :

(*a*) 1.9 lakh (*b*) 2.184 lakh
(*c*) 2.2 lakh (*d*) 2.5 lakh.

Q. 13. The suitable method for forecasting population for a young and a rapidly developing city is :

(*a*) arithmetic mean method
(*b*) geometric mean method
(*c*) comparative graphical method
(*d*) none of these.

Q. 14. The suitable method for forecasting population for an old developed large city, is :

(*a*) arithmetic mean method
(*b*) geometric mean method
(*c*) comparative graphical method
(*d*) none of these.

Q. 15. The growth of population can be conveniently represented by a curve, which is amenable to mathematical solution. The type of this curve is :

(*a*) semi-log curve (*b*) straight line curve
(*c*) logistic curve (*d*) exponential curve.

Q. 16. The average domestic water consumption per capita per day for an Indian city, as per IS 1172-1963, may be taken as :
(a) 135 l/c/d (b) 210 l/c/d
(c) 240 l/c/d (d) 270 l/c/d.

Q. 17. The total water consumption per capita per day, including domestic, commercial, and industrial demands, for an average Indian city, as per IS code, may be taken as :
(a) 135 l/c/d (b) 210 l/c/d
(c) 240 l/c/d (d) 270 l/c/d.

Q. 18. If the population of a city is 2 lakh, and average water consumption is 200 lpcd, then the capacity of the pipe mains, should be :
(a) 108 Mld (b) 72 Mld
(c) 60 Mld (d) 40 Mld.

Q. 19. If the population of a central congested high valued city is 2,00,000, and the fire demand is computed to be 45,000 litres per minute, the formula used for the calculation must have been :
(a) Freeman's formula
(b) National Board of Underwriters formula
(c) Kuichling's formula
(d) Buston's formula.

Q. 20. If the average water consumption of a city is 300 lpcd, and its population is 4,00,000, the maximum hourly draft of the maximum day will be :
(a) 120 Mld (b) 216 Mld
(c) 324 Mld (d) none of these.

Q. 21. The pipe mains carrying water from the source to the reservoir are designed for the :
(a) maximum daily draft
(b) average daily draft
(c) maximum hourly draft of the maximum day
(d) maximum weekly draft.

Q. 22. Water losses in water supply system, are assumed as :
(a) 5% (b) 7.5%
(c) 15% (d) 25%.

CHAPTER 3

Q. 23. Which source of water, among the following, is not a surface source ?
(a) river (b) well
(c) lake (d) ocean.

Q. 24. Which source of water, among the following, is not a subsurface source ?

(a) spring (b) well
(c) storage reservoir (d) infiltration gallery.

Q. 25. The earth's water circulatory system is known as :
(a) water cycle (b) precipitation cycle
(c) hydrological cycle (d) none of them.

Q. 26. Most of the weather phenomena take place in the :
(a) mesophere (b) stratosphere
(c) ionosphere (d) troposphere.

Q. 27. The average annual rainfall of a country is a figure, which is averaged over a period of :
(a) 10 years (b) 15 years
(c) 35 years (d) 100 years.

Q. 28. A large whirling mass of air, at the centre of which the barometric pressure is low, is known as a :
(a) cyclone (b) tornado
(c) anti cyclone (d) none of these.

Q. 29. The major quantity of rain comes to India, as :
(a) convective precipitation (b) cyclonic precipitation
(c) orographic precipitation (d) none of these.

Q. 30. Ground water accretion starts during a rain, after satisfying the demand for :
(a) interception (b) depression storage
(c) surface detention (d) all of these.

Q. 31. I.M.D. has changed over from Symon's rain gauge to fibre glass rain gauges of two sizes. The collector areas of these gauges are :
(a) 1000 cm^2 and 500 cm^2 (b) 400 cm^2 and 200 cm^2
(c) 200 cm^2 and 100 cm^2 (d) none of these.

Q. 32. The standard recording rain gauge adopted in India, is of the type :
(a) weighing bucket (b) tipping bucket
(c) natural siphon (d) telemetry.

Q. 33. The type of rain gauge, which produces a continuous graphical record as mass curve of rainfall, is known as :
(a) Symon's gauge (b) storage gauge
(c) natural siphon gauge (d) none of these.

Q. 34. Natural siphon type gauge is also known as :
(a) floating type (b) weighing type
(c) tipping bucket type (d) none of these.

Q. 35. Out of the following, the only non-recording type of rain gauge, is :
(a) weighing type (b) tipping bucket type
(c) Symon's type (d) none of the above.

Q. 36. The type of rain gauge, which is installed at hilly and inaccessible areas, is :

(*a*) tipping bucket type (*b*) float type
(*c*) weighing type (*d*) none of these.

Q. 37. Index of wetness, directly represents in a given year at a given place, the :

(*a*) extent of rainfall
(*b*) extent of deficiency of rainfall
(*c*) comparative rainfall with respect to annual average value
(*d*) none of these.

Q. 38. The dead storage zone in a reservoir is provided for storage of :

(*a*) water for firm power (*b*) sand and silt
(*c*) water for water supplies (*d*) none of these.

Q. 39. Trap efficiency of a storage reservoir indicates the :

(*a*) sediment volume trapped in the reservoir
(*b*) sediment volume let out from the reservoir
(*c*) sediment volume trapped inrelation to the sediment volume entering the reservoir
(*d*) none of these.

Q. 40. The percentage of the total sediment flow, getting deposited in a reservoir, is called :

(*a*) capacity-inflow ratio (*b*) sediment coefficient
(*c*) trap efficiency (*d*) displacement efficiency.

Q. 41. With the reduction in the reservoir capacity, the quantum of sediment trapped by it :

(*a*) increases
(*b*) decreases
(*c*) does not get affected.

Q. 42. The evaporation losses from the surface of a reservoir can be reduced by sprinkling :

(*a*) DDT (*b*) acetyl alcohol
(*c*) potassium permanganate (*d*) none of these.

Q. 43. Pick up the incorrect statement :

(*a*) surface run off equals run off minus base flow
(*b*) the water that seeps into the ground reservoir is called infiltration
(*c*) the yield of a drainage basin in depth, when multiplied by the basin area, gives us the yearly available water volume
(*d*) the coarser sand and silt gets deposited near the face of the dam, and the finer one away from it
(*e*) none of the above.

Q. 44. Pick up the incorrect statement :

(*a*) proper location and the operation of the sluiceways of a reservoir can help in increasing the density currents, to reduce reservoir silting

(*b*) about 1/4 to 1/5 of the total reservoir capacity is earmarked for dead storage, in a storage reservoir
(*c*) small reservoirs on large rivers, silt very slowly
(*d*) surface waters are generally soft and less corrosive than the ground waters
(*e*) none of the above.

Q. 45. The maximum quantity of water that can be supplied from a reservoir with full guarantee during critical periods, is called :

(*a*) reservoir yield (*b*) design yield
(*c*) secondary yield (*d*) firm yield.

CHAPTER 4

Q. 46. Pick up the incorrect statement :

(*a*) porosity of a rock, having well sorted uniform grains, is on a lower side
(*b*) quartzite stone has much lower porosity as compared to limestone or sandstone
(*c*) all porous rocks possess enough capacity to store water, and hence, are considered as potential sources of ground water, irrespective of their permeability
(*d*) zone of saturation, which is tapped for the ground water, lies below the zone of aeration.

Q. 47. The zone of aeration in a ground profile does not consist of :

(*a*) capillary fringe (*b*) soil water zone
(*c*) intermediate zone (*d*) saturation zone.

Q. 48. The velocity of flow through a ground soil sample can be measured by :

(*a*) Manning's formula (*b*) Darcy's law
(*c*) Dupuit's formula (*d*) none of these.

Q. 49. The appropriate percentage that represents the world's water resources in the form of ground water, is :

(*a*) 2% (*b*) 12%
(*c*) 22% (*d*) 32%.

Q. 50. A geological formation, which not only stores water, but can yield it in sufficient quantity, is known as an :

(*a*) aquiclude (*b*) aquitard
(*c*) aquifer (*d*) aquifuse.

Q. 51. Shales are :

(*a*) porous and permeable (*b*) porous but not permeable
(*c*) only permeable (*d*) none of these.

Q. 52. The water yielding capacity of a ground formation is represented by its :

(*a*) field capacity (*b*) specific yield
(*c*) specific capacity (*d*) none of these.

Q. 53. A flowing well is essentially :
(a) an artesian well (b) a non-artesian well
(c) a gravity well (d) none of these.

Q. 54. Rate of flow from a well per unit of drawdown is known as its:
(a) specific yield (b) specific capacity
(c) field capacity (d) none of these.

Q. 55. An aquifer, sandwiched between the top and the bottom aquicludes, will supply water to a well, called :
(a) non-artesian well (b) artesian well
(c) flowing well (d) none of these.

Q. 56. A perched aquifer is essentially found within :
(a) an unconfined aquifer (b) a confined aquifer
(c) an aquiclude (d) none of these.

Q. 57. A rainy well, largely used for tapping ground water for public supplies, receives ground water, from :
(a) its bottom (b) its vertical permeable sides
(c) radial horizontal collectors (d) none of these.

Q. 58. For an aquifer of width W and thickness B, and having its co-efficient of permeability as K, the transmissibility (T) is given by T =
(a) KB (b) KW
(c) $\frac{K}{B}$ (d) $\frac{KB}{W}$.

Q. 59. The most widely used type of a tubewell in India is :
(a) a cavity well (b) a strainer well
(c) a dug well (d) a rainy well.

Q. 60. Abyssinian tubewell is a special variety of :
(a) a dug well (b) a rainy well
(c) a strainer well (d) a cavity type well.

Q. 61. Strainer type tubewells are unsuitable for :
(a) hard strata (b) fine sandy strata
(c) coarse gravel (d) clayey strata.

Q. 62. The area of flow in a strainer tubewell can be increased by :
(a) decreasing the size of the screen openings
(b) increasing the length of the strainer pipe
(c) increasing the size of the cavity
(d) none of these.

Q. 63. The depression of watertable in a well due to pumping, is maximum at :
(a) a distance R from the well (b) a distance $R/2$ from the well
(c) close to the well (d) none of these.
(where R is the radius of influence of the well)

Q. 64. Ground water is usually free from :
(a) suspended impurities (b) dissolved impurities
(c) both (a) and (b) (d) none of the above.

Q. 65. When compared to shallow dug wells, deep dug wells have :
(a) more depth
(b) more discharge
(c) more depth and more discharge
(d) none of these.

Q. 66. Select the correct relationship between porosity (N), specific yield (Y), and specific retention (R) :
(a) $R = N + Y$ (b) $N = Y + R$
(c) $R > N + Y$ (d) $Y = N + R$.

Q. 67. The normal maximum discharge of a tubewell is of the order of :
(a) 5 litres/sec (b) 50 litres/sec
(c) 500 litres/sec (d) 5000 litres/sec.

Q. 68. The quantity of water available from an infiltration gallery, depends upon the :
(a) size of the gallery (b) efficiency of the drain pipes
(c) yield of the aquifer source (d) all of these.

Q. 69.* If the ground water is saline and the subsoil depth in the river bed is shallow, then the best (medium size) source to develop is :
(a) Tubewell (b) Infiltration gallery
(c) Spring (d) River.

Q. 70. The portion of the soil formation, through which lateral movement of water takes place, is called :
(a) water-table (b) zone of saturation
(c) zone of gravity water (d) none of these.

Q. 71. Pick up the incorrect statement :
(a) the quality of tubewell-water is generally better than that of a surface source
(b) the discharge of a tubewell is generally more than that of an open well
(c) the water bearing stratum is called an aquifer
(d) a tubewell should preferably draw water from the top most pervious stratum of the soil.

Q. 72. The types of pumps used in tubewells are :
(a) submergible pumps (b) centrifugal pumps
(c) turbine type pumps (d) all of these.

Q. 73. Generally, heavy treatment is required to be given to the water, obtained from a :
(a) river (b) ranney well
(c) tubewell (d) lake.

*Hints for solving starred questions are given at the end of the chapter.

Q. 74.* In an artesian aquifer, the drawdowns in two observation wells, located at distances of 100 m and 300 m, were found to be the same after one hour and x hours, respectively. The value of x in such a case is :

(*a*) 1/9 hours (*b*) 3 hours
(*c*) 9 hours (*d*) 27 hours.

Q. 75. If the specific capacity of a well is 1.166 litres/sec, then the discharge from this well under a depression head of 3 m will be :

(*a*) 1.166 l/s (*b*) 3.5 l/s
(*c*) 10.5 l/s (*d*) none of these.

Q. 76. Pick up the incorrect statement :

(*a*) a shallow dug well receives supplies from its surroundings
(*b*) a deep dug well receives supplies from the bottom, through a bore hole in the mota layer
(*c*) shallow dug wells are always of smaller depths than those of deep dug wells
(*d*) none of them.

Q. 77. 'Shrouding' is essentially provided in :

(*a*) strainer type tubewells (*b*) cavity type tubewells
(*c*) slotted pipe tubewells (*d*) all of the above.

Q. 78. Percussion drilling is most suitable for drilling tube wells in :

(*a*) unconsolidated sands (*b*) consolidated rocks
(*c*) black cotton soils (*d*) quick sands.

Q. 79. Rotary drilling is unsuitable for drilling tubewells in :

(*a*) unconsolidated sands (*b*) consolidated rocks
(*c*) quick sands (*d*) all of them.

Q. 80. Large wells, of say 1.2 m diameter or so, in unconsolidated ground strata, can be best drilled by :

(*a*) percussion drilling rig (*b*) direct rotary drilling rig
(*c*) reverse rotary drilling rig (*d*) none of them.

Q. 81. The maximum non-verticality of a drilled tubewell bore of 100 m depth, may be :

(*a*) 5 cm (*b*) 10 cm
(*c*) 50 cm (*d*) 100 cm.

Q. 82. The minimum untapped land area required per tube-well in north India, without causing any material effect on the watertable, is of the order of :

(*a*) 0.5 sq km (*b*) 1.5 sq km
(*c*) 10 sq km (*d*) 20 sq km.

Q. 83. The appropriate life of a tubewell in India is of the order of :

(*a*) 1 year (*b*) 2 years
(*c*) 5 years (*d*) 15 years.

Q. 84. Corrosion of tubewell pipes may cause :

(*a*) reduced discharge from the tubewell
(*b*) excessive discharge of sand with water
(*c*) discharge of highly corrosive acidic water
(*d*) none of these.

Q. 85. Incrustation of the tubewell pipes may lead to :

(*a*) reduced discharge from the tubewell
(*b*) excessive discharge of sand with water
(*c*) discharge of highly alkaline water
(*d*) none of these.

Q. 86. The efficiency of the centrifugal pump, used for lifting water from a tubewell, may be assumed to be :

(*a*) 30% (*b*) 65%
(*c*) 80% (*d*) 90%.

CHAPTER 5

Q. 87. The devices, which are installed for drawing water from different water sources, are called :

(*a*) filters (*b*) intakes
(*c*) outlets (*d*) inlets.

Q. 88. Pick up the incorrect statement, in relation to water supplies :

(*a*) the intake point should, as far as possible, be located in deep waters
(*b*) the intake point should be located upstream of the point of sewage disposal
(*c*) the intake point on the meandering rivers, should preferably be located on the convex banks
(*d*) none of these.

Q. 89. Simple submerged intakes may be suitable to :

(*a*) large water supply projects on rivers
(*b*) small water supply projects on streams
(*c*) small water supply projects on lakes
(*d*) both (*b*) and (*c*).

Q. 90. Dry intake towers, when compared to wet intake towers, need :

(*a*) heavier and costly construction
(*b*) lighter and cheaper construction
(*c*) practically the same section.

Q. 91. Pick up the incorrect statement :

(*a*) dry intake towers are preferred to wet intake towers
(*b*) wet intake towers help in withdrawing water from different levels of a storage reservoir
(*c*) a medium sized river intake well can preferably be adopted, when the water is withdrawn from the upstream of a barrage.
(*d*) none of these.

Q. 92. The intake towers, which remain surrounded by the water from all sides and stand in the river, are called :

(*a*) wet intake towers (*b*) valve towers
(*c*) dry intake towers (*d*) both (*a*) and (*c*).

CHAPTER 6

Q. 93. Pressure pipes are exclusively used for carrying water supplies, because :

(*a*) such pipes are closed, and hence not exposed to pollution anywhere, enroute
(*b*) since the water runs under pressure in these pipes, pollution from outside surroundings cannot find entry, even if some joints are loose
(*c*) such pipes can go up and down the hills and valleys, thus requiring lesser length
(*d*) for all of the above reasons.

Q. 94. Pick up the incorrect statement :

(*a*) The inner surface of a water supply pressure pipe offers resistance to the flow, leading to dissipation of pressure head
(*b*) In the design of water supply conduits, the frictional loss offered by the pipe length is balanced against the available water head
(*c*) the flow velocity in water supply conduits is generally kept between 0.9 to 1.5 m/s, as an optimum velocity, depending on the cost of the pipe and the pumping.
(*d*) none of the above.

Q. 95. The formula, which is most appropriate to the design of pressure pipes is :

(*a*) Darcy Weisbach formula (*b*) Mannings formula
(*c*) Chezy's formula (*d*) Dupuit's formula.

Q. 96. The formula, which has absolutely no relevance with the design of water supply conduits, is :

(*a*) Hazen-William's formula
(*b*) Thiem's formula
(*c*) Darcy-Weisbach formula
(*d*) Modified Darcy Weisbach formula.

Q. 97. In the design of water supply conduits, the smaller value of the design velocity, if adopted, will :

(*a*) increase the cost of the pipe (*b*) reduce the cost of pumping
(*c*) both (*a*) and (*b*) above (*d*) none of the above.

Q. 98. In the design of water supply conduits, the larger value of the design velocity, if adopted, will :

(*a*) reduce the cost of the pipe (*b*) increase the cost of pumping
(*c*) both (*a*) and (*b*) above (*d*) none of the above.

Q. 99. The appropriate diameter of a water main for supplying 9 Mld of water, with a velocity of 1.5 m/sec, is :

(*a*) 0.3 m (*b*) 0.6 m
(*c*) 0.9 m (*d*) 1.0 m.

Q. 100. The internal pressure, to which a water supply pipe is subjected to, is :

(*a*) full hydrostatic pressure when water in the pipe is at rest
(*b*) pressure head and velocity head in the pipe, when flow at full velocity is taking place
(*c*) (*a*) + water hammer pressure
(*d*) (*b*) + water hammer pressure.

Q. 101. The hoop tension, needing steel reinforcement along the circumference of a water supply conduit, is caused by :

(*a*) internal pressure due to water in the pipe
(*b*) water hammer pressure
(*c*) pressure due to traffic load
(*d*) both (*a*) and (*b*).

Q. 102. Pick up the incorrect statement :

(*a*) water hammer pressure is caused by opening and closing of gates, valves, pumps, etc. situated along a water main
(*b*) water hammer pressure is caused only momentarily
(*c*) the greater the haste with which a gate valve is opened or closed on a water main, the greater will be the water hammer pressure developed
(*d*) none of these.

Q. 103. Water hammer pressures can be reduced by using :

(*a*) fast closing valves (*b*) slow closing valves
(*c*) critically closing time, valves (*d*) none of these.

Q. 104.* A C.I., 1 m dia water pipe, is subjected to a water hammer pressure of 8 kg/cm^2. The maximum static head on this pipe at its lowest point enroute is 120 m. The suggested thickness of this pipe is :

(*a*) 10 mm (*b*) 12 mm
(*c*) 15 mm (*d*) 25 mm.

Q. 105. The water supply mains, normally need meticulous design, to withstand :

(*a*) compressive stresses (*b*) tensile stresses
(*c*) flexural stresses (*d*) all of these.

Q. 106. The maximum pressure, which a pipe can withstand without any leakage, during hydrostatic pressure test, is called the :

(*a*) working pressure (*b*) test pressure
(*c*) design pressure (*d*) hydrostatic pressure.

Q. 107. The commonly used material for water mains, which is strong, non-corrodable, very very durable (100 years or so), but heavy and brittle, is :

(*a*) steel (*b*) R.C.C.
(*c*) copper (*d*) cast iron.

Q. 108. The cast iron water mains are :

(*a*) vary durable
(*b*) can resist very high pressures
(*c*) liable to corrosion
(*d*) unaffected in their discharging capacities, over time.

Q. 109. As compared to the cast iron pipes, steel water pipes are :

(*a*) heavier (*b*) stronger
(*c*) durable (*d*) all of these.

Q. 110. Steel pipes are :

(*a*) easily affected by acidic and alkaline atmospheric effects
(*b*) unable to withstand high negative pressures
(*c*) liable to quick rusting
(*d*) all of the above.

Q. 111. Match the columns, most appropriately :

Water pipe material	*Average age of pipe*
(*a*) cast iron	(*i*) 75 years
(*b*) steel	(*ii*) 100 years
(*c*) R.C.C.	(*iii*) 40 years.

Q. 112. Cast iron pipes, which are cast in horizontal position by ordinary sand moulding process, are called :

(*a*) Pit cast pipes (*b*) Delavaud pipes
(*c*) Sand spun pipes (*d*) Mcwane pipes.

Q. 113. Asbestos pipes are made of :

(*a*) asbestos (*b*) silica and asbestos
(*c*) asbestos cement (*d*) none of these.

Q. 114. Cast iron pipes, made by centrifugal process using metallic moulds, are called :

(*a*) Pit cast pipes (*b*) Delavaud pipes
(*c*) Sand spun pipes (*d*) Mcwane pipes.

Q. 115. The valve, which allows the flow only in one direction, is a :

(*a*) reflux valve (*b*) sluice valve
(*c*) gate valve (*d*) none of these.

Q. 116. The valve, which is provided in the water distributing pipes at street corners and pipe junctions to control the flow in the distribution system, is :

(*a*) an air valve (*b*) a sluice valve
(*c*) a scour valve (*d*) a reflux valve.

Q. 117. A check valve is also known as a :

(*a*) relief valve (*b*) reflux valve
(*c*) blow off valve (*d*) none of these.

Q. 118. A sluice valve is also known as :

(*a*) air-inlet valve (*b*) scour valve
(*c*) gate valve (*d*) none of these.

Q. 119. Pressure-relief valves installed along water mains may be provided for relieving :

(*a*) air pressure (*b*) water hammer pressure
(*c*) ice pressure (*d*) all of these.

Q. 120. Anchrorages in water pipelines are provided :

(*a*) at all bends
(*b*) at all steep slopes
(*c*) at other points of unbalanced pressure
(*d*) all of the above.

Q. 121. Scour valves are provided :

(*a*) at the street corners to control the flow
(*b*) at the dead ends to drain out the wastewater
(*c*) at every summit of the rising main
(*d*) at the foot of the rising main along the slope, to prevent back running of water.

Q. 122. A blow off valve is provided in a water distribution system, at :

(*a*) low points (*b*) high points
(*c*) junction points (*d*) all of the above.

Q. 123. Cast iron pipes having plain ends, are joined by a joint, called :

(*a*) flanged joint (*b*) spigot and socket joint
(*c*) dresser coupling (*d*) none of these.

Q. 124. Pick up the incorrect statement :

(*a*) sluice valves are provided to allow flow of water only in one direction, thereby preventing back flow
(*b*) air valves are provided at 'summits' along a pipe line, to admit or release air
(*c*) scour valves are provided at low points to empty a pipe line
(*d*) gate valves are provided to regulate flow of water through the pipe lines.

Q. 125. Summits are the points of :

(*a*) high pressures (*b*) low pressures
(*c*) equal pressures (*d*) none of these.

CHAPTER 7

Q. 126. The pumps required for feeding the treated water into the distribution system, when compared to the pumps required for feeding the raw water into the treatment plant, are generally of :

(*a*) higher capacity (*b*) lower capacity
(*c*) equal capacity (*d*) none of the above.

Q. 127. High lift pumps are generally required to feed water into the :

(*a*) treatment plant (*b*) distribution system
(*c*) both of them (*d*) neither of them.

Q. 128. Low lift pumps are generally required to feed water into the :
(a) the treatment plant (b) the distribution system
(c) both of them (d) neither of them.

Q. 129. Centrifugal pumps are rotodynamic machines of the :
(a) axial flow type (b) radial flow type
(c) mixed flow type (d) (b) and (c) both
(e) (a) and (b) both (f) (a) and (c) both
(g) none of these.

Q. 130. Axial flow pumps are of :
(a) rotodynamic type (b) displacement type
(c) centrifugal type (d) none of them.

Q. 131. Examples of the displacement pumps are :
(a) reciprocating pumps (b) centrifugal pumps
(c) rotary pumps (d) (a) and (c) both
(e) (a) and (b) both (f) (b) and (c) both
(g) none of these (h) all (a), (b) and (c) above.

Q. 132. The most commonly adopted pumps in water supplies are :
(a) centrifugal pumps (b) reciprocating pumps
(c) hydraulic rams (d) none of these.

Q. 133. The efficiency of closed impeller type of centrifugal pumps, when compared to that of open impeller types, is :
(a) much high
(b) much less
(c) almost equal.

Q. 134. The chances of clogging of the centrifugal pumps are more for the type of :
(a) open impellers (b) closed impellers
(c) equal for both types (d) no chances at all.

Q. 135. Closed impeller type of centrifugal pumps are largely used for pumping water supplies, because :
(a) their efficiency is high (b) they do not clog easily
(c) both (a) and (b) above (d) none of these.

Q. 136. Open impeller type of centrifugal pumps are largely used to pump sewage, because :
(a) their efficiency is high (b) they do not clog easily
(c) both (a) and (b) above (d) none of them.

Q. 137. Diffuser type of centrifugal pumps are those centrifugal pumps, which have :
(a) volute type of casing (b) turbine type of casing
(c) any of the above two (d) none of these.

Q. 138. Turbine casing type of centrifugal pumps, when compared to volute casing type of centrifugal pumps, possess :
(a) higher efficiency (b) lower efficiency
(c) equal efficiency.

Q. 139. A bore hole pump is essentially a centrifugal pump of :
(a) single stage (b) double stage
(c) multi-stage (d) none of these.

Q. 140. The driving motor is placed below the pump bowl in a :
(a) turbine pump (b) submergible pump
(c) monoblock pump (d) none of these.

Q. 141. Priming of a pump, involving the filling of the pump casing with water for removing entrapped air, is essentially required for :
(a) centrifugal pumps installed below the water level
(b) centrifugal pumps installed above the water level
(c) both (a) and (b)
(d) none of the above.

Q. 142. While installing a monoblock centrifugal pump, the inlet pipe drawing water from the sump well or the ground watertable, is provided with a foot valve, in order to :
(a) prevent water from leaving and emptying the pump casing
(b) prevent backflow of lifted water into the pump
(c) prevent sudden and full loading of the pump motor
(d) none of the above.

Q. 143. If the sluice valve provided at the outlet end of a monoblock centrifugal pump installation is kept fully open at the start of the motor, then the chances are :
(a) that the pump motor will work perfectly
(b) that the pump motor may burn
(c) that the motor gets loaded slowly, reducing the discharge
(d) none of the above.

Q. 144. The rated discharge of a centrifugal pump represents the discharge at which the pump efficiency is :
(a) minimum (b) maximum
(c) workable (d) none of these.

Q. 145. The discharge from a given centrifugal pump will vary, depending on the :
(a) lifted head (b) speed of its motor
(c) both (a) and (b) (d) none of the above.

Q. 146.* A single speed centrifugal pump, feeding a small water supply distribution system of a block of houses, works at :
(a) maximum efficiency (b) minimum efficiency
(c) reduced efficiency (d) none of the above.

Q. 147. Axial flow pumps are most-particularly suitable for pumping :
(a) water supplies from tubewells
(b) water supplies from treatment plants
(c) raw sewage into the treatment plants
(d) storm water from low lying areas into the drains.

Q. 148. Radial flow centrifugal pumps perform well at specific speeds of the order of :
(a) 100-500 rpm (b) 1000-4000 rpm
(c) 4000-7000 rpm (d) above 7000 rpm.

Q. 149. Mixed flow centrifugal pumps perform well at specific speeds of the order of :
(a) 100-500 rpm (b) 1000-4000 rpm
(c) 4000-7000 rpm (d) above 7000 rpm.

Q. 150. Axial flow rotodynamic pumps perform well at specific speeds of the order of :
(a) 100-500 rpm (b) 1000-4000 rpm
(c) 4000-7000 rpm (d) above 7000 rpm.

Q. 151. Hand pumps make use of :
(a) centrifugal pumping (b) reciprocating pumping
(c) rotary pumping (d) none of the above.

Q. 152. The pump, suitable for lifting water from very deep tubewells of the order of 100 to 150 m deep, is :
(a) submergible pump (b) jet pump
(c) air-lift pump (d) hydraulic ram.

Q. 153. The pump, which can preferably be used for lifting water from a number of tubewells, simultaneously, is :
(a) bore hole turbine pump (b) rotary pump
(c) monoblock centrifugal pump (d) air-lift pump.

Q. 154. Jet pumps may preferably be used for :
(a) pumping waters from State tubewells
(b) pumping water supplies into the distribution mains
(c) pumping water from small sized domestic bores, where watertable is low
(d) none of the above.

Q. 155. The only pump among the following, which is not a kinetic energy pump, is :
(a) axial flow pump (b) rotary pump
(c) radial flow pump (d) mixed flow pump.

Q. 156. Reciprocating pumps are used for :
(a) high discharge and high head
(b) low discharge and high head
(c) high discharge and low head
(d) low discharge and low head.

Q. 157. Adoption of the economical diameter for water mains, ensures :
(a) the least cost of pipe
(b) the least cost of pumping
(c) the least cost for pipe and pumping together
(d) none of the above.

Q. 158. The economical diameter of a pipe, through which a discharge of 0.3 cumecs is to be passed, is :
(a) 1.2 m (b) 0.8 m
(c) 0.6 m (d) 0.4 m.

Q. 159. Water is to be lifted from a ground reservoir to an elevated reservoir, so as to fill up the elevated reservoir of 576 m^3 capacity in one hour. The economic diameter of the rising main may then, be chosen as :
(a) 30 cm (b) 45 cm
(c) 60 cm (d) 1 m.

Q. 160.* Water is to be lifted from a ground reservoir of 4 m depth to an elevated reservoir. The difference in their water levels is 40 m. In order to fill this elevated reservoir of 90 cu. m capacity in one hour, the appropriate H.P. of the centrifugal pump chosen, will be :
(a) 10 H.P. (b) 25 H.P.
(c) 50 H.P. (d) 100 H.P.

CHAPTER 8

Q. 161. Water is said to be contaminated, if it contains :
(a) pathogens
(b) undesirable suspended matter, making it unfit for drinking and domestic use
(c) dissolved salts
(d) none of the above.

Q. 162. Pick up the incorrect one :
(a) contaminated water is always polluted
(b) polluted water is always contaminated
(c) absolute pure water is not good for health
(d) none of the above.

Q. 163. 'Safe water' is the one, which does not contain :
(a) pathogenic bacteria (b) turbidity
(c) any taste (d) any colour.

Q. 164. 'Wholesome water' is the one, which does not contain :
(a) pathogenic bacteria
(b) suspended matter in quantities harmful to man
(c) dissolved matter in quantities harmful to man
(d) all of the above
(e) none of the above.

Q. 165. Polluted water is the one, which :
(a) contains pathogenic bacteria
(b) contains undesirable matter, making it unfit for drinking and domestic use
(c) is contaminated
(d) is safe for drinking, but poses problems in special industrial uses.

Q. 166. Suspended solids should not be present in water, because :
(*a*) they cause turbidity
(*b*) they provide absorption sites for biological and chemical agents
(*c*) they are aesthetically displeasing
(*d*) all of the above.

Q. 167. The measure of the amount, to which light is adsorbed or scattered by the suspended material in water, is called :
(*a*) opacity (*b*) turbidity
(*c*) celerity (*d*) diffraction.

Q. 168. When the mode of measurement of turbidity is the 'adsorption of light', the measuring apparatus, is called a :
(*a*) Spectrometer (*b*) Tintometer
(*c*) Turbidimeter (*d*) Nephelometer.

Q. 169. Modern turbidimeters, working on the principle of 'scattering of light', are, known as :
(*a*) Spectrometers (*b*) Optimeters
(*c*) Tintometers (*d*) Nephelometers.

Q. 170. The standard turbidity produced by one mg of silicon dioxide (silica) in one litre of distilled water, is called :
(*a*) one Jackson turbidity unit (Jtu)
(*b*) one Formazin turbidity unit (Ftu)
(*c*) one Nephelometry turbidity unit (Ntu)
(*d*) none of the above.

Q. 171. The modern standard low turbidity, produced by one mg of formazin in one litre of distilled water, is called :
(*a*) FTU (*b*) JTU
(*c*) NTU (*d*) none of the above.

Q. 172. The turbidity, which can be seen easily on naked eye, is of the order of :
(*a*) 1 Jtu (*b*) 2 Jtu
(*c*) 3 Jtu (*d*) 5 Jtu.

Q. 173. The permissible turbidity of 5 Jtu, sometimes, be misguiding as even a trace of turbidity, may be a sign of serious pollution, as may happen in :
(*a*) chalk well waters (*b*) silica-well waters
(*c*) sand-well waters (*d*) none of the above.

Q. 174. Bayli's turbidimeter, when compared to Jackson turbidimeter, can preferably be used, to measure :
(*a*) lower turbidity values (0—5 ppm)
(*b*) higher turbidity values (20—1000 ppm)
(*c*) equal turbidity values (0—2000 ppm)
(*d*) none of the above.

Q. 175. The suspended solids present in water may give colour to the water, which is known as :
(*a*) true colour (*b*) apparent colour
(*c*) colour (*d*) none of the above.

Q. 176. Pick up the correct statement(s) :
(*a*) turbidimeters are frequently used to measure turbidities of raw supplies
(*b*) turbidimeters are frequently installed on line in treatment plant, to measure turbidities of sedimented filtered waters
(*c*) Nephelometers are frequently used to check and measure the turbidities of final disinfected supplies
(*d*) all of the above.

Q. 177. One true colour unit is the colour produced by :
(*a*) one mg of formazin in one litre of distilled water
(*b*) one mg of silicon in one litre of distilled water
(*c*) one mg of ferric silicon in one litre of distilled water
(*d*) one mg of platinum as chloroplatinate ions in one litre of distilled water.

Q. 178. The colour of water, contributed by dissolved solids, is :
(*a*) apparent colour (*b*) true colour
(*c*) colour (*d*) (*b*) and (*c*) above.

Q. 179. The true colour of water is measured on :
(*a*) platinum cobalt scale (*b*) silica scale
(*c*) nickel scale (*d*) all of the above.

Q. 180. The colour of water for domestic supplies, on standard platinum cobalt scale, should not exceed :
(*a*) 0—5 ppm (*b*) 5—10 ppm
(*c*) 10—20 ppm (*d*) 20—50 ppm.

Q. 181. The specific conductivity of water helps in knowing the extent of :
(*a*) organic matter (*b*) inorganic matter
(*c*) dissolved salts (*d*) none of these.

Q. 182. With increase in temperature, the specific conductivity of water :
(*a*) decreases (*b*) increases
(*c*) remains unchanged.

Q. 183. The colour of water can be determined on an instrument, known as :
(*a*) dionic water tester (*b*) turbidimeter
(*c*) tintometer (*d*) none of the above.

Q. 184. The threshold odour number (TON) for a water sample of 40 ml, diluted to standard 200 ml mixture, in which odour is just barely detectable to the sense of smell, is :
(*a*) 8 (*b*) 5
(*c*) 50 (*d*) none of the above.

Q. 185. pH value of water indicates its :
(a) acidity (b) alkalinity
(c) both (a) and (b) above (d) none of these.

Q. 186.* The most common cause of acidity in water, is :
(a) oxygen (b) hydrogen
(c) nitrogen (d) carbon dioxide.

Q. 187. A water having 1 unit lesser pH, will be :
(a) 1 time more acidic (b) 10 times more acidic
(c) 100 times more acidic (d) none of these.

Q. 188. A water having pH equal to 7, is :
(a) acidic (b) alkaline
(c) neutral (d) none of these.

Q. 189. A water having pH more than 7, is :
(a) acidic (b) alkaline
(c) neutral (d) none of these.

Q. 190. A water having pH less than 7, is :
(a) acidic (b) alkaline
(c) neutral (d) none of these.

Q. 191. A water having pH = 9, will have hydrogen ion concentration equal to :
(a) 9 mol/l (b) 10^{-9} mol/l
(c) 10^{9} mole/l (d) none of these.

Q. 192. The product of H ions and OH ions in a stronger acid, is :
(a) 0 (b) 1
(c) 10^{-7} (d) 10^{-14}.

Q. 193. Higher values of pH indicate :
(a) stronger acids (b) stronger alkalis
(d) stronger contamination (d) none of the above.

Q. 194. Ethylene Diamine Tetra Acetic acid (EDTA) solution is used to determine, by titrating it against water, the :
(a) hardness of water
(b) turbidity of water
(c) dissolved oxygen in water
(d) residual chlorine in water.

Q. 195. A water having pH = 9, will have hydroxyl ion concentration, equal to :
(a) 10^{9} (b) 10^{-5}
(c) 10^{-9} (d) 10^{5}.

Q. 196. Temporary hardness in water is caused by :
(a) carbonates and bicarbonates of calcium and magnesium
(b) bicarbonates of sodium and potassium
(c) carbonates of calcium and magnesium
(d) dissolved carbon dioxide.

Q. 197. For a water sample having a total hardness of 200 mg/l as $CaCO_3$, and alkalinity of 250 mg/l as $CaCO_3$, the carbonate hardness is ..... and non-carbonate hardness is .....

Q. 198. One French degree of hardness equals to :
(a) 1 ppm (b) 10 ppm
(c) 14.25 ppm (d) none of the above.

Q. 199. One British degree of hardness equals to a hardness of :
(a) 1 ppm (b) 10 ppm
(c) 14.25 ppm (d) none of the above.

Q. 200. The most appropriate hardness, in drinking water supplies, is of the order of :
(a) 50 ppm (b) 100 ppm
(c) 200 ppm (d) 300 ppm.

Q. 201. Waters are considered 'hard', if their hardness is of the order of :
(a) 50 ppm (b) 100 ppm
(c) 200 ppm (d) 300 ppm.

Q. 202. Waters are considered 'soft', if their hardness does not exceed :
(a) 75 ppm (b) 100 ppm
(c) 150 ppm (d) 300 ppm.

Q. 203. Hard water is tastier than soft water, due to the presence of :
(a) sodium (b) calcium
(c) carbonates (d) bicarbonates.

Q. 204.* When fluoride concentration in water exceeds 1.5 mg/l or so, the disease that may be caused, is :
(a) methemoglobinemia (b) fluorosis
(c) dental caries in children (d) poliomyelitis.

Q. 205.* Dental caries in children may be caused due to water supplies which are deficient in :
(a) calcium (b) iron
(c) fluorides (d) none of these.

Q. 206. Higher quantities of copper, more than 2.5 mg/l or so, may cause diseases pertaining to :
(a) kidneys (b) lungs
(c) lever (d) teeth.

Q. 207. Blue baby disease may be caused in infants due to drinking waters, containing higher concentrations of :
(a) nitrites (b) nitrates
(c) lead (d) arsenic.

Q. 208. The only metal among the following, which is toxic to human beings, is :
(a) calcium (b) barium
(c) iron (d) magnesium.

Q. 209. The metal which is the most hazardous to human beings, among the following, is :
(a) Iron (b) Barium
(c) Silver (d) Arsenic
(e) Chromium.

Q. 210. The only metal among the following, which is not toxic to human beings, is :
(a) Sodium (b) Mercury
(c) Lead (d) Cadmium.

Q. 211. The maximum allowable concentration of iron in water is :
(a) 1.0 ppm (b) 0.05 ppm
(c) 0.3 ppm (d) 0.03 ppm.

Q. 212. The commonly used indicator for measuring iron concentration in water is :
(a) erichrome black T (b) 1, 10, phenanthroline
(c) phenolphthalein (d) blue litmus.

Q. 213. The maximum safe permissible limit of chlorides in domestic water supplies is :
(a) 0.5 mg/l (b) 2.5 mg/l
(c) 200 mg/l (d) 100 mg/l.

Q. 214. The maximum safe permissible limit of sulphates in domestic water supplies is :
(a) 100 mg/l (b) 200 mg/l
(c) 500 mg/l (d) 1000 mg/l.

Q. 215. The maximum permissible concentration of sulphates in waters to be used for curing and mixing of concrete is :
(a) 250 mg/l (b) 500 mg/l
(c) 1000 mg/l (d) 2000 mg/l.

Q. 216. Soap is a sodium salt of :
(a) acetic acid (b) stearic acid
(c) formic acid (d) oxalic acid.

Q. 217. A pathogenic organism of unicellular/protozoal group is :
(a) escherichia coli (b) salmonella typhi
(c) entamoeba hystolytica (d) none of the above.

Q. 218. The bacteria which survive in the absence of oxygen, are called :
(a) anaerobic (b) aerobic
(c) facultative (d) E-coli.

Q. 219. The bacteria which survive in the presence as well as absence of oxygen, are called :
(a) anaerobic (b) facultative
(c) B-coli (d) E-coli.

Q. 220.* A harmful organism, which may be present in fecal matter, may be :
(a) bacteria-coli (b) escherichia-coli
(c) vibrio cholerae (d) none of these.

Q. 221. Waters required for pulp and paper industry should generally be free from :
(a) iron (b) manganese
(c) hardness (d) all of the above.

Q. 222. Breweries and distilleries preferably require :
(a) hard waters (b) soft waters
(c) potable waters (d) none of the above.

Q. 223. For steel plants, the chloride content of process water, as compared to that of potable water, should be :
(a) higher (b) lower
(c) equal (d) all of these can do.

Q. 224. The waters to be used for boilers should preferably be :
(a) hard (b) soft
(c) potable (d) none of the above.

CHAPTER 9

Q. 225. The treatments which are generally given to treat raw water supplies, follow the sequence :
(a) screening, sedimentation, disinfection, filtration
(b) screening, sedimentation, filtration, disinfection
(c) screening, sedimentation, disinfection, aeration
(d) screening, sedimentation, coagulation, filtration, disinfection.

Q. 226. The fine screens are generally not used these days, in water treatment, as the fine suspended impurities are removed in :
(a) filtration (b) sedimentation
(c) aeration (d) disinfection.

Q. 227. The settling velocity of inorganic particles in a sedimentation tank of a water treatment plant, is governed by :
(a) Darcy's law (b) Dupuit's law
(c) Stoke's law (d) none of the above.

Q. 228. The settling velocity of inorganic particles of less than 0.1 mm dia, varies with the dia (d), in proportion to :
(a) d^3 (b) d^2
(c) d
(d) none of them, as dia does not affect the settling velocity.

Q. 229. The settling velocity of inorganic particles, larger than 0.1 mm dia, varies with the dia (d), in proportion to :
(a) d^3 (b) d^2
(c) d
(d) none of them, as dia (d) does not affect the settling velocity.

Q. 230. Sedimentation can remove inorganic particles, having specific gravity upto, say :
(a) 2.65 (b) 1.65
(c) 1.2 (d) 1.03.

Q. 231. Water sedimentation process involves the settling of the impurities in a tank, under the action of :
(a) sun rays
(b) gravitational force
(c) biological action
(d) flow velocity of particles.

Q. 232. If the temperature of a sedimentation tank is increased, the sedimentation speed will :
(a) get hastened
(b) get slowed down
(c) not be affected at all.

Q. 233. A water sedimentation tank, under normal conditions, can remove suspended impurities, up to, say :
(a) 30% (b) 50%
(c) 70% (d) 90%.

Q. 234. Particles intended to be removed in a continuous flow water sedimentation tank, should have settling velocity :
(a) more than the surface loading of the tank
(b) less than the surface loading of the tank
(c) neither of the above.

Q. 235. In order to exclude as many fine particles as possible in a continuous flow sedimentation tank, the surface loading of the tank should, as far as possible, be :
(a) higher (b) lower
(c) neither of the two.

Q. 236. Surface loading or overflow rate of a sedimentation tank, passing a discharge Q, and having length = L, depth = D, width = B, is given by :

(a) $\frac{Q}{B.D}$ (b) $\frac{Q}{B.L}$

(c) $Q.B.D$ (d) $\frac{Q}{B.D.L}$.

Q. 237. The efficiency of sediment removal in a continuous sedimentation tank does not depend upon the :
(a) discharge through the tank (b) width of the tank
(c) length of the tank (d) depth of the tank.

Q. 238. The depth of water sedimentation tanks is decided, only on practical considerations, and may range between :
(a) 1 to 1.8 m (b) 3 to 4.5 m
(c) 6 to 8 m (d) none of the above.

Q. 239. Detention time for a sedimentation tank (continuous flow type), is given for a tank, passing a discharge = Q, and having length = L, width = B, and depth = H, as :

(a) $\frac{BLH}{Q}$ (b) $\frac{Q}{B.L.H}$

(c) $\frac{Q}{B.L}$ (d) none of the above.

Q. 240. Flowing through period of a sedimentation tank, when compared to its detention time, is always :
(a) more (b) less
(c) equal (d) none of these.

Q. 241. When the raw water passes in a sedimentation tank, without being detained for the intended time, due to the existing currents in the flow, then the flow is said to be :
(a) long-circuited (b) short-circuited
(c) disturbed (d) none of these.

Q. 242. Displacement efficiency of a sedimentation tank is given by :
(a) detention period (b) flowing through period
(c) $\frac{(a)}{(b)}$ above (d) $\frac{(b)}{(a)}$ above.

Q. 243. Displacement efficiency of water sedimentation tanks, generally varies, between :
(a) 0 to 25% (b) 25 to 50%
(c) 50 to 75% (d) 75% to 100%.

Q. 244. Surface loading or overflow velocity of a sedimentation tank (plain) may vary in the range :
(a) 100—500 l/hr/m^2
(b) 500—750 l/hr/m^2
(c) 1000—1250 l/hr/m^2
(d) none of the above.

Q. 245. Surface loading or overflow rate of a coagulant-sedimentation tank may vary between :
(a) 100—500 l/hr/m^2
(b) 500—750 l/hr/m^2
(c) 1000—1250 l/hr/m^2
(d) none of the above.

Q. 246. The detention time for a water sedimentation tank may vary between :
(a) 1—2 hr (b) 2—4 hr
(c) 4—8 hr (d) 16—24 hr.

Q. 247. The detention time for a water sedimentation tank, using coagulated raw supplies, may vary between :
(a) 1—2 hr (b) 2—4 hr
(c) 4—8 hr (d) 16—24 hr.

Q. 248. The horizontal flow velocity in a sedimentation tank, passing a discharge = Q, and having length = L, width = B, depth = H is given by :

(a) $\frac{Q}{B.H}$ (b) $\frac{Q}{B.L}$

(c) $B.H$ (d) none of the above.

Q. 249. The velocity of flow of water in a sedimentation tank may normally be kept between :

(a) 15—30 cm/sec (b) 15—30 cm/min

(c) 15—30 cm/hr (d) none of these.

Q. 250. For a given discharge, the efficiency of a sedimentation tank can be increased by :

(a) increasing the depth of the tank

(b) decreasing the depth of the tank

(c) increasing the surface area of the tank

(d) decreasing the surface area of the tank.

Q. 251. The most widely used coagulant for water-treatment, is :

(a) lime and soda (b) ferrous sulphate

(c) chlorinated copperas (d) alum.

Q. 252. The amount of coagulant needed in water treatment, increases with the :

(a) increase in temperature of water

(b) increase in turbidity of water

(c) decrease in turbidity of water

(d) none of the above.

Q. 253. Coagulants, used in water treatment, function better when the raw water is :

(a) acidic (b) alkaline

(c) neutral (d) none of the above.

Q. 254. Chlorinated copperas may be a preferred coagulant, even in water treatment, when raw water is :

(a) acidic (b) alkaline

(c) neutral (d) none of the above.

Q. 255. A clariflocculator is a :

(a) plain sedimentation unit

(b) aeration unit

(c) coagulation-sedimentation unit

(d) none of the above.

Q. 256*. Particles of around 1 micron (10^{-6} m) size are best removed by :

(a) filtration (b) plain sedimentation

(c) chemical precipitation (d) chemical coagulation.

(GATE—90)

Q. 257. In water treatment, slow sand filters, when compared to rapid gravity filters, produce :

(a) lesser contaminated effluent

(b) more contaminated effluent

(c) equally contaminated effluent

Q. 258. Slow sand filters when compared to rapid gravity filters, may normally, filter :

(a) 5 times slower (b) 10 times slower

(c) 20 times slower (d) 30 times slower.

Q. 259. Slow sand filters remove bacteria, to as much as :

(a) 80—90% (b) 90—95%

(c) 98—99% (d) none the these.

Q. 260. Rapid gravity filters remove bacteria to as much as :

(a) 80—90% (b) 90—95%

(c) 98—99% (d) none the these.

Q. 261. Cleaning of slow sand filters is done by :

(a) scraping and removal of sand (b) back washing

(c) any of the above (d) none of the above.

Q. 262. Cleaning of rapid sand filters is done by :

(a) scraping and removal of sand (b) back washing

(c) any of the above (d) none of the above.

Q. 263.* The rate of filtration in slow sand filters is of the order of :

(a) 10—20 Ml/d/ha (b) 25—50 Ml/d/ha

(c) 100—500 Ml/d/ha (d) 700—1400 Ml/d/ha.

Q. 264. The rate of filtration in rapid gravity filters is of the order of :

(a) 10—20 Ml/d/ha (b) 25—50 Ml/d/ha

(c) 100—500 Ml/d/ha (d) 700—1400 Ml/d/ha.

Q. 265. The periodicity, with which slow sand filters need cleaning, is of the order of :

(a) 24—48 hours (b) 10—15 days

(c) 1—3 months (d) 1—2 years.

Q. 266. The periodicity, with which rapid gravity filters need cleaning, is of the order of :

(a) 24—48 hours (b) 10—15 days

(c) 1—3 months (d) 1—2 years.

Q. 267. The percentage of filtered water, which is used for back washing a rapid gravity filter, is about :

(a) 0.2—0.5 (b) 0.5—1.0

(c) 2—4 (d) 5—7.

Q. 268. Back washing of rapid gravity filters, may face rough weather, due to :

(a) air-binding (b) mud-balls

(c) negative head (d) cracking of filters.

Q. 269. Air-binding phenomenon in rapid sand filters may occur due to :
(a) excessive negative pressure
(b) mud ball formation
(c) higher turbidity in the effluent
(d) low temperature.

Q. 270. The quality of the filtered effluent and the working of the filter, in a rapid gravity filter, is adversely affected by :
(a) air-binding (b) mud-balls
(c) cracking of filter (d) none of these.

Q. 271. A pressure filter is a compact unit, which carries out the job, as accomplished by a :
(a) flocculation tank (b) sedimentation tank
(c) filtration unit (d) all of the above in one unit.

Q. 272. The rate of filtration in pressure filters, when compared to that of rapid gravity filters, is about :
(a) half (b) twice
(c) 2 to 5 times (d) 5 to 10 times.

Q. 273. The quality of pressure filter-effluents, when compared with that of rapid gravity filters, is :
(a) poorer (b) better
(c) equally good.

Q. 274. A roughening filter, as used in treating water supplies, is like a :
(a) slow sand filter
(b) rapid gravity filter
(c) rapid gravity filter with coarser sand
(d) none of the above.

Q. 275. Double filtration is used :
(a) to increase the filtration capacity of slow sand filters
(b) to increase the filtration capacity of rapid sand filters
(c) for isolated colonies of a few houses
(d) for none of the above.

Q. 276. In double filtration, the roughening filter unit is placed :
(a) prior to the slow sand filter (b) after the slow sand filter
(c) any of (a) and (b) above (d) none of the above.

Q. 277. Disinfection of water helps in :
(a) removing turbidity (b) removing hardness
(c) killing pathogenic bacteria (d) complete sterilisation.

Q. 278. The efficiency of disinfection by chlorine, in water treatment, increases by :
(a) decrease in time of contact
(b) decrease in temperature of water
(c) increase in temperature of water
(d) none of the above.

Q. 279. Chlorine demand of water is equal to :
(a) applied chlorine (b) residual chlorine
(c) (a) + (b) above (d) (a) – (b) above.

Q. 280. The process, which involves chlorination beyond break point chlorination, is known as :
(a) prechlorination (b) super chlorination
(c) post chlorination (d) dechlorination.

Q. 281. The percentage of chlorine in fresh bleaching powder is about :
(a) 10—15 (b) 20—25
(c) 30—35 (d) 50—60.

Q. 282. The treatment of water with bleaching powder is known as :
(a) prechlorination (b) dechlorination
(c) super chlorination (d) hypochlorination.

Q. 283. Which of the following chemical compounds can be used for dechlorination of water :
(a) carbon dioxide (b) bleaching powder
(c) sulphur dioxide (d) chloramines.

Q. 284. The suitable method for disinfection of swimming pool water is :
(a) ultra violet rays treatment
(b) lime treatment
(c) chlorination
(d) use of pottasium permanganate.

Q. 285. Disinfection efficiency of chlorine treatment :
(a) is decreased at higher pH values of water
(b) is increased at higher pH values of water
(c) is unaffected at higher pH values of water
(d) is highest at pH = 7 of water.

Q. 286. At lower pH values of water, the contact period required for chlorination, is :
(a) lower (b) higher
(c) same (d) none of the above.

Q. 287. In lime-soda process of water softening :
(a) only carbonate hardness is removed
(b) only noncarbonate hardness is removed
(c) lime reduces noncarbonate hardness, and soda removes carbonate hardness
(d) lime reduces carbonate hardness, and soda removes non-carbonate hardness.

Q. 288. The main disadvantage of lime-soda process of water softening is that :
(a) it is unsuitable for turbid and acidic waters
(b) zero hardness-effluent, can not be obtained

(c) excessive hard waters cannot be softened by it
(d) huge amount of precipitate is formed, posing disposal problem.

Q. 289. Which of the following compounds is used in algae control?
(a) ferric sulphate (b) calcium chloride
(c) copper sulphate (d) ferric chloride.

Q. 290. Activated carbon is used in water treatment for removing:
(a) colour (b) tastes and odours
(c) turbidity (d) corrosiveness.

Q. 291. Activated carbon is added to water to remove tastes and odours:
(a) before coagulation (b) after coagulation
(c) before filtration (d) any of the above.

Q. 292. Iron and manganese can be removed from waters, by:
(a) boiling
(b) aeration followed by coagulation
(c) chlorination
(d) activated carbon addition.

Q. 293. The process of desalination of water, which makes use of microporous membranes, is:
(a) electrodialysis (b) solar distillation
(c) freezing (d) none of the above.

Q. 294. The process of desalination, which makes use of microporous membranes, is:
(a) solar distillation (b) reverse osmosis
(c) evaporation and distillation (d) none of the above.

CHAPTER 10

Q. 295. The suitable layout for a water supply distribution system, for an irregularly grown town, is:
(a) dead end system (b) grid iron system
(c) ring system (d) radial system.

Q. 296. The suitable layout for a water supply distribution system, for a city of roads of rectangular pattern, is:
(a) dead end system (b) grid iron system
(c) ring system (d) radial system.

Q. 297. The method, which is most widely used for analysing and designing the pipes of all types of complex water distribution networks, is:
(a) equivalent pipe method (b) Hardy cross method
(c) circle method (d) all of the above.

Q. 298. The design technique, adopted in design of large water supply networks, as an aid to simplify and separate the smaller loops, is:
(a) Hardy cross method (b) Circle method
(c) electrical analyser method (d) equivalent pipe method.

Q. 299. The water meter, which is installed on individual house connections, on municipal supplies, is:
(a) a velocity meter (b) an inferential meter
(c) a displacement meter (d) none of the above.

Q. 300. The water-tap of your house is known as:
(a) sluice tap (b) stop cock
(c) bib cock (d) ferrule.

ANSWERS TO OBJECTIVE QUESTIONS

1. (b)	2. (d)	3. (b)	4. (a)	5. (d)
6. (d)	7. (d)	8. (b)	9. (c)	10. (d)
11. (a)	12. (b)	13. (b)	14. (a)	15. (c)
16. (a)	17. (d)	18. (b)	19. (c)	20. (c)
21. (a)	22. (c)	23. (b)	24. (c)	25. (c)
26. (d)	27. (c)	28. (a)	29. (c)	30. (a)
31. (c)	32. (c)	33. (c)	34. (a)	35. (c)
36. (a)	37. (c)	38. (b)	39. (c)	40. (c)
41. (b)	42. (b)	43. (d)	44. (a)	45. (d)
46. (c)	47. (d)	48. (b)	49. (c)	50. (c)
51. (b)	52. (b)	53. (a)	54. (b)	55. (b)
56. (a)	57. (c)	58. (a)	59. (b)	60. (c)
61. (b)	62. (b)	63. (c)	64. (a)	65. (b)
66. (b)	67. (b)	68. (d)	69. (d)	70. (b)
71. (d)	72. (d)	73. (a)	74. (c)	75. (b)
76. (c)	77. (c)	78. (b)	79. (b)	80. (c)
81. (c)	82. (b)	83. (d)	84. (b)	85. (a)
86. (b)	87. (b)	88. (c)	89. (d)	90. (a)
91. (b)	92. (d)	93. (d)	94. (d)	95. (a)
96. (b)	97. (c)	98. (c)	99. (a)	100. (c)
101. (d)	102. (d)	103. (b)	104. (c)	105. (b)
106. (b)	107. (d)	108. (a)	109. (b)	110. (d)
111. (a)—*ii* (b)—*iii* (c)—*i*		112. (d)	113. (c)	114. (b)
115. (a)	116. (b)	117. (b)	118. (c)	119. (b)
120. (d)	121. (b)	122. (a)	123. (c)	124. (a)
125. (b)	126. (a)	127. (b)	128. (a)	129. (d)
130. (a)	131. (d)	132. (a)	133. (a)	134. (b)
135. (a)	136. (b)	137. (b)	138. (a)	139. (c)
140. (b)	141. (a)	142. (a)	143. (b)	144. (b)
145. (c)	146. (c)	147. (d)	148. (b)	149. (c)
150. (d)	151. (b)	152. (c)	153. (d)	154. (c)
155. (b)	156. (b)	157. (c)	158. (c)	159. (b)
160. (b)	161. (a)	162. (b)	163. (a)	164. (d)
165. (b)	166. (d)	167. (b)	168. (c)	169. (d)
170. (a)	171. (a)	172. (d)	173. (a)	174. (a)

175. (*b*)	**176.** (*d*)	**177.** (*d*)	**178.** (*d*)	**179.** (*a*)
180. (*c*)	**181.** (*c*)	**182.** (*b*)	**183.** (*c*)	**184.** (*b*)
185. (*c*)	**186.** (*d*)	**187.** (*b*)	**188.** (*c*)	**189.** (*b*)
190. (*a*)	**191.** (*b*)	**192.** (*d*)	**193.** (*b*)	**194.** (*a*)
195. (*b*)	**196.** (*a*)	**197.** 200 mg/l, zero		**198.** (*b*)
199. (*c*)	**200.** (*b*)	**201.** (*d*)	**202.** (*a*)	**203.** (*d*)
204. (*b*)	**205.** (*c*)	**206.** (*b*)	**207.** (*b*)	**208.** (*b*)
209. (*c*)	**210.** (*a*)	**211.** (*c*)	**212.** (*b*)	**213.** (*c*)
214. (*b*)	**215.** (*b*)	**216.** (*b*)	**217.** (*c*)	**218.** (*a*)
219. (*b*)	**220.** (*c*)	**221.** (*d*)	**222.** (*a*)	**223.** (*b*)
224. (*b*)	**225.** (*b*)	**226.** (*b*)	**227.** (*c*)	**228.** (*b*)
229. (*c*)	**230.** (*c*)	**231.** (*b*)	**232.** (*a*)	**233.** (*c*)
234. (*a*)	**235.** (*b*)	**236.** (*b*)	**237.** (*d*)	**238.** (*b*)
239. (*a*)	**240.** (*b*)	**241.** (*b*)	**242.** (*d*)	**243.** (*b*)
244. (*b*)	**245.** (*c*)	**246.** (*c*)	**247.** (*b*)	**248.** (*c*)
249. (*b*)	**250.** (*c*)	**251.** (*d*)	**252.** (*b*)	**253.** (*b*)
254. (*a*)	**255.** (*c*)	**256.** (*d*)	**257.** (*a*)	**258.** (*d*)
259. (*c*)	**260.** (*a*)	**261.** (*a*)	**262.** (*b*)	**263.** (*b*)
264. (*d*)	**265.** (*c*)	**266.** (*a*)	**267.** (*c*)	**268.** (*b*)
269. (*a*)	**270.** (*a*)	**271.** (*d*)	**272.** (*c*)	**273.** (*a*)
274. (*c*)	**275.** (*a*)	**276.** (*a*)	**277.** (*c*)	**278.** (*c*)
279. (*d*)	**280.** (*b*)	**281.** (*c*)	**282.** (*d*)	**283.** (*c*)
284. (*a*)	**285.** (*a*)	**286.** (*a*)	**287.** (*d*)	**288.** (*d*)
289. (*c*)	**290.** (*b*)	**291.** (*d*)	**292.** (*b*)	**293.** (*a*)
294. (*b*)	**295.** (*a*)	**296.** (*b*)	**297.** (*b*)	**298.** (*d*)
299. (*c*)	**300.** (*c*)			

Hints to Solve Starred Questions

Q. 69. Sources at (*a*), (*b*) and (*c*) all, will yield underground water, which is saline, and hence '*River*' can give us good supply.

Q. 74. Use Eqn. $\frac{t_1}{r_1^2} = \frac{t_2}{r_2^2}$ (Please see Solved Example 4.13).

Q. 104. $t = \frac{pd}{2f} = \frac{20 \times 1}{2 \times 770} = 13$ mm ; add 2 mm for corrosion.

Q. 146. As the discharge used by the residents vary with time, depending on their needs, pump will have to work to give variable discharge, thereby giving reduced efficiency.

Q. 160. $H \approx 4 +$ losses ≈ 50 m (say)

$$\frac{wQH}{75\eta} = 1000 \times \frac{1}{40} \times \frac{50}{75 \times 0.65} = 25.$$

Q. 186. CO_2 reacts with water to form carbonic acid.

Q. 204. *Higher concentrations of fluorides* in drinking waters, exceeding about 1.5 mg/l or so, such as found at several places in India, cause **mottling of teeth,** called **fluorosis**. The disease increases with the increasing

fluoride consumption ; and excessive consumption of fluorides over long periods may even cause *bone problems*, including *crippling of bones*. *Mottled teeth* are characterised by black spots and streaks found on the teeth, and with the increasing consumption of excess fluorides, the teeth may even become brittle and weak. This disease particularly occurs in children and in teenagers during the calcination of their permanent teeth, which usually continues from 5 years of age to 17 years of age.

Q. 205. *Lower concentrations of fluorides* in drinking waters, lesser than about 1 mg/l or so, such as found at several places in USA, is found to be causing **dental caries** (*tooth decay*). Optimum fluoride concentrations (0.7—1.2 mg/l) in the drinking waters, infact, are useful to the teeth, as the fluorides are absorbed by the enamel during the growth of teeth, to resist the acids that form cavities* in the teeth. Presence of optimum fluorides in waters are therefore helpful in reducing cavity formation in the teeth of children, and the resultant tooth decay. Fluoridation of public supplies and the use of tooth pastes containing fluorides, are therefore very common in America. However, Indian water supplies are generally rich in fluorides, and do not need fluoridation.

Q. 220. *Vibriocholarae* are the pathogenic bacteria, causing cholera. E-coli and B-coli are harmless bacteria, as explained in the text.

Q. 256. Particles of size larger than 10^{-2} mm (10^{-5} m or 10 micron) are classified as *settleable solids*. Smaller particles in the size range of 10^{-2} mm (10 μm) to 10^{-6} mm (10^{-3} μm) are removed only by *coagulation*, followed by sedimentation, since they donot settle down of their own in plain sedimentation.

The term '*chemical precipitation*' is ruled out since it covers *only the removal of dissolved ions from water by addition of certain chemicals, which precipitate the impurity particles by changing their liquid phase to the solid phase, such as in the removal of hardness causing salts by the addition of lime or soda, etc.*

Note. The terms, 'chemical precipitation' and 'chemical coagulation' differ in the above respect, in the strict sense, although they have often been used as synonyms to each other.

Q. 263. 100—200 l/hr/m^2 = 24—48 Ml/day/ha
≈ 25—50 Ml/day/ha.

*Cavities in teeth need filling and treatment, and if not checked, may cause gradual tooth decay, as the cavities may progress through the *enamel*, *dentin*, and *pulp*, ultimately causing the tooth to come out.

—17

Conventional Questions of the Engineering Services Exams.

The Union Public Service Commission, Dholpur House, Shahjahan Road, New Delhi-110011, conducts every year two competitions, in which, you as civil engineering graduates (even with exam. results awaited) can participate for securing a direct class I Govt. appointment.

The first competition in named as the **Combined Engineering Services (CES) Exam** and the other competition is known as the **Civil Services Exam**, popularly called *IAS competition*.

Press notifications for submitting application forms, are released by Secretary UPSC, through leading news papers including "Employment News" sometimes in February every year. The exams are usually held in August.

On the basis of the first competition, you are likely to be appointed as Asstt. Executive Engineer or Asstt. Director or on an equivalent post in civil engineering side of the various Government Departments, like Railways, CPWD, P & T, Central Water Commission, etc., whereas, on the basis of the second competition, you are liable to become an IFS, IAS, IPS, or an officer on allied services, such as Asstt. Custom Collector or Asstt. Excise Collector, etc.

Most of the engineering degree students aspire to clear any one of these two competitions, for an excellent career full of promotional avenues, just after doing the BE degree. The syllabuses for both these competitions, now a days, provide full opportunities to Civil Engineering degree holders, to make through these competitions. We will now discuss the syllabuses and examination schemes, as may be of use to you as civil engineering graduates, for these competitions.

COMBINED ENGINEERING SERVICES EXAM. (Civil Engg.)

In this competition, there will be 5 papers in all ; 3 being of objective type, and 2 of conventional type. Each paper carries 200 marks, with a total of 1000 marks. 200 marks are reserved for Personal Interview, making a grand total of 1200 marks, as below :

Section I—Objective Type

1.	*General Ability Test*	2 hr.	200 marks
2.	*Civil Engineering Paper I*	2 hr.	200 marks
3.	*Civil Engineering Paper II*	2 hr.	200 marks

Section II—Conventional Papers

4.	*Civil Engineering Paper I*	3 hr.	200 marks
5.	*Civil Engineering Paper II*	3 hr.	200 marks
	Total :		1000 marks
	Personality Test cum Interview		200 marks
	Grand Total Marks :		1200

The syllabus for the above papers is given in *"Water Resources Engineering Vol. I—Hydrology and Water Resources Engineering"* written by the same author. The conventional questions of these exams. of the past years on 'Water Supply Engineering' are, however, given below.

Engineering Services Exam. Questions of last years on Water Supply Engg. *(Civil Engg. Paper II)*

Year 1979 (*Engineering Services*)

Q. 6. *(a) In a continuous flow settling tank 3 m deep and 60 m long, what flow velocity of water would you recommend for effective removal of 0.025 mm particles at 25°C. The specific gravity of particles is 2.65 and kinematic viscosity ν for water may be taken as 0.01 cm²/sec.*

[Note. *Refer solved example 9.6*]

Year 1980 (*Engineering Services*)

Q. 1. *(b) A water supply system consisting of a reservoir with lift pump, elevated storage tank, piping and withdrawal points is shown in the diagram below :*

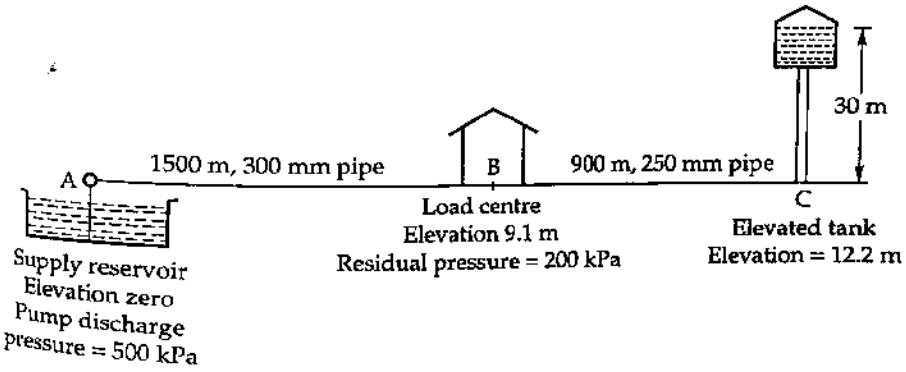


Fig. 17.1

(i) Based on the following data, sketch the hydraulic gradient for the system :

$Z_A = 0$ *m*, $P_A = 550$ *kPa*, $Z_B = 9.1$ *m*, $P_B = 200$ *kPa*,
$Z_C = 12.2$ *m*, $P_C = 30$ *m (water level in the tank)*

(ii) For these conditions, compute the flow available at point B from both the supply pumps and elevated storage. Use C = 100, and pipe sizes as shown in the diagram. Hydraulic values are given in the table below :

or $$\frac{\partial}{\partial v}\left[\rho \cdot a \cdot 2g \cdot v\left(H - \frac{4f \cdot L \cdot V^2}{2gD}\right)\right] = 0.$$

Where $av = A.V$ where A, V and D relates to the hose pipe ; and a, v and d to the nozzle

or $$\frac{\partial}{\partial v}\left[\rho a \cdot 2g \cdot v\left(H - \frac{4f.L}{2gD} v^2 \cdot \frac{a^2}{A^2}\right)\right] = 0$$

or $$\frac{\partial}{\partial v}\left[\rho \cdot a \cdot 2g\left(H \cdot v - \frac{4f.L}{2gD} v^3 \cdot \frac{a^2}{A^2}\right)\right] = 0$$

or $$\rho.a.2g.\left[H - \frac{4f.L}{2gD} 3v^2 \cdot \frac{a^2}{A^2}\right] = 0$$

or $$\rho a.2g \cdot \left[H - \frac{4f.L}{2gD} v^2 \cdot \frac{a^2}{A^2} \cdot 3\right] = 0$$

or $$(H - 3.H_L) = 0$$

$\therefore$ $$H_L = \frac{H}{3}.$$

Hence, for the momentum of the issuing jet to be maximum, H_L will be equal to $\frac{H}{3}$.

$\therefore$ $$H_L = \frac{4f.LV^2}{2gD} = \frac{H}{3}$$

or $$\frac{4 \times 0.032 \times 75}{2 \times 9.81 \times 0.075} V^2 = \frac{30}{3} \qquad [\because \quad H = 3 \text{ kg/cm}^2 = 30 \text{ m head of water}]$$

or $$V^2 = \frac{10 \times 2 \times 9.81 \times 0.075}{4 \times 0.032 \times 75}$$

or $$V = 1.238 \text{ m/sec.}$$

Now, $$a \cdot v = A.V$$

or $$\frac{\pi}{4} d^2 \cdot v = \frac{\pi}{4} \times (0.075)^2 \times 1.238$$

$\therefore$ $$v = \frac{(0.075)^2 \times 1.238}{d^2}$$

or $$v = \frac{0.00696}{d^2} \qquad ...(i)$$

Also $$\frac{v^2}{2g} = H - H_L = H - \frac{H}{3} = \frac{2}{3} H$$

$$= \frac{2}{3} \times 30 = 20$$

$\therefore$ $$v^2 = 2 \times 9.81 \times 20 = 392.4 \qquad ...(ii)$$

or $$v = 19.81 \text{ m/sec.}$$

$\therefore$ $$\frac{0.00696}{d^2} = 19.81$$

or $$d = \sqrt{\frac{0.00696}{19.81}} = 0.01875 \text{ m}$$

$$= 1.875 \text{ cm} = 18.75 \text{ mm.} \quad \textbf{Ans.}$$

[**Alternatively** The formula worked out in Fluid Mechanics for such a case, on the above principles, can be directly used as

$$d = \left[\frac{D^5}{8 \cdot f \cdot L}\right]^{1/4}$$

$$= \left[\frac{(0.075)^5}{8 \times 0.032 \times 75}\right]^{1/4} \qquad ...(17.1)$$

$$= 0.01875 \text{ m} = 1.875 \text{ cm} = 18.75. \quad \textbf{Ans.} \quad]$$

Q. 7. (*b*) *Describe briefly the theories of "chemical coagulation".* (10)

(*c*) *Discuss relative merits of Rapid Sand Filters and Slow Sand Filters, indicating the conditions favourable for the choice of each.*

Describe the working of any one of them with the aid of neat sketches. (15)

Year 1984 (*Engineering Services*)

Q. 6. (*b*) *Why is chlorination considered necessary in the case of all public water supplies even when filtration is provided ? Mention the various methods by which chlorination of water is effected.* (12)

(*c*) *Write short notes on (i) pH value ; and (ii) BOD.* (12)

Year 1985 (*Engineering Services*)

Q. 1. (*a*) *Two pipes each of length L and diameter D_1 and D_2 are arranged in parallel ; the loss of head, when a total quantity of water Q flows through them being h_1. If the pipes are arranged in series and the same quantity of water, Q, flows through them, the loss of head is h_2. If $D_1 = 2D_2$, find the ratio of h_1 and h_2. Neglect minor losses and assume friction factor, f, to be constant.* (20)

[**Solution.** When pipes are connected in parallel, then

$$\left(\frac{4}{\pi}\right)^2 \frac{f \cdot L \cdot Q_1^2}{2g \cdot D_1^5} = \left(\frac{4}{\pi}\right)^2 \frac{f \cdot L \cdot Q_2^2}{2gD_2^5} = h_1 \qquad [f = 4\,f]$$

or $$\left(\frac{4}{\pi}\right)^2 \frac{f \cdot L \cdot Q_1^2}{2g(2D_2)^5} = \left(\frac{4}{\pi}\right)^2 \frac{f \cdot L\,(Q - Q_1)^2}{2g \cdot D_2^5} = h_1$$

$(\because \quad D_1 = 2D_2 \text{ and } Q_2 = Q - Q_1)$

(ii) A steel penstock 60 cm in diameter has a shell thickness of 1.2 cm. The modulus of elasticity of the pipe shell material is 2.4×10^6 kg/cm^2 (2.1×10^5 N/mm^2) with Poisson's ratio of $\frac{1}{4}$ and the volume of modulus of elasticity of water as 2.1×10^4 kg/cm^2 (2.1×10^3 N/mm^2). The pipe is designed to discharge water at mean velocity of 2.1 m/sec. Determine the water hammer pressure rise caused by sudden closure of the valve at the downstream end : (1) neglecting the elasticity of the pipe material ; and (2) considering the elasticity of the pipe material. (10)

[**Note.** *Refer solved example 6.12*]

Year 1991 (*Engineering Services*)

Q. 2. (*a*) *Draw the flow sheet showing sequence of various treatment units of a typical water treatment plant. Also list these treatment units sequentially.* (15)

Q. 5. (*a*) *Enumerate the assumptions on which Dupuit-Thiem theory of ground-water flow is based, and state the range of validity of the law governing the flow. Define the following terms*

(*i*) *Storage coefficient* ; (*ii*) *Specific yield* ;

(*iii*) *Specific retention* ; *and* (*iv*) *Porosity.*

stating clearly the relationship which exists among some of these terms. (20)

Year 1992 (*Engineering Services*)

Q. 7. (*a*) *A settling basin is designed to have a surface overflow rate of 32.6 m/day. Determine the overall removal obtained for a suspension with size distribution given below. The specific gravity of the particles is 1.2 and the water temperature is 20°C, at which the dynamic viscosity is 1.027 centipoise and the density 0.997 g/cm^3.*

Particle size, mm	*0.10*	*0.08*	*0.07*	*0.06*	*0.04*	*0.02*	*0.01*
Weight fraction greater than size (per cent)	*10*	*15*	*40*	*70*	*93*	*99*	*100*

(25)

[**Note.** *Please refer solved example 9.9*]

Q. 8. (*a*) *Water flows in a 80 mm pipe at Reynolds number 80,000. The pipe is estimated to have an equivalent sand grain roughness of size 0.16 mm. Determine the head loss expected in 500 mm length of the pipe. How much head loss would be expected if this pipe were smooth ? Take kinematic viscosity of water as 10^{-6} m^2/s. The following explicit equation for friction factor may be used*

$$\frac{1}{\sqrt{f}} = 1.14 - 2 \log \left(\frac{k}{d} + \frac{21.25}{R^{0.9}} \right)$$

in which k is the equivalent sand grain roughness, d the pipe diameter, and R the pipe Reynolds number. (15)

[**Note.** *Please refer solved example 6.9*]

Q. 8. (*c*) *For water supply of a town, water is pumped from a river 3 km away into a reservoir. The maximum difference of levels of water in river and the reservoir is 20 m. The population of the town is 50,000 and per capita water demand is 120 litre per day. If the pumps are to operate for a total of 8 hours and the efficiency of pumps is 80%, determine the horsepower of the pumps. Assume friction factor as 0.03, the velocity of flow as 2 m/s, and maximum daily demand as 1.5 times the average daily demand.* (15)

[**Note.** *Please refer solved example 7.8*]

Year 1993 (*Engineering Services*)

Q. 2. (*b*) *Make an itemwise comparison of slow sand and rapid gravity filters and show it in a tabular form.* (15)

[**Hint.** *Refer table 9.5*]

Q. 4. (*c*) *In a wide rectangular channel, if the normal depth is increased by 20%, determine the corresponding percentage increase in the discharge, use Manning's equation.*

[**Solution.** $Q = \frac{1}{n} . A R^{2/3} . \sqrt{S}$

$\therefore \quad Q = \frac{1}{n} . (B.y) . y^{2/3} . \sqrt{S}$ [$\because$ for wide channels $R \approx y$]

or $\quad Q \propto y^{5/3}$

$\therefore \quad Q_1 = K . y_1^{5/3}$

$Q_2 = K . (1.2\, y_1)^{5/3} = 1.3552\, K . y_1^{5/3}$

%age increase in discharge

$= \frac{Q_2 - Q_1}{Q_1} \times 100 =$ **35.52%. Ans.**]

Q. 5. (*b*) *Pressures on the suction side and the discharge side of a pump are read by the pressure gauge as :*

(*i*) *Vacuum of 25 cm of mercury, and* (*ii*) *1.4 kg/cm^2 respectively. The gauge on the suction side is placed 60 cm below the centre line of the pump, whereas the gauge on the discharge side is placed 17 cm above the centre line. If the diameters of pipes on suction and discharge sides are 15 cm and 10 cm respectively, and the quantity of water to be pumped is 30 lit/sec, calculate the horsepower required for the pump, assuming its efficiency as 60%.* (15)

[**Solution.** *On suction side (1)*

$p_1 = -$ 25 cm of Hg $= -$ 250 mm of Hg

$= \frac{-250}{760} \times 10.33$ m of water head = (–) 3.4 m

[$\because$ 760 mm of Hg = 1 At. Pressure = 10.33 m of water
Pl. see Appendix table A-7]

$Z_1 =$ (–) 60 cm = (–) 0.6 m

$$\text{Velocity head} = \frac{V_1^2}{2g} = \frac{(Q_1/A_1)^2}{2g}$$

$$= \left[\frac{(30/1000)}{\pi/4 \times (15/100)^2}\right]^2 \cdot \frac{1}{2 \times 9.81}$$

$$= 0.15 \text{ m}$$

Total pressure at suction end

$$= Z_1 + p_1 + \frac{V_1^2}{2g} = (-)\ 0.6 + (-)\ 3.4 + 0.15$$

$$= (-)\ \mathbf{3.85\ m} \quad \text{...(i)}$$

On discharge side (2)

$p_2 = 1.4$ kg/cm^2 = 14 m of water head

$Z_2 = +\ 0.17$ m (given)

$$\text{Velocity head} = \frac{V_2^2}{2g}$$

$$= \left[\frac{(30/1000)}{\pi/4 \times (10/100)^2}\right]^2 \times \frac{1}{2 \times 9.81}$$

$$= 0.74 \text{ m}$$

Total pressure at discharge end

$$= Z_2 + p_2 + \frac{V_2^2}{2g} = 14 + 0.17 + 0.74$$

$$= \mathbf{14.91\ m} \quad \text{...(ii)}$$

Diff. in pressure head between 1 and 2

$$= 14.91 - (-\ 3.85) = \mathbf{18.76\ m}$$

Required H.P. of pump

$$= \left(\frac{w \cdot QH}{75}\right) \cdot \frac{1}{\eta}$$

$$= \left(\frac{1000 \times \frac{30}{1000} \times 18.76}{75}\right) \frac{1}{0.6}$$

$$= \mathbf{12.5\ H.P.}\quad \textbf{Ans.}$$

Year 1994 (*Engineering Services*)

Q. 1. (*a*) *A 0.5 m diameter and 100 m long pipe line carrying 0.5 m^3/sec discharge is fitted with a valve at downstream end. Calculate the rise of pressure caused within the pipe due to the valve closure in (i) 1 second; and (ii) instantaneously.*

Take the sonic velocity as 1430 m/s. (10)

[**Note.** *Refer solved example 6.13*]

Q. 2. (*a*) *A coagulation treatment plant with a flow of 0.5 m^3/sec is dosing alum at 23 mg/L. No other chemicals are being added. The raw water suspended solids concentration is 37 mg/L. The effluent suspended solids concentration is measured at 12 mg/L. The sludge content is 1 percent and the specific gravity of sludge solids is 3.01. What volume of sludge must be disposed of each day ? Based on your result, remark with reference to water treatment process.* (20)

[**Note.** *Refer solved example 9.15*]

Q. 6. (*c*) *A pump lifts 93,600 litre of water per hour against a total head of 21 m. Pump has an efficiency of 72 percent. If the direct drive electric motor, having an efficiency of 80 percent is used to operate the pump, compute the cost of electric energy in a month. The pump is operated 12 hours daily for 30 days. The cost of electric energy is 65 np per unit.* (10)

[**Hint.**

$$\text{H.P.} = \frac{w \cdot QH}{75 \cdot \eta} = \frac{1000 \times (93.6/3600) \times 21}{75 \times (0.72 \times 0.8)}$$

$$= 12.64$$

Electrical energy consumed in 1 month

$$= 12.64 \times 0.735 \times (12 \times 30) \text{ kWh}$$

($\because$ 1 H.P. = 0.735 kW)

$$= 3344 \text{ kWh}$$

Cost of electrical energy per month

$$= ₹\ 0.65\ \text{P} \times 3344 = \mathbf{₹\ 2174.}\quad \textbf{Ans.}\ ]$$

Year 1995 (*Engineering Services*)

Q. 2. (*b*) *Determine the surface area of a settling tank for 0.5 m^3/sec design flow, using the design overflow rate as 32.5 m^3/day/m^2. Find the depth of the clarifier for this overflow rate and detention time of 95 min. Assume length to width ratio for settling tank between 2 : 1 and 5 : 1, and length not to exceed 100 m. Recommend the dimensions of the tank.* (15)

[**Solution.** $Q = 0.5 \times 24 \times 3600$ m^3/d = 43,200 m^3/d

$$\text{Overflow rate} = \frac{Q}{BL} = 32.5 \text{ m}^3/\text{d/m}^2 \text{ (given)}$$

$$\therefore \quad BL = \frac{43{,}200}{32.5} \text{ m}^2 = 1329 \text{ m}^2$$

$\therefore$ Surface area of the tank = **1329 m^2.** **Ans.**

Capacity of the tank (V)

$$= \frac{Q}{24} \cdot t\ ;$$

where t = detention time = 95 min = $\frac{95}{60}$ h

$$= \frac{43{,}200}{24} \text{ m}^3/\text{h} \times \frac{95}{60} \text{ h} = 2850 \text{ m}^3$$

But capacity $= BLH = 2850$ m^3

$$\therefore \quad H = \frac{2850}{1329} = \mathbf{2.15\ m}$$

$\therefore$ *Depth of clarifier (water depth)* = **2.15 m.** **Ans.**

To find dimensions of the tank, we first assume $L : B$ ratio as 2 : 1.

$$\therefore \quad 2B^2H = \text{Volume} = 2850 \text{ m}^3$$

$$\therefore \quad B = \sqrt{\frac{2850}{2 \times 2.15}} = 25.7 \text{ m}$$

$$\therefore \quad L = 2 \times 25.75 = 51.5 \text{ m} < 100 \text{ m (O.K.)}.$$

Let us check the horizontal flow velocity through the tank, which should be between 0.15 to 0.6 m/min (more generally as 0.3 m/min) ; as :

Flow velocity × Detention time = Length of the tank

or $\quad v_h \times 95 \text{ min} = 51.5 \text{ m}$

or $\quad v_h{}^* = \dfrac{51.5}{95} \text{ m/min}$

$$= 0.54 \text{ m/min} < 0.6 \text{ m/min (O.K.)}$$

If the higher ratio for $L : B$ *i.e.* 3 : 1 is taken, then v_h exceeds 0.6 m/min ; and hence the above ratio of 2 : 1 may be adopted. Hence use a tank of size **51.5 m × 25.75 m × (2.15 + 0.5) m size.**

Ans.]

Q. 3. (*b*) *An aquifer of 20 m average thickness is overlain by an impervious layer of 30 m thickness. A test well of 0.5 m diameter and two observation wells at a distance of 10 m and 60 m from the test well are drilled through the aquifer. After pumping at a rate of 0.1 m³/sec for a long time, the following drawdowns are stabilised in these wells : first observation well, 4 m ; second observation well, 3 m. Show the arrangement in a diagram. Determine the coefficient of permeability and the drawdown in the test well. State the validity of Darcy's law.* (15)

[**Solution.** This is the case of Thiem's formula for a confined aquifer, and the conditions are shown in Fig. 4.40.

The given values are now symbolised as

$H = 20$ m

$r_w = 0.25$ m ; $\quad s_w = ?$

$r_1 = 10$ m ; $\quad s_1 = 4$ m

$r_2 = 6$ m ; $\quad s_2 = 3$ m

$Q = 0.1$ m³/s, $\quad K = ?$

*v_h can also be calculated using

$$v_h = \frac{Q}{B.H.} = \frac{43200 \text{ m}^3/\text{d}}{25.7 \text{ m} \times 2.15 \text{ m}} = 780 \text{ m/d}$$

$$= \frac{780}{24 \times 60} \text{ m/min} = \mathbf{0.54 \text{ m/min}}.$$

Using eqn. (4.34), which is applicable to such cases, we have

$$Q = \frac{2\pi KH(s_1 - s_2)}{2.3 \log_{10} \frac{r_2}{r_1}}$$

$$\therefore \quad 0.1 = \frac{2 \times 3.14 \times K \times 20(4 - 3)}{2.3 \log_{10} \frac{60}{10}}$$

$$\therefore \quad K = \mathbf{1.425 \times 10^{-3} \text{ m/sec.}} \quad \textbf{Ans.}$$

To compute the drawdown in the main pumped well, we may use the above eqn. by using r_w in place of r_1 and s_w in place of s_1, as :

$$Q = \frac{2\pi KH(s_w - s_2)}{2.3 \log_{10} \frac{r_2}{r_w}}$$

$$\therefore \quad 0.1 = \frac{2 \times 3.14 \times (1.425 \times 10^{-3}) \times 20(s_w - 3)}{2.3 \log_{10} \frac{60}{0.25}}$$

or $\quad s_w = \mathbf{6.059 \text{ m.}}$ **Ans.**

For validity of Darcy's law, please read article 4.4, wherein in footnote on page 101, it is clearly stated *that the Darcy's law has been demonstrated to be valid only for laminar flow conditions, which as for as soils are concerned, is not at all a problem, because the flow in sands, silts and clays is invariably laminar.* **Ans.**]

Q. 7. (*a*) *In a pumping station, 18000 cum water is to be raised per day from an intake well to a sedimentation tank under the static head of 21 m. Lengths of suction pipe and rising main are 40 m and 150 m respectively. Diameter of pipes is 50 cm. There are two shifts of working of pumps each of 8 hours. Take coefficient of friction as 0.01, and combined efficiency of motor and pump as 80%. Recommend the number of units of pumps each having BHP of 30.* (15)

[**Solution.** Q = Discharge through the pumps = ?

$$Q \text{ per shift of 8 hr} = \frac{18000 \text{ m}^3}{2} = 9000 \text{ m}^3/8 \text{ hr}$$

$$\therefore \quad Q = \frac{9000}{8 \times 60 \times 60} \text{ m}^3/\text{s} = 0.3125 \text{ m}^3/\text{s}$$

$f' = 0.01$

$L = 40 + 150 = 190$ m

$d = 0.5$ m

H_L = Head loss in pipes

$$= \frac{f' . L . \left(\frac{Q}{\pi/4}\right)^2}{2g . d} = \frac{8}{\pi^2 . g} . \frac{f' L Q^2}{d^5}$$

$$= \frac{f' . LQ^2}{12.1 d^5} = \frac{1000 \times 190 \times (0.3125)^2}{12.1 \times (0.5)^5}$$

$$= \mathbf{0.49\ m}$$

Total head against which pumping is done

= Static head + Head loss

= 21 + 0.49 = 21.49 m

Required Brake Horse Power (BHP) of pumping

$$= \frac{w . QH}{7.5\eta} = \frac{1000 \times (0.3125) \times 21.49}{75 \times 0.8}$$

$$= \mathbf{111.92\ BHP}$$

Hence, use 4 No. pumps each of 30 BHP. **Ans.**]

Year 1996 (*Engineering Services*)

Q. 2. (c) *In a continuous flow settling tank, 3.5 m deep and 65 m long, if the flow velocity of water observed is 1.22 cm/s, what size of the particles of specific gravity 2.65 may be effectively removed ? Assume temperature 25°C and kinematic viscosity of water as 0.01 cm²/s.* (20)

[**Hint.** Use eqn. (9.12) as

$$\frac{v}{v_s} = \frac{L}{H} = \frac{65}{3^*}, \quad \text{where } v = 1.22 \text{ cm/s}$$

$$\therefore \quad \frac{1.22}{v_s} = \frac{65}{3}$$

$$\text{or} \quad v_s = \frac{3}{65} \times 1.22 \text{ cm/s} = 0.0563 \text{ cm/s}$$

Now, use eqn. (9.1) as

$$v_s = \frac{g}{18} (G - 1) \frac{d^2}{\nu} \quad (\text{for } d < 0.1 \text{ mm})$$

$$\therefore \quad 0.0563 = \frac{981}{18} (1.65) \frac{d^2}{0.01}$$

$$\text{or} \quad d = 2.5 \times 10^{-3} \text{ cm}$$

$$= \frac{2.5}{1000} \times 10 \text{ mm}$$

$$= \mathbf{0.025\ mm} \text{ (which is < 0.1 mm)}$$

Hence, the particles of size 0.025 mm and above shall be effectively removed. **Ans.**]

*assuming 0.5 m free board ; water depth in tank = 3 m

Q. 3. (a) *During a recuperation test, the water in an open well was depressed by pumping by 2.1 m and it recuperated 1.6 m in 90 minutes. Find the diameter of a well to yield 10 litres/sec under a depression head of 2 m.* (20)

[**Hint.** Follow exactly similar solved example 4.6, and determine the requisite well dia as **4.9 m.** **Ans.**]

Q. 3. (c) *For a town with a population of 2 lakhs, a water supply scheme is to be designed. The maximum daily demand may be assumed as 200 litres/capita/day. The storage reservoir is situated 5 km away from the town. Assuming loss of head from source to the town as 10 m and coefficient of friction for the pipe material as 0.012, recommend the size of the supply main. 50% of daily demand has to be pumped in 8 hours for the proposed scheme.* (20)

[**Hint.** *Follow exactly similar solved example 6.3 and compute :*

Max. water demand per day

= 2,00,000 × 200 L/Day = 40 MLD

Max. water demand for which supply main is to be designed

= 50% × 40 ML per 8 hr

$$= \frac{20 \times 10^6}{10^3} \text{ m}^3\text{/8 hr} = \frac{20 \times 10^3}{8 \times 60 \times 60} \text{ m}^3\text{/s} = 0.694 \text{ m}^3\text{/s}$$

Now, for supply main

$Q = 0.694$ m³/s, $f' = 0.012$

$L = 5$ km $= 5000$ m, $H_L = 10$ m

Using eqn. (6.1), we have

$$H_L = \frac{f' LV^2}{2gd} \text{ where } V = \frac{Q}{A} = \frac{0.694}{(\pi/4)\, d^2}$$

$$\therefore \quad 10 = \frac{0.012 \times 5000 \times \left[\frac{0.694}{(\pi/4)\, d^2}\right]^2}{2 \times 9.81 \times d}$$

Solving d = dia of supply main = **0.75 m.** **Ans.**]

Q. 6. (b) *Determine the distribution of flow in the pipe network shown. The head loss, h_L, may be assumed as kQ^n. The flow is turbulent and pipes are rough. The value of k for each pipe is indicated in the figure. Use Hardy-Cross method.* (20)

[**Hint.** Please refer Solved Example 10.11.]

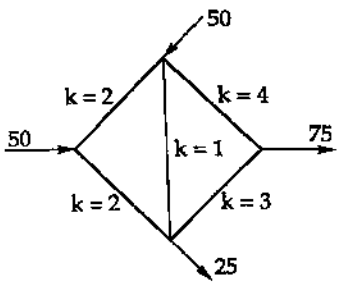


Fig. 17.2

Year 1998 (*Engineering Services*)

Q. 2. (*c*) *Suggest a suitable disinfective device for water supplies in rural areas with open dug wells as the source of water.* (10)

Q. 4. (*a*) *A rectangular settling tank without mechanical equipment is to treat 1 million litres of raw water per day. If the design criteria are that : the detention period is 2.5 hours, the velocity of flow is 8 cm/minute, and depth of water and sediment is 4.5 m, then what would be :*

(i) the length of the tank

(ii) the width of the tank if an allowance of 1.5 m is to be made for sediment

(iii) the overflow rate of the tank ? (15)

[**Hint.** Follow exactly similar solved example, and compute the answers as :

(*i*) $L = 12$ m ; (*ii*) $B = 2.9$ m ;

(*iii*) $\frac{Q}{BL} = \frac{10^6 \text{ L/day}}{(2.9 \times 12)\text{ m}^2} = 1197.3$ L/hr/m^2. **Ans.**]

Q. 5. (*a*) *A fully penetrating well of radius 0.2 m pumps at a constant rate of 400 m^3/hour from a confined sandy aquifer of thickness 40 m and porosity 0.25. Considering only macroscopic flow velocity, how long will it take for a pollutant to reach the well if introduced into the aquifer at 100 m from it ?* (10)

[**Solution.** Use Eq. (4.20), *i.e.* $Q = n \,.\, v_a \,.\, A_s$

where Q = discharge of well = 400 m^3/hr

n = porosity = 0.25

v_a = actual flow velocity of ground water

A_s = surface area of well opening

But $A_s = (\pi \,.\, d)\, H$

d = dia of well = 0.4 m (given)

H = Thickness of confined aquifer = 40 m

$\therefore \quad A_s = \pi \times 0.4 \text{ m} \times 40 \text{ m} = 16\pi \text{ m}^2$

$\therefore \quad 400 \text{ m}^3/\text{day} = 0.25 \times v_a \times 16\pi \text{ m}^2 = 4\pi\, v_a \text{ m}^2$

or $\quad v_a = \frac{400}{4\pi} \text{ m/d} = \frac{100}{\pi} \text{ m/d}.$

Time taken by the pollutant to travel with ground water from a distance of 100 m to the well

$$= \frac{\text{Distance}}{\text{Velocity of travel}} = \frac{100 \text{ m}}{\frac{100}{\pi} \text{ m/d}}$$

$= \pi$ day = **3.14 day.** **Ans.**]

Year 1999 (*Engineering Services*)

Q. 3. (*a*) *A rapid sand filter is proposed for water supply treatment plant for a town with a population of 75,000. The average rate of water supply is 150 litres/capita/day and rate of filtration is to be 100 litres/min/sqm. Design :*

(i) size and number of filter beds.

(ii) manifold-lateral under-drainage system.

(iii) wash water discharge required, if the rate of washing is 45 cm rise/min. (18)

[**Hint.** Follow Solved Examples 9.20 & 9.21.]

Q. 4. (*b*) *A town of 2,00,000 population is to be supplied water from a source 2500 m away. The lowest water level in the source is 15 m below the waterworks of the town. The demand of water is estimated as 150 litres/capita/day. A pump of 300 HP is operated for 15 hours. If the maximum demand is 1.5 times the average demand, the velocity of flow through the rising main is 1.3 m/sec and the pump efficiency is 70%, determine :*

(i) the hydraulic gradient, and

(ii) friction factor for the pipe. (15)

[**Hint.** Follow Solved Example 6.4, and compute as follows :

Max. demand = 1.5 [150 × 2,00,000] l/day

$$= 45 \text{ Mld} = \frac{45 \times 10^6}{10^3 \times 24 \times 3600} \text{ m}^3/\text{s} = 0.5208 \text{ m}^3/\text{s}.$$

Since the pumping is done for 15 hrs a day, max. draft for pumping

$$= 0.5208 \times \frac{24}{15} = 0.833 \text{ m}^3/\text{s}.$$

H.P. of pump = 300.

But $\quad \text{H.P.} = \frac{w \,.\, QH}{0.735\, \eta}$;

where w = unit wt. of water = 9.81 kN/m^3

$Q = 0.833$ m^3/s

$\eta = 70\%$ (given)

$\therefore \quad 300 = \frac{9.81 \times 0.833 \times H}{0.735 \times 0.70}$

or $\quad H = \frac{300 \times 0.735 \times 0.70}{9.81 \times 0.833} = 18.8$ m.

The total head lift = 18.8 m

= head lost in rising main of 2500 m length having flow velocity of 1.3 m/s (H_L) + head difference between water source and waterworks

or $\quad 18.8 \text{ m} = H_L + 15 \text{ m}$

$\therefore$ $H_L = 3.5$ m

Length of the pipe = 2500 m

$\therefore$ Hydraulic gradient = $\frac{H_L}{L} = \frac{3.5}{2500} = \frac{1}{714}$. **Ans.**

Friction factor = ?

$$H_L = \frac{f \cdot L \cdot V^2}{2gd};$$

where d is pipe dia to be computed as below:

$$A = \frac{\pi}{4} d^2 = \frac{Q}{V} = \frac{0.833}{1.3} = 0.6407$$

$$d = \sqrt{0.6407 \times \frac{4}{\pi}} = 0.9 \text{ m}$$

or $d = 0.9$ m

$\therefore$ $3.5 = \frac{f \times 2500 \times (1.3)^2}{2 \times 9.81 \times 0.9}$

or $f = 0.0146$. **Ans.**]

Year 2000 (*Engineering Services*)

Q. 2. (*a*) *Water flows from a reservoir through a pipe of 0.15 m diameter and 180 m long to a point 13.5 m below the open surface of the reservoir. Here it branches into two pipes, each of 0.1 m diameter, one of which is 48 m long discharging to atmosphere at a point 18 m below reservoir level, and the other 60 m long discharging to atmosphere 24 m below reservoir level. Assuming constant friction coefficient of 0.032, calculate the discharge from each pipe. Neglect any losses at the junction.* (20)

[**Solution.** Such questions have been solved in chapter 6, as solved examples 6.5 to 6.8. Study them and solve as follows :

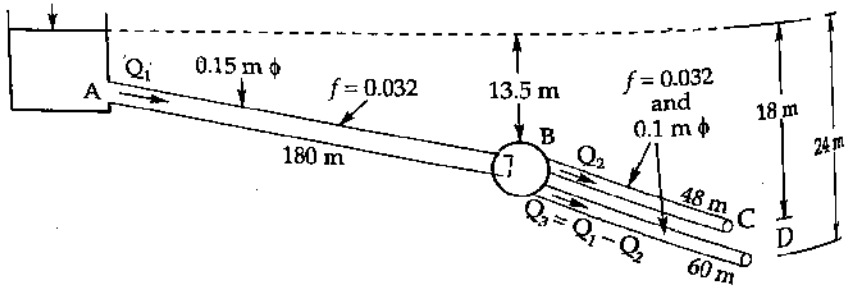


Fig. 17.3

H_L between ABC = 18 m

Using Eq. $H_L = \frac{8f'}{g\pi^2} \frac{L \cdot Q^2}{d^5}$, we get

$$\frac{8 \times 0.032}{9.81\,(3.14)^2} \left[\frac{180 \cdot Q_1^2}{(0.15)^5} + \frac{48\, Q_2^2}{(0.1)^5} \right] = 18$$

or $[2370370\, Q_1^2 + 4800000\, Q_2^2] = 6800.81$

or $Q_2^2 + 0.4938\, Q_1^2 = 0.0014168$

or $Q_2 = \sqrt{0.0014168 - 0.4938\, Q_1^2}$

Now, H_L between ABD = 24 m.

$\therefore$ $\frac{8 \times 0.032}{9.81 \times (3.14)^2} \left[\frac{180\, Q_1^2}{(0.15)^5} + \frac{60\,(Q_1 - Q_2)^2}{(0.1)^5} \right] = 24$

or $\frac{180\, Q_1^2}{(0.15)^5} + \frac{60\,(Q_1 - Q_2)^2}{(0.1)^5} = 9067.75$

or $2370370\, Q_1^2 + 6000000$

$$\times \left[Q_1 - \sqrt{0.0014168 - 0.43938\, Q_1^2} \right]^2 = 9067.75$$

or $Q_1^2 + 2.5312 \left[Q_1 - \sqrt{0.0014168 - 0.4938\, Q_1^2} \right]^2 = 0.003825$

Solving by hit and trial, we get

$Q_1 = 0.0458$ cumecs

Then, compute $Q_2 = 0.0195$ cumecs

$Q_3 = Q_1 - Q_2 = 0.0263$ cumecs.] **Ans.**

Q. 4. (*a*) *Draw the flow sheet showing sequence of various treatment units of a typical water treatment plant with perennial river as the source of water. List these treatment units sequentially.* (10)

[**Hint.** Please refer Fig. 13.1.]

Q. 8. (*c*) *Design the approximate dimensions of a set of two rapid gravity filters for treating water required for a population of 50,000 ; the rate of supply being 180 litres/day/person. The filters are rated to work 5000 litres/hour/sq. m. Assume maximum daily demand to be 1.8 times the average daily demand. Assume any other data, if necessary.* (10)

[Please refer Solved Example 9.20. **This numerical question was set from this book.**]

Year 2001 (*Engineering Services*)

Q. 1. (*a*) *Following mean monthly flows were observed at a site on a stream in a typical year :*

Months	*J*	*F*	*M*	*A*	*M*	*J*
Mean monthly flow (cumec)	15	10	8	6	5	12
Months	*J*	*A*	*S*	*O*	*N*	*D*
Mean monthly flow (cumec)	25	40	71	60	40	20

Assuming that the stream flow is to be fully utilised for delivering water to a city to meet its fixed monthly demand by diversion of flow, find the capacity of the conduit in cumec for which it is to be designed. Determine the minimum storage capacity in ha. m. of the storage reservoir to ensure that all the available flow is fully utilised to meet the constant monthly demand of the city. (20)

Solution. Calculations are carried out in Table 17.2 as usual, and minimum storage capacity of reservoir is worked out to be **281.578 Mm³** ; *i.e.* **28157.8 ha.m. Ans.**

Table 17.2. Calculation Table for Q.1 (*a*) of Year 2001

S. No.	*Month*	*Monthly inflow in cumec*	*Monthly inflow in M.m³ = (3) × 3600 × 24 × days in the months*	*Monthly outflow from reservoir Mm³ = Σ col (4) / 12*	*Monthly deficit in the reservoir to be supplied from storage in Mm³*	*Monthly surplus in reservoir in Mm³*	*Cumulative deficit in Mm³*	*Cumulative surplus in Mm³*
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
1	Jan.	15	40.176	68.4936	28.3176			
2	Feb.	10	24.192	68.4936	44.3016			
3	March	8	21.4272	68.4936	47.0664			
4	April	6	15.552	68.4936	52.9416			
5	May	5	13.392	68.4936	50.1016			
6	June	12	31.104	68.4936	37.3896			
7	July	25	66.690	68.4936	1.5336		261.652	
8	August	40	107.136	68.4936	—	38.6424		
9	Sept.	71	184.032	68.4936	—	115.5384		
10	Oct.	60	160.704	68.4936	—	92.2104		
11	Nov.	40	103.680	68.4936	—	35.1868		281.578*
12	Dec.	20	53.568	68.4936	14.9256		14.9256	
		Σ =	821.9232	821.9232				

*Highest value in col. (8) and (9).

Discharge in cumec for the conduit

$$= \frac{\text{Total outflow in the year (m}^3\text{)}}{\text{No. of secs in the year}}$$

$$= \frac{821.9232^* \times 10^6}{86400 \times 365} = \mathbf{26.063\ m^3/s.}\ \textbf{Ans.}$$

Q. 2. (*a*) *Explain how you will prepare the sand to be used in the rapid sand filters from the available stock sand in case of a large water supply scheme.* (15)

Year 2002 (*Engineering Services*)

Q. 1. (*a*) (*ii*) *Water is pumped through a 600 mm diameter pipe with a head loss of 20 m. It is intended to reduce the power consumption by laying another parallel pipe of appropriate diameter laid by the side of the existing pipeline with the same overall length and the same friction factor-by reducing the head loss to 10 m, but still delivering the same discharge jointly through both pipes. What should be the diameter of this additional pipe ?*

Solution. In first case,

$$H_L = \frac{f' . L . V^2}{2gd} \; ; \; V = \frac{Q}{A} = \frac{Q}{\frac{\pi}{4} . d_1^{\,2}} = \frac{4Q}{\pi d_1^{\,2}}$$

$$\therefore \quad H_L = \frac{f' . L . 16Q^2}{2gd . \pi^2 . d_1^{\,4}} = \frac{8}{\pi^2} \frac{f' L Q^2}{g d_1^{\,5}}$$

$$\therefore \quad 20 = \frac{8}{g . \pi^2} \left[\frac{f' . L . Q^2}{d_1^{\,5}} \right]$$

or $$Q = \sqrt{\frac{24181 d_1^{\,5}}{f' L}} = 15.55 \sqrt{\frac{d_1^{\,5}}{f' L}}$$

In second case, when pipe of dia. d_2 is connected in parallel to first pipe of dia. d_1, we get

$$10 = \frac{8}{g\pi^2} \left[\frac{f' . L . Q_1^{\,2}}{d_1^{\,5}} \right] ; \quad \text{or} \quad Q_1 = 11 \sqrt{\frac{d_1^{\,5}}{f' L}}$$

Also $$10 = \frac{8}{g\pi^2} \left[\frac{f' . L . Q_2^{\,2}}{d_2^{\,5}} \right] ; \quad \text{or} \quad Q_2 = 11 \sqrt{\frac{d_2^{\,5}}{f' L}}$$

But $Q_1 + Q_2 = Q$

$$\therefore \quad 11 \sqrt{\frac{d_1^{\,5}}{f' L}} + 11 \sqrt{\frac{d_2^{\,5}}{f' L}} = 15.55 \sqrt{\frac{d_1^{\,5}}{f' L}}$$

But $d_1 = 600$ mm $= 0.6$ m ; $\sqrt{d_1^{\,5}} = 0.279$

*See Σ of col. (5) in Table 17.2.

$$\therefore \quad 11 \times 0.279 \cdot \sqrt{\frac{1}{f'L}} + 11 \times \sqrt{d_2^5} \cdot \sqrt{\frac{1}{f'L}}$$

$$= 15.55 \times 0.279 \sqrt{\frac{1}{f'L}}$$

$$\text{or} \quad 11 \times 0.279 + 11 \times \sqrt{d_2^5} = 15.55 \times 0.279$$

or $d = 0.422$ m. Hence use 422 mm dia. pipe. **Ans.**]

Q. 2 (*a*) (*i*) *Defining coagulation, explain the different mechanisms of coagulation and its usefulness in removal of particles.*

(*ii*) *A flocculation chamber 30 m long, 12 m wide and 4.5 m deep is to treat 75 MLD of water. It is equipped with 12 m long, 0.3 m wide paddles supported parallel to and moved by four horizontal shafts which rotate at a speed of 2.5 r.p.m. The centreline of paddles is 1.8 m from the shaft which is at mid-depth of the tank. Two paddles are mounted on each shaft, one opposite to another. If the mean velocity of water is 1/4th velocity of paddles, find: (1) power consumption, (2) time of flocculation; and (3) value of G. Kinematic viscosity of water = 1.31 × 10^{-6} m^2/s.*

[**Solution.** Dimensions of flocculation chamber

= 30 m × 12 m × 4.5 m (Fig. 17.4)

Volume (V) of flocculation chamber

$= 30 \times 12 \times 4.5 = 1620$ m^3

$$Q = 75\ MLD = \frac{75 \times 10^6}{10^3} \text{ m}^3\text{/d} = 75000 \text{ m}^3\text{/d}.$$

Dimensions of paddle = (12 m × 0.3 m) [2 paddles per shaft × 4 shafts]

Paddle speed $N = 2.5$ r.p.m.

Power consumption, $P = F_D \cdot v_p$ ...(17.2)

where F_D = Drag force ; and v_p = vel. of paddle

But, $v_p = \omega \cdot r$ ...(17.3)

where ω = angular velocity = $\frac{2\pi N}{60^\circ}$ (N = rpm)

$$\therefore \quad v_p = \frac{2\pi N}{60^\circ} \cdot r = \frac{\pi N}{60^\circ} \cdot D_i \qquad \text{...(17.4)}$$

where D_i is the dia. of impeller = 3.6 m (Fig. 17.4)

$$\therefore \quad v_p = \frac{\pi \times 2.5}{60} \times 3.6 = 0.471 \text{ m/s}$$

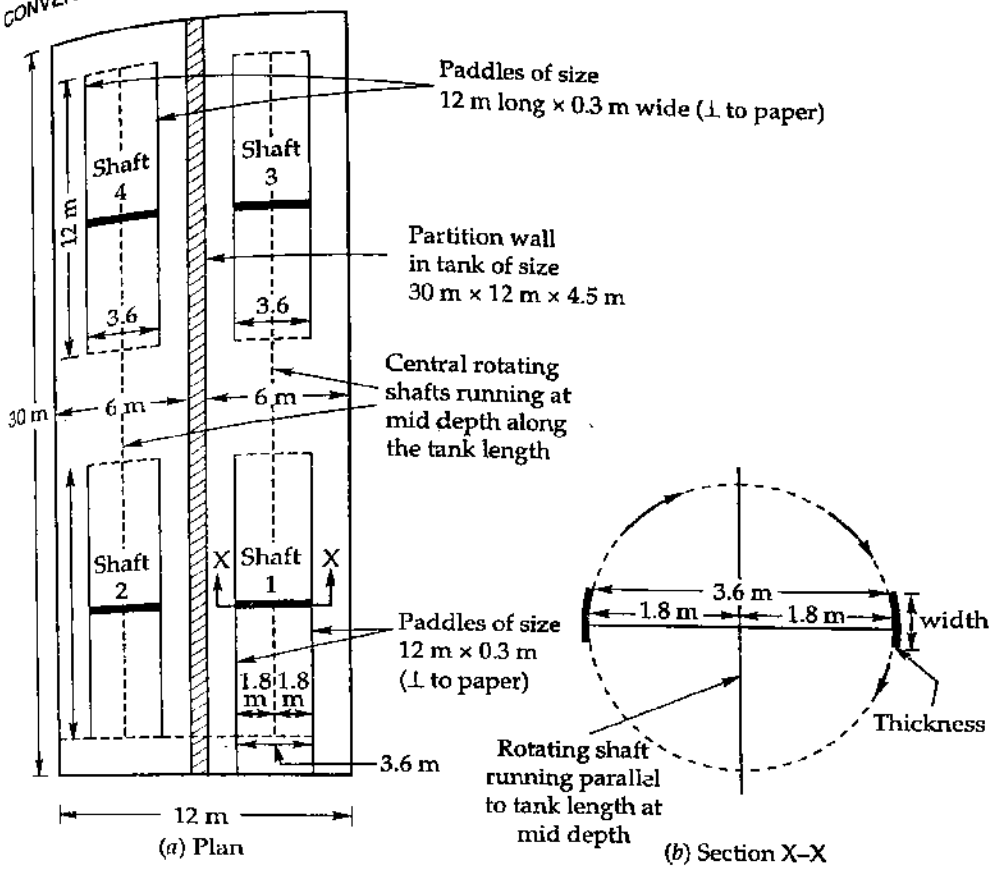


Fig. 17.4

But drag force is given as :

$$F_D = C_D \cdot \rho_w \ A_P \cdot \frac{v_p^2}{2} \ \frac{\text{kgm}}{\text{sec}^2} \textit{ i.e. } \text{Newton (N)} \qquad \text{...(17.5)}$$

P = Power consumption = $F_D \cdot v_p$

$$\text{or} \quad P = C_D \cdot \rho_w \cdot A_p \cdot \frac{v_p^3}{2} \text{ (N.m/s)} \qquad \text{...(17.6)}$$

where C_D = Coefficient of drag = 1.8 for flat plate

A_P = Area of (2 × 4) Nos of paddles

$= (2 \times 4) \times 0.3 \text{ m} \times 12 \text{ m} = 28.8 \text{ m}^2$

ρ_w = density of water = 1000 kg/m^3

$$\therefore \quad P = \left[1.8 \times 1000 \times 28.8 \times \frac{(0.47)^3}{2}\right] \frac{\text{Nm}}{\text{s}} \textit{ i.e. } \text{Watts}$$

$= 2.71 \times 10^3$ watts = **2.71 kW. Ans.**

(*ii*) Time of flocculation, $t = \dfrac{\text{Vol}\,(V)}{Q} = \dfrac{1620\ \text{m}^3}{75 \times 10^3\ \text{m}^3/\text{d}}$

$= 0.0216$ day

$= (0.0216 \times 24 \times 60)$ min.

$=$ **31.1 minutes. Ans.**

(*iii*) ***Temporal Mean Velocity Gradient (G')*** is given by eqn. (9.25) as

$$G' = \left[\frac{P}{\mu . V}\right]^{1/2}$$

where P = Power dissipated in watts $= 2.71 \times 10^3\ \text{W}\left(i.e. \dfrac{\text{Nm}}{\text{s}}\right)$

μ = Absolute or dynamic viscosity of water

$= \rho$. ν where ν is kinematic viscosity

$$= 1000 \times 1.31 \times 10^{-6}\left(\frac{\text{kg}}{\text{m}^3} . \frac{\text{m}^2}{\text{s}} = \text{kg/ms}\right)$$

V = Volume of raw water to which P is applied in $\text{m}^3 = 1620\ \text{m}^3$.

$$\therefore \quad G' = \sqrt{\frac{2.71 \times 10^3}{[1000 \times 1.31 \times 10^{-6}] \times 1620}} = \textbf{35.73 per sec.}$$

Ans.]

Q. 4. (*b*) (*i*) *Design and draw a bell mouth canal intake for 1 MLD of water supposing canal runs 10 hours a day.*

(*ii*) *What is the relationship between efficiency and depth of an ideal sedimentation basin and on what assumptions ?*

Year 2003 (*Engineering Services*)

Q. 5. (*a*) *Why is sedimentation considered a necessity for treating raw water ? What are the factors that effect sedimentation and how is sedimentation done ? What is the purpose of coagulation in treatment of raw water ? Give a neat sketch of 'Baffle type of mixing basin' for coagulation.* (15)

Year 2004 (*Engineering Services*)

Q. 4. (*b*) *The turbine as shown in the given figure (Fig. 17.5), extracts 45 mhp (metric horse power) from the water flowing through it. The 30 cm diameter 100 m long pipe has a friction factor of 0.02. Neglecting minor losses, determine the flow rate through the pipe and the turbine.* (20)

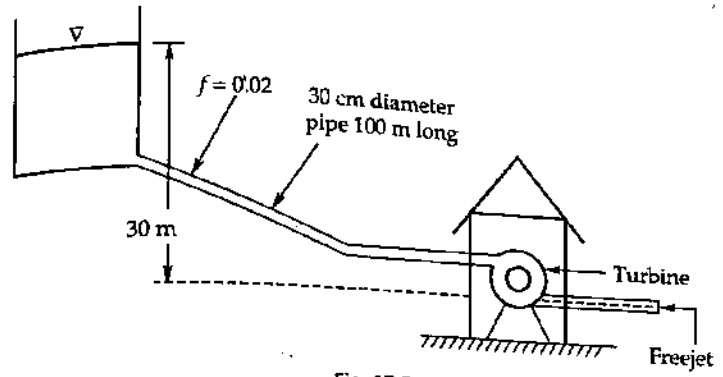


Fig. 17.5

Solution. Head loss in pipe line

$$= H_L = \frac{f . L . Q^2}{2gd . \left(\frac{\pi}{4} . d^2\right)^2}$$

or $$H_L = \frac{0.02 \times 100 \times Q^2}{2 \times 9.81 \times \left(\frac{\pi}{4}\right)^2 . (0.3)^5}$$

or $$H_L = 68Q^2 \qquad \ldots(i)$$

Now, Effective Head on Turbine

$$H = 30\ \text{m} - H_L = (30 - 68Q^2)$$

$$\underset{\text{(metric)}}{\text{H.P.}} = \frac{wQH}{75} \qquad \text{(in MKS units)}$$

where w = the unit wt. of water in MKS units $= 1000\ \text{kg/m}^3$

Q = discharge in m^3/s

H = Head in m.

$$\therefore \quad 45 = \frac{1000 \times Q \times [30 - 68Q^2]}{75}$$

or $$\left[30 - 68Q^2\right] = \frac{45 \times 75}{1000} = 3.375$$

or $$68Q^3 - 30Q + 3.375 = 0$$

Solving by hit and trial

$$Q = \textbf{0.60 m}^3\textbf{/s. Ans.}$$]

Q. 5. (*a*) *A rapid sand filter is to be provided in a water treatment plant, to process the water for a town with a population of 2,75,000. The water demand is 200 litres/capita/day. The rate of filtration is 15 m^3/m^2/hour. Allow 5% of filtered water for storage to meet the backwash requirements. Each backwashing period is of 30 min. Determine the number of filters required*

allowing one as a standby unit. The available surface area configuration of filter unit is 10 m × 4 m. Also compute the up-flow velocity and headloss to expand the bed to 0.66 m. The porosity of the bed is 0.50. Specific gravity is 2.5. The average particle size is 0.6 mm. The drag coefficient is 5.02 and the kinematic viscosity is 0.10136 × 10^{-5} m^2/sec. The flow is assumed to be transitional flow. (15)

[**Hint.** Please see Solved Example 9.26.]

Year 2005 (*Engineering Services*)

Q. 1. (*b*) *Explain the phenomenon of "negative head" in filter. Why it should be avoided ? How it is avoided in filter operation ?* (10)

Q. 4. (*a*) *What are the differences between head balance method and quantity balance method in analysing the water distribution pipe network ? Explain in detail.* (10)

[**Hint:** In **head balance method,** the pipe circuit can be reduced into a *single equivalent pipe* by using the following two principles of hydraulics :

(*a*) The loss of head caused by a given flow of water through the pipes connected in series is additive; and

(*b*) The quantity of discharge flowing through the different pipes connected in parallel will be such as to cause the same head loss through each pipe.

This method is popularly called the ***equivalent pipe method*** of analysing water distribution networks.

In **quantity balanced method,** the flow in each pipe is assumed by the designer in magnitude as well as in direction in such a way that the principle of continuity is satisified at each junction (*i.e.* the inflow at any junction becomes equal to the outflow at that junction). A correction to these assumed flows is then computed successively for each pipe loop in the network, untill the correction is reduced to an acceptable value. This method is popularly called the ***Hardy-Cross Method*** of analysing Water distribution pipe networks.]

Q. 5. (*c*) *Water is being pumped from a lower to an upper reservoir through a pipeline as shown in Fig. 17.6. The elevation difference between the reservoirs is 15.2 m. A steel pipe of 21.3 m connects these two reservoirs. It has an equivalent roughness height of 0.046 mm. The diameter of the pipe is 50 mm. The performance curve of the pump is given by* $h_p = 24.2 - 7.65\ Q_p^2$ *in which* h_p *is the pump head and* Q_p *in lps. Using this pump, what flow do you expect in the pipeline ? If the motor rotates at 2400 rpm, what is the specific speed ? State the type of pump that should be used. Minor losses can be neglected.*

$g = 9.81\ ms^{-2}$. *Viscosity* $1 \times 10^{-6}\ m^2\ s^{-1}$.

[**Hint.** Jain equation

$$\frac{1}{\sqrt{f}} = -2 \log\left[\frac{\varepsilon/D}{3.7} + \frac{5.74}{\mathrm{Re}^{0.9}}\right]$$ can be used

(0.9 is the exponent)

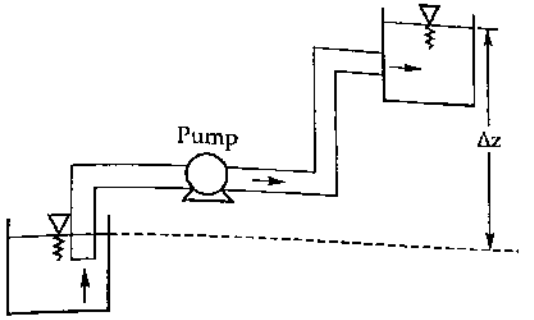


Fig. 17.6

(25)

Year 2006 (*Engineering Services*)

Q. 5. (*b*) *Derive equations for settling of a discrete particle in liquid.* (15)

[**Hint.** Pl. see artitle 9.4 and derive Eq. (9.1).]

Year 2007 (*Engineering Services*)

Q. 2. (*d*) *Differentiate between a rapid sand filter and a slow sand filter in respect of the following :*

(*i*) *Sand specifications through UC and E.S.*

(*ii*) *Method of collection of filtered water*

(*iii*) *Media arrangement*

(*iv*) *Penetration and removal of impurities.* (8)

[**Hint:** (*i*) The ***Uniformity Coefficient*** Values (UC) for rapid sand*filters are : 1.2-1.8 ; and for slow sand filters are : 1.8–2.5; while the ***Effective Size*** (ES) values for these filters are 0.35–0.55 and 0.2–0.4 mm, respectively, as indicated in SNo. 3 table 9.5]

Q. 3. (*d*) *Compute the 'fire demand' for a city of 2 lac population by any two formulae including that of the National Board of Fire Underwriters.* (8)

Q. 6. (*a*) *Water hammer occurs in an elastic pipe due to rapid closure of the valve. Derive an expression for the rise of pressure in the pipeline.* (8)

[**Hint.** Pl. see article 6.5.1.1 and derive equation 6.19.]

Q. 8. (*d*) *Discuss the functioning and applicability of the following :*

(*i*) *Air Valve* (*ii*) *Reflux Valve*

(*iii*) *Flanged Joint* (*iv*) *Expansion Joint* (8)

Year 2008 (*Engineering Services*)

Q. 1. (*c*) *Design a suction well (intake well) for a flow of 400 L/min, if the highest water level and the lowest water level are 9.0 m and 4.5 m above the river bed, respectively. Ground level is 9.7 m above the river bed. Assume all other data and mention it.* (10)

[**Hint.** Follow Solved Example 5.1. Assuming that the pumps work for 16 hr a day, the discharge to be pumped

$$= \frac{400}{60} \times \left(\frac{24}{16}\right) \text{ L/s} = 10 \text{ L/s} = 0.01 \text{ m}^3/\text{s}. \ ]$$

Q. 3. (*b*) *Derive Darcy-Weisbach equation for the head loss due to friction in circular pipe. Assume flow is steady and incompressible.* (10)

Q. 6 (*c*) *What is Coagulation ? Discuss the mechanisms of coagulation.* (10)

Year 2009 (*Engineering Services*)

Q. 2. (*d*) (*ii*) *Design slow sand filters for a population of 40,000 with an average rate of water supply of 150 litres per capita per day.* (5)

[**Hint.** Pl. follow Solved Example 9.19. The additional data given in that example can be assumed while solving this question.]

Q. 4. (*a*) *A pipe network in the form of a triangle ABC has inflows of 5 m³/s and 4 m³/s at A and B respectively. The outflow at C is 9 m³/s. Given $K_{AB} = 10$, $K_{BC} = 50$ and $K_{AC} = 20$, compute discharges in each pipeline [$h_f = KQ^2$].* (10)

Solution. Use Hardy Cross method and follow Solved Example 10.11, to compute first correction for flows assumed (Q_a) as shown in Fig. 17.7. Q_a and H_L may be assumed as +ve in clockwise direction.

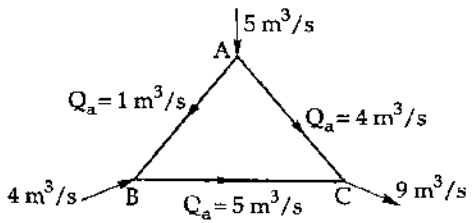


Fig. 17.7

Table 17.3. Hardy Cross procedure for 1st correction

Pipe	*Assumed flow* Q_a *m³/s*	*K (given)*	$H_L = KQ_a^2$ *+ve or −ve*	$\left\lvert \frac{H_L}{Q_a} \right\rvert$	*Corrected Q after first correction* $Q_{a_1} = Q_a + \Delta_1^*$
(1)	(2)	(3)	(4)	(5)	(6)
AB	− 1	10	(−) 10	10	0.38
BC	− 5	50	(−) 1250	250	− 3.62
CA	4	20	320	80	5.38
		Σ	(−) 940	340	

$$*\Delta_1 = \frac{-\Sigma H_L}{x \Sigma \left\lvert \frac{H_L}{Q_a} \right\rvert} = \frac{(-)(-940)}{2 \times 340} = 1.38 \qquad \text{since } x = 2 \left[\because H = K.Q_a^x\right]$$

The discharges in three pipes after first correction are computed in col. (6) of Table 17.3.

The procedure is again repeated in Table 17.4 to compute the second correction and the corrected flows after the 2nd correction.

Table 17.4. Hardy Cross procedure for second correction

Pipe	*Assumed flow* (Q_{a_1}) (m^3/s)	*K (given)*	$H_L = K.Q_{a_1}^2$ *(+ve or −ve)*	$\left\lvert \frac{H_L}{Q_{a_1}} \right\rvert$	*Corrected Q after second correction* $Q_{a_2} = Q_{a_1} + \Delta_2^*$ (m^3/s)
(1)	(2)	(3)	(4)	(5)	(6)
AB	0.38	10	1.44	3.79	0.251
BC	(−) 3.62	50	(−) 655.22	181.00	(−) 3.749
CA	5.38	20	578.89	107.60	5.251
		Σ	(−) 74.89	292.39	

$$*\Delta_2 = \frac{-\Sigma H_L}{x \cdot \Sigma \left\lvert \frac{H_L}{Q_{a_1}} \right\rvert} = \frac{(-)74.89}{2 \times 292.39} = (-)\,0.129$$

The corrected flows after 2nd correction are computed in col. (6) Table 17.4. The procedure is again repeated in Table 17.5 to get the third correction.

Table 17.5. Hardy Cross procedure for third correction

Pipe	*Assumed flow* (Q_{a_2}) m^3/s	*K (given)*	$H_L = K.Q_{a_2}^2$ *+ve or −ve*	$\left\lvert \frac{H_L}{Q_{a_2}} \right\rvert$	*Corrected Q after third correction* $Q_{a_3} = Q_{a_2} + \Delta_3^*$
(1)	(2)	(3)	(4)	(5)	(6)
AB	0.251	10	0.63	2.51	0.506
BC	(−) 3.749	50	(−) 702.75	187.45	(−) 3.494
CA	5.251	20	551.46	105.02	5.506
			(−) 150.65	294.98	

$$*\Delta_3 = \frac{(-)\Sigma H_L}{x \cdot \Sigma \left\lvert \frac{H_L}{Q_{a_2}} \right\rvert} = \frac{(-)(-150.65)}{2 \times 294.98} = \frac{150.65}{2 \times 294.98} = 0.255$$

The corrected flows after 3rd correction are now plotted on the pipe network in Fig. 17.8 to counter check balancing of flows at points *A*, *B* and *C*.

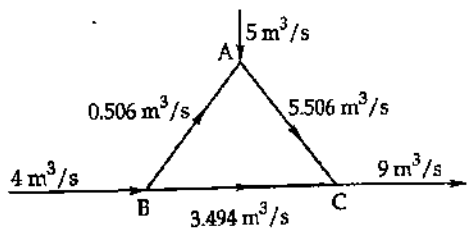


Fig. 17.8

[**Note:** If we carry out further estimation of correction, it is found to be very very small, almost negligible, and hence the above computed flows can be taken as the final discharges. **Ans.**]

Year 2010 (*Engineering Services*)

Q. 1. (*a*) (*iv*) *The diameters of the impeller of a centrifugal pump at inlet and outlet are 20 cm and 40 cm respectively. Determine the minimum starting speed of the pump against a head of 25 m.* (2)

Solution. The required relation is given by the following equation :

$$\left(\frac{\pi N}{60}\right)^2 \left[D_2^2 - D_1^2\right] = 2gH \quad \text{...(17.1)}$$

where N = Speed in RPM
D_1 = Impeller dia. at inlet
D_2 = Impeller dia. at outlet
H = Head

Substituting given values, we get

$$\left(\frac{\pi N}{60}\right)^2 \left[(0.4)^2 - (0.2)^2\right] = 2 \times 9.81 \times 25$$

or $N = 1221$ rpm. **Ans.**

Q. 1. (*b*) (*iii*) *An aquifer of 25 m average thickness is overlain by an impervious layer of 30 m thickness. A test well 0.4 m in diameter and two observation wells at a distance of 12 m and 48 m are located in the aquifer. After pumping at a rate of 0.2 m^3/s for a long time, drawdowns in the wells were observed to be 3.5 m and 1.5 m respectively. Determine the coefficient of permeability in m/day.* (2)

Solution. The Thiem's discharge equation for the given case of a confined aquifer is given as :

$$Q = \frac{2\pi KH(s_1 - s_2)}{2.3 \log_{10} \frac{r_2}{r_1}} \quad \text{...(i.e. Eq. 4.34)}$$

Here $Q = 0.2$ m^3/s, $K = ?$, $H = 25$ m
$s_1 = 3.5$ m $s_2 = 1.5$ m, $r_1 = 12$ m, $r_2 = 48$ m

Substituting values, we get

$$0.2 = \frac{2\pi K \times 25(3.5 - 1.5)}{2.3 \log_{10} \frac{48}{12}}$$

or $K = 8.816$ m/s

or $K = 8.816 \times (3600 \times 24)$ m/d

or $K =$ **76.17 mld. Ans.**

Q. 1. (*c*) (*iii*) *If the alum dose to coagulate is 10 p.p.m., find out the amount of alum (in quintals) needed to treat 10 mld of water.* (2)

Solution. Water to be treated daily = 10 MLD

Alum dose required = 10 ppm = 10 mg/L

∴ Alum required = (10×10^6) L/d × 10 mg/L
= 100×10^6 mg/day
= 100 kg/day = **1 quintal. Ans.**

Q. 1. (*c*) (*iv*) *For a population of 2 lac having per capita daily demand of water of 150 litres, determine the water horse power to raise the water from a river of R.L. 120 m to treatment plant of R.L. 140 m through a rising main 1 m in diameter. Assume 2 m as the total head loss due to friction, etc. and pumping efficiency of 70%.* (2)

Solution. Water pumped daily

$= 2,00,000 \times 150$ L $= 30 \times 10^6$ L $= 30 \times 10^3$ m^3

Discharge pumped = $\frac{30 \times 10^3}{24 \times 3600}$ m^3/s = 0.347 m^3/s

Water Power = $\gamma_w Q . H$

where H = Total head lift = 20 + 2 = 22 m

∴ Water Power required = $9.81 \times 0.347 \times 22 = 74.94$ kW

∴ Water H.P. = $\frac{74.94}{0.735}$ HP = 101.96 HP.

Since pumping efficiency is 70%, HP of motor required

$= \frac{101.96}{0.7} =$ **145.66 HP. Ans.**

Q. 2. (*c*) *Explaining their qualitative differences in respect of total dissolved solids, turbidity and bacterial quality, suggest the units needed for the treatment of water drawn from (a) ground, (b) lake and (c) river.* (8)

Q. 3. (*c*) *Differentiate between a slow sand filter and a rapid sand filter in respect of*

(*i*) *Mechanisms of removal of impurities,*
(*ii*) *Cleaning process,*
(*iii*) *Effluent quality,*
(*iv*) *Bed size in qualitative term.* (8)

Q. 4. (*a*) (*i*) *A five cylinders reciprocating pump raises the water level by 150 m and the theoretical discharge is 0.20 m^3/s. The velocity in the delivery pipe is 2.0 m/s. Total head loss in pipes is 20 m. What is the input power, if the efficiency of the pump is 0.87 ?* (4)

Solution. Water Power Required = $\gamma_w . Q . H$

BHP or Power of Motor required = $\frac{\gamma_w . Q . H}{\eta}$

where $H = H_d + H_L = 150 + 20 = 170$ m

$$\therefore \quad \text{BHP} = \frac{9.81 \times 0.2 \times 170}{0.87} \text{ kN m/s } (i.e. \text{ kJ/s or kW})$$

$$= \mathbf{383.4\ kW.} \quad \textbf{Ans.}$$

$$= \frac{383.4}{0.735} \text{ HP} = \mathbf{521.6\ HP.} \quad \textbf{Ans.}$$

Q. 4. (*a*) (*ii*) *Two pipes of lengths 2500 m each and diameters of 80 cm and 60 cm respectively, are connected in parallel. The friction factor for each pipe is 4f = 0.024. Total flow is equal to 250 litres per second. Find the discharge in each pipe.* (4)

Solution.

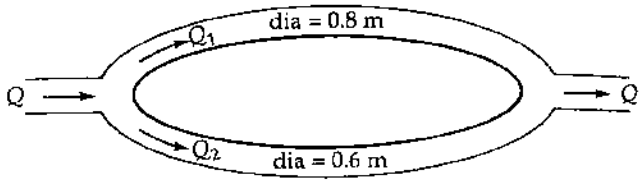


Fig. 17.9

Discharge in a pipe is given by Darcy's-Weisbach formula as

$$H_L = \frac{(4f)\,LV^2}{2gd} \qquad \text{...(}i.e.\text{ Eq. 6.1)}$$

$$\text{where } V^2 = \frac{Q^2}{A^2} = \frac{Q^2}{\left(\frac{\pi}{4}d^2\right)^2}$$

$$\text{or} \qquad H_L = \frac{(4f)\,L}{2gd} \cdot \frac{16Q^2}{\pi^2 . d^4}$$

$$\text{or} \qquad H_L = \left[\frac{8}{g\pi^2} \cdot \frac{(4f)\,LQ^2}{d^5}\right]$$

For first pipe :

$$H_L = \frac{8}{9.81 \times (3.14)^2}\left[\frac{0.024 \times 2500 \times Q_1^2}{(0.8)^5}\right] \qquad \text{...(}i\text{)}$$

$$\text{or} \qquad H_L = 15.129\, Q_1^2$$

For 2nd pipe, we have

$$H_L = \frac{8}{g\pi^2}\left[\frac{0.024 \times 2500 \times Q_2^2}{(0.6)^5}\right] \qquad \text{...(}ii\text{)}$$

$$\text{or} \qquad H_L = 63.76\, Q_2^2$$

Equating (*i*) and (*ii*), we get

$$15.129\, Q_1^2 = 63.76\, Q_2^2$$

$$\text{or} \qquad Q_1 = 2.05\, Q_2$$

But $Q_1 + Q_2 = Q = 250$ l/s

$$\therefore \quad 2.05\, Q_2 + Q_2 = 250 \text{ l/s}$$

$$\text{or} \quad Q_2 = \mathbf{82\ l/s}$$

$$\text{and} \quad Q_1 = \mathbf{168\ l/s.} \quad \textbf{Ans.}$$

Q. 4. (*c*) *How much will be the settling velocity of a spherical particle A (having specific gravity of 2.65) of diameter 10×10^{-3} cm. Determine the size for a floating spherical particle having a specific gravity of 0.80 (rising with the same velocity as that of particle A).*

Assume kinematic viscosity of water as 1.012×10^{-2} cm²/sec. (8)

Solution. The settling velocity is given by Stoke's law, given by Eq. (9.1)

$$\text{as} \qquad v_s = \frac{g}{18}(G-1)\frac{d^2}{\nu} \text{ for } d \leq 0.1 \text{ mm.} \quad \text{...(}i.e.\text{ Eq. 9.1)}$$

In this case, particle size d_1 in case $= 10 \times 10^{-3}$ cm

$= 10 \times 10^{-2}$ mm $= 0.1$ mm.

Hence, the above equation can be used.

$$\therefore \qquad v_s = \frac{9.81}{18}\frac{\text{m}}{\text{s}^2}(2.65-1)\left[\frac{(10 \times 10^{-5}\text{m})^2}{1.012 \times 10^{-6}\ \text{m}^2/\text{s}}\right]$$

$$\text{or} \qquad v_s = \mathbf{8.89 \times 10^{-3}\ m/s.} \quad \textbf{Ans.}$$

In the second case, we have to determine the particle dia. (d_2), assuming the rising velocity to be equal to (–) 8.89×10^{-3} m/s, as :

$$(-)\ 8.89 \times 10^{-3} = \frac{9.81}{18}(0.8-1)\left[\frac{d_2^2}{1.012 \times 10^{-6}}\right]$$

$$\text{or} \qquad d = 2.87 \times 10^{-4}\ \text{m} = \mathbf{0.287\ mm.} \quad \textbf{Ans.}$$

Q. 5. (*b*) (*i*) *Two reservoirs are connected by a pipe 100 m long and 100 mm in diameter followed by another pipe 60 m long and 50 mm in diameter. The total head loss between the reservoirs is 10.3 m. Given f = 0.03. Computer discharge neglecting minor losses.* (4)

Solution. The total head loss = Head loss in two pipes, where

Head loss in a pipe is given as

$$= \frac{f'LV^2}{2gd} = \frac{8}{g\pi^2}\left[\frac{f'LQ^2}{d^5}\right] \qquad \text{...(4.1)}$$

For 1st pipe, the head loss is given as :

$$H_{L_1} = \frac{8}{g\pi^2}\left[\frac{0.03 \times 100 \times Q^2}{(0.1)^5}\right]$$

$$\text{or} \qquad H_{L_1} = \frac{8}{g\pi^2}\left[3 \times 10^5\right]Q^2 \qquad \text{...(}i\text{)}$$

$$\text{Similarly,} \quad H_{L_2} = \frac{8}{g\pi^2}\left[\frac{0.03 \times 60 \times Q^2}{(0.05)^5}\right]$$

or $$H_{L_2} = \frac{8}{g\pi^2}\left[57.60 \times 10^5\right] Q^2 \quad \text{...(ii)}$$

But $H_{L_1} + H_{L_2} = 10.3$ m (given)

$$\therefore \quad \frac{8}{g\pi^2}\left[3 \times 10^5\right] Q^2 + \frac{8}{g\pi^2}\left[57.60 \times 10^5\right] Q^2 = 10.3$$

or $$\frac{8 \times 10^5}{g\pi^2} \cdot Q^2 \left[3 + 57.60\right] = 10.3$$

or $$\frac{8 \times 10^5 \times 60.6}{g\pi^2} Q^2 = 10.3$$

or $$Q = 4.54 \times 10^{-3} \text{ m}^3/\text{s} = \mathbf{4.54\ l/s.}\ \ \textbf{Ans.}$$

Year 2011 (*Engineering Services*)

Q. 1. (c) *In a water treatment plant, raw water undergoes the following treatment process viz. coagulation, flocculation, sedimentation and filtration. A plant of capacity to treat 3 × 10⁴ m³/d of raw water is installed for a township. The raw water source is having a suspended solid concentration of 400 ppm. Alum [$Al_2 SO_4)_3$ 14 H_2O] is used as a primary coagulant at a dose of 45 ppm. Compute the daily production of sludge if 95% of the total suspended solids are removed by the treatment processes. Neglect aluminium hydroxide quantity formed due to Alum dose.* (10)

Solution. Pl. refer Solved Example 9.15, and compute as under :

Total suspended solids in raw water = 400 ppm = 400 mg/L

Suspended solids removed = 95% × 400 mg/L = 380 mg/L

Neglecting the dry sludge produced by alum (dose of 45 mg/L) forming Aluminium hydroxide ppt; we have the total dry sludge (solids) removed = 380 mg/l.

Qty. of water treated = 3×10^4 m³/d

∴ Suspended solids removed daily

$$= 3 \times 10^4 \frac{\text{m}^3}{\text{d}} \times 380 \frac{\text{mg}}{\text{L}}$$

$$= 3 \times 10^4 \frac{\text{m}^3}{\text{d}} \times 380 \frac{\text{g}}{\text{m}^3} = 11400 \times 10^3 \text{ g/d}$$

$$= 11400 \text{ kg/d} = \mathbf{11.4\ t/d.}\ \ \textbf{Ans.}$$

Q. 3. (d) (*ii*) *Estimate the maximum water hammer pressure generated in a rigid pipe of a 3 m dia. pipe with an initial velocity of 3.0 ms^{-1} and the pipe is 8 km long. The downstream valve at the pipe end is closed in 4 seconds. The bulk modulus of water is 2.25 × 10⁶ kPa and the mass density is 995.7 kg/m³. Also determine the critical time of closure.* (6)

Solution. Max. water hammer pressure is given by Eq. (6.16) as :

$$p_{h(\max)} = \rho_w \cdot U_p V$$

where ρ_w = Density of water = 995.7 kg/m³

V = 3 m/s

U_p = Vel. of the generated pressure wave, given by Eq. (6.17) as:

$$U_p = \sqrt{\frac{E_w}{\rho_w}} \cdot \frac{1}{\sqrt{1 + \frac{E_w}{E_p} \cdot \frac{d}{t}}} \quad \text{..(Eq. 6.17)}$$

Since the thickness of pipe t and its Elasticity E_p is not given, the elasticity of the pipe can be ignored with conservative results, and the value U_p is then given by Eq. (6.17*a*) as :

$$U_P' = \sqrt{\frac{E_w}{\rho_w}} \quad \text{...(Eq. 6.17}a\text{)}$$

$$= \sqrt{\frac{2.25 \times 10^6 \text{ k} \cdot \frac{\text{N}}{\text{m}^2}}{995.7 \text{ kg/m}^3}} \sqrt{\frac{2.25 \times 10^6}{995.7} \cdot \frac{\text{k} \cdot \frac{\text{kgm}}{\text{s}^2} \cdot \frac{1}{\text{m}^2} \cdot \text{m}^3}{\text{kg}}}$$

$$= \sqrt{\frac{2.25 \times 10^6}{995.7} \times 1000 \times \frac{\text{m}^2}{\text{s}^2}} = \sqrt{\frac{2.25 \times 10^9}{995.7}} \text{ m/s}$$

$$= \mathbf{1503.24\ m/s}$$

Now, $$p_{h\max} = \left[995.7 \frac{\text{kg}}{\text{m}^3} \times 1503.24 \frac{\text{m}}{\text{s}} \times 3 \frac{\text{m}}{\text{s}}\right]$$

$$= 4490 \times 10^3 \frac{\text{kg}}{\text{m.s}^2} = 4490 \times 10^3 \frac{\text{Ns}^2}{\text{m}} \frac{1}{\text{ms}^2}$$

$$= 4490 \times 10^3 \frac{\text{N}}{\text{m}^2} \qquad \left[\because\ 1\text{kg} = 1\frac{\text{Ns}^2}{\text{m}}\right]$$

$$= \mathbf{4490\ kN/m^2}\ \ \textbf{Ans.}$$

Critical time of closure of valve is now given by Eq. (6.20) as :

$$T_c = \frac{2S}{U_p}$$

where S = Distance of the valve from the reservoir

= 8 km = 8000 m

$$\therefore \quad T_c = \frac{2 \times 8000 \text{ m}}{1503.24 \text{ m/s}} = \mathbf{10.64\ sec}\ \ \textbf{Ans.}$$

Since the valve was closed in 4 seconds, which time (T) is $\le T_c$, full maximum water hammer will be developed. Hence, the water hammer pressure developed in the given case

$$= \mathbf{4490\ kN/m^2.}\ \ \textbf{Ans.}$$

Q. 5. (a) (*i*) *Drawdown from multiple observation wells in a confined aquifer under steady state flow condition when 100 m³/day is pumped is given below.*

Determine the transmissivity assuming the linear variation semi log graph on the given data :

r(m) distance	*s(m) drawdown*
30	*2.79*
40	*2.505*
50	2.38
100	*1.83*
200	*1.28*
400	*0.73*

(4)

[**Solution.** The given values of r and s are used to plot a curve on a semi log graph paper, as shown in Fig. 17.10 st. line of best fit is

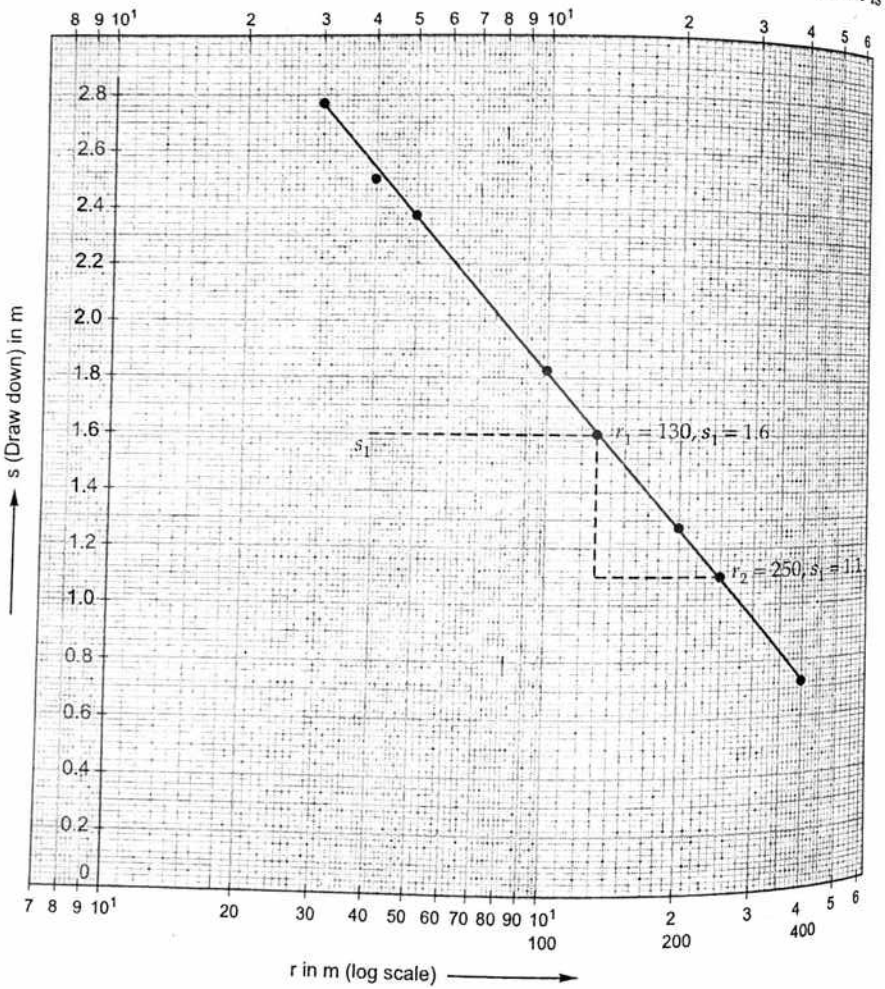


Fig. 17.10

drawn, from which any two values of r and s, lying on the st. line are chosen, say as :

$r_1 = 130$ m ; $s_1 = 1.6$ m

$r_2 = 250$ m ; $s_2 = 1.1$

Now, for confined aquifers, from Thiem's formula, we have from Eq. (4.35) as

$$Q = \frac{2\pi T(s_1 - s_2)}{2.3 \log_{10} \frac{r_2}{r_1}} \quad \text{...(Eq. 4.35)}$$

Substituting the values, $Q = 100$ m³/day. and above mentioned values of r_1, s_1, r_2 and s_2, we get

$$100 \text{ m}^3/\text{d} = \frac{2\pi T(1.6 - 1.1)}{2.3 \log_{10} \frac{250}{130}} = 4.81\ T$$

or $$T = \frac{100}{4.81} = \mathbf{20.79\ m^2/d.}\quad \textbf{Ans.}\ ]$$

[**Note.** If the semi log graph paper is not available in the Exam, then any of the given two values may be chosen as the values of r_1, r_2 and corresponding values of s_1 and s_2. For example, let us choose

$r_1 = 50$ m, $s_1 = 2.38$ m

$r_2 = 100$ m, $s_2 = 1.83$ m.

Then $$Q = \frac{2\pi T(s_1 - s_2)}{2.3 \log_{10} r_2/r_1}$$

or $$100 = \frac{2\pi T(2.38 - 1.83)}{2.3 \log_{10} \frac{100}{50}}$$

or $$T = \mathbf{20.04\ m^2/d.}\quad \textbf{Ans.}\ ]$$

Q. 5. (*a*) (*ii*) *Water flows at a velocity of 1 ms⁻¹ in a 150 mm dia new ductile iron pipe. Estimate the head loss over 500 m using Darcy-Weisbach formula for the given $k_s = 0.26$ mm.*

Assume viscosity = 1.00×10^{-6} m² s¹.

Hint : $\dfrac{1}{\sqrt{f}} = 2 \log \left[\dfrac{K_s}{3.7\ D} + \dfrac{2.51}{R_e \sqrt{f}} \right]$ (6)

[**Solution.** $$R_e = \frac{V.d}{\nu} = \frac{1 \frac{\text{m}}{\text{s}} \times \frac{150}{1000} \text{m}}{1 \times 10^{-6} \frac{\text{m}^2}{\text{s}}}$$

$$= 1.50{,}000$$

k_s (or δ or e) = 0.26 mm = 0.26×10^{-3} m.

$$\frac{1}{\sqrt{f}} = 2\log\left[\frac{\left(\frac{0.26}{1000}\right)}{3.7\times 0.150} + \frac{2.51}{1{,}50{,}000\sqrt{f}}\right]$$

$(\because \quad D = d = 150 \text{ mm} = 0.150 \text{ m})$

$$= 2\log\left[4.68\times 10^{-4} + \frac{1.673\times 10^{-5}}{\sqrt{f}}\right]$$

$$= 2\log\left[4.68\times 10^{-4} + \frac{0.1673\times 10^{-4}}{\sqrt{f}}\right]$$

or $$\frac{1}{\sqrt{f}} = 2\times 4\log\left[4.68 + \frac{0.1673}{\sqrt{f}}\right]$$

$$\frac{1}{\sqrt{f}} = 8\log\left[4.68 + \frac{0.1673}{\sqrt{f}}\right]$$

or $$\frac{1}{\sqrt{f}} = 8\log\left[\frac{4.68\sqrt{f} + 0.1673}{\sqrt{f}}\right]$$

Solving by Hit and Trial, we get

f = 0.0275

Head loss by Darcy-Weisbatch formula is given as :

$$H_L = \frac{f\,.\,LV^2}{2gd} = \frac{0.0275\times 500\times (1)^2}{2\times 9.81\times 0.15} \text{ m} = 4.67 \text{ m}$$

Hence, H_L = **4.67 m. Ans.**]

Q. 5. (*b*) *What are the four tests to be carried out for physical examination of water quality in a natural river flowing over an alluvial bed ?*

Explain briefly the procedure and instruments required to carry out the tests. (10)

[**Hint.** The four tests for physical examination of water are for : Turbidily, Colour, Tastes, and Odours, and specific conductivity of water. as explained in the text.]

Q. 6. (*b*) (*ii*) *What is the intrinsic permeability of a water saturated medium that has hydraulic conductivity of 15.24 m / day ? Assume the ground water is at atmospheric pressure at 20°C and has a density of 998.2 kg / m³ and viscosity of 1.002 × 10⁻³ kg/ms.* (4)

[**Solution.** Intrinsic permeability (K_0) is given by Eq. (4.12) as :

$$K_0 = K\,.\,\frac{\nu}{g} \qquad \text{...(i.e. Eq. 4.12)}$$

or $$K_0 = K\,.\left(\frac{\mu}{\frac{\rho}{g}}\right) = \frac{K\,.\,\mu}{\rho\,.\,g}$$

where μ is dynamic viscosity, which is given as = 1.002×10^{-3} kg/m.s

Here $K = 15.24 \text{ m/d} = \frac{15.24}{24\times 3600} \text{ m/s}$

$= 1.764\times 10^{-4}$ m/s

$\mu = 1.002\times 10^{-3}$ kg/m.s

$\rho = 998.2$ kg/m³

$g = 9.81$ m/s².

Substituting values, we get

$$K_0 = \frac{1.764\times 10^{-4}\ \frac{\text{m}}{\text{s}} \times 1.002\times 10^{-3}\ \frac{\text{kg}}{\text{m.s}}}{998.2\ \frac{\text{kg}}{\text{m}^3} \times 9.81\ \frac{\text{m}}{\text{s}^2}}$$

or $K_0 = 1.805\times 10^{-11}$ m². **Ans.**

But 1 Darcy = 9.87×10^{-13} m²

$$\therefore \quad K_0 = \frac{1.805\times 10^{-11}}{9.87\times 10^{-13}} = \textbf{18.29 Darcy.} \quad \textbf{Ans.}$$]

Year 2012 (*Engineering Services*)

Q. 3. (*b*) *In fine sand aquifer region, design a well to get 10 litres / sec yield, under depression head of 2.5 metres. Value of* $\frac{C}{A}$ *in cum/h per sqm of area may be taken as 0.60.* (10)

[**Solution.** Follow exactly similar Solved Example 4.5 for designing an open well, and compute as under :

Q = Discharge required = 10 l/s = $\frac{10}{1000}$ m³/s = 0.01 m³/s

$= 0.01\times 60\times 60$ m³/h = 36 m³/h

$\frac{C'}{A}$ = Sp. capacity or Specific yield = 0.60 m³/h/m² per m of drawdown*

Using Eq. (4.23), we have

$$Q = \left(\frac{C'}{A}\right) A.s \qquad \text{...(Eq. 4.23)}$$

Substituting given values, we get

$$36\ \frac{\text{m}^3}{\text{h}} = 0.6\ \frac{\text{m}^3}{\text{h}\cdot\text{m}^2\cdot\text{m}} \times A \times 2.5 \text{ m}$$

or $A = 24$ m²

If d_w is the dia. of the well, then $\frac{\pi d_w^{\,2}}{4} = 24$

or d_w = **5.53 m. Ans.**]

*The correct units of $\frac{C'}{A}$ is m³/h per m² of area per m of drawdown, as given in Table 4.5. The Examiner has missed mentioning per m of drawdown.

Q. 5. *(c) Calculate the size of a rectangular tank to treat 2.0 million litres of water per day. The detention period may be assumed as 3 hours and overflow rate less than 40,000 lit. per sqm of the surface area per day.* (10)

Solution. Pl. refer Solved Example 13.1 and compute as under :

Qty. of water to be treated per day = Q = 2 Ml = 2×10^6 litres

$\therefore$ Qty. of water to be treated during the detention period of 3 hours

$$= \left[\frac{2\times10^6}{24}\times 3\right] \text{ litres} = 25 \times 10^4 \text{ litres/day}$$

$\therefore$ The capacity of the tank required = 25×10^4 litres.

Now, the *overflow rate, i.e.* the discharge per unit of surface area per day $\le$ 40000 lit/m^2.d

$$\therefore \quad \frac{Q}{\text{Surface area}} = \frac{2\times10^6 \text{ l/day}}{L\times B} \le 40000 \text{ l/m}^2\text{.d}$$

or $$(L \times B) \ge \frac{2\times10^6}{40000} \text{ m}^2$$

or $$(L \times B) \ge 50 \text{ m}^2$$

Depth of tank

$(D) \times (L \times B)$ = Vol. of tank = 25×10^4 litres

$= 250$ m^3

$$\therefore \quad D = \frac{250 \text{ m}^3}{(L\times B)} = \frac{250 \text{ m}^3}{\ge 50 \text{ m}^2}$$

or $$D \le 5 \text{ m}$$

Let us assume water depth D = 4 m (< 5 m)

$$\therefore \quad BL = \frac{250}{4} = 62.5 \text{ m}^2$$

Also assume $L = 3B$ (say)

$$\therefore \quad 3B.B = 62.5$$

$$\therefore \quad B = \sqrt{\frac{62.5}{3}} = 4.57 \text{ m, say } 4.6 \text{ m}$$

$$\therefore \quad L = 3 \times 4.6 = 13.8 \text{ m}$$

$$\therefore \text{ Overflow rate} = \frac{Q}{BL} = \frac{2\times10^6 \text{ l/day}}{4.6\times13.8} = 31506$$

(which is OK being < 40000)

Using a free-board of 0.5 m, Overall depth of tank

$= 4 + 0.5 + 4.5$ m

Hence, use a rectangular tank with an overall size

13.8 m × 4.6 m × 4.5 m. **Ans.**

Q. 6. *(a) (i) A tube well of 15 cm diameter penetrates fully an artesian aquifer 27 m thick. Determine the permeability of the aquifer if a steady discharge of 30 lps is obtained from the well under a draw down of 3.0 m at the well face. Take radius of influence equal to 200 m.* (6)

Solution. Use Dupuit's formula for an artesian aquifer as :

$$Q = \frac{2\pi KHs}{2.3 \log_{10}\left(\frac{R}{r_w}\right)} \quad \text{...(Eq. 4.39)}$$

where K = ?

H = 127 m

s = 3.0 m

R = 200 m

r_w = 15 cm = 0.15 m

$$Q = 30 \text{ lps} = \frac{30}{1000} \text{ m}^3\text{/s} = 0.03 \text{ m}^3\text{/s}$$

$$\therefore \quad 0.03 = \frac{2\times3.14\times K\times27\times3}{2.3\log_{10}\left(\frac{200}{0.03}\right)}$$

or $$\frac{0.03\times2.3\times3.824}{2\times3.14\times27\times3} = K \quad \text{or} \quad K = 5.18 \times 10^{-4} \text{ m/s.} \quad \textbf{Ans.}$$

Q. 7. *(a) (i) Determine the maximum time for rapid closure of pipe 60 cm diameter, 450 m long made of steel ($E = 2.07 \times 10^{11}$ N/m^2) with a wall thickness of 1.25 cm. The pipe carries a liquid of specific gravity 0.88 and bulk modulus $K = 1.03 \times 10^9$ N/m^2 flowing at 1 m^3/sec. (E = Young's modulus of steel)* (4)

Solution. The velocity of pressure wave generated due to rapid closure of a valve in a pipe line is given by Eq. (6.17) as :

$$U_p = \sqrt{\frac{E_w}{\rho_w}} \cdot \frac{1}{\sqrt{1+\frac{E_w}{E_p}\cdot\frac{d}{t}}} \quad \text{...(Eq. 6.17)}$$

where E_w = bulk modulus of water/liquid, *i.e.* K

$= 1.03 \times 10^9$ N/m^2 = 1.03×10^6 kN/m^2

E_p = elasticity of pipe = E

= 2.07×10^{11} N/m^2 = 2.07×10^8 kN/m^2

d = diameter of the pipe = 60 cm = 0.6 m

t = thickness of pipe = 1.25 cm = 0.0125 m

ρ_w = density of liquid = 0.88 × density of water

$$= 0.88 \left(\frac{\rho_w}{g}\right)$$

$$= 0.88 \frac{9.81 \text{ kN/m}^3}{9.81 \text{ m/s}^2}$$

$$= 0.88 \text{ kN} \frac{\text{kN}}{\text{m}^4}.\text{s}^2$$

Substituting values, we get

$$U_p = \sqrt{\frac{1.03 \times 10^6 \frac{\text{kN}}{\text{m}^2}}{0.88 \frac{\text{kN}}{\text{m}^4}.\text{s}^2}} \frac{1}{\sqrt{1 + \frac{1.03 \times 10^6}{2.07 \times 10^8} \times \frac{0.6}{0.0125}}}$$

$$= 1081.88 \frac{\text{m}}{\text{s}} \times 0.898 = 972 \text{ m/sec}$$

Critical time of closure $(T_c) = \frac{2S}{U_p}$ ...(Eq. 6.20)

or $$T_c = \frac{2 \times 450 \text{ m}}{972 \text{ m/sec}} = \mathbf{0.93 \text{ sec.}}$$

∴ **Maximum Time for rapid closure = 0.93 sec. Ans.**

Q. 8. *(a) (ii) A pipeline of 0.6 m diameter is 1.5 km long. To augment the discharge, another pipe-line of the same diameter is introduced parallel to the first in the second half of the length neglecting minor losses, find the increase in discharge if f = 0.04 and head above the outlet is 30 m.* (6)

[Hint. Pl. refer Solved Example 6.6]

Year 2013 (*Engineering Services*)

Q. 1. *(e) What are the sources and impact of hardness in water supplies ? Also mention standard of hardness in water supply.* (4)

Q. 2. *(a) A new 5 km long pipeline connects two reservoirs. The water surface elevation of the upper reservoir is 1100 m and that of the lower reservoir is 835 m. The pipeline is 400 mm nominal diameter of welded steel (Hazen Williams Coefficient C = 130) with a square mouth inlet (14.32 m long) and including the following fittings :*

1. Gate valves	*10 numbers*	*Each gate valve of equivalent length*	*0.98 m*
2. Standard radius 90° elbow	*19 numbers*	*Each elbow of equivalent length*	*6.4 m*
3. Standard radius 45° elbow	*37 numbers*	*Each elbow of equivalent length*	*4.57 m*
4. Straight Tees	*8 numbers*	*Each Tee length*	*2.19 m*

What is the water flow rate between the reservoirs ? Use the concept of equivalent pipe.

Given $h_f = \frac{10.7 Q^{1.85} L}{C^{1.85} D^{4.87}}$. (8)

Solution. The total head loss H_L or h_f

= Difference in water levels of two reservoirs connected by a pipe of 400 mm dia and of 5 km length

= 1100 m – 835 m = **265 m.**

The above total head loss in flow is caused by the loss of head in pipe of 5 km length and additional head loss in inlet connection and fixtures (like valves, elbows, tees, etc.)

The equivalent length of all fixtures

= 14.32 m (inlet) 10 × 0.98 (Gate valves)
+ 19 × 6.4 m (90° elbows) + 37
× 4.57 m (45° elbows) + 8 × 2.19 m (Tees)

= (14.32 + 9.8 + 121.6 + 169.09 + 17.52) m

= **332.33 m**

This means that the head loss caused in fixtures will be the same as will be caused in a pipe of 332.33 m length and of 0.4 m (400 mm) dia with the given discharge Q (to be computed).

∴ The total head loss will thus be caused by discharge Q in pipe of length

= 5000 m (*i.e.* 5 km) + 332.33 m = 5332.33 m.

Since the dia of the pipe (D) is 0.4 m (*i.e.* 400 mm) and the Hazen William coefficient of its material (C) is 130, we get total head loss

$$h_f = \frac{10.7 Q^{1.85} . L}{C^{1.85} . D^{4.87}} = \mathbf{265 \text{ m}}$$

or $$\frac{10.7 Q^{1.85}}{(130)^{1.85} (0.4)^{4.87}} = 265$$

or $$Q = 66.04 \text{ cumecs.} \quad \textbf{Ans.}$$

Q. 5. *(b) What is type I settling ? How analysis of type I settling is done ? Find the settling velocity of a discrete particle in water having diameter 4 × 10⁻³ cm and specific gravity 2.65. Take kinematic viscosity of water = 1.02 × 10⁻² cm²/sec.* (10)

Solution. Type I settling is plain sadimentation, the analysis of which is explained in article 9.5.2. For the numerical question, pl. follow Solved Example 9.6, and compute as under :

The settling velocity of a discrete particle in water (v_s) is given by Eq. (9.1) as :

by Eq. (9.1) as :

$$v_s = \frac{g}{18}(G-1)\frac{d^2}{\nu} \qquad \text{for } d < 0.1 \text{ mm}$$

In this question,

$d = 4 \times 10^{-3}$ cm, *i.e.* 4×10^{-2} mm = 0.04 mm,

which is < 0.1 mm. Hence, the formula is applicable.

Using the given values, we have

$g = 981$ cm/s^2

$G = 2.65$

$d = 4 \times 10^{-3}$ cm = 0.004 cm

$\nu = 1.02 \times 10^{-2}$ cm^2/sec = 0.0102 cm^2/s.

$$\therefore \quad v_s = \frac{981}{18}(2.65-1)\frac{(0.004)^2}{0.0102} \text{ cm/sec}$$

$$= \mathbf{0.141 \text{ cm/sec. Ans.}}$$

Year 2014 (*Engineering Services*)

Q. 2. (c) *Two pipelines have been connected to a large water reservoir. One of the pipes is 15 cm × 300 m length with an outlet 4 m below the reservoir water level, while the other pipe is 20 cm × 600 m length. Both the pipes have free discharge at outlet end and the total discharge rate is 50 L/s. Calculate the difference of elevation between the reservoir water level and the outlet of 20 cm pipe. Neglect entrance losses, and for all pipes f = 0.08 in Darcy's formula*

$$h_f = \frac{flv^2}{2gd} \qquad (8)$$

Solution. Head loss h_{f1} in first pipe = 4 m

Let outlet discharge from 1st pipe = Q_1

Now, using Eq. $h_f = \dfrac{flv^2}{2gd}$, we have

$$h_f = \frac{f.l.\left(\dfrac{Q}{\dfrac{\pi}{4}d^2}\right)^2}{2gd} = \frac{f.l.Q^2}{\left(\dfrac{\pi}{4}\right)^2.2g.d^5}$$

or $$h_f = \frac{flQ^2}{12.1d^5}$$

Substituting values for 1st pipe, we have

$$4 = \frac{0.08 \times 300 \times Q_1^2}{12.1(0.15)^5}$$

or $$Q_1 = 0.012375 \text{ m}^3/\text{s} = 12.375 \text{ L/s}$$

Total discharge $Q = Q_1 + Q_2 = 50$ L/s (given)

$\therefore$ Discharge through 2nd pipe = $Q_2 = (50 - 12.375)$ L/s

= 37.625 L/s = 0.037625 m^3/s.

For 2nd pipe, h_{f_2} is computed as :

$$h_{f_2} = \frac{0.08 \times 600 \times (0.037625)^2}{12.1 \times (0.20)^5} = \mathbf{17.54 \text{ m. Ans.}}$$

Q. 2. (e) *A square filter box is to be designed for a filtration rate of 2.8 L/m^2 s. What are the required surface area and side dimension of the unit if the flow rate is 6 ML/d? If the filter is backwashed once a day for 12 minutes at a rate of 10 L/m^2 s, what percentage of the total flow rate is used for cleaning the filter?* (8)

Solution. Flow rate = 6 ML/d

Filtration rate = 2.8 L/m^2.s

= [2.8 × (24 × 3600)] L/m^2 per day,

$\therefore$ Area of filter reqd.

$$= \frac{6 \times 10^6 \text{ L/d}}{(2.8 \times 24 \times 3600) \text{ L/m}^2.\text{d}} = 24.8 \text{ m}^2.$$

Side dimension of square filter = $\sqrt{24.8}$ m

= 4.98 m; **say 5 m Ans.**

$\therefore$ Surface area of filter

= 25 m^2. Ans.

For backwashing of filter, quantity of water reqd. per day @ 10 L/m^2.s for 12 minutes washing per day

= 10 L/m^2.s × (25 m^2) × (12 × 60 s)

= 1,80,000 L/d. ≈ 3% of 6 MLD.

Hence, 3% of total flow rate of 6 MLD is to be used for backwashing. **Ans.**

Q. 3. (c) *A water treatment plant is designed to treat a flow of 20000 m^3/day. The chlorine dosage is 1 mg/L. What size of containers should be used in this plant? What is the minimum weight of chlorine which should be kept in hand?* (8)

Solution. Water treated per day = 20,000 m^3/d

= 20,000 × 1000 l/d = 20 MLD

Qty. of chlorine reqd. at 1 mg/L dosage per day

$$= (20 \times 10^6)\frac{L}{d} \times 1 \text{ mg/L}$$

= 20 × 10^6 mg/d = 20 kg/d

Hence, a minimum of 20 kg of chlorine has to be made available at the above water treatment plant (WTP) for meeting daily requirement. However, the chlorine cylinders, which are generally available in India for use at water treatment plants, contain 900 kg of chlorine (liquid form).* Accordingly, even if one such cylinder (container) is used at this plant, it will suffice for 45 days. We can, hence, go in for use of one such cylinder of this size. One additional cylinder can be kept in storage, as stand by storage for change, when needed.

Ans.]

Q. 4. (*a*) *A 300 mm diameter pipe of mild steel having 6 mm thickness carries water at the rate of 200 litres/s. What will be the rise in pressure if the valve at the downstream end is closed instantaneously? Compare the results assuming the pipe to be rigid as well as elastic. What should be the maximum closing time for the computed results to be valid? Take pipe length as 8.0 km. The ratio of modulus of elasticity and bulk modulus of water may be taken as 100. Assume any other data only if required and state the same clearly.* (8)

[**Solution.** Pl. refer Solved Example 6.11 and compute as under :

Velocity of water in the pipe of 0.3 m dia

$$(300 \text{ mm dia}) = \frac{\text{Discharge}}{\text{Area}} = \frac{200 \text{ l/s}}{\frac{\pi}{4} \times d^2}$$

$$= \frac{0.2 \text{ m}^3/\text{s}}{\frac{\pi}{4} \times (0.3 \text{ m})^2} = 2.83 \text{ m/s}$$

(*i*) **Rigid pipe case :** When the elasticity of pipe in ignored, *i.e.* when the pipe is rigid, the velocity of the pressure wave caused by water hammer is given as :

$$U_p' = \sqrt{\frac{E_w}{\rho_w}} \quad \text{... (}i.e.\text{ Eq 6.17 } a\text{)}$$

where $\sqrt{\frac{E_w}{\rho_w}}$ equals the velocity of sound in water

$= 1433$ m/s.

∴ Max. water hammer pressure $p_{h(\max)}$ is then given as :

$$p_{h(\max)} = \rho_w . U_p' V$$

$$= 1433 \rho_w : V = 1433 (1 \times 2.83) \quad ...(i)$$

$$= \mathbf{4055.4 \text{ kN/m}^2}. \quad \textbf{Ans.} \;]$$

*Cylinders containing 100 kg of chlorine were available in olden days, and such smaller cylinders can also be used at the above plant if available or can be got manufactured on special orders, but will need change of cylinder every 5 days. A number of such smaller cylinders will therefore have to be kept at the plant as stand by.

(*ii*) **Elastic pipe case :** When the elasticity of the pipe meterial is also considered, then the velocity of pressure wave is given as :

$$U_p = \sqrt{\frac{K}{\rho_w}} = \sqrt{\frac{E_w}{\rho_w}} \cdot \frac{1}{\sqrt{1 + \frac{E_w}{E_p} \cdot \frac{d}{t}}} \quad \text{... } i.e.\text{ Eq. (6.17)}$$

where $\sqrt{\frac{E_w}{\rho_w}}$ = velocity of sound in water

≈ 1433 m/sec

$$\therefore \quad U_p = 1433 \frac{1}{\sqrt{1 + \frac{E_w}{E_p} \cdot \frac{d}{t}}}$$

where $\frac{E_w}{E_p} = \frac{\text{Modulus of elasticity of water}}{\text{Modulus of elasticity of pipe metrial}}$

$$= \frac{1}{100} = 0.01 \text{ (given)}$$

$$\therefore \quad U_p = 1433 \frac{1}{\sqrt{1 + 0.01 \times \left(\frac{300 \text{ mm}}{6 \text{ mm}}\right)}}$$

$$= 1433 \times \frac{1}{\sqrt{1.5}} = \mathbf{1170 \text{ m/s}}$$

Now $p_{h(\max)} = \rho_w . U_p . V = [1 \times 1170 \times 2.83] \text{ kN/m}^2$

$$= \mathbf{3311.2 \text{ kN/m}^2}. \quad \textbf{Ans.} \;] \quad ...(ii)$$

(*iii*) Maximum closing time for the valve at the downstream end for development of max. water hammer pressures (computed above) is given by the critcal time (T_c), which is given by Eq. (6.20) as :

$$T_c = \frac{2S}{U_p}$$

$$= \frac{2 \times 8000 \text{ m}}{1170 \text{ m/s}} = \mathbf{13.68 \text{ seconds}}. \quad \textbf{Ans.} \;]$$

Year 2015 (*Engineering Services*)

Q. 1. (e) *Give suitable reasons why the following are important parameters for drinking water quality:*

(*i*) *Nitrate* (*ii*) *Fluoride*

(*iii*) *Total coliforms* (*iv*) *Iron.* (8)

Q. 3. (c) *Explain the significance of alkalinity in coagulation practice and in lime-soda softening.* (8)

Q. 4. (c) *In a treatment plant, two tanks of rapid sand filtration are proposed after sedimentation. Each filter bed has a surface area of 3 m × 2 m. The design flow rate to each tank of filters is 0.044 m^3/sec. The design loading rate to each tank of filters is 150 $m^3/day/m^2$. Determine the number of filter beds in each tank of filters. Determine the loading rate when one filter is out of service. The maximum acceptable loading rate is 235 $m^3/day/m^2$.* (8)

Solution. Design flow to each filter tank = 0.044 m^3/s

Total daily flow to each filter tank

$$= 0.044\ m^3/s \times (24 \times 3600)\ s/d = 3801.6\ m^3/d$$

When loading rate to each tank of filters is 150 $m^3/d/m^2$, we have the total area of filter beds required in each tank

$$= \frac{3801.6\ m^3/d}{150\ m^3/d/m^2} = 25.34\ m^2$$

The area of one filtered = 3 m × 2 m = 6 m^2

∴ No. of filter beds required in each tank

$$= \frac{25.34\ m^2}{6\ m^2} = 4.22;\ \text{say } \mathbf{4\ Nos.\ Ans.}$$

Say, let us use 4 filter beds of size 3 m × 2 m in each filter tank.

Area of filters provided in each filter tank

$$= 4\ \text{Nos.} \times (3\ m \times 2\ m) = 24\ m^2.$$

$$\text{Actual loading rate} = \frac{3801.6\ m^3/d}{24\ m^2}$$

$$= 158.4\ m^3/d/m^2 < 235\ m^3/d/m^2\ (\text{OK})$$

Hence, we can provide 4 filter beds in each tank. **ANS.**

When one filter (out of four) is out of service, we will have the loading rate for 3 working filters

$$\frac{3801.6\ m^3/d}{3\ \text{Nos.} \times (3\ m \times 2\ m)} = \mathbf{211.2\ m^3/d/m^2} < 235$$

(max. permissible) ; and hence OK. **Ans.**

Hence, the loading rate will not be exceed the max. permissible, even when one out of four filter beds, goes out of service. **Ans.**

—18

*Objective Questions of the Engineering Services Exams.**

Year 1993

Q. 1. Match List I and List II and select the correct answer using the codes given below in the lists :

List I (Source of water supply)	List II (Water quality parameter)
A. Ground water	1. Low D.O. and colour
B. Lake water at the bottom	2. Carbon dioxide and hardness
C. Canal water	3. Low organic content and high D.O.
D. River water in floods	4. Silt and bacteria
	5. High turbidity and D.O.

Codes	A	B	C	D
(a)	2	1	3	5
(b)	3	4	2	1
(c)	3	2	4	1
(d)	1	2	3	5.

Q. 2. The given figure shows roughly the daily mass curves of supply to and demand from an elevated service reservoir (E.S.R.). The minimum required capacity of the reservoir is given by :

(a) $(A - B)$
(b) $(A + B)$
(c) $(A \times B)$
(d) larger of A or B.

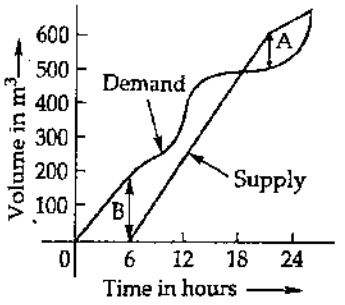


Fig. 18.1

Q. 3. If present in water, chlorination of water does not reduce the :

(a) ammonia content (b) organic matter content
(c) B.O.D. (d) dissolved oxygen content.

Q. 4. Dechlorination of water is achieved by adding :

(a) sodium thiosulphate (b) sodium sulphate
(c) sodium hexametaphosphate
(d) sodium bisulphate.

*Hints for solving starred Questions are given at the end of the chapter.

Q. 5. The efficiency of sedimentation tank does not depend open :
(*a*) detention time (*b*) depth of the tank
(*c*) length of the tank (*d*) horizontal velocity of water.

Q. 6. Which of the following treatments reduce salinity of water :
1. Flocculation and sedimentation
2. Filtration
3. Reverse osmosis
4. Electrodialysis.
Select the correct answer using the codes given below :
Codes
(*a*) 1 and 2 (*b*) 3 and 4.
(*c*) 2 and 3 (*d*) 1 and 4.

Q. 7. A city supply of 15000 cubic metres of water per day is treated with a chlorine dosage of 0.5 ppm. For this purpose, the requirement of 25% bleaching powder per day would be :
(*a*) 300 kg (*b*) 75 kg
(*c*) 30 kg (*d*) 75 kg.

Q. 8. The distribution configuration shown in the figure is called :
(*a*) dead end system
(*b*) grid iron system
(*c*) radial system
(*d*) ring system.

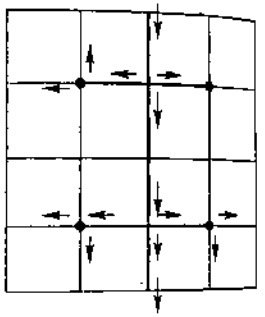


Fig. 18.2

Q. 9. The results of analysis of a sample of water is given below :
Turbidity = 10 mg/L ; colour = slightly yellowish : pH = 7.4 ; taste and odour = inky taste ; total dissolved solids = 450 mg/L : total hardness = 200 mg/L ; Chlorides = 100 mg/L ; Sulphates = 200 mg/L ; Fluorides = 0.5 mg/L; Nitrates = 30 mg/L ; Calcium = 75 mg/L ; Magnesium = 200 mg/l ; Iron = 3.0 mg/L. From the data given above, it can be inferred that the water needs :
(*a*) no treatment except disinfection
(*b*) treatment for removal of iron, hardness, and disinfection
(*c*) treatment for removal of colour, taste and odour
(*d*) treatment for removal of iron, taste, odour and disinfection.

Q. 10. Match List I with List II, and select the correct answer using to codes given below the lists :

List I	List II
A. Cast iron pipe	1. Simplex joint
B. Asbestos cement pipe	2. Spigot and socket joint
C. Plain ended pipes subjected to frequent vibrations	3. Screwed joint
D. G.I. pipe	4. Victaulic joint

Codes :	A	B	C	D
(*a*)	1	3	2	4
(*b*)	2	1	4	3
(*c*)	3	4	2	1
(*d*)	4	2	1	3.

Q. 11. The water supply to a house begins with the connection of the service pipe with the municipal water mains. The connection comprises :
1. Stop cock ; 2. Goose neck ;
3. Ferrule ; 4. Water meter.
The correct sequence of these connections is :
(*a*) 1, 2, 3, 4 (*b*) 3, 1, 2, 4
(*c*) 3, 2, 1, 4 (*d*) 1, 2, 4, 3.

Q. 12. When the coefficient of rugosity is increased from 0.01 to 0.02, the gradient of a pipe of a given diameter to carry the same flow at the same velocity will be :
(*a*) increased by 4 times (*b*) increased by 2 times
(*c*) decreased by $\sqrt{2}$ times (*d*) decreased by 4 times.

Year 1994

Q. 13. Consider the following statements regarding confined aquifers :
1. The aquifer is bound at top and below by impervious strata.
2. The pressure of water is greater than atmospheric pressure.
3. A tubewell sunk in such an aquifer starts flowing always by itself.
4. The aquifer is fully saturated
Of the above statements :
(*a*) 1, 2 and 3 are correct (*b*) 2, 3 and 4 are correct
(*c*) 1, 2 and 4 are correct (*d*) 1, 3 and 4 are correct.

Q. 14. Consider the following statements in respect of slow sand filters :
1. Qualitywise they are more efficient than Rapid sand filters.
2. They need periodic back washing.
3. Their flow rate is much smaller than that of pressure filters
Of these statements :
(*a*) 1, 2 and 3 are correct (*b*) 1 and 2 are correct
(*c*) 1 and 3 are correct (*d*) 2 and 3 are correct.

Q. 15.* Type II settling in water treatment is defined as :
(*a*) settling of discrete particles in dilute suspensions
(*b*) settling of flocculent particles in dilute suspensions
(*c*) settling of flocculent particles in concentrated suspensions
(*d*) settling of particles in the form of a sludge blanket.

Q. 16. Which of the following pairs are correctly matched ?
1. Lime soda process ..... Softening
2. Nalgonda technique ..... Fluoride removal
3. Aeration ..... Coagulation
4. Ozonation ..... Disinfection.

Select the correct answer using the codes given below :

Codes

(*a*) 1, 2 and 3 (*b*) 1, 3 and 4

(*c*) 1, 2 and 4 (*d*) 2, 3 and 4.

Q. 17.* The chlorine demand of a water sample was found to be 0.2 mg/l. The amount of bleaching powder containing 30% available chlorine to be added to treat one litre of such a water sample is

(*a*) 0.67 mg (*b*) 0.06 mg

(*c*) 1.33 mg (*d*) 0.14 mg.

Q. 18. Consider the following statements :

In water-supply distribution network, the dead end system is not favoured because

1. it is cumbersome in terms of design.
2. the pressure at the dead-ends becomes undesirably low in the case of additional extensions.
3. it is difficult to maintain residual chlorine levels at the dead ends.
4. at the time of repairs, service connections beyond the point of repair are deprived of water.

Of these statements.

(*a*) 1, 2 and 3 are correct (*b*) 2, 3 and 4 are correct

(*c*) 1, 3 and 4 are correct (*d*) 1, 2 and 4 are correct.

Q. 19.* The ideal residual pressure at the farthest consumer's tap in a properly designed water distribution system is in the range of

(*a*) 0.06 to 0.20 N/mm^2 (*b*) 0.21 to 0.25 N/mm^2

(*c*) 0.26 to 0.30 N/mm^2 (*d*) 0.31 to 0.35 N/mm^2.

Year 1995

Q. 20. The following residual chlorine compounds are formed during chlorination of water :

(1) NH_2Cl, (2) $NHCl_2$, (3) HOCL, (4) OCl^-

The correct sequence of formation of these residual chlorine compounds is :

(*a*) 2, 1, 3, 4 (*b*) 1, 2, 4, 3

(*c*) 1, 2, 3, 4 (*d*) 2, 1, 4, 3.

Q. 21. Match List I with List II and select the correct answer using the codes given below the lists :

List I *(Type of water source)*	List II *(Treatment to be given)*
A. Surface water (river or canal)	1. Aeration, coagulation, sedimentation and disinfection
B. Water from infiltration gallery	2. Disinfection
C. Lake/Pond water	3. $CuSO_4$ treatment, coagulation, sedimentation, filtration and disinfection
D. Tubewell water	4. Coagulation, flocculation, sedimentation, filtration, and disinfection

Codes	A	B	C	D
(*a*)	4	1	3	2
(*b*)	1	4	3	2
(*c*)	1	4	2	3
(*d*)	4	1	2	3

Q. 22. The flow chart of a water treatment plant is shown in the following figure :

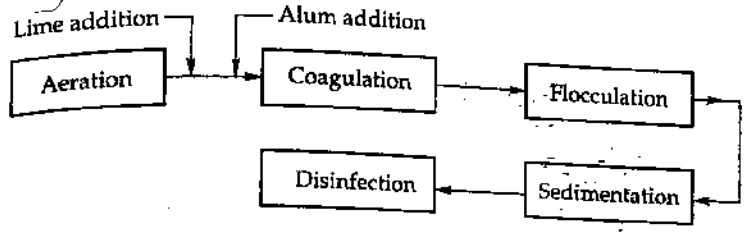


Fig. 18.3

It is proposed to, defluoridate the water using 'Nalgonda treatment', then it should be done

(*a*) by adjusting the dose of lime and alum

(*b*) after sedimentation

(*c*) after filtration (*d*) before aeration.

Q. 23. Two reservoirs at different levels are connected by two parallel pipes of diameter $2d$ and d. The ratio of the flows in the two pipes (larger : smaller) is

(*a*) $\sqrt{2}$: 1 (*b*) 2 : 1 (*c*) 4 : 1 (*d*) $4\sqrt{2}$: 1.

Q. 24. If the total hardness and alkalinity of a sample of water are 300 mg/l and 100 mg/l ($CaCO_3$ scale) respectively, then the carbonate and non-carbonate hardness (in units of mg/l) will be respectively

(*a*) 100 and 200 (*b*) 400 and 300

(*c*) 100 and 400 (*d*) 400 and zero.

Q. 25. Electrical conductivity (EC) of water and total dissolved solids (TDS) are interrelated. The value of EC will

(*a*) decrease with increase in TDS

(*b*) increase with increase in TDS

(*c*) decrease initially and then increase with increase in TDS

(*d*) increase initially and then decrease with the increase in TDS.

Q. 26. When the identical centrifugal pumps are operating in series on a common rising main, then

(*a*) the pressure in the rising main will be nearly doubled while the discharge will remain the same.

(*b*) the discharge will be nearly doubled while the pressure remains the same

(*c*) discharge as well as pressure in the rising main will be doubled

(*d*) discharge as well as the pressure in the rising main will increase but not become double.

Year 1996

Q. 27. The yield of a well depends upon :
(*a*) permeability of soil
(*b*) area of aquifer opening into the wells
(*c*) actual flow velocity
(*d*) all of the above.

Q. 28. The construction of impounding reservoir is required when :
(*a*) average annual flow in the stream is lower than average demand
(*b*) the rate of flow in the stream, in dry season is more than the demand
(*c*) the rate of flow in the stream, in dry season is less than the demand
(*d*) the rate of flow in the stream is equal to the demand.

Q. 29. Which of the following treatments reduce salinity of water ?
1. Flash mixing and sedimentation
2. Electrodialysis
3. Reverse osmosis
4. Freezing
5. Filtration

Select the correct answer using the codes given below :
Codes :
(*a*) 1, 2, 3, 4 and 5 (*b*) 2, 3 and 4
(*c*) 1, 3 and 5 (*d*) 1, 2 and 4.

Q. 30. The cleaning of slow sand filter is done by :
(*a*) reversing the direction of flow of water
(*b*) passing air through the filter
(*c*) passing a solution of alum and lime through the filter
(*d*) scraping off the top layers of sand and admitting water.

Q. 31. What is the correct sequence of formation of the following compounds during chlorination of water in which ammonia is present ?
1. NCl_3 2. NH_2Cl 3. $NHCl_2$

Select the correct answer using the codes given below :
Codes :
(*a*) 1, 2, 3 (*b*) 2, 3, 1 (*c*) 3, 1, 2 (*d*) 2, 1, 3.

Q. 32. Match List I with List II and select the correct answer using the codes given below the lists :

List I *(Name of impurity in water)*	List II *(Removed by)*
A. Fluorides	1. Activated carbon
B. Manganese	2. Activated alumina
C. Taste and odour	3. Maganese zeolite

Codes	A	B	C
(*a*)	1	2	3
(*b*)	2	3	1
(*c*)	2	1	3
(*d*)	3	2	1

Q. 33. Which one of the following pairs is not correctly matched ?
(*a*) Check valve — To check water flow in all direction
(*b*) Sluice valve — To control flow of water through pipe lines
(*c*) Air valve — To release the accumulated air
(*d*) Scour valve — To remove silt in a pipe line.

Q. 34. Which one of the following would contain water with the maximum amount of turbidity ?
(*a*) Lakes (*b*) Oceans (*c*) Rivers (*d*) Wells.

Year 1997

Q. 35. The following steps are involved in making a spigot and socket joint of cast iron pipes used in water supply systems :
1. Tarred gasket or hemp yarn is wrapped around the spigot.
2. The spigot end is centred into the socket end of the preceding pipe.
3. A jointing ring is placed around the barrel and against the face of the socket.
4. The gasket or hemp yarn is caulked slightly.
5. Molten pig lead is poured and then caulked.

The correct sequence of these steps is
(*a*) 2, 1, 4, 3, 5 (*b*) 2, 1, 3, 4, 5
(*c*) 1, 2, 4, 5, 3 (*d*) 1, 2, 3, 5, 4.

Q. 36. Match List I (Soil classification of an aquifer) with List II (Values of the range of hydraulic conductivities in metre per day) and select the correct answer using the codes given below the Lists :

List I	List II
A. Fine gravel	1. 1 to 10
B. Very fine sand	2. 0.1 to 0.01
C. Silt	3. 100 to 1000
D. Pure clay	4. 10^{-5} to 10^{-6}

Codes	A	B	C	D
(*a*)	1	3	4	2
(*b*)	1	3	2	4
(*c*)	3	1	4	2
(*d*)	3	1	2	4

Q. 37. Which of the following statements about Design Period are true ?
1. It is concerned with economy of investments.
2. It takes into account aspects like life and durability and ease or difficulty of capacity increase of installations.
3. It considers the frequency of occurrence of extremes of river flow.
4. It is concerned with estimating future requirements.

Select the correct answer using the codes given below :
Codes :
(*a*) 1, 2, 3 and 4 (*b*) 2 and 3
(*c*) 1, 2 and 4 (*d*) 1, 3 and 4.

Q. 38. A river is the source of water for water supply to a town. Its water is very turbid and polluted. The correct sequence of steps for treating the river water would be

(*a*) presedimentation → prechlorination → coagulation → sedimentation → filtration → post-chlorination

(*b*) coagulation → sedimentation → post-chlorination

(*c*) coagulation → filtration → sedimentation → post-chlorination

(*d*) sedimentation → post-chlorination.

Q. 39. Uniformity coefficient of filter sand is given by

(*a*) D_{50}/D_5 (*b*) D_{50}/D_{10} (*c*) D_{60}/D_5 (*d*) D_{60}/D_{10}.

Q. 40. Zero hardness of water is achieved by

(*a*) using lime soda process (*b*) excess lime treatment

(*c*) ion exchange method (*d*) using excess alum dosage.

Q. 41. Which of the following statements is/are true in relation to the term 'detention period' in a settling tank ?

1. It may be determined by introducing a dye in the inlet and timing its appearance at the outlet.
2. Greater the detention period, greater the efficiency of removal of settleable matter.
3. It is the time taken for any unit of water to pass through the settling basin.
4. It is usually more than the flow-through period.

Select the correct answer using the codes given below :

Codes :

(*a*) 1, 2, 3 and 4 (*b*) 2, 3 and 4 (*c*) 1 and 3 (*d*) 4 alone.

Q. 42. Match List I with List II and select the correct answer using the codes given below the Lists :

List I	List II
A. Dead-end system	1. Equal pressures and multiple flow paths
B. Grid-iron system	2. Both economy and reasonably equal pressures
C. Ring system	3. Economy and simplicity
D. Radial system	4. Zonal distribution

Codes	A	B	C	D
(*a*)	3	2	1	4
(*b*)	2	4	1	3
(*c*)	3	1	2	4
(*d*)	2	1	4	3.

Q. 43. Match List I (Parameters) with List II (Permissible concentration in drinking water) and select the correct answer using the codes given below the List :

List I	List II
A. Hardness	1. 0.1 mg/l
B. Nitrate concentration	2. 0.5 mg/l
C. Iron concentration	3. 200 mg/l
D. Fluoride concentration	4. 45 mg/l

Codes	A	B	C	D
(*a*)	3	4	2	1
(*b*)	3	4	1	2
(*c*)	4	3	2	1
(*d*)	4	3	1	2.

Q. 44. Match List I with List II and select the correct answer using the codes given below the Lists :

List I	List II
A. Absence of fluorides	1. Methemo-globinaemia
B. Excess of lead	2. Goitre
C. Presence of excess nitrates	3. Dental caries
D. Absence of iodide	4. Anemia

Codes	A	B	C	D
(*a*)	3	4	2	1
(*b*)	2	3	4	1
(*c*)	3	4	1	2
(*d*)	1	2	4	3.

Q. 45. Which of the following treatment(s) will be indicated for a rural water supply from a deep groundwater source ?

1. Sedimentation. 2. Alum dosage.
3. Potassium permanganate dosing.
4. Bleaching powder application.

Select the correct answer using the codes given below :

Codes :

(*a*) 1, 2 and 3 (*b*) 1, 2 and 4 (*c*) 3 and 4 (*d*) 4 alone.

Q. 46. Which of the following determinations are NOT necessary for raw water from a lake for use as source of supply of water for boiler-feed ?

1. Turbidity. 2. Bacterial count.
3. Iron. 4. Hardness.

Select the correct answer using the codes given below :

Codes :

(*a*) 1, 2 and 3 (*b*) 1, 2 and 4 (*c*) 1, 3 and 4 (*d*) 2, 3 and 4.

Year 1998

Q. 47. Two identical centrifugal pumps are operated in parallel so as to deliver into a common delivery pipe. Speed for both is also identical. At what total discharge (Q) and total head (H) will the system operate as compared to discharge and head of each of the pumps operated singly ?

(*a*) Both total Q and total H would increase, each approximately by 50%

(*b*) Total Q would be approximately doubled, but H would remain the same

(*c*) Total H would be approximately doubled, but Q would remain the same

(*d*) Total H would be doubled, but Q would be approximately halved.

Q. 48. Storage of water by impounding is required where
(*a*) plenty of water is available in the stream in all seasons
(*b*) excess of suspended and dissolved matter are present in the water
(*c*) there is a large variation in quantity of the river flow from time to time
(*d*) the flow is uniform throughout the year but is insufficient.

Q. 49. Air-binding in rapid sand filters is encountered when
(*a*) there is excessive negative head
(*b*) the water is subjected to prolonged aeration
(*c*) the raw water contains dissolved gases
(*d*) the filter bed comprises largely of coarse sand

Q. 50. In a water treatment plant, dissolved iron and manganese can be removed from the water by
(*a*) aeration (*b*) aeration and coagulation
(*c*) aeration and flocculation (*d*) aeration and sedimentation.

Q. 51*. Which of the following is/are the characteristic(s) of coliform organism?
1. Bacillus. 2. Gram-negative.
3. Ferments lactose. 4. Spore-forming.
Select the correct answer using the codes given below :
Codes :
(*a*) 1 alone (*b*) 1, 2 and 4 (*c*) 1, 2 and 3 (*d*) 2, 3 and 4.

Q. 52. The various treatment processes in a water treatment plant are listed below :
1. Filtration. 2. Chlorination. 3. Sedimentation.
4. Coagulation. 5. Flocculation.
The correct sequence of these processes in water treatment is
(*a*) 1, 2, 3, 4, 5 (*b*) 4, 5, 3, 1, 2 (*c*) 2, 3, 1, 5, 4 (*d*) 1, 2, 5, 3, 4.

Q. 53. The effective size (ES) of sand and its uniformity coefficient (UC) are the usually specified parameters for sand filters. In slow sand filters, as compared to rapid sand filters.
(*a*) *ES* is less but *UC* is more (*b*) *ES* is more but *UC* is less
(*c*) both *ES* and *UC* are more (*d*) both *ES* and *UC* are less.

Q. 54. For proper slow mixing in the flocculator of a water treatment plant, the temporal mean velocity gradient G needs to be of the order of
(*a*) 5 to 10 s^{-1} (*b*) 20 to 80 s^{-1}
(*c*) 100 to 200 s^{-1} (*d*) 250 to 350 s^{-1}.

Q. 55. If only ammonia was present in water, the only change in the above diagram (Fig. 18.4) would have been that the curve would
(*a*) be a straight line
(*b*) become parallel to *Y*-axis
(*c*) become parallel to *X*-axis after '*D*'
(*d*) be passing through the origin.

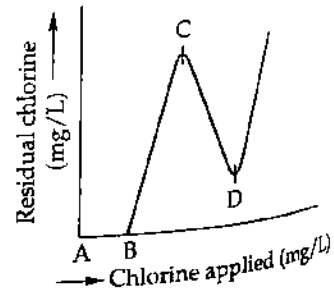


Fig. 18.4

Q. 56. Which one of the following is the purpose of providing a surge tank in a pipeline carrying water ?
(*a*) To store water
(*b*) To increase the pressure throughout the pipeline
(*c*) To store overflowing water
(*d*) To protect the pipeline against water hammer.

Q. 57. Match List I (Water treatment units) with List II (Detention time) and select the correct answer using the codes given below the lists :

List I	List II
A. Rapid mixing unit	1. $1\frac{1}{2}$ hours
B. Flocculator	2. 10 seconds
C. Propeller mixing unit	3. 30 seconds
D. Sedimentation tank	4. 30 minutes

Codes	A	B	C	D
(*a*)	3	4	2	1
(*b*)	4	3	1	2
(*c*)	4	3	2	1
(*d*)	3	4	1	2.

Year 1999

Q. 58. Water present in artesian aquifer is usually :
(*a*) at sub-atmospheric pressure
(*b*) at atmospheric pressure
(*c*) at 0.5 times the atmospheric pressure
(*d*) above the atmospheric pressure.

Q. 59. Consider the following statements :
The daily per capita consumption of water apparently increases with :
1. higher standard of living of people
2. availability of sewerage in the city
3. metered water supply
4. wholesome and potable public supply of water.
Which of the statements are correct ?
(*a*) 1, 2 and 3 (*b*) 2, 3 and 4
(*c*) 1, 3 and 4 (*d*) 1, 2 and 4.

Q. 60. Match List I (Operational problems in rapid gravity filters) with List II (Methods to overcome the problems) and select the correct answer using the codes given below the Lists :

List I	*List II*
A. Incrustation of filter sand media	1. Compressed air scouring for about 4 minutes at the time of backwashing and manual surface raking.
B. Air binding and development of negative head	2. Washing the filter with sodium hydroxide or bleaching powder occasionally.

C. Mud ball formation

3. Thorough back washing with salt solution after soaking the filter in it.

D. Slime growth on filter

4. Increasing the depth of water during filter operation by about 15-20 cm as compared to the normal depth maintained during daily operation and more frequent backwashing.

Codes	A	B	C	D
(a)	2	4	3	1
(b)	4	2	1	3
(c)	2	4	1	3
(d)	4	2	3	1

Q. 61. In the plot of residual chlorine versus chlorine dose applied shown in the figure, the curve will not have any (0, 0) point because

(a) of an experimental error
(b) chlorine escapes into the atmosphere
(c) chlorine requires some contact time
(d) chlorine is consumed for disinfection.

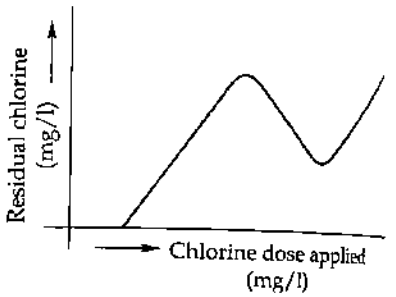


Fig. 18.5

Q. 62.* If the methyl orange alkalinity of water equals or exceeds total hardness, all of the hardness is :

(a) non-carbonate hardness
(b) carbonate hardness
(c) pseudo hardness
(d) negative non-carbonate hardness.

Q. 63. Consider the following statements :

Some amount of chlorides is allowed in drinking water because :

1. it helps in killing bacteria
2. small quantity of chlorides adds to the taste
3. it is not injurious to human health
4. it is not economical to remove it completely.

Which of these statements are correct ?

(a) 1, 2 and 4 (b) 1, 2 and 3
(c) 2, 3 and 4 (d) 1, 3 and 4.

Q. 64. A commonly used hand pump is the :

(a) centrifugal pump (b) reciprocating pump
(c) rotary pump (d) axial flow pump.

Q. 65. Match List I with List II and select the correct answer using the codes given below the lists :

List I (*Pump*)	*List II* (*Requirement*)
A. Multistage centrifugal pump of moderate sized impellers	1. High head and low discharge.
B. Archimedean screw type pump	2. Low head and large discharge
C. Simple reciprocating pump	3. High head and moderate discharge.
D. Axial flow propeller pump	4. Low head and low discharge

Codes	A	B	C	D
(a)	4	3	1	2
(b)	3	4	1	2
(c)	4	3	2	1
(d)	3	4	2	1

Q. 66. Service connection consists of :

(a) ferrule, stop cock and goose neck
(b) ferrule, check valve and goose neck
(c) stop cock, meter and sluice valve
(d) sluice valve, check valve and meter.

Q. 67. The correct sequence of processes in a water treatment plant for rural water supply is :

(a) chlorination, aeration, sedimentation, rapid sand filter
(b) coagulation, sedimentation, slow sand filter, chlorination
(c) coagulation, flocculation, clarification, pressure filter
(d) aeration, plain sedimentation, slow sand filter, chlorination.

Q. 68. Match List I (Industry and unit of production) with List II (water need in kl/unit/day) and select the correct answer by using the codes given below the lists :

List I	List II
A. Automobile (per vehicle)	1. 1 to 2
B. Leather (per 100 kg)	2. 4
C. Paper (per tonne)	3. 40
D. Crude petroleum refinery (per tonne)	4. 50-100
	5. 200-400

Codes	A	B	C	D
(a)	3	2	5	1
(b)	1	3	5	4
(c)	3	2	4	5
(d)	5	4	2	1

Q. 69. Examine the 'Assertion A' and 'Reason R', and decide if the assertion A and the Reason R are individually true, and if so, whether the Reason is a correct explanation of the Assertion ; then mark your choice using the codes given below :

Codes

(*a*) *Both A and R are true and R is the correct explanation of A*

(*b*) *Both A and R are true and R is not a correct explanation of A*

(*c*) *A is true but R is false*

(*d*) *A is false but R is true.*

(*i*) **Assertion A :** At the ferrule point of a water supply distribution pipe network for Indian towns having a majority of two-storey buildings, the ideal minimum pressure head of water is 12 m.

Reason R : The magnitude of pressure head at ferrule is necessary to reach the highest floor of the two-storey buildings.

Year 2000

Q. 70. A rural water supply scheme serves a population of 10,000 at the rate of 50 litres per capita per day. For the chlorine dose of 2 ppm, the required amount of bleaching powder with 20% available chlorine will be :

(*a*) 0.5 kg (*b*) 5 kg (*c*) 10 kg (*d*) 15 kg.

Q. 71. The usual size of residential ferrule bore varies from :

(*a*) 1 mm to 5 mm (*b*) 10 mm to 50 mm

(*c*) 100 mm to 500 mm (*d*) 1000 mm to 5000 mm.

Q. 72. Reciprocating pumps are suitable for :

(*a*) low discharge and high heads (*b*) high discharge and low heads

(*c*) low discharge and low heads (*d*) high discharge and high heads.

Q. 73.* The raw water entering an ideal horizontal settling tank contains the following two types of particles :

Particle type	*Settling velocity (m/h)*	*Concentration (mg/l)*
I	3	200
II	1	300

When the surface of overflow rate of the settling tank is 3 $m^3/m^2/h$, the concentration of the particles in the settled water will be :

(*a*) 100 mg/l (*b*) 200 mg/l

(*c*) 300 mg/l (*d*) 400 mg/l.

Q. 74. Which one of the following filters will produce water of higher bacteriological quality ?

(*a*) Slow sand filter (*b*) Rapid sand filter

(*c*) Pressure filter (*d*) Dual media filter.

Q. 75. Examine Assertion (A) and Reason (R) and select your answer using the codes given below :

(*a*) *Both A and R are true and R is the correct explanation of A*

(*b*) *Both A and R are true but R is not the correct explanation of A*

(*c*) *A is true but R is false*

(*d*) *A is false but R is true.*

(A) **Assertion A :** The future population is predicted on the basis of knowledge of the city and its environment.

Reason R : The future population depends on the trade and expansion of the city, discovery of mineral deposits, power generation etc.

(B) **Assertion A :** The leakage losses are less when the water supply is intermittent.

Reason R : Pressure is less in intermittent water supply.

(C) **Assertion A :** Alum is the most commonly used coagulant in water treatment.

Reason R : Alum is effective in killing pathogens present in water.

Year 2001

Q. 76. Match List I (Unit in water treatment plant) with List II (Impurities removed) and select the correct answer using the codes given below the lists :

List I	*List II*
A. Aerator	1. Excess CO_2 and H_2S
B. Rapid sand filter	2. Settleable and colloidal matter
C. Slow sand filter	3. Suspended matter
D. Sedimentation tank (after coagulation and flocculation)	4. Suspended colloidal and bacteriological matter

Codes	A	B	C	D
(*a*)	1	3	2	4
(*b*)	3	1	2	4
(*c*)	3	1	4	2
(*d*)	1	3	4	2

Q. 77. Which of the following statements are correct ?

1. Ground water is generally free from suspended and dissolved impurities.
2. Suspended matter often contain pathogenic bacteria.
3. Rain water is soft and tasteless.
4. Lake water may contain microscopic organisms.

Select the correct answer using the codes given below :

Codes

(*a*) 1, 2, 3 and 4 (*b*) 1 and 2

(*c*) 2, 3 and 4 (*d*) 1, 3 and 4.

Q. 78. The following sketch (Fig. 18.6) shows a water supply main from a storage reservoir provided with a sluice valve.

The type of valve most suitable for this pipeline is :

(*a*) air valve

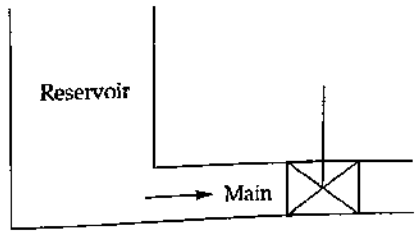


Fig. 18.6

(*b*) scour valve
(*c*) pressure relief valve
(*d*) None of the above.

Q. 79. In the figure shown here, x_2x_4 represents :
(*a*) accumulated supply
(*b*) storage requirement
(*c*) accumulated demand for the period 1 to 2
(*d*) surplus in the reservoir.

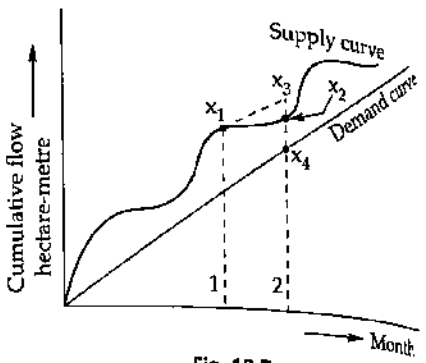


Fig. 18.7

Q. 80. Consider the following impurities
1. CO_2 and H_2S
2. Finely divided suspended matter
3. Disease causing bacteria
4. Excess alkalinity

The correct sequence of the removal of these impurities in a water treatment plant is :
(*a*) 1, 2, 3, 4 (*b*) 1, 4, 3, 2 (*c*) 1, 4, 2, 3 (*d*) 4, 1, 3, 2.

Q. 81. Which of the following are the common problems associated with the operation of rapid sand filter ?
1. Air binding
2. Cracking of sand beds
3. Bumping of filter beds
4. Mud balls

Select the correct answer using the codes given below :
Codes :
(*a*) 1 and 2 (*b*) 2 and 3 (*c*) 2, 3 and 4 (*d*) 1, 2, 3 and 4.

Q. 82. The purpose of providing a balancing reservoir in a water supply distribution system is to
(*a*) equalize pressure in the distribution system
(*b*) store adequate quantity of water to meet requirements in case of breakdown of flow
(*c*) store adequate fire fighting reserve
(*d*) take care of fluctuations in the rate of consumption.

Q. 83. Match List I (Impurities) with List II (Effects) and select the correct answer using the codes given below the lists :

List I (*Impurities*)	*List II* (*Effects*)
A. Dissolved sulphates and chlorides of Ca and Mg	1. Hardness and corrossion
B. Dissolved bicarbonates of Ca and Mg	2. Bacterial infection
C. Dissolved fluorides of Na	3. Alkalinity and softness
D. Dissolved organic matter	4. Impairment of dental health

Codes	A	B	C	D
(*a*)	2	3	4	1
(*b*)	1	4	3	2
(*c*)	2	4	3	1
(*d*)	1	3	4	2

Q. 84. Match List I (Water quality) with List II (Method of determination) and select the correct answer using the codes given below the lists :

List I (*Water quality*)	*List II* (*Method of determination*)
A. Hardness	1. Winkler method
B. Chlorine	2. EDTA method
C. DO	3. Orthotolidine test
D. Chloride	4. Mohr Method

Codes	A	B	C	D
(*a*)	2	3	1	4
(*b*)	2	4	1	3
(*c*)	1	3	2	4
(*d*)	1	4	2	3

Q. 85. The amount of bleaching powder containing 20% available chlorine neaded to chlorinate a rural water supply covering a population of 10000 at 50 lpcd at the rate of 2 ppm is
(*a*) 1 kg (*b*) 5 kg (*c*) 0.2 kg (*d*) 20 kg.

Q. 86. Which of the following statement(s) regarding industrial water supply is/are correct ?
1. Industrial water supplies need not be disinfected.
2. Water for industrial use requires chemical treatment.
3. Standards of purity and methods of treatment of water for industrial use are often different from those for domestic water supplies.
4. It is more economical to use water from surface source than from ground water sources for industrial supplies.

Select the correct answer using the codes given below :
Codes :
(*a*) 1, 2 and 4 (*b*) 2, 3 and 4 (*c*) 3 and 4 (*d*) 3 only.

Year 2002

Q. 87. Match List I (Valves) with List II (Uses) and select the correct answer using the codes given below the lists :

List I (*Valves*)	*List II* (*Uses*)
A. Sluice valve	1. Used where gravity flow is required through pipe line
B. Check valve	2. Used to maintain constant level of water
C. Air inlet valve	3. Used for reversal of flow
D. Ball valve	4. Used for isolating

Codes	A	B	C	D
(a)	1	3	4	2
(b)	4	2	1	3
(c)	1	2	4	3
(d)	4	3	1	2

Q. 88. Consider the following statements :
While deciding to locate an intake structure for a city situated on a river bank, intake for water supply should be located
1. in deep waters.
2. sufficiently away from shore lines.
3. upstream of the populated city.
4. near navigational channel.
Which of these are correct ?
(a) 1, 2 and 4 (b) 1, 2 and 3 (c) 2, 3 and 4 (d) 1, 3 and 4.

Q. 89. The population figures in a growing town are as follows :

Year	*Population*
1970	40,000
1980	46,000
1990	53,000
2000	58,000

The predicted population in 2010 by Arithmetic Regression method is
(a) 62,000 (b) 63,000 (c) 64,000 (d) 65,000.

Year 2003

Q. 90. On which of the following factors, does the population growth in a town normally depends ?
(1) Birth and death rates (2) Migrations
(3) Probabilistic growth (4) Logistic growth
Select the correct answer using the codes given below :
(a) 1 and 4 (b) 1 and 2 (c) 1, 2 and 3 (d) 2 and 4.

Q. 91. Match *List I (Tests)* with *List B (Features)* and select the correct answer using the codes given below the Lists :

List I (Tests)	*List II (Features)*
A. Pumping Test	1. The gradual rise of water level in well is observed as time progresses
B. Recuperation Test	2. Rate of pumping is adjusted to constant level of water in well
C. Pressure Test	3. Vigorous mixing of the chemical followed by slow mixing
D. Jar Test	4. Pipeline is filled up with water, allowed to stand for sometime and then at least double the maximum pressure is applied.

Codes	A	B	C	D
(a)	1	2	3	4
(b)	2	1	4	3
(c)	1	2	4	3
(d)	2	1	3	4

Q. 92. The purpose of re-carbonation after lime-soda process of water softening is the
(a) removal of excess soda from water
(b) removal of non-carbonate hardness
(c) recovery of lime
(d) conversion of precipitates to soluble form.

Q. 93. Which of the following treatment processes are necessary for removing suspended solids from water ?
1. Coagulation 2. Flocculation
3. Sedimentation 4. Disinfection.
Select the correct answer using the codes given below :
(a) 1 and 2 (b) 1, 2 and 3 (c) 2 and 3 (d) 1 and 4.

Q. 94. Match *List I (Filter Operating Problems)* with *List II (Effects)* and select the correct answer using the codes given below the Lists

List I (Filter Operating Problems)	*List II (Effects)*
A. Air binding	1. Changes effective size of sand
B. Mud deposition	2. Mud penetrates deeper inside the bed
C. Cracking of bed	3. Mounds and balls of mud are formed in the bed
D. Sand incrustation	4. Air and gases get locked in the bed.

Codes	A	B	C	D
(a)	4	3	2	1
(b)	3	4	1	2
(c)	4	3	1	2
(d)	3	4	2	1

Q. 95.* Which of the following are removed by rapid sand filter from water ?
1. Dissolved solids 2. Suspended solids
3. Bacteria 4. Helminths
Select the correct answer using the codes given below :
Codes
(a) 1 and 2 (b) 2 and 3
(c) 1 and 3 (d) 2, 3 and 4.

Q. 96. Consider the following valves in a water distribution system :
1. Check valve 2. Pressure reducing valve
3. Air relief valve 4. Scour valve
5. Sluice valve

Which of the following work automatically ?

(a) 1, 3 and 4 (b) 2, 4 and 5
(c) 3, 4 and 5 (d) 1, 2 and 3.

Year 2004

Q. 97. Two long pipes in parallel are used to carry water between two reservoirs. The diameter of one pipe is twice that of the other. Both the pipes have the same value of friction factor. Neglect minor losses, what is the ratio of flow rates through the two pipes ?

(a) 2.8 (b) 5.6
(c) 8 (d) 11.3.

Q. 98. Which one of the following filters should be recommended for protected rural water supply project ?

(a) Pressure filter (b) Slow sand filter
(c) Diatomaceous filter (d) Rapid sand filter.

Q. 99. Match List-I (*Fixture*) with List-II (*Purpose*) and select the correct answer using the codes given below the lists :

List I (*Fixture*)	List II (*Purpose*)
(A) Surge arrester	1. Preventing reversal of flow in a pipe line
(B) Butterfly valve	2. Regulating or stopping the flow especially in large size conduit
(C) Scour valve	3. Control of water hammer
(D) Check valve	4. Drainage or emptying the line section

Codes	A	B	C	D
(a)	1	2	4	3
(b)	3	2	4	1
(c)	1	4	2	3
(d)	3	4	2	1

Q. 100. Consider the following statements :

Aeration in water treatment helps in

1. killing pathogens
2. correcting pH
3. precipitating dissolved iron and manganese
4. expelling excess CO_2 and H_2S.
5. expelling volatile oils.

Which of the statements given above are correct ?

(a) 2, 4 and 5 (b) 3, 4 and 5
(c) 1, 2, 3 and 5 (d) 1, 2, 3 and 4.

Q. 101. The following items consists of two statements, one labelled as the '**Assertion (A)**' and the other as '**Reason (R)**', you are to examine these two statements carefully and select the answer to these items using the codes given below :

Codes :

(a) Both A and R are individually true and R is the correct explanation of A

(b) Both A and R are individually true but R is **not** the correct explanation of A

(c) A is true but R is false

(d) A is false but R is true.

(i) **Assertion (A) :** The yield of a well varied from 10 m^3/day to 20 m^3/day when the aquifer area changed from 50 m^2 to 75 m^2.
Reasons (R) : The yield is found to be directly proportional to the area of an aquifer opening into a well.

(ii) **Assertion (A) :** The fluoride content in drinking water should neither be too low nor too high.
Reason (R) : Deficiency of fluoride content causes mottling of teeth and its excess causes dental cavities in children.

(iii) **Assertion (A) :** The settling velocity of a discrete particle will become five times when its diameter doubles.
Reason (R) : The settling velocity of a discrete particle is almost proportional to be square of the particle diameter.

Q. 102. Match List-I with List-II and select the correct answer using the codes given below the lists :

List I	*List II*
(A) Protozoa	1. Methemoglobinemia
(B) Bacteria	2. Poliomyelitis
(C) Presence of nitrate > 45 mg/l	3. Dysentery
(D) Virus	4. Typhoid fever.

Codes	A	B	C	D
(a)	3	2	1	4
(b)	1	4	3	2
(c)	3	4	1	2
(d)	1	2	3	4

Q. 103. *Which one of the following organism is responsible for enteric fever ?

(a) ECHO (b) Salmonella typhi
(c) Entamoeba histolytica (d) Echinococcus.

Q. 104. Which one of the following statements is correct ? If the specific gravity of a suspended particle is increased from 2 to 3, the settling velocity will

(a) not change (b) get doubled
(c) get increased by 1.5 times (d) get increased by 2.25 times.

Q. 105. Which one of the following is **not** a specific criterion for calculating surface overflow rate in sedimentation tank design ?

(a) Total quantity of water to be treated
(b) Total surface area available in the tank
(c) Total length of the tank
(d) Total depth of the tank.

Q. 106. Which of the following remedial measures are taken to avoid negative head and airbinding in a rapid sand filter ?

1. Avoiding the occurrence of excessive negative head
2. Pumping in air
3. Avoiding increase in water temperature
4. Control of algae growth

Select the correct answer using the codes given below :

Codes

(*a*) 1, 3 and 4 (*b*) 2, 3 and 4 (*c*) 1 and 2 (*d*) 1, 2, 3 and 4.

Q. 107. Match List-I (*Water/Waste Water Parameter*) with List-II (*Test*) and select the correct answer using the codes given below the lists :

List-I (*Water/Waste Water Parameter*)	List-II (*Test*)
(A) Potability of water	1. Mohr's method
(B) Chloride	2. Orthotolidine method
(C) Residual chlorine	3. EDTA method
(D) Hardness of water	4. M.F. technique

Codes	A	B	C	D
(*a*)	4	3	2	1
(*b*)	2	1	4	3
(*c*)	2	3	4	1
(*d*)	4	1	2	3

Q. 108. Consider the following statements :

The basic difference between water pipes and sewer pipes is

1. in the material used for the pipes
2. in the pressure of the liquid flow
3. in the suspended solids they carry

Which of the statements given above is/are correct ?

(*a*) 1 and 3 (*b*) 1 only
(*c*) 2 and 3 (*d*) 1, 2 and 3.

Q. 109. Match List-I (*Type of pipe*) with List-II (*Purpose*) and select the correct answer using the codes given below the Lists :

List-I (*Type of Pipe*)	List-II (*Purpose*)
(A) Steel pipe	1. House plumbing
(B) Cast iron pipe	2. Hot water carrying
(C) G.I. pipe	3. Distribution main
(D) PVC pipe	4. Pumping main

Codes	A	B	C	D
(*a*)	4	1	2	3
(*b*)	4	3	2	1
(*c*)	2	1	4	3
(*d*)	2	3	4	1

Year 2005 (*Engineering Services*)

Q. 110. *Directions :* The following item consists of two statements, one labelled as the 'Assertion (A)' and the other as 'Reason (R)'. You are to examine these two statements carefully and select the answer to the item using the codes given below :

(*a*) Both A and R are individually true and R is the correct explanation of A

(*b*) Both A and R are individually true but R is *not* the correct explanation of A

(*c*) A is true but R is false

(*d*) A is false but R is true

(i) Assertion (A) : A small quantity of ammonia is added to water before carrying out disinfection using chlorine.

Reason (R) : Chloramines are persistent disinfectants, which provide continued protection against regrowth of micro-organisms in a water distribution system.

Q. 111. Match List I (*Type of Impurity*) with List II (*Effect*) and select the correct answer using the code given below the Lists :

List I	*List II*
A. Carbonates and bicarbonates of Ca and Mg	1. Permanent hardness
B. Carbonates and bicarbonates of sodium	2. Temporary hardness
C. Sulphates and chlorides of Ca and Mg	3. Alkanity and softness
D. Oxides of iron and manganese	4. Colour and taste

	A	B	C	D
(*a*)	1	3	2	4
(*b*)	2	4	1	3
(*c*)	1	4	2	3
(*d*)	2	3	1	4

Q. 112. Match List I (*Equation/Law*) with List II (*Related Application*) and select the correct answer using the code given below the Lists :

List I	*List II*
A. Chick's law	1. Discrete particle settling
B. Darcy-Weisbach equation	2. Head loss in a pipe
C. Stoke's equation	3. Head loss in filters
D. Carmen-Kozeny equation	4. Rate of bacterial kill

	A	B	C	D
(*a*)	4	2	1	3
(*b*)	3	1	2	4
(*c*)	4	1	2	3
(*d*)	3	2	1	4

Q. 113. A flash mixer of 2.0 m^3, with a velocity gradient of mixing mechanism equal to 600/s, and fluid absolute viscosity of 1.0×10^{-3} Ns/m^2 is continuously operated. What is the power input per unit volume ?
(*a*) 360 W (*b*) 720 W (*c*) 1440 W (*d*) 300 W.

Q. 114. Consider the following statements :
In water supply distribution network,
1. The grid-iron system requires more length of pipe lines and larger number of cut-off valves.
2. The design of the grid-iron system is difficult but economical.
3. By employing a grid-iron system, the dead ends are completely eliminated.
4. Employing a grid-iron system permits more water to be diverted towards the affected point from various directions.

Which of the statements given above are correct ?
(*a*) 1, 2 and 4 (*b*) 1, 3 and 4 (*c*) 2, 3 and 4 (*d*) 1, 2 and 3.

Q. 115. A high efficiency pump is required for low discharge, high head and low maintenance cost. Delivery of water need not be continuous. The pump need not run at high speed. Which one of the following is the correct choice ?
(*a*) Centrifugal pump (*b*) Reciprocating pump
(*c*) Air lift pump (*d*) Hydraulic ram.

Q. 116. Match List I (*Treatment Process*) with List II (*Removed Matter*) and select the correct answer using the code given below the Lists :

List I	*List II*
A. Plain Sedimentation	1. Dissolved gases
B. Chemical Precipitation	2. Dissolved solids
C. Slow Sand Filtration	3. Suspended solids with specific gravity more than 1.0
D. Aeration	4. Floating solids
	5. Bacterial cells

	A	B	C	D
(*a*)	5	1	4	2
(*b*)	3	2	5	1
(*c*)	5	2	4	1
(*d*)	3	1	5	2

Q. 117. A sample of ground water at a pH of 7.0 contains 122 mg/l of bicarbonates. What is the alkalinity of this water (in terms of $CaCO_3$) ?
(*a*) 120 mg/l (*b*) 60 mg/l (*c*) 100 mg/l (*d*) 200 mg/l.

Q. 118. Consider the following statements :
1. Infiltration galleries are placed along the river beds at a depth of 4 to 6 m.
2. The draw-down for the infiltration galleries is more than that for radial wells.
3. Clogging of pipe pores in infiltration galleries is less than that for radial wells.
4. The cost of extracting unit volume of water is more in case of infiltration galleries as compared to radial wells.

Which of the statements given above is/are correct ?
(*a*) 1 and 2 (*b*) 1, 2 and 3
(*c*) 2, 3 and 4 (*d*) 4 only.

Q. 119. Among the following, which is/are not pre-treatment unit(s) ?
(*a*) Bar screen and grit chamber
(*b*) Flow equalization and proportioning tank
(*c*) Neutralization for pH adjustment tank
(*d*) Nutrient removal tank.

Q. 120. According to the theory of filtration in water treatment, which of the following mechanisms come into play when water is filtered through a filter bed ?
1. Mechanical straining 2. Capillary action
3. Centrifugal force 4. Electro kinetic phenomenon
5. Osmotic force 6. Bacteriological action

Select the correct answer using the code given below :
(*a*) 1, 2, 4 and 6 (*b*) 2, 3 and 5
(*c*) 3, 4, 5 and 6 (*d*) 1, 3, 4 and 6.

Q. 121. Which of the following are associated with alum coagulation ?
1. A decrease of alkalinity in treated water
2. Formation of hydroxide flocs of aluminium
3. A slight decrease of pH in treated water
4. An increase of permanent hardness

Select the correct answer using the code given below :
(*a*) 1, 2 and 3 (*b*) 1, 3 and 4
(*c*) 1, 2, 3 and 4 (*d*) 2 and 4.

Year 2006

Q. 122. Which one of the following statements is ***not*** correct ?
(*a*) Solution of pipe network by Hardy Cross method is a trial and error solution
(*b*) At a junction of pipes, total inflow is equal to the total outflow
(*c*) Loss of head due to flow in a clockwise direction should be equal to loss of head in flow in a counter clockwise direction
(*d*) Hardy Cross method can be extended to open channel flow also.

Q. 123*. Water flows into junction J from reservoirs A and B through connecting pipes, the head loss through these being, respectively $10.Q_A^2$ and $4.Q_B^2$. The water level elevations at the reservoirs at A and B are 25.9 m and 18 m, respectively. The inflow at J is discharged out at C into the atmosphere. The head loss through pipe JC is $1.Q_C^2$. The gauge

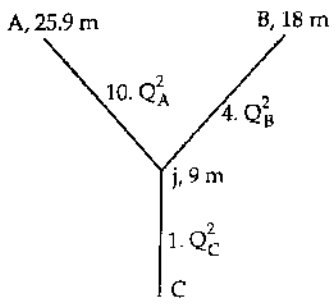


Fig. 18.8

pressure at J is 9 m. What is the residual gauge pressure of the outflow at C ?

(a) 2.56 m (b) 1.86 m
(c) 1.16 m (d) 0.46 m

Q. 124. Which one of the following inferences is ***not*** drawn by studying performance curves of centrifugal pumps ?

(a) Discharge increases with speed
(b) Power decreases with speed
(c) Head increases with speed
(d) Manometric head decreases with discharge.

Q. 125. The centrifugal pump should be installed above the water level in the sump such that

(a) Its height is not more than 10.3 m at room temperature of liquid
(b) Its height is not allowed to exceed 6.7 m
(c) The negative pressure does not reach as low as the vapour pressure
(d) The negative pressure is not allowed to develop in the impeller.

Q. 126. Which one of the following statements is correct ?

(a) Pumps operating in series boost the discharge ; whereas, pumps operating in parallel boost the head
(b) Pumps operating in parallel boost the discharge ; whereas, pumps operating in series boost the head
(c) In both the above cases, there would be a boost in discharge only
(d) In both the above cases, there would be a boost in head only.

Q. 127. A standard multiple-tube fermentation test was conducted on a sample of water. The results of the analysis for the confirmed test are given below :

Sample Size (ml)	*No. of positive results out of 5 tubes*	*No. of negative results out of 5 tubes*
10	4	1
1	2	3
0.1	1	4
0.01	0	5

MPN Index for combination of positive results when 5 tubes used per dilutions (10 ml, 1.0 ml, 0.1 ml)

Combination of positives	*MPN Index per 100 ml*	*Combination of positives*	*MPN Index per 100 ml*
5-4-3	280	4-2-1	26
4-3-1	33	2-1-0	7

Using the above MPN Index table, what is the most probable number (MPN) of the sample ?

(a) 280 (b) 33
(c) 26 (d) 70

Q. 128. Chlorides from water are removed by

(a) Lime soda process (b) Reverse osmosis
(c) Cation exchange process (d) Chemical coagulation.

Q. 129. Chlorination with Cl_2 produces hypochlorous acid (HOCl), which may further dissociate as hypochlorite ion (OCl^-) depending upon pH of the water. The reaction is represented as

$$HOCl \Leftrightarrow H^+ + OCl^- \ (k = 2.5 \times 10^{-8} \text{ moles/l at } 20°C).$$

What is the fraction of HOCl in the water at pH 7.0 ?

(a) 0.95 (b) 0.80
(c) 0.20 (d) 0.05.

Q. 130. In which treatment unit is "Schmutz decke" formed ?

(a) Sedimentation tank (b) Rapid sand filter
(c) Coagulation tank (d) Slow sand filter.

Q. 131. Which one of the following chemicals is employed for dechlorination of water ?

(a) Sodium sulphite (b) Sodium bicarbonate
(c) Calcium carbonate (d) Hydrogen peroxide.

Q. 132*. Which one of the following is the correct sequence of slow sand filter (SSF), rapid sand filter (RSF), dual media filter (DMF) and mixed media filter (MMF) in the decreasing order of their filtration rates ?

(a) MMF = DMF > RSF > SSF (b) DMF > RSF > SSF > MMF
(c) RSF > SSF > MMF » DMF (d) SSF > MMF » DMF > RSF.

Q. 133. After which of the following water treatment units, the turbidity is maximum ?

(a) Chlorination (b) Primary sedimentation
(c) Flocculation basin (d) Secondary sedimentation.

Year 2007

Q. 134. Match List I with List II and select the correct answer using the code given below the lists :

List I (Pathogen)	*List II (Epidemic)*
A. Bacteria	1. Gastroenteritis
B. Virus	2. Cholera
C. Protozoa	3. Worms
D. Helminth	4. Polio

Code :	A	B	C	D
(a)	2	4	1	3
(b)	3	1	4	2
(c)	2	1	4	3
(d)	3	4	1	2

Q. 135. Match List I with List II and select the correct answer using the code given below the lists :

List I (Equation / Method)	*List II (Application)*
A. Manning's Equation	1. Frictional head loss estimation in pipe flow
B. Darcy-Weisbach Equation	2. Sanitary sewer design
C. Hardy Cross Method	3. Storm sewer design
D. Rational Method	4. Water distribution system design

Code :	A	B	C	D
(*a*)	2	1	4	3
(*b*)	1	4	3	2
(*c*)	2	1	3	4
(*d*)	1	4	2	3

Q. 136. What is the predominating coagulation mechanism for raw water having high turbidity and high alkalinity ?

(*a*) Ionic layer compression
(*b*) Adsorption and charge neutralization
(*c*) Sweep coagulation
(*d*) Inter particle bridging

Q. 137. Match List I with List II and select the correct answer using the code given below the lists :

List I (Disinfectant)	*List II (Property)*
A. Chlorine	1. No carcinogenics result
B. Ozone	2. Ineffective in the presence of suspended solids
C. Iodine	3. Not affected by the Ammonium ion
D. Ultra-violet rays	4. Feasible residual content

Code :	A	B	C	D
(*a*)	4	3	1	2
(*b*)	1	2	4	3
(*c*)	4	2	1	3
(*d*)	1	3	4	2

Q. 138. Which one of the following treatments is economically effective in the control of guinea worm disease ?

(*a*) Chlorination (*b*) Filtration
(*c*) Ozonation (*d*) Sedimentation

Q. 139. The concentration of hardness producing cations may be estimated using which one of the following ?

(*a*) Conductivity meter (*b*) pH meter
(*c*) Spectrophotometer (*d*) Flame photometer

Q. 140. Match List I with List II and select the correct answer using the code given below the lists :

List I (Parameter)	*List II (Impact)*
A. Excess sulphates	1. Greater soap consumption
B. Lack of iodide	2. Laxative effect
C. Excess hardness	3. Goitre
D. Excess dissolved oxygen	4. Corrosion of pipes

Code :	A	B	C	D
(*a*)	2	1	3	4
(*b*)	4	3	1	2
(*c*)	2	3	1	4
(*d*)	4	1	3	2

Q. 141. Directions :

The item given below consist of two statements, one labelled as the 'Assertion (A)' and the other as 'Reason (R)'. You are to examine these two statements carefully and select the answer to the item using the codes given below :

Code :

(*a*) Both A and R are individually true and R is the correct explanation of A
(*b*) Both A and R are individually true but R is not the correct explanation of A
(*c*) A is true but R is false
(*d*) A is false but R is true

Assertion (A) : A discrete particle (of diameter d_0) settling in a circular sedimentation tank follows a parabolic path.

Reason (R) : The downward settling velocity (v_0) of the discrete particle (of diameter d_0) in a circular sedimentation tank does not change with time.

Q. 142.* A penstock is 2000 m long and the velocity of pressure wave in it is 1000 m/s. Water hammer pressure head for instantaneous closure of valve at the downstream end of the pipe is 60 m. If the valve is closed in 4 s, then what is the peak water hammer pressure in *m* of water ?

(*a*) 15 (*b*) 30
(*c*) 60 (*d*) 120

Year 2008

Q. 143. For water supply to a medium town, what is the daily variation factor ?

(*a*) 1.5 (*b*) 2.5
(*c*) 3 (*d*) 3.5

Q. 144. Which one of the following factors has the maximum effect on figure of per capita demand of water supply of a given town ?

(*a*) Method of charging of the consumption
(*b*) Quality of water
(*c*) System of supply-intermittent or continuous
(*d*) Industrial demand

Q. 145*. Match List-I with List-II and select the correct answer using the code given below the lists :

List I (*Predominance of Compounds*)	*List II* (*pH range*)
A. Monochloramine	1. Below ph 4.4
B. Dichloramine	2. Over pH 7.5
C. Nitrogen trichloride	3. Between pH 5 to 6.5

Code :	A	B	C
(*a*)	1	2	3
(*b*)	2	3	1
(*c*)	3	1	2
(*d*)	3	2	1

Q. 146. Which of the following are the advantages of cast-iron pipe for its use in water supply ?

1. Resistant to corrosion to a reasonable extent
2. Very easy to join the pipes
3. Easy to transport
4. Longer life

Select the correct answer using the code given below :

(*a*) 1, 3 and 4 (*b*) 1, 2 and 3 (*c*) 1 and 4 only (*d*) 2, 3 and 4

Q. 147*. Which one of the following valves is seldom used in water distribution systems because of high head loss characteristics ?

(*a*) Butterfly (*b*) Globe (*c*) Plug (*d*) Sluice

Q. 148. Sonoscope is used for which one of the following ?

(*a*) Checking the accuracy of water meters
(*b*) Regulating the fire hydrants
(*c*) As a replacement of venturimeter for discharge measurement
(*d*) Detection of leakage in underground water mains

Year 2009

Q. 149. What is the most important design parameter used in designing a continuous flow rectangular sedimentation tank for removal of discrete particles ?

(*a*) Length of the tank
(*b*) Surface overflow rate
(*c*) Depth of the tank
(*d*) Temperature of the water to be treated

Q. 150. Match List-I with List-II and select the correct answer using the code given below the Lists :

List I (*Type of impurity*)	List II (*Harm caused*)
A. Excess of nitrates	1. Brackish water
B. Excess of fluorides	2. Goiter
C. Lack of iodides	3. Fragile bones
D. Excess of chlorides	4. Blue babies

Code :	A	B	C	D
(*a*)	4	2	3	1
(*b*)	1	2	3	4
(*c*)	4	3	2	1
(*d*)	1	3	2	4

Q. 151*. The concentration of OH^- ion in a water sample is measured as 17 mg/L at 25°C. What is the pH of the water sample ?

(*a*) 10 (*b*) 11 (*c*) 12 (*d*) 13

Q. 152*. Which combination of surface water quality parameters will indicate *sweep coagulation* as the preferred mechanism of coagulation ?

(*a*) High turbidity—low alkalinity (*b*) High turbidity—high alkalinity
(*c*) Low turbidity—high alkalinity (*d*) Low turbidity—low alkalinity

Q. 153. Which one of the following processes of water softening requires recarbonation ?

(*a*) Lime-soda ash process
(*b*) Hydrogen-cation exchanger process
(*c*) Sodium-cation exchanger process
(*d*) Demineralization.

Q. 154. Consider the following treatment process units in a water treatment plant:

1. Coagulation 2. Disinfection
3. Sedimentation 4. Filtration

Which is the correct sequence of the process units in the water treatment plant ?

(*a*) 2–4–3–1 (*b*) 1–4–3–2 (*c*) 2–3–4–1 (*d*) 1–3–4–2

Q. 155. Which one of the following tests of water/wastewater employs Erichrome Black T as an indicator ?

(*a*) Hardness (*b*) COD
(*c*) Residual chlorine (*d*) DO.

Year 2010

Q. 156. Two identical pumps, each capable of delivering 0.2 cumec, against a head of 30 m are connected in parallel. The resulting discharge is

(*a*) 0.4 cumec against a head of 30 m
(*b*) 0.4 cumec against a head of 60 m
(*c*) 0.2 cumec against a head of 30 m
(*d*) 0.2 cumec against a head of 60 m

Q. 157. If organic sources of carcinogenic compounds in water persist even after chlorination, then what is the correct sequence among treatment processes listed below. If all these are considered compulsory?

1. Coagulation 2. Sedimentation
3. Filtration in general 4. Activated carbon bed filtration
5. Flocculation 6. Chlorination

(*a*) 4–5–3–2–6 and 1 (*b*) 1–2–3–4–5 and 6
(*c*) 4–2–3–1–5 and 6 (*d*) 1–5–2–3–4 and 6

Q. 158. Match List-I with List-II and select the correct answer using the codes given below the Lists :

List I (*Parameters*)	List II (*Units*)
A. Turbidity	1. TON
B. Pathogen	2. TCU
C. Odour	3. JTU
D. Colour	4. MPN

Codes	A	B	C	D
(*a*)	2	1	4	3
(*b*)	3	1	4	2
(*c*)	2	4	1	3
(*d*)	3	4	1	2

Q. 159. Match List-I with List-II and select the correct answer using the codes given below the Lists :

List I	List II
A. Viruses in water	1. Parasite-based diseases
B. Depletion of oxygen	2. Fish extinction
C. Excess nitrates in water	3. Methemeglobinemia
D. Excess fluorides in water	4. Mottling of teeth

Codes	A	B	C	D
(*a*)	1	2	3	4
(*b*)	4	2	3	1
(*c*)	1	3	2	4
(*d*)	4	3	2	1

Q. 160. The role of gravel bed in a rapid send filter is :

1. To filter out large suspended matter
2. To support the sand bed above it
3. To prevent the escape of sand particles
4. To uniformly distribute the backwash water
5. To prevent algae growth

Which of these statements are correct ?

(*a*) 1, 2, 3, 4 and 5 (*b*) 2, 3 and 4 only
(*c*) 3, 4 and 5 only (*d*) 1, 2 and 3 only

Q. 161. Consider the following statements :

The following factors relate to the process of coagulation :

1. Percentage removal is higher when turbidity is more
2. Addition of activated silica aids in the process of coagulation
3. pH of water is an important consideration for selecting a coagulant.

Which of these statements are correct ?

(*a*) 1, 2, and 3 (*b*) 1 and 2 only
(*c*) 2 and 3 only (*d*) 1 and 3 only

Q. 162.* Which of the following methods are employed for determination of free and combined chlorine residuals in water ?

1. Starch-iodide test
2. Orthotolidine test
3. Amperometric titration method
4. SNORT method
5. DPD method

Which of these are correct ?

(*a*) 1, 2, 4 and 5 (*b*) 1, 2 and 3 only
(*c*) 3, 4 and 5 only (*d*) 2, 3 and 4 only

Q. 163. Match List-I with List-II and select the correct answer using the codes given below the Lists :

List I	List II
A. Primary sedimentation	1. Differential settling
B. Coagulation	2. Hindered settling
C. Flocculation	3. Charge neutralization
D. Secondary treatment	4. Growth of flocs
	5. Flow through velocity

Codes	A	B	C	D
(*a*)	2	5	4	1
(*b*)	1	3	4	2
(*c*)	2	4	3	1
(*d*)	5	4	3	2

Q. 164. Which of the following operational problems relate to the functioning of rapid gravity filter ?

1. Inadequate media comprising filter bed
2. Sludge bulking
3. Mud balls
4. Negative head
5. Incrustation of media

(*a*) 1, 3, 4 and 5 only (*b*) 1, 2, 3 and 4 only
(*c*) 2, 3, 4 and 5 only (*d*) 1, 2, 3, 4 and 5 only

Q. 165. Conversion of dynamic velocity head into static pressure head in a centrifugal pump is the result of :

(*a*) Increasing area of flow between adjacent vanes from inlet to outlet
(*b*) Difference in pressure between suction and delivery ends
(*c*) Radial thrust in pumps
(*d*) Stuffing Box.

Q. 166. Match List I and List II and select the correct answer using the codes given below in the lists :

List I	List II
A. Steel pipe	1. Highly resistant to corrosion but can break easily
B. Concrete pipe	2. Virtually corrosion resistant
C. AC pipe	3. Sulphide corrosion
D. Vitrified clay pipe	4. Electrolyte corrosion

Codes	A	B	C	D
(*a*)	2	3	1	4
(*b*)	4	3	1	2
(*c*)	2	1	3	4
(*d*)	4	1	3	2

Q. 167. Which one of the following tests employs ferroin indicator ?
(*a*) Chemical oxygen demand (*b*) Ammonia nitrogen
(*c*) Nitrate nitrogen (*d*) Fluoride.

Q. 168.* In a pressure penstock 4500 m long, water is flowing at a velocity of 4 m/s. If the velocity of the pressure wave travelling in the pipe, due to sudden complete closure of a valve at the down stream end, is given as 1500 m/s, what would be the period of oscillation in seconds under frictionless conditions ?
(*a*) 6 (*b*) 8 (*c*) 9 (*d*) 11

Directions : *Each of the next six items consists of two statements, one labelled as the 'Assertion (A)' and the other as 'Reason (R)'. You are to examine these two statements carefully and select the answer to these items using the codes given below :*

Codes :
(*a*) *Both A and R are individually true and R is the correct explanation of A*
(*b*) *Both A and R are individually true but R is not the correct explanation of A*
(*c*) *A is true but R is false*
(*d*) *A is false but R is true.*

Q. 169. ***Assertion (A) :*** Loss of head at sudden contraction in a pipe is smaller than the loss at a sudden expansion in the pipe.
Reason (R) : Increase in turbulence level is higher at sudden expansion than at a sudden contraction.

Q. 170. ***Assertion (A) :*** The efficiency of a reciprocating pump is 10-20 percent higher than that of a centrifugal pump for comparable head conditions.
Reason (R) : The discharge from a reciprocating pump is dependent upon speed.

Q. 171. ***Assertion (A) :*** In centrifugal pumps, flow takes place from low pressure zone to high pressure zone.
Reason (R) : Possibility of separation occurring in pumps can be more; and characterizing efficiency of pumps is less than that of turbines.

Q. 172. ***Assertion (A) :*** Fluorides should always be present in drinking water up to a value of 1.5 mg/l.
Reason (R) : Such a water helps clean the teeth well.

Q. 173. ***Assertion (A) :*** The bottom layers of water in a deep reservoir are usually not acceptable as raw water in a water supply scheme.
Reason (R) : The bottom water layers may contain products of anaerobic degradation.

Q. 174. ***Assertion (A) :*** Disinfection is the last treatment given to water before it is supplied.
Reason (R) : Any other treatment after disinfection may accidently also contaminate the water.

Year 2011

Q. 175. Consider the following statements :
1. Drain valves are provided at elevated or higher points to remove accumulated air.
2. Reflux valve allows flow in one direction only.
3. Drain valves are provided as low points to remove silt and other deposits.

Which of the above statements is/are correct ?
(*a*) 1, 2 and 3 (*b*) 2 only (*c*) 2 and 3 only (*d*) 3 only

Q. 176. Consider the following statements :
The disadvantage of employing steel pipes in conveyance and distribution of water are :
They
1. cannot withstand high negative pressures or vacuums that may be created in them, especially the combined effect of vacuum and external loads of backfill and traffic
2. are easily affected by acidic and alkaline waters and even atmospheric agencies may produce adverse effects on them.
3. cannot be used for high pressures (generally not used for pressure above 7 kg/cm^2).

Which of the above statement is/are correct ?
(*a*) 1, 2 and 3 only (*b*) 1 and 2 only
(*c*) 2 and 3 only (*d*) 1 only

Q. 177.* Consider the following statements :
In reciprocating pumps, the air vessels are used
1. to get continuous supply of liquid at a uniform rate
2. to save the power required to drive the pump
3. to run at much higher speed without any danger of separation.

Which of the above statements are correct ?
(*a*) 1 and 2 only (*b*) 2 and 3 only
(*c*) 1, 2 and 3 (*d*) 2 and 3 only

Directions : *Each of the two items given below, consists of two statements, one labelled as the 'Assertion (A)' and the other as 'Reason (R)'. You are to examine these two statements carefully and select the answers to the item using the codes given below :*

Codes :
(*a*) *Both A and R are individually true and R is the correct explanation of A*
(*b*) *Both A and R are individually true but R is not the correct explanation of A*
(*c*) *A is true but R is false*
(*d*) *A is false but R is true*

Q. 178. ***Assertion (A) :*** In a strainer type tube well, strainer pipes are surrounded by wire mesh.
Reason (R) : This prevents the fine particles from entering the well pipe.

Q. 179. ***Assertion (A) :*** Break point chlorination ensures a residual of free available chlorine.
Reason (R) : A super high chlorine dose inactivates the pathogens in a very short time.

Q. 180. In a pipe network of municipal water supply, a parallel pipe is sometimes installed over a portion of the pipe mainly for :
(*a*) reducing water hammer pressure
(*b*) decreasing the pumping power need
(*c*) increasing the head available at the node
(*d*) increasing the discharge.

Q. 180.* An unconfined aquifer of porosity 30% and permeability 35 m/day, specific yield of 0.20 has an area of 100 km^2. If the water table falls by 0.25 m during a drought, the volume of water lost from storage in million cubic metres, is
(*a*) 2.0 (*b*) 5.0 (*c*) 1.0 (*d*) 4.0

Q. 182.* In a water treatment, the optimum time of flocculation is usually given as 30 minutes. In case the time of flocculation is increased beyond this value, then the flocs will
(*a*) become heavy and settle down in flocculation itself
(*b*) entrap air and will float in the sedimentation tank
(*c*) break up and defeat the purpose of flocculation
(*d*) stick to the paddles.

Q. 183.* Consider the following statements :
In water treatment, the addition of chlorine inactivates the cells of pathogenic bacteria through
1. Penetration of the chlorine species through the cells walls
2. Suffocation of the cells
3. Reaction of the chlorine species with the enzyme system of the cells
Which of the above statement are correct ?
(*a*) 1, 2 and 3 (*b*) 1 and 2 only
(*c*) 2 and 3 only (*d*) 1 and 3 only

Q. 184. Consider the following statements :
1. Pump in series operation cause the head to increase
2. Pumps in series operation increase the flow rate
3. Pumping in parallel operation increase the flow rate
4. Pumps in parallel operation cause the head to increase.
Which of the above statement are correct ?
(*a*) 1 and 3 (*b*) 1 and 2
(*c*) 2 and 3 (*d*) 3 and 4

Q. 185. Consider the following statements :
The appropriate method(s) for removal of fluorides from water comprises :
1. Addition of alum and lime followed by clarification.
2. Passing through beds of activated alumina.

Which of the above statement is/are correct ?
(*a*) Neither 1 nor 2 (*b*) Both 1 and 2
(*c*) 1 only (*d*) 2 only

Q. 186. The correct sequence of treatment of typical turbid surface water is :
(*a*) Flocculation, Coagulation, Sedimentation, Filtration
(*b*) Flocculation, Coagulation, Filtration, Sedimentation
(*c*) Coagulation, Flocculation, Filtration, Sedimentation
(*d*) Coagulation, Flocculation, Sedimentation, Filtration.

Year 2012

Q. 187. EDTA titration method of hardness determination of water sample uses an indicator which combines with hardness-causing divalent cations and forms a coloured complex. The name of the indicator and the colour of the formed complex respectively are
(*a*) Ferroin and dark blue
(*b*) Ferroin and wine red
(*c*) Eriochrome Black T and dark blue
(*d*) Eriochrome Black T and wine red

Q. 188. Consider the following statements :
1. Carbonate hardness is due to bicarbonates.
2. Non-carbonate hardness is due to sulphates and chlorides of Ca and Mg.
3. Both the hardnesses can be removed by lime-soda method.
4. Both the hardnesses can be removed by ion-exchange method.
Which of these statements are correct ?
(*a*) 1, 2 and 3 only (*b*) 1,2 and 4 only
(*c*) 2, 3 and 4 only (*d*) 1, 2, 3 and 4.

Q. 189. If the velocity of flow as well as the diameter of the flowing pipe are respectively doubled through a pipe system in use since long, the head loss will thereafter be
(*a*) Halved (*b*) Doubled
(*c*) Increased 4 times (*d*) No change

Q. 190. Consider the following statements :
The total head against which a pump has to work must include, besides any other items,
1. the suction lift.
2. the delivery head.
3. the head lost due to friction at entrance in the rising main.
4. the head lost due to friction at exit in the rising main.
Which of these statements are correct ?
(*a*) 1, 2 and 3 only (*b*) 2 and 3 only
(*c*) 1, 2, 3 and 4 (*d*) 3 and 4 only.

Q. 191. The most common constituents of alkalinity in natural water are measured by titrating the water sample with 0.02 N H_2SO_4 using

(a) Eriochrome Black T and Ferroin indicators
(b) Ferroin and Phenolphthalein indicators
(c) Phenolphthalein and Methyl Orange indicators
(d) methyl Orange and Eriochrome Black T indicators

Q. 192. An approximate estimation of total dissolved solids of a given water sample is often made by measuring
(a) Electrical conductivity of the water sample
(b) Electro-magnetic conductivity of the water sample
(c) Sound conductivity of the water sample
(d) Thermal conductivity of the water sample

Q. 193. One nephelometry turbidity unit (NTU) is equal to the turbidity produced by
(a) 1 mg SiO_2 dissolved in 1 *l* of distilled water with the test being run according to absorption principle
(b) 1 mg SiO_2 dissolved in 1 *l* of distilled water with the test being run according to scattering principle
(c) 1 mg Formazin dissolved in 1 *l* of distilled water with the test being run according to absorption principle
(d) 1 mg Formazin dissolved in 1 *l* of distilled water with the test being run according to scattering principle.

Q. 194. Which one of the following tests employs Ethylene Diamine Tetra Acetic Acid as a titrating agent ?
(a) Chlorides (b) Dissolved oxygen
(c) Hardness (d) Residual chlorine

Q. 195. Which of the following reasons are responsible for adoption of post-chlorination of water ?
1. Chlorine demand is reduced.
2. Possibility of taste and odour formation is reduced.
3. Possibility of carcinogenic compounds is reduced.
4. Chloramines are formed.

(a) 1, 2, 3 and 4 (b) 1, 2 and 3 only
(c) 1 and 4 only (d) 2, 3 and 4 only

Directions : *Each of the next given five items consists of two statements, one labelled as the 'Statement (I)' and the other as 'Statement (II)'. You are to examine these two statements carefully and select the answers to these items using the codes given below :*

Codes :

(a) *Both Statement (I) and Statement (II) are individually true and Statement (II) is the correct explanation of Statement (I)*
(b) *Both Statement (I) and Statement (II) are individually true but Statement (II) is* **not** *the correct explanation of Statement (I)*
(c) *Statement (I) is true but Statement (II) is false.*
(d) *Statement (I) is false but Statement (II) is true.*

Q. 196. ***Statement (I)*** **:** Hypochlorites are added to kill pathogens as a disinfection process in the treatment of water.
Statement (II) **:** It forms hypochlorous acid to oxidize the organic compounds including bacteria.

Q. 197. ***Statement (I)*** **:** Fluoride concentrations of approximately 1.0 mg/l in drinking water help to prevent dental cavities in children.
Statement (II) **:** During formation of permanent teeth, fluoride combines chemically with tooth enamel resulting in softer and weaker teeth that are less resistant to decay.

Q. 198. ***Statement (I)*** **:** An epidemic of infective hepatitis is transmitted by drinking contaminated water.
Statement (II) **:** Since infective hepatitis is transmitted by bacteria, it can be controlled by filtration and disinfection of water.

Q. 199. ***Statement (I)*** **:** The ability of water to conduct electricity, known and measured as the specific conductance, and concentration of total dissolved solids are not relatable on a one-to-one basis.
Statement (II) **:** Many organic molecules and compounds dissolve in water without ionizing and hence are not taken into account while measuring specific conductance.

Q. 200. ***Statement (I)*** **:** Water with heavy algal growth often has pH values as high as 9 to 10.
Statement (II) **:** Non-utilization of the bicarbonate ion as a carbon source by algae can result in substantial accumulation of OH^- ions.

Year 2013

Q. 201.* In a pipeline 1920 m long, the velocity of propagation of pressure wave is 960 m/s. Rapid closure of a downstream valve will entail, the maximum time for the closure is :
(a) 8 s (b) 6 s
(c) 4 s (d) 2 s

Q. 202. The dissolved oxygen in a water sample is generally estimated by modified Winkler method. The water sample is taken in a 300 ml BOD bottle and the dissolved oxygen is fixed by adding :
(a) 1 ml of conc. H_2SO_4 followed by addition of 1 ml of $MnSO_4$ reagent and contents mixed thoroughly, thereafter waiting for 5–10 minutes, and then 1 ml of alkali-iodide-azide reagent is added
(b) 1 ml of alkali-iodide-azide reagent followed by addition of 1 ml of conc. H_2SO_4 and then the contents mixed thoroughly, thereafter waiting for 5–10 minutes, 1 ml of $MnSO_4$ reagent is added
(c) 1 ml of $MnSO_4$ followed by addition of 1 ml of alkali-iodide-azide reagents and then the contents mixed thoroughly, thereafter waiting for 5-10 minutes and then 1 ml of conc. H_2SO_4 is added
(d) 1 ml of conc. of H_2SO_4 followed by addition of 1 ml of alkali-iodide-azide reagent and then the contents mixed thoroughly, thereafter waiting for 5–10 minutes and then 1 ml of $MnSO_4$ reagent is added.

Q. 203. Consider the following statements :
1. Typhoid fever is caused by viral infection.
2. Infectious hepatitis is caused by bacterial infection.
3. Cholera is caused by bacteria
4. Amoebic dysentery is caused by protozoal infection.

Which of these statements are correct ?
(*a*) 1, 2, 3 and 4 (*b*) 1, 2 and 3 only
(*c*) 3 and 4 only (*d*) 2 and 3 only.

Q. 204. Which of the following statements are the important characteristics of a slow sand filter ?
1. Cleaning of filter is done by scraping and sand removal.
2. Lack of pretreatment.
3. Greater efficiency of bacterial removal as compared to rapid sand filter.
4. Efficient in colour, taste and odour removal.

(*a*) 1, 2, 3 and 4 (*b*) 1, 2 and 4 only
(*c*) 2 , 3 and 4 only (*d*) 1, 2 and 3 only

Q. 205. Which of the reasons given below favour the use of cast iron pipe for city water supply ?
1. Strong in tension; and pipe of large diameter (up to 6 m) can be made
2. Easy to make joints
3. Durable
4. Corrosion-resistant

(*a*) 1, 2, 3 and 4 (*b*) 2, 3 and 4 only
(*c*) 1, 2 and 4 only (*d*) 1, 3 and 4 only

Directions : *Each of the next given items consists of two statements, one labelled as the 'Statement (I)' and the other as 'Statement (II)'. You are to examine these two statements carefully and select the answers to these items using the codes given below :*

Codes :
(*a*) *Both Statement (I) and Statement (II) are individually true and Statement (II) is the correct explanation of Statement (I)*
(*b*) *Both Statement (I) and Statement (II) are individually true but Statement (II) is not the correct explanation of Statement (I)*
(*c*) *Statement (I) is true but Statement (II) is false.*
(*d*) *Statement (I) is false but Statement (II) is true.*

Q. 206. ***Statement (I) :*** Rapid sand filters consisting of carefully sieved sand on top of a bed of gravel will only filter particles larger than its pore size.

Statement (II) : In addition to straining, adsorption, continued flocculation and sedimentation in the pore spaces help in particle removal mechanism.

Q. 207. ***Statement (I) :*** Suspended as well as colloidal particulates are removed from water by addition of 25% alum dosage ?

Statement (II) : The optimum dosage of alum is determined by performing the 'Jar test'.

Q. 208. ***Statement (I) :*** Gravity is a determinant in estimating pipe flow magnitudes.

Statement (II) : The term '*g*' appears explicitly in Darcy-Weisbach formula for pipe flow resistance.

Q. 209. ***Statement (I) :*** Water with heavy algal growths often has pH values as high as 9 to 10.

Statement (II) : Non-utilization of the bicarbonate ion as a carbon source by algae can result in substantial accumulation of OH ions.

Year 2014 (*Engineering Services Objectives*)

Q. 210.* The total hardness value obtained from the complete analysis of a water sample was found to be 120 mg/l. If the value of carbonate hardness is 50 mg/l, the non-carbonate hardness and alkalinity are, respectively
(*a*) 170 mg/l and 70 mg/l (*b*) 170 mg/l and 50 mg/l
(*c*) 70 mg/l and 50 mg/l (*d*) 50 mg/l and 70 mg/l

Q. 211.* Population levels over 5 decades of a small town are given below.

Year	*Population*
1960	2,50,000
1970	2,80,000
1980	3,40,000
1990	4,20,000
2000	4,90,000

The population of the town in the year 2020 estimated by Arithmetic Increasea method will be
(*a*) 5,10,000 (*b*) 5,90,000
(*c*) 6,10,000 (*d*) 6,90,000

Q. 212. The dissolved oxygen in a water sample is generally estimated by modified Winkler method. Accordingly, approximately 200 ml volume of dissolved-oxygen-fixed solution shall be titrated with
(*a*) Sodium thiosulphate reagent using soluble starch as an indicator
(*b*) Sodium thiosulphate reagent using ferroin as an indicator
(*c*) Ferrous ammonium sulphate reagent using soluble starch as an indicator
(*d*) Ferrous ammonium sulphate reagent using ferroin as an indicator

Q. 213. A water treatment plant treats 6000 m^3 of water per day. If it consumes 20 kg chlorine per day, then the chlorine dosage would be
(*a*) 3.00 mg/l (*b*) 3.75 mg/l
(*c*) 4.25 mg/l (*d*) 3.33 mg/l

Q. 214.* When a turbid stream flows into the ocean, the ionic content of the water increases drastically. This causes coagulation and settling

which leads to formation of deposits (deltas). Such coagulation mechanism is called.

(*a*) Adsorption and charge neutralization

(*b*) Interparticle bridging

(*c*) Ionic layer compression

(*d*) Sweep coagulation

Q. 215.* A rapid sand filter for a town with water repuirement of 1 MLD is to be provided with rate of filtration at 4000 l/hr/m^2, with backwash system. What would be the size of the filter and its underdrainage system to the nearest approximation ?

(*a*) 10 and 0.032 sq. m (*b*) 11 and 0.022 sq. m

(*c*) 12 and 0.045 sq. m (*d*) 15 and 0.062 sq. m

Q. 216. Consider the following statements in respect of injecting chlorine between rapid sand filter and storage tank :

1. More contact time is available.
2. Chlorine demand is reduced.
3. $NHCl_2$ is formed which is more effective than residual chlorine.
4. Break point chlorination takes place.

Which of the above statements are correct ?

(*a*) 1, 2, 3 and 4 (*b*) 1, 2 and 3 only

(*c*) 1 and 2 only (*d*) 3 and 4 only

Q. 217.* What is the settling velocity of a discrete particle in a wide body of water when the relevant Reynolds number is less than 0.5 ? The diameter and specific gravity of the particle are 2×10^{-3} cm and 2.65, respectively. Water temperature is 20°C. (Kinematic viscosity = 2×10^{-2} cm^2/sec.)

(*a*) 0.018 cm/sec (*b*) 0.025 cm/sec

(*c*) 0.18 cm/sec (*d*) 0.25 cm/sec

Q. 218. If the length dimension of a square filter bed increases to two times (while the rate of filtration remains unchanged), the amount of water filtered would become

(*a*) 4 times (*b*) 2 times

(*c*) 1 time (*d*) 16 times

Directions : *Each of the next given item(s) consists of two statements, one labelled as the 'Statement (I)' and the other as 'Statement (II)'. You are to examine these two statements carefully and select the answers to these items using the codes given below :*

Codes :

(*a*) *Both Statement (I) and Statement (II) are individually true and Statement (II) is the correct explanation of Statement (I)*

(*b*) *Both Statement (I) and Statement (II) are individually true but Statement (II) is not the correct explanation of Statement (I)*

(*c*) *Statement (I) is true but Statement (II) is false*

(*d*) *Statement (I) is false but Statement (II) is true*

Q. 219.* ***Statement (I) :*** During an epidemic of infective hepatitis, the supplied water is super chlorinated.

Statement (II) : Spore-forming bacteria which cause this disease are effectively removed from water by super chlorination.

Year 2015 (*Engineering Services Objectives*)

Q. 220. A certain water needs alum treatment to the extent of 10 p.p.m. How much alum, in quintals per day, would be needed to treat 10 MLD of water?

(*a*) 10 (*b*) 1.0

(*c*) 100 (*d*) 1000

Q. 221. 10 MLD water is to be chlorinated in a clear water reservoir (CWR) with 0.8 mg/l chlorine dose with providing contact time of 40 minutes. The required CWR capacity is nearly,

(*a*) 220 m^3 (*b*) 280 m^3

(*c*) 28 m^3 (*d*) 22 m^3

Q. 222. Consider the following statements in respect of cast iron pipes employed for water supply:

1. Easy to make joints 2. Strong and durable
3. Corrosion resistant 4. Long life

Which of the above statements are correct?

(*a*) 1, 2 and 3 only (*b*) 1, 3 and 4 only

(*c*) 2, 3 and 4 only (*d*) 1, 2, 3 and 4

Q. 223. Two reservoir are connected by two pipes P and Q. The pipes have the same diameter and length and are placed in parallel. If the friction factor of P is 9 times that of Q, then the discharge in P to that in Q is

(*a*) 0.5 (*b*) 0.45 (*c*) 0.33 (*d*) 0.27

Q. 224. Consider the following statements regarding valves in a pipe line:

1. In long pipe lines, air will accumulate in the low point of the line and will interfere with the flow.
2. Pressure relief valves are used in pipe lines where pressure may increase beyond the maximum permissible pressure.
3. Non-return valves prevent water flowing back, *i.e.*, in the opposite direction.

Which of the above statements are correct?

(*a*) 1 and 2 only (*b*) 2 and 3 only

(*c*) 1 and 3 only (*d*) 1, 2 and 3

Q. 225. The population of a city in the year 2000 was 82,300. If average per cent increase in population per decade is 35%, the population of the city in the year 2020 estimated geometrical increase will nearly be

(*a*) 1,00,000 (*b*) 1,25,000

(*c*) 1,50,000 (*d*) 1,75,000

Q. 226. What is the rapid sand filter surface area required for filtering of 10 MLD water assuming a filtration rate of 100,000 l/m^2/day?

(*a*) 100 m^2 (*b*) 10 m^2 (*c*) 1 m^2 (*d*) 1000 m^2

Q. 227. Consider the following statements in respect of slow sand filter and rapid sand filter:

1. The two filters differ in respect of the standards regarding non-uniformity of the sand used in their filtering media.
2. The two filters do not differ in respect of the effective size of the sand used in them.
3. The two filters differ in respect of their respective under-drainage system.
4. The two filters differ in respect of their respective rate of filtration.

Which of the above statements are correct?

(*a*) 1, 2 and 3 (*b*) 1, 2 and 4 (*c*) 2, 3 and 4 (*d*) 1, 3 and 4

Q. 228. How many kg of bleaching powder is needed per day to chlorinate 4 MLD of water so that, after 40 minutes of contact, there remains residual chlorine of 0.25 mg/l. The input water has a chlorine demand of 1.25 mg/l, and that the bleaching powder has only 25% available chlorine.

(*a*) 8 kg (*b*) 20 kg (*c*) 24 kg (*d*) 6.6 kg

Directions : *Each of the next listed items consists of two statements, one labelled as the 'Statement (I)' and the other as 'Statement (II)'. Examine these two statements carefully and select the answers to these items using the codes given below:*

Codes :

(*a*) *Both statement (I) and Statement (II) are individually true and Statement (II) is the correct explanation of Statement (I)*

(*b*) *Both Statement (I) and Statement (II) are individually true but Statement (II) is not the correct explanation of Statement (I)*

(*c*) *Statement (I) is true but Statement (II) is false*

(*d*) *Statement (I) is false but Statement (II) is true*

Q. 229. ***Statement (I) :*** By providing Air Vessels on the suction and delivery sides of a reciprocating pump, it is possible to increase the delivery head of the pump.

Statement (II) : The air vessel terminates the acceleration head and contributes to the outgoing discharge becoming reasonably steady and uniform.

Q. 230.* ***Statement (I) :*** Dracontiasis is transmitted by drinking contaminated water.

Statement (II) : Dracontiasis can be controlled by filtration of the drinking water.

Q. 231. ***Statement (I) :*** Coagulation is the process of charge neutralization on colloids.

Statement (II) : Flocculation is the process to grow the chargeless colloids into settleable flocs.

Q. 232. ***Statement (I) :*** The specific speed (N_s) of a centrifugal pump is defined as the speed (in rpm) at which it works most efficiently.

Statement (II) : The specific speed is a characteristic of pumps that can be used as a basis for comparing the performance of centrifugal pumps.

ANSWERS TO THE OBJECTIVE QUESTIONS OF THE ENGINEERING SERVICES EXAMS

1. (*a*)	**2.** (*b*)	**3.** (*d*)	**4.** (*a*)	**5.** (*b*)
6. (*b*)	**7.** (*c*)	**8.** (*b*)	**9.** (*b*)	**10.** (*b*)
11. (*c*)	**12.** (*a*)	**13.** (*c*)	**14.** (*c*)	**15.** (*b*)
16. (*c*)	**17.** (*a*)	**18.** (*b*)	**19.** (*d*)	**20.** (*a*)
21. (*a*)	**22.** (*a*)	**23.** (*d*)	**24.** (*a*)	**25.** (*b*)
26. (*d*)	**27.** (*a*)	**28.** (*c*)	**29.** (*b*)	**30.** (*d*)
31. (*b*)	**32.** (*b*)	**33.** (*a*)	**34.** (*c*)	**35.** (*d*)
36. (*d*)	**37.** (*c*)	**38.** (*a*)	**39.** (*d*)	**40.** (*c*)
41. (*b*)	**42.** (*c*)	**43.** (*b*)	**44.** (*c*)	**45.** (*d*)
46. (*c*)	**47.** (*b*)	**48.** (*c*)	**49.** (*a*)	**50.** (*d*)
51. (*c*)	**52.** (*b*)	**53.** (*a*)	**54.** (*b*)	**55.** (*d*)
56. (*d*)	**57.** (*a*)	**58.** (*a*)	**59.** (*d*)	**60.** (*c*)
61. (*d*)	**62.** (*b*)	**63.** (*c*)	**64.** (*b*)	**65.** (*b*)
66. (*d*)	**67.** (*b*)	**68.** (*a*)	**69.** (*a*)	**70.** (*b*)
71. (*b*)	**72.** (*a*)	**73.** (*b*)	**74.** (*d*)	
75. (A) (*a*) ; (B) (*c*) ; (C) (*c*)		**76.** (*d*)	**77.** (*c*)	**78.** (*c*)
79. (*d*)	**80.** (*c*)	**81.** (*d*)	**82.** (*d*)	**83.** (*d*)
84. (*a*)	**85.** (*b*)	**86.** (*d*)	**87.** (*d*)	**88.** (*b*)
89. (*c*)	**90.** (*b*)	**91.** (*b*)	**92.** (*d*)	**93.** (*b*)
94. (*a*)	**95.** (*d*)	**96.** (*d*)	**97.** (*b*)	**98.** (*b*)
99. (*b*)	**100.** (*b*)			
101. (*i*) (*d*)	(*ii*) (*c*)	(*iii*) (*c*)	**102.** (*c*)	**103.** (*b*)
104. (*b*)	**105.** (*d*)	**106.** (*a*)	**107.** (*d*)	**108.** (*d*)
109. (*b*)	**110.** (*a*)	**111.** (*d*)	**112.** (*a*)	**113.** (*b*)
114. (*b*)	**115.** (*b*)	**116.** (*b*)	**117.** (*c*)	**118.** (*c*)
119. (*a*)	**120.** (*a*)	**121.** (*c*)	**122.** (*d*)	**123.** (*c*)
124. (*a*)	**125.** (*c*)	**126.** (*b*)	**127.** (*d*)	**128.** (*b*)
129. (*b*)	**130.** (*d*)	**131.** (*a*)	**132.** (*a*)	**133.** (*c*)
134. (*a*)	**135.** (*a*)	**136.** (*b*)	**137.** (*a*)	**138.** (*a*)
139. (*a*)	**140.** (*c*)	**141.** (*b*)	**142.** (*c*)	**143.** (*a*)
144. (*d*)	**145.** (*b*)	**146.** (*c*)	**147.** (*c*)	**148.** (*d*)
149. (*b*)	**150.** (*c*)	**151.** (*b*)	**152.** (*b*)	**153.** (*a*)
154. (*d*)	**155.** (*a*)	**156.** (*a*)	**157.** (*d*)	**158.** (*d*)
159. (*a*)	**160.** (*b*)	**161.** (*a*)	**162.** (*a*)	**163.** (*b*)

164. (*a*)	165. (*b*)	166. (*b*)	167. (*a*)	168. (*a*)
169. (*a*)	170. (*b*)	171. (*c*)	172. (*c*)	173. (*a*)
174. (*a*)	175. (*c*)	176. (*b*)	177. (*c*)	178. (*a*)
179. (*b*)	180. (*d*)	181. (*b*)	182. (*c*)	183. (*d*)
184. (*a*)	185. (*b*)	186. (*d*)	187. (*d*)	188. (*d*)
189. (*b*)	190. (*c*)	191. (*c*)	192. (*a*)	193. (*d*)
194. (*c*)	195. (*d*)	196. (*a*)	197. (*c*)	198. (*c*)
199. (*a*)	200. (*c*)	201. (*c*)	202. (*c*)	203. (*c*)
204. (*a*)	205. (*b*)	206. (*b*)	207. (*d*)	208. (*a*)
209. (*a*)	210. (*c*)	211. (*c*)	212. (*a*)	213. (*d*)
214. (*c*)	215. (*c*)	216. (*c*)	217. (*a*)	218. (*b*)
219. (*c*)	220. (*b*)	221. (*b*)	222. (*a*)	223. (*c*)
224. (*b*)	225. (*c*)	226. (*b*)	227. (*d*)	228. (*c*)
229. (*d*)	230. (*b*)	231. (*b*)	232. (*b*)	

Hints for Solving Starred Questions

Q. 15. The settling/sedimentation involved in raw and waste waters has generally been divided into the following four types :

Type I settling. This type of settling refers to the settling of **discrete particles,** such as the removal of grit and sand from raw waters, containing low concentrations of solids. *The settling of discrete particles in dilute concentrations is thus covered under this type of settling.*

Type 2 settling. This type of settling refers to *the settling of flocculent particles in rather dilute suspensions.* This settling helps in the removal of a portion of the suspended solids in the primary settling tanks, and in upper portions of the secondary settling tanks.

Type 3 settling. This type of settling, called **hindered** or **zoned settling,** involves the *settling of the flocculent particles in concentrated suspensions*, in which interparticle forces are sufficient to hinder the settling of the neighbouring particles, thereby tending the particles to remain in fixed positions with respect to each other, causing settling of the mass of particles as a unit, developing a solid-liquid interface at the top of the settling mass. The settling that occurs in the secondary settling tank of a Biological Sewage Treatment Unit, is an example of this type of settling.

Type 4 settling. When solids are present in excessive concentration, thereby forming a '*structure* or *sludge blanket*, such as at the bottom of a deep secondary settling tank or in a sludge thickner unit, the settling occurs only by compression, caused by the weight of the particles which are constantly being added to the structure by settling from the supernatant liquid. *Such settling of particles in the form of sludge blanket in wastewater treatment is usually represented as Type 4 settling.*

Q. 17. Chlorine demand to treat 1 litre of water = 0.2 mg.

x mg of bleaching powder, containing 30% chlorine, will supply $x \times 0.3$ mg of chlorine. To balance, we have

$$0.2 = (x \times 0.3)$$

or
$$x = \frac{0.2}{0.3}\ (\text{mg}) = 0.67\ \text{mg}.$$

Hence, the choice at (*a*) is correct. **Ans.**

Q. 19. The residual water pressure at the farthest consumer's tap in the distribution system, should under no circumstances be kept less than 14 m of water head (≈ 140 kPa)*, so as to permit the Pumpers of the Fire Deptt to obtain adequate flows from the hydrants, since this much head is usually lost in the hydrant and suction pipe. *Ideally, however the minimum pressure should be kept to be about 28 m (≈ 280 kPa) for residential service connections.* Pressure head of about 50 m of water (≈ 500 kPa) is, however, minimally required for fire fighting, where pumpers are not available, and direct water streams from the hydrants are to be used in the fire fighting. Pressures in excess of about 70 m (690 kPa) are generally undesirable, since leakage or bursting of pipes may occur at higher pressures.

The American Water Works Association recommends a normal static pressure of about 400 to 500 kPa.

When we consider the above provisions, we find the minimum ideal residual pressure to be 280 kPa, which is equal to 280 kN/m^2, which equals to $280 \times 1000\ \text{N/m}^2 = \frac{280 \times 1000}{10^6}\ \text{N/mm}^2 = 0.28\ \text{N/mm}^2$. Since this minimum pressure value can be provided only by choice (*d*), this choice can be considered to be the correct choice. **Ans.**

The case of non-availability of pumpers or necessity of maintaining higher pressures is ruled out by the given choices, since no choice is available with min. pressure head of 400 to 500 kN/m^2.

Q. 51. The bacteria residing in the colon (lower portion of large intestine) of man and animals, which are **lactose fermenters** are termed as *coliform bacilli. Escharichia coli (E-coli)* is the most common member of these bacilli ; while *klebsiella* is another important member. All such lactose fermenting enteric bacilli are, thus, included in the term *coliforms* or *coliform bacilli.* These bacteria are essentially **non-sporing** and **Gram negative.** They are also *non acid fast*, as explained in the Text.

Colon bacillus were first described by Mr. Escherich in 1885, and were called as *Bacterium coli commune (B-coli).* These colon bacillus not only include the lactose fermenters (coliforms) but also include *late lactose fermenters* (pathogens like Shigella Sonnei and Paracolons) as well as *non-lactose fermenters* (pathogens like Salmonella and Shigella).

*10.2 m of water head = 100 kPa.

The E-coli, as stated above, is the most commonly found coliform, and is further sub-divided into biotype and serotype. Unlike other coliforms, E-coli is a parasite living only in the human or animal intestines. Voided in faeces, it remains visible in the environment only for some days. *Detection of E-coli in drinking water, therefore, is taken as evidence of recent pollution with human or animal faeces.*

With the above knowledge, you can easily state that the terms bacillus, gram negative, and lactose fermenters do characterise coliforms (of course the term bacillus includes all kind of bacteria and not only coliforms). Hence, the choice (*c*) is correct, which includes these three terms but excludes spore-forming (since *coliforms are non-sporing*).

Q. 62. We have stated and explained in the Text that the carbonate hardness is equal to the Alkalinity or the total hardness, whichever is less. In other words, when Alkalinity exceeds or equals the total hardness, then carbonate hardness will be equal to the total hardness. Hence, choice (*b*) is correct. The 'methyl orange alkalinity', simply indicates the use of methyl orange indicator in titration, adopted for determining Alkalinity which changes its colour from orange at pH = 4.6 to pink at pH = 4.0.

Alkalinity is infact computed for a given water sample, by titrating it against 0.02(N)* H_2SO_4. For highly alkaline samples, the first step is titrating to pH of 8.3. The second phase, or the first in case of water with an initial pH of less than 8.3, is its titration to an indicated pH of 4.5. The end points of titration can be determined either by using pH meters to check pH changes ; or by using indicators (added to water sample) which changes colour as *p*H changes with the addition of H_2SO_4. *Phenolphthalein indicator is used in first stage titration, since it changes colour from pink (pH > 8.3) to colourless (at pH = 8.3).* For second stage titration, methyl orange may be used which changes from red (at pH > 4.6) to pink. *Bromocresol green methyl* red indicator may be used as a preferred substitute to methyl orange, since it changes from bluish gray at pH 4.8 to light pink at pH = 4.6.

The alkalinity in titration is computed by the equation :

$$\text{Alkalinity as } CaCO_3 = \frac{\text{ml of titrant used} \times \text{Normality of acid} \times 50 \times 1000}{\text{ml of water sample}}$$

(*i.e.* Eq. 15.3)

Q. 73. The surface overflow rate $\left(\frac{Q}{BL}\right)$ represents the settling velocity of the particles, which shall be fully (100%) removed. Hence, if the settling

*1.00 N solution of H_2SO_4 contains one gram of available hydrogen ions or its equivalent per litre of solution. A 0.02 N H_2SO_4 $\left(i.e.\ \frac{N}{50}H_2SO_4\right)$ solution contains 0.98 gm of pure sulphuric acid, which is 0.02 times the equivalent mass of sulphuric acid (*i.e.* 49).

velocity of a particle is more than or equal to surface overflow rate (S.R.O.), it shall be fully removed. However, if the settling velocity of a particle size is less than the surface overflow rate, it wan't get removed.

In the given case, the surface overflow rate is 3 m/h (*i.e.* 3 $m^3/m^2/h$), while the settling velocity of particle type I is also 3 m/h ; and hence, type I particles, having concentration of 200 mg/l, will completely settle down. Particle type II, having settling velocity of 1 m/h (less than that of S.O.R.) will however not at all be removed in the ideal horizontal settling tank (which does not involve partial removal of particles). Hence, the concentration of particles settling in the tank will be equal to 200 mg/l. Hence, choice (*b*) is correct. **Ans.**

Q. 95. **Helminths** are worms, which naturally, would get removed in rapid sand filters, which are even capable of removing 80-90% of bacteria, which are much smaller than helminths.

Q. 103. **Enteric fever** is another name of typhoid fever, which is caused by Salmonella typhi organism, as explained in the text. Hence, choice (*b*) is correct. **Ans.**

Q. 123. $25.9 - 10\,Q_A^2 = 9$

$\therefore \quad Q_A = 1.3$

Also, $18 - 4\,Q_B^2 = 9$

$\therefore \quad Q_B = 1.5$

Between *AJC*, we have

$$25.9 - 10\,Q_A^2 - 1\,Q_C^2 = P_C$$

But $Q_C = Q_A + Q_B = 1.3 + 1.5 = 2.8$

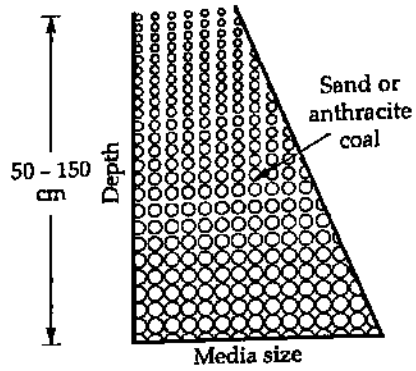
$\therefore \quad 25.9 - 10 \times (1.3)^2 - 1 \times (2.8)^2 = P_C$

or $25.9 - 16.9 - 7.84 = P_C$

or $P_C = \mathbf{1.16\ m.}$ **Ans.**

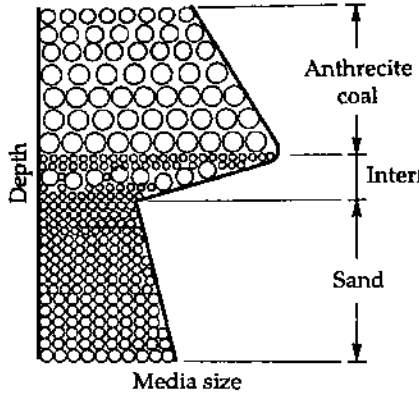
Hence, choice (*c*) is correct. **Ans.**

Q. 132. Both the commonly used *slow sand filters* (SSF) as well as the *rapid sand filters* (*RSF*), described in the text, utilize a single material, *i.e.*, well graded sand. In such a single-medium filter, *after backwashing*, larger grains settle down faster than the smaller grains, causing formation of *stratification*, called ***reverse gradation,*** as shown in Fig. 18.9 (*a*). This phenomena offers a major disadvantage, as only the top 4 or 5 cm of filter depth is utilised, because the smaller media grains at the top of the filter bed will absorb most of the (particles) contained in the water being filtered. Impurities particles that pass through this layer are less likely to be removed by the layer media grains below. This also leads to quicker clogging of the filter, since only the top 4 or 5 cm of filter bed is practically being used for catching the impurities (particles). This problem is sometimes minimized by using a large-diameter medium with a uniformity coefficient close to unity (*i.e.* less than 1.4) and by using a deeper medium bed. Such filters, called **monomedium filters,** often utilize



(*i*) *For sand*
Effective size = 0.45 – 1.0 mm
Uniformity coeff. = 1.2 – 1.7
Depth = 50 – 150 cm.
Or
(*ii*) *For Anthracite Coal*
Effective size = 0.5 – 1.5 mm
Uniformity coeff. = 1.2 – 1.7
Depth = 50 – 150 cm

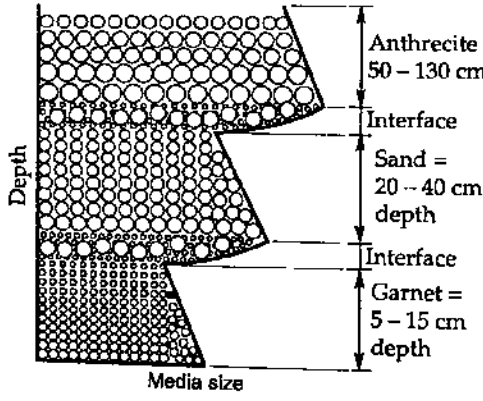
(*a*) Single medium filter



30 – 60 cm depth
Effective size = 0.7 – 2.0 mm
Uniformity coeff. 1.3–1.8

20 – 40 cm depth
Effective size = 0.45 – 0.6 mm
Uniformity coeff. = 1.2 – 1.7

(*b*) Dual Medium Filter



(*i*) *Top Anthracite Coal layer*
Depth = 50 – 130 cm
Effective size 1.0 – 2.0 mm
Uniformity coeff. = 1.4 – 1.8
(*ii*) *Middle Sand Layer*
Depth = 20 – 40 cm
Effective Size = 0.4 – 0.8 mm
Uniformity coeff. = 1.2 – 1.7.
(*iii*) *Bottom Garnet layer*
Depth = 5 – 15 cm
Effective Size = 0.2 – 0.8 mm
Uniformity Coeff. = 1.5 – 1.8

(*c*) Multi-Media Filter

Fig. 18.9. Single Medium, Dual Media and Multi Media Filters.

anthracite coal as the filter media, and have been successfully used in California and Texas (USA) to treat water *with higher filtration rates*. Whereas, the usual filtration rate for rapid sand filters may range from 75 to 150 $m^3/d.m^2$ (highest used being about 235 $m^3/d.m^2$), the filtration rates for anthracite coal filters of 1.5 m to 2.5 m depth (*i.e., deep bed monomedium filters*) may be as high as 800 $m^3/d.m^2$.

Another solution to the problem of *reverse gradation* is to use **dual-Media Filter (DMF).** Typical *dual media filter (DMF)* uses anthracite coal at the top 30—60 cm depth of the filter media, and filter sand in the bottom 20—40 cm depth portion, as shown in Fig. 18.9 (*b*). The anthracite with a specific gravity of 1.55 is lighter than the sand with a sp. gravity of 2.65. Therefore, a larger anthracite grain has the same settling velocity as a much smaller sand grain. This characteristic allows larger anthracite grains to be placed on top of the smaller sand grains.

The effective size and Uniformity coefficient of top anthracite coal may vary between 0.7—2.0 mm ; and 1.3—1.8, respectively. These respective values for the bottom sand layer may range between 0.45—0.60, and 1.2—1.7.

Dual medium filter may behave much like the two-single-medium filters, placed in series, one above the other, each with a different grain size. The top layer of larger anthracite particles removes the larger sized impurities (particles) as well as some smaller ones ; while the bottom sand, in its turn, captures the smaller impurities (particles). As a consequence, more depth of filter is utilized for removal of solids (impurities) than in the stratified single-medium filter. Such DMFs help in providing higher filtration rates (than the RSFs) up to as high as 300 $m^3/d.m^2$ (12, 500 $l/h/m^2$ of filter).

Multi Media Filters (MMFs), also called ***Mixed-Media Filters,*** are similar to dual-media filters, except that they use three or more materials for filter layers. Typically, a three layered filter, consisting of anthracite coal (sp. gravity 1.55), filter sand, (sp. gravity 2.65) and garnet (sp. gravity = 4.05) are used from the top to the bottom, as shown in Fig. 18.9 (*c*).

The multi-media filters are basically improved dual-media filters, with increased filter run times and better water quality. The rates of filtration, however, are not much improved, and almost equals those of dual-media filters. The cost of a multi-media filter, is, however, much higher than that of a dual-media filter, because garnet is quite costly. The improved performance of multi-media filters often cannot justify the additional cost.

With the above knowledge about dual-media filters and multi-media filters (not described in our text), you can easily mark, the correct choice in the given objective question, as (*a*). **Ans.**]

Q. 142. From Eq. (6.16), we have

$$P_{h(max)} = \rho_w \cdot U_p \cdot V$$

where $P_{h(max)}$ = Max. water hammer pressure developed (due to instantaneous closure of valve) = 60 m

$= 60 \times 9.81$ kN/m^2

$= 60 \times 9.81 \times 1000)$ N/m^2

ρ_w = density of water = 1000 kg/m^3

U_p = velocity of the pressure wave generated = 1000 m/s (given)

V = Velocity of water

Substituting the given values, we get

$$(60 \times 9.81 \times 1000)\ \frac{\text{N}}{\text{m}^2} = 1000\ \text{kg/m}^3 \times 1000\ \text{m/s} \times \text{V}$$

or $$(60 \times 9.81 \times 1000)\ \frac{\text{kgm}}{\text{sec}^2} \cdot \frac{1}{\text{m}^2} = 1000\ \frac{\text{kg}}{\text{m}^3} \times 1000\ \frac{\text{m}}{\text{s}} \times \text{V}$$

or $$\left(\frac{60 \times 9.81}{1000}\right) \frac{\text{m}}{\text{s}} = \text{V}$$

$$\therefore \quad V = \frac{60 \times 9.81}{1000}\ \text{m/s} = 0.589\ \text{m/s}$$

From Eq. (6.20), we have

$$T_c = \text{Critical time of closure} = \frac{2S}{U_p}, \text{ where } S = 2000\ \text{m (given)}$$

$$\therefore \quad T_c = \frac{2 \times 2000}{0.589} = 6791\ \text{sec.}$$

Since the actual closure time (T) of 4 sec is less than the critical time T_c, full water hammer pressure of 60 m of water will be developed. Hence, choice (*c*) is correct. **Ans.**

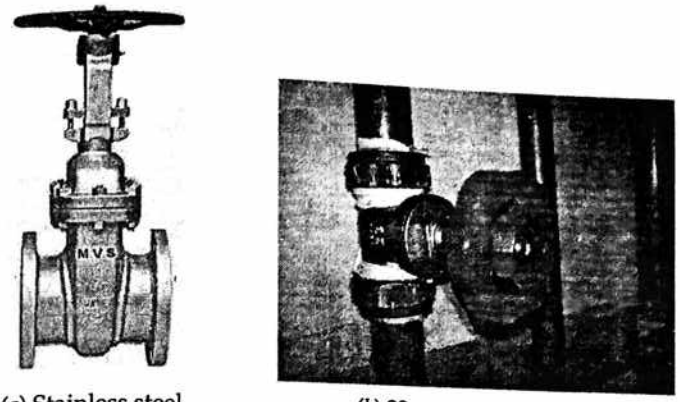
Q. 145. Please see article 9.24.1.

Q. 147. The various types of shut off valves that are used in plumbing are discussed below :

(1) **Sluice Valve.** A sluice valve, also known as a **gate valve,** as explained in the text under article 6.9.1 (*a*), is a valve that opens by lifting a round or a rectangular gate/wedge out of the path of the fluid. Since the sealing surfaces between the gate and the seat are planar, the *gate valves are often used when a straight line flow with minimum restriction is desired.* The gate faces can take a wedge shape (Fig. 6.27), or they can be parallel (Fig. 6.28). *Typical gate valves are usually not to be used for throttling purposes for regulating flow, unless they are specifically designed for that purpose, because on opening the gate valve, the flow path is enlarged in a highly non-linear manner with respect to the percentage of opening.* This means that the flow rate does not change evenly with stem travel, *Moreover, a partially open gate disk tends to vibrate from the fluid flow. Most of the flow change occurs near shut-off with relatively high fluid velocity, causing quicker wear and tear of the disk and the seat, eventually leading to leakage, if such a valve is often used to regulate the flow,* ***Typical gate valves are, hence, designed to be fully opened or closed.*** When fully open, the typical gate valve has no obstruction in the flow path, resulting in very low friction loss.

Gate valves may be provided with a rising stem attached to a wheel gate (Fig. 6.27 and 18.10); or they may have a non-rising stem with the operating

nut at the top, as the gate travels up or down the stem on threads (Fig. 6.28). Such non-rising stems are used underground, or where vertical space is limited.



(*a*) Stainless steel gate valve

(*b*) 22 mm gate valve on domestic hot water pipe

Fig. 18.10. Typical gate valves provided with rising stems.

(2) **Butterfly Valve.** A butterfly valve is a valve which may be used for isolating or regulating flow. The closing mechanism is in the form of a moveable metal disc that rotates on a spindal or an axle, set in the body shell. The disc is positioned in the centre of the pipe, as shown in Fig. 18.11. Passing through the disc is a rod, connected to an *actuator* on the outside of the valve. Rotating the actuator rotates the disc, either parallel or perpendicular to the flow. *Thus when the disc is parallel to the flow, the valve will be 'fully open', and when the disc is perpendicular to 'the flow, the valve* will be 'fully closed. Eventually, the disc is always present within the flow, which induces some pressure drop, regardless of the valve position.

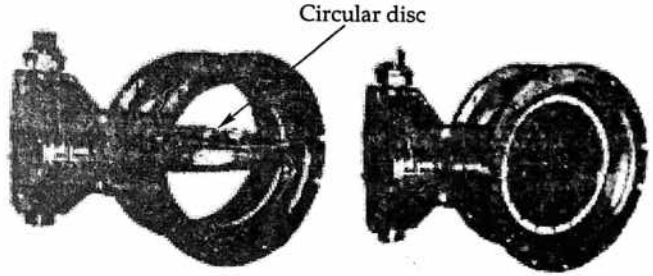


Fig. 18.11. Views of a butterfly valve in open and closed positions.

To close the valve, the disc needs only a quarter ($\frac{1}{4}$ th) of a turn, which completely blocks the passageway, thereby completely and quickly shutting off the flow. To open the valve, the disc is again rotated a quarter turn, so that it allows an almost unrestricted passage of fluid, *The valve may also be easily opened incrementally to throttle flow,* Butterfly valves are generally favoured in water supply systems, because *they are lower in cost and lighter in weight.*

Other advantages of a butterfly valve are :

(*i*) it can be shut off quickly and tightly

(*ii*) *it has a low head loss*

(*iii*) it has a small space requirement; and

(*iv*) it has good throttling capabilities.

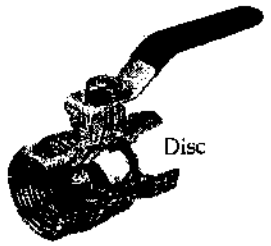
Due to the good throttling capabilities, they are largely used for regulating the rate of discharge from gravity filters in a water treatment plant.

The *main disadvantage* of the butterfly valve is caused due to the fact that its disk always remains in the flow direction, restricting the use of pipe cleaning tools.

There are different kinds of butterfly valves, each adapted for different pressures and different usage. The ***resilient butterfly valve***, which uses the flexibility of rubber, has the lowest pressure rating. The ***high performance butterfly valve***, used in slightly higher-pressure systems, features a slight offset in the way the disc is positioned, which increases the valve's sealing ability and decreases its tendency to wear. The valve best suited for high-pressure systems is the ***tricentric butterfly valve***, which makes use of a metal seat, and is therefore able to withstand a greater amount of pressure.

In recent times, ***rubber seated butterfly valves*** have also been devised for use in larger sized distribution mains. Because the large pressure difference across the disk of the butterfly valve in larger mains tends to close the valve, a mechanical arrangement has to be made to overcome the torque while opening the valve, and to resist 'this' force while closing the valve to prevent slamming. The mechanical arrangement so made is in the form of a hydraulic cylinder and piston rod assembly, which can hold the valve disk in open or closed or in any intermediate position.

(3) **Ball Valve.** A ball valve (Fig. 18.12) contains a spherical ball type disc, having a hole or a port through the middle, so that when this hole (opening) is in line with both ends of the valve body, flow will occur. However, when the valve is closed, the hole is perpendicular to the ends of the valve, which blocks the flow. The handle or lever will be in line with the hole (port) position, which permits us to "see" the valve position. The ball valve, along with butterfly valve and plug valve, are part of the family of '***quarter turn valves***'.



Cut-away of a ball-valve mechanism

Fig. 18.12. Figure showing the mechanism of a ball valve.

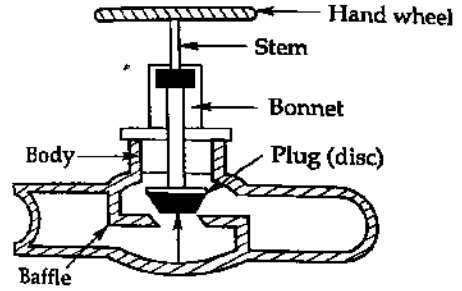
Ball valves are durable and usually work to achieve perfect shutoff even after years of use. *These are therefore an excellent choice for shutoff applications, and are often preferred to globe valves and gate valves for this purpose.* They, however, *do not offer the fine control that may be necessary in throttling applications, but are sometimes used for this purpose also.*

Ball valves are used extensively in industry because they are very versatile, supporting pressures up to 700 bars and temperatures up to 200°C. Sizes typically range from 0.5 cm to 30 cm. They are easy to repair and operate.

The body of ball valves may be made of metal, plastic or metal with a ceramic centre. The ball is often chrome plated to make it more durable.

(4) **Globe Valve.** A *globe valve* is used for regulating flow in a pipeline. It consists of a moveable disk-type element and a stationary ring seat in a generally spherical body.

Globe valves are named for their spherical (globe type) body shape, with two halfs of the body being separated by an internal baffle, having an opening which gets closed by the plug to shut the valve, as shown in Fig. 19.13(*a*). The plug is called the disk or the disc. The plug is connected to a stem, which is operated by skrew action in manual valves. Typically, automated valves are sliding stems. Automated globe valves have a smooth stem rather then threaded, and are opened and closed by an actuator assembly. When a globe valve is manually operated, the stem is turned by a hand wheel.



(*a*) Internal parts of a typical globe valve

(*b*) Stainless steel globe valve

Fig. 18.13. Globe valve.

Although the globe valves being manufactured in the past had the spherical bodies which gave them their name, many modern designs do not have much of a spherical shape. However, the term globe valve is still often used for valves that have such an internal mechanism. *In plumbing operations, valves with such a machanism are also often called as* **stop valves**, since they do not have the global appearance; but this term *stop valve* my be confusing, since all stop valves may not have such an internal mechanism as that of a globe valve.

Globe valves are generally used for applications requiring throttling and frequent operations. For example, such valves may be used as **sampling valves**, which are normally shut, except when liquid samples are being taken. *Since the baffle restricts the flow, they are not recommended where full unobstructed flow is required.*

(5) **Plug Valve.** Plug valves (Fig. 18.14) use either cylindrical or a conically tapered plug (disc), which can be rotated inside the valve body to control flow through the valve. The plugs in such a valve will have one or more hollow passage-ways going sideways through the plug, so that the fluid can flow through the plug when the valve is open.

When the plug is conically tapered, the stem/handle is typically attached to the larger diameter end of the plug. In such an assembly, there is not much length of stem; and the stem and handle often come in one piece. The other end of the plug is often exposed to the outside of the valve too, but with a mechanism which retains the plug in the body.

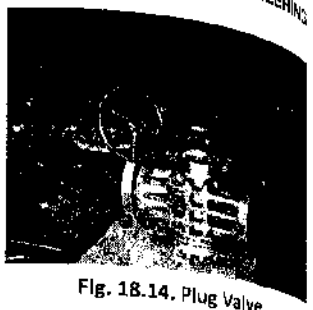


Fig. 18.14. Plug Valve.

The simplest and most common general type of plug valve is a 2-port* valve, which has two positions, *open* to allow flow, and *shut* to stop flow. The plug in this kind of valve has one passage way going through it. The ports are typically at opposite ends of the body, when the plug is rotated a fourth of a full turn, to change from open to shut position and vice versa. This makes this kind of plug valve a *quarter-turn valve*. There is often a mechanism, limiting motion of the handle to a quarter turn, but not in glass-cocks.

Plug valves are simple and economical shut off valves which are generally not used in plumbing operations because of high head loss and non-throttling operational characteristics. However, due to their simple and economical shut off qualities, slightly conical-tapered metal (often brass) plug valves are often used as simple shut off valves in household natural gas lines and LPG cylinders. Conically tapered glass plug valves are also widely used in laboratory glassware.

Q. 151. If the concentration of OH^- (ions) in moles/L is x, then is concentration in mg/L

$= x$ mole/L $\times$ 17 gm/mole (Molecular mass of OH)

$= 17x$ g/L $= 17x \times 10^3$ mg/L

This is measured to be **17 mg/L**

$\therefore \quad 17x \times 10^3 = 17; \quad$ or $\quad x = 10^{-3}$.

Hence, OH ions $= 10^{-3}$ moles/L

Since $H^+ \times OH^- = 10^{-14}$, we have

$\therefore \quad H^+ = 10^{-11}$ moles/L

$\therefore$ pH of water $= \log_{10} \dfrac{1}{H^+}$

$= \log_{10} \dfrac{1}{10^{-11}}$

$= \log_{10} (10^{11})$

$= 11$

Hence, choice (b) is correct. **Ans.**

Q. 152. We have explained in article 9.8 that the alum, also called *filter alum;* is primarily used as a coagulant, as it helps in forming a gelatinous precipitate of aluminium hydroxide by reacting with the bicarbonate alkalinity present in raw water. The precipitate so formed produces a *floc*, which

*Ports are the openings in the valve body through which fluid can enter or leave.

tends to collect the colloids present in raw water, as to grow in size, for subsequent settlement, by gravity. Since the floc so formed tends to collect/attach the colloids with itself, it is also known as **sweep floc**, *since the floc sweeps away the fine suspended matter. The coagulation so caused is also known as* ***sweep coagulation.***

Since the prime requirement for the success of formation of sweep floc is the presence of sufficient alkalinity in water, it *follows that the sweep coagulation needs high alkalinity.* Moreover, the greater is the quantity of colloids (causing turbidity in raw water) the larger will be the size of the sweep floc, since more colloids will get attached with sweep floc, causing the formation of a heavier floc, leading to successful coagulation. Hence, *high turbidity* also leads to successful sweep coagulation, Naturally, the choice of *high alkalinity-high turbidity* given at (b) is correct. **Ans.**

Q. 162. The tests which are commonly used for determining chlorine residuals (free and combined) given in the text are numbred at S. No. 1, 2 and 5 of the Question. Since the given choices having 1, 2 and 5 is only one, *i.e.* the one given at (A), which includes 1, 2, 4 and 5. The same gives the correct choice **'Ans'.** It may also be informed that another test called the ***Stabilised Neutral Orthotolidine* (*SNORT*)** colourimetric method is another test which was developed along with several others, such as *Leuco Crystal Violet (LCV)* and *LCD methods*, but none is of much practical significance.

Q. 168. Period of oscillation is given by Eq.

$$T = \frac{2L}{V_p}$$

where T = Period of oscillation

L = Length of pipe/penstroke

V_p = Velocity of pressure wave.

Here $L = 4500$ m

$V_p = 1500$ m/s

$\therefore \quad T = \dfrac{2 \times 4500 \text{ m}}{1500 \text{ m/s}} = 6$ sec

Hence, choice (a) is correct. **Ans.**

Q. 177. In general, a **pump** may be defined as a mechanical device which when interposed in a pipeline, converts the ***mechanical energy*** supplied to it from some external source into hydraulic energy and transfers the same to the liquid through the pipeline, thereby increasing the energy of the flowing liquid. Almost all the ***pumps*** increase the pressure energy of the liquid which is subsequently converted into potential energy as the liquid is lifted from a lower level to a ***higher level.*** The various pumps may broadly be *classified* into the following two types :

(*i*) *Rotodynamic pumps* (or *dynamic-pressure pumps*)

(*ii*) *Positive-displacement pumps.*

The ***rotodynamic pumps*** have a rotating element, called impeller, through which-as the liquid passes its ***angular momentum*** changes, due to which the pressure energy of the liquid is increased. As such, a ***rotodynamic pump does***

not push the liquid as in the case of a positive displacement pump. The centrifugal pumps are the most common examples of rotodynamic pumps.

The ***'positive displacement'*** pumps are those pumps in which the liquid is sucked and then it is actually pushed or displaced due to the thrust exerted on it by a moving member, which results in lifting the liquid to the required height. These pumps usually have one or more chambers which are alternately filled with the liquid to be pumped and then emptied again. As such, the discharge of liquid pumped by these pumps almost wholly depends on the speed of the pump. The most common example of the positive displacement type of pumps is that of reciprocating pumps.

Main components and working of a Reciprocating Pump

A reciprocating pump essentially consists of a plunger and a cylinder, where the plunger is enclosed by cylinder. The cylinder is connected to suction and delivery pipes, each of which is provided with a non-return or one way valve, called suction valve and delivery valve respectively.

The function of non-return or one way valve is to admit liquid in one direction only. Thus the suction valve allows the liquid only to enter the cylinder and the delivery valve permits only its discharge from the cylinder. The piston or the plunger is connected to a crank by means of a *connecting rod.*

As the crank is rotated at uniform speed by a driving engine or motor, the piston or plunger moves to and fro (or backward and forward) in the cylinder. When the crank rotates from $e = 0'$ and $e = 180'$, the piston or plunger which is initially at its extreme left position (that is, it is completely inside the cylinder), move to its extreme right position, (that is, it moves outwardly from the cylinder). During the outward movement of the piston or plunger, a partial vacuum (pressure below atmospheric) is created in the cylinder, which enables the atmospheric pressure acting on the liquid surface in the well or sump below, to force the liquid up the suction pipe and fill the cylinder by forcing open the suction valve.

Since during this operation of the pump, the liquid is sucked from below, it is known as its suction stroke. Thus at the end of the suction stroke, the piston or plunger is at its extreme right position, the crank is at $e = 180'$ (*i.e.*, at its outer dead centre), the cylinder is full of liquid, the suction valve is closed and the delivery valve is just at the point of opening.

Types of Reciprocating Pumps

The reciprocating pumps can be classified according to the liquid being in contact with one side or both the sides of the piston or plunger; and according to the number of cylinders provided.

According to the first basis of classification, the reciprocating pumps may be classified as :

(*i*) *Single acting pump.*

(*ii*) *Double acting pump.*

If the liquid is in contact with one side of the piston or plunger only, it is known as ***single acting pump.*** Thus, a single acting pump has one suction and one deliver pipe, and in one complete revolution of crank there are only two strokes-one suction and one delivery stroke. On the other hand, if the liquid is in contact with both the sides of the piston or plunger, it is known as ***double acting pump.*** A double acting pump has two suction and two delivery pipes with appropriate valves, so that during each stroke when suction takes place on one side of the piston, the other side delivers the liquid. In this way, in the case of a double acting pump, in one complete revolution of the crank, there are two suction strokes and two delivery strokes.

According to the number of cylinders provided, the reciprocating pumps may be classified as :

(*i*) Single cylinder pump.7

(*ii*) Double cylinder pump.

(*iii*) Triple cylinder pump.

(*iv*) Duplex double cylinder pump.

(*v*) Quintuplex pump.

Functions of an Air Vessel in a Reciprocating Pump

A reciprocating positive displacement pump is shown in Fig. 18.15. The pump is double acting, *i.e.* the liquid is admitted to either side of the piston,

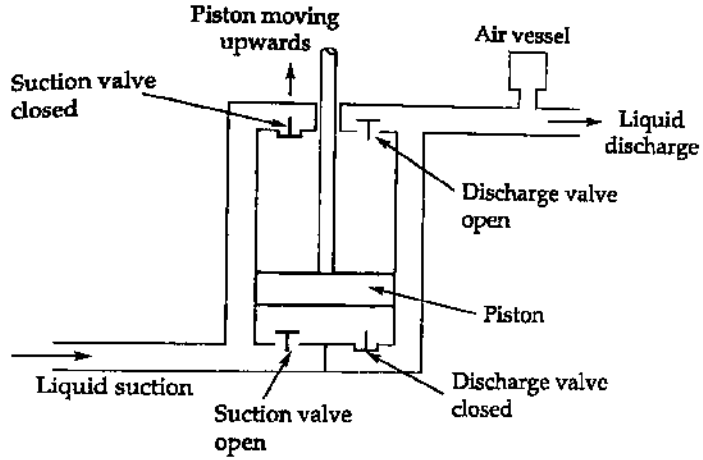


Fig. 18.15. Reciprocating piston pump with an air vessel.

where it is alternatively drawn in and discharged. *As the piston moves upwards,* suction takes place below the piston and liquid is drawn in, the valve arrangement ensuring that the discharge valve cannot open during suction stroke. Above the piston, liquid is discharged and the suction valve remains closed.

As the piston travels down, the operations of suction and discharge occur now on opposite sides.

An ***air vessel*** is usually fitted in the discharge pipe, to dampen out the pressure variations during discharge. As the discharge pressure rises, the air is compressed in the vessel ; and as the pressure falls, the air expands. The peak pressure energy is, thus, stored in the air and returned to the system when pressure falls.

A small air chamber fixed to the pipeline on the discharge side of a reciprocating pump, thus, acts as a cushion to minimize the shock produced by the

pulsations of the pump. The various advantages offered by the air vessel are listed below.

(*i*) Gives continuous supply at uniform rate

(*ii*) Reduces the possibility of separation and cavitation, even at higher speeds.

(*iii*) Allows pump to run at high speed

(*iv*) Suction head can be increased by increasing the length of the pipe below the air vessel.

(*v*) Large amount of power is saved due to low acceleration head.

Air vessels are, however, not fitted on the reciprocating boiler feed pumps, since they may introduce air into the de-aerated water. In view of the aforesaid knowledge, you may make the choice correctly in the given question as (*c*), since air vessels in reciprocating pumps do help in obtaining continuous uniform flow, to save power, and to run the pump at higher speed without the danger of separation. Hence, choice (*c*) is the correct choice. **Ans.**

Q. 181. Specific yield $= \dfrac{\text{Vol. of water drained } (V_w)}{\text{Vol. of aquifer drainaged}}$

or $$0.20 = \frac{V_w}{100 \text{ km}^2 \times 0.25 \text{ m}}$$

or $$V_w = 0.20 \times (100 \times 10^6 \text{ m}^2) \times 0.25 \text{ m} = 5 \text{ Mm}^3$$

Hence choice (*b*) is correct. **Ans.**

Q. 182. In flocculation, the best floc size is 0.1 to 3 mm. Smaller floc may not settle. Similarly, larger floc does not settle as well, and is more subject to break up in the flocculation basin. Hence, if flocculation is continued beyond the optimum period (varying from 15 to 30 minutes—extending up to 45 minutes in extreme cases), larger floc may form, which may break, defeating the purpose of flocculation. Hence choice (*c*) is correct. **Ans.**

Q. 183. Chlorine, as we all know, effectively kills a large variety of microbial water born pathogens, including those that cause typhoid fever, dysentery, cholera and legionnaires disease. Chlorine is widely credited to have virtually eliminated the outbreaks of water born diseases, even in developed countries like USA, where typhoid fever used to kill about 25 per 1,00,000 population every year, before the introduction of chlorine for disinfecting public water supplies in 1908.

Chlorine is currently being used almost universally for disinfection of public supplies throughout the world. Even USA, uses chlorine for disinfection of public supplies in more than 98% of all U.S. water utilities that disinfect water.

Although chlorin's value as a disinfectant has been known for nearly a century, the mechanism by which the compound kills or inactivates the microorganisms is not clearly understood. Researchers from their research from 1940s to 1970s postulated that chlorine, which exists in water as hypochlorite and hypochlorous acid, reacts with biomolecules in the bacterial cell to destroy the organisms.

Further research led to the so-called ***"multiple hit" theory of chlorine inactivation.*** It asserted that bacterial death probably results from chlorine

attacking a variety of bacterial molecules or targets, including enzymes, nucleic acids, and membrane lipids. *Further research has proclaimed that chlorine acts by attacking cell wall. It is postulated that chlorine exposure destroys the cell wall*—by altering it physically, chemically and biochemically, thereby terminating the vital functions of cells, killing the micro-organisms.

A possible sequence of events during chlorination would be :

(*i*) disruption of the cell barrier by reactions of chlorine with target sites on the cell surface.

(*ii*) release of vital cellular constituents from the cell.

(*iii*) termination of membrane – associated functions.

(*iv*) termination of cellular functions within the cell.

It, therefore, becomes clear that the cell walls are penetrated by chlorine, and the DNA (*i.e.* nucleic acid) and enzymes are destroyed killing the micro-organisms, and there is no suffocation of the cells. Hence choice (*d*) is correct. **Ans.**

Q. 197. As explained in the text, *statement-1* is perfectly correct, since the optimum fluoride concentration of about 1 mg/l in drinking water helps in preventing dental cavities in children. However, *statement-2* seems incorrect, since fluorides, during formation of permanent teeth in children, makes the tooth structure stronger and more resistant to acid attacks. Fluoride strengthens tooth enamel, making it more resistant to tooth decay. It also reduces the amount of acid that the bacteria on teeth produce. It also helps repair the early stages of tooth decay (before formation of a cavity). *Fluoride infact, strengthens teeth by penetrating the tooth structure and replacing the lost minerals to repair acid damage.*

No where the fluoride is mentioned to result in softer and weaker teeth ; and also the softer and weaker teeth can not be more resistant to decay. The given statement-2, thus seems to be self contradictory, and may hence be treated as in correct. The choice at (*c*) is hence chosen as the correct choice. **Ans.**

Q. 200. Statement-I is evidently correct. Heavy algel growth occurs due to photosynthesis ; and carbon dioxide absorption during photosynthesis couses the water to become alkaline with high OH ions. Thus, statement II is not correct. Hence, the choice (*c*). **Ans.**

Q. 201. Critical time (T_c), *i.e.* the max. time for quick (rapid) closure of valve, is given by Eq. (6.80) as :

$$T_c = \frac{2S}{U_p} = \frac{2 \times 1920 \text{ m}}{960 \text{ m/s}} = 4 \text{ s.}$$

Hence, choice (*c*) is correct. **Ans.**

Q. 210. Total hardness = Carbonate hardness + Non carbonate hardness

$\therefore$ 120 mg/l = 50 mg/l + Non carbonate hardness

$\therefore$ Non carbonate hardness = 70 mg/l

Also, the carbonate hardness (*i.e.*, 50 mg/l given) is equal to the alkalinity or the total hardness which ever is less (Pl. see page 502).

$\therefore$ Carbonate hardness = Alkalinity = 50 mg/l

Hence, choice (*c*) is correct. **Ans.**

Q. 211. Follow Solved example 2.2 and compute as :

Year	*Population*	*Increases in population*
1960	2,50,000	
		30,000
1970	2,80,000	
		60,000
1980	3,40,000	
		80,000
1990	4,20,000	
		70,000
2000	4,90,000	Σ = 2,40,000

Average increase per decade $\bar{x} = \frac{2,40,000}{4} = 6000$

Future population of 2020 (*i.e.*, 2 decades after the year 2000) = ?

Using $P_n = P_0 + n\ \bar{x}$ ...(Eq. 2.1)

we have $P_{2020} = P_{2000} + 2 \times 60000 = 4,90,000 + 1,20,000 = 6,10,000$

Hence, choice (*c*) is correct. **Ans.**

Q. 214. In coagulation and flocculation, the mixing of coagulant chemical with raw water, does act on the colloids present in raw water, as to cause their agglomeration and settlement. The mechanisms, which can cause such agglomeration of smaller particles, can be divided into the following four categories;

(*i*) *Sweep coagulation;*

(*ii*) *Adsorption and charge neutralization;*

(*iii*) *Adsorption and Interparticle Bridging;* and

(*iv*) *Ionic layer compression.*

These mechanisms are briefly discussed below :

(*i*) **Sweep Coagulation.** As discussed in the text, the coagulant such as alum or ferric chloride is used as a coagulant, which in association with natural or man-added alkalinity (such as calcium bicarbonate) forms a gelatinous precipitate or floc of Aluminium hydroxide, *or* ferric hydroxide, which attracts the smaller particles and colloids present in raw water, to form a bigger and heavier ***sweep floc***, which settles down and is removed. This is the most commonly adopted mechanism of coagulation in water treatment plants. Since the floc sweeps away the fine suspended matter, it is known as ***sweep floc***, and the coagulation so caused is known as ***sweep coagulation***.

In ***sweep coagulation***, the optimum coagulant dose is inversely proportional to the colloids concentration, because the colloidal particles will be acting as a nucleus, and so the precipitation will be much faster. Thus, if we have very little colloids in raw water, there will not be any nucleus available, and more alum will be required to form the aluminium hydroxide precipitate. Hence, dose of alum will have to be optimised and determined experimentally (Jar test).

(*ii*) **Adsorption and Charge Neutralization.** In this mechanism, the +vely charged aluminium ions*, resulting from the dissociation of alum in raw water, do react with the –vely charged colloids in raw water, to neutralise their charge, thereby helping the colloids to come closer and agglomerate to larger particles, to finally settle down and get removed.

The dose of alum in this mechanism must however, be well controlled, since over-dose will again result in giving +ve charges to the colloids, which will repel each other due to similar charges, leading to their separation inplace of agglomeration.

*aqua metallic ions.

(*iii*) **Adsorption and Interparticle Bridging.** ***Synthetic organic polymers*** as coagulants can be used for removal of colloids from water inplace of coagulants such as alum, etc. Although anionic, or cationic or neutral (non-ionic) polymers can be used, yet the **anionic polymers are the most effective**, compared to others, and these polymers may be linear or branched, with high reactive surfaces. Several colloids may get attracted to one polymer; then several of the polymer colloidal group may become enmeshed, to improve the settling. But if the coagulant dose in high, then restabilization of colloids will take place. Moreover, low concentration of coagulant will remain insufficient for the effective removal of colloids.

Hence, an optimum dose of polymer coagulant is absolutely essential.

As against using synthetic polymers, even **natural polymers,** such as muringa *olifera seeds* have been found to be useful. Muringa olifera seeds, are nothing but the drumstick seeds that are usually used for cooking. If we make a paste of this drumstick seeds, and put it in water, we can see that the colloidal particles get settled. Thus, muringa seeds offer an example of a natural polymer.

(*iv*) **Ionic Layer Compression.** Although ionic layer compression is also one of the four categories of mechanisms for agglomeration of colloids, yet the same does not come under water treatment, because it is not involved in our usual water treatment processes. However, the same is explained below :

We know that diffused layer of ions surrounding the colloidal particles (Fig. 9.16) does not allow them to come closer. However, if we can decrease that diffused layer distance, the particles can come more and more closer.

In the *ionic layer compression,* the chemical compresses the ionic layer to bring the colloidal particles together. So how can we compress the ionic layer ? The ion surrounding a particle is a function of electrostatic potential, and hence if we can give high ionic strength by increasing the ionic concentration in the liquid, it will result in bringing more and more ions closer to the colloids, reducing the charge effect drastically, causing the diffused layer length to reduce. If the diffused layer is sufficiently compressed, then the Van der Waals force predominates, and particles get attracted. This is what happens in ionic layer compression.

An example of ionic layer compression is offered when a turbid stream flows into the ocean to form an estuary. In such a meeting length (estuary), the salinity of the river water suddenly increases, because the salty sea water and non-salty river water mixes together. This increases the ionic strength of the mixed water as compared to that of original river water by a large amount. Because of this high ionic strength, the *ionic* layer or the *diffused layer* of the colloidal particles (present in muddy river water) will get compressed. Due to

compression or reduction in diffused layer, Van der Waals force will pre-dominate to bring the colloidal particles together, and causing them to agglomerate. *That is why, deltas of sand and other agglomerated particle deposits are formed in estuaries.*

Thus, the colloidal particles which remain very stable in river water, become unstable when the river water becomes salty, due to increase in ionic strength causing compression of ionic layer, finally leading to settlement of these unstable particles.

The above explained ionic layer compression, however, does not play any major role in water treatment, because whatever chemicals we add as coagulants in water treatment, will have very low concentration*, as not to create high ionic strength, or ionic layer compression. Hence, this mechanism of ionic layer compression has no roll in water treatment.

With the above explanations of the four mechanisms, you can easily tick choice (c) as the correct choice to the Question. **Ans.**

Q. 216. Pl. see Solved Example 9.21 and compute as under :

Filtered water reqd. for the town = 1 ML/d

Assuming that 4% of filtered water is required for back washing the filter, we have

$$\text{Total filtered water reqd.} = \frac{1 \text{ ML}}{0.96} = 1.0417 \text{ ML/d}$$

Now, assuming that 0.5 hr is lost evryday in washing the filter, we have

$$\text{filtered water reqd. per hour} = \frac{1.0417 \text{ ML/h}}{23.5} = 0.0443 \text{ ML/h}$$

Now, the rate of filtration = 4000 L/hr/m^2 (given)

$$\therefore \text{ The area of filter reqd.} = \frac{(0.0443 \times 10^6)}{4000} \text{ m}^2 = 11.08 \text{ m}^2 \approx 12 \text{ m}^2$$

Hence, area of filter reqd. = **12 m^2.**

The under-drainage system of the above filter can be a "manifold and lateral system" below the filter bed for receiving the filtered water, and to allow back washing for cleaning the filter, as shown in Figs. 9.33 and 9.34. This consists of central manifold pipe, with laterals having perforations, at their bottom. To design this system, let us assume that the total area of the perforations in all the laterals is 0.2% of the total filter area.

$\therefore$ Total area of perforations in under-drainage pipes

$$= 12 \text{ m}^2 \times 0.2\% = 12 \times \frac{0.2}{100} = \mathbf{0.024 \text{ sq. m.}}$$

Now, assuming the area of each lateral

= 2 times the area of perforation in it,

(for 13 mm dia perforations), we have

Total area of laterals = 2 × Total area of perforations

$$= 2 \times 0.024 = \mathbf{0.048 \text{ m}^2}$$

We can hence choose choice (c) as correct. **Ans.**

*Of the order of 5 to 40 mg/l of alum, etc.

Q. 217. Pl. refer Solved Example 9.1, and compute as under :

Use Stoke's equation for settling velocity, v_s in cm/sec, as :

$$v_s = \frac{g}{18}\left[(G-1).\frac{d^2}{\nu}\right] \cdot \qquad \text{when } d < 0.1 \text{ mm}$$

Here $d = 2 \times 10^{-3}$ cm $= 2 \times 10^{-2}$ mm $= 0.02$ mm < 0.1 mm

Hence, equation (9.1) can be well used.

Substituting given values, we have

$g = 981$ cm^2/sec

$G = 2.65$

$d = 2 \times 10^{-3}$ cm

$\nu = 2 \times 10^{-2}$ cm^2/sec

$$\therefore \qquad v_s = \frac{981}{18}\left[(2.65-1)\frac{(2\times10^{-3})^2}{2\times10^{-2}}\right] \text{ cm/sec}$$

$$= \frac{981}{18} \times 1.65 \times \frac{4\times10^{-6}}{2\times10^{-2}} = \text{cm/sec}$$

$$= 179.85 \times 10^{-4} = \mathbf{0.018 \text{ cm/sec.}}$$

Hence, choice (a) is correct. **Ans.**

Q. 219. Hepatitis is a viral disease and not a bacterial disease. Hence, statement I is true, but statement II is false. Hence, choice (c) is correct. **Ans.**

Q. 230. **Dracontiasis** or *Dracunuliasis*, also called **Guinea worm disease** (GWD) is an infection by the *guinea worm*. Persons become infected when they drink water that contains *water fleas* infected with guinea worm larvae. Initially, there are no symptoms. About one year later, the person develops a painful burning feeling as the female worm forms a blister in the skin, usually on the lower limb. The worm then comes out of the skin over a few weeks. During this time, it may be difficult to walk or work. It is very uncommon for the diseases to cause death.

Humans and possibly dogs are the only known animals that guinea worms infect. The worm is about one to two millimeters wide and an adult female is 60 to 100 centimeters long (males are much shorter). Outside of humans, the eggs can survive up to three weeks. They must be eaten by water fleas before this. The larva inside water fleas may survive up to four months. Thus the disease must occur each year in humans to stay in an area. A diagnosis of the disease can usually be made based on the signs and symptoms of the disease.

Prevention is by early diagnosis of the disease, followed by keeping the person from putting the wound in drinking water, to decrease spread of the parasite. Other efforts include **improving access to clean water and otherwise filtering water if it is not clean. Filtering through a cloth is often enough.** Contaminated drinking water may be treated with a chemical called **temefos** to kill the larva. There is no medication or vaccine against the disease. The worm may be slowly removed over a few weeks by rolling it over a stick. The ulcers formed by the emerging worm may get infected by bacteria. Pain may continue for months after the worm has been removed. **Ans.**

19

Conventional Questions of Civil Services Exams. (Water Supply Engg. Portion)

The syllabus and examination scheme prescribed for Civil Services Examination (*revised wef year 2000*) is given in "*Water Resources Engineering (Vol. I)*", by the same author.

The various conventional questions that have been set during the last more than 30 years (since 1978) in this competitive examination, in Paper II-Civil Engineering, from the subject of "Water Supply Engineering", are however reproduced here, with hints and solutions where required.

Conventional Questions of the Previous Years of Civil Services Exams. On Water Supply Engg.

Year 1978 (*Civil Services*)

Q. 6. (*b*) *The following are the data given of a change in diameter effected in laying a water supply pipeline. The change in diameter is gradual from 20 cm at A to 50 cm at B. Pressures at A and B are 0.8 kg/cm² and 0.6 kg/cm², respectively with the end B being 3 metres higher than A. If the flow in the pipeline is 200 litres/sec, find*

(*i*) *Direction of flow ; and*

(*ii*) *The head lost in friction between A and B.* (30)

[**Solution.** Vel. through pipe $= \frac{Q}{A}$

Vel. head $= \frac{V^2}{2g}$

$\therefore$ Vel. head at $A = \dfrac{\left[\dfrac{200}{1000} \times \dfrac{1}{(\pi/4) \times (0.2)^2}\right]^2}{2 \times 9.81}$ m $= 2.06$ m

Vel. head at $B = \dfrac{\left[\dfrac{200}{1000} \times \dfrac{1}{(\pi/4) \times (0.5)^2}\right]^2}{2 \times 9.81}$ m $= 0.05$ m

Pressure head at $A = 0.8$ kg/cm² $= 0.8 \times 10^4$ kg/m²

$= \dfrac{0.8 \times 10^4}{1000}$ m of water $\left(i.e. \dfrac{p}{w}\right) = 8$ m

Pressure head at $B = 0.6$ kg/cm² $= 6$ m

Using Bernoulli's theorem between A and B.

$$8 + 2.06 + Z_a = 6 + 0.05 + (Z_a + 3) + H_L$$

or $H_L = 10.06 - 9.05 = \mathbf{1.01\ m.}$ **Ans.**

Since the total head at A is more than the head at B, evidently the flow will occur from A to B. **Ans.**]

Q. 11. (*a*) *Derive an expression for determination of permeability in a well with (i) unconfined aquifer and (ii) confined aquifer. What factors affect permeability of soil forming the aquifer ?*

(*b*) *For conducting permeability tests on a well which has an unconfined aquifer, two observation wells A and B are bored at distances 15 m and 30 m respectively from the centre of the well. When water is pumped at the rate of 5 litres per sec, it is observed that the elevations of the watertable above the impervious layer up to which the well is excavated are 12 m and 12.5 m respectively at A and B. Calculate the permeability of the aquifer in m/day.* (30)

[**Hint.** (*a*) Derive Eqns. (4.30) and (4.33) in terms of K

(*b*) Use Eq. (4.30) as

$$K = \frac{2.3\, Q \log_{10}(r_2/r_1)}{\pi(h_2^2 - h_1^2)}$$

where $Q = \dfrac{5}{1000}$ m³/sec

$r_1 = 15$ m $\quad h_1 = 12$ m

$r_2 = 30$ m $\quad h_2 = 12.5$ m

Hence, K will be equal to 9×10^{-5} m/sec. **Ans.**]

Year 1979 (*Civil Services*)

Q. 11. (*a*) *Fresh water containing nitrate ions, sulphate ions, and dissolved oxygen is sealed in a jar. In what sequence are these compounds reduced by the bacteria ? Where would obnoxious odour appear ?* (30)

[**Hint.** When fresh water *i.e.* untreated raw water, containing DO, NO_3 and SO_4 ions are sealed in jar, then the aerobic and facultative bacteria will start oxidising the organic mater present in raw water, depleting its dissolved oxygen. The oxidation process is given by Eqn. (19.1) as :

$$\text{Organics} + \text{Oxygen} \xrightarrow{\text{aerobics}} CO_2 + H_2O + \text{Energy} \quad \ldots(19.1)$$

Although, with reducing DO, the aerobes cannot continue to function, but then the anaerobics will start working by utilising the oxygen bounded in nitrate ions, releasing nitrogen gas, as given by Eqn. (19.2) :

$$NO_3 + \text{anaerobics} \longrightarrow N_2 \uparrow + CO_2 + \text{Energy} \quad \text{...(19.2)}$$

After the decomposing of nitrate ions, the next most accessible oxygen is in sulphate ions, which is utilised by anaerobics, releasing H_2S gas under Eqn. (19.3) :

$$SO_4^{--} + \text{Anaerobics} \longrightarrow H_2S \uparrow + CO_2 + \text{Energy} \quad (19.3)$$

The evolution of **H_2S gas** *will produce the obnoxious odour of rotten gas.*

During the above process, other anaerobic and facultative bacteria will partially decompose the organic matter to *organic acids, alcohols,* and *other compounds. The organic acids,* so formed, may further decompose to carbon dioxide and **methane,** if *methane forming bacteria* are also present, thus,

causing further evolution of pungent smell of methane gas.]

Q. 12. *(a) A surface water treatment plant coagulates a raw water having a turbidity of 9 Jackson candle units by applying an alum dosage of 30 mg/l. Estimate the total sludge solids produced in grams per cubic metre of water processed. Compute the volume of sludge from the settling basin and filter backwash water using 1.0% solid concentration in the sludge and 500 mg/l of solids in the wash-water.*

Assume that 30% of the total solids are removed in the filter. Use the following equation.

$$\text{Sludge solids (mg/l)} = \frac{\text{Alum dosage (mg/l)}}{4} + \text{raw water turbidity in Jtu.} \quad (30)$$

[**Note.** *Refer solved example 9.23*]

Year 1980 (*Civil Services*)

Q. 18. *Design a rapid sand filter unit for a population of 1,00,000 which is to be served by a 200 litres/head/day water supply.*

Assume rate of filtration $= 3 \times 10^5\ m^3/ha/day$
Amount of wash water reqd. $= 5\%$ *filtered water*
Dimensions of one filter unit $= 17.5\ m \times 10\ m.$ (60)

[**Hint.** Follow Solved Example 9.21 and work out :

Total filtered water reqd.
$= 1.05\ [1,00,000 \times 200]$ l/day
$= 21$ Ml/day $= 21000$ cu m/day.

$$\text{Filter area reqd.} = \left[\frac{21000}{3 \times 10^5}\right] \text{ ha}$$

$$= \frac{7}{100} \times 10000\ m^2 = 700\ m^2$$

$$\text{No. of units reqd.} = \frac{700}{17.5 \times 10} = 4$$

Other parts may also be designed. **Ans.**]

Q. 19. *(c) Fresh water containing nitrate ions, sulphate ions, and dissolved oxygen is placed in a sealed jar. In what sequence are these oxidised compounds reduced by the bacteria ? When would obnoxious odours appear ?* (15)

(e) What are the functions of water storage in a water distribution system ? (7)

Q. 20. *(a) The source of water supply for a town is deep tube well. What is the essential treatment to be given to the water before supplying to the town ?* (10)

(b) Draw the layout of a water treatment plant of a city with river as the source. Label various units. (10)

[**Note.** *Refer Fig. 13.1*]

(c) Results of chlorine demand test on a raw water are as given below :

Sample No.	*Chlorine Dosage mg/l*	*Residual chlorine after 10 min contact (mg/l)*
1	*0.2*	*0.19*
2	*0.4*	*0.36*
3	*0.6*	*0.50*
4	*0.8*	*0.48*
5	*1.0*	*0.2*
6	*1.2*	*0.4*
7	*1.4*	*0.6*
8	*1.6*	*0.8*

Sketch a 'chlorine demand curve'. What is the break point dosage and what is the 'chlorine demand' at dosage of 1.2 mg/l ? (40)

[**Note.** *Refer Solved Example 9.29*]

Year 1981 (*Civil Services*)

Q. 13. *(a) Water is to be supplied to three cities. The source and the quality of raw water are given below :*

City	*Source of Supply*	*Quality of raw water*	*Suggested treatment*
A	*Deep tubewell*	*Clean, no turbiduty, no colour*	*?*
B	*River*	*Highly turbid and coloured*	*?*
C	*River*	*Moderate turbidity and colour, contains very high quantity of Ca and Mg Sulphates and bicarbonates and high fluoride*	*?*

Suggest the most efficient method of water treatment. The answer should be precise with reason. (30)

[**Hint.** (*A*) Only *chlorination* may be required.

(*B*) *Sedimentation with coagulation, filtration* and *chlorination* may be required.

(*C*) *Filtration, softening* with line-soda process with the addition of magnesium oxide or hydroxide, and *chorination* may be required.]

Q. 13. (*b*) *Two primary settling basins are 26 m in diameter with a 2.1 m side water depth. Single effluent weirs are located on the pheripheries of the tank. For a water flow of 26000 m^3/d, calculate*

(*i*) *Surface area and volume ;* (*ii*) *Overflow rate in m^3/m^2. d ;*

(*iii*) *Detention time in hours ; and* (*iv*) *Weir loading in m^3/m day.* (30)

[**Note.** *Refer Solved Example 9.7*]

Q. 16. (*c*) *In precipitation water softening, lime is used to remove calcium harness by the reaction :*

$$CaO + Ca(HCO)_3)_2 = 2\ CaCO_3 + H_2O$$

what dosage of lime with 78% purity of CaO is required to combine with 70 mg/l of calcium ?

[*Given : Ca = 40 ; O = 16 ; H = 1 ; C = 12*] (20)

[**Hint.** 100 mg/l of $CaCO_3$ hardness requires 56 mg/l of pure lime i.e. CaO

$\therefore$ 70 mg/l of $CaCO_3$ hardness will need CaO

$$= \frac{56}{100} \times 70 \text{ mg} = 39.2 \text{ mg/l of pure CaO}$$

Impure lime reqd. $= \frac{39.2}{0.78}$ mg/l = **50.3 mg/l. Ans.**]

Year 1982 (*Civil Services*)

Q. 13. *Write brief but relavant answers for :* (20)

(*a*) *Explain and distinguish among the following :*

(*i*) *Prechlorination ;* (*ii*) *Break point chlorination ;*

(*iii*) *Double chlorination.*

(*b*) *Outline the various sources of water used in city water supply schemes and critically evaluate their merits and demerits from the view points of quality and quantity.* (20)

(*c*) *Describe the various layouts of distribution networks in a water supply system and state their relative merits and demerits. Suggest with reasons which of the systems is appropriate for rural areas.* (20)

Q. 14. (*a*) *Discuss why the size of population of a community should influence appreciably the per capita consumption of water. Also discuss factors effecting effective recycling of water.* (10)

(*b*) *Derive the expression for predicting the future (extrapolated) population of a community by the Geometric increase method. Determine the future population of a statellite town in the year 2011 by the G.I. method, given the following data :*

Year	*1951*	*1961*	*1971*	*1981*	.....	*2011*
Population in thousands	*93*	*111*	*132*	*161*	.....	*?*

(20)

[**Note.** *Refer Solved Example 2.5*]

Q. 15. (*a*) *Explain the main functions of service reservoir and describe the procedure of determining their capacity.* (10)

(*b*) *A clear water tank is receiving water from the treatment plant at a rate of 200 m^3/hr for 24 hours. The high lift pumps are lifting water from the same tank at the following rates 4—14 hr @ 120 m^3/hr, 15—24 hr @ 400 m^3/hr. Determine the capacity required for the clear water tank.* (15)

[**Note.** *Refer Solved Example 10.8*]

Q. 16. (*a*) *Discuss the scope and importance of sanitary survey in water supply scheme.* (15)

(*c*) *Describe the design procedure of a rapid sand filter with illustrative sketch.* (15)

(*d*) *Where and why do you soften the water ? Describe clearly the lime-soda process of softening water.* (15)

(*e*) *Explain the residual head method for the design of water supply plumbing system in a building.* (15)

Year 1983 (*Civil Services*)

Q. 13. (*d*) *Draw a neat sketch of a slow sand filter and label its parts.* (20)

Q. 15. (*c*) *Name the three important coagulants used in the primary treatment of water. Give the mechanism of their functioning.* (25)

Q. 16. (*a*) *Calculate the yield of a tubewell 40 cm in diameter if the depth before and after the pumping are 11 m and 5 m, respectively. Given the radius of influence zone as 320 m and K as 2400 m/day.* (15)

[**Hint.** Use Dupuit's equilibrium formula Eqn. (4.36) *i.e.*

$$Q = \frac{\pi K[d^2 - h_w^2]}{2.3 \log_{10} \frac{R}{r_w}}$$

where K = 2400 m/day = 0.0277 m/sec

d = depth before pumping = 11 m

h_w = depth after pumping = 5 m
R = 320 m
r_w = radius of well = 0.2 m

$\therefore$ Q = 1.136 cu m/sec. **Ans.**]

Q. 16. (*b*) *Where do you recommend an infiltration gallery ? Draw a neat sketch of it.* (15)

(*c*) *Explain how peak hourly demand of water is calculated.* (15)

(*d*) *Sketch and describe a 'Reservoir intake'.* (15)

Year 1984 (*Civil Services*)

Q. 13. (*c*) *Write the acceptable standards of purified water.* (20)

(*d*) *Design a rapid sand filter to treat 2.25 Ml/d water supply for a town ; given*

(*i*) *filter works for 12 hours a day*

(*ii*) *rate of filtration is 1250 Ml/d/ha.* (20)

[**Hint.** Water required = $\frac{2.25 \times 10^6}{10^3}$ m³/day

Since filter works only for 12 hrs a day ;

Filter output required = $\frac{2.25 \times 10^3}{12}$ m³/hr

Rate of filtration = 1250 Ml/d/ha

$= \frac{1250 \times 10^6}{10^3 \times 24}$ m³/hr/ha.

$\therefore$ Filter area reqd. $= \left[\frac{\frac{2.25 \times 10^3}{12}}{\frac{1250 \times 10^6}{10^3 \times 24}}\right]$ ha

= 36 m². **Ans.**]

Q. 16. *Design a bell mouth canal intake for a city of 75,000 persons, drawing water from a canal which runs only for 10 hours a day with a depth of 1.5 m. Calculate the head loss in the intake conduit if the treatment works are $\frac{1}{4}$ km away. Draw a neat sketch of the canal intake. Given, average consumption per person = 150 l/d.*

Assume the velocity through the screens and bell mouth to be less than 16 cm/s and 32 cm/s, respectively. (60)

[Note. *Refer Solved Example 5.2*]

Year 1985 (*Civil Services*)

Q. 13. (*c*) *Discuss the factors affecting the per capita demand of water supply. Also explain why it increases with the population size of the city. Give acceptable limits of turbidity, fluorides, and phenolic substances in potable water and explain the deleterious effects when these limits are exceeded.* (20)

Q. 14. (*a*) *Explaining why water is softened, describe with sketches the zeolite process of softening hard water.* (12)

(*b*) *Distinguish between 'standard' rate and 'high' rate trickling filters indicating the range of design parameters for both. Also evaluate their relative performances.* (12)

(*c*) *A filter unit is 4.5 m by 9.0 m. After filtering 10,000 cum per day in 24 hour period, the filter is back-washed at a rate of 10 l/m²/sec for 15 min. Compute the average filtration rate and percentage of treated water used in washing, and the rate of wash water flow in each trough. Assume 4 troughs.* (24)

[Note. *Refer Solved Example 9.22*]

Q. 15. (*a*) *For a continuous flow settling tank 3 m deep and 60 m long, design the flow velocity of water for affecting removal of 0.025 mm particles at 25°C. The specific gravity of particles is 2.65 and kinematic viscosity for water may be taken as 0.01 cm²/sec.* (20)

[Note. *Refer Solved Example 9.6*]

(*b*) *Describe briefly the theories of chemical coagulation.* (10)

(*c*) *Explain why chlorination is considered necessary even when filtration is provided. What is double chlorination ?* (10)

(*d*) *Enumerating the causes for wastage of water in public water supplies, describe briefly methods for their prevention ?* (10)

Q. 16. (*c*) *You are to lay 200 mm water main in a street in which there are reported to be 500 mm storm sewer and a 250 mm sanitary sewer. By sketches of working drawing indicate the locations of these pipe structures and explain the precautions in laying them.* (10)

(*d*) *Stating the requirements of water distribution system, evaluate the two principal types indicating the conditions for which each type is suited.* (15)

Year 1986 (*Civil Services*)

Q. 13. (*a*) *List at least six indicators used in measurement of pH in a water supply organisation. Indicate either the respective pH range values or the respective colour changes that occur.* (20)

(*b*) *What is meant by coagulation ? What are the common coagulants used ? Describe the chemical reaction involved and indicate the formula (or equation) for the chemical reaction, the flocs that form ; explain the reactions by stages, if any.* (20)

Q. 15. (*a*) *What are the reasons for tastes and odours to occur in water supply ? List at least six methods for water treatment for removal of tastes and odours.* (15)

(b) What does Clark process achieve in water treatment ? Indicate the chemical reactions involved. (15)

(c) How are water-borne diseases classified ? List at least five such diseases indicating them under the respective class as the case may be. (15)

(d) Derive the basic formula used in alternatively correcting the assumed or estimated discharges by Hazen-William method in pipe network. (15)

Q. 16. *(b) Design a rapid sand filter system for a water supply of 9 M.l.d. to a township. All the principle components shall be designed. Enumerate your assumptions during the design steps.* (30)

[**Hint.** *Follow Solved Example 9.21*]

Year 1987 (*Civil Services*)

Q. 13. *(c) List the several distinctions between slow sand filters and rapid gravity filters used in treatment works in water supply.* (20)

(d) A compound piping system consists of 1800 m of 50 cm, and 1200 m of 40 cm, and 600 m of 30 cm pipes of the same material, connected in series. (i) what is the equivalent length of a 40 cm pipe of the same material, (ii) what is the equivalent size of a pipe 3600 m long, (iii) if the three pipes are in parallel, what is the equivalent length of 50 cm pipe ? (20)

Q. 14. *(c) A centrifugal pump running at 1450 rpm has the characteristics given below :*

Discharge lps	*11.3*	*16.9*	*22.6*	*28.3*	*34.0*	*39.6*	*45.2*
Head, m	*25.8*	*25.0*	*24.1*	*23.2*	*21.4*	*18.9*	*15.8*
Efficiency %	*65*	*70*	*73*	*74*	*72*	*69*	*62*

Draw the operating characteristics of the pump and determine its specific speed. (15)

[**Note.** *Refer Solved Example 7.1*]

Q. 15. *(a) How are impurities in water classified ? What are their respective causes and their effects.* (20)

(c) Indicate the complete set of design criteria for the design of a rapid gravity sand filter. (20)

Q. 16. *(a) Other than chlorination, what are the different methods of disinfection of drinking water ? Write briefly on each of them.* (18)

Year 1988 (*Civil Services*)

Q. 13. *(a) Explain any three methods of forecasting the future population of a city. Discuss their relative merits.* (20)

(b) State the permissible limits for the following in the drinking water :

(i) turbidity *(ii) chlorides*

(iii) nitrates *(iv) hardness, and*

(v) flourides.

What are the harmful effects when the permissible limits are exceeded ? (20)

Q. 14. *(a) Discuss the different physical tests for determining the suitability of drinking water.* (20)

(b) Explain with neat sketches, the working of a rapid sand filter (20)

(d) Describe the method of softening of water by the zeolite process. (20)

Q. 15. *(c) State the factors affecting the efficiency of sedimentation tanks. With the help of a neat sketch, describe a sedimentation tank.* (20)

Q. 16. *(b) The population of a town is 1,00,000 and the average per capita demand is 135 litres / day / capita. Design the coagulation-cum sedimentation tank for supplying water to the town. The maximum demand may be taken as 1.5 times the average demand. Detention period is 5 hours for settling tank and 30 minutes for flocculent chamber. Flow rate is 900 litres / hour / m² of plan area.* (30)

[**Hint.** *Follow similar Solved Example 9.17 and determine :*

(A) Design of Sedimentation Tank

(*i*) Maximum daily water consumption

$= 1.5 \times (1,00,000 \times 135)$ l/d

$= 20.25 \times 10^6$ l/d $= 20.25 \times 10^3$ m³/d.

(*ii*) Quantity to be treated in 5 hr detention time

$$= 20.25 \times 10^3 \times \frac{5}{24} \text{ m}^3 = 4218.75 \text{ m}^3.$$

$\therefore$ *Capacity of Tank* = **4218.75 m³**

Overflow rate $= \dfrac{Q}{BL} = 900$ l/hr/m² (given)

$$\therefore \quad BL = \left[\frac{4218.75 \times 1000 \text{ l/hr}}{900 \text{ l/hr/m}^2}\right] = 937.5 \text{ m}^2.$$

Using 12 m width (B), $L = 78$ m

$$\therefore \quad \text{Depth} = \frac{4218.75}{78 \times 12} = 4.5 \text{ m}.$$

Hence, use 78 m × 12 m × 4.5 m tank, with extra depth for sludge storage, say use 5.0 m depth at starting end ; and $5 + \dfrac{78}{50} =$ 6.56 m at the *d/s* end (using *1* in 50 slope). Use a free-board of 0.5 m above *W.L.* **Ans.**

(B) Floc Chamber Design. Use 2.5 m effective depth ($\frac{1}{2}$ of depth of sedimentation tank).

Detention time given is 30 minutes.

$$\text{Capacity} = \frac{20.25 \times 10^3}{24}\left(\frac{30}{60}\right) \text{m}^3 = 421.88 \text{ m}^3$$

$$\text{Plan area} = \frac{421.88}{2.5} = 168.75 \text{ m}^2$$

$B = 12$ m (same as in sedimentation tank)

$$\therefore \quad L = \frac{168.75}{12} = 14 \text{ m}$$

Use 14 m × 12 m × 2.25 m floc chamber. **Ans.**]

Year 1989 (*Civil Services*)

Q. 13. (*a*) *What are the sources of temperature increases in natural water bodies ? What are the impacts of such elevated temperatures ?* (20)

[**Hint.** The major source of temperature increase in natural water bodies, like reservoirs, tanks, lakes, rivers, etc. is the heat produced by *decomposition of organic matter either by aerobic or anaerobic oxidation processes*. Although of course, other natural causes like increased atmospheric temperature, may also increase their temperatures.

The major impact of temperature increase in waters is that the *dissolved oxygen (DO) content of water gets reduced*. Due to reduction in its DO, the water becomes more susceptible to contamination, even with the addition of smaller quantities of sewage or organic matter. The fish needing more oxygen for their survival, may also become extinct, due to excessive temperature increases. **Ans.**]

Q. 13. (*b*) (*ii*) *What is an indicator organism ? List the characteristics required in an ideal pathogen indicator. Which organisms most nearly exhibit these characteristics ?* (8)

[**Hint.** *Study article 8.2.3 and work out as follows :*

An **indicator organism** is that organism whose presence/ detection in water, leads us to presume that water has been contaminated. It also helps us to reflect the type and degree of contamination.

Characteristics. (*i*) An ideal pathogen indicator should therefore always be present whenever pathogens are present in water and *vice-versa*.

(*ii*) It should in itself, be not a pathogen, so as not to cause hazards to laboratory technicians ;

(*iii*) It should be easily applicable to all types of waters ; and

(*iv*) It should be easily detectable on easy testing, without being interfered with by other organisms.

Examples. (*i*) The **E-coli** (*Escherichia coli*) organisms ; belonging to the *fecal coliform group of organisms** is the principal ***pathogen indicator.***

(*ii*) Another group of coliform organisms particularly **Helminths**, *found in the soils and decaying vegetations*, also serve as good pathogen indicators.

*Found in the feces of warm blooded animals (including humans). They are non-pathogens, and survive for longer duration than most of the pathogens.

(*iii*) **Total Coliform Count** or **M.P.N.** of **Coliforms** (*including both of fecal as well as non-fecal coliforms*) also serve as a good-pathogen indicator to indicate the sanitary quality of water. **Ans.**]

Q. 13. (*b*) *Write brief notes on the following items in the context of a water distribution system :*

(*i*) *Service reservoir* (*ii*) *Leak detection*

(*iii*) *Plastic pipes* (*iv*) *Laying of pipes.* (20)

Q. 14. (*a*) *Describe the process of sedimentation with coagulation, as adopted in water treatment plants. Include in your answer the factors affecting coagulation, the list of commonly used chemicals, and the description of mixing devices and flocculators.* (20)

Q. 14. (*c*) *Describe briefly the Hardy-cross method of analysis of flow in pipe networks.* (20)

Q. 16. (*c*) *Describe with neat sketches the various valves used in urban water distribution system.* (20)

Year 1990 (*Civil Services*)

Q. 13. (*i*) *Write brief notes on :*

(*i*) *Leakage in water distribution system*

(*ii*) *Break point chlorination*

(*iii*) *Water softening*

(*iv*) *Removal of taste and odour from water.* (20)

Q. 13. (*d*) (*i*) *The drinking water needs of some of the communities are to be met from the following sources :*

(*i*) *Alluvial river* (*ii*) *Reservoir*

(*iii*) *Mountainous river.*

Briefly describe the important problems that may be encountered at the intake in these sources. Sketch and describe a typical river intake. (20)

Q. 13. (*d*) (*ii*) *State the WHO international water quality standards relating to the presence of :*

calcium ; chloride ; copper ; iron ; total solids. (10)

Q. 14. (*a*) *A water supply main trifurcates at a junction point J into three branches, each feeding a separate reservoir. The details of the pipes and reservoirs are as follows :*

Pipe	*Diameter*	*Length*	*Darcy Weisbach friction factor (f)*	*Terminal reservoir*	*Reservoir elevation in m*
JA	*20 cm*	*2.0 km*	*0.02*	*A*	*80.00*
JB	*20 cm*	*2.5 km*	*0.02*	*B*	*70.00*
JC	*20 cm*	*3.0 km*	*0.02*	*C*	*60.000*

If the inflow from the main at the junction is 0.250 m^2/s, determine the delivery into each reservoir. (30)

[**Hint.** *Refer Solved Example 6.8*]

Q. 15. (*b*) *A rectangular sedimentation basin is to handle 10 million litres/day of raw water. A detention basin of width to length ratio of $\frac{1}{3}$ is proposed to trap all impurities larger than 0.04 mm in size. Assuming a relative density of 2.65 for the particles and 20°C as the average temperature, compute the basin dimensions. If the depth of the tank is 3.5 m, calculate the detention time.* (30)

[**Hint.** *Refer Solved Example 9.8*]

Q. 15. (*c*) *Describe the salient features of a tubewell. What are the possible causes of failure and what remedial measures can be adopted to prevent such failures.* (20)

Q. 16. (*a*) *The waterworks of city treats 50 million litres/day of water in a coagulation-sedimentation tank. The quantity of alum consumed is 15 mg/litre of water. If the alkalinity of raw water is equivalent to 1.0 mg/litre of $CaCO_3$, determine the quantity of alum and quicklime (containing 80% CaO) required per month by the waterworks.* (10)

[**Solution.** *Follow exactly similar Solved Example 9.14*, and compute :

Qty. of alum reqd. daily

$$= 50 \times 10^6\ l \times 15\ \text{mg/l}$$

$$= \frac{750 \times 10^6}{10^6}\ \text{kg} = 750\ \text{kg}$$

(*i*) Alum consumed per month

$$= 750 \times 30\ \text{kg} = 22500\ \text{kg} = \mathbf{22.5\ tonnes.}\ \textbf{Ans.}$$

(*ii*) M. mass of alum = 666

M. mass of $Ca(HCO_3)_2$ = 162

M. mass of $CaCO_3$ = 100.

3 moles of $Ca(HCO_3)_2$ reacts with one mole of alum.

∴ *3 × 100 parts of alkalinity (as $CaCO_3$) is thus, reqd. by 666 parts of alum ; i.e.*

Alkality (as $CaCO_3$ reqd. to fully react with 15 mg/l of alum

$$= \frac{3 \times 100}{666} \times 15\ \text{mg/l} = 6.757\ \text{mg/l.}$$

Alkalinity of raw water = 1.0 mg/l

Additional outside alkalinity reqd. = 6.757 – 1.0

$$= 5.757\ \text{mg/l as } CaCO_3.$$

Since 56 parts of pure CaO produces 100 parts of $CaCO_3$.

$$\text{Pure CaO reqd.} = \frac{5.757}{100} \times 56\ \text{mg/l} = 3.224\ \text{mg/l.}$$

∴ $$\text{CaO of 80\% purity reqd.} = \frac{3.224}{0.8}\ \text{over mg/l} = 4.03\ \text{mg/l.}$$

Total quick lime reqd. for treating 50 Ml of raw water daily

$$= \frac{4.03 \times 50 \times 10^6}{10^6}\ \text{kg} = 201.5\ \text{kg/day.}$$

∴ Monthly requirement of quicklime

$$= 201.5 \times 30\ \text{kg} = 6045\ \text{kg}$$

$$= \mathbf{6.045\ tonnes.}\ \textbf{Ans.}$$]

Year 1991 (*Civil Services*)

Q. 13. (*c*) *A pump is to deliver water from an underground tank against a static head of 40 m. The suction pipe is 50 m long and is of 25 cm diameter with Darcy-Weisbach friction factor f = 0.02. The delivery pipe is of 20 cm diameter, 1600 m long and has f = 0.022. The pump characteristics can be expressed as*

$$H_p = 100 - 6000\ Q^2$$

where H_p = pump head in metres, and

Q = discharge in m^3/s

Calculate the head and discharge of the pump. (20)

[**Note.** *Refer Solved Example 7.7*]

(*d*) *Write brief notes on :*

(*i*) *Water-borne diseases.* (10)

Q. 14. (*b*) *What are the objectives of well development ? List the commonly used methods of developing tubewells and describe briefly the backwashing techniques.*

Q. 15. (*b*) *Describe the salient characteristics of filter media used in :*

(*i*) *Slow sand filter* (*ii*) *Rapid sand filter*

(*iii*) *Dual media filter* (*iv*) *Multi media filter.* (30)

Q. 16. (*b*) *Describe, briefly the 'floating on line' type of balancing reservoir.* (10)

(*ii*) *The population of a town is 50,000 and its per capita demand of water is 150 litres per day. The hourly variation of the demand is given below in Table 1. Calculate the minimum capacity of the storage reservoir to be provided to meet this demand when the pumping is for 16 hours and at a constant rate during 10 PM to 6 AM and 10 AM to 6 PM.*

Table 1. Hourly Variation of Demand

Time period	*Percentage of average hourly flow*	*Time period*	*Percentage of average hourly flow*
11 PM—4 AM	*20*	*1 PM—2 PM*	*225*
4 AM—5 AM	*40*	*2 PM—5 PM*	*70*
5 AM—6 AM	*80*	*5 PM—6 PM*	*225*
6 AM—10 AM	*225*	*6 PM—8 PM*	*90*
10 AM—12 Noon	*100*	*8 PM—10 PM*	*70*
12 Noon—1 PM	*60*	*10 PM—11 PM*	*40*

(20)

Solution. Average daily water reqd.

$= 50{,}000 \times 150$ litres $= \mathbf{7.5\ Ml.}$

$\therefore$ Average hourly demand $(x) = \dfrac{7.5\ Ml}{24} = \mathbf{0.3125\ Ml/hr.}$

Total daily pumping for 16 hr only = 7.5 Ml

Hourly pumping rate $= \dfrac{7.5}{16}$ Ml/hr. $= \mathbf{0.46875\ Ml/hr.}$

The calculations are now carried out, in the **Solution Table 19.1**, as are done in other similar questions, and the values are found out as :

Max. excess of demand = 0.34375 Ml ...(i)

Max. excess of supply = 2.59375 Ml ...(ii)

Total storage reqd. = (i) + (ii)

= 2.9375 Ml. Ans.

Year 1992 (*Civil Services*)

Q. 13. (*b*) *Compare a rapid gravity filter with a slow sand filter inrespect of the following features :*

Rate of filtration ; size of bed ; length of run between cleaning ; and efficiency. (20)

Q. 14. (*a*) *How would you define hardness of water ? Describe a method of softening of water for industrial uses.* (20)

(*b*) *Discuss the possible causes of failure of tubewells in coastal environment.* (20)

(*c*) *Describe the process of sedimentation with coagulation adopted in water treatment plants. Indicate the factors affecting coagulation.* (20)

Q. 15. (*c*) *State the W.H.O. recommended water quality standards relating to the presence of :* (6)

(*i*) *total solids ;* (*ii*) *iron ;* (*iii*) *heavy metals.*

Year 1993 (*Civil Services*)

Q. 13. (*b*) *Discuss with neat sketches :* (10)

(*ii*) *Hydraulic devices for rapid mixing and flocculation.*

(*d*) *Write short notes on the following :*

(*ii*) *Removal of pathogens in slow sand filters* (5)

(*iii*) *Removal of chromium and Arsenic from water.* (5)

Q. 15. (*a*) *Differentiate between Trunk Main and Service Main in connection with water supply. Give specifications for materials and laying of water supply mains.* (15)

(*b*) *Explain Hardy Cross method of distribution network analysis. Illustrate your answer with a suitable example.* (15)

Solution Table 19.1: For Question 16 (b) (iii) of Year 1991 (*Civil Services*)

Time interval hr of day	*Duration of interval in hr*	*%age of av. hourly flow in each interval*	*Water demand in each interval (3) × (2).x*	*Cumulative water demand*	*Cumulative water demand using x = 0.3125 Ml/hr*	*Pumping in given interval Ml*	*Cumulative pumping Ml*	*Excess of demand (6) – (7) + ve values only Ml*	*Excess of demand (8) – (6) + ve values only Ml*
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
23—4	5	20%	1x	1.0x	0.3125	5 × 0.46875	2.34375	—	2.03124
4—5	1	40%	0.4x	1.4x	0.4375	1 × 0.46875	2.8125	—	2.375
5—6	1	80%	0.8x	2.2x	0.6875	1 × 0.46875	2.28125	—	**2.59375**
6—10	4	225%	9x	11.2x	3.5000	—	3.28125	0.21875	—
10—12	2	100%	2x	13.2x	4.1250	2 × 0.46875	4.21875	—	0.09375
12—13	1	60%	0.6x	13.8x	4.3125	1 × 0.46875	4.6875	—	0.375
13—14	1	225%	2.25x	16.05x	5.0156	1 × 0.46875	5.15625	—	0.14065
14—17	3	70%	2.1x	18.15x	5.6718	3 × 0.46875	6.5625	—	0.8907
17—18	1	225%	2.25x	20.4x	6.3750	1 × 0.46876	7.03125	—	0.65625
18—20	2	90%	1.80x	22.2x	6.9375	—	7.03125	—	0.09375
20—22	2	70%	1.4x	23.6x	7.3750	—	7.03125	**0.34375**	—
22—23	1	40%	0.4x	24x	**7.5000**	1 × 0.46875	**7.5000**	—	—
Check	Σ = 24								

Max. excess of demand = 0.34375 ...(i)

Max. excess of supply = 2.59375 ...(ii)

$\therefore$ **Reqd. Total Storage = (i) + (ii) = 2.9375 Ml. Ans.**

(c) *The result of chemical analysis of a sample of raw water is given below.*

Ca^{++}	$= 80\ mg/l$
Na^{+}	$= 2.5\ m.eq/l$
Total alkalinity	$= 80\ mg/l\ as\ CaCO_3$
Total hardness	$= 120\ mg/l\ as\ CaCO_3$
SO_4^{-}	$= 20\ mg/l\ as\ CaCO_3$
Cl^{-}	$= 140\ mg/l\ as\ CaCO_3$
NO_3^{-}	$= 5\ mg/l\ as\ CaCO_3$

(i) Prepare a bar diagram for the raw water.

(ii) Estimate the quantity in kg/day of CaO (90% pure) and Soda ash (95% pure) required to soften 4 million litres per day of this water. (30)

[Note. *Refer Solved Example 9.35*]

Year 1994 (*Civil Services*)

Q. 14. (*a*) *Determine the distribution of flow by Hardy Cross method in the pipe network shown in the figure 19.1. The head loss* $h_L = K \cdot Q^n$. *Assume suitable value of n. The flow in the network is turbulent and pipes are rough. The value of K for each pipe is indicated in the figure.* (30)

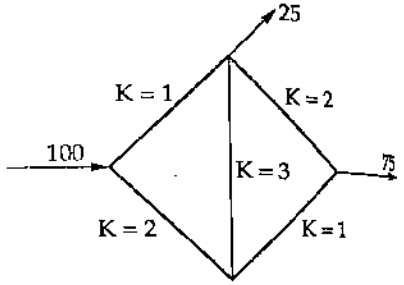


Fig. 19.1

[Hint. Follow the solution of a similar question given under the questions of Engineering Services Exam. 1996.]

Q. 15. (c) *What is meant by the process of coagulations ? Sketch a coagulation solution dosing system.* (15)

Year 1995 (*Civil Services*)

Q. 13 is compulsory. Answer any three sub-divisions.

Q. 13. (*b*) *It is required to treat water from surface source. Draw a flow diagram of conventional water treatment units indicating their names and purposes for installing them.* (20)

Q. 13. (*d*) *Distinguish between the following :* (10)

(i) Waste water treatment and water treatment

Q. 15. (*a*) *At a town of present population 50,000, water supply is provided through an overhead tank. Water is pumped into this tank at a uniform rate continuously for 24 hours of the day. Length of the supply mains is 500 m. For the demand pattern*

Hours	*Demand in % of 24 hr demand*
0—8	*5*
8—10	*40*
10—16	*20*
16—20	*25*
20—24	*10*

Determine the following :

(i) Daily demand in mld. Assume value of per capita demand.

(ii) Required capacity of overhead tank

(iii) Diameter of supply mains. Assume f = 0.01 and velocity through the mains as 2 m/s.

(iv) Minimum height of lowest water level in the tank to supply a terminal head of 15 m. (35)

Solution. (*i*) Daily demand

= Population × Per capita demand

= 50,000 × 160 l/c/d (assumed) = **7.5 MLD. Ans.**

(*ii*) Puming is done 24 hrs at uniform rate, hence 7.5 MLD water is pumped in 24 hrs.

$\therefore$ Uniform rate of pumping $= \frac{7.5}{24}$ ML/hr

$= 0.3125$ ML/h.

Demand and pumping values are analysed in **Solution Table 19.2**, to work out capacity of storage tank.

Solution Table 19.2 For Question 15 (*a*) of year 1995

Time hr of day	*No. of hr*	*Demand in M.L*	*Cumulative demand in M.L*	*Pumping in each interval in M.L 0.3125 ML × col (2)*	*Cumulative pumping*	*Excess of demand col (4) – col (6)*	*Excess of supply col 6 – col (4) + ve values only*
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
0–8	8	5% × 7.5 = 0.375	0.375	2.500	2.500	—	**2.125**
8–10	2	40% × 7.5 = 3.000	3.375	0.625	3.125	0.250	—
10–16	6	20% × 7.5 = 1.500	4.875	1.875	5.000		0.125
16–20	4	25% × 7.5 = 1.875	6.750	1.250	6.250	**0.500**	—
20–24	4	10% × 7.5 = 0.750	7.500	1.250	7.500	—	—

From Solution Table 19.2, we get

Max. excess of demand = 0.500 ML

Max. excess of supply = 2.125 ML ...(*i*)

∴ Total storage required = (*i*) + (*ii*) = 2.625 ML ...(*ii*)

Hence, the required capacity of overhead tank = **2.625 ML.** Ans.

(*iii*) Velocity (*V*) through the supply mains of dia *d* and 500 m length = 2 m/s,

where peak supply rate between 8–10 hrs

$$\text{is} = \frac{3.0\ \text{ML}}{2\ \text{hr}} = 1.5\ \text{ML/hr}$$

$$\therefore \quad Q = 1.5\ \text{ML/hr}$$

$$= \frac{1.5 \times 10^6}{10^3} \times \frac{1}{3600}\ \text{m}^3/\text{sec} = 0.416\ \text{m}^3/\text{s}$$

$$\text{Area of pipe} \quad = A = \frac{\pi}{4} \cdot d^2 = \frac{Q}{V} = \frac{0.416}{2\ \text{m/s}}\ \text{m}^3/\text{s}$$

$$\therefore \quad d = 0.52\ \text{m}.$$

(*iv*) Head lost in supply main at peak rate of flow

$$H_L = \frac{fLV^2}{2gd}$$

$$= \frac{0.01 \times 500 \times (2)^2}{2 \times 9.81 \times 0.52}\ \text{m} = 1.96\ \text{m}$$

Minimum height of lowest water level in tank to provide terminal head of 15 m

= 15 m + Head lost in supply pipe

= 15 m + 1.96 m

= 16.96 m ; say **17 m. Ans.**]

Q. 15. (*b*) *Describe in details the techniques to treat water to make it potable. Explain with schematic sketches the procedures for partial softening for removal of iron and manganese from such water.* (25)

Year 1996 (*Civil Services*)

Question 13 is compulsory. Answer any three subdivisions.

Q. 13. (*c*) *Draw plan, cross-section and longitudinal section of a slow sand filter, with dimensions, noting the sizes and depths of filtering material used?* (20)

Q. 14. (*a*) *Describe the process of sedimentation with coagulation adopted in water treatment plants. Design the dimensions of a suitable sedimentation tank for raw supplies from a town of population of 1,00,000 with a daily per capita water supply of 120 litres. Assume detention period of 6 hours and the velocity of flow as 20 cm/minute.* (35)

[**Hint.** The sedimentation tank can be designed for maximum daily demand ≈ 1.8 times the per capita demand of 120 litres, by following Solved Example 9.2, as below :

The max. daily demand = 216 l/c/d × 1 lakh = 21.6 MLD

Qty. of water to be treated in 6 hr (D.P.)

$$= 21.6 \times \frac{6}{24} = 5.4\ \text{ML} = 5400\ \text{m}^3$$

Capacity of tank = 5400 m³

Length of the tank = Vel. × Dct. period

$$= 0.2\ \frac{\text{m}}{\text{min}} \times (6 \times 60\ \text{min}) = 72\ \text{m}$$

$$A = \frac{5400}{72} = 75\ \text{m}^2$$

D = 4 m (assumed water depth)

$$B = \frac{75}{4} = 18.75\ \text{m}$$

Hence, use rectangular tank **72 m × 18.75 m × 4.75 m size. Ans.**]

Q. 15. (*a*) *A distribution reservoir is to be designed for a locality of a town for 900 persons. The average supply may be assumed as 250 litres/capita/day. The pattern of demand is as follows :*

7 am to 8 am	*30%% of day's supply*
8 am to 5 pm	*35% of day's supply*
5 pm to 6.30 pm	*30% of day's supply*
6.30 pm to 7 am	*5% of days supply*

The pumping to be done at a constant rate of 8 hours per day (8.am to 4 pm). Draw a mass curve and determine the capacity of the reservoir. (25)

[**Hint.** Please follow similar Solved Example 10.2]

Q. 16. (*b*) (*i*) *State the points of contrast in the design and using of settling tank used in water and waste water treatment.* (5)

[**Hint.** The raw water supplies which are sedimented in a water sedimentation tank usually contain inorganic solids like sand, silt, clay, etc. of sp. gravity 2.65 to 1.2 ; whereas, the sewage which is sedimented in a sewage sedimentation tank usually contain organic solids having sp. gravity 1.20 to 1.01.

The inorganic solids removed and settled in a water sedimentation tank will not pose as much problem as may be posed by the organic solids settled in a sewage sedimentation tank. The settled sludge of a sewage sedimentation tank,

being organic in nature, is thus likely to become septic and stale with the passage of time. *Longer detention periods cannot therefore be provided for sewage sedimentation tanks, as otherwise it may cause septic conditions leading to uplift of settled particles by the rising gases of decomposition.*

Additionally, the sewage solids usually behave as flocculent particles ; whereas the solids of natural raw water supplies usually behave as discrete particles. The laws of sedimentation and surface loading adopted in water treatment will, therefore, have to be altered, while applying them in sewage settling. The surface area, depth as well as detention time become quite important in sewage sedimentation tanks.

The design values of various parameters for both these types of settling tanks are indicated below :

S.No.	*Parameter*	*For water sedimentation tank*	*For sewage sedimentation tanks (plain)*
(1)	(2)	(3)	(4)
(*i*)	Detention time	4 to 8 hrs for plain sedimentation; and 2 to 4 hrs for coagulated sedimentation tanks	1 to 2 hrs
(*ii*)	Horizontal flow velocity in the tank (v) to compute A, as: (B.H.) = Q/v ...Eq. (9.11)	0.15 to 0.9 m/min, usually kept at 0.3 m/min	0.3 m/min (average value)
(*iii*)	Length of tank	= vel. of flow × detention time (4 to 5 times the width)	(4 to 5 times the width)
(*iv*)	Depth of tank* (overall) (*does not have any effect on efficiency of sediment removal)	3 to 4.5 m with 1.8 m as minimum and 6 m as maximum	3 to 4 m, generally not exceeding 3.5 m or so
(*v*)	Width of tank (B)	10 m with 12 m as maximum	6 m with 7.5 m as maximum
(*vi*)	Surface loading or overflow rate (Q/BL)	12000 – 18000 l/m²/d for plain sedimentation ; and 24000 – 30000 l/m²/d for coagulation-sedimentation	40,000 – 50,000 l/m²/d

Q. 16. (*c*) *What is meant by 'break point chlorination'. Discuss all stages of disinfection reactions that appear before it reaches break point.* (15)

Year 1997 (*Civil Services*)

Q. 13. *Answer any three subdivisions :*

(*a*) *Name the different types of pumps used generally in water supply schemes. What are the factors on which their selection depends ?* (20)

(*b*) *Explain the sedimentation process used in a water treatment plant. Draw neat sketch of a sedimentation tank in which coagulant is used.* (20)

(*c*) *Public water supply is conveyed from the source to the centre of supply by gravitational system or pumping system or combined system. Draw three separate longitudinal sections, indicating these systems and show the positions of various details in each.* (20)

Q. 15. (*a*) *Compare a rapid gravity filter with a slow-sand filter in respect of the following features :*

Rate of filtration, size of bed, length of run between cleaning, and efficiency.

Sketch a slow-sand filter. Briefly discuss recent development in filtration process for water treatment. (30)

(*b*) *For water supply of a town, water is pumped from a river 3 km away into a reservoir. The maximum difference of levels of water in river and the reservoir is 20 m. The population of the town is 50,000 and per capita water demand is 120 lit / day. If the pumps are to operate for a total of 8 hours and the efficiency of pumps is 80%, find the h.p. of the pumps. Assume friction factor as 0.03, the velocity of flow as 2 m / s and maximum daily demand as 1.5 times the average daily demand.* (30)

Hint. Follow similar Solved Example 6.4, and compute as under :

Max. daily demand

$$= 1.5\,[50{,}000 \times 120\ \text{L}] = 9 \times 10^6\ \text{l/day}$$

Now, 9×10^6 L *i.e.* 9×10^3 m³ of water is to be lifted by pumps in 8 hrs,

∴ Discharge to be lifted by pumps

$$= \frac{9 \times 10^3}{8 \times 60 \times 60}\ \text{m}^3/\text{s} = \mathbf{0.3125\ m^3/s}$$

Velocity of flow (v) from the pressure pipe = 2 m/s (given)

$$\therefore \quad \text{Area of pipe} = \frac{Q}{v} = \frac{0.3125}{2}\ \text{m}^2 = 0.15625\ \text{m}^2$$

$$\text{Dia. of pipe} \quad = \sqrt{\frac{4 \times 0.15625}{\pi}} = 0.446\ \text{m}$$

Now, head loss through the pipe

$$= \frac{f' L v^2}{2gd} = \frac{0.03 \times 3000 \times (2)^2}{2 \times 9.81 \times 0.446} = 41.14 \text{ m}$$

$\therefore \quad H_{L_1} = 41.14$ m

H_{L_2} = max. difference of water levels between river and storage reservoir = 20 m (given)

$\therefore$ Total head loss

$$= H_L = H_{L_1} + H_{L_2} = 41.14 + 20 = \mathbf{61.14 \text{ m}}$$

Required H.P. of pumps

$$= \frac{\gamma_w \cdot Q H_L}{0.735\eta} = \frac{9.81 \times 0.3125 \times 61.14}{0.735 \times 0.8} = 318.76$$

$$= \textbf{say 320 HP.} \quad \textbf{Ans.}$$

Year 1998 (*Civil Services*)

Question 13 is compulsory.

Q. 13. (*a*) *What is meant by pH value ? How is it determined ? What is its importance in water analysis ?* (20)

(*b*) *Draw the longitudinal section through a rapid sand filter with dimensions, also giving the sizes and depths of different filtering materials* (20)

Q. 14. (*a*) *Describe the cost-head loss ratio criterion method for optimal design of branched water distribution systems.* (20)

Q. 15. (*a*) *Draw a neat sketch of a 'check valve' and explain its working. Cite a practical situation in which provision of a check valve would be necessary.* (20)

(*b*) *Describe various factors which influence the selection of pumps for lifting water.* (20)

Q. 16. (*a*) *The waterworks of a town with a population of 20,000 meets its water demand at the rate of 135 lpcd. If the disinfection is done by bleaching powder having 45% available chlorine, determine the required quantity of bleaching powder per year. Required dose of chlorine at water-works is 0.003 ppm for disinfection. Also suggest the method of applying the prescribed dose of bleaching powder.* (20)

[**Hint.** Please refer similar Solved Example 9.27 and compute the answer to be 6.57 kg.]

(*b*) *State different impurities that may be present in water. Briefly describe how you will remove these impurities and make the water potable.* (20)

Year 1999 (*Civil Services*)

Question 13 is Compulsory.

Q. 13. (*a*) *Compare cast iron pipes with steel pipes and concrete pipes for urban water supply distribution system. Sketch and describe a spigot and socket joint for cast iron pipes.* (20)

(*d*) (*i*) *Suggest suitable methods of predicting demand of water for planning public water supply schemes.*

(*ii*) *The population figures of a town during the last four consecutive decades are : 20,000 ; 24,500 ; 29,500 ; 32,200 respectively. Predict the population in the next decade using incremental increase method.* (20)

Q. 14. (*a*) *What is an ideal settling tank ? An ideal settling tank with surface area of 100 m^2 receives an inflow of 20 million litres/day of raw water. Referring to the tabular data given below for particles of specific gravity 1.2, find out what percentage of each size would settle down in the tank :* (20)

Diameter of particles in mm	*Settling velocity in cm/sec*
0.5	*0.62*
0.4	*0.49*
0.3	*0.38*
0.2	*0.22*
0.1	*0.08*

[**Solution.** As discussed in article 9.5.2, the surface overflow rate (v_0) is given as :

$$v_0 = \frac{Q}{A_s}$$

or $$v_0 = \frac{Q}{\text{Surface area of settling tank}} = \frac{20 \text{ Mld}}{100 \text{ m}^2}$$

$$= \frac{\frac{20 \times 10^3}{24 \times 60 \times 60} \text{ m}^3/\text{s}}{100 \text{ m}^2} = \frac{0.23}{100} \text{ m/s} = 0.23 \text{ cm/s}.$$

Now, particles having settling velocity greater than or equal to v_0 will settle down in ideally settling horizontal tank to the extent of 100% ; while the lighter particles having settling velocity (v_s) lesser than v_0 will settle in the ratio of $\frac{v_s}{v_0} \times 100$ (as given by Eq. 9.14).

In the given question, the settling velocities of particles of sizes 0.5 mm, 0.4 mm and 0.3 mm are given to be more than v_0

i.e. 0.23 cm/sec ; and hence these particles will settle down 100%. For other two sizes, the settling velocities given are less than v_0 (*i.e.* 0.23 cm/s), and hence their percentage settlements are worked out as below :

Percent settlement of particles of 0.2 mm size having v_s of 0.22 cm/s $= \frac{0.22}{0.23} \times 100 = \mathbf{95.65\%}$.

Similarly, percent settlement of particles of 0.1 mm size having v_s of 0.08 cm/sec $= \frac{0.08}{0.23} \times 100 = \mathbf{34.78\%}$.

Hence, the percent settlement of various sized particles shall be as :

Particle size in mm	*0.5*	*0.4*	*0.3*	*0.2*	*0.1*
% age settlement	*100%*	*100%*	*100%*	*95.65%*	*34.78%*

Ans.]

Q. 14. (*b*) *What is meant by economic diameter of a rising main ? How will you determine the economic diameter of a rising main for pumping water from a filter plant to the service reservoir of a town water supply ?* (20)

Q. 15. (*a*) *A large perennial river is used as a source of water supply to a town with a population of 1,00,000. Indicate by sketches the general layout of the water works and show a detailed section of the intake.*

Q. 16. (*a*) *What are waterborne diseases ? Describe the MPN test for bacteriological analysis of water. State the bacteriological standards for drinking water.* (20)

Year 2000 (*Civil Services*)

Q. 5. (*f*) *Compulsory : For disinfecting water supply, it is required to treat 3,00,000 litres of daily supply with 0.5 ppm of chlorine. If the disinfectant is available in the form of bleaching powder containing 30% of available chlorine, calculate the amount of bleaching powder required to treat the daily flow of water.* (12)

[**Hint.** Please follow *exactly similar Solved Example 9.27*, and calculate :

Amount of chlorine reqd. daily

$= 0.5 \frac{\text{mg}}{l}$ (*i.e.* ppm) $\times (3 \times 10^5\ l)$

$= 1.5 \times 10^5$ mg $= 0.5$ kg

Amt. of bleaching powder reqd. (containing 30% chlorine)

$= \frac{0.5 \times 100}{30} = \frac{0.5}{0.3} = \mathbf{1.67\ kg.}$ **Ans.**]

Q. 6. (*a*) *Define storage coefficient and transmissibility coefficient of an aquifer.*

A well is 30 cm in diameter and fully penetrates 50 m below the static water-table. After 36 hours of pumping at steady state rate of 4 m^3/minute, the water level in a test well 200 m apart is lowered by 1.2 m ; and in a well 40 m away, the drawdown is 2.7 m. What is the transmissibility of the aquifer ? (20)

[**Hint.** Please follow *exactly similar Solved Question 4.11* and compute $T = \mathbf{0.71\ m^2/min.}$ **Ans.**]

Year 2001 (*Civil Services*)

Q. 5. (*a*) *What is the difference between a 'confined aquifer' and an 'unconfined aquifer' ? Define and describe Darcy's law of flow of water through permeable soils.* (12)

Q. 5. (*d*) *What is the necessity of treatment of water for drinking purposes ? Enumerate briefly the impurities in raw water and corresponding treatment method for making water fit for drinking.* (12)

Q. 7. (*a*) *For a large metropolitan city, how do we estimate the total quantity of water required per 24 hours. Assume typical data regarding water requirement and population.* (20)

Q. 7. (*c*) *What is the purpose of elevated reservoirs in water supply ? What are the types of reservoirs ? How is the capacity of an overhead reservoir determined ?* (20)

Q. 8. (*c*) *What are the different methods of distribution of water ? What are the controlling factors for efficient distribution of drinking water ? Discuss radial system of distribution.* (20)

Year 2002 (*Civil Services*)

Q. 5 (*e*) *Compulsory : In a water treatment plant, raw water is passed through a filter bed of uniform sand at a velocity of 5 m/hour. The filter is made of sand grains of diameter = 0.4 mm, shape factor = 0.85, and specific gravity = 2.65. The depth of the bed is 0.67 m and the porosity is 0.4. Determine the head loss through the bed (Take density of water = 998 kg/m^3, and dynamic viscosity = 1.0×10^{-3} kg/ms.)* (12)

[**Solution.** Pl. See Solved Example 9.25]]

Q. 7. (*b*) *Describe briefly the process of disinfection of water supply by chlorination. What are the problems associated with it ? Indicate other means of disinfection as a replacement to chlorination and their constraints.*

Year 2003 (*Civil Services*)

Q. 5. (*d*) *Compulsory. The chemical analysis of a water indicates the presence of cations as Na^+ = 20 mg/L ; Mg^{++} = 60 mg/L ; Ca^{++} = 45 mg/L. Compute the hardness (as equivalent of $CaCO_3$). The combining weights of Ca, Mg and $CaCO_3$ are 20, 12 and 50 respectively.* (12)

[**Solution.** Using eqn. (8.1), we have

Total hardness in mg/l as $CaCO_3$

$$= \left[Ca^{++} \text{ in mg/l} \times \frac{\text{Combining wt. of } CaCO_3}{\text{Combining wt. of } Ca^{++}}\right] + \left[Mg^{++} \text{ in mg/l} \times \frac{\text{Combining wt. of } CaCO_3}{\text{Combining wt. of } Mg^{+}}\right]$$

$$= \left[45 \times \frac{50}{20} + 60 \times \frac{50}{12}\right]$$

$$= 112.5 + 250 = \mathbf{362.5\ mg/l.}\quad \mathbf{Ans.}\ ]$$

Q. 5. (*e*) (*i*) *Define the term "Optimum Coagulant Dose" and explain its significance in water treatment.* (4)

[**Hint.** Please refer article 9.11. *Optimum coagulant (alum) dose* may be defined as the minimum quantity of the given coagulant (in mg) which produces a good floc when mixed with the given raw water (1 litre). The quantities greater than this as well as the quantities lesser than this optimum qty. will give unsatisfactory floc formations.]

(*ii*) *Explain how the information on "optimum coagulant dose" is utilised to administer actual dose in the plant. Highlight also the consequences if proper dosing was not carried out.* (8)

Q. 5. (*f*) *Determine the resultant pH of two electrolytic solutions, one with a pH of 6.0 and the other with a pH of 8.0.*

[**Solution.** $pH_1 = \log \frac{1}{H_1^+} = 6.0$

or $\frac{1}{H_1^+} = 10^{+6}$

or $H_1^+ = 10^{-6}$

Similarly, $H_2^+ = 10^{-8}$

When two solutions are mixed (equally), the hydrogen ion concentration (H^+) will become

$$= H^+ = \frac{H_1^+ + H_2^+}{2} = \frac{10^{-6} + 10^{-8}}{2}$$

$$= 10^{-8}\left[\frac{10^2 + 1}{2}\right] = 10^{-8}\left(\frac{101}{2}\right)$$

$$= 50.5 \times 10^{-8} = 5.05 \times 10^{-7}$$

$$\text{pH} = \log_{10}\frac{1}{H} = \log_{10}\left(\frac{1}{5.05 \times 10^{-7}}\right)$$

$$= \log_{10}\left(\frac{10^7}{5.5}\right) = \log_{10} 10^6\left(\frac{10}{5.5}\right)$$

$$= \log_{10}(10^6 \times 1.818)$$

$$= \mathbf{6.26.}\quad \mathbf{Ans.}\ ]$$

Q. 6. (*b*) *The rate of inflow of pumping into the reservoir is constant for 16 hours period beginning from 4.00 hours in the morning to 20.00 hours in the evening each day.*

Determine analytically the minimum capacity of service reservoir to meet the demand given below :

Time (h)	*Demand m^3/s*	*Time (h)*	*Demand m^3/s*
0.0	0.00	14.0	0.60
2.0	0.20	16.0	0.30
4.0	0.30	18.0	0.40
6.0	0.40	20.0	0.50
8.0	1.00	22.0	0.30
10.0	1.20	24.0	0.00
12.0	0.80		

(20)

[**Solution.** Follow Solved Examples 10.3 to 10.5 and compute as under :

2 Hourly demands are given in m^3/s. Hence, demand for each interval in m^3 will be obtained by multiplying m^3/sec with

$$2 \times 3600 = 7200 \text{ sec.}$$

The total daily demand in m^3

$= 7200\ \Sigma$ given values of m^3/s of various intervals

$= 7200 \times 6 = 43200\ m^3$.

This demand is pumped in 16 hours at a constant rate. Hence, constant rate of pumping

$$= \frac{43200}{16} = \mathbf{2700\ m^3/h.}$$

Pumping done in 2 hr interval = **5400 m^3**

The solution is now worked out in **Solution Table 19.3**, given on next page, which is self explanatory.

From this table, we obtain :

Maximum of excess of demand = 6480 m^3

Maximum of excess of supply = 2160 m^3

Total storage required = 8640 m^3. **Ans.**]

Solution Table 19.3 : For Question 6 (*b*) of Year 2003

Time (h)	*Demand m^3/s*	*Demand in m^3 in each interval (2) × 7200*	*Cumulative demand in m^3*	*Qty. pumped in each interval*	*Cumulative pumping in m^3*	*Excess of demand in m^3 (4) – (6)*	*Excess of supply in m^3 (6) – (4)*
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
0.0	0.00	0	0	0	0	0	
2.0	0.20	1440	1440	0	0	1440	
4.0	0.30	2160	3600	0	0	3600	
6.0	0.40	2880	6480	5400	5400	1080	
8.0	1.00	7200	13680	5400	10800	2880	
10.0	1.20	8640	22320	5400	16200	6120	
12.0	0.80	5760	28080	5400	21600	6480	
14.0	0.60	4320	32400	5400	27000	5400	
16.0	0.30	2160	34560	5400	32400	2160	
18.0	0.40	2880	37440	5400	37800	—	360
20.0	0.50	3600	41040	5400	43200	—	2160
22.0	0.30	2160	43200	0	43200	—	—
24.0	0.00	0	43200	0	43200	—	—

Q. 6. (*c*) *Filter sand is to be prepared for a rapid sand filter from the stock sand for which the details of sieve analysis are given below :*

Sieve size mm	*Weight retained (g)*	*Sieve size mm*	*Weight retained (g)*
2.0	0	0.50	168
1.5	100	0.40	159
1.2	125	0.30	165
1.0	150	0.20	95
0.9	210	0.10	120
0.8	115	0.08	51
0.75	100	0.06	33
0.70	170	Finer	04
0.60	235		

The required effective size and the uniformity coefficient of the filter media is 0.3 mm and 2.5, respectively.

Determine amount of usable sand and percentage and sizes above and below which the sand is too course or too fine.

[**Solution.** Pl. See Solved Example 9.24]

Q. 7. (*b*) *Using Darcy-weisbach, Chezy-Manning, and Hazen-William formulae, find the head loss in 300 m of 2 m diameter smooth concrete pipe (assume $f = 0.014$) carrying 2 m^3/s of water (assume kinematic viscosity of water as 9.248×10^{-7} m^2/s), taking Manning's n as 0.013 and Hazen-William coefficient C_H as 130. Reason the differences in the three values.* (20)

[**Solution.** (*i*) **Darcy-Weisbatch formula** is given by Eq. (6.1) as :

$$H_L = \frac{f' L}{d} \cdot \frac{V^2}{2g}$$

where $f' = 0.014$

$L = 300$ m

$d = 2$ m

$Q = 2$ m^3/s

But V = velocity $= \dfrac{Q}{A} = \dfrac{Q}{\frac{\pi}{4} d^2} = \dfrac{Q}{\frac{\pi}{4} \times (2)^2}$ m/s

$= 0.6366$ m/s

$$\therefore \quad H_L = \frac{0.014 \times 300}{2} \frac{(0.6366)^2}{2 \times 9.81} \text{ m}$$

$= 0.0434$ m $=$ **4.34 cm. Ans.**]

(*ii*) **Mannings formula** is given by eqn.

$$V = \frac{1}{n} . R^{2/3} . \sqrt{S}$$

or $$V = \frac{1}{n} . R^{2/3} . \sqrt{\frac{H_L}{L}}$$

or $$V^2 = \frac{1}{n^2} . R^{4/3} \frac{H_L}{L}$$

$$H_L = \frac{n^2 . V^2 . L}{R^{4/3}} \qquad \text{...(i.e. Eq. (6.11))}$$

where $R = \dfrac{A}{P} = \dfrac{\frac{\pi d^2}{4}}{\pi d} = \dfrac{d}{2} = \dfrac{2}{4} = 0.5$ m

$$\therefore \quad H_L = \frac{(0.013)^2 \times (0.6366)^2 \times 300}{(0.5)^{4/3}}$$

$= 0.0518$ m $=$ **5.18 cm. Ans.**

(*iii*) **Hazen-William formula** is given by Eq. (6.12) as :

$$V = 0.85\, C_H . R^{0.63} . S^{0.54}$$

where $C_H = 130$

$$R = \frac{d}{4} = \frac{2}{4} = 0.5 \text{ m}$$

$$S = \frac{H_L}{L} = \frac{H_L}{300}$$

$$V = \frac{Q}{A} = \frac{2}{\frac{\pi}{4}(2)^2} = 0.6366$$

$$\therefore \quad 0.6366 = 0.85 \times 130 \times (0.5)^{0.63} \cdot \left(\frac{H_L}{300}\right)^{0.54}$$

$$\text{or} \quad \left(\frac{H_L}{300}\right)^{0.54} = \frac{0.6366}{0.85 \times 130 \times (0.5)^{0.63}} = 8.9159 \times 10^{-3}$$

$$\text{or} \quad H_L = 0.048 \text{ m} = \mathbf{4.8 \text{ cm}}.$$

Comments on results. It can be seen that the values of head loss computed from the three formulas are 4.34 cm (Darcy-Weisbatch), 5.18 cm (Mannings), and 4.8 cm (Hazen-William). All the three values are some what different with Darcy-Weisbatch giving the lowest head loss ; while Mannings is giving the highest value. The Hazen-William formula is giving value in between the two (*i.e.* almost average of the two values).

The reason for getting different results is the primary fact that all these formulas are based on assumed values of friction factors f', n and C_H, which depend upon several factors and can never be precise.

Moreover, *Manning's formula* is basically given for conduits (pipes) flowing under gravity and not for pressure pipes (turbulent flows), although it is implied to be applicable to pipes having flows under pressure also, if Manning's friction factor (n) is accurately estimated for each case, since it increases with the value of hydraulic mean depth (R). Since it is not feasible to compute n for each R, the results obtained from Mannings formula can not be precise.

Hazen-William's formula, though most widely used for pipe flows, yet suffers from the drawback that the value of constant 0.85 (in MKS units) as appearing in this formula, has been calculated for an assumed value of $R\left(i.e., \frac{d}{4}\right)$ of 0.3 m and friction slope (H_L) of 1 in 1000, although the formula is used for all values of pipe dia. and friction slopes, as we have used it in our case where R is equal to 0.5 and friction slope is $\frac{H_L}{300} = \frac{0.048}{300} = \frac{1}{6250}\left(i.e. \text{ much flatter than } \frac{1}{1000}\right)$. This factor is likely to introduce errors upto ± 30% in the evaluation of velocity and ± 55% in head loss (H_L). For these very reasons, Hazen-William's formula has now-a-days been modified by Darcy-Weisbatch and Cole-brook-White introducing viscosity term, as :

$$V = \text{velocity} = \frac{3.83\, C_R \left[d^{0.6575} (g \cdot S)^{0.5525}\right]}{\nu^{0.105}} \quad \text{...(Eq. 6.13)}$$

The value of C_R for RCC pipes of size upto 2 m is obtained from table 6.3 as 1.0. The value of H_L as obtained from this formula is computed here just for the sake of knowing as to how much different result is obtained by this modified formula, as :

$$V = \frac{3.83 \times 1\left[(2)^{0.6575} (9.81)^{0.5525} \left(\frac{H_L}{300}\right)^{0.5525}\right]}{[9.284 \times 10^{-7}]^{0.105}}$$

$$0.6366 = 3.9262 \cdot H_L^{0.5525}$$

$$\text{or} \quad H_L = 0.0371 = \mathbf{3.71 \text{ cm}}$$

This is the lowest value of all the four computations, and will be more true to realistic value.]

Year 2004 (*Civil Services*)

Q. 5. (*d*) *List the tests to be performed to determine impurities in water. In this context explain the terms "Hardness" and "pH" of water interms of causes, classifications as appropriate, and effects due to consumption and the tests relevant therein.* (12)

Q. 5. (*e*) *What considerations rule in the selection of pumps for a water supply scheme with a lake as a source ? What are the appurtenances to be installed in the pipeline through the pumps upto the overhead reservoir at the area being served ?* (12)

Q. 6. (*c*) *Why are coagulants required in the sedimentation phase of water treatment ? What factors affect the dosage of coagulants ?* (10)

Q. 6. (*d*) *A coagulation cum sedimentation tank is to be designed for the water works of a town having a population of 1.5 lakh. The average demand of water is 135 lpd/head and maximum demand is 1.5 times the average. The detention period is 6 hours. Flow rate is to be in the range near 1000 lph/m² of plan area. Design a system of rectangular sedimentation tanks.* (10)

[**Hint.** Pl. follow similar Solved Example 9.17.]

Years 2005 (*Civil Services*)

Q. 5. (*d*) (*i*) *What are the standards for potable water in respect of*

(1) *Turbidity*

(2) *Total Solids*

(3) *pH*

(4) *BOD*

(*ii*) *What is the scope of B-Coli test ? Describe Membrane Filter Technique.* (4 + 8 = 12)

Q. 5. (*e*) *What is meant by "Dual System of Water Distribution" ? What are its advantages ?* (12)

Q. 6. (*c*) (*i*) *A source of water supply to a town on a hill is under investigation and the following data are being considered :*

Population—2 lakh ; Water demand—200 lpcd ; Height of point of inlet to water works above the lowest water level in the source—30 m ; Friction and minor losses in the intake pipe system—to be around 4 m ; Efficiency of pump—80% ; Duration of pumping—20 hrs/day ; Flow velocity through pipe—to be about 1.0 m/sec.

Determine the size of the rising main and the horse power of the pump required. (10)

Solution : Follow similar Solved Example 6.4 and compute as under :

Average quantity of water reqd. per day

$$= 200 \times 2{,}00{,}000 \text{ l/d} = 40 \text{ MLD}$$

Max. demand $= 1.8 \times 40 = 72$ MLD

$$= \frac{72 \times 10^6}{10^3 \times 24 \times 3600} \text{ m}^3/\text{s} = 0.833 \text{ m}^3/\text{s}$$

Max. draft reqd. for 20 hr/d working of pumps (Q)

$$= 0.833 \times \frac{24}{20} = 1 \text{ m}^3/\text{s}$$

Area of pipe reqd. $(A) = \frac{Q}{V} = \frac{1 \text{ m}^3/\text{s}}{1 \text{ m/s}} = 1 \text{ m}^2$

Dia. (d) of pipe (rising main) $= \sqrt{1 \times \frac{4}{\pi}} = 1.13$ m $\left(\because \frac{\pi}{4} d^2 = 1\right)$

Hence, use the nearest available size of 1.2 m (say). **Ans.**

$$\text{Power of motor} = \frac{\gamma_\omega \cdot QH}{\eta} \text{ (kN/s ; } i.e. \text{ kW)}$$

where γ_w is in kN/m^3

$$\text{Now, H.P. of pump} = \frac{\text{kilo watts}}{0.735} = \frac{\gamma_\omega QH}{0.735\,\eta}$$

where $H = 30 + 4 = 34$ m

$\gamma_w = 9.81$ kN/m^3

$$\therefore \quad \text{H.P. reqd.} = \frac{9.81 \times 1 \times 34}{0.735 \times 0.80}$$

$$= \mathbf{567.2\ HP} \text{ ; say 570 HP. } \mathbf{Ans.}$$

Q. 6. (*c*) (*ii*) *What are the factors that guide in deciding the water pressure to be maintained in a distribution system ? How do service reservoirs help in maintaining the pressure, discharge rate and economy of the supply ?* (10)

Q. 8. (*b*) (*ii*) *Draw a neat sketch of a river intake on the straight reach of a river for supplying water to a township. Name the components on the sketch and briefly state the functions of each.* (10)

Year 2006 (*Civil Services*)

Q. 5. (*d*) (*i*) *Explain the significance of the following from the point of view of water quality criteria :* (6)

(1) *Fluoride*

(2) *Nitrate*

(3) *Arsenic*

(4) *Specific conductivity*

(5) *Turbidity*

(6) *pH value*

(*ii*) *Describe operating problems of rapid gravity filter.* (6)

Q. 5. (*e*) *Name water-borne diseases caused by bacteria, viruses, protozoa and helminths. How are the incidence of these diseases prevented ?* (12)

Q. 7. (*b*) (*ii*) *Name the chemicals, which are used for coagulation. Discuss their merits and demerits.* (10)

Q. 8. (*c*) *Write notes on :*

(*i*) *Breakpoint Chlorination*

(*ii*) *Fire Demand* (10)

Year 2007 (*Civil Services*)

Q. 7. (*c*) *Discuss the following aspects of water distribution system :* (15)

(*i*) *Cross-connections*

(*ii*) *Leakage*

Q. 8. (*b*) (*i*) *A 15 cm diameter borehole well was drilled in an area to supply 360 l.p.m. of water. The water level in the well is found to be 18 m below ground level. Water is conveyed to a place 50 m away from the well. If the drawdown is 20 m and water is lifted 2 m above the ground level, determine the power in kW of the motor of the pump, given the overall efficiency as 65%. Friction including strainer and other losses may be taken as 5 cm per metre length of delivery pipe. What type of pump would you suggest ?* (20)

Solution : Power of pump (kW)

$$= \frac{\omega QH}{\eta} \text{ (Eq. 4.61)}$$

Here total head loss against which motor has to work

= 10 (Depth of W.T.) + 20 m (drawdown)
+ 2 m (Lift above ground)
+ 5% × 50 m (Friction losses)

$= 18 + 20 + 2 + 2.5$

$= 42.5$ m

$$Q = 360 \text{ l.p.m.} = \frac{360}{1000} \times \frac{1}{60} \text{ m}^3/\text{sec}$$

$$= \frac{0.360}{60} = 6 \times 10^{-3} \text{ m}^3/\text{s}$$

$$\eta = 0.65$$

$$w = 9.81 \text{ kN/m}^3$$

$$\therefore \quad \text{Power} = \frac{9.81 \times 6 \times 10^{-3} \times 42.5}{0.65} = \mathbf{3.85\ kW.}\ \textbf{Ans.}$$

$$\text{HP} = \frac{3.85}{0.735} = \mathbf{5.24\ ;\ say\ 5.25\ HP.}\ \textbf{Ans.}$$

A submergible or a turbine type of pump set can be used, depending upon the market availability of a motor, having HP slightly exceeding 5.25 HP. Ans.

Year 2008 (*Civil Services*)

Q. 5. (e) (i) *Draw a neat line sketch of a water intake tower on a lake serving as a source of water for a town and discuss the constructional details and operation of the tower.* (7)

(ii) *Differentiate between a slow sand and a rapid sand filter in respect of (i) arrangement of the media (sand), (ii) method of cleaning, (iii) penetration of impurities and (iv) under drainage system.* (5)

Q. 6. (c) (i) *Three towns A, B and C were developed in a similar fashion. The past record of their population (in lacs) are shown in the following table:*

Town	*Population (in lacs) in the year*				
	1920	*1940*	*1960*	*1980*	*2000*
A	*1.0*	*1.1*	*1.5*	*2.1*	*2.7*
B	*3.0*	*3.2*	*3.8*	*4.3*	*5.0*
C	*2.0*	*2.3*	*2.8*	*3.0*	—

Determine the populations in the years 2020 and 2040 respectively of a town W (which developed on a basis/circumstances similar to those of towns A, B and C having populations of 0.5, 0.7, 0.95, 1.3 and 1.7 lac, in the years 1920, 1940, 1960, 1980 and 2000, respectively. (10)

Solution. The question is to be solved as explained in article 2.10.6. The plotting is done as shown in Fig. 19.2 for town X, to determine the point P corresponding to the highest known population of 1.7 lakh and year 2000. The curve OP corresponds to the known population-year data for town X. Now, the given data for town A is analysed to estimate that its population would have become 1.7 lakh in the year 1967*. The given data of town A is now plotted by starting the year 1967 from point P on time

*The population of town A was 1.5 lac in 1960, which rose to 2.1 lac in 1980, giving increase of 0.6 lac in 20 years. Thus, the increase of 0.2 lac will occur in about 7 years to make its population at 1.7 lac in the year 1967.

scale. Similarly for town B, the population of 1.7 lakh is estimated to have reached in the year 1860. The plotting for data of town B is hence started from point P assuming the year to be 1860. Similarly, for town C, the population of 1.7 lakh is estimated to have reached in the year 1900. The time scale from point P as year 1900 is thus started for town C. The curves are thus plotted for three towns A, B and C. The curve

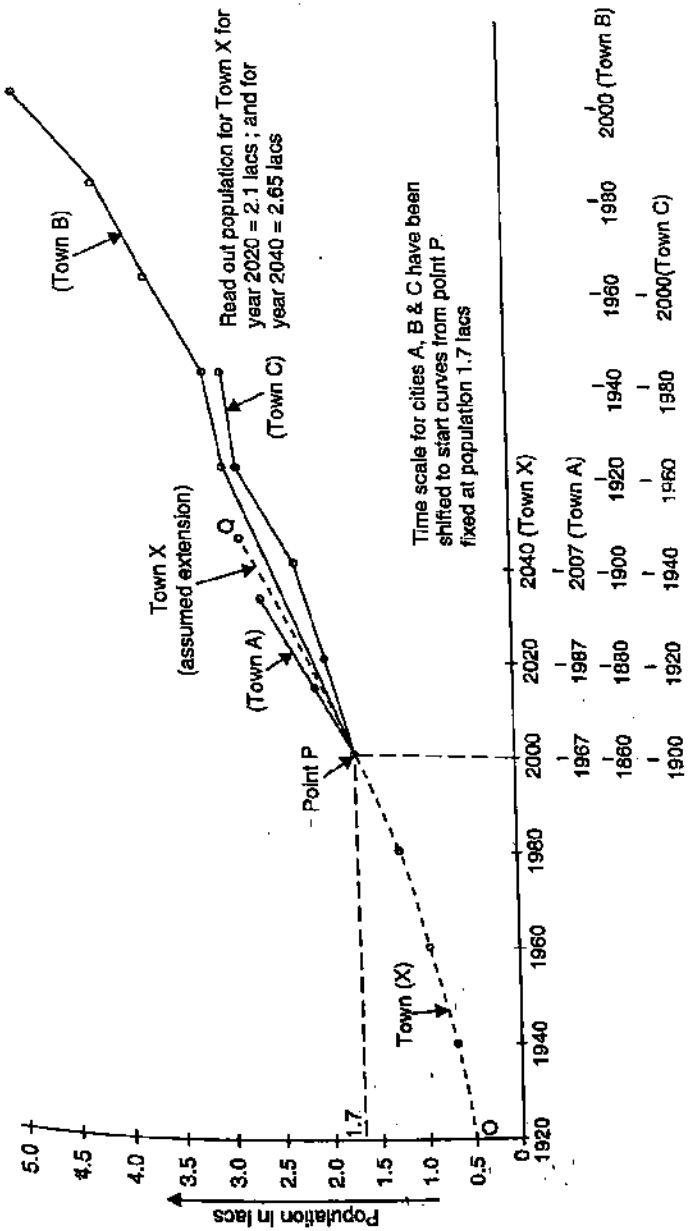


Fig. 19.2

PQ for town *X* is now assumed to be somewhere in between the curves of three towns *A*, *B* and *C*, which fits well with its initial portion *OP*. Thus, the curve *OPQ* represents the curve for town *X*. The values of P_{2020} and P_{2040} are read out from this curve as 2.1 lakh and 2.65 lakh, respectively. **Ans.**]

Q. 6. (*c*) (*ii*) *A water treatment plant is to treat* 10×10^6 *litres of water per day by international standards. Determine the amount (in kg) of bleaching powder (having 30% available chlorine) needed per day if the chlorine demand of the water is 1.5 mg/l.* (5)

[**Solution.** Pl. follow exactly similar Solved Example 9.27, and compute as follows:

Qty. of water treated = 10×10^6 L/day

Chlorine demand (no additional residual chlorine is reqd. under international standards) = 1.5 mg/L

∴ Amount of chlorine required daily

$$= 1.5 \frac{\text{mg}}{\text{L}} \times 10 \times 10^6 \text{ L/d} = 15 \text{ kg}$$

Since the bleaching power contain 30% of available chlorine, the amount of bleaching powder required

$$= 15 \text{ kg} \times \frac{100}{30} = \mathbf{50 \text{ kg/day}}. \quad \textbf{Ans.}$$]

Q. 6. (*c*) (*iii*) *Differentiate between coagulation and flocculation. Give a line sketch of any one system of flocculation for water treatment.* (5)

Year 2009 (*Civil Services*)

Q. 5. (*d*) *What is the significance of the following substances when they exceed the permissible limit in potable water? Also mention the permissible limit for the same:*

Fluoride, Nitrate, Nitrite, Chloride, Arsenic (12)

Q. 8. (*c*) *Design a plain sedimentation tank to treat 4 million litres water per day. The detention period may be assumed as 6 hours and the depth may be assumed as 3.5 m. Assume velocity of flow as 10 cm/minute.* (5)

[**Solution.** Pl. follow exactly similar Solved Example 9.2, and compute as follows:

Qty. of water to be treated per day

$= 4 \text{ ML} = 4 \times 10^6 \text{ L} = 4000 \text{ m}^3$

Qty. of water to be treated during the detention period of 6 hr.

$$= 4000 \times \frac{6}{24} = 1000 \text{ m}^3$$

∴ Capacity of the tank required = 1000 m^3

Velocity of flow to be maintained through the tank

= 10 cm/min

∴ Length of the tank required

= Velocity of flow × Detention period

$$= 10 \frac{\text{cm}}{\text{min}} \times (6 \times 60 \text{ min})$$

$$= 0.1 \frac{\text{m}}{\text{min}} \times 360 = 36 \text{ m}$$

Cross-section area of the rectangular tank required

$$= \frac{\text{Capacity}}{\text{Length}} = \frac{1000}{36} \text{ m}^2$$

$= 27.78 \text{ m}^2$

Total depth of the tank

= 3.5 m (given)

Assuming free board of 0.5 m, we have the water depth in the tank

$= 3.5 - 0.5 = 3$ m

∴ Width of tank

$$= \frac{\text{Area}}{\text{Water depth}} = \frac{27.78 \text{ m}^2}{3 \text{ m}}$$

$= \mathbf{9.26 \text{ m}}$; say 9.3 m

Hence, use a rectangular sedimentation tank with an overall size of 36 m × 9.3 m × 3.5 m. **Ans.**]

Year 2010 (*Civil Services*)

Q. 5. (*c*) *A small town with a population of 1000 persons is to be supplied with 96 lpcd. The demand of water during different periods is given as follows :*

Time hr.	*0–3*	*3–6*	*6–9*	*9–12*	*12–15*	*15–18*	*18–21*	*21–24*
Total demand in litres	*9000*	*10000*	*12000*	*18000*	*14000*	*12000*	*10000*	*11000*

Determine the balancing capacity of a service reservoir if pumping is done in blocks of 24 hours, the delivery rate being constant. (12)

[**Solution.** Follow similar Solved Example 10.3–Table 10.5, and analyse the given data as shown in **Solution Table 19.4.**

From Solution Table 19.4, it can be observed that :

the maximum of excess of demand = 3000 litres ...(*i*)

and, the maximum of excess of supply = 5000 litres ...(*ii*)

Solution Table 19.4. For Q. 5 (*c*) of Year 2010 (Civil Services)

Time in hrs of the day	*Demand in each 3 hr period in litres*	*Cumulative demand in litres*	*Constant pumping rate in each of the 3 hr period in litres*	*Cumulative pumping in litres*	*Excess of demand in litres (3) – (5) +ve values only*	*Excess of supply in litres (5) – (3) +ve values only*
(1)	(2)	(3)	(4)	(5)	(6)	(7)
0–3	9000	9000	12000	12000	–	3000
3–6	10000	19000	12000	24000	–	5000
6–9	12000	31000	12000	36000	–	6000
9–12	18000	49000	12000	48000	1000	–
12–15	14000	63000	12000	60000	3000	–
15–18	12000	75000	12000	72000	**3000**	–
18–21	10000	85000	12000	84000	1000	–
21–24	11000	96000	12000	96000	–	–
Σ	96000		96000			

Hence the storage or balancing capacity reqd.

$= (i) + (ii) = 3000 + 5000$

= 8000 litres. Ans.

Q. 5. (*d*) *Draw a neat sketch of a service pipe connection explaining the functioning and requirements of the various parts in this connection.* (12)

Q. 5. (*f*) *Stating a use-oriented definition of water pollution, list the various sources of water pollution alongwith the major characteristics of each of the stated sources.* (12)

Hint : Pl. refer Article 9.1 and 9.3 in author's another book *"Ecology and Environmental Studies"* ; from where you can easily explain the following four sources of water pollution :

(1) *Agricultural runoff* ; (2) *Rain runoff from colonies* ; (3) *Domestic waste waters* ; and (4) *Industrial waste waters*. In addition, water pollution may also be sourced from ***human activities*** such as by large scale bathing in rivers, as happens on the occasions of *Kumbh Mela* and on other festivals ; or by large scale immersions of Lord Ganesha and Durga idols in various cities during Ganesh Chaturthi and Navratra festivals. Sometimes, even dead bodies are thrown away by humans in holy rivers, such as in Ganga at Varanasi. Throwing of garbage and temple wastes is another such example of human activities, which cause water pollution in our precious river resources.

Q. 6. (*b*) (*i*) *Differentiate between super-chlorination and break-point chlorination. Explain break-point chlorination with a neat sketch explaining the types of chlorine form available in the various stages of the break-point curve. What is the significance of residual chlorine ?* (10)

Q. 6. (*b*) (*ii*) *Describe the step-by-step procedure used for backwashing of a rapid sand filter. What is the significance of the backwash time and backwash velocity ?* (10)

Q. 6. (*c*) (*i*) *What is Darcy's coefficient of permeability ? How is it determined ? Draw the necessary sketches.* (10)

Q. 8. (*b*) (*i*) *Enumerate and discuss the various factors that play a role in estimating the per capita demand of water supply. Also explain the implications if heavy metals are present in concentrations above their permissible levels.* (10)

Q. 8. (*c*) (*i*) *How would the radial flow in an open well be determined under steady state flow conditions when the aquifer is* (*a*) *unconfined, and* (*b*) *confined ?* (10)

Q. 8. (*c*) (*ii*) *Write short notes on each of :*

(1) *Reflux valve*

(2) *Air-Relief valve*

(3) *Expansion joint*

in a water supply distribution system. (10)

Year 2011 (*Civil Services*)

Q. 5. (*c*) *What is an aquifer ? How are they classified ? Describe salient features of aquifers.* (10)

Q. 6. (*a*) (*i*) *Explain the types of sedimentation with examples.* (2)

Q. 6. (*a*) (*ii*) *What path trajectory the settling particles follow in circular and rectangular tanks ?* (2)

Q. 6. (*a*) (*iii*) *Given the following data below, design a circular sedimentation tank with launders and find out :*

(*i*) *The diameter of the settling tank.*

(*ii*) *Total depth of the settling tank with free board including the depth of the hopper portion.*

Total township area	*20 ha*
%age areas for roads, public utilities, and open spaces	*10%, 15%, 15%*
Floor to space ratio	*1.5*
Floor area required for each person	*10 m^2*
Water losses accounts for	*4%*
Working hours for plant	*23 hrs.*
Surface loading rate	*1.25 $m^3/hr/m^2$*
Detention time	*2.5 hrs*
Slope for floor slab	*1 in 12*
Incoming velocity of flow	*0.6 m/sec.*

Check the design with weir loading calculations. Assume water supply rate as 135 lpcd. Give a neat sketch of final designed circular tank showing all design parameters. (16)

[**Hint :** Total township area = 20 ha = 20×10^4 m^2

% age of areas for roads + public utilities + open spaces
= 10 + 15 + 15 = 40%

∴ Area meant for housing
= (60% × 20 × 10^4) m^2 = (12×10^4) m^2

Floor area = 1.5 space area
= ($1.5 \times 12 \times 10^4$) m^2 = ($18 \times 10^4$) m^2

Floor area reqd. per person = 10 m^2

∴ No. of persons to reside = $\dfrac{18 \times 10^4}{10}$ = **18000**

Water supply rate = 135 lpcd

∴ Total water reqd. = 18000 × 135 litres/day = **2.43 MLD**

Water losses = 4%

∴ Raw water reqd. to be treated to produce 2.43 MLD of treated water

$$= \frac{100}{96} \times 2.43 = \frac{2.43}{0.96} = \mathbf{2.53125\ MLD}$$

∴ Qty. of water to be treated per day = 2.53125 ML

Detention time = 2.5 hr

∴ Qty. of raw water to be treated during the detention period of 2.5 hrs; *i.e.* the capacity of the tank

$$= \left[\frac{2.53125 \times 10^6}{24 \times 10^3} \times 2.5\right] \text{m}^3 = \mathbf{263.67\ m^3}$$

The capacity (Vol.) of the circular tank of depth H and dia D is given from Eq. (9.18) as

$$\text{Vol.} = D^2\left[0.011\,D + 0.785\,H\right] \quad \text{...(Eq. 9.18)}$$

Here D and H are both to be determined.

Hence, we use another given factor ; *i.e. surface loading rate*
= 1.25 m^3/hr/m^2,
which is defined as the discharge per unit of surface area (A_s)

$$\therefore \quad \frac{\text{Discharge}}{A_s} = 1.25\ \text{m}^3/\text{hr/m}^2$$

or
$$\frac{\left[\dfrac{2.53125 \times 10^6}{10^3 \times 24}\right] \text{m}^3/\text{h}}{A_s} = 1.25\ \text{m}^3/\text{hr/m}^2$$

or
$$A_s = 84.375$$

∴
$$A_s = \text{Surface area of tank} = \frac{\pi}{4}D^2 = 84.375$$

or
$$D = \sqrt{84.375 \times \frac{4}{\pi}} = \mathbf{10.36\ m.}\quad \textbf{say 10.4 m}\quad \textbf{Ans.}$$

Using Eq. (9.18), we get

$$263.67 = (10.4)^2\left[0.011 \times 10.4 + 0.785\,H\right]$$

or
$$2.438 = 0.114 + 0.785\,H$$

or
$$2.323 = 0.785\,H$$

or
$$H = \mathbf{2.96\ m}$$

∴ Side water depth of the tank = 2.96 m

With slope of floor slab at 1 in 12, the depth at centre of the circular tank of radius 5.2 m = $2.96 + \dfrac{1}{12} \times 5.2 = 3.4$ m

Let us provide a sludge hopper at bottom (in the centre) of the tank having depth of 0.6 m. Also provide a free board of 0.5 m.

∴ The total depth of the tank including hopper and freeboard
= 3.4 + 0.6 + 0.5 = **4.5 m. Ans.**

Weir loading

Length of the weir along peripheri of the tank for effluents
= πD = 3.14 × 10.4 m = 32.67 m

Weir loading per day

$$= \frac{\text{Discharge per day}}{\text{Length of weir}} = \frac{\dfrac{2.53125 \times 10^6}{10^3}}{32.67} = \frac{2531.25}{32.67}\ \text{m}^3/\text{d.m.}$$

= 77.48 m^3/day/m of weir length.

(Ok, being less than 120 m^3/d.m*)

Typical design details of the sedimentation tank with launders (Influent as well as Effluent) *i.e.* the tank with peripherial feed are shown in Fig. 19.3.

Such a tank will have the sludge-removal equipment supported on beams spanning the tank. Tanks larger than 10 m or so in diameter will have a central pier, as shown in Fig. 19.3, to support the mechanism and is reached by a walkway or a bridge (***Since dia is 10.4 m in our design, we can even certainly omit the central pier***). The bottom of the tank is sloped at 1 in 12 as given, and the sludge is scraped to a relatively small hopper (of say 0.6 m depth) located near the centre of the tank.

In such a ***rim feed design***, a suspended circular baffle at a short distance from the tank wall forms an annular space into which the wastewater is

*According to GULMRB (The Great Lakes – Upper Mississippi River Board of State Sanitary Engineers), the weir loading should not exceed about 120 m^3/day/m of weir length for plants with average flows of less than 0.04 cumec (3450 m^3/d) ; and 190 m^3/day/m for for larger flows.

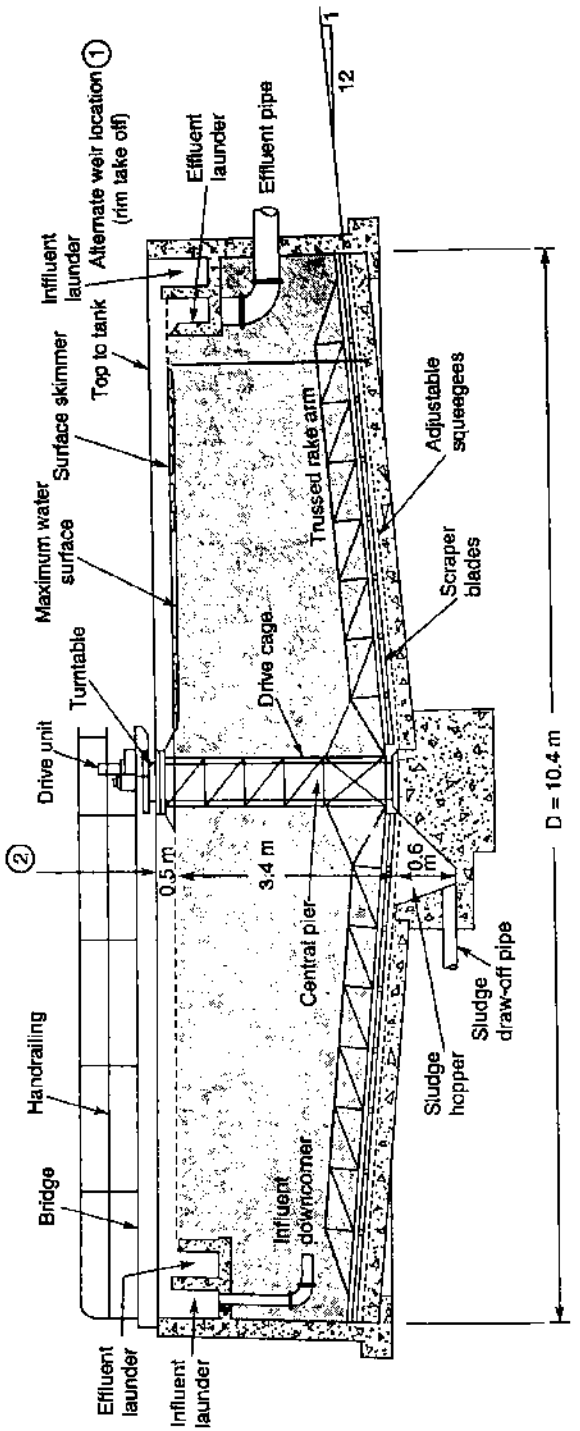


Fig. 19.3. Sedimentation tank for Q. 6 (*a*) (*iii*) of Year 2011 with **Peripherial Flow** (*i.e.* having Influent & Effluent Launders), called the **Rim Feed** type.

discharged in a tangential direction. The wastewater flow spirally around the tank and underneath the baffle, and clarified liquid effluent is skimmed off over weirs on both sides of a centrally located weir trough. Grease and scum are confined to the surface of the annular space. Sludge should be removed by pumping, atleast once per shift and more frequently in hot weather to avoid deterioration of the effluent.

Year 2012 (*Civil Services*)

Q. 5. (*d*) (*i*) *Discuss any two methods for predicting population with their relative advantages and disadvantages.* (6)

Q. 5. (*d*) (*ii*) *What are waterborne diseases ? Also discuss their control measures.* (6)

Q. 5. (*e*) (*i*) *Explain the break point chlorination and super chlorination.* (2)

Q. 5. (*e*) (*ii*) *Determine the amount of bleaching powder required annually in a water treatment plant treating 10 million litre per day of water if 0.2 ppm of chlorine dose is required. Available bleaching powder contains 30 per cent of available chlorine.* (10)

[**Hint.** Pl. follow Solved Example 9.27.]

Year 2013 (*Civil Services*)

Q. 5. (*b*) *Explain what is meant by filtration for drinking water. Draw a schematic diagram of a slow sand filter showing inlet, outlet and other components.* (10)

Q. 7. (*b*) *Explain with the help of diagrams the Grid iron system and the Radial system of water distribution for a town.* (10)

Q. 8. (*c*) *A water treatment plant is to treat a design discharge of 60 million litres per day. The plant has a paddle flocculator 20 m long, 12 m wide and 3.5 m deep. It is fitted with four horizontal shafts to which revolving paddles are attached which rotate at 150 rpm. Each shaft supports four paddles which are 10 m long, 20 cm wide and centred 1.5 m from the shaft.*

Assume mean velocity of flow of water as 20% of the paddle velocity, temperature of water as 10°C and $C_D = 1.8$.

Determine the difference in velocity between paddle and water, mean velocity gradient and detention time.

Dynamic viscosity can be taken as 1.31×10^{-4} *m²/sec at temperature of 10°C.* (20)

[**Hint.** Pl. see Q. 2 (*a*) (*i*) of Year 2002 of Engineering Services Exam on Page 870.]

Year 2014 (*Civil Services*)

Q. 5. (*a*) *The difference between the level of catchment and a Service Reservoir is 150 m and the distance between them is 48 km. The Reservoir was originally constructed by a single pipeline designed to carry 0.26 m^3/sec. It was later found necessary to increase the supply to 0.365 m^3/sec and it was decided to lay another pipeline of same diameter along the first part of the length and cross connected. Calculate the diameters of the pipes and the length of second pipe.* (10)

Solution. In the first case, when a single pipe line is laid, we have

$H_L = 150$ m

$L_1 = 48$ km $= 48000$ m

$Q_1 = 0.26$ m^3/s

d = dia. of pipe = ?

From Eq. 6.1(*a*), we have

$$H_L = \frac{8}{g\pi^2}\frac{f' L_1 Q^2}{d^5}$$

Let us assume that the Darcy's friction factor for the pipe (f') is equal to 0.024 (Pl. see footnote one Page 224).

$$\therefore \quad 150 = \frac{8}{9.81 \times (3.14)^2} \cdot \frac{0.024 \times 48000 \times (0.26)^2}{d^5}$$

or $\quad d^5 = 0.0429$

or $\quad \mathbf{d = 0.533\ m}$

In the 2nd case, let another pipe for length L_2 and dia d (0.53 m) be laid alongside the already existing pipe. Then equating the total head loss in this

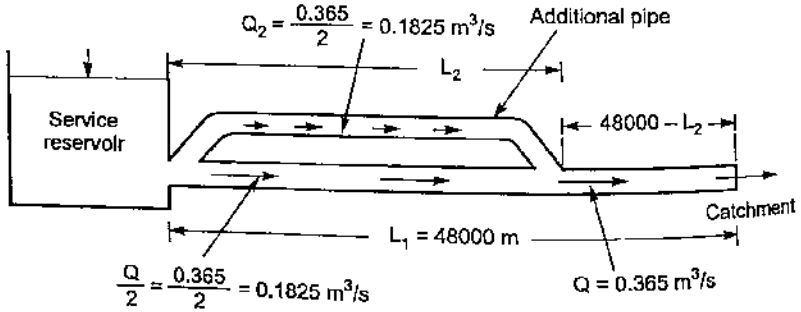


Fig. 19.4

pipe system to the available head, we get

150 m = Head loss in Length L_2 with discharge $\frac{0.365}{2} = 0.1825$ m^3/s

+ Head loss in Length $(48000 - L_2)$ with 0.365 m^3/s

$$\therefore \quad 150 = \frac{8}{g\pi^2} \times \frac{0.024 \times L_2 (0.1825)^2}{(0.533)^5} + \frac{\frac{8}{g\pi^2} \times 0.024 \times (48000 - L_2)(0.365)^2}{(0.533)^5}$$

or $\quad 150 = 1.535 \times 10^{-3} L_2 + 6.142 \times 10^{-3} (48000 - L_2)$

or $\quad 150 = 294.79 - 4.607 \times 10^{-3} L_2$

or $\quad 144.79 = 4.607 \times 10^{-3} L_2$

or $\quad L_2 = \frac{144.79}{4.607 \times 10^{-3}}$ m $= 31{,}428$ m

$= \mathbf{31.43\ km.}$ **Ans.**

Hence lay an additional pipe of 31.43 km along side the original pipe of 48 km in its initial portion. The dia of both pipes being 0.533 m. **Ans.**

Q. 6. (*b*) *What do you mean by the term Per Capita Demand? How is it estimated? The population of a locality as obtained from Census record is as follows :*

Year	*1970*	*1980*	*1990*	*2000*	*2010*
Population	*15000*	*20000*	*24500*	*29500*	*32500*

Estimate the population of the locality in 2040 by Arithmetic Increase, Geometrical increase, Incremental Increase and Rate of Decrease Methods. (20)

Hint: This question can be easily solved by following the calculations adopted for different methods, such as Arithmetic method, Geometric method, Incremental Increase method, and Decrease Rate of Growth methods by following the Solved Examples 2.2, 2.3, 2.6 and 2.10, respectively.

Q. 7. (*a*) *Discuss the significance of Nitrogen Content, BOD, COD, DO, Arsenic and Fluoride in Water Quality.* (15)

Hint: For BOD, COD and DO, pl. refer Environmental Engg. Vol. II – Sewage Disposal and Air Pollution Enginecring.

Q. 7. (*b*) *Design a Rapid Sand Filter unit for 4.5 MLD, with all its principal components.* (15)

Pl. refer Solved Example 9.21.

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Objective Questions of AMIE Exams

Q. 1. Fill in the blanks :

(i) *Particles larger than ..... size are considered to be suspended solids in an aqueous media.

(ii) The abbreviation TCU stands for ..... colour unit.

(iii) Iron may cause colour problem even at a low concentration like ..... mg/l.

Q. 2. Indicate whether the following statements are True or False :

(i) If alkalinity is much higher than the total hardness, non carbonate hardness is expected to be absent.

(ii) Mottled and discoloured teeth occur when drinking water supply contains excessive fluorides.

(iii) Total dissolved solids concentrations are usually very high in surface waters.

(iv) *In any unpolluted natural system, the alkalinity is caused mostly by the bicarbonates.

(v) Lead, chromium, cadmium, zinc, copper, etc. are known as heavy metals, as their atomic numbers are more than 23.

Q. 3. Fill in the blanks :

(i) Lead, chromium, cadmium, zinc, copper, iron, etc., are known as 'heavy metals' because their atomic numbers are more than .....

(ii) *Most common rate of filtration in rapid sand filters is ..... m/h.

(iii) Part of the total hardness, which is equivalent to alkalinity is known as ..... hardness.

Q. 4. Match the following sets :

Set A	Set B
(*i*) Mottled teeth	(*a*) Nitrate
(*ii*) Methaemoglobinemia	(*b*) Slow sand filters
(*iii*) Schmutzdecke	(*c*) Fluoride.

Q. 5. Indicate whether the following statements are True or False :

(i) Chlorine existing in the form of HOCl and/or OCl^- is defined as free available chlorine.

(ii) Ferrule is one of the important appurtenances installed in a combined sewage system.

*Hints for solving starred questions are given at the end of the chapter.

Q. 6. Choose the correct statement in the following :

(i) The microorganism group *E-Coli* is used as an indicator of faecal contamination of water because :

(*a*) E-Coli is a known pathogen

(*b*) it is very easy to detect

(*c*) they are discharged along with faeces in large number

(*d*) all of the above.

(ii) Pathogenic organisms may be found in contaminated waters and are responsible for diseases like

(*a*) typhoid fever (*b*) infectious hepatitis

(*c*) gastro enteritis (*d*) all of the above.

(iii) If the chlorine fed and chlorine demand of water is 1.5 mg/l and 1.3 mg/l respectively, the residual chlorine in gm/m^3 is

(*a*) 0.2 (*b*) 2.0

(*c*) 20 (*d*) 0.02.

Q. 7. Choose the correct answer in the following :

(i) The design period for water supply project is taken as :

(*a*) 5—10 yr (*b*) 10—15 yr

(*c*) 50—60 yr (*d*) 20—30 yr.

(ii) The method adopted for removing bushes, debris and wood from water is known as

(*a*) sedimentation

(*b*) coagulation

(*c*) screening

(*d*) filtration.

(iii) In the design of sedimentation tank, the essential factor to be considered is

(*a*) surface loading

(*b*) depth of tank

(*c*) turbidity

(*d*) concentration of suspended solids.

(iv) A good trap should

(*a*) not have self cleaning property

(*b*) restrict the flow of water

(*c*) provide an adequate water seal at all times

(*d*) all the above.

Q. 8. Fill in the blanks :

(i) In sedimentation, the Stokes law is applicable for Reynold's number less than .....

(ii) The population of a young and rapidly developing city is forecast by ..... method.

(iii) pH value is a symbol for ..... concentration.

Q. 9. Match the following sets :

Group A	Group B
(*i*) Arithmetical increase method	(*a*) Septic tank
(*ii*) Anaerobic treatment	(*b*) Old city
(*iii*) Water bearing strata	(*c*) Pathogenic bacteria
(*iv*) Water borne diseases	(*d*) Particulate matter
(*v*) Electrostatic precipitators	(*e*) Aquifer.

Q. 10. Indicate whether the following statements are True or False :

(i) The quality of tubewell water is poor as compared to surface water.

(ii) Biochemical oxygen demand of safe drinking water is zero.

(iii) The uniformity coefficient of sand for rapid sand filter is more as compared to slow sand filter.

(iv) Killing of pathogenic bacteria is called sterilization.

Q. 11. Choose the correct statement in the following :

(i) The valve which allows unidirectional flow of water is called

(*a*) Sluice valve (*b*) Gate valve

(*c*) Reflux valve (*d*) Washout valve.

(ii) Total hardness of a water sample is 500 mg/l as $CaCO_3$; if the total alkalinity of the same sample is 270 mg/l as $CaCO_3$, the temporary hardness of the water is

(*a*) 500 mg/l as $CaCO_3$ (*b*) 270 mg/l as $CaCO_3$

(*c*) 230 mg/l as $CaCO_3$ (*d*) nil.

Q. 12. Fill in the blanks :

(i) A particle in water will be considered as a colloid if its size ranges in between ..... and ..... μm.

(ii) Net positive suction head in a centrifugal pump should not be more than ..... m.

(iii) Iron in water may cause colour problem at concentrations more than ..... mg/l.

Q. 13. State whether the following statements are True or False :

(i) Fluoride in water in excess is responsible for a disease known as 'blue baby disease'.

(ii) In submersible pumps, the pumping unit is above the ground level.

(iii) In the development of well, the fines in the aquifer material in the immediate vicinity is removed.

Q. 14.* Match the following sets :

Set A	Set B
(*i*) Recarbonation	(*a*) Activated carbon
(*ii*) Chlorination	(*b*) Trihalomethanes
(*iii*) Taste and odour	(*c*) Lime soda process

Q. 15. Choose the correct statement in the following :

(i) *Presence of higher algal content in water indicates that water is

(*a*) Hard (*b*) Soft

(*c*) Acidic (*d*) Alkaline

(*e*) Free of turbidity.

(ii) If the sedimentation tank is rectangular in shape having length L, width W and depth D, then for discharge equal to Q, the settling velocity of a particle would be

(*a*) $\frac{Q}{W \times D}$ (*b*) $\frac{Q}{L \times W}$

(*c*) $\frac{Q}{D \times L}$ (*d*) $\frac{Q}{L \times W \times D}$

(iii) As per IS : 1172 the water consumption per head for domestic purposes for average conditions is taken as :

(*a*) 75 lit/day (*b*) 100 lit/day

(*c*) 135 lit/day (*d*) 155 lit/day

(*e*) 175 lit/day.

(iv) The acration of water is done for the removal of

(*a*) Hardness (*b*) Turbidity

(*c*) Colour (*d*) Odour

(*e*) Dissolved solids.

(v) *Zeolite is

(*a*) A naturally occurring salt (*b*) Hydrated silica

(*c*) Silicon carbide (*d*) Dehydrated calcium silicate

(*e*) Hydrated alumino-silicate.

(vi) *Addition of alum in water treatment increases :

(*a*) Acidity and Hardness of water

(*b*) Alkalinity and Hardness of water

(*c*) Sulphates and Sulphides in water

(*d*) Carbonates and Bicarbonates in water

(*e*) None of the above.

(vii) The pipes for branches in bathrooms and lavatories in domestic water supply are usually

(*a*) 10 mm (*b*) 12 mm

(*c*) 15 mm (*d*) 20 mm

(*e*) 25 mm.

(viii) The joints on cast-iron pipes for water supply are generally

(*a*) Flanged

(*b*) Welded

(*c*) Spigot and socket (turned and bored)

(*d*) Spigot and socket (lead)

(*e*) None of the above.

(ix) If with the minimum quantity of potassium permanganate, pink colour disappears soon, then it show that
(a) The water is hard and alkaline
(b) The water is soft and acidic
(c) Organic matter is present in water
(d) Organic matter is not present in water
(e) Chlorine content is high in water.

(x) Priming of a centrifugal pump is necessary
(a) If it is located above the reservoir level
(b) If it is located below the reservoir level
(c) If it is located at the reservoir level
(d) If delivery head is high.

(xi) As minimum of chlorine as possible is used for disinfection
(a) To reduce the cost of disinfection
(b) To protect consumers from potential health risks
(c) To avoid adverse consumer from potential health risks
(d) To protect distribution system from possible corrosion.

(xii) The disease 'Methaemoglobinaemia' is caused when drinking water supply contains high concentrations of
(a) Fluoride (b) Iron
(c) Nitrite (d) Nitrate
(e) Chloride.

Q. 16. Choose the correct statement in the following :

(i) The pipe joint commonly used in pumping station is
(a) flexible joint (b) expansion joint
(c) flanged joint (d) socket and spigot joint.

(ii) If the diameter of the main pipe is taken less than economical diameter then
(a) head loss will be high (b) cost of pipe will be less
(c) cost of pumping will be more
(d) all the above.

(iii) The permissible limit of nitrate content in potable water is
(a) 25 ppm (b) 45 ppm
(c) 100 ppm (d) 150 ppm.

Q. 17. Fill in the blanks :

(i) The influent to the rapid sand filters is pretreated with .....
(ii) The average detention period in sedimentation water tank is .....
(iii) The product of H^+ ions and OH^- ions in water solution is .....

Q. 18. State whether the following statements are true or false :

(i) Mottling of teeth is associated with the presence of sulphur water.
(ii) Rapid sand filters are usually cleaned in 2 to 3 months.

Q. 19. Match the following sets :

Set A	Set B
(i) Acidity	(I) Particulate matter
(ii) Alum	(II) Oils and greases
(iii) Cyclone	(III) Flocculation
(iv) Skimming tank	(IV) Carbon dioxide.

Q. 20. Choose the correct statement in the following :

(i) Presence of higher algal content in water indicates that water is
(a) Having high BOD (b) Free from oxygen
(c) Saturated with oxygen (d) Obeying none above.

(ii) Maximum permissible limit of fluoride in drinking water should not exceed
(a) 0.5 ppm (b) 1.5 ppm
(c) 5.0 ppm (d) 10.0 ppm.

(iii) Iron and manganese content in domestic water should not exceed
(a) 0.35 ppm (b) 3.5 ppm
(c) 35 ppm (d) 350 ppm.

(iv) In a right angle triangular notch if θ is the discharge, H is the head over the crest, then
(a) $\theta \propto H$ (b) $\theta \propto \frac{1}{H}$
(c) $\theta \propto H^{3/2}$ (d) $\theta \propto H^{5/2}$.

(v) In a tube-well, the yield will
(a) Increase with higher drawdown
(b) Decrease with higher drawdown
(c) Not depend on drawdown
(d) Maximum at zero drawdown.

(vi) In water distribution system at consumers end, the minimum residual gauge pressure should be
(a) 0.7 kg/cm^2 (b) 1.0 kg/cm^2
(c) 1.4 kg/cm^2 (d) 5.0 kg/cm^2.

(vii) For internal water distribution system within building, the most suitable material amongst the given four is
(a) Steel pipes (b) C.I. pipes
(c) R.C.C. pipes (d) G.I. pipes.

(viii) If V is the velocity of flow in ground water and S is the slope of the hydraulic gradient line and the conditions obeying Darcy's equation, then
(a) $V \propto S$ (b) $V \propto S^2$
(c) $V \propto \frac{1}{S}$ (d) None as (a), (b) or (c).

(ix) Molecular formula of bleaching powder is
(a) ClO_2 (b) $CaCl_2$
(c) $Ca(OH)_2$ (d) $CaO(Cl)Cl$.

(x) Aeration of water is employed to remove
(a) Iron (b) Hardness
(c) Alkalinity (d) Oxygen.

(xi) The disinfection by chlorination is most efficient at pH
(a) 2.0 (b) 5.0
(c) 7.0 (d) 10.0.

(xii) If h_f is the head loss, Q the discharge, l the length of pipeline, f the friction factor and d the diameter of pipe, then a relationship may be given by

(a) $h_f = \frac{f l Q^2}{12.1 d^5}$ (b) $h_f = \frac{f l^2 Q^2}{d^2}$

(c) $h_f = \frac{f l_Q}{d}$ (d) $\frac{f l Q^2}{4 d^2}$.

(xiii) If Q in m^3/s be the discharge of a pump against a head of H in m and w in kg/m^3 is the unit wt. of water, then horse power of the pump is given by
(a) QwH (b) QwH^2

(c) $\frac{QwH}{75}$ (d) $\frac{Q^2wH^2}{2}$.

(xiv) For rural water supply where house connections are not contemplated, per capita supply should be
(a) 7 l.p.c.d. (b) 25 l.p.c.d.
(c) 40 l.p.c.d. (d) 100 l.p.c.d.

(xv) For rural water supply where house connections are contemplated, per capita supply should be
(a) 40 l.p.c.d (b) 50 l.p.c.d.
(c) 60 l.p.c.d. (d) 70 l.p.c.d.

Q. 21. Fill in the blanks :

(i) Dental caries in children may be caused due to the deficiency of ..... in the food and water.

(ii) Blue baby disease may be caused in infants due to water supplies containing higher concentration of .....

(iii) Total hardness of water sample is 500 mg/L as $CaCO_3$; if the total alkalinity of the sample is 270 mg/L of $CaCO_3$, the temporary (carbonate) hardness of water should be ..... mg/L as $CaCO_3$.

(iv)* If mineral acids are not present, the carbonic acid alone cannot depress the pH of the water beyond .....

(v) Hazen-William C for a 20 year old C.I. pipe could be around .....

(vi) In the analysis of water, 'muffle furnace' is used to estimate ..... in the sample.

(vii) Iron in water supplies may cause colour problem if its concentration exceeds ..... mg/L.

(viii) Product of molar concentration of $[H^+]$ and $[OH^-]$ in any water sample is .....

Q. 22. Choose the correct answer in the following :

(i) Growth of algae in a natural water body tends to increase the latter's
(a) turbidity (b) alkalinity
(c) hardness (d) acidity.

(ii) The valve which allows uni-directional flow of water in a pipe is called :
(a) reflux valve (b) gate valve
(c) sluice valve (d) washout valve.

(iii) Recarbonation after lime-soda process of water treatment is practiced for the :
(a) removal of excess lime and soda from water
(b) removal of non carbonate hardness
(c) recovery of lime and soda
(d) conversion of precipitates back to soluble form.

(iv) Rotary drills operate rapidly (in connection with the construction of deep tubewells) when the type of formation encountered is :
(a) loose boulders (b) clay and silt
(c) metamorphic rock (d) limestone with small cracks.

Q. 23. Fill in the blanks :

(i) The free residual chlorine maintained in supplied water at all points of the distribution system is about .....

(ii) Two systems of supply of water are ..... and .....

Q. 23. Choose the correct answer :

(i) The removal of bacteria is accomplished in slow sand filter by means of a jelly like substance :
(a) Zoogloeal (b) Metabolism
(c) Schmutdecke (d) Phytoplankton.

(ii) Pressure filter is used mostly in :
(a) Urban water supply schemes
(b) Rural water supply schemes
(c) Swimming pools
(d) Industrial schemes.

(iii) Overflow pipes are provided in service reservoir at :
(a) Floor level of tank (b) Full supply level
(c) Bottom of tank (d) Roof level of tank.

Q. 24. State whether true or false.

(i) The mass curve is used to work out storage capacity of a reservoir.

(ii) The Kuitchling's formula for calculation of water demand for fire is $\frac{1136.5P}{P+10}$.

(iii) The water taken from infiltration gallery is free from suspended solids.

(iv) The dead end system is adopted in irregularly expanding localities.

Q. 25. Choose the correct answer :

(i) As compared to arithmetical increase method of forecasting population, geometric increase method gives :
(a) lower value (b) higher value
(c) same value
(d) sometimes higher value and sometimes lower value depending on the population figure.

(ii) The design period for a water supply project is generally taken as :
(a) 10 years (b) 20-30 years
(c) 50 years (d) 50-100 years.

(iii) A potable water is one which is :
(a) unpolluted
(b) unpolluted and free from disease producing bacteria
(c) unpolluted and free from disease producing bacteria and toxic substances
(d) unpolluted, tree from pathogens and organic matter.

(iv) Water losses in a water supply system are taken as :
(a) 5-10% (b) 10-20%
(c) 20-30% (d) 30-40%.

(v) Which metal is toxic is drinking water even at low concentration :
(a) aluminium (b) chromium
(c) copper (d) zinc.

(vi) The apparatus in which the turbidity is measured as a function of intensity of light scattered as it passes through the water sample is called :
(a) spectrometer (b) nephelometer
(c) tintometer (d) turbidimeter.

(vii) EDTA (Ethylene Diamine Tetra Acetic Acid) method is used to determine :
(a) chlorides in water (b) turbidity of water
(c) iron in water (d) hardness of water.

(viii) Alum as a coagulant is most effective in the pH range :
(a) 2.5—4.5 (b) 4.5—6.5
(c) 6.5—8.5 (d) 8.5—10.5.

(ix) Slow sand filters can remove bacteria as much as :
(a) 70—80% (b) 80—90%
(c) 90—95% (d) 96—97%.

(x) In distribution pipes, air valves are provide, at :
(a) lower points (b) higher points
(c) junction points (d) any where.

(xi) The basic purpose of surge tank is :
(a) to remove impurities from water
(b) to supply more water when needed
(c) to increase velocity of flow in a pipe
(d) to provide safety against water hammer.

(xii) The water mains are provided usually of cast iron as :
(a) it is very desirable
(b) it is able to resist very high pressure
(c) it is resistant to corrosion
(d) its discharge remains uneffected with the service time.

Q. 26. Choose the correct answer :

(i) The peak flow factor in water demand in relation to average daily demand is :
(a) 1.8 (b) 1.5
(c) 2.7 (d) 2.0.

(ii) The distribution system in water supplies is designed on the basis of :
(a) average daily demand (b) peak hourly demand
(c) coincident draft (d) greater of (b) and (c).

Q. 27. Fill in the blanks :

(i) The under-drainage system of a slow sand filter consists of pipes which are laid with ..... .

(ii) Hardness in water is determined by a standard test known as ..... .

(iii) For a newly laid pipeline, alongwith the pressure test, ... test is also conducted.

(iv) Dissolved gases and odour from water can be removed by ..... .

(v) The most commonly used disinfectant in water purification is ..... .

(vi) The most commonly used method of water supply in village is ..... .

Q. 28. Fill in the blanks :

(i) ..... source of water is unreliable.

(ii) The surface loading in the design of sedimentation tank is independent of ..... .

(iii) The causative bacteria for jaundice is ..... .

Q. 29. State whether the following statements are true or false :

(i) The bacteria in sewage/water sample is detected by circumstantial evidences.

(ii) The removal of suspended solids is necessary before effective disinfection.

(iii) Gastroenteritis is a viral disease.

Q. 30. Match the following :

Column I	*Column II*
(*i*) Circular or ring system	(*a*) Hydrocarbon, carbon monoxide, Nitrous oxide
(*ii*) Depletion of oxygen in stream	(*b*) Mottling of teeth
(*iii*) Inverted sewer	(*c*) Carry sewage below road/pavement
(*iv*) Excess fluorides in water	(*d*) Fish killing
(*v*) Automobile exhaust	(*e*) Suitable layout of pipes for well planned locality

Q. 31. Choose the correct answer :

(i) Which one of the operations is termed as development of tubewell?

(*a*) Drilling of the tubewell

(*b*) Lowering of strainer pipe in the tubewell

(*c*) Improving yield of tubewell

(*d*) Measuring yield of tubewell.

(ii) In the flocculator, water is gently churned to

(*a*) accelerate floc particles

(*b*) accelerate coagulant reactions

(*c*) prevent settling of floc

(*d*) mix the coagulant uniformly.

Q. 32. Choose the correct answer :

(i) The settling mostly encountered in water treatment for primary sedimentation is

(*a*) discrete settling (*b*) flocculent settling

(*c*) zone settling (*d*) None of the above.

Q. 33. State whether the following statements are True or False :

(i) When head loss is high, the rapid sand filters are back washed.

(ii) Most probable number of bacteria in drinking water should be zero.

(iii) Viruses in drinking water can be killed by normal disinfection.

Q. 34. Fill in the blanks :

(i) The average detention period in sedimentation water tank is ..... .

Q. 35. State whether the following statements are True or False :

(i) The disease Methaemoglobinaemia is caused when drinking water supply contains high concentration of chlorides.

(ii) Mottling of teeth is associated with excess presence of fluorides.

Q. 36. Match the following sets :

Set A	*Set B*
(*i*) Acidity	(*a*) Particulate matter
(*ii*) Alum	(*b*) Oils and grease
(*iii*) Cyclone	(*c*) Flocculation
(*iv*) Skimming tank	(*d*) Carbon dioxide

Q. 37. Choose the correct answer :

(i) The suitable layout of distribution system for a city with roads of rectangular pattern is

(*a*) grid iron system (*b*) dead end system

(*c*) ring system (*d*) radial system.

(ii) The suitable method for disinfection of swimming pool water is

(*a*) ultraviolet rays treatment

(*b*) lime treatment

(*c*) by using potassium permanganate

(*d*) chlorination.

(iii) The causative organism for jaundice is

(*a*) Escherichia coli (*b*) Viruses

(*c*) Salmonella typhi (*d*) Poliomyelitis.

Q. 38. Fill in the blanks :

(i) ..... is the indicator organism for assessing the bacterial quality of water.

(ii) For drinking water, viruses are primarily destroyed by ..... .

(iii) Pumps are connected in ..... to increase the head and in ..... to augment the discharge.

Q. 39. State whether the following statements are True or False :

(i) Bacteriological examination of water for the presence of Escherichia coliform is conducted as coliform organisms are pathogenic.

(ii) Quality of water obtained from infiltration galleries is generally better than that obtained from intakes.

Q. 40. Match the following :

(*i*) Tricking filter	(*a*) Do not require oxygen
(*ii*) Nitrates	(*b*) Oxidation pond
(*iii*) Zeolites	(*c*) Attached growth process
(*iv*) Anaerobes	(*d*) Cause mathemoglobinemia
(*v*) Bacterial algal symbiosis	(*e*) Remove hardness in water

Q. 41. Select correct answer :

(i) Negative head in rapid sand filters

(*a*) decreases the rate of filtration

(*b*) improves the filter performance

(*c*) releases dissolved air and gases in water

(*d*) adds dissolved air in water.

(ii) If pH of water is increased from 7 to 8
(a) hydrogen ion concentration is doubled
(b) hydrogen ion concentration is halved
(c) alkalinity is increased ten folds
(d) hydrogen ion concentration decreases 10 folds.

(iii) Compared to rapid sand filter, slow sand filter
(a) is better for colour removal
(b) is better in the removal of colloids and microorganism
(c) requires more back wash water due to large area
(d) entails high maintenance cost.

(iv) The water supply pipeline of the following material, which has long life and is durable, not easily corroded but heavy and brittle, is
(a) ductile iron (b) cast iron
(c) mild steel (d) reinforced cement concrete.

Q. 42. Choose the correct answer :

(i) Mathemoglobinemia (blue baby syndrome) is due to excess of the following in water :
(a) Coliforms (b) Nitrates
(c) Sulphates (d) Iron.

(ii) Perched aquifers generally occur
(a) below water table (b) above water table
(c) in acquicludes (d) in Artesian aquifers.

(iii) Copperas is the commercial name for :
(a) aluminium sulphate (b) ferrous sulphate
(c) ferric sulphate (d) copper sulphate.

(iv) In rapid sand filters, the ratio of length to diameter should not exceed :
(a) 10 (b) 15
(c) 20 (d) 25.

(v) Bleaching powder is
(a) ClO_2 (b) $CaCl_2$
(c) $Ca(OH)_2$ (d) $Ca(OCl)_2$.

(vi) PVC pipes can withstand pressure head of water up to :
(a) 25 m (b) 50 m
(c) 75 m (d) 100 m.

(vii) The chloride content in treated water for public supplies should not exceed
(a) 100 ppm (b) 150 ppm
(c) 200 ppm (d) 250 ppm.

Q. 43. Fill in the blanks :

(i) If the settling velocity of particle is 1.4 mm/s and overflow rate is 0.056 mm/s, the percentage of solids removed in sedimentation tank is ..... .

Q. 44. State whether the following statements are True or False :

(i) Wholesomeness is must, while palatability is desirable for drinking water.

(ii) Nitrate is an indicator of recent water pollution.

(iii) Slow sand filter gives biologically good quality of water.

Q. 45. Choose the correct answer :

(i) E-Coliforms are used as ideal pathogen indicator of the contamination of water with domestic wastewater, because :
(a) Pathogens and coliforms are of the same origin (human intestinal tract)
(b) These are always absent when pathogens are present
(c) They are always present when pathogens are absent
(d) All of the above.

Q. 46. Match the following :

(i) (a) Mathemoglobinemia	1. SSF
(b) Arithmetic method	2. Flocculation
(c) Telescopic outlet	3. Nitrates
(d) Orthokinetic	4. Forecasting population

Q. 47. Choose the correct answer :

(i) The maximum permissible fluoride content in drinking water is
(a) 50 mg/L (b) 1 mg/L
(c) 20 mg/L (d) 5 mg/L.

(ii) The following is an algicide
(a) Alum (b) Bleaching powder
(c) Copper sulphate (d) Lime.

(iii) The discharge of a centrifugal pump
(a) decreases with increase of the head
(b) is independent of head
(c) increases with increase of the head
(d) None of the above.

(iv) The colour of water is measured on
(a) turbidity scale (b) threshold scale
(c) platinum-cobalt scale (d) calcium carbonate scale.

Q. 48. Choose the correct answer :

(i) Presence of high algal content in water indicates water is
(a) hard (b) soft
(c) acidic (d) alkaline.

(ii) Zeolite is a
(a) natural salt (b) hydrated silica
(c) silicon carbide (d) hydrated aluminium silicate

(iii) Pipes for branches to bath rooms and labatories in domestic water supply are usually
(a) 5 mm (b) 12 mm
(c) 20 mm (d) 25 mm.

(iv) Priming of the centrifugal pump is necessary
(a) if it is located above the reservoir level
(b) if it is located below the reservoir level
(c) if the delivery head is high
(d) if the discharge is high.

(v) *Schistosomiasis is caused by
(a) Bacteria
(b) Protozoa
(c) Virus
(d) Crustacean.

(vi) Air valves are provided in water mains at
(a) pipe junctions (b) summits
(c) low points (d) near service pipes.

(vii) If w is the weight of water in kg per cubic metre, Q is the discharge in m^3/s and H is the total head, the water horse power of pump is
(a) $\frac{wQH}{75}$ (b) $\frac{wQH}{360}$
(c) $\frac{wQH}{220}$ (d) $\frac{wQH}{550}$

(viii) Maximum permissible colour for domestic water supply on cobalt scale is
(a) 5 ppm (b) 10 ppm
(c) 15 ppm (d) 20 ppm.

(ix) For rural water supply, where house connections are not contemplated, per capita supply should be
(a) 7 lpcd (b) 25 lpcd
(c) 40 lpcd (d) 70 lpcd.

Q. 49. Choose the correct statement/answer(s) :

(i) Coincident draft is
(a) maximum hourly demand + fire demand
(b) maximum daily demand + fire demand
(c) maximum daily demand + public demand
(d) maximum hourly demand + industrial demand.

(ii) Coagulation with alum increases
(a) acidity and hardness of water
(b) Sulphates in water
(c) Carbonates and bicarbonates of water
(d) None of the above.

(iii) Schmutzedecks will be formed in
(a) slow sand filters (b) Rapid sand filters
(c) Pressure filters (d) Trickling filters.

Q. 50. Match the pairs in the following sets :

(i) Flocculator (a) Orthotolodine
(ii) Residual chlorine (b) Soluble BOD
(iii) Trickling filter (c) Velocity gradient.

ANSWERS

1 (i). 1 μm (micron) 1 (ii). True 1 (iii). 0.3
2 (i). True 2 (ii). True 2 (iii). False 2 (iv). True
2 (v). True
3 (i). 23 3 (ii). 5 3 (iii). carbonate
4 (i)—(a)
(ii)—(a)
(iii)—(b)
5 (i). True 5 (ii). False 5 (iii). False
6 (i). (c) 6 (ii). (d) 6 (iii). (a)
7. (i). (d) 7 (ii). (c) 7 (iii). (a) 7 (iv). (c)
8 (i). 0.5 8 (ii). geometric increase
8 (iii). hydrogen ion
9. (i)—(b)
(ii)—(a)
(iii)—(e)
(iv)—(c)
(v)—(d)
10 (i). False 10 (ii). True 10 (iii). False 10 (iv). True
11 (i). (c) 11 (ii). (b)
12 (i). 10^{-3}, 1 12 (ii). 6 12 (iii). 0.3
13 (i). False 13 (ii). False 13 (iii). True
14. (i)—(c), (ii)—(b), (iii)—(a)
15 (i). (d) 15 (ii). (b) 15 (iii). (c) 15 (iv). (d)
15 (v). (a) 15 (vi). (a) 15 (vii). (a) 15 (viii). (d)
15 (ix). (c) 15 (x). (a) 15 (xi). (c) 15 (xii). (d)
16 (i). (c) 16 (ii). (d) 16 (iii). (b)
17 (i). clariflocculators or coagulation-sedimentation
17 (ii). 4 hours 17 (iii). 10^{-14} moles/Litre

18 (*i*). False 18 (*ii*). False
19 (*i*)—(*IV*), (*ii*)—(*III*), (*iii*)—(*I*), (*iv*)—(*II*)
20 (*ii*). (*b*) 20 (*iii*). (*a*) 20 (*iv*). (*d*) 20 (*v*). (*a*)
20 (*vi*). (*c*) 20 (*vii*). (*d*) 20 (*viii*). (*a*) 20 (*ix*). (*d*)
20 (*x*). (*c*) 20 (*xi*). (*c*) 20 (*xii*). (*a*) 20 (*xiii*). (*c*)
20 (*xiv*). (*b*) 20 (*xv*). (*b*)
21 (*i*). (Fluoride) 21 (*ii*). (Nitrates)
21 (*iii*). (270 mg/L) 21 (*iv*). (5.6)
21 (*v*). (100) 21 (*vi*). (Volatile solids)
21 (*vii*). (0.3 mg/L) 21 (*viii*). (10^{-14} moles/L)
22 (*i*). (*a*) 22 (*ii*). (*a*) 22 (*iii*). (*d*) 22 (*iv*). (*b*)
23 (*i*). (*c*) 23 (*ii*). (*c*) 23 (*iii*). (*b*)
24 (*i*). (True) 24 (*ii*). (False) 24 (*iii*). (True) 24 (*iv*). (True)
25 (*i*). (*b*) 25 (*ii*). (*b*) 25 (*iii*). (*c*) 25 (*iv*). (*b*)
25 (*v*). (*b*) 25 (*vi*). (*b*) 25 (*vii*). (*d*) 25 (*viii*). (*c*)
25 (*ix*). (*d*) 25 (*x*). (*b*) 25 (*xi*). (*d*) 25 (*xii*). (*c*)
26 (*i*). (*c*) 26 (*ii*). (*d*)
27 (*i*). open joints @ 3-5 m apart and sloping towards the main covered central drain
27 (*ii*). EDTA method 27 (*iii*) leakage
27 (*iv*). by Aeration and by adding Activated carbon before filtration (before or after coagulation) in the water treatment process.
27 (*v*). Chlorine 27 (*vi*). through wells and tubewells.
28 (*i*) Turbid 28 (*ii*) depth of tank
28 (*iii*) hepatitis virus.
29 (*i*) True 29 (*ii*) True 29 (*iii*) False
30 (*i*)—(*e*)
(*ii*)—(*d*)
(*iii*)—(*c*)
(*iv*)—(*b*)
(*v*)—(*a*)
31 (*i*)—(*c*) 31 (*ii*)—(*d*)
32 (*i*) (*b*) 33 (*i*) True 33 (*ii*) False 33 (*iii*) True
34 (*i*) – 4 hr 35 (*i*) False 35 (*ii*) True
36 (*i*)—(*d*)
(*ii*)—(*c*)
(*iii*)—(*a*)
(*iv*)—(*b*)
37 (*i*)—(*a*) 37 (*ii*)—(*d*) 37 (*iii*)—(*b*)
38 (*i*)—(E. coli) 38 (*ii*)—(Chlorine)
38 (*iii*)—(Series, parallel) 39 (*i*)—(False)

39 (*ii*)—(True) 39 (*iii*)—(True) 40 (*i*)—(*c*)
(*ii*)—(*d*)
(*iii*)—(*e*)
(*iv*)—(*a*)
(*v*)—(*b*)
41 (*i*)—(*c*) 41 (*ii*)—(*c*) 41 (*iii*)—(*b*)
41 (*iv*)—(*b*)
42 (*i*)—(*b*) 42 (*ii*)—(*b*) 42 (*iii*)—(*b*)
42 (*iv*)—(*a*) 42 (*v*)—(*d*)
42 (*vi*)—(*d*) 42 (*vii*)—(*d*) 43 (*i*)—(100%)
44 (*i*)—(True) 44 (*ii*)—(False) 44 (*iii*)—(True)
45 (*i*)—(*a*) 46 (*i*)—(*a*)—(3) 47 (*i*)—(*b*)
(*b*)—(4)
(*c*)—(1)
(*d*)—(2)
47 (*ii*)—(*c*) 47 (*iii*)—(*a*) 47 (*iv*)—(*c*)
48 (*i*)—(*d*) 48 (*ii*)—(*a*) 48 (*iii*)—(*c*)
48 (*iv*)—(*a*) 48 (*v*)—(*d*) 48 (*vi*)—(*b*)
49 (*i*)—(*b*) 49 (*ii*)—(*b*) 49 (*iii*)—(*a*)
50 (*i*)—(*c*)
(*ii*)—(*a*)
(*iii*)—(*b*)

Hints for Solving Starred Objective Questions of AMIE Exams.

Q. 1. (i) Particles larger than 1 micron *i.e.* 1 μm (10^{-3} mm) are classified as **suspended particles**; between 10^{-3} μm to 1 micron as **colloids**; and particles smaller than 10^{-3} μm are classified as **dissolved solids.**

Q. 2. (iv) If the pH of a water is less than 8.3, the entire alkalinity exists in the form of bicarbonates.

Q. 3. (ii) As stated in the Text, the rate of filtration for rapid sand filters is kept in the range of 3000—6000 *l*/h/m² *i.e.* 3—6 m³/h/m² (m/h). A figure of 4.8 m/h is recommended by GoI manual. A figure of 5 m/h has however, been traditionally considered to be a conservative standard value, although filtration rates as high as 10—12 m/h have been obtained.

Q. 14. Recarbonation. During the softening of hard water in lime soda process, insoluble $Ca(CO_3)_2 \downarrow$ and $Mg(OH)_2 \downarrow$ are formed, which are removed by sedimenting the water in a sedimentation tank. However, the effluent of this sedimentation tank may still contain some quantity of their finally divided particles, which if not removed, may cause troubles by getting deposited in the filter or in the pipes of the distribution system. In order to avoid such

difficulties, the sedimented water is recarbonated by passing CO_2 gas through it, in a *recarbonation plant*, to convert the residual insoluble $Ca(CO_3)_2$ into soluble $Ca(HCO_3)_2$. *Recarbonation is, thus, found associated with lime soda process as discussed in article 9.4 in the Text.*

Trihalomethanes (THMs) like *trichloromethane (Chloro-form), bromodichloromethane*, etc. are found to have been formed in chlorinated drinking waters, due to the interaction of chlorine (applied for disinfection) with the organic substances like humic and fulvic acids naturally present in the water. Such organic substances are usually produced from the decaying vegetation. The above reaction can be represented as :

Chlorine + (bromide ion + iodide ion) + organic substances = Trihalomethanes and other halogenated compounds.

Following chlorination, the formation of THM's is not instantaneous, but continues for an extended period of time. Concentration of THMs can, thus, increase in chlorinated water held in the distribution system. Since THM's are found to be carcinogenic in nature, their total 12 month running average concentration in drinking water should stringently be limited to 0.1 mg/l. This testing should be carried out on atleast 4 samples per quarter (25% samples to be collected from extremities of the pipe network, and 75% from the population distribution). The total THM concentration is the sum of the individual concentrations of $CHCl_3$, $CHBrCl_2$, $CHBr_2Cl$ and $CHBr_3$.

Trihalomethanes are thus closely associated with chlorination. Hence, choice matching is between (*ii*) and (*b*).

Activated carbon, as discussed in article 9.25.2, is used to remove tastes and odours from public supplies. Hence, the choice matching between (*iii*) and (*a*) is quite evident.

Q. 15. (i) CO_2 is consumed during photosynthesis, which helps in the growth of algae. Since CO_2 causes acidity in water due to formation of carbonic acid as $CO_2 + H_2O \longrightarrow H_2CO_3$ (Carbonic acid), its consumption during photosynthesis, will reduce acidity, thereby increasing the alkalinity of the water body supporting algal growth, upto a pH level of 9 to 10. *Hence, a water, supporting higher algal growth, is always alkaline.* Such a water will also contain lesser turbidity, because greater turbidity will check sun rays, retarding photosynthesis. Hence, such a water will be less turbid, but will not necessarily be free of turbidity. Therefore, the choice (*d*) is correct and not the choice (*e*). **Ans.**

Q. 15. (v) Zeolite infact is a hydrated sodium-aluminium-silicate. It is available in nature in the form of green sand or clay, which due to the presence of sodium, can be termed as a 'naturally occurring salt'. Since all other choices except (*a*) donot match with the above definition of a zeolite, we may reject all other choices, and can accept (*a*) as correct. **Ans.**

Q. 15. (vi) The alum, when added to raw water, reacts with the bicarbonate alkalinity, which generally exists in raw supplies, as

$$Al_2(SO_4)_3 \cdot 18H_2O + 3Ca(HCO_3)_2 \longrightarrow 3CaSO_4 + 2Al(OH)_3 \downarrow + 6CO_2 \uparrow$$

The $CaSO_4$, so produced, imparts permanent hardness to water ; and the produced CO_2 gas causes increased acidity. *Hence, the addition of alum increases the acidity and hardness of water.* The choice (*a*) is, therefore, correct. **Ans.**

Since the alkalinity is consumed and acidity is produced in water due to the addition of alum, external alkalinity is added when alum is used as a coagulant.

Q. 21. (iv) Carbonic acid as you know is formed by the dissolution of CO_2 gas in water as:

$$CO_2 + H_2O \longrightarrow \underset{\text{Carbonic acid}}{H_2CO_3}$$

It may dissociate to bicarbonate ions (HCO_3^-) and H^+ ions as:

$$H_2CO_3 \longrightarrow HCO_3^- + H^+$$

It, thus contains acidity as well as alkalinity. Hence, it is a **weak acid.**

The pH of carbonic acid is of the order is 5.6 – 5.7. It is infact the presence of mineral acids particularly HCL which lowers the pH value of acid rain below 5.6 (since pH of HCL various from 0 – 2), otherwise carbonic acid which is generally present in acid rain, will not be able to lower its pH value to less than about 5.6.

Hence, in the absence of mineral acids, carbonic acid cannot depress pH below 6.5 or so.

Q. 48. (v) **Schistosomiasis,** also known as **Bilharzia** or **Sleeping Sickness** is caused by snails carrying the intermediate vector for the disease. These snails (***crustaceans***)* thrive in clean water canals in many tropical countries, and can create a major health hazard. *Molluscicides* are needed to control the snail, combined with adequate education of the local population with regard to sensible personal hygiene practice.

*The **Crustacean** is an aerobic, heterotrophic and multicellular animal, having a hard body or shell. They constitute an important source of food for fish, and as such, are normal occupants in most natural waters. Except for their occasional presence in under-loaded oxidation ponds, they donot exist in biological treatment systems to any noticeable extent. Their presence is indicative of an effluent that is low in organic matter and high in D.O.

Rotifers, like crustaceans, are also aerobic heterotrophic and multicellular animals, having two sets of rotating cilia on its head which are used for motality and capturing food. Rotifers are very effective in consuming dispersed and flocculated bacteria, and

small particles of organic matter. Their presence in an effluent indicates a high efficient aerobic biological purification process.

Protozoa. Protozoa are *motile, microscopic* single cell animals, representing the simplest and lowest form of animal life. They can be directly counted under a microscope. The majority of protozoa are aerobic heterotrophs, although a few are anaerobic. Protozoa are generally in order of magnitude *larger than bacteria* and often consume bacteria as an energy source. In effect, the protozoa act as polishers of the effluents from biological waste treatment processes by consuming bacteria and particulate organic matter. They are usually divided into the following five groups :

(*i*) *Sarcodina*

(*ii*) *Mastigophora*

(*iii*) *Sporozoa*

(*iv*) *Infusoria o.· Ciliate*, and

(*v*) *Suctoria.*

Virusus are the smallest biological structure (*smaller than bacteria*), containing all information necessary for its reproduction. Viruses are so small that they can be seen only with an electron microscope. They are obligate parasites, and as such, require a host in which to live. Once they have a host, they redirect its complex machinery to produce new virus particles. Eventually, the host cell ruptures, releasing new virus particles, which can go on to infect new cells.

Viruses are usually classified by the host that they infect. Many viruses that produce disease in man are also known to be excreted in the feces of man. Thus, in wastewater treatment, the engineers must ensure the effective removal of viruses. This is usually done by chlorination and proper disposal of plant effluent.

Appendix Tables

Appendix Table A-1. Existing Water Rates for Domestic Water Supplies over Various States of India

S.No.	*Name of State*	*Prevailing Water Rates for Domestic Water Supplies*
(1)	(2)	(3)
1.	Andhra Pradesh	NA
2.	Assam	NA
3.	Bihar	₹ 4.50 per 4.5 kL
4.	Chhatisgarh	₹ 0.20 P wef. 1.4.2000 with annual increase of 2 Paisa
5.	Delhi	(*i*) *For metered homes* (*a*) ₹ 0.35 P + 5% per kL upto 10 kL per month (*b*) ₹ 1.00 + 50% per kL from 10 to 20 kL per month (*c*) ₹ 1.50 + 50% per kL from 20 to 30 kL per month (*d*) ₹ 2.00 + 50% per kL above 30 kL per month Minimum charges per month per connection = ₹ 20 + 50% (*ii*) *For unmetered homes* (*a*) ₹ 20 + 50% per month — upto 3 months (*b*) Double the above rate after 3 months.
6.	Goa	(*i*) ₹ 2.00 per kL upto 30 kL per month (*ii*) ₹ 4.00 per kL above 30 kL per month Minimum charges @ ₹ 20.00 per month
7.	Gujarat	₹ 0.50 P per kL
8.	Haryana	₹ 10.00 per 70 kL (2500 cu ft.)
9.	Himachal Pradesh	₹ 2.00 per kL in Urban areas ₹ 10.00 per connection in Rural areas ₹ 5.00 per connection in Tribal areas

(1)	(2)	(3)
10.	J & K	(*i*) *For metered homes* ₹ 3.60 per kL (*ii*) *For unmetered homes* (*a*) ₹ 15.00 per connection per month for ½" pipe connection (*b*) ₹ 28.00 per connection per month for ¾" pipe connection
11.	Jharkhand	₹ 4.50 per 4.5 kL
12.	Karnataka	₹ 375.00 per 28000 kL (M cuft) for drinking purposes, which works out to **₹ 0.013 per kL,** ₹ 12.00 per 1000 kL where maintenance is entrusted to WUC's, which works out to **₹ 0.012 per kL**
13.	Kerala	(*a*) ₹ 20.00 per month (Min. charges upto 12.68 kL consumption) (*b*) ₹ 3.00 per 1.27 kL (for consumption above 12.68 kL and upto 38.04 kL) with min. charges of ₹ 80.00 upto 38.04 kL. (*c*) ₹ 5.00 per 1.27 kL (for consumption from 38.04 kL to 63.04 kL), with min. charges of ₹ 180.00 upto 63.41 kL (*d*) ₹ 7.35 per 1.27 kL (for consumption from above 63.41 kL)
14.	Madhya Pradesh	₹ 0.20 P per kL wef. 1.4.2000 with annual increase of ₹ 0.02 P.
15.	Maharashtra	(*a*) ₹ 1.00 per 10 kL for reservoirs of dams constructed on rivers (*b*) ₹ 2.50 per 10 kL from storage tanks (through canals of dams) as per norm (*c*) ₹ 3.50 per 10 kL from storage tanks (through canals of dams) not as per norm (*d*) ₹ 2.00 per 10 kL from storage tanks (through river down stream of dam) as per norm

(1)	(2)	(3)
		(*e*) ₹ 3.50 per 10 kL from storage tank (through river down stream of dam) not as per norms (*f*) ₹ 0.75 P per 10 kL from dams constructed by user institutes at own cost (*g*) ₹ 0.75 P per 10 kL from dams with construction expr. in proportion to water use (*h*) ₹ 0.75 P per 10 kL from dam not constructed anywhere upstream of place of water use
16.	Manipur	₹ 0.01 P per 4.5 kL for raw water for drinking purposes supplied to SPHED
17.	Mizoram	(*a*) ₹ 10.00 per kL upto 13 kL consumption (*b*) ₹ 20.00 per kL for consumption between 13 kL & 39 kL (*c*) ₹ 30.00 per kL for consumption in excess of 39 kL New connection charges = ₹ 500.
18.	Orissa	(*a*) ₹ 30.00 per 10000 cuft (per 283 cum, or 283 kL) for drinking and washing purposes (*b*) ₹ 15.00 per 10000 cuft (per 283 cum, or 283 kL) for filling tanks for drinking purposes
19.	Punjab	(*a*) ₹ 6.00 per 6000 cuft (*i.e.* per 170 kL) (*b*) ₹ 8.00 per 2500 cuft (*i.e.* per 71 kL) for Bulk supply
20.	Rajasthan	(*a*) ₹ 0.80 P per 1000 cuft (*i.e.* per 38 kL) for drinking purposes (*b*) ₹ 1.00 per 1000 cuft (*i.e.* per 28 kL) for Bulk supply for other purposes
21.	Tamil Nadu	Various municipalities collect charges for supplying water for domestic demand to individual homes. In Chennai, a monthly rate of ₹ 50.00 per connection is being charged.

(1)	(2)	(3)
22.	Tripura	₹ 30/-, 40/-, 60/- and ₹ 100/-, respectively, per month per connection for 6 mm, 10 mm, 12.5 mm and 20 mm sized ferrule connections in municipal areas; and ₹ 30/- per month per connection in rural areas.
23.	U.P.	NA
24.	Uttranchal	NA
25.	West Bengal	₹ 64/- per month per connection in Bolpur (Urban area); and ₹ 42.50 P per month per connection in Bolpur (rural); ₹ 96/- per month per connection in Raghunathpur area (urban) ₹ 75/- per month per connection in Raghunathpur area (rural)
26.	Andaman and Nicobar Islands	(*a*) ₹ 18/- per month in urban areas (*b*) ₹ 10/- per month in rural areas (*c*) ₹ 10/- per month in Tribal areas
27.	Chandigarh UT	(*i*) *For Metered Homes* (*a*) ₹ 1.40 per kL upto first 15 kL (*b*) ₹ 2.60 per kL from 15 kL to 30 kL (*c*) ₹ 4.00 per kL above 30 kL (*ii*) *For Unmetered Homes* (*a*) ₹ 100/- per connection per month (*b*) ₹ 200/- per tanker for supply through tankers
28.	Dadar and Nagar Haveli (UT)	₹ 20/-, 35/- & ₹ 55/- per month for 12 mm, 20 mm and 25 mm connections, respectively
29.	Daman and Diu (UT)	₹ 15/-, 40/- and 100/- per month for ½", ¾" and 1" tap connections, respectively
30.	Pondicherry	(*a*) ₹ 500/- per 1000 kL for domestic purposes (*b*) ₹ 100/- per tap for public taps.

Appendix Table A-2. Existing Water Rates for Industrial and Commercial Water Supplies over Various States of India

S.No.	*Name of State*	*Prevailing Water Rates for Industrial Uses*
(1)	(2)	(3)
1.	Andhra Pradesh	(*a*) ₹ 150/- per 4.5 kL for water drawn from natural sources (*b*) ₹ 300/- per 4.5 kL for water drawn from reservoirs (*c*) ₹ 450/- per 4.5 kL for water drawn from canals
2.	Assam	NA
3.	Bihar	₹ 4.50 per 4.5 kL
4.	Chhatisgarh	(*a*) ₹ 1.50 per kL from Govt. sources (*b*) ₹ 0.45 per kL from natural sources
5.	Delhi	(*a*) ₹ 12/- per kL upto 50 kL per month (*b*) ₹ 18/- per kL for 50 to 100 kL per month (*c*) ₹ 24/- per kL for above 100 kL, per month
6.	Goa	(*a*) ₹ 20/- per kL for commercial and industries, hotels, etc. (*b*) ₹ 7/- per kL for small hotels not registered under State Sales Tax Act
7.	Gujarat	(*a*) ₹ 3.25 per kL for industries having their own source of water, or having participation in development of sources of water, or those drawing water from river nala at their own expenses, or industries of GIDC as well as Gujarat Electricity Board (*b*) ₹ 4.00 per kL for industries drawing water from Dams and Canals
8.	Haryana	₹ 100.00 per 2500 cft (*i.e.* about 71 kL) *i.e.* ₹ 1.41 per kL
9.	Himachal Pradesh	₹ 4/- per kL
10.	Jammu & Kashmir	(*a*) ₹ 500/- per connection/annum for 1/2" dia pipe (*b*) ₹ 900/- per connection/annum for 3/4" dia pipe (*c*) ₹ 2700/- per connection/annum for 1" dia pipe (*d*) ₹ 5400/- per connection/annum for 1½" dia pipe

(1)	(2)	(3)
11.	Jharkhand	₹ 4.50 per 4.5 kL *i.e.* ₹ 1/- per kL
12.	Karnataka	(*a*) ₹ 1800/- per M cft from natural sources/ rivers (*b*) ₹ 3200/- per M cft from canals or irrigation projects
13.	Kerala	₹ 10.60 per 1.27 kL (with minimum charges of ₹ 200/-) *i.e.* ₹ 8.35 per kL
14.	Madhya Pradesh	(*a*) ₹ 1/- per kL for Govt. sources (*b*) ₹ 0.30 per kL for natural/created sources
15.	Maharashtra	(*a*) ₹ 3.50 per kL from reservoirs of dams on rivers (*b*) ₹ 6.50 per kL from storage tanks (through canals of dams) as per norm (*c*) ₹ 8.50 per kL from storage tanks (through canals of dams) not as per norm (*d*) ₹ 5.00 per kL from storage tank (through river downstream of dam) as per norm (*e*) ₹ 8.50 per kL from storage tank (through river downstream of dam) not as per norm (*f*) ₹ 1.00 per kL from dams constructed by users at own cost (*g*) ₹ 1.00 per kL from dams with const. expr. in proportion to water use (*h*) ₹ 100 per kL from dams not constructed anywhere upstream of place of water use **Note :** Separate rates for industrial (raw commodity uses)
16.	Manipur	NA
17.	Mizoram	NA
18.	Orissa	₹ 250/- per lakh gallons *i.e.* 4.5 lakh litres, *i.e.* 450 kL ; *i.e.* ₹ 0.56 per kL
19.	Punjab	₹ 8/- per 2500 cft, *i.e.* ₹ 0.112 per kL
20.	Rajasthan	₹ 20/- per 1000 cft, *i.e.* ₹ 0.71 per kL
21.	Tamil Nadu	(*a*) ₹ 300/- per 1000 kL for new industries upto 5 yrs. with minimum annual charge of ₹ 3000/- (*b*) ₹ 500/- per 1000 kL for all industries after 5 yrs. with minimum annual charge of ₹ 5000/-

(1)	(2)	(3)
22.	Tripura	(*a*) ₹ 100/- per connection per month for establishments in rural areas (*b*) ₹ 200/- per connection per month for establishments in Nagar Panchayat areas (*c*) ₹ 250/- per connection per month for establishments in Municipal areas
23.	Uttar Pradesh	NA
24.	Uttranchal Pradesh	NA
25.	West Bengal	(*a*) ₹ 98/- per month/connection for Bolpur Area (urban) (*b*) ₹ 85/- per month/connection for Bolpur Area (rural) (*c*) ₹ 149/- per month/connection for Raghunathpur area (urban) (*d*) ₹ 128/- per month/connection for Raghunathpur (rural)
26.	Andaman and Nicobar Islands	₹ 15.90 per 4.5 kL *i.e.* ₹ 3.53 per kL
27.	Chandigarh (UT)	₹ 6/- per kL
28.	Dadar and Nagar Haveli (UT)	(*a*) ₹ 150/- per month per 12 mm connection (*b*) ₹ 280/- per month per 20 mm connection (*c*) ₹ 430/- per month per 25 mm connection
29.	Daman and Diu (UT)	(*a*) ₹ 75/- per month per 1/2" connection (*b*) ₹ 250/- per month per 3/4" connection (*c*) ₹ 500/- per month per 1" connection
30.	Pondicherry	₹ 1.50 per kL

Appendix Table A-3. Drinking Water Pricing around the World in U.S. dollars ($) per kL*

S.No.	Name of Country	Average Water Tariff in $ per kL
1.	India	0.03
2.	Australia	0.50
3.	U.S.A.	0.51
4.	U.K.	1.18
5.	France	1.23
6.	Netherlands	1.25
7.	Belgium	1.25
8.	Denmark	1.54
9.	Germany	1.91

***Source** : Watertech online and study by Public-Private-Infrastructure Advisory Facility (PPIAF)

Appendix Table A-4. Density of Water at Different Temperatures

Temp °C	Density of water kg / m^3
0	999.8
5	1000.0
10	999.7
15	999.1
20	998.2
25	997.0
30	992.2
40	988.0
50	983.2
60	977.8
70	971.8
80	965.3
90	958.4
100	935.0
125	918.0
150	893.0
175	863.0
200	

Appendix Table A-5. Boiling Point of Water at Different Pressures

Gauge Pressure i.e. the pressure over the local atmospheric pressure		Absolute Pressure i.e. Gauge Pressure + Local atmospheric pressure in kPa (Assume Local Atmospheric Pressure = 101 kPa)	Boiling Point of Water in °C
m head of water	kPa		
0	0	101	100
10	98.1	199.1	119.5
20	196.2	297.2	132.6
30	294.3	395.3	142.5
40	392.4	493.4	150.6
50	490.5	591.5	157.3
60	588.6	689.6	163.7
70	686.7	787.7	169.1
For Negative Gauge Pressures i.e. for Vacuum pressures, the Boiling point of water is less than 100°C, as shown below :			
– 0.2	(–) 1.96	99.04	98.92
– 0.6	(–) 5.89	95.11	97.84
– 0.9	(–) 8.83	92.17	96.76
– 1.2	(–) 11.77	89.23	95.68
– 1.5	(–) 14.72	86.28	94.60
– 1.8	(–) 17.66	83.34	93.30
– 2.5	(–) 24.53	76.47	92.03

Appendix Table A-6. Saturation Value of Dissolved Oxygen (D.O.) in Fresh Water at Different Temperatures and 1 Atmospheric Pressure (i.e. 101.325 kPa)*

Temperature (°C)	*Dissolved oxygen* (mg/l)	*Temperature* (°C)	*Dissolved oxygen* (mg/l)
0	14.62	20	9.17
1	14.23	21	8.99
2	13.84	22	8.83
3	13.48	23	8.68
4	13.13	24	8.53
5	12.80	25	8.38
6	12.48	26	8.22
7	12.17	27	8.07
8	11.87	28	7.92
9	11.59	29	7.77
10	11.33	30	7.63
11	11.08	31	7.51
12	10.83	32	7.42
13	10.60	33	7.28
14	10.37	34	7.17
15	10.15	35	7.07
16	9.95	36	6.96
17	9.74	37	6.86
18	9.54	38	6.75
19	9.35		

* For other barometric pressures, the solubilities vary approximately in proportion to the ratios of those pressures to the standard pressure.

Appendix Table A-7. Useful Conversions between Different Units

Length

1 metre = 3.281 ft ; 1 ft = 0.3048 m

1 km = 0.622 mi ; 1 mi = 1.6093 km

1 cm = 0.394 in ; 1 in = 2.54 cm

1 mm = 1000 μ (micron) 1 micron (μ) = 10^{-6} m

Area

1 m^2 = 10.761 ft^2 ; 1 ft^2 = 0.0929 m^2

1 km^2 = 10^6 m^2 = 100 ha ; 1 mi^2 = 2.59 km^2

= 0.386 mi^2 = 247 acres = 251 ha

= 640 acre

1 ha = 10^4 m^2 1 acre = 4840 sq yards

= 2.47 acres ; = 0.4047 ha

Volume and Discharge

1 m^3 = 35.307 ft^3 ; 1 ft^2 = 0.02832 m^2

1 cumec = 35.307 cusec ; 1 cusec = 0.02832 cumec

= 0.538 MGD

= 2.447 MLD

1 ha.m = 8.13 acre ft ; **1 acre ft** = 43560 ft^3

= 1233.6 m^3

= 0.123 ham

≈ $\frac{1}{2}$ **cusec day**

(*i.e.* 43200 ft^3)

1 m^3 = 1000 litres (L) **1 gallon** (*i.e.* imp gallon)

1 L = 1.76 pints (imp) = 4.546 **litres**

= 0.22 gallon (imp) 1 MGD = $\frac{1}{2}$ cusec

1 U.S. gallon = 3.785 litres.

Force

1 kg wt = 9.807 N (Newton) ; 1 N = 1 kgm/sec^2

1 N = 0.102 kg wt

1 gm wt = 9807.7 dynes

1 lb wt = 32.2 poundals

1 kg wt = 2.204 lb wt.

1 kg (wt) = 9.807 N = 2.204 lb (wt) = 9.807 × 10^5 dynes

Or

1 N = 0.102 kg (wt) = 0.2247 lb (wt) = 10^5 dynes

1 poundal = 138.26 dynes = 14.102 gm (wt).

Pressure

Units : N/m^2 = Pascal (P_a) ; kg/m^2 (*i.e.* kg (wt)/m^2) : lb (wt)/ft^2

1 kg/m^2 = 9.807 N/m^2 ; = 9.807 P_a

1 kg/cm^2 = 10^4 kg/m^2

1 Standard Atmospheric pressure
= 760 mm of Hg
= 101.325 kP_a
= 1.033 kg/cm^2
= 10.33 m water head

1 P_a = 1 N/m^2 = 0.102 kg/m^2

1 kP_a = 102 kg/cm^2

100 kP_a = 10,200 kg/m^2
= 1.02 kg/cm^2
= **1 bar**
= 1000 mb

1 kg/cm^2 = 98.07 kP_a
= 10 m of water

100 k.P_a = 1.02 kg/cm^2
= 10.2 m of water

1 lb/ft^2 = 47.88 k.P_a
= 16.02 ft of water ;

1 psi = 1 lb/in^2
= 0.0703 kg/cm^2

1 lb/ft^2 = 4.882 kg/m^2 ;

100 k.P_a = 2.089 lb/ft^2
= 33.45 ft of water

1 kg/cm^2 = 14.22 psi

1 kg/m^2 = 0.205 lb/ft^2.

Density (ρ)

1 kg/m^3 = 0.0624 lb/cft ; 1 lb/cft = 16.02 kg/m^3

Unit wt. (γ or $w = \rho.g.$)

Units : N/m^3, kg/m^3 (*i.e.* kg (wt)/m^3) ; lb (wt)/ft^3

1 kg/m^3 (*i.e.* kg (wt)/m^3) = 9.807 N/m^3

1 N/m^3 = 0.102 kg/m^3

Energy

Units : Joule (N.m) ; kg (wt).m ; lb (wt).ft

1 Joule (j) = 1 N.m.

1 kg (wt).m = 9.807 N.m

i.e. 1 kgm = 9.807 J ; 1 J = 0.102 kgm

1 ft lb = 0.138 kgm = 1.356 J

1 kgm = 7.23 ft.lb = 9.807 J

1 J = 0.102 kgm = 0.737 ft lb.

Heat Energy

Joule, Calorie, British Thermal Unit (B.T.U.)

1 kilo Joule = 1.0551 B.T.U.

4.12 kilo Joule = 1 kilo Calorie

∴ **1 kJ = 0.2427 kC = 1.0551 B.T.U.**
1 kC = 4.12 kJ = 4.347 B.T.U.

Power

Units : J/s ; kgm/sec ; ft lb/sec

1 J/s = 1 watt.

1 J/s = 1 watt = 0.102 kgm/sec = 0.737 ft lb/sec

1 HP (fps) = 550 ft lbs/sec = 746 J/s = 746 watts.

1 HP (metric) = 735 kgm/sec = 735 J/s = 735 watts.

Dynamic Viscosity (μ or η)

Units : N.s/m^2 ; kg (wt) sec/m^3 ; dyne sec/cm^2 (Poise)

1 N.s/m^2 = 1 Pa.s = 10 Poise
= 0.102 kg (wt) sec/m^2

1 dyne.sec/cm^2 (Poise) = 0.0102 kg (wt) sec/m^2

Kinematic Viscosity (ν)

Units : m^2/s ; m^2/sec ; cm^2/sec (Stoke)

1 m^2/s = 10^4 cm^2/sec = 10^4 **Stoke** (*S*)

1 centi stoke (CS) = $\frac{1}{100}$ S

Radioactivity

1 pico curie (pCi) = 10^{-12} Curie (Ci)

= 0.037 dps (disintegration per second)

1 Bacqueral (Bq) = 1 dps

$$= \frac{1}{0.037} \text{ pCi} = 27 \text{ pCi.}$$

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Year 1997 (*Engineering Services*)

Q. 3. (c) *Results of chlorine demand test on a raw water are tabulated below :*

Sample No.	*Chlorine dosage mg/l*	*Residual chlorine after 10 min contact mg/l*
1	0.2	0.19
2	0.4	0.36
3	0.6	0.50
4	0.8	0.48
5	1.0	0.20
6	1.2	0.40
7	1.4	0.60
8	1.6	0.80

By plotting the data, determine 'break point dosage'. What is the 'chlorine demand' at dosage 1.2 mg/l ? (10)

[Note. Please refer solved Example 9.29.
This question was set from this book.]

Q. 7. (c) *Water has to be supplied to a town with one lakh population at the rate of 150 litres/capita/day from a river 2000 m away. The difference in elevation between the lowest water level in the sump and reservoir is 36 m. If the demand has to be supplied in 8 hours, determine the size of the main and the brake horse power of the pumps required. Assume maximum demand as 1.5 times the average demand. Assume f = 0.03, velocity in the pipe 2.4 m/s and efficiency of pump 80 percent.* (15)

Hint. Refer similar Solved Example 6.4, and compute as under :

Max. daily demand to be met in 8 hrs of pumping

$$= 1.5\ [10^5 \times 150\ \text{L}]$$

$$= \frac{225 \times 10^5}{10^3}\ \text{m}^3 = 2.25 \times 10^4\ \text{m}^3$$

$\therefore$ Max. discharge required

$$= \frac{2.25 \times 10^4}{8 \times 60 \times 60}\ \text{m}^3/\text{sec} = 0.781\ \text{m}^3/\text{s}.$$

Area of pipe $A = \frac{\pi}{4} d^2 = \frac{Q}{V} = \frac{0.781}{2.4}$ ($\because$ $V = 2.4$ m/s given)

or $$d = \sqrt{\frac{0.781}{2.4} \times \frac{4}{\pi}}$$

$$= 0.644\ \text{m}\ ;\ \textbf{say 0.65 m.}\quad \textbf{Ans.}$$

Head loss in the main

$$= \frac{fLV^2}{2gd}$$

$$= \frac{0.03 \times 2000 \times (2.4)^2}{2 \times 9.81 \times 0.65} = 27.09\ \text{m}.$$

Required lift head between sump and reservoir

$$= 36\ \text{m}$$

Total head against which pumps have to work

$$= 27.09 + 36 = \textbf{63.09 m.}$$

$$\text{H.P.} = \frac{\gamma_w \cdot QH}{0.735 \cdot \eta}$$

$$= \frac{9.81 \times 0.781 \times 63.09}{0.735 \times 0.8} = \textbf{822.}\quad \textbf{Ans.}$$

Q. 8. (c) *Two reservoirs with 15 m difference in their water levels are connected by a 300 mm diameter pipe line of 3000 m length. Calculate the discharge. If a parallel pipe line of 300 mm diameter is attached to the last 1500 m length of existing pipe, determine the modified discharge Take only wall friction into account. Assume f = 0.04 in Darcy-Weisbach formula.* (15)

Hint. Please refer Solved Example 6.6, and Compute as follows :

$$H_L = \frac{f' LV^2}{2gd} = \frac{f' L \left(\dfrac{Q}{(\pi/4)\, d^2}\right)^2}{2gd}$$

or $$H_L = \frac{8}{g\pi^2} \frac{f' LQ^2}{d^5}\ ,\quad \text{where}\quad H_L = 15\ \text{m}.$$

In the 1st case, let Q_1 be the discharge flowing through the pipe of 0.3 m dia and of 3000 m length, having $f' = 0.04$. Then

$$15 = \frac{8 \times 0.04 \times 3000 \times Q_1^{\,2}}{g\pi^2\, (0.3)^5}$$

or $$Q_1 = 0.06063\ \text{m}^3/\text{s}$$

$$= \textbf{60.6} \times \textbf{10}^{-3}\ \textbf{m}^3/\textbf{s}.\quad \textbf{Ans.}$$

In the 2nd case, when additional pipe of 1500 m length is laid in the second half in parallel to the original pipe, let the discharge through the initial length of pipe be Q_2. The discharge through the original 1500 m length will thus be Q_2, and that through the two parallel pipes shall be $\frac{Q_2}{2}$ each. The total head loss will then be

$$15 = \frac{8}{g\pi^2} \cdot \frac{0.04}{(0.3)^5} \left[1500\, Q_2^{\,2} + 1500 \left(\frac{Q_2}{2}\right)^2\right]$$

or $$15 = \frac{8 \times 0.04 \times 1500}{9.81 \times \pi^2 (0.3)^5} \times 1.25\, Q_2^{\,2}$$

or $$Q_2 = 0.0767\ \text{m}^3/\text{s}$$

Modified discharge = $\textbf{76.7} \times \textbf{10}^{-3}\ \textbf{m}^3/\textbf{s}.$ **Ans.**

or $$\frac{Q_1^2}{32} = \frac{(Q - Q_1)^2}{1} = h_1$$

or $$\frac{Q_1}{\sqrt{32}} = (Q - Q_1)$$

or $$Q_1 = 5.66\,(Q - Q_1)$$

or $$Q_1 = 0.85\,Q$$

$$\therefore \quad h_1 = \left(\frac{4}{\pi}\right)^2 \cdot \frac{f \,.\, L\,(0.85\,Q)^2}{2g \,.\, D_1^5} \qquad ...(i)$$

when pipes are connected in series, then

$$\left(\frac{4}{\pi}\right)^2 \frac{fLQ^2}{2gD_1^5} + \left(\frac{4}{\pi}\right)^2 \cdot \frac{fLQ^2}{2g(D_1/2)^5} = h_2$$

or $$\left(\frac{4}{\pi}\right)^2 \frac{fLQ^2}{2gD_1^5}\left[1 + 32\right] = h_2$$

or $$h_2 = 33 \,.\, \left(\frac{4}{\pi}\right)^2 \frac{fLQ^2}{2gD_1^5} \qquad ...(ii)$$

$$\therefore \quad \frac{h_1}{h_2} = \frac{\text{Eq. } (i)}{\text{Eq. } (ii)} = \frac{(0.85)^2}{33} = \mathbf{0.0219.}\ \textbf{Ans.}\ ]$$

Q. 5. (*a*) *How are various impurities of water classified ? Give examples for each type.* (11)

(*c*) *Write short notes on :*

(*i*) *Characteristics of algae* (7)

(*ii*) *Break point chlorination* (7)

Year 1986 (*Engineering Services*)

Q. 5. (*b*) *Write short notes on :*

(*iii*) *Chemical methods of removing suspended impurities from very large quantities of water.* (8)

Year 1987 (*Engineering Services*)

Q. 1. (*a*) *A pipe line of 0.6 m diameter is 1.5 km long. To augment the discharge, another pipe line of the same diameter is introduced in parallel to the first in the second half of its length. Find the increase in discharge if f = 0.04 and head at the inlet is 30 m.* (10)

[**Note.** *Refer solved example 6.6*]

(*b*) *Define soft water. Write down the names of the 4 methods of water softening.* (10)

Q. 7. (*b*) *In a town, it has been decided to provide 200 litre per head per day in the 21st century.*

Estimate the domestic water requirements of this town in the year AD 2000 by projecting the population of the town by incremental increase method.

Year	*Population*
1940	*2,37,98,624*
1950	*4,69,78,325*
1960	*5,47,86,437*
1970	*6,34,67,823*
1980	*6,90,77,421*

(20)

[**Note.** *Refer solved example 2.9*]

Year 1988 (*Engineering Services*)

Q. 2. (*c*) *Water is pumped from a low level reservoir to a high level reservoir through a main pipeline of 0.45 m diameter and 1400 m length. The pump is located at the low level reservoir. At a point along the main line at a distance of 450 m from the high level reservoir, a branch line of 0.3 m dia and 360 m length takes off to discharge 180 l/sec in the atmosphere.*

Level of water surface in high level reservoir	*= + 30.00 m*
Level of water in the open end of the 0.3 m dia branch line	*= 25.50 m*
Level of water in the low level reservoir	*= 18.00 m*
Darcy's frictional coeff. for both pipes	*= 0.032*

Determine the flow rate into the high level reservoir and the theoretical H.P. of the pump, assuming the delivery valve of the pump to be at + 20.00 m. (20)

[**Note.** *Refer solved example 7.6*]

Year 1989 (*Engineering Services*)

Q. 7. (*a*) *What are the essential requirements in performance of a water meter in domestic supplies ?* (12)

(*b*) *What are the functional and design features that help to increase the overall efficiency of a centrifugal pump ?* (12)

Q. 8. (*a*) *Draw out a typical set of specifications for placement of galvanised mild steel pipes in a building, including supply thereof.* (15)

Year 1990 (*Engineering Services*)

Q. 4. (*b*) *For supplying water to a colony, it is intended to store water in a large tank, whose water surface elevation is presently 85.37 m RL. This tank is being supplied by a pipe line from a junction J, the friction loss in metres of water in this pipe being $0.6\,Q^{1.95}$, where Q is the discharge in m^3/sec. Junction J receives water under pressure and without leakage from (i) a large reservoir I whose water surface elevation is constant at 100.00 m RL through a pipe characterised by $h_f = 0.35\,Q^2$; and (ii) a second large reservoir II with its constant water level at 96 m RL, through a pipe characterised by $h_f = 0.46\,Q^{1.85}$. Neglecting losses other than due to pipe friction, determine the incoming flow into the tank in the colony.* (15)

[**Note.** *Refer solved example 6.7*]

Q. 5. (*i*) *Derive the expression for the pressure rise due to water hammer caused by a sudden closure of the discharge through the pipe if : (1) the pipe elasticity is considered ; and (2) the pipe elasticity is fully neglected.* (15)

[**Note.** *Refer solved example 6.11*]

Table for hydraulic values for C = 100

d (mm)	h_L (m/1000)	Q(l/s)	V(m/s)
300	17.8	133	1.7
250	14.1	117	1.5
200	20.0	48	1.6
150	30.0	30	1.7
100	40.0	13	1.5

(30)

[**Note.** *Refer solved example 10.9*]

Q. 2. *(a) A filter unit is 4.5 m by 9.0 m. After filtering 10000 cubic metre per day in 24 hour period, the filter is backwashed at a rate of 10 l/sq. m/sec for 15 min. Compute the average filtration rate, quantity, and percentage of treated water used in washing, and the rate of wash water flow in each trough. Assume 4 troughs.* (25)

[**Note.** *Refer solved example 9.22*]

(b) Chlorine usage in the treatment of 20,000 cubic metre per day is 8.0 kg/ day. The residual after 10 min contact is 0.20 mg/l. Calculate the dosage in milligram per litre and chlorine demand of water. (15)

[**Note.** *Refer solved example 9.28*]

Year 1981 (*Engineering Services*)

Q. 4. *An overhead tank is to be provided for a city water supply. Given the following data, calculate the minimum capacity of the tank without any fire demand storage. Assume the tank to be empty between 12 to 15 hours. The draw off factors, water pumped and water consumed, etc. are shown in Table below:*

Table 17.1. Table for Q. 4 of year 1981

No.	Time (hr.)	Hours	Draw off factor	Water pumped	Water consumed	Accumulation (+) draw-off (–)	Water in service reservoir (SR)
	(1)	(2)	(3)	(4)	(5)	(6)	(7)
A	0—4	4	0.2	Zero	13,333		
B	4—6	2	1.2	50,000	40,000		
C	6—10	4	1.8	1,00,000	1,20,000		
D	10—12	2	1.0	50,000	33,333		
E	12—15	3	0.5	Zero	25,000		
F	15—16	1	0.5	25,000	8,333		
G	16—20	4	1.6	1,00,000	1,06,666		
H	20—23	3	1.0	75,000	50,000		
I	23—24	1	0.2	Zero	3,333		

Note. Only columns 6 and 7 may be reproduced in the answer-book to save time.

Compute columns 6 and 7 of the table to calculate the storage capacity of the tank. Given :

(i) Average water supply = 16,666 litres/hr.
(ii) Rate of pumping = 25,000 litres/hr.
(iii) Hours of pumping = 4—12 and 15—23. (40)

[**Note.** *Refer solved example 10.6*]

Year 1982 (*Engineering Services*)

Q. 6. *(a) A town with a population of 1 lakh is to be supplied with water daily at 200 litres per head. The variation of demand is as follows :*

6 am to 9 am = 40% of total
9 am to 12 noon = 10% of total
12 pm to 3 pm = 10% of total
3 pm to 6 pm = 15% of total
6 pm to 9 pm = 25% of total.

Determine the capacity of the service reservoir assuming pumping to be at uniform rate and the period of pumping to be from 6 am to 6 pm. Neglect fire demand. Solve by the mass curve method. (30)

[**Note.** *Refer solved example 10.7*]

Year 1983 (*Engineering Services*)

Q. 1. *(a) A fire engine supplies water to a hose pipe, 75 m long and 0.075 m in diameter, at a pressure of 3 kg/cm² (gauge). The discharge end of the hose pipe has a nozzle of diameter d fixed to it. Taking friction factor f as 0.032, determine the dia d of the nozzle, so that the momentum of the issuing jet may be maximum.* (20)

Solution. Let the head at the inlet of the hose pipe = H.

Head loss in the hose pipe = H_L.

$\therefore$ Head left at nozzle for conversion into vel. head $= H - H_L$.

$\therefore$ $\dfrac{v^2}{2g} = H - H_L$ (where v is the vel. of jet)

or $v^2 = 2g\,[H - H_L]$

Now, the momentum of the jet is $\rho.a.v(v)$, where ρav is the mass jetting out per second.

The momentum (ρav^2) will not only be maximum when ρav^2 is maximum, but it will be maximum when kinetic energy of the jet *i.e.*, ρav^3 is also maximum (v for the jet being more than 1 m/sec).

But $\rho av^3 = \rho a.v\left[2g(H - H_L)\right]$

For ρav^3 to be maximum, $\dfrac{\partial}{\partial v}(\rho av^3) = 0$.

$\therefore$ $\dfrac{\partial}{\partial v}\left[\rho.a.v.(2g)(H - H_L)\right] = 0.$

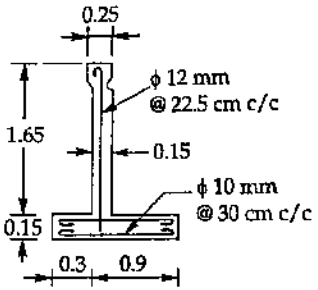


Fig. 14.2. (*f*) Section C - C

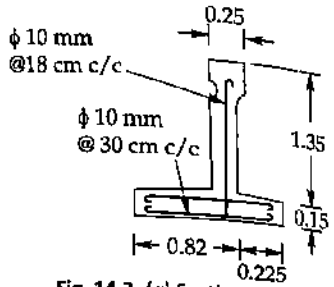


Fig. 14.2. (*g*) Section D - D

Fig. 14.2. Structural details of a 21 m × 9 m Swimming Pool.

The **pool hall** should also have a pleasant environment with an efficient ventilation system, so as to

(*i*) maintain flow of fresh air without draughts ;

(*ii*) control humidity and prevent condensation of water vapour ; and

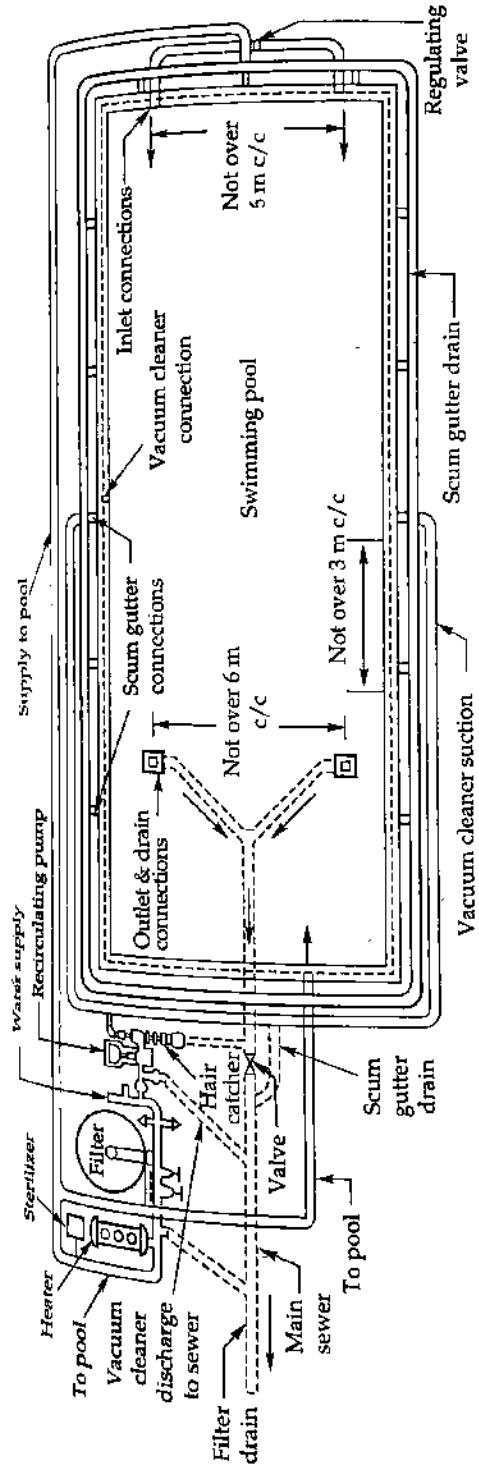
(*iii*) remove chlorine effects (such as the pungent nitrogen trichloride gas) eminating from the badly functioning chlorinated pool water. If not removed efficiently, it may cause coughing and irritation of eyes and nose.

14.3. Systems of Supply in Swimming Pools and Recycling of Pool Water

Many of the old swimming pools are still found to be of **fill and empty type**. Depending upon the number of users, the quality of water will deteriorate very fast in such pools, resulting in infection hazards. Chlorination helps but the concentration is likely to rise in static water, and the pool water has to be replaced in such tanks quite often and usually within 18—48 hours. Such pools can only be considered for construction, when the number of users are few, and a large quantity of fresh water is constantly available. The used and polluted water from such a pool will also have to be disposed of efficiently. Such pools will therefore need heavy quantity of fresh water supply almost every day or at the most every alternate day. Such pools may however be constructed in farm houses, where the waste water may be used for irrigation purposes.

Pools in general are however provided with a **flow through system**, in which fresh water is added at a constant rate, thereby daily replacing the used water at 10—15% of the pool volume. When constructed in a farm, the waste water in such a case in again normally used for irrigation in the farm land, as in the first case. But disposal of such huge quantities of water is, however, not feasible, in case the pools are constructed in non-agricultural areas, such as in hotels, schools, etc. Moreover, supply of fresh water for replacing the old one is again very costly and difficult.

Recycling of the pool water is, therefore, widely practised in swimming pools, constructed in cities, hotels, and institutional areas. Reuse of water is accomplished by providing a **filtration system**, comprising of a *pressure filter* (or a conventional one) *chlorinator and chemical dosing, and a pumping system* to recirculate the entire pool water over a period of 4 to 16 hours (generally not over 8 hours). Water is sometimes *aerated* to eliminate dissolved



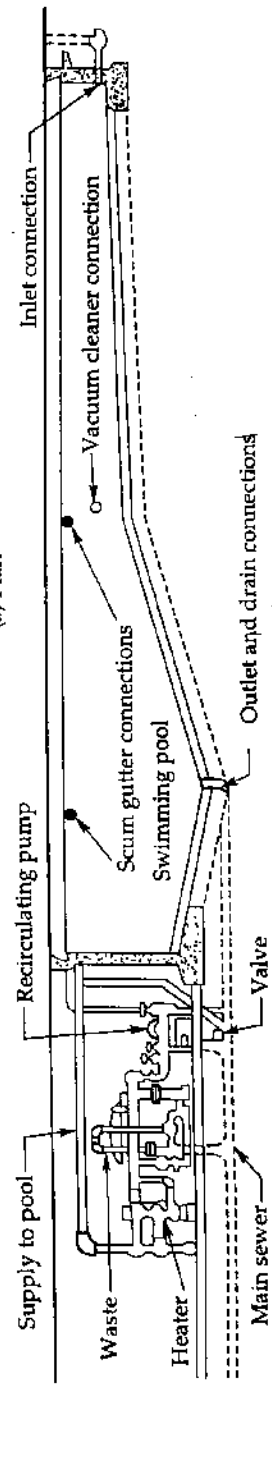


Fig. 14.3. Typical plumbing layout for a Recycling type of swimming pool (Fig. Continues on next page).

or $$P_B = \frac{200}{100} \times 10.1937 \text{ m (of water)}$$

$$= 20.387 \text{ m ; say } \mathbf{20.39\ m}.$$

Also $$Z_A = 0$$

$$Z_B = 9.1$$

∴ Total head (*i/c* datum head) at A

$$= Z_A + P_A$$

$$= 0 + 56.07$$

$$= \mathbf{56.07\ m} \quad ...(i)$$

Similarly the total head (*i/c* datum head) at B

$$= Z_B + P_A$$

$$= 9.1 + 20.39$$

$$= \mathbf{29.49\ m} \quad ...(ii)$$

Similarly the total head (*i/c* datum head) at C

$$= Z_C + P_C$$

$$= 12.2 + 30$$

$$= \mathbf{42.2\ m} \quad ...(iii)$$

With these total heads at A, B and C being equal to 56.07 m, 29.49 m, and 42.2 m, respectively, we can easily draw the H.G. line, as shown in Fig. 10.21 (*b*).

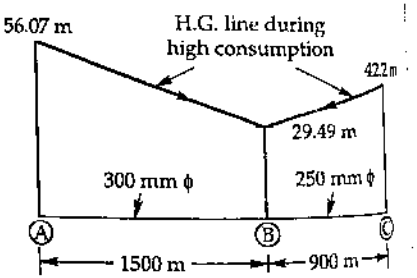


Fig. 10.21 (*b*) H.G. line for example 10.9.

Now, for pipe AB of 300 mm ϕ, we find

S = gradient of energy line

$$= \frac{56.07 - 29.49}{1500} \times 1000 \text{ m/1000 m length}$$

$$= 17.72 \text{ m/1000 m ; say } 17.8 \text{ m/1000 m}.$$

The given table of hydraulic values of the pipe shows that for h_L of 17.8 m/1000 m, and 300 mm ϕ pipe, the discharge is 133 l/s. Hence, the discharge at B due to AB *i.e.* due to pumps

$$= \mathbf{133\ l/s.}\quad \textbf{Ans.} \quad ...(1)$$

Similarly, the gradient of energy line for pipe CB of 250 mm ϕ

$$= \frac{42.2 - 29.47}{900} \times 1000 \text{ m/1000 m length}$$

$$= 14.1 \text{ m/1000 m}.$$

From the given table, for a pipe of 250 mm and h_L of 14.1 m/1000 m, the discharge is 117 l/s.

Hence, the discharge at B caused by the storage reservoir

$$= \mathbf{117\ l/sec.}\quad \textbf{Ans.} \quad ...(2)$$

Hence, the flow available at point B from pumps is 133 l/sec, and from the elevated storage is 117 l/sec, respectively, giving a total flow of 250 l/s. **Ans.**

WASTAGE OF WATER IN THE DISTRIBUTION SYSTEM

Considerable amount of water gets lost and wasted in the distribution system, either through leakage or through unauthorised water connections. So much so, that even in the best managed distribution system, these losses may amount to about 15% of the total consumption ; while in a faulty distribution system, they may be as high as 40% of the total consumption. Obviously, this wastage of water will increase the per unit cost of the filtered water. Hence, for the proper management of the distribution system, it is necessary to detect the places where water is being wasted, and to take immediate steps in order to stop that wastage.

The various *factors which are responsible for the wastage of the water and the steps required to be taken to control such wastages* have already been discussed in chapter 2 ; and the readers are advised to refer back. However, the methods which are adopted for detecting the leakage of water through the pipe joints and from other junctions, are given below :

10.13. Detection of Leakage in the Underground Distribution Pipes

The following methods may be used for detecting the leakage of water from the underground water mains :

(*i*) *by direct observations* ;

(*ii*) *by using sounding rods* ;

(*iii*) *by plotting hydraulic gradient line* ; and

(*iv*) *by using waste detecting meters.*

These methods are discussed below :

(*i*) **By direct observations.** This method consists in practically observing a wet soft spot on the unpaved ground, or a luxuriously grown grass in a lawn, or the emergence of a spring at an odd place. All such hints will indicate the presence of the underground water pipe leaking somewhere in the vicinity of the area. Such indications will be generally available at places, where the pipes are laid below the loamy or the clayey soils. However, in sandy soils, the leaks are difficult to be traced by direct observation. Nevertheless, even at the places where such preliminary indications are available, the exact location of the leak will have to be confirmed by using sounding rods, etc.

(*ii*) **By using sounding rods.** In this method, a sharp pointed metal rod is thrust into the ground along the pipe line and pulled up for inspection. Its moist or muddy point will preliminarily indicate the presence of leakage. The sound of the escaping water can also be heard by placing the ear on the top of the inserted rod. This will confirm the leakage point. Such notices produced by leaks can also be magnified by a stethoscope type instrument called an **aquaphone** or a **sonoscope** ; so as to facilitate better hearing of sounds. The sound hearing job should preferably be done during the night hours, when the general atmosphere is calm.

(*iii*) **By plotting the hydraulic gradient line.** In this method, the pressures at various points along a suspected pipe line are measured, and the hydraulic gradient line is plotted. The appearance of any *kink* or change in the slope of the hydraulic gradient line will indicate the location of a leak in the pipe line.

9.19. Hydraulics of Sand Gravity Filters

9.19.1 Computing Initial Loss of Head in a Clean Filter Bed. We have explained in article 9.17.2 that loss of head is caused in the filter when water moves through the filter bed, since the downward movement of water under gravitational force is opposed by the resistance offered by the sand grains and the impurities arrested in them. In a newly commissioned filter (or after every backwash), this **initial head loss** is caused only by the sand grains, as no impurities are present. This initial loss of head in a clean stratified filter primarily depends upon the **porosity (n)** of the filter material, besides depending to different degrees on other factors, such as filtration velocity (v), depth of filter (D), dia. of the sand grain (d), acceleration due to gravity (g), drag coefficient (C_D), etc. **Rose** has given the following equation for computing this ***initial head loss through clean stratified sand filter :***

$$\text{Initial Head Loss} = h_L = \frac{1.067\, v^2 \,.\, D}{\phi g \,.\, n^4} \sum \frac{C_D \,.\, (f)}{d} \quad ...(9.34)$$

where h_L = frictional head loss through the filter in m
v = approach velocity or filtration velocity in m/s
D = depth of filter in m
ϕ = shape factor (for non-spherical sand particles)
d = diameter of sand particles in m
g = acceleration due to gravity in m/s^2
n = porosity
C_D = Newton's drag coefficient
f = mass fraction of sand particles of dia. d

The value of Drag Coefficient (C_D) can be computed from equations (9.4) to (9.6) depending upon the value of Reynold number (R_e). While calculating the value of R_e by using Eq. (9.3), the value of d (dia. of sand grain) is multiplied by ϕ (*i.e.* shape factor). Thus, d becomes equal to $\phi\, d$ for non-spherical particles.

The Rt. hand side summation term $\sum \frac{C_D \,.\, f}{d}$ in Eq. (9.34) can be calculated using the size distribution of the sand particles found from sieve analysis. However, for uniform sand, having representative size d, this term equals to $\frac{C_D}{d}$.

The Rose equation then becomes :

$$h_L = \left[\frac{1.067\, v^2 \,.\, D}{\phi g n^4} \,.\, \frac{C_D}{d}\right] \text{ for uniform sand of representative particle size } d \quad ...(9.34a)$$

Although, the Rose equation is limited to clean filter beds, it does provide an opportunity to examine the initial stage of filtration and the effects of sand grain size distribution on head loss. As the filter clogs, the head loss will increase, so that the calculated head loss is the minimum expected head loss.

Initial head loss in excess of 0.6 m imply that either the loading rate is too high or that the sand has too large proportion of fine grain sizes. The design of filter must account for the additional losses that will occur as the filter runs. *Thus, the filter box must be atleast as deep as the highest design head loss. This value is about 3 m, maximum.*

9.19.2. Hydraulic Head Loss and Expansion of the Filter Bed during Backwash. Backwashing of granular media filters, as discussed earlier, is accomplished by reversing the flow and forcing clean water upward through the sand media. *To clean the interior of the filter bed, it is necessary to expand it, so that the granules are no longer in contact with each other, thus exposing all the surfaces for cleaning.* To hydraulically expand a porous bed, the head loss h_{Le} (or uplift) must be atleast equal to the buoyant weight of the filter bed. For a unit area of the filter, this is expressed as :

$$h_{Le} \,.\, \gamma_w = D \,.\, \gamma_{sub} \quad ...(9.35)$$

where h_{Le} = head loss through the filter bed required to initiate expansion, in m
γ_w = unit wt. of water in kN/m^3
D = depth of filter bed in m
γ_{sub} = submerged unit wt. of sand in bed of depth D in kN/m^3

But $\gamma_{sub} = \frac{\gamma_w (G-1)}{1+e}$ ...(Eq. derived in Soil Mechanics)

where e = void ratio
G = sp. gravity of sand grains

But porosity $n = \frac{e}{1+e}$ ...(Eq. derived in Soil Mechanics)

or $(1-n) = 1 - \frac{e}{1+e} = \frac{1}{1+e}$

$\therefore \quad \gamma_{sub} = \gamma_w \,.\, (G-1)\,(1-n)$

Eq. (9.35) hence becomes

$$h_{Le} \,.\, \gamma_w = D \,.\, \gamma_w (G-1)(1-n)$$

or $$\boxed{h_{Le} = D.(1-n)\,(G-1)} \quad ...(9.36)$$

Even when the bed gets expanded to depth D_e, the head loss through the expanded bed essentially remains unchanged, because the total buoyant weight of the bed is constant.

But wt. of fluidized/expanded bed = $D_e(1-n_e)\,(G-1)$

$\therefore \quad h_{Le} = D_e \,.\, (1-n_e)(G-1)$ ...(9.37)

where D_e = the depth of expanded/ fluidized bed in m
n_e = the porosity of the expanded/ fluidized bed.

Equating Eq. (9.36) and (9.37), we get ...(9.38)

$\therefore \quad D(1-n)(G-1) = D_e(1-n_e(G-1)$

sedimentation tank. In such a tank, a plain floc-chamber without any mechanical devices is provided before the water enters the sedimentation chamber. *The detention period for the floc-chamber is kept about 15 to 40 minutes, and that for the settling tank, at about 2 to 4 hours. The depth in the floc chamber may be kept about half that of in the settling chamber.*

The water from the mixing basin enters this tank, and the clarified water comes out of the outlet end. The design principles for such a tank are the same as those applied to a plain sedimentation tank except that these are kept deeper. A depth varying from 3 to 6 m is generally provided. They may be cleaned at intervals of about 6 months or so.

Example 9.17. *Design a coagulation-cum-sedimentation tank with continuous flow for a population of 60,000 persons with a daily per capita water allowance of 120 litres. Make suitable assumptions where needed.*

Solution. First of all, we will design the settling tank, and then the floc-chamber.

Design of the Settling Tank

Average daily consumption

= Population × Per capita demand

= 60,000 × 120 = 7.2×10^6 litres

Assuming the max. daily demand as 1.8 times the average daily demand (Eq. 2.7), we get

The max. daily consumption

= $1.8 \times (7.2 \times 10^6)$ litres = 12.96×10^6 litres

∴ Quantity of water to be treated during an assumed detention period of 4 hours (*i.e.* between 2 to 4 hrs, as indicated in article 9.9.4)

$$= \frac{12.96 \times 10^6}{24} \times 4 = 2.16 \times 10^6 \text{ litres} = 2.16 \times 10^3 \text{ cu-m.}$$

Hence, the capacity of the tank reqd.

= 2.16×10^3 cu-m.

Assuming an overflow rate* of 1000 litres/hr/m^2 of plan area (*i.e.* between 1000 to 1250 l/hr/m^2 as given in article 9.10) we get

$$\frac{Q}{B.L} = 1000$$

where $Q = \frac{2.16 \times 10^6}{4}$ litres/hr = 540×10^3 litres/hr.

∴ Plan area = $B.L$.

$$= \frac{Q}{1000} = \frac{540 \times 10^3}{1000} = 540 \text{ m}^2.$$

Using the width of the tank (B) as 12 m, we get

the length (L) of the tank = $\frac{540}{12}$ = **45 m.**

**Alternatively*, adopt the water depth as 4 m and get the plan area as

$$= \frac{\text{Capacity}}{\text{Depth}} = \frac{2.16 \times 10^3}{4} = 540 \text{ m}^2$$

Hence, use a tank of 45 m × 12 m × 4 m. Provide extra depth for sludge storage, say use 4.5 m depth at the starting end, and $4.5 + \frac{45}{50} = 5.4$ m at the d/s end (using 1 in 50 slope). Use a freeboard of 0.5 m above the water level, as shown in Fig. 9.27.

Design of the Floc Chamber

In addition to 45 m length of the settling tank, the floc chamber at the entry has to be provided. Assume the effective depth in the floc chamber as half of the depth in the tank near the floc chamber, (Pl see article 9.10)

i.e. $= \frac{4.5}{2} = 2.25$ m.

Assume the period of flocculation or detention period as 20 minutes. (*i.e.* between 15 to 40 min, as given in article 9.10)

Now, the capacity of the chamber

= Flow reqd. in 20 minutes

$$= \frac{12.96 \times 10^3}{24} \times \frac{20}{60} \text{ cu. m} = 180 \text{ cu. m.}$$

The plan area reqd. $= \frac{\text{Capacity}}{\text{Depth}} = \frac{180}{2.25}$ m^2 = **80 m^2.**

Using the same width as 12 m, we get the length of flocculation chamber

$$= \frac{80}{12} = 6.67 \text{ m (say 6.7 m)}$$

The details and dimensions are shown in Fig. 9.27.

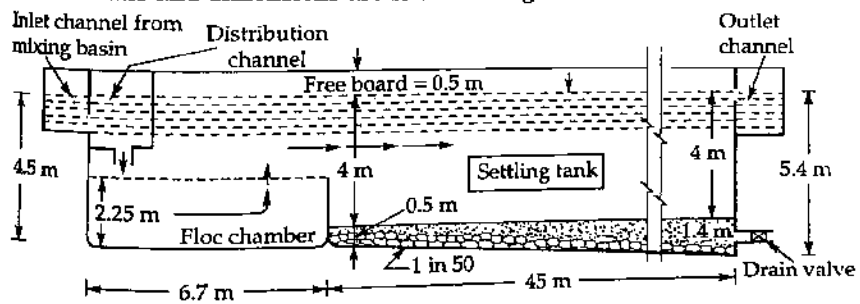


Fig. 9.27. Dimensions of the Coagulation-cum-Sedimentation tank in example 9.17.

9.11. Laboratory Testing for Determining Optimum Coagulant Quantities

It was pointed out earlier that the best coagulant dose is determined first in the laboratory and then adjusted by actual observations at the treatment plant. The common test which is performed to determine this approximate optimum quantity of coagulant is known as ***jar test.***

The apparatus which is used in this test is shown in Fig. 9.28. The sample of raw water to be tested is placed in a number of jars each having a capacity of about 1 litre. Normally, six jars are used. Different amounts of coagulant are then added to each jar. The driving unit is started. The paddles placed inside the jars, and connected with the driving shaft through vertical stirring rods,

	trans-1, 2-Dichloro-ethylene	0	0.005
	1, 2-Dichloropropane	0	0.005
	Di[2-ethyl]adipate	0.4	0.4
	Diethylhexyl phthlate	0	0.006
	Dinoseb	0.007	0.007
	Diquat	0.02	0.02
	Endothall	0.1	0.1
	Endrin	0.002	0.002
	Epichlorohydrin	0	TT[a]
	Ethylenedibromide (EDB)	0	0.00005
	Ethylbenzene	0.7	0.7
	Glyphosate	0.7	0.7
	Heptachlor	0	0.0004
	Heptachlor epoxide	0	0.0002
	Hexachlorobenzene	0	0.001
	Hexachlorocyclo-pentadiene	0.05	0.05
	γ-HCH (lindane)	0.0002	0.0002
	Indeno[1, 2, 3-c, *d*] pyrene	0[c]	0.0004[c]
	Methoxychlor	0.04	0.04
	Monochlorobenzene	0.1	0.1
	Oxamyl	0.2	0.2
	Pentachlorophenol	0	0.001
	Polychlorinated biphenyls	0	0.0005
	Simazine	0.004	0.004
	Styrene	0.1	0.1
	2, 3, 7, 8-TCDD (dioxin)	0	3×10^{-8}
	2, 4, 5-TP [2(2, 4, 5-tri-chlorophenoxy) propionic acid]	0.05	0.05
	Tetrachloroethylene	—	0.005
	Toluene	1	1
	Toxaphene	0	0.003
	1, 2, 4-Trichloro-benzene	0.07	0.07
	1, 1, 1-Trichloroethane	0.2	0.2

Contd...

	1, 1, 2-Trichloroethane	0.003	0.005
	Trichloroethylene	0	0.005
	Trihalomethanes	—	0.08[e]
	Vinyl chloride	0	0.002
	Xylene	10	10
(3)	**Aesthetic Parameters**		
	(*i*) Aluminium	—	0.05 to 0.2
	(*ii*) Chloride	—	250
	(*iii*) Colour	—	15 Units
	(*iv*) Copper	—	TT[a]
	(*v*) Corrosivity	—	Non corrosive
	(*vi*) Foaming agents	—	0.5
	(*vii*) Iron	—	0.3
	(*viii*) Manganese	—	0.05
	(*ix*) Taste and odour	—	3 TON (Threshold Odour No.)
	(*x*) pH	—	6.5—8.5
	(*xi*) Silver	—	0.1
	(*xii*) Solids total dissolved	—	500
	(*xiii*) Sulphate	—	250
	(*xiv*) Turbidity (NTU)	—	0.5—1.0
	(*xv*) Zinc	—	5
(4)	**Radiological Parameters**		
	(*i*) Radium-226	0	20 pCi/L[f]
	(*ii*) Radium-228	0	20 pCi/L[f]
	(*iii*) Radon	0	300 pCi/L[f]
	(*iv*) Uranium	0	20 µg/L
	(*v*) Gross alpha activity	0	15 pCi/L[f]
	(*vi*) Gross beta activity	0	4 mili rem

[a]Treatment technology specified.
[b]Draft
[c]Proposed
[d]The total THM cannot exceed 0.08 mg/L
[e]Aesthetic objective
[f]1 pCi = 2.22 Bq (Bacquerel)

The *free ammonia* or *ammonia nitrogen* indicates the very first stage of decomposition of organic matter and consequent release of free ammonia (thus indicating recent pollution) ; *albuminoid or organic nitrogen* indicates the quantity of nitrogen present in water in the form of undecomposed organic matter (mainly in the form of proteins and amino acids) which on digestion will release ammonia ; the *nitrites* indicate the presence of partly decomposed (not fully oxidised) organic matter ; and *nitrates* indicate the presence of fully oxidised organic matter in water (thus representing old pollution).

The presence of ***free ammonia*** in water indicates the start of decomposition of organic matter present in water; and for potable waters, its value should not exceed 0.15 mg/l. It can be easily measured by simply boiling the water and measuring the liberated ammonia gas, by *distillation process*.

The presence of ***organic nitrogen*** in water also indicates pollution and presence of undecomposed organic matter. For potable waters, it should not exceed a value of about 0.3 mg/l. It can be determined by digestion of the organic matter and measuring the released ammnonia gas. This can be easily done by adding strong alkaline solution of potassium permanganate ($KMnO_4$) to already boiled* water sample and again boiling the same, when ammonia gas is liberated, which is measured, so as to indicate the amount of organic nitrogen present in water. The sum total of ammonia nitrogen and the organic nitrogen is called the **Kjedahl nitrogen.**

The partly oxidised organic matter present in water in the form of ***nitrites*** *is highly dangerous and, therefore, the permissible amount of nitrites present in potable water should be nil.* The ***nitrates***, on the other hand, represent the fully oxidised organic matter, and such waters may not be harmful. However, the presence of too much of nitrate in water may adversely affect the health of infants, causing a disease technically called ***"methemoglobinemia"*** (commonly called **"blue baby disease"**). Children suffering from this disease may vomit, their skin colour may become dark, and may die in extreme cases. The nitrate concentration in domestic water supplies is generally limited to 45 mg/l. The amount of nitrites or nitrates, present in a water sample, can be measured by colour matching methods. For nitrites, the colour is developed by adding sulphonic acid and napthamine ; whereas for nitrates, the colour is developed by adding phenol-di-sulphonic acid and potassium hydroxide. The colour developed in water is finally compared with the standard colours of known concentrations.

Excess presence of nitrogen of any kind indicates pollution, and the form in which it is present indicates the period of this pollution. In olden days, when bacteriological tests of water were not properly developed, the nitrogen tests were of considerable importance, as they were the only methods for detecting pollution caused by sewage or organic waste, and thus indicating the presence or absence of bacteria and other such pollutants. But in modern times, these test are taken only as indicative or as inference tests, and the bacteriological tests are taken as the real confirmatory tests in this respect. Besides serving as inference tests, they also reveal the history of water pollution, if has taken place at any time.

(*vi*) **Metals and other Chemical substances.** Tests are carried out in order to determine the amounts of various metals and other substances, such as iron, manganese, copper, lead, barium, cadmium, arsenic, selenium, fluorine, etc., present in water. The amounts of these minerals or metals in potable waters are then limited to their permissible standard values (Refer Tables 8.8 to 8.10), so as to avoid their harmful effects on human life.

The type of harmful effects that are caused by various elements and compounds when consumed in excess of their permissible values are summarised in Table 8.2 and also indicated in Table 8.10.

*thus removing free ammonia.

Table 8.2. Summary of Harmful Effects Caused by Various Toxic Inorganic Elements and Compounds present in water, when consumed in excess of their specified values

S. No.	*Inorganic contaminant*	*Max. permissible concentration in potable waters as per WHO guidelines in mg/L*	*Max. permissible concentration as per BIS standards (IS 10500 : 1991) in mg/L*	*Harmful health effects*
(1)	(2)	(3)	(4)	(5)
1.	*Antimony, Sb*	0.005	Not specified (N.S.)	Antimony (Sb) may accumulate in liver and is detrimental to the heart in humans. Can also be accumulated by marine organisms.
2.	**Arsenic, As**	0.01	0.01	May cause cancer, diabetes, and liver damage. Also increases risk of hypertension. Causes acute or chronic arsenic toxicity (called ***arsenicosis***) to humans. Toxic to all life. Contributed as by-product of smelting ores and used in other industries. Ground waters are often found to contain arsenic in specific areas such as in West Bengal-Bangladesh belt.
3.	*Barium, Ba*	0.70	N.S.	Ingested Ba salts are highly toxic to humans. Usually found in trace amounts in natural waters but surface water concentrations are sometimes as high as 0.34 mg/L. May be toxic to plants if present above trace amounts.
4.	*Beryllium, Be*	N.S.	N.S.	Extremely toxic to all life. Usually, naturally present in concentrations less than 0.0001 gm/L in surface waters. Oxides and hydro-xides are insoluble within normal pH ranges.
5.	*Boron, B*	0.3	1.0	No evidence of accumulation in humans. Large amounts may produce digestive difficulties and nerve disorders.
6.	**Cadmium, Cd**	0.003	0.01	Cumulative, highly toxic in humans and livestock. Affects all life. Protects other metals against oxidation ; also used in other industries.

Contd...

Water Supply Engineering

This book provides the requisite details of planning and designing an efficient and economical water supply system in urban as well as rural areas under different site conditions. The simple and lucid language adopted by the author has made this book highly popular amongst the students and teachers of engineering colleges in India and abroad.

The book is thoroughly revised and updated covering complete *Hydraulic Design of a water Treatment Plant, Laboratory Experiments in Water Supply Engineering* alongwith latest topics of interest like *Roof Top Rain Water Harvesting; Design of Swimming Pools; and Packaged Drinking Waters; etc.* The text matter is well supported by over *500 Objective Questions* and a vast number of numerical examples, chosen from various universities and competitive examinations. This book will be of immense help to the students of the engineering colleges and for the preparation of various competitive examinations, particularly the Engineering Services, Civil Services (IAS), and GATE examinations.

The Author, **Santosh Kumar Garg,** retired as Superintending Engineer from the Irrigation & Flood Control Deptt. of the Govt. of Delhi, obtained his basic degree in civil engineering from the University of Delhi in 1969, securing a **First Rank at the university** with **Record breaking percentage of marks**, getting ***Certificates of merit in seven subjects.*** He since remained closely associated with the planning, design and maintenance of several water resources and drainage projects of the Delhi Govt., including the Preparation of the Master plan of Drainage.

Sh. Garg, in fact, is a veteran in the field of Civil and Environmental engineering, and has eight widely acclaimed books to his credit. His books on *Water Resources Engineering (Vol. I & II),* and *Environmental Engineering (Vol. I & II),* are being followed in almost all the engineering colleges in India, including the prestigious IITs and NITs, and also in certain Asian and African countries. His three other publications titled, *Soil Mechanics & Foundation Engineering, Physical and Engineering Geology,* and *Ecology & Environmental Studies,* have also gained a lot of popularity with the students and teachers alike. His publication on *River Water Disputes* is also of interest to the academicians, field engineers, and serious students desirous of getting knowledge.

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