

16th Edition

IRRIGATION AND WATER POWER ENGINEERING



**Dr. B. C. PUNMIA
Dr. PANDE B. B. LAL
ASHOK KUMAR JAIN
ARUN KUMAR JAIN**

IRRIGATION AND WATER POWER ENGINEERING

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By

Dr. B.C. PUNMIA

Formerly,

Professor and Head, Deptt. of Civil Engineering, &
Dean, Faculty of Engineering
M.B.M. Engineering College,
Jodhpur

Dr. PANDE BRIJ BASI LAL

Formerly,

Principal, M.M.M. Engg. College
Gorakhpur

Er. ASHOK KUMAR JAIN

Director,
Arihant Consultants,
Jodhpur

Dr. ARUN KUMAR JAIN

Assistant Professor
M.B.M. Engineering College,
Jodhpur

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IRRIGATION AND WATER POWER ENGINEERING

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Preface

India has unequal distribution of rainfall over the country—varying from less than 25 cm in the North West deserts of Rajasthan to about 1150 cm in the Assam hills. Substantial variations in the quantity, incidence and duration of rainfall in individual tracts from year to year, make irrigation of supreme necessity in the country. The science of Irrigation Engineering in India has developed by leaps and bounds during the last six decades. The Irrigation Engineering occupy prominent place in the World.

The purpose of this book is to present the science of the practice of irrigation engineering in a concise form comprising practically all the modern developments. The book, written entirely in Metric Units covers the syllabi of Indian Universities and A.M.I.E. Examination in 'Irrigation and Water Power.' The various original formulae in F.P.S. units have also been given side by side. Most of the diagrams and the charts have been given in metric units.

The practices described are mainly those prevailing or coming into use in India, although many references are made to the work of engineers in other countries. The approach to the subject has been practical throughout. Indications are given to the most efficient way in which the work could be undertaken, without neglecting the basic principles involved.

The book has been primarily written for the students to use it as a text book at the first degree level. Some advanced topics included in this book will be useful at the M.E. level. The book will also be useful to the practising engineers, as a reference. A number of solved examples and detailed designs of major irrigation works have been included.

The authors gratefully acknowledge the help received from many standard text books and publications on irrigation engineering. The authors are thankful to Sri Kanhaiya Lal who has very nicely traced the diagrams. The authors are also thankful for the co-operation received and efficiency with which the printing was carried out.

Every effort was made to eliminate errors in the book, but should the reader discover some, we would appreciate having these brought to our attention. Suggestions from the readers for the improvement in the book will be most gratefully acknowledged.

Gandhi Jayanti

2-10-1969

B.C. PUNMIA

PANDE B.B. LAL

Preface to the Second Edition

The revised and enlarged edition of the book incorporates the latest Indian Standards on the design of lined canals and on the criteria for design of hydraulic jump type stilling basins with horizontal and sloping aprons.

The authors are thankful to Shri R.S. Varshney, S.C. Gupta and R.L. Gupta for permitting us to use the Garret's and Lacey's diagrams in metric units from their book "Theory and Design of Irrigation Structures."

(iii)

(iv)

Account has been taken throughout of suggestions offered by the many users of the book and grateful acknowledgement is made to them. Further suggestions will be greatly appreciated.

Gandhi Jayanti
2-10-1972

B.C. PUNMIA
PANDE B.B. LAL

Preface to the Third Edition

In the Third Edition of the book the following three chapters have been completely rewritten : Chapter 7. Dams-1 : General ; Chapter 8. Dams-2 : Gravity Dams, and Chapter 11. Dams-5 : Spillways. In addition to these, Unsolved Problems, with Answers, have been added at the end of each chapter to enable the student to test his reading at different stages of his reading. Suggestions from the readers for the improvements in the book will be most gratefully acknowledged.

1-7-1975

B.C. PUNMIA

Preface to the Fifth Edition

In this edition, Chapter 3 has been rewritten and many new articles have been added. An Appendix on 'Khoala's Theory' has been added at the end. The subject matter of the entire book has been updated. The authors are thankful to many readers of the book for their useful suggestions. Further suggestions will be greatly appreciated.

26th Jan., 1979

B.C. PUNMIA

Preface to the Sixth Edition

In the Sixth Edition, the subject matter has been thoroughly revised and updated. Latest Indian Standard Recommendations and Codes have been incorporated in the text.

1-1-1981

B.C. PUNMIA

Preface to the Seventh Edition

In the Seventh Edition, few articles have been added in the chapter on Hydrology. Some solved illustrative examples have also been added on the computations for S-curve and unit hydrographs of different durations.

2-10-1982

B.C. PUNMIA

(v)

Preface to the Eleventh Edition

In the Eleventh Edition of the book, the Subject matter has been revised and updated. Many new articles have been added in various chapters.

Jodhpur
DEEPAWALI
29-10-1989

B.C. PUNMIA

Preface to the Twelfth Edition

In the Twelfth Edition, the book has been thoroughly revised and enlarged. The book has now been divided into Four Sections. Section 1, containing three chapters is on *Irrigation Practice*. Section 2 devoted to *Water Resources Engineering* has nine chapters. Section 3 is on *Distribution System*, and has seven chapters. Lastly, Section 4 has five chapters on *Miscellaneous Topics*. Chapter 24, on *Important Dams of India*, contains eight photographic plates on important dams. Thus, in the Twelfth Edition of the book, which has 24 chapters, three new chapters have been added.

The entire matter of the book has been composed with DTP process and laser print out. It is hoped that with these changes, the New Edition will be more useful of both the students as well as Field Engineers.

Jodhpur
15-5-92

B.C. PUNMIA
PANDE B.B. LAL

Preface to the Sixteenth Edition

In the Sixteenth Edition, the entire book has been completely rewritten by way of revising, updating, rearranging and enlarging the earlier contents. In each chapter, many new articles and illustrative examples have been added. Questions from prominent competitive examinations, such as Indian Civil Services Examination, Engineering Services Examinations, Gate etc. have been added at the end of majority of the chapters. Latest IS Code recommendations have been included in each chapter. All the diagrams have been drawn using computer graphics and the entire book has been laser typeset using bigger size format. The authors are thankful to Shri M.S. Gehlot for fine laser type setting. The Authors are also thankful to Shri R.K. Gupta, Managing Director, Laxmi Publications, for taking keen interest in the publication of the book.

Jodhpur
Ashay Tritya,
27th April, 2009

B.C. PUNMIA
ASHOK KUMAR JAIN
ARUN KUMAR JAIN

Contents

SECTION I : IRRIGATION PRACTICE

CHAPTER 1	INTRODUCTION	
1.1.	DEFINITION	3
1.2.	NECESSITY	3
1.3.	SCOPE OF IRRIGATION SCIENCE	4
1.4.	MULTIPURPOSE RIVER VALLEY PROJECT	6
1.5.	BENEFITS OF IRRIGATION	6
1.6.	ILL-EFFECTS OF IRRIGATION	8
1.7.	TYPES OF IRRIGATION	8
1.8.	HISTORY OF IRRIGATION DEVELOPMENT IN INDIA	12
CHAPTER 2	METHODS OF IRRIGATION	
2.1.	MODES OR METHODS OF APPLYING WATER TO CROPS	15
2.2.	UNCONTROLLED OR WILD FLOODING	16
2.3.	FREE FLOODING OR FLOODING FROM FIELD CHANNELS	16
2.4.	FLOODING BY CONTOUR LATERALS	17
2.5.	BORDER STRIP FLOODING METHOD	17
2.6.	CHECK FLOODING	23
2.7.	RING BASIN FLOODING	26
2.8.	THE ZIGZAG METHOD	27
2.9.	FURROW METHOD	27
2.10.	CONTOUR FARMING	33
2.11.	SUB-SURFACE IRRIGATION	34
2.12.	SPRINKLER IRRIGATION	35
2.13.	DRIP IRRIGATION	37
2.14.	EXAMPLES FROM COMPETITIVE EXAMINATIONS	40
CHAPTER 3	WATER REQUIREMENTS OF CROPS	
3.1.	FUNCTIONS OF IRRIGATION WATER	41
3.2.	QUALITY OF IRRIGATION WATER	41
3.3.	TYPES OF SOILS	45
3.4.	PREPARATION OF LAND FOR IRRIGATION	48
3.5.	CLASSES AND AVAILABILITY OF SOIL WATER	49
3.6.	LIMITING SOIL MOISTURE CONDITIONS	54
3.7.	DEPTH AND FREQUENCY OF IRRIGATION	54
3.8.	PRINCIPAL CROPS AND CROP SEASONS	56
3.9.	DUTY AND DELTA	58
3.10.	FACTORS AFFECTING DUTY	61
3.11.	METHODS OF IMPROVING DUTY	63
3.12.	SOME DEFINITIONS	63
3.13.	CONSUMPTIVE USE OF WATER (EVAPO-TRANSPIRATION)	70
3.14.	IRRIGATION EFFICIENCIES	79
3.15.	SOIL FERTILITY	84

(vii)

(viii)

3.16	CROP ROTATION	85
3.17	ASSESSMENT OF IRRIGATION WATER	86
3.18	EXAMPLES FROM COMPETITIVE EXAMINATIONS	88

SECTION - II : WATER RESOURCES ENGINEERING

CHAPTER 4 HYDROLOGY

4.1	HISTORY OF HYDROLOGY	99
4.2	THE HYDROLOGIC CYCLE	100
4.3	METEOROLOGICAL DATA	101
4.4	HYDROLOGICAL DATA	104
4.5	THE WATER BUDGET	104
4.6	PRECIPITATION	105
4.7	MEASUREMENT OF RAIN FALL	108
4.8	RAINGAUGE NETWORK	112
4.9	ESTIMATION OF MISSING RAINFALL DATA	114
4.10	COMPUTATION OF AVERAGE RAINFALL OVER A BASIN	116
4.11	PRESENTATION OF RAINFALL DATA	121
4.12	INTERPRETATION OF RAINFALL DATA	122
4.13	PROBABLE MAXIMUM PRECIPITATION (PMP)	129
4.14	LOSSES OR ABSTRACTIONS FROM PRECIPITATION	139
4.15	EVAPORATION	140
4.16	INFILTRATION	145
4.17	RUN-OFF	161
4.18	FACTORS AFFECTING RUN-OFF	164
4.19	COMPUTATION OF RUN-OFF	166
4.20	HYDROGRAPH ANALYSIS	168
4.21	COMPUTATION OF DIRECT RUN-OFF OR RAIN FALL EXCESS FROM STORM HYDROGRAPH.	173
4.22	UNIT HYDROGRAPH (UH)	174
4.23	S-HYDROGRAPH (SUMMATION HYDROGRAPH)	178
4.24	CONSTRUCTION OF UNIT HYDROGRAPH OF DIFFERENT UNIT DURATION FROM A UNIT HYDROGRAPH OF SOME GIVEN UNIT DURATION	181
4.25	DERIVATION OF UNIT HYDROGRAPH FROM COMPLEX STORMS	183
4.26	DISTRIBUTION GRAPH	191
4.27	INSTANTANEOUS UNIT HYDROGRAPH	192
4.28	SYNTHETIC UNIT HYDROGRAPH	195
4.29	PEAK FLOW DETERMINATION	197
4.30	EMPIRICAL FORMULAE FOR FLOOD DISCHARGE	198
4.31	FLOOD FREQUENCY STUDIES	199
4.32	FLOOD DISCHARGE BY RATIONAL FORMULA	207
4.33	METHOD OF ENVELOPING CURVES	209
4.34	EXAMPLES FROM COMPETITIVE EXAMINATIONS	216

(ix)

CHAPTER 5 GROUND WATER : WELL IRRIGATION

5.1	INTRODUCTION	239
5.2	SOME DEFINITIONS	239
5.3	DIVISIONS OF SUB-SURFACE WATER	241
5.4	TYPES OF AQUIFERS	242
5.5	STORAGE COEFFICIENT	244
5.6	WELL HYDRAULICS	245
5.7	DETERMINATION OF AQUIFER CONSTANT T	250
5.8	CHARACTERISTIC WELL LOSSES : SPECIFIC CAPACITY OF WELL	252
5.9	INTERFERENCE AMONG WELLS	253
5.10	FULLY PENETRATING ARTESIAN GRAVITY WELL	254
5.11	PARTIALLY PENETRATING ARTESIAN WELL	254
5.12	SPHERICAL FLOW IN A WELL	255
5.13	TUBE WELLS	256
5.14	METHODS FOR DRILLING TUBE WELLS	262
5.15	WELL SHROUDING AND WELL DEVELOPMENT	266
5.16	OPEN WELL	268
5.17	YIELD OF AN OPEN WELL	270
5.18	METHODS OF LIFTING WATER	273
5.19	ADVANTAGES AND DISADVANTAGES OF WELL IRRIGATION OVER CANAL IRRIGATION	275
5.20	SELECTION OF SUITABLE SITE FOR A TUBE WELL	276
5.21	OTHER SOURCES OF UNDER GROUND WATER	287
5.22	UNSTEADY FLOW TOWARDS WELLS*	290
5.23	EXAMPLES FROM COMPETITIVE EXAMINATIONS	299

CHAPTER 6 RESERVOIR PLANNING

6.1	INTRODUCTION	306
6.2	INVESTIGATIONS FOR RESERVOIR PLANNING	307
6.3	SELECTION OF SITE FOR A RESERVOIR	310
6.4	ZONES OF STORAGE IN A RESERVOIR	310
6.5	STORAGE CAPACITY AND YIELD	311
6.6	CALCULATION OF RESERVOIR CAPACITY FOR A SPECIFIED YIELD, FROM THE MASS INFLOW CURVE	313
6.7	DETERMINATION OF SAFE YIELD FROM A RESERVOIR OF A GIVEN CAPACITY	314
6.8	SEDIMENT FLOW IN STREAMS : RESERVOIR SEDIMENTATION	314
6.9	RESERVOIR SEDIMENT CONTROL	317
6.10	SINGLE PURPOSE FLOOD CONTROL RESERVOIRS	318
6.11	MULTIPURPOSE RESERVOIRS	319
6.12	APPORTIONMENT OF TOTAL COST OF A MULTIPURPOSE RESERVOIR	321
6.13	FLOOD ROUTING	322
6.14	EXAMPLES FROM COMPETITIVE EXAMINATIONS	341

(x)

CHAPTER 7	DAMS-I : GENERAL	
7.1.	INTRODUCTION	346
7.2.	CLASSIFICATION ACCORDING TO USE	346
7.3.	CLASSIFICATION ACCORDING TO HYDRAULIC DESIGN	347
7.4.	CLASSIFICATION ACCORDING TO MATERIAL	348
7.5.	GRAVITY DAMS	348
7.6.	ARCH DAMS	349
7.7.	BUTTRESS DAMS	350
7.8.	STEEL DAMS	352
7.9.	TIMBER DAMS	353
7.10.	EARTH DAMS AND ROCKFILL DAMS	354
7.11.	PHYSICAL FACTORS GOVERNING SELECTION OF TYPE OF A DAM	356
7.12.	SELECTION OF SITE FOR A DAM	357
CHAPTER 8	DAMS-II : GRAVITY DAMS	
8.1.	INTRODUCTION	359
8.2.	FORCES ACTING ON A GRAVITY DAM	359
8.3.	WATER PRESSURE	359
8.4.	WEIGHT OF THE DAM	360
8.5.	UPLIFT PRESSURE	360
8.6.	PRESSURE DUE TO EARTHQUAKE	362
8.7.	ICE PRESSURE	373
8.8.	WAVE PRESSURE	373
8.9.	SILT PRESSURE	374
8.10.	WIND PRESSURE	375
8.11.	COMBINATION OF LOADING FOR DESIGN	375
8.12.	MODES OF FAILURE : STABILITY REQUIREMENTS	383
8.13.	PRINCIPAL AND SHEAR STRESSES	386
8.14.	QUALITY AND STRENGTH OF CONCRETE/MASONRY (IS : 6512-1984)	388
8.15.	STABILITY ANALYSIS	390
8.16.	ELEMENTARY PROFILE OF A GRAVITY DAM	393
8.17.	PRACTICAL PROFILE OF A GRAVITY DAM	396
8.18.	LIMITING HEIGHT OF A GRAVITY DAM : HIGH AND LOW GRAVITY DAMS	398
8.19.	PROFILE OF HIGH MASONRY GRAVITY DAM	398
8.20.	DESIGN OF GRAVITY DAMS	399
8.21.	GALLERIES	405
8.22.	JOINTS, KEYS AND WATER SEALS	405
8.23.	CONTROL OF CRACKING IN CONCRETE DAMS	408
8.24.	EXAMPLES FROM COMPETITIVE EXAMINATIONS	430
CHAPTER 9	DAMS-III : ARCH AND BUTTRESS DAMS	
9.1.	TYPES OF ARCH DAMS	434
9.2.	FORCES ON AN ARCH DAM	436
9.3.	DESIGN METHODS FOR ARCH DAMS	438
9.4.	THE ELASTIC THEORY	438

(xi)

9.5.	THE TRIAL LOAD ANALYSIS	440
9.6.	BUTTRESS DAMS	440
9.7.	TYPES OF BUTTRESS DAMS	441
CHAPTER 10	DAMS-IV : EARTH AND ROCKFILL DAMS	
10.1.	INTRODUCTION : TYPES OF EARTH DAMS	446
10.2.	CAUSES OF FAILURES OF EARTH DAMS	447
10.3.	CRITERIA FOR SAFE DESIGN OF EARTH DAM	452
10.4.	SECTION OF AN EARTH DAM	453
10.5.	DOWNSTREAM DRAINAGE SYSTEM	458
10.6.	SEEPAGE ANALYSIS	461
10.7.	STABILITY ANALYSIS	473
10.8.	STABILITY OF DOWNSTREAM SLOPE DURING STEADY SEEPAGE	476
10.9.	STABILITY OF UPSTREAM SLOPE DURING SUDDEN DRAWDOWN	479
10.10.	STABILITY OF US AND DS SLOPES DURING CONSTRUCTION	482
10.11.	STABILITY OF FOUNDATION AGAINST SHEAR	484
10.12.	DESIGN TO SUIT AVAILABLE MATERIALS	485
10.13.	SLOPE PROTECTION	488
10.14.	SEEPAGE CONTROL MEASURES	489
10.15.	SLOPE STABILITY ANALYSIS UNDER EARTH QUAKE EFFECTS	496
10.16.	DESIGN CONSIDERATIONS IN EARTHQUAKE REGIONS	499
10.17.	COMPACTION OF ROLLED FILL DAM	500
10.18.	ROCKFILL DAMS	501
CHAPTER 11	DAMS-V : SPILLWAYS	
11.1.	INTRODUCTION	511
11.2.	STRAIGHT DROP SPILLWAY	512
11.3.	OGEE OR OVERFLOW SPILLWAY	513
11.4.	SIDE CHANNEL SPILLWAY	527
11.5.	CHUTE OR TROUGH SPILLWAY	528
11.6.	CONDUIT OR TUNNEL SPILLWAY	528
11.7.	SHAFT SPILLWAY	529
11.8.	SIPHON SPILLWAYS	530
11.9.	DYNAMIC FORCE ON SPILLWAY	538
11.10.	ENERGY DISSIPATION BELOW SPILLWAYS	543
11.11.	INDIAN STANDARD ON CRITERIA FOR DESIGN OF HYDRAULIC JUMP TYPE STILLING BASINS WITH HORIZONTAL AND SLOPING APRONS	549
11.12.	SPILLWAY CREST GATES	556
11.13.	OUTLET WORKS	560
11.14.	EXAMPLES FROM COMPETITIVE EXAMINATIONS	562
CHAPTER 12	DIVERSION HEADWORKS	
12.1.	INTRODUCTION	566
12.2.	COMPONENT PARTS OF A DIVERSION HEADWORK	567
12.3.	THE WEIR	568
12.4.	LOCATION OF HEADWORKS	569
12.5.	EFFECTS OF CONSTRUCTION OF A WEIR ON THE REGIME OF RIVER	571

(xii)

12.6.	CAUSES OF FAILURE OF WEIRS AND THEIR REMEDIES	571
12.7.	DESIGN OF IMPERVIOUS FLOOR FOR SUB-SURFACE FLOW	573
12.8.	KHOSLA'S THEORY	577
12.9.	ELEMENTS OF DESIGN FOR SURFACE FLOW	592
12.10.	DESIGN OF VERTICAL DROP WEIR	599
12.11.	DESIGN OF SLOPING GLACIS WEIR	611
12.12.	DIVIDE WALL	624
12.13.	THE FISH LADDER	625
12.14.	THE UNDER SLUICES	626
12.15.	THE CANAL HEAD REGULATOR	627
12.16.	SILT CONTROL AT HEADWORKS	629
12.17.	EXAMPLES FROM COMPETITIVE EXAMINATIONS	631

SECTION - III : DISTRIBUTION SYSTEM

CHAPTER 13 FLOW IRRIGATION

13.1.	CANALS : CLASSIFICATION	643
13.2.	CANAL ALIGNMENT	645
13.3.	CURVES	647
13.4.	INUNDATION CANALS	647
13.5.	BANDHARA IRRIGATION	650

CHAPTER 14 IRRIGATION CHANNELS - I : SILT THEORIES

14.1.	TYPE OF CHANNELS	653
14.2.	DESIGN OF ALLUVIAL CHANNELS	653
14.3.	KENNEDY'S THEORY	654
14.4.	KENNEDY'S METHOD OF CHANNEL DESIGN	656
14.5.	SILT SUPPORTING CAPACITY ACCORDING TO KENNEDY'S THEORY	661
14.6.	DRAWBACKS IN KENNEDY'S THEORY	664
14.7.	FURTHER WORK ON KENNEDY'S THEORY	669
14.8.	LACEY'S REGIME THEORY	677
14.9.	LACEY'S THEORY APPLIED TO CHANNEL DESIGN	678
14.10.	COMPARISON OF KENNEDY'S AND LACEY'S THEORIES	679
14.11.	DEFECTS IN LACEY'S THEORY	679
14.12.	TRACTIVE FORCE APPROACH FOR CHANNEL DESIGN	680
14.13.	DESIGN OF UNLINED CHANNELS IN ALLUVIAL SOIL AS PER IS : 7112-1973	683
14.14.	SEDIMENT TRANSPORT	694
14.15.	DESIGN OF NON-ALLUVIAL CHANNELS	698
14.16.	EXAMPLES FROM COMPETITIVE EXAMINATIONS	698

CHAPTER 15 IRRIGATION CHANNELS - II : DESIGN PROCEDURE

15.1.	LONGITUDINAL SECTION OF CANAL	710
15.2.	BALANCING DEPTH	712
15.3.	LOSSES IN CANALS	713
15.4.	SCHEDULE OF AREA STATISTICS AND CHANNEL DIMENSIONS	714

(xiii)

15.5.	USE OF GARRET'S DIAGRAMS FOR CHANNEL DESIGN ON KENNEDY'S THEORY	715
15.6.	USE OF LACEY'S REGIME DIAGRAMS	718
15.7.	CROSS-SECTION OF AN IRRIGATION CHANNEL	718
15.8.	BORROW PIT	728
15.9.	SPOIL BANK	728
15.10.	PERMANENT LAND WIDTH	728
15.11.	BACK BERM, COUNTER BERM	729
15.12.	MAINTENANCE OF IRRIGATION CHANNELS	729
15.13.	MAINTENANCE OF SERVICE ROADS	731
15.14.	CANAL BREACHES	732
15.15.	REGULATION OF CANAL SYSTEM	732
15.16.	MANAGEMENT OF IRRIGATION WATER	733
15.17.	MEASUREMENT OF DISCHARGE OF A CANAL	734
15.18.	AREA VELOCITY METHOD.	734
15.19.	CHEMICAL METHOD	737
15.20.	WEIR METHOD	738
15.21.	METER FLUME	738
15.22.	STAGE DISCHARGE CURVE	739

CHAPTER 16 WATERLOGGING AND CANAL LINING

16.1.	WATERLOGGING	746
16.2.	EFFECTS OF WATERLOGGING	747
16.3.	CAUSES OF WATERLOGGING	748
16.4.	REMEDIAL MEASURES	749
16.5.	LOSSES IN CANAL	750
16.6.	LAND DRAINAGE	752
16.7.	DESIGN AND MAINTENANCE OF OPEN DRAINS	753
16.8.	UNDER-DRAINS OR TILE DRAINS	754
16.9.	LAYOUT OF A TILE DRAIN SYSTEM	754
16.10.	FLOW OF GROUND WATER TO DRAINS	754
16.11.	LINING OF IRRIGATION CHANNELS	757
16.12.	TYPES OF LINING	758
16.13.	DESIGN OF LINED CANAL	761
16.14.	INDIAN STANDARD ON DESIGN OF LINED CANALS	762
16.15.	ECONOMICS OF CANAL LINING	769
16.16.	DRAINAGE AND PRESSURE RELEASE ARRANGEMENTS BEHIND CANAL LINING	770
16.17.	SALT PROBLEMS IN IRRIGATED SOILS	772
16.18.	EXAMPLES FROM COMPETITIVE EXAMINATIONS	776

CHAPTER 17 CANAL OUTLETS

17.1.	INTRODUCTION	781
17.2.	TYPES OF OUTLETS	781
17.3.	IMPORTANT DEFINITIONS	783
17.4.	NON-MODULAR OUTLETS : PIPE OUTLET	783

(xiv)

17.5.	SEMI-MODULE OR FLEXIBLE OUTLETS	789
17.6.	RIGID MODULE	796
CHAPTER 18	CANAL REGULATION WORKS	
18.1.	INTRODUCTION	799
18.2.	NECESSITY AND LOCATION OF FALLS	799
18.3.	DEVELOPMENT OF FALLS	800
18.4.	CLASSIFICATION OF FALLS	802
18.5.	CISTERN DESIGN	804
18.6.	DESIGN OF SARDA TYPE FALL	808
18.7.	DESIGN OF STRAIGHT GLACIS FALL	818
18.8.	MONTAGU TYPE FALL	825
18.9.	INGLIS TYPE FALL	827
18.10.	OFF-TAKE ALIGNMENT	827
18.11.	HEAD REGULATORS AND CROSS-REGULATORS	828
18.12.	DESIGN OF CROSS-REGULATOR AND DISTRIBUTARY HEAD REGULATOR	829
18.13.	CANAL ESCAPES	839
CHAPTER 19	CROSS DRAINAGE WORKS	
19.1.	INTRODUCTION	843
19.2.	TYPES OF CROSS-DRAINAGE WORKS	843
19.3.	SELECTION OF SUITABLE TYPE OF CROSS-DRAINAGE WORK	846
19.4.	CLASSIFICATION OF AQUEDUCTS AND SYPHON AQUEDUCTS	847
19.5.	FEATURES OF DESIGN OF CROSS-DRAINAGE WORKS	848
19.6.	FIXATION OF WATERWAY OF THE DRAIN	849
19.7.	CLEARANCE AND FREEBOARD : IS CODE RECOMMENDATIONS	850
19.8.	CONTRACTION OF CANAL WATERWAY	852
19.9.	HEAD LOSS THROUGH SYPHON BARRELS	856
19.10.	UPLIFT PRESSURE ON THE ROOF OR TROUGH	857
19.11.	UPLIFT PRESSURE ON THE FLOOR OF THE AQUEDUCT	858
19.12.	DESIGN OF BANK CONNECTIONS	860
19.13.	DESIGN EXAMPLE OF A SYPHON AQUEDUCT	860
	SECTION - IV : MISCELLANEOUS TOPICS	
CHAPTER 20	RIVER ENGINEERING	
20.1.	CLASSIFICATION OF RIVERS	871
20.2.	MEANDERING	871
20.3.	CAUSES OF MEANDERING	872
20.4.	BASIC FACTORS CONTROLLING PROCESS OF MEANDERING	872
20.5.	GENERAL FEATURES OF MEANDERING	872
20.6.	THE AGGRADING TYPE OF RIVER	873
20.7.	DEGRADING TYPE OF RIVER	873
20.8.	CUT-OFF	874
20.9.	RIVER TRAINING	875
20.10.	CLASSIFICATION OF RIVER TRAINING WORKS	875
20.11.	TYPES OF RIVER TRAINING WORKS	876

(xv)

20.12.	GUIDE BANK SYSTEM	876
20.13.	GROYNES OR SPURS	879
20.14.	REPELLING GROUYNE	880
20.15.	DEFLECTING GROYNES	880
20.16.	ATTRACTING GROUYNE	881
20.17.	LENGTH OF GROYNES	881
20.18.	SPACING OF GROYNES	881
20.19.	IMPERMEABLE GROYNES	882
20.20.	PERMEABLE GROYNES	882
20.21.	DENEHY'S GROYNES	883
20.22.	HOCKEY GROUYNE	883
20.23.	MARGINAL BUND OR LEVEES	883
20.24.	ARTIFICIAL CUTOFF	886
20.25.	PITCHED ISLANDS	886
20.26.	BANK PROTECTION	887
20.27.	PITCHED BANK	887
20.28.	BANDALLING	888
20.29.	NANAL PLANTATION	888
CHAPTER 21	WATER POWER ENGINEERING	
21.1.	GENERAL	891
21.2.	TYPES OF WATER POWER DEVELOPMENT	892
21.3.	PRINCIPAL COMPONENTS OF HYDROELECTRIC SCHEME	895
21.4.	FOREBAY	895
21.5.	INTAKE STRUCTURE	896
21.6.	PENSTOCKS	896
21.7.	SURGE TANK	896
21.8.	TURBINES	897
21.9.	SELECTION OF SUITABLE TYPE OF TURBINE	899
21.10.	THE POWER-HOUSE	900
21.11.	SCROLL CASING, DRAFT TUBE AND TAIL RACE	902
21.12.	ASSESSMENT OF WATER POWER POTENTIAL	904
CHAPTER 22	WATER RESOURCES PLANNING	
22.1.	INTRODUCTION	908
22.2.	INDIA'S WATER RESOURCES	908
22.3.	SCENARIO OF WATER USE	909
22.4.	PURPOSES OF WATER RESOURCES DEVELOPMENT	909
22.5.	CLASSIFICATION OF WATER RESOURCES DEVELOPMENT PROJECTS	910
22.6.	FUNCTIONAL REQUIREMENTS IN MULTI-PURPOSE PROJECTS	911
22.7.	PROCESS OF PROJECT FORMULATION	911
22.8.	PROJECT EVALUATION	912
22.9.	STRATEGIES FOR THE FUTURE	913
22.10.	PLANNING STRATEGIES	914
22.11.	MANAGEMENT STRATEGIES	915

(xvi)

CHAPTER 23 IMPORTANT DAMS OF INDIA

23.1. MAJOR RIVER BASINS OF INDIA	918
23.2. BHAKRA DAM	918
23.3. KOYNA DAM	921
23.4. NAGAR JUN SAGAR DAM	922
23.5. GANDHI SAGAR DAM	923
23.6. RIHAND DAM	925
23.7. LOWER BHAVANI DAM	927
23.8. HIRAKUD DAM	928
23.9. KRISHNARAJA SAGAR DAM	930
23.10. UKAI DAM	931
23.11. THEIN DAM	933
23.12. TEHRI DAM	935
23.13. RAMGANGA DAM	936
23.14. MAHI BAJAJ SAGAR DAM	937
23.15. UJJANI DAM	938
23.16. TUNGBHADRA DAM	940
23.17. DANTIWADA DAM	942
APPENDIX - I	944
INDEX	961

SECTION-I**IRRIGATION PRACTICE****Chapters :**

1. INTRODUCTION
2. METHODS OF IRRIGATION
3. WATER REQUIREMENTS OF CROPS

Apprise Education, Reprise Innovations

Introduction

1.1. DEFINITION

Irrigation may be defined as the process of artificially supplying water to soil for raising crops. It is a science of planning and designing an efficient, low-cost, economic irrigation system tailored to fit natural conditions. It is the engineering of controlling and harnessing the various natural sources of water, by the construction of dams and reservoirs, canals and headworks and finally distributing the water to the agricultural fields. Irrigation engineering includes the study and design of works in connection with river control, drainage of water-logged areas, and generation of hydroelectric power.

1.2. NECESSITY

India is basically an agricultural country, and all its resources depend on the agricultural output. Water is evidently the most vital element in the plant life. Water is normally supplied to the plants by nature through rains. However, the total rainfall in a particular area may be either insufficient, or ill-timed. In order to get the maximum yield, it is essential to supply the optimum quantity of water, and to maintain correct timing of water. This is possible only through a systematic irrigation system—by collecting water during the periods of excess rainfall and releasing it to the crop as and when it is needed. Thus, the necessity of irrigation can be summarised in the following points:

1. Less Rainfall

When the total rainfall is less than needed for the crop, artificial supply is necessary. In such a case, irrigation works may be constructed at a place where more water is available, and then to convey the water to the area where there is deficiency of water. Rajasthan canal is one such example. It conveys water to the arid zones of Rajasthan, where the annual rainfall hardly exceeds 100 to 200 mm.

2. Non-uniform Rainfall

The rainfall in a particular area may not be uniform over the crop period. During the early periods of the crop, rains may be there, but no rain water may be available at the end, with the result that either the yield may be less, or the crop may die altogether. By the collection of water during the excess-rainfall period, water may be supplied to the crop during the period when there may be no rainfall. Most of the irrigation projects in India are based on this premise. The rainfall during the winter

is very scanty, and hence rabi crops need artificial supply of water through the irrigation works.

3. Growing a number of crops during a year

The rainfall in an area may be sufficient to raise only one type of crops during the rainy season (i.e. kharif crops), for which no irrigation may be required. However, with the provision of irrigation facilities in that area, crops can be raised in other season also (i.e. rabi crops).

4. Growing perennial crops

Perennial crops, such as sugar cane etc, which need water throughout the year, can be raised only through the provision of irrigation facilities in the area.

5. Commercial Crops with Additional Water

The rainfall in a particular area may be sufficient to raise the-usual crops, but more water may be necessary for raising-commercial and cash crops.

6. Controlled Water Supply

By the construction of proper distribution system, the yield of the crop may be increased because of controlled supply of water.

The importance of irrigation is well stated by Shri N.D. Gulati : "Irrigation in many countries is an old art—as old as civilisation—but for the whole World it is modern science—the science of survival".

Advantage of application of water by modern methods.

Application of water to the soil by modern methods of irrigation serves the following purposes :

1. It adds water to the soil to supply the moisture essential for the plant growth.
2. It saves the crops from drying during short duration droughts.
3. It cools the soil and the atmosphere, and thus makes more favourable environment for healthy plant growth.
4. It washes out or dilutes salts in the soil.
5. It reduces the hazard of soil piping.
6. It softens the tillage pans.

1.3. SCOPE OF IRRIGATION SCIENCE

The scope of irrigation is not limited to the application of water to the soil. It deals with all aspects and problems extending from the watershed to the agricultural farms. It deals with the design and construction of all works, such as dams, weirs, head regulators etc. in connection with the storage or diversion of water, as well as the problems of subsoil drainage, soil reclamation and water-soil-crop relationships. An irrigation engineer is also required to have the knowledge of cultivation of various crops, their maturing and protection from pests. Briefly speaking, the scope of irrigation can be divided into two heads :

- (a) Engineering aspect.
- (b) Agricultural aspect.

(a) Engineering Aspect

1. Storage, Diversion, or Lifting of Water

This is the first phase of irrigation engineering. By the construction of a dam across the river, a suitable reservoir can be created and water can be stored. Alternatively, if river is perennial and carries sufficient discharge, suitable diversion works, such as a weir, barrage and bandhara can be constructed across the river and water can be diverted to the canal. Places where ground water table is high, suitable wells can be dug and water can be lifted and fed to small channels or pipes.

2. Conveyance of Water to the Agricultural Fields

The stored or diverted water is conveyed to the agricultural fields through some suitable distribution system. If the project is big, the distribution system will consist of two or more main canals and a number of distributaries, and minors. From the canal, the water is led to the field through 'water course' or 'field channel'. These field channels take off from the distributaries or minors through modular or non-modular outlets, so that water supply can be regulated. Thus, this second phase includes the design and construction of suitable canal system, along with various regulatory works such as head regulators, cross regulators, falls etc. for the efficient working of the canal. In addition to these, suitable cross drainage works, such as aqueducts, super passages, level crossings, bridges, etc. will have to be designed and constructed at the places where the canal crosses a natural drain or a road.

3. Application of Water to Agricultural Fields

The water is applied to the crop by flooding, furrows, corrugations, subsoil irrigation or by sprinkling. A detailed account of these methods has been given in the next Chapter.

4. Drainage and Relieving Water-Logging

Proper disposition of excess water by suitable drainage methods is very important. Due to the conveyance losses, and excess application of water to the crop, the sub-soil water level is raised, and sometimes the soil is water-logged and becomes useless from production point of view. Hence the design of surface and sub-surface drainage system is of vital importance in maintaining the high productivity of irrigated lands.

5. Development of Water Power

Most of the irrigation projects are associated with the generation of the hydroelectric power either at the dam site or at canal falls.

(b) Agricultural Aspect

The agricultural aspect deals with the thorough study of the following points :

1. Proper depths of water necessary in single application of water for various crops.
2. Distribution of water uniformly and periodically.
3. Capacities of different soils for irrigation water, and the flow of water in soils.
4. Reclamation of waste and alkaline lands, where this can be carried out through the agency of water.

1.4. MULTIPURPOSE RIVER VALLEY PROJECT

A *multipurpose river valley project* includes the following *aspects* :

1. Irrigation
2. Water supply for public health and sanitation
3. Generation of hydro-electric power
4. Flood control and river training
5. Erosion and sediment control : Soil conservation
6. Inland navigation
7. Fish culture.
8. Recreation
9. Drainage

A *multipurpose river valley scheme* embodies the study of the following *sciences*:

1. Agronomy and animal husbandry
2. Soil science and mechanics
3. Engineering : Civil, mechanical, electrical and agricultural
4. Hydrography and hydrology
5. Hydraulics
6. Geology
7. Sociology
8. Economics
9. Infrastructure
10. Administration.

1.5. BENEFITS OF IRRIGATION

All the irrigation schemes are so designed that they increase the food production of the country. Apart from the increase in food, there are many indirect benefits or advantages of irrigation, as summarised below :

1. **Increase in Food Production.** Due to the controlled and timely supply of water to the crop, the yield is increased. With the assured supply of water, superior crops take the place of inferior crops, and thus the value of crops is increased. For a given crop, the optimum quantity of water required may be determined experimentally, and then applied scientifically to the crop.

2. **Protection from Famine.** The construction of irrigation works helps during the famine and drought period in two ways. During the construction of the irrigation works, employment is carried to the people and thus they get relief against famine. After the construction of such works, continuous water supply is maintained during drought.

3. **Cultivation of Cash Crops.** Irrigation makes it possible to grow cash crops such as sugar-cane, indigo, tobacco, cotton, etc.

4. **Elimination of mixed cropping.** If irrigation facilities are not available in an area, the farmers have a tendency to adopt *mixed cropping* whereby they grow two or more crops together in the same field. The advantage of mixed cropping is that if the weather conditions are not suitable for one of the crops, it may be suitable

for the others and thus the farmer may get atleast some yield from the same field. But, mixed cropping has several drawbacks, specially due to the fact that each crop has different requirements for field preparation, manuring, watering etc. However the availability of irrigation facilities in the area gives the farmers the protection against the vagaries of the weather, resulting in the elimination of the mixed cropping system.

5. **Addition to the Wealth of the Country.** Almost all the irrigation projects are so designed that they bring some revenue to the state. In addition to the water tax obtained from the farmers, the bumper crops produced due to irrigation facilities makes country self-sufficient in food requirements, and thus saves the foreign exchange.

6. **Increase in Prosperity of People.** Due to irrigation facility, the value of land is increased. Also due to increase in the yield of the crop and due to the growing of cash crops, the farmers become prosperous, and thereby their living standard is improved.

7. **Generation of Hydro-Electric Power.** Major river valley projects are usually planned to provide hydro-electric power together with irrigation. Also, falls on the irrigation channels can be utilised to generate electricity which may help in industrialising the rural area, and also in solving the problem of fuel shortage.

8. **Domestic and Industrial Water Supply.** The canals may also be utilised for domestic and industrial water supply. Some of the irrigation reservoirs also supply water to nearby rural and urban areas. The canals also provide facilities for bathing, cattle watering, boating and others recreations.

9. **Inland Navigation.** Large canals can also be used for inland navigation, and thus transportation of agricultural products to the mandis.

10. **Improvements of Communication.** Almost all the irrigation channels are provided with inspection roads. These roads can be metalled, and be used as means of communication.

11. **Canal Plantations.** The stripped area along the canal is always damp, and therefore, trees etc. are planted along the canal banks, water courses and field boundaries. This increases the timber wealth of the country and also checks the soil erosion.

12. **Improvement in the Ground Water Storage.** Due to the constant percolation and seepage of water, the ground water table is raised in the area where irrigation facilities are prevalent. So long as this water table remains well below the root zone of the crop, this improvement in the ground water storage is beneficial.

13. **Aid in Civilisation.** Due to the introduction of river valley projects, the violent tribes adopt irrigation as their profession, and settle peacefully. Also due to increased yield and value of the crop, the standard of living of a common cultivator is improved, and he becomes more civilised.

14. **General Development of the Country.** The most important benefit derived from the irrigation is the general development of an agricultural country such as ours in which majority of people live in villages and have agriculture as their profession. Due to the increased yield and value of the crop, means of communication, such as road-ways, railways and post and telegraph facilities are introduced. Due to the increase

in agricultural revenue of the state, and due to rise in the living standards of the people, schools, hospitals and other facilities are provided. All these in turn have direct influence in making the people better citizens. There are instances in which the most dry and backward areas have become prosperous and civilised, mainly due to the introduction of irrigation facilities.

1.6. ILL-EFFECTS OF IRRIGATION

Excess irrigation and unscientific use of irrigation water may give rise to the following ill-effects :

✓ **Breeding Places for Mosquitoes** : Due to excess application of water, and due to leakage of water, ponds and depressions get filled up with water and create breeding places for mosquitoes. Also, if the canal is leaky, mosquitoes breed all along the canal and spread malarial conditions.

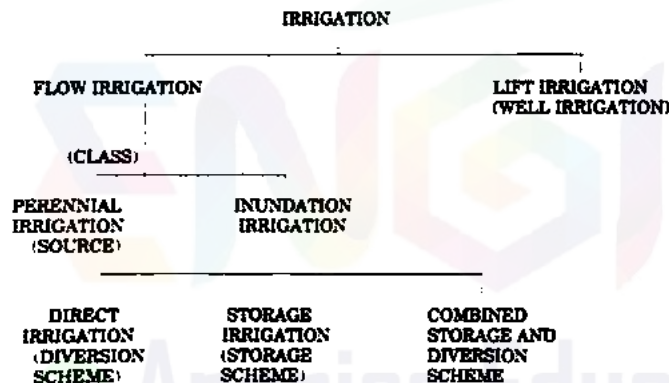
✓ **Water-Logging** : If the water table is near the ground surface, over-irrigation may raise the water table. This saturates the crop root-zone completely, causes efflorescence and the whole area becomes waterlogged.

✓ **Damp Climate** : The areas which are already damp and cold, become damper and colder due to irrigation.

1.7. TYPES OF IRRIGATION

Irrigation has the following main types or classes :

(a) Flow Irrigation, and (b) Lift Irrigation.



Flow Irrigation

Flow irrigation is that type of irrigation in which the supply of irrigation water available is at such a level that it is conveyed on to the land by the gravity flow.

Flow irrigation may further be divided into two classes :

(i) Perennial irrigation system (ii) Inundation or flood irrigation system.

Perennial Irrigation : In perennial irrigation system, the water required for irrigation is supplied in accordance with the crop requirements throughout the crop period. For such a system, therefore, some storage head works, such as dams and storage weirs or barrages are required to store the excess water during floods and release it to the crops as and when it is required.

Inundation Irrigation : Inundation irrigation is carried out by deep flooding and thorough saturation of the land to be cultivated which is then drained off prior to the planting of the crop.

Depending upon the source from which the water is drawn, flow irrigation can be further subdivided into three types :

- Direct irrigation (River canal irrigation) : Diversion scheme
- Storage irrigation (Reservoir or tank irrigation) : Storage scheme
- Combined storage and diversion scheme.

1. Direct Irrigation or River Canal Irrigation

In this direct irrigation system, water is directly diverted to the canal without attempting to store the water. For such a system, a low diversion weir or diversion barrage is constructed across the river. This raises the water level in the river and

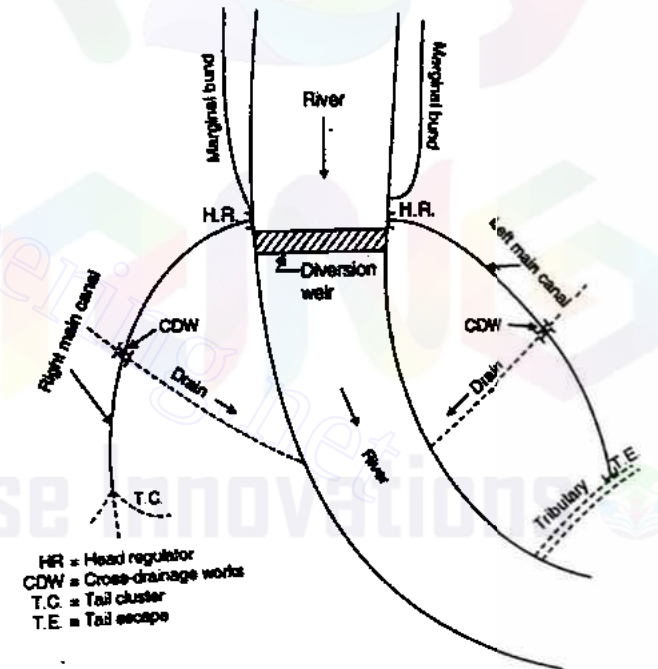


FIG. 1.1. DIRECT IRRIGATION SCHEME.

thus diverts the water to the canal taking off upstream of the weir, as shown schematically in Fig. 1.1.

Generally, a direct irrigation scheme is of a smaller magnitude, since there are no rigid controls over the supplies. One or two main canals may take off directly from the river. Cross-drainage works are constructed wherever natural drains or distributary streams cross the canals. In a bigger scheme, there may be branch canal taking off from the main canal.

2. Storage Irrigation or Tank Irrigation

In storage irrigation system, a solid barrier, such as a dam or a storage weir is constructed across the river and water is stored in the reservoir or lake so formed.

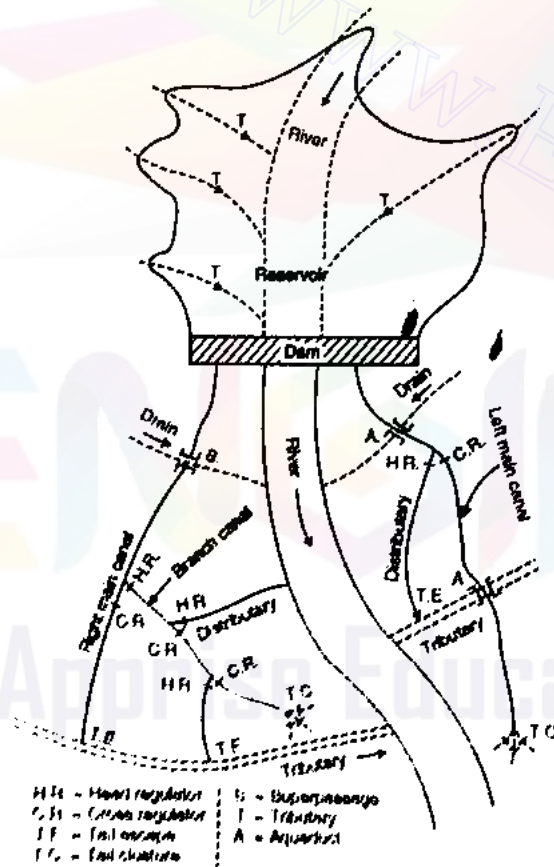


FIG. 1.2. STORAGE IRRIGATION SCHEME.

Depending upon the water requirements of crops, or the hydroelectric power generation, and upon the flow of water in the river, the volume of storage required is decided. From the contour plan of the basin at the site of construction, the elevation storage curve for the reservoir is known. The height of the dam is then decided from this curve, corresponding to the storage-volume required. In India, most of the irrigation schemes fall under this category. Fig. 1.2 shows the schematic plan for such a scheme.

Storage irrigation scheme is comparatively of a bigger magnitude, and involves much more expenditure than a direct irrigation scheme. One or two main canals take off from the reservoir. Due to the formation of reservoir, some land property may be submerged to the upstream of the dam. A network of canal system convey water to the agricultural fields, through various regulatory works. Cross-drainage works such as aqueducts, syphon aqueducts, super passages and canal syphons are constructed wherever natural drains cross the canals.

3. Combined System (Storage cum Diversion Scheme)

We have seen that in the storage irrigation system, water is stored in the reservoir, since the river is not perennial, while in the direct irrigation system, the river is

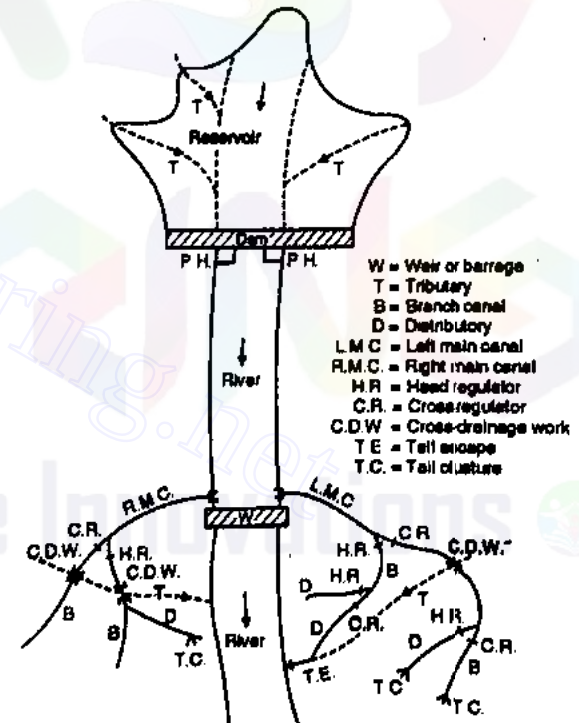


FIG. 1.3. STORAGE-CUM-DIVERSION SCHEME.

perennial and hence the water is *diverted* from the river to the canal. Sometimes, a combined scheme is adopted in which the water is first stored in the reservoir formed at the upstream side of the dam, and this water is used for water power generation. The discharge from the power house is fed back into the river, to the downstream side of the dam. Thus, sufficient quantity of flow is again available in the river. At a suitable location in the downstream, a *pick-up weir* is constructed. This weir diverts the water from the river to the canal. Fig. 1.3 shows such a storage-cum-diversion scheme. One typical example of such a scheme is the Kota dam and Kota barrage on Chambal river in Rajasthan.

Choice Between the Systems

Direct irrigation scheme is adopted in the circumstances where the river is perennial and has a normal flow throughout the irrigation season, never less at any time than the requirements of the field. On the contrary, storage irrigation system is adopted when the river flow is either not perennial, or where flow is insufficient during certain parts of the crop season for irrigation requirements. In a multistage river-valley development, a combined storage-cum-diversion scheme is more useful.

Lift Irrigation

Lift irrigation is practised when the water-supply is at too low a level to run by gravitation on to the land. In such a circumstances water is lifted up by mechanical means. Irrigation from wells is an example of lift irrigation, in which sub-soil water is lifted up to the surface and is then conveyed to the agricultural fields.

1.8. HISTORY OF IRRIGATION DEVELOPMENTS IN INDIA

Irrigation practice¹ in India can be traced to prehistoric times. Perhaps the earliest reference to irrigation is given by the great sage, Narada, who came to see the Emperor, Yudhishtira, around 3150 B.C. and asked "Are the farmers sturdy and prosperous? Are the dams full of water and big enough and distributed in different parts of the kingdom?"² Ancient Hindu rulers took very keen interest in construction of irrigation works. The account left by Megasthenes - the Greek Ambassador of Seleukos Nikator to the court of Chandra Gupta Maurya - also gives some idea about irrigation in India around 300 B.C. when "the whole country was under irrigation". Rock inscriptions found in the State of Kathiawar also credit Chandra Gupta with the construction of *Sudarsan lake* (300 B.C. - 457 A.D.). An excellent description of the *pleasant looking dam* is found from the inscriptions recording two major disasters that befell it, in one of which "by a breadth, four hundred and twenty cubits long, just as many broad (and) seventy-five cubits deep, all the water flowed out so that (the lake), almost like a sandy desert, (became) extremely ugly (to look at)."² Chola rulers of South India also constructed necessary irrigation works.

The Muslim rulers also took keen interest in constructing canals for irrigation. In the 14th century, Sultan Firoz Tughlaq constructed a number of canals from Sutlej

1. "Irrigation in India : Past and Present" by A. K. Biswas, Journal of the Irrigation and Drainage Division, ASCE March, 1965.

2. "Selection from Sanskrit Inscriptions", Vol. I, part II, by D.B. Diskalkar.

and Yamuna rivers. Akbar remodelled Firozah canal in 16th century and put it in use again. Shah Jahan also constructed a number of new canals and remodelled old ones.

State-controlled irrigation works have begun to play a major role only in the last about 150-200 years. In the nineteenth century, the *British Raj* was more concerned with the improvement of existing irrigation works than with building new ones. Three important irrigation works - Western Yamuna Canal, Eastern Yamuna Canal and Cauvery Delta System - were remodelled and opened.

Sir Proby Cautley and Sir Arthur Cotton were perhaps amongst the leaders who laid the foundation of modern irrigation engineering in India. Cautley's finest work is the aqueduct over Solani river. He was also responsible for the Great Ganga Canal and the Eastern Yamuna Canal. Cotton worked on the Southern side of subcontinental peninsula, and built the upper anicut across Coleroon, a tributary of Cauvery river. He also developed Godavari and Krishna deltas. Table 1.1 shows the major irrigation works completed from 1820-1930.

TABLE 1.1
MAJOR IRRIGATION WORKS COMPLETED FROM 1820-1930

Project	Year of completion	Area irrigated in thousand hectares
Western Yamuna Canal	1820	431
Ganga Canal	1856	655
Lower Ganga Canal	1880	508
Sirhind Canal	1884	935
Cauvery Delta System	1889	434
Godavari Delta System	1890	487
Krishna delta System	1898	405
Sarda Canal	1930	525

Perhaps the first hydro-electric work in India was undertaken in 1897 near Darjeeling. The State of Mysore constructed in 1902, the Sivasamudram hydroelectric works with an installed capacity of 4500 kW which was later increased to 42,000 kW. Tatas constructed Khapoli hydroelectric works (1914) with an installed capacity of 50,000 kW, Bhivpuri hydroelectric works (in 1920) with a capacity of 72,000 kW, and Bhira Scheme (1927) with an installed capacity of 110,000 kW. During the period between the two World Wars some hydroelectric schemes were taken up by various states. They include Jogindernagar hydroelectric scheme (48,000 kW), Ganga Canal power stations (17,400 kW), Pykara hydroelectric scheme (38,750 kW), Mettur hydroelectric scheme (30,000 kW) and Pillivasal scheme (13,500 kW). After Second World War and before independence, only two hydroelectric installations, i.e., Papansam (21,000 kW) and Jog (48,000 kW) were made.

After independence (1947), the Government of India was concerned with the need for food for a vastly growing population and irrigation was given a much needed impetus. Nearly 27.2% of the total outlay in the First Five Year plan (1951-56) was set aside for irrigation and power. In the Second Five Year Plan (1956-61), 19% of the total outlay was used for irrigation and power. In the Third Five Year Plan (1961-66), about 22 % of the total outlay was spent on irrigation and power development. Table 1.2 shows the major irrigation schemes undertaken after 1947.

TABLE 1.2
MAJOR IRRIGATION SCHEMES AFTER 1947

Project	Area to be irrigated (million hectares)	Total cost in crore rupees (of irrigation part only)
Bhakra Nangal	1.46	101.9
Gandak	1.35	49.5
Hirakud	0.87	93.3
Nagarjuna Sagar	0.83	91.1
Chambal	0.57	54.9
Kosi	0.56	24.8
Damodar Valley	0.51	34.7
Mahi	0.30	41.0
Rajasthan canal	1.41	206.0

Table 1.3. gives the plan wise irrigation potential created, irrigation potential utilised and approximate expenditure incurred in India.

TABLE 1.3. PLANWISE IRRIGATION POTENTIAL CREATED, POTENTIAL UTILISED (CUMULATIVE) AND EXPENDITURE INCURRED

S.N.	Plan with Period	Potential created (cumulative in Mha)	Potential utilised (cumulative in Mha)	Expenditure in curred (Rs. Crores)
1.	Pre - Plan (upto 1951) and 1 st plan (1951-56)	26.25	25.05	+ 443
2.	Second Plan (1956-61)	29.06	27.78	542
3.	Third Plan (1961-66)	33.57	32.18	1019
4.	Annual Plans (1966-69)	37.12	35.77	991
5.	Fourth Plan (1969-74)	44.20	42.09	2415
6.	Fifth Plan (1974-78)	52.02	48.26	3928
7.	Annual Plans (1978-80)	56.61	52.65	3061
8.	Sixth Plan (1980-85)	65.22	58.82	10786
9.	Seventh Plan (1985-90)	76.53	68.59	17387
10.	Annual Plan (1990-92)	81.09	72.86	27000
11.	Eighth Plan (1992-97)	95.43	80.37	
12.	Ninth Plan (1997-2002)	105.33	83.91	47000

India has unequal distribution of rainfall over the country varying from less than 25 cm in the north-west deserts of Rajasthan to about 1150 cm at Cherrapunji in the Assam hills. Substantial variations in the quantity, incidence and duration of rainfall in individual tracts from year to year, make irrigation of supreme necessity in the country.

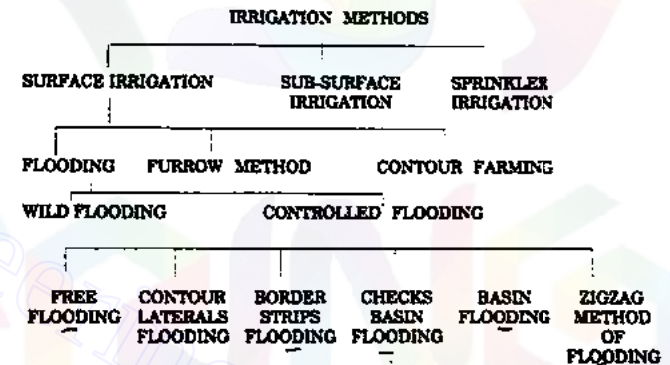
PROBLEMS

1. Define irrigation. What is the necessity of irrigation ?
2. Write a note on scope of irrigation. What is meant by a multipurpose river valley project ?
3. Discuss in brief the benefits and ill-effects of irrigation.
4. Write an essay on 'History of irrigation developments in India'.
5. Describe in brief some of the important irrigation projects and multipurpose river valley projects undertaken or completed after Independence of our country.

Methods of Irrigation

2.1. MODES OR METHODS OF APPLYING WATER TO CROPS

Irrigation water may be applied to the crops by three basic methods : (1) Surface irrigation method, (2) Sprinkler irrigation method, and (3) Sub-surface irrigation method. The surface irrigation method can further be sub-divided into various methods as indicated below:



Every irrigation method has both advantages and disadvantages. Before selecting a particular method, the irrigation engineer (or agricultural engineer) must evaluate all the factors, and choose that method which is best suited to the local conditions.

The basic requirements for adaptation of any irrigation method or mode of application of water are as follows :

1. The method should be such that uniform water distribution with as small as 6 cm water depth applications can be made for light irrigation.
2. At the same time, it should afford heavy uniform application of 15 to 20 cm water depth.
3. It should allow the use of large concentrated water flows for reduction of conveyance losses, and labour cost.
4. It should be suitable for use with economic conveyance structure.

(15)

5. It should be such that mechanised farming is facilitated.

Good irrigation method results in increased yield and conservation of resources with soil productivity maintained and water utilised economically. Over irrigation results in soil erosion, leached fertilizers, drainage troubles and salt accumulation.

2.2. UNCONTROLLED OR WILD FLOODING

In this method, water is spread or flooded on a rather smooth flat land, without much control or prior preparation. This method is generally used in the inundation irrigation system in which water is forced to spread over vast tracts during the season of high stream flow. It is wasteful use of water, and is practised largely where irrigation water is abundant and inexpensive.

2.3. FREE FLOODING OR FLOODING FROM FIELD CHANNELS

In the *controlled flooding*, water is spread over the land, with proper methods to control the depth of application. Controlled flooding can be achieved by free flooding, contour laterals, border strips or by check flooding.

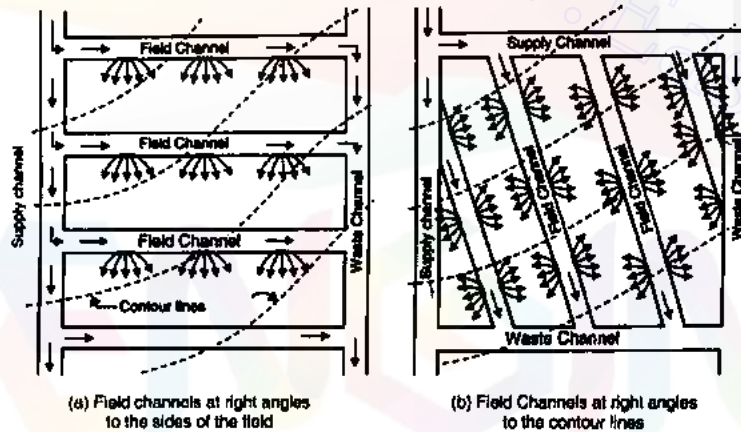


FIG. 2.1. FREE FLOODING METHOD.

Free flooding method consists of dividing the entire land to be irrigated into small strips by a number of field channels, known as *laterals*. These laterals may be either at right angles to the sides of the field, or at right angles to the contour lines, as shown in Fig. 2.1 (a) and (b) respectively. When the laterals are aligned at right angles to the sides of the field, irrigation is possible only to one side of the lateral. However, when the laterals are aligned at right angles to the contours, irrigation done to both the sides of the lateral (Fig. 2.1 b). The field channels receive water from the supply channel and discharge excess water in the waste channel. Water is applied to the strip of a land through feeding points provided at some interval along the laterals. This method of water application can be used both for flat lands as well as for relatively steep lands. The alignment and spacing of the laterals depends upon

the type of soil (i.e. its permeability characteristics) and topography of the land. If the slopes are steeper, closer spacing of laterals is required. Similarly, closer spacing is required for relatively more permeable soils. For flat land and for relatively less pervious soils, the spacing of the laterals may be increased. Normally the spacing of the laterals may vary from 10 to 50 metres. However, the method is more suited for irrigation fields containing medium type soil and having slopes ranging from 1 in 100 to 1 in 300.

2.4. FLOODING BY CONTOUR LATERALS

This is a special case of *free flooding* (or flooding from field channels) in which the field channels or laterals are aligned approximately along the contour lines.

This method is applicable for steeper terrain. The field is cut by relatively dense network of small contour laterals, the spacing of which depends upon the prominent grade of the field between two adjacent ditches or laterals, the uniformity of slope and the soil type.

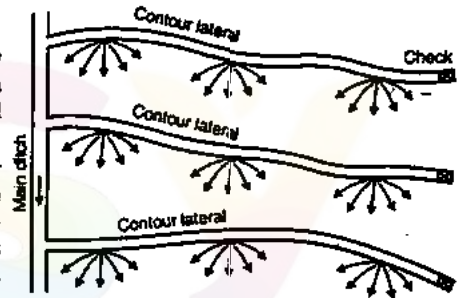


FIG. 2.2. CONTOUR LATERALS.

2.5. BORDER STRIP FLOODING METHOD

In the border strip flooding method, the farm is divided into a series of strips 10 to 20 metres wide and 100 to 300 metres long. These strips are separated by low levees or borders (low flat dikes) and run down the predominant or any other desired slope. To irrigate, water (discharge varying from 0.014 to 0.028 cumec) is turned from the supply ditch onto the head of the border. Water advances — confined and guided by two borders — in a thin sheet towards the lower end of the strip. The surface is essentially level between two borders so that the advancing sheet of water covers the entire width of the strip. The lengthwise slope varies from 0.2 to 0.8%. The border strip should be level cross-wise.

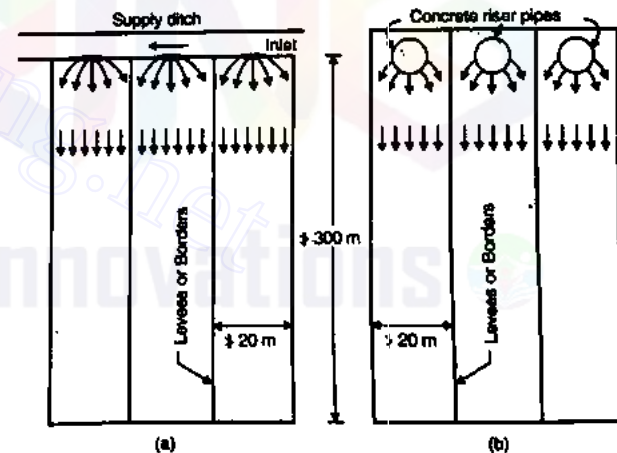


FIG. 2.3. BORDER STRIP METHOD.

Let A = Area of land covered at any time t ,
 I = Rate of infiltration in m/hour
 Q = Discharge for the strip in cumecs or ha-m/hour
 t = Time required to cover area A .
 y = Average depth of sheet of flowing water, in metres.

Total quantity of water flowing in small time interval dt is equal to the quantity of water infiltrated in this time dt over the area A plus the quantity of surface flow over the area dA .

$$Q dt = y dA + IA dt$$

$$\text{From which } dt = \frac{y dA}{Q - IA} \quad \dots(1)$$

Considering I and y as constants and integrating, we get

$$t = \frac{y}{I} \log_e \frac{Q}{Q - IA}$$

$$= 2.303 \frac{y}{I} \log_{10} \frac{Q}{Q - IA} \quad \dots(2) \dots(2.1)$$

The constant of integration being zero since $A = 0$ when $t = 0$. In the above equation, I has been considered constant, but actually I decreases as the soil gets saturated. Eq. (2) can be rewritten in the form below:

$$\log_{10} \frac{Q}{Q - IA} = \frac{I \cdot t}{2.303 y} = x \text{ (say)}$$

or

$$10^x = \frac{Q}{Q - IA}$$

$$\text{From which } A = \frac{(10^x - 1) Q}{10^x I} = \frac{Q}{I} \left(1 - 10^{-x} \right) \quad \dots(2.2)$$

Equation (3) is very important since it gives the maximum area that can be irrigated with a stream of discharge Q .

If B is the width of strip and L is the length of strip or length of run, we have $A = B \cdot L$. Similarly, q = discharge per unit width = Q/B . Hence from Eq. 2.1, we have

$$t = \frac{y}{I} \log_e \frac{Q}{Q - I B L} = \frac{y}{I} \log_e \frac{q}{q - IL} \quad \dots(2.1 a)$$

$$\text{Hence } L = \frac{q}{I} \left[1 - e^{-It/y} \right] \quad \dots(4) \dots(2.3)$$

If t is the time required for infiltration of water to the desired depth d of irrigation to the root zone, we have

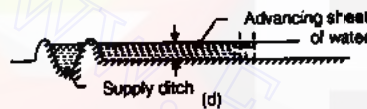
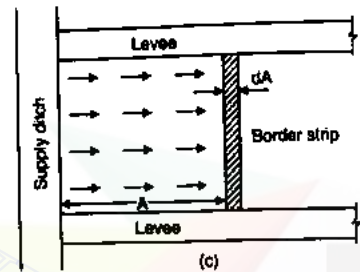


FIG. 2.5

$$d = I \cdot t$$

...(5)

Substituting the value of $I \cdot t$ in Eq. (4), we get

$$L = \frac{q}{I} \left[1 - e^{-d/y} \right] \quad \dots(2.4)$$

Eq. 2.4 can be used for determining the length of border (L) for given values of I , q , d and y . Similarly, for given values of L , I , d and y , the unit flow rate q can be determined.

Average depth y of surface flow

The average depth of surface flow, y , is related to the normal or uniform depth y_0 by the relation

$$y = 0.665 y_0 \quad \dots(2.5)$$

Also, the normal or uniform depth y_0 is given by Manning's equation

$$y_0 = \left(\frac{nq}{\sqrt{s}} \right)^{0.48} \quad \dots(2.6)$$

where s is the slope of the border strip and n is the roughness coefficient given by the following equation :

$$n = 0.55425 - 0.0001386 q' \quad \dots(2.7)$$

where q' = inflow per unit width of border strip, in $\text{cm}^3/\text{min}/\text{cm}$

Time of application of water to the strip

If the infiltration rate (I) and desired depth (d) of infiltration are known, the time of application is given by

$$t = d/I \quad \dots(5a) \dots(2.8)$$

Example 2.1. Find the time required to cover an area of 0.1 hectares when a tube well is discharging at the rate of 0.03 cumec for irrigating rabi crops. Average depth of flow is expected to be 7.5 cm. Average infiltration rate for the soil may be taken as 5 cm/hour.

Solution

$$Q = 0.03 \text{ cumec} = 0.03 \times 3600 \text{ m}^3/\text{hour} = 0.108 \text{ hectare-metre/hour}$$

$$y = 7.5 \text{ cm} = 0.075 \text{ m} : I = 5 \text{ cm/hour} = 0.05 \text{ m/hour}$$

$$A = 0.1 \text{ hectare}$$

Substituting in Eq. 2.1, we have

$$t = 2.303 \frac{y}{I} \log_{10} \frac{Q}{Q - IA} = \frac{2.303 \times 0.075}{0.05} \log_{10} \left(\frac{0.108}{0.108 - 0.1 \times 0.05} \right) \text{ hours} = 56 \text{ minutes.}$$

Example 2.2. For the above example, find the maximum area that can be irrigated by the available discharge of 0.03 cumec.

Solution : From Eq. 2.2, it is obvious that the maximum area that can be irrigated is Q/I . After this area, the surface flow will stop and total percolation will result.

$$A = \frac{Q}{I} \text{ hectares} = \frac{0.108}{0.05} = 2.16 \text{ hectare.}$$

Example 2.3. For a given field soil, the average infiltration rate is 45 mm per hour, and desired depth of water application corresponding to the depth of root zone is 50 mm. Determine (a) optimum length of each border strip if the discharge of the water source entering into the border strip is 18 litres per second and width of each border strip is 9 m, and (b) the inflow rate if the length of each border strip is 150 m and its width is 9 m. What will be the time of water application in each case? Take the slope of the border strip as 0.35%.

Solution $d = 50 \text{ mm} = 0.05 \text{ m}$

Also, $I = 45 \text{ mm/hour} = \frac{45}{1000 \times 3600} = 1.25 \times 10^{-5} \text{ m/s}$

Time of water application, $t = \frac{d}{I} = \frac{50}{45} = 1.111 \text{ hours} \approx 1 \text{ hour } 7 \text{ min.}$

(a) **Optimum length of border strip :**

$$q = 18 \text{ lps per } 9 \text{ m width} = \frac{18}{9 \times 1000} = 2 \times 10^{-3} \text{ m}^3/\text{s/m width}$$

$$q' = (\text{in cm}^3/\text{min/cm width}) = \frac{2 \times 10^{-3} (100)^3 (60)}{100} = 1200 \text{ cm}^3/\text{min/cm width}$$

From Eq. 2.7, $n = 0.55425 - 0.0001386 q' = 0.55425 - 0.0001386 \times 1200 = 0.3879$

From Eq. 2.6, $y_0 = \left(\frac{nq}{\sqrt{s}} \right)^{0.6} = \left(\frac{0.3879 \times 2 \times 10^{-3}}{\sqrt{0.0035}} \right)^{0.6} = 0.0742 \text{ m}$

Hence from Eq. 2.5, $y = 0.665 y_0 = 0.665 \times 0.0742 = 0.049 \text{ m}$

From Eq. 2.4, $L = \frac{q}{I} \left[1 - e^{-d/I} \right] = \frac{2 \times 10^{-3}}{1.25 \times 10^{-5}} \left[1 - e^{(-0.05/0.049)} \right] = 102.3 \text{ m}$

(b) **Inflow rate for $L = 150 \text{ m}$.**

For a given value of L , q is determined from Eq. 2.4. However, Eq. 2.4 also contains depth y which is dependent on y_0 and depth y_0 is again dependent on q as evident from Eq. 2.6. Hence the determination of discharge, for a given value of L , is essentially a process of trial and error.

First trial : In part (a), given inflow rate is $18/9 = 2 \text{ lps per m width}$ for $L \approx 102.3 \text{ m}$. Hence the inflow rate for $L = 150 \text{ m}$ will be approximately equal to $(150/102.3) 2 = 2.93 \text{ lps}$. In the first trial, let $q = 3 \text{ lps/m width}$.

Thus $q = 3 \text{ lps/m} = 3 \times 10^{-3} \text{ m}^3/\text{s/m width}$

$$q' = \frac{3 \times 10^{-3} (100)^3 (60)}{100} = 1800 \text{ cm}^3/\text{min./cm width}$$

$$n = 0.55425 - 0.0001386 \times 1800 = 0.3048$$

$$y_0 = \left(\frac{0.3048 \times 3 \times 10^{-3}}{\sqrt{0.0035}} \right)^{0.6} = 0.0819 \text{ m}$$

$$y = 0.665 \times 0.0819 = 0.0545 \text{ m}$$

$$L = \frac{3 \times 10^{-3}}{1.25 \times 10^{-5}} \left[1 - e^{(-0.05/0.0545)} \right] = 144.11 \text{ m}$$

But given value of $L = 150 \text{ m}$

Second trial : In the second trial, let $q = \frac{150}{144.11} \times 3 \approx 3.12 \text{ lps/m}$

Assume $q = 3.15 \text{ lps/m} = 3.15 \times 10^{-3} \text{ m}^3/\text{s/m width}$

$$q' = \frac{3.15 \times 10^{-3} (100)^3 (60)}{100} = 1890 \text{ cm}^3/\text{min./cm}$$

$$n = 0.55425 - 0.0001386 \times 1890 = 0.2923$$

$$y_0 = \left(\frac{0.2923 \times 3.15 \times 10^{-3}}{\sqrt{0.0035}} \right)^{0.6} = 0.0823 \text{ m}$$

$$y = 0.665 \times 0.0823 \approx 0.055 \text{ m}$$

$$L = \frac{3.15 \times 10^{-3}}{1.25 \times 10^{-5}} \left[1 - e^{(-0.05/0.055)} \right] = 150.9 \text{ m}$$

This is approximately equal to the given length $L = 150 \text{ m}$. Hence no further trial is necessary. Thus, required discharge $q = \frac{150}{150.9} \times 3.15 = 3.13 \text{ lps/m width}$, and the discharge for each border strip $= 3.13 \times 9 \approx 28.2 \text{ lps}$.

2.6. CHECK FLOODING

Check flooding is similar to free flooding except that the water is controlled by surrounding the check area with low, flat levees surrounding each check while in free flooding no such levees are provided and the strips are divided by field channels.

It is the most common method of irrigation used in India as well as in many other countries. The method is also known as *irrigation by plots* since the entire area is divided into several plots obtained by subdividing the entire area by levees. Each plot is practically level. The size of the levee depends upon the depth of water to be applied as well as the stability of the soil when it is wet. Water is conveyed to the land by a system of *supply channels* (known as *main ditch*) as well as *laterals* (known as *field channels*). Usually, there is one field channel for every two rows of plots as shown in Fig. 2.6. Water is admitted to these plots at the higher end and the supply is cutoff as soon as the lower part of the plot has received

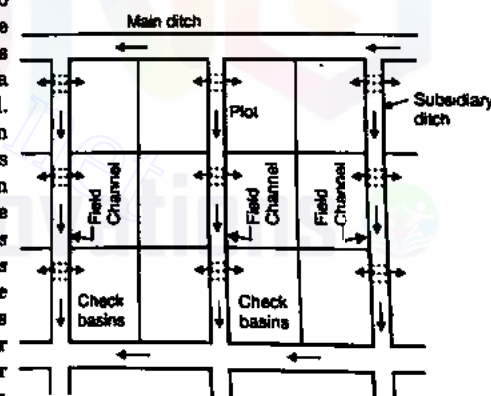


FIG. 2.6. CHECK FLOODING OR IRRIGATION BY PLOTS

the sufficient depth of water. Oblong plots are preferred to the square plots. The size of the plots depends on the porosity of the soil.

In a levelled ground, the plots are generally rectangular, but if the ground has same initial slope, the checks or levees may follow contours, as shown in Fig. 2.7. Contour checks are prepared by constructing ridges along contours at vertical intervals of 6 to 12 cm and connecting them with cross-ridges at intervals. Very little crop land is perfectly level, and the water does not stand at a uniform depth over the entire check. This results in uneven distribution of water within the plot. Low area will receive too much water and high gets too little.

The size of check basins (also known as checks or plots) is dependent on the infiltration rate (also known as intake rate or the depth of water entering the soil per unit time). It may vary from 1 m² for growing vegetables and other intensive irrigation to 1-2 hectares for growing rice under wet land conditions. However, the more common size varies from 0.03 to 0.06 hectares for medium soils. The size of the stream should be sufficient to cover the entire basin in a relatively small portion of time required to apply the desired depth of irrigation water.

Since water is supplied through large stream, the method is more suitable for permeable soils which must be quickly covered with water in order to prevent excessive percolation losses near the supply ditches. The method is also suitable for impermeable soils in which the percolation rate is so slow that infiltration would be inadequate in the time required for the sheet of water flowing over it.

Design Features

Let t_1 = time of advance (i.e. the time required to cover the entire length L of the basin)

t_d = time of ponding (Refer Fig. 2.4)

Since some time (t_1) is required for water to spread over the entire basin, the time of ponding (t_d) and hence the depth of water absorbed by the soil will not be the same for the entire basin. Thus, in order to obtain a higher water application efficiency, the time of advance should be as small as possible. The general practice is to keep the time to spread water in the entire base equal to one fourth the time required to infiltrate the net depth (d) of irrigation water.

The rate of infiltration (I) is a function of time t and can be expressed by the expression :

$$I = k \cdot t^n \quad \dots(1) \dots(2.9a)$$

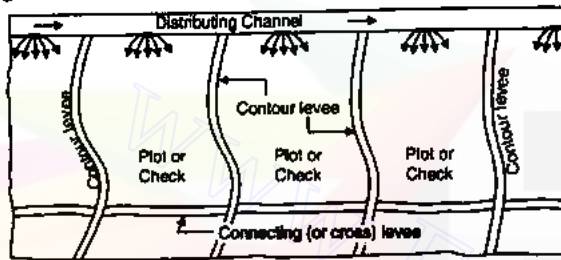


FIG. 2.7. CHECK FLOODING WITH CONTOUR LEVEES.

where k and n are the constants depending upon the type of the soil.

Hence the accumulated depth of infiltration during the time of ponding (t_d) is given by

$$I_c = \int_0^{t_d} k t^n dt = \frac{k \cdot t_d^{n+1}}{n+1} = k' t_d^{n'} \quad \dots(2) \dots(2.9)$$

where $k' = k/(n+1)$ and $n' = n+1$

Neglecting the volume of water absorbed during the time of advance, the total volume of water (V_d) absorbed over a length L , per unit width of the check basin is given by

$$V_d = \int_0^L I_c \cdot dL = \frac{k}{n+1} \int_0^L t_d^{n+1} dL = \frac{k L t_d^{n+1}}{n+1} = k' L t_d^{n'} \quad \dots(3) \dots(2.10)$$

Let T = Total time of irrigating the check basin = $t_1 + t_d$

The time of advance (t_1) is a function of stream size and soil characteristics, and can be expressed by the following advance function

$$L = a (t_1)^b \quad \dots(4) \dots(2.11)$$

where a and b are constants.

The total volume (V_T) absorbed over a length L , per unit width of check basin, during the total time $T (= t_1 + t_d)$ is obtained from Eq. (3) in the form

$$V_T = \frac{k L T^{n'+1}}{n'+1} = k' L T^{n'} \quad \dots(5)$$

Hence deep percolation loss, expressed as a fraction of total volume of water is given by

$$p = \frac{V_T - V_d}{V_T} = \frac{T^{n'+1} - t_d^{n'+1}}{T^{n'+1}} = \left[1 - \left(\frac{t_d}{T} \right)^{n'+1} \right] \quad \dots(6) \dots(2.12)$$

Hence water application efficiency (η_a) is given by

$$\eta_a = \frac{V_d}{V_T} \times 100 = \left(\frac{t_d}{T} \right)^{n'+1} \times 100 = \left(\frac{t_d}{t_1 + t_d} \right)^{n'+1} \times 100 = (1-p) 100 \quad \dots(7) \dots(2.13)$$

$$\text{Volume of water to be applied} = \frac{d L B}{\eta_a} \quad \dots(8) \dots(2.14)$$

where B is the width of the basin

$$\text{Time of irrigation} = \frac{d L B}{\eta_a Q} \quad \dots(a) \dots(2.15)$$

where Q is the discharge of the irrigating ditch or stream.

Example 2.4. For a check basin of size 12 m × 36 m, the following functions are available :

1. Cumulative infiltration $I_c = 9 t^{0.42}$, where I_c is in mm and t is in minutes

2. Advance function :(a) $L = 5t_L^{0.6}$ for a stream of size 1.5 lps/m width(b) $L = 8t_L^{0.6}$ for a stream of size 3 lps/m width

If the depth of irrigation is 70 mm, determine (a) deep percolation loss, (b) water application efficiency, and (a) time of irrigation for each of the above sizes of stream.

Solution : Given : $B = 12$ m ; $L = 36$ m and $d = 70$ mm; $k' = 9$; $n' = 0.42$

(a) For stream of 1.5 lps/m width : Given : $a = 5$ and $b = 0.6$

$$q = 1.5 \text{ lps and hence } Q = qB = (1.5 \times 12) \text{ lps} = 1.5 \times 12 \times 10^{-3} \text{ m}^3/\text{s}$$

$$\text{From Eq. 2.11, } t_L = \left(\frac{L}{a}\right)^{1/b} = \left(\frac{36}{5}\right)^{1/0.6} = 26.85 \text{ minutes}$$

$$\text{From Eq. 2.9, } t_d = \left(\frac{I_c}{k'}\right)^{1/n'} = \left(\frac{70}{9}\right)^{1/0.42} = 132.16 \text{ minutes}$$

$$\therefore \text{Total time } T = t_L + t_d = 26.85 + 132.16 = 159.01 \text{ min.}$$

$$\text{Now } p = \left[1 - \left(\frac{t_d}{T}\right)^{n'}\right] = \left[1 - \left(\frac{132.16}{159.01}\right)^{0.42}\right] = 0.0747 = 7.47\%$$

$$\text{Also, } \eta_a = (1 - p) 100 = (1 - 0.0747) 100 = 92.53\%$$

$$\text{Time of irrigation} = \frac{dLB}{\eta_a Q} = \frac{(70 \times 10^{-3})(36)(12)}{0.9253 \times 1.5 \times 12 \times 10^{-3}} \times \frac{1}{60} = 30.3 \text{ min.}$$

(b) For stream of 3 lps/m width : Given $a = 8$ and $b = 0.6$

$$q = 3 \text{ lps and hence } Q = qB = (3 \times 12) \text{ lps} = 3 \times 12 \times 10^{-3} \text{ m}^3/\text{s}$$

$$t_L = \left(\frac{L}{a}\right)^{1/b} = \left(\frac{36}{8}\right)^{1/0.6} = 12.27 \text{ minutes}$$

$$t_d = \left(\frac{I_c}{k'}\right)^{1/n'} = \left(\frac{70}{9}\right)^{1/0.42} = 132.16 \text{ minutes (as before)}$$

$$T = t_L + t_d = 12.27 + 132.16 = 144.43 \text{ minutes}$$

$$p = \left[1 - \left(\frac{t_d}{T}\right)^{n'}\right] = \left[1 - \left(\frac{132.16}{144.43}\right)^{0.42}\right] = 0.0366 = 3.66\%$$

$$\eta_a = (1 - p) 100 = (1 - 0.0366) 100 = 96.34\%$$

$$\text{Time of irrigation} = \frac{dLB}{\eta_a Q} = \frac{(70 \times 10^{-3})(36)(12)}{0.9634 \times 3 \times 12 \times 10^{-3}} \times \frac{1}{60} = 14.6 \text{ min.}$$

2.7. RING BASIN FLOODING

The ring basin flooding (Fig. 2.8) is a special form of check basin flooding adapted to orchards. Ring basins are formed for each tree ; in some cases one ring basin may be formed for two or more trees. Water is supplied to these basins through a supply ditch. In some cases, a number of ring basins are inter-connected. Portable pipes or large hoses may also be used in place of ditches.

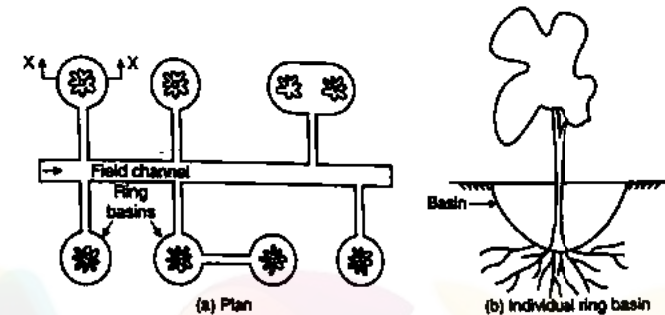


FIG. 2.8. RING BASIN FLOODING

2.8. THE ZIGZAG METHOD

It is special method of flooding where water takes circuitous route before reaching the dead end of each plot (Fig. 2.9). The whole area is divided into a number of square or rectangular plots ; each plot is then subdivided with the help of low bunds or levees. This method is suitable for relatively level plots. It is, however, highly unsuitable for farming operations with modern farm machinery.

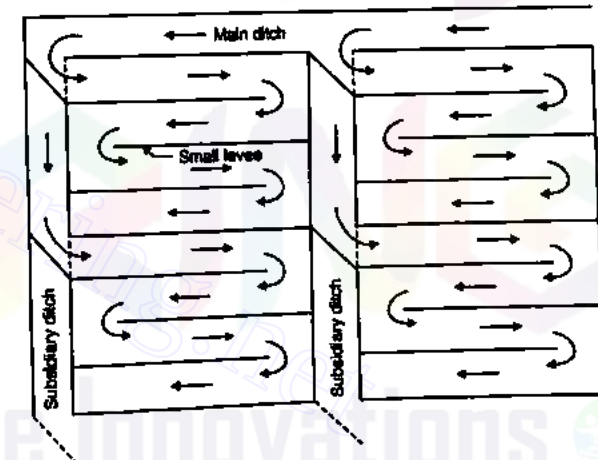


FIG. 2.9. THE ZIGZAG METHOD.

2.9. FURROW METHOD

The furrow method of irrigation is very much used for row crops like maize, jowar, sugarcane, cotton, tobacco, groundnut, potatoes etc. In the other irrigation methods described so far, almost the entire land surface is wetted, but in this method only one half to one-fifth of the surface is wetted, and thus evaporation losses are very much reduced. A furrow consists of a narrow ditch between rows of plants. The length

of furrows varies from 3.0 m or less for gardens to as much as 500 m for field crops, the common length being 100 to 200 m. If the furrows are of excessive length, deep percolation losses and soil erosion near the upper end of the field may result. The general slopes provided for furrows may vary from 0.2 to 6%.

A minimum furrow grade of 0.05% is needed to assure surface drainage.

Furrow spacing for corn, potatoes, sugarcane and other row crops is determined by the proper spacing of the plant rows, one irrigation furrow being provided for each row. The spacing of furrows is kept from 1 to 2 m in orchard irrigation. If the spacing is kept more, it is essential to check the distribution of moisture after each watering by auger boring. For soil of low permeability, the depth of furrows may vary from 20 to 30 cm. It is essential in irrigating the root crops (such as sugarcane etc.) to have furrows deep enough, and the stream in each small enough so that water cannot come in contact with the plant. The common size of furrow for row crops such as cotton, tobacco and potatoes is about 25 cm wide and 8 to 10 cm deep.

Type of Furrows

Depending upon the alignment, furrows may be of two types :

(i) Straight furrows, and (ii) Contour furrows.

(i) **Straight furrows** : Straight furrows are used where the land slope is nominal. These are aligned more or less along straight lines, parallel to each other, and along the slope of the land. These are normally adopted where the slopes do not exceed 0.6%.

(ii) **Contour furrows** : Contour furrows are practically laid along the contours. Therefore, these are not straight but are curvilinear in plan. With *contour furrows irrigation*, the direction of flow is across a sloping field rather than down the slope to reduce water velocity. The furrows are laid out with enough grade to carry the irrigation streams. *Head ditches* are run across the slope or down hill using drop structures as needed, to feed the individual furrow. Contour furrows method can be successfully used in nearly all irrigable soils. Light soils can be irrigated successfully across slopes upto 5 percent. Where the soils are stable and will not be cultivated (as in orchards), slopes upto 20 percent can be irrigated by contour furrowing.

Construction of furrows

Furrows are made before planting, at the time of planting or after the plants have grown large enough not to be covered up. The time of furrowing depends upon the

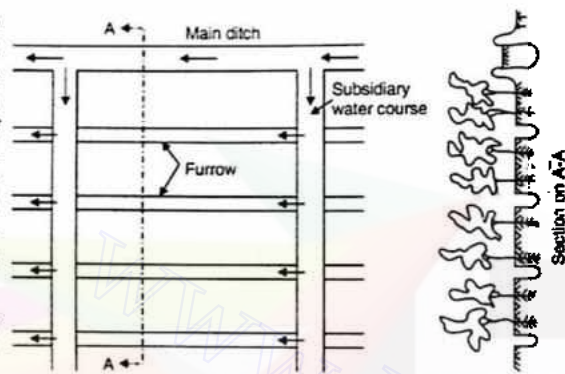


FIG. 2.10. FURROW METHOD.

crop grown and the method of planting used. Often young plants are irrigated by small furrows until a good root system is developed. Thereafter, the furrow is made larger. The furrows at any stage must be large enough to carry the water needed for irrigation.

In most soils, crops are grown on the top of the ridge while in deep sand, it is better to have the seedling near the bottom of the small furrow. Alternate method specially adapted to sandy soils is to transplant vegetables in the furrow, irrigate it once or twice and then establish furrows between the rows after plants have grown larger.

Furrows are made with various cultivating tools depending on the type of crop to be grown. Large furrows are normally made with a double mould board plough or lister. A wooden plough with furrower attachment can also be used in place of listers. Disc-drum corrugator furrower is very useful to make small size furrows in sandy soils.

Advantages of Furrow Irrigation

Furrow irrigation method has the following advantages :

- (i) In the furrow irrigation, water contacts only 1/5 to 1/2 of the land surface, thereby reducing puddling and crusting of the soil. Evaporation losses are also reduced.
- (ii) Earlier cultivation is possible in heavy soil and may be adapted to use without erosion on a wide range of natural slope by carrying furrows across a sloping field rather than down the slope.
- (iii) It is specially suitable for those crops (like maize etc.) that are injured by contact with water.
- (iv) Labour requirements in land preparation and irrigation are very much reduced.
- (v) There is no wastage of land in field ditches.

Corrugation Method

Furrows and corrugations are essentially similar—in both the cases water is conveyed to the plant through long, small-capacity channelets dug at regular intervals. While furrows are channelets of a comparatively large cross-section and are used on flat slopes, corrugations are channelets of smaller cross-sections and run down the predominant slope. Thus, corrugations are small furrows and are used for grain and other forage crops such as alfalfa. The corrugations are advantageous when the available irrigation streams are small and also for land of uneven topography.

Design Features for furrows irrigation

The discharge in the furrows (also called *furrow stream*) should be such that it does not cause soil erosion in the upper reaches and at the same time uniform irrigation is obtained. The maximum *non-erosive* flow rate (q_m) in a furrow naturally depends upon the slope (s) of the furrows and is estimated by the following empirical expression:

$$q_m = \frac{0.6}{s} \quad \dots(1) \quad \dots(2.16)$$

where s = slope of furrow, expressed as percent
and q_m = max. non-erosive discharge in lps.

Water is fed to the furrows at the above rate, to start with, till the water reaches the lower end of the furrows. After that, the stream size (i.e. the discharge) is reduced, the reduction being known as *cut back* and the reduced stream is known as *cut-back stream*. The cut-back stream is allowed to flow in the furrow till the required amount of water is applied. If q is the discharge in the cut-back stream, the *average depth of water application* (d) applied during an irrigation of duration t is given by

$$d = \frac{3600 q \cdot t}{WL} \quad \dots(2.17)$$

where d = average depth of water applied (mm) during irrigation.

t = duration of irrigation, in hours.

q = discharge in the furrow, in litres/second.

L = length of furrow in metres.

W = spacing of furrows in metres.

Determination of Infiltration Characteristics

The determination of infiltration characteristics, such as infiltration rate and time required to infiltrate the given depth of water for irrigation is done by two methods: (1) inflow-outflow method, and (2) ponding method. Out of these, the inflow-outflow method gives better results and is commonly followed.

For the inflow-outflow method, a 30 m long reach is picked up and uniform stream of water is applied to three consecutive furrows consisting of the central furrow and two outer furrows, known as *guard furrows*. The inflow and outflow rates are measured in the central furrow, for the reach of 30 m. If q_1 and q_2 are the inflow and outflow rates, in litres per minute at any instant of time t minutes after the start of the test, the quantity of water absorbed per unit time (i.e. infiltration rate) is given by

$$I = (q_1 - q_2) \text{ lpm, per 30 m length of furrows} \quad \dots(1)$$

If the values of I are obtained for different values of time t , and plotted on a log-log scale (i.e. I versus t), a straight line is obtained which can be expressed by the equation :

$$I = k t^n \quad \dots(2) \quad \dots(2.18)$$

in which k and n are constants.

The accumulated infiltration (V) in a given time T is given by

$$V = \int_0^T I dt = \int_0^T k t^n dt = \frac{k T^{n+1}}{(n+1)} \text{ litres} \quad \dots(3) \quad \dots(2.19)$$

However, if d is the total depth of infiltration in mm, we have

$$V = \frac{d}{1000} \times W \times 30 \text{ m}^3 = \frac{d}{1000} \times 30 W \times 1000 \text{ litres} = 30 d \cdot W \quad \dots(4)$$

Equating (3) and (4), we get

$$30 d W = \frac{k T^{n+1}}{n+1}$$

$$\text{From which } d = \frac{k T^{n+1}}{30 W (n+1)} \quad \dots(5) \quad \dots(2.20)$$

Also, the total time T to infiltrate the desired depth d is given by

$$T = \left[\frac{30 d W (n+1)}{k} \right]^{1/(n+1)} \quad \dots(6) \quad \dots(2.21)$$

Computation of instantaneous rate of infiltration

Let i = instantaneous rate of infiltration, in mm/hour

$$\text{Then } I = \left(\frac{i}{1000} \times 30 W \right) \text{ m}^3/\text{hour}$$

$$\text{But from observation, } I = (q_1 - q_2) \text{ lpm} = \frac{q_1 - q_2}{1000} \times 60 \text{ m}^3/\text{hour}$$

$$\text{Hence } \frac{i}{1000} \times 30 W = \frac{q_1 - q_2}{1000} \times 60$$

$$\text{or } i = \frac{2(q_1 - q_2)}{W} \text{ mm/hour} \quad \dots(7) \quad \dots(2.22)$$

Example 2.5. During an inflow-outflow test for an irrigation furrow 100 m long, following observations were taken for time of advance :

Distance (m)	Time of advance t_d (min.)	Distance (m)	Time of advance t_d (min.)
0	0	90	88
30	23	120	127
60	52		

The stream size used during the test was 24.5 lpm. The infiltration characteristics for the furrow can be expressed by the following equation :

$$I = 38 t^{-0.6} \text{ lpm per 30 m length}$$

Irrigation was continued with original stream size for 16 minutes after the water reached the end of the furrow and there after the stream size was cut back. Compute (a) size of cut back stream, (b) the time of irrigation and average depth of water applied if the crop needs 37.5 mm of water. Assume furrow spacing of 1 m.

Solution : 1. Determination of size of cut-back stream

At any observation station along the furrow, the time of ponding (t_d) is evidently equal to the total time of irrigation T ($= 127 + 16 = 143$ min.) minus the time of advance t_L . Hence the time of ponding (t_d) at every station can be computed and entered in column (3) of the Table given below. Similarly, the furrow infiltration rate (I) is given by Eq. 2.18, expressed as below as per data given :

$$I = 38 t^{-0.6} \text{ per 30 m length (where } t = t_d)$$

Hence the value of furrow infiltration can be computed for every observation station, corresponding to t_d value (column 3) of that station and tabulated in column (4) of the Table. Column (5) of the Table gives the average infiltration for the stretch

between consecutive stations while column (6) gives the cumulative infiltration for the stretch between the stations.

Distance (m)	Time of advance t_L (min.)	Time of ponding $t_d = T - T_L$	Furrow infiltration I (lpm/30 m)	Average infiltration I_{av} (lpm/30 m)	Cumulative infiltration I_c (lpm)
(1)	(2)	(3)	(4)	(5)	(6)
0	0	143	3.34		
				3.49	3.49
30	23	120	3.64		
				3.90	7.39
60	52	91	4.17		
				4.75	12.14
90	88	55	5.33		
				7.55	19.69
120	127	16	9.77		

From the above Table, we find that cumulative infiltration rate for the entire furrow is 19.69 lpm. Hence the size of the cut-back stream should be equal to 19.69 lpm or slightly less than this.

2. Time required for putting 37.5 mm depth of water

From Eq. 2.21, putting $d = 37.5$ mm, $W = 1$ m; $n = -0.49$ and $k = 38$, we get

$$T = \left(\frac{30 d W (n + 1)}{k} \right)^{1/(n+1)} = \left[\frac{30 \times 37.5 \times 1 \times (-0.49 + 1)}{38} \right]^{1/(-0.49 - 1)} = 205 \text{ min.}$$

3. Average depth of water applied

The average depth of water may be calculated by taking the size of stream as 24.5 lpm for the first 143 min. and taking the size of cutback stream as 19.69 lpm for the remaining period of $205 - 143 = 62$ minutes.

$$d_{av} = \left(\frac{24.5 \times 143 + 19.69 \times 62}{1000} \right) \frac{1}{120 \times 1} \times 1000 = 39.4 \text{ mm}$$

However, let q be the size of cut back stream to have 37.5 mm average depth of water application. In that case

$$d_{av} = 37.5 = \left(\frac{24.5 \times 143 + q \times 62}{1000} \right) \frac{1}{120 \times 1} \times 1000$$

From which, we get $q = 16.07$ lpm

Hence maximum size of cut back stream is 19.69 lpm and the minimum size of cut back stream is 16.07 lpm.

Example 2.6. Furrows, 100 m long and 1 m apart, having longitudinal slope of 0.3 percent are initially irrigated by a non-erosive stream for 80 minutes. The stream size is then reduced to 40% and continued for another 35 minutes. Determine the average depth of irrigation.

Solution : Given $L = 100$ m; $W = 1$ m ; $s = 0.3\%$

From Eq. 2.6, $q_m = \frac{0.6}{s} = \frac{0.6}{0.3} = 2 \text{ lps}$

Cut back discharge $q = 0.4 q_m = 0.4 \times 2 = 0.8 \text{ lps}$

Hence average depth of water applied

$$= \frac{2 \times 80 \times 60 + 0.8 \times 35 \times 60}{1000} \times \frac{1}{100 \times 1} \times 1000 = 112.8 \text{ mm}$$

2.10. CONTOUR FARMING

Contour farming is practised in hilly areas having steep slopes with quickly falling contour. Contour farming is the practice of conducting field operations, such as ploughing, planting and cultivating land, across the slope rather than up and downhill. The area is divided into longitudinal curved plots, the bunds of the plots following the contours. The irrigation water, stored in some depression higher up, flows between the bunds. Before starting contour farming on land which is not terraced or banded, contour guide lines are established first. On average hill sides, the first contour should ordinarily be laid about 1.2 to 1.5 m vertically below the top of the hill. Additional contour lines are located with the same spacing as the first.

The design of contour bunds involves the selection of the vertical and horizontal interval and determining dimensions of the bund cross-section. Table 2.1 gives Gadkary's recommendations for the spacing of contour bunds on different land slopes. Table 2.2 gives Ghumare's recommendations for the dimensions of contour bunds of various types of soils.

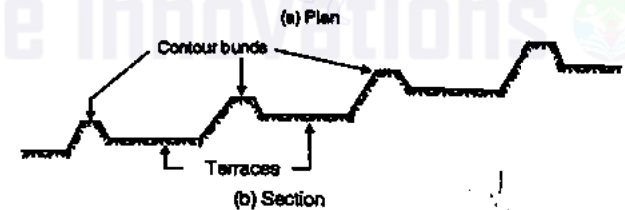
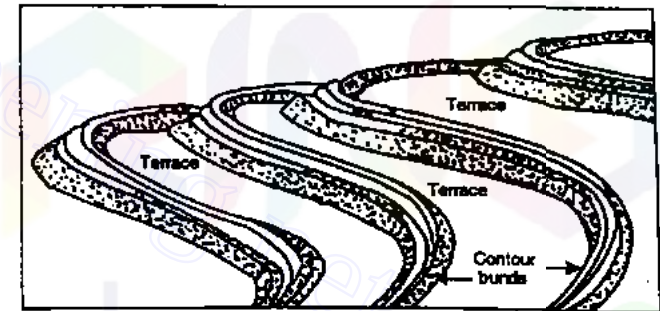


FIG. 2.11. CONTOUR FARMING

TABLE 2.1. SPACING OF CONTOUR BUNDS

Slope of land (%)	Vertical interval (m)	Horizontal distance (m)
0 to 1	1.05	105
1 to $1\frac{1}{2}$	1.20	98
$1\frac{1}{2}$ to 2	1.35	70
2 to 3	1.50	60
3 to 4	1.65	52

TABLE 2.2. SPECIFICATION FOR BUND CROSS-SECTION

	Depth of soil	Base width (m)	Top width (m)	Height (m)	Side Slopes
1.	Shallow soils (7.5 to 22.5 cm)	2.67	0.38	0.75	$1\frac{1}{2} : 1$
2.	Medium soil (22.5 to 45 cm)	3.12	0.60	0.85	$1\frac{1}{2} : 1$
3.	Medium deep soil (45 to 90 cm)	4.25	0.90	0.90	2 : 1

2.11. SUB-SURFACE IRRIGATION

The sub-surface irrigation method consists of supplying water directly to the root zone of the crop. The favourable conditions for the sub-surface irrigation practice are:

1. Impervious sub-soil at reasonable depth (2 to 3 m) or existence of high water table.
2. Permeable soil such as loam or sandy loam in the root zone of the soil.
3. Uniform topographic conditions.
4. Moderate slopes.
5. Good quality irrigation water.

If all these favourable conditions are fulfilled and if proper precautions are taken to prevent alkali accumulation or excess water-logging, the method results in economical use of water, high crop yield and low labour cost in preparing the irrigation plots.

Subsurface irrigation may be of two classes :

- (i) Natural sub-irrigation and
- (ii) Artificial sub-irrigation

Natural sub-irrigation

In this system, water is supplied to the root zone of the plants by controlling the level of local water table. Such a high level of water table in the area may be available due to water seeping from earthen canals, drains, rivers etc. In order to ensure the requisite supply of water to the root zone, it is essential to maintain the desired water level by artificial means. For this purpose, water is supplied to a series of ditches half to one metre deep and 25 to 50 cm wide having vertical sides. These ditches are spaced from 15 to 100 m apart depending upon the permeability of subsoil and topography of land. The depth of the ditches may vary from 0.3 to 1 m and the width may vary from 0.3 to 0.5 m. These channels have relatively flat slopes.

Water flows at a slow rate and seeps into the ground to maintain the water table at a height such that water from the capillary fringe is available to the crops. Proper drainage of excess water is permitted, either naturally or with the other drainage works to prevent water logging of the fields. Sometimes the upward capillary water flow from shallow water table may produce saline and alkali conditions and may make the land less productive. Under such circumstances, the sub-surface irrigation method has to be discontinued and irrigation has to be resorted to by sprinkling method.

Artificial sub-irrigation

This method consists of supplying water directly to the root zone of crops through a net work of buried perforated pipes. Water is made to pass under pressure, through these underground perforated pipes. This method is suitable only for those soils formations which have high horizontal permeability to permit free lateral movement though the root zone of the crops and low vertical permeability so that deep percolation losses are minimised. The pipe are buried 0.3 to 0.4 m deep, so that cultivation operations are not hindered, and are spaced 0.4 to 0.5 m horizontally, for uniform distribution of water.

2.12. SPRINKLER IRRIGATION

The sprinkler method consists of applying the water in the form of a spray, somewhat as in ordinary rain, as is done in the garden lawn sprinkling. The greatest advantage of sprinkler irrigation is its adaptabilities to use under conditions where surface irrigation methods are not efficient. This method is more useful where :

- (i) The land cannot be prepared for surface methods
- (ii) Slopes are excessive
- (iii) Topography is irregular
- (iv) Soil is erosive
- (v) Soil is excessively permeable or impermeable
- (vi) Depth of soil is shallow over gravel or sand.

In this system, the cost of land preparation and permanent water delivery system of channels or conduits is less. However, there is large initial investment in the purchase of the pumping and sprinkling equipment.

Sprinkler system can be classified under three heads :

- (i) Permanent system,
- (ii) Semi-permanent system, and
- (iii) Portable system.

Earlier, stationary over-head perforated pipe installations were used. However, with the introduction of light weight steel pipes and quick couplers, portable sprinkler system were developed. In the permanent system, pipes are permanently buried in such a way that they do not interfere with tillage operations. In the semi-permanent system, the main lines are buried while the laterals are portable. Portable system have both portable mains lines and laterals. These systems are designed to be moved from around the farm from field to field. A pump usually lifts the water from the source, pushes it through the distribution system and through the sprinkler nozzle on the sprinkler heads mounted on rising pipes attached to the laterals. Turbine and horizontal centrifugal

pumps are used. Sprinkler system usually is composed of perforated pipes or revolving head sprinklers and may be 'high pressure' (2.1 kg/cm²) or 'low pressure' (1.4 kg/cm²) system. Generally, a perforated pipe system operates on the low pressure where as the revolving head sprinklers operate in both ranges depending on the type of rotary head used.

Type of sprinklers

Sprinklers may be of three types : (i) fixed nozzle pipe system (ii) perforated pipe system and (iii) rotating sprinklers system. In the *fixed nozzle pipe system* lateral pipes, supported on row of posts, are installed parallel to each other at a distance of 15 m apart. These pipes have a line of small holes drilled at top at regular interval along their length, and small nozzles are fitted in these holes. The spray of water is developed through these nozzles. By turning the pipes through 135°, the entire width of 15 metres between the pipe lines can be irrigated. In the *perforated pipe system*, the laterals, having perforations drilled along the sides and top, are laid on the ground parallel to each other at distances of 6 to 15 m. Sprays of water are developed from both the sides and top of the pipe. However, the more commonly used *rotating sprinkler system* consists of single-nozzle or twin-nozzle rotating sprinkler mounted on a body which is rotated slowly about its vertical axis by the action of deflecting vane connected to it. The spray of water developed through the nozzles sprinkles over a circular area of land around each sprinkler. The rotating sprinklers are mounted on the riser pipes and are located just above the crop to be irrigated, the height of riser pipes being dependent on the maximum height of crops. Such riser pipes fitted with sprinklers, are fixed at regular intervals along the length of the laterals. The spacing of the sprinklers is so selected that area of spread from each sprinkler overlap with the spread from the adjacent sprinkler, so that uniform application of water is achieved. The percentage overlap depends on wind velocity. Normally, the spacing between the sprinklers varies from 65% to 30% of the diameter of water spread area, for no wind condition to high wind condition (> 13 km/hour) respectively.

The discharge, required through each rotating sprinkler is given by the following expression :

$$q = \frac{S_l \times S_m \times I}{3600}$$

where q = discharge required from each sprinkler (litres/sec.)

S_l = spacing of sprinklers along the laterals (m)

S_m = spacing of laterals along mains (m)

I = optimum water application rate (mm/hour)

Advantages of Sprinkler Irrigation

- (i) Erosion can be controlled
- (ii) Uniform application for water is possible.
- (iii) Irrigation is better controlled; light irrigation is possible for seedling and plants which are young.
- (iv) Land preparation is not required. Labour cost is reduced. More land is available for cropping, as borders and ditches are not required. Surface run off is eliminated.

- (v) Small streams of irrigation water can be used efficiently.
- (vi) Time and amount of fertilizers can be controlled for application.
- (vii) Crop damage from frost can be reduced.
- (viii) It is a stand-by drainage pumping plant.

Limitations

- (i) Wind may distort sprinkling pattern.
- (ii) A constant water supply is needed for commercial use of equipment.
- (iii) Water must be clean and free from sand etc.
- (iv) The power requirement is high.
- (v) Heavy soil with poor intake cannot be irrigated efficiently.

Efficiency : If η_a is the water application efficiency of the plant, W_r is the irrigation water stored in root zone and W_p = water pumped into the system, then

$$\eta_a = \frac{W_r}{W_p} \times 100.$$

About 80 % irrigation efficiency is possible with sprinkler irrigation particularly in semi-arid and humid areas. The efficiency decreases by 5 % for each 7.5 km per hour of wind.

2.13. DRIP IRRIGATION

In drip irrigation, also known as *trickle irrigation*, water is applied in the form of drops directly near the base of the plant. Water is conveyed through a system of flexible pipe lines, operating at low pressure, and is applied to the plants through drip nozzles. This technique is also known as 'feeding bottle' technique where by the soil is maintained in the moist congenial form by keeping the soil-water-air proportions in the optimum range. Drip irrigation limits the water supplied for consumptive use of the plant by maintaining a minimum soil moisture, equal to the field capacity, thereby maximising the saving. The system permits the fine control on the application of moisture and nutrients at stated frequencies.

The method of drip irrigation was first introduced in Israel, but is now practiced in many countries of the World. Along with irrigation water, nutrients (fertilizer solution) are also fed to the system. Water is first filtered so that the impurities may not clog the fine holes of the drippers. The whole arrangement consists of the following components:

- (i) a pump to lift water.
- (ii) a head tank to store the water and to maintain a pressure head of 5 to 7 m.

(iii) *Central distribution system* which filters the water, adds nutrients (or fertilizer solution) and regulates the pressure and amount of water to be applied.

(iv) *Mains and secondary lines* made of polyethylene, polyvinyl chloride or a alkathylene material. The diameter of the piping may vary from 20 mm to 40 mm. A water meter may be fixed at the beginning of the mains.

(v) *Trickle lines* consisting of 10 to 20 mm dia. PVC pipes with perforations at a distance equal to the spacing of the crops. The trickle lines are fitted to the secondary lines at a distance equal to the row to row spacing of the crops, which

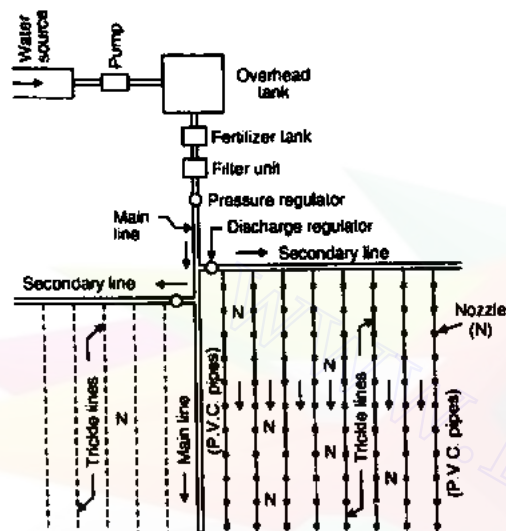


FIG. 2.12 ARRANGEMENT FOR DRIP IRRIGATION SYSTEM.

may vary between 60 cm to 90 cm, a more common value being 75 cm for the most of the crops. The spacing of the nozzles, to be fitted in the perforations of the trickle lines, depends upon the planting distance along a row, and this distance may vary between 30 cm to 50 cm.

(vi) *Plastic nozzles* having perforations attached to the trickle line. The perforations are so designed that water leaves the nozzle at a very slow rate, usually ranging between 2 to 10 litres per hour, depending upon the irrigation requirements of the crop.

The whole field is divided into suitable plots. A secondary line is provided for each such plot, and a number of trickle lines are connected to each secondary line. A discharge regulator is provided at the beginning of each secondary line, and its capacity is fixed in accordance with the size and number of nozzles used. The automatic valve at the head is so adjusted to deliver the desired quantity of water and the irrigation terminates automatically after this amount is discharged. The drip irrigation system is suitable to practically all types of crops, except paddy which requires standing water. It is also suitable to all climatic conditions and to all types of soils. However, it is most suited to coarse sandy formations.

Advantages Of Drip Irrigation System

1. **Less requirement of irrigation water.** The evaporation and percolation losses, commonly associated with surface irrigation method are reduced.

2. **Water supply at optimum level.** An optimum water level, equal to the field capacity can always be maintained, by the water flow controls available in this method. Soil is maintained in most congenial form.

3. **Water logging avoided.** Because of check over deep percolation losses, the chances of water logging are all together removed.

4. **High yield.** Because of proper control over the factors governing soil-water-crop relationships, higher yield is obtained.

5. **Cultivation of cash crops.** The method is specially suited to cash crops such as vegetables, cotton, tobacco and orchard products.

6. **No over-irrigation.** In surface methods of water application, over-irrigation is sometimes done to provide sufficient moisture at deficient area. This is avoided in this method since water is supplied to the root zone directly for each crop.

7. **Variation in application rate.** It is possible to vary the application rate, throughout the crop period. When the crop is infant, application rates can be easily increased.

8. **Weed control.** The nuisance of weeds commonly associated with surface application methods is avoided. This will, in turn, result in increase in crop production, and saving in farm application.

9. **Increase in net irrigable area.** In surface application, a lot of area is wasted in the construction of field channels etc. As water is carried through small diameter pipes, there is considerable saving in land.

10. **Nutrients preservation.** Soil nutrients are preserved because of reduction in deep percolation losses. Moreover, feeding of water enriched with fertilizers results in proper maintenance and preservation of nutrients at optimum level.

11. **Effective pest control.** Insect and pest combating chemicals can be directly conducted into the root zone.

12. **Reduced labour cost.** The labour cost, otherwise associated with the maintenance of field channels, furrows etc., is reduced.

13. **No soil erosion.** The problems of surface soil erosion, associated with surface application methods, are practically absent in this method.

14. **Suitability for saline soils.** The saline, alkaline or brackish soils otherwise unsuitable, can be utilized by cultivation by the drip irrigation method.

15. **Maintenance of high surface temperature.** In surface application method, soil temperature is lowered because of water spread over the entire area. However, in this method, entire soil surface is not wetted. Hence high soil temperature is maintained which will be useful in hastening the ripening or maturity of the crop.

16. **Suitable for any topography.** No land levelling is required, and is suitable even for sloping and undulating terrain.

Disadvantages of Drip Irrigation System

(i) **High initial cost.** The initial cost of various components such as pumping unit, filter units, pipelines, nozzles etc. is very high.

(ii) **Danger of blockade of nozzles.** The nozzle holes are very small in size (0.5 to 1 mm dia.). Hence they are very frequently clogged, if the water has high degree of total dissolved salts which cannot be removed by filtration.

(iii) *Change in spacing of nozzles* due to change in the crops may result in frequent replacement of trickle lines.

(iv) *Shallow root depth* of the crops, specially for fruit trees etc., may result in instability of the crop or tree which may topple during high winds.

2.14. EXAMPLES FROM COMPETITIVE EXAMINATIONS

Example 2.7. For border strip method of irrigation, determine the time required to irrigate strip of land of 0.04 hectare in area from tube well with a discharge of 0.02 cumec. The infiltration capacity of the soil may be taken as 5 cm/hour and the average depth of flow on the field as 10 cm. Also, determine the maximum area that can be irrigated from this tube well. (Engg. services Exam. 2000)

Solution

This example is similar to Example 2.1 of the book.

$$Q = 0.02 \text{ cumec} = 0.02 \times 3600 \text{ m}^3/\text{hour} = 0.0072 \text{ hectare-m/hour}$$

$$y = 10 \text{ cm} = 0.1 \text{ m.}$$

$$I = 5 \text{ cm/hour} = 0.05 \text{ m/hour}$$

$$A = 0.04 \text{ hectare}$$

$$t = 2.303 \frac{y}{I} \log_{10} \frac{Q}{Q - IA} = \frac{2.303 \times 0.1}{0.05} \log_{10} \frac{0.0072}{0.0072 - 0.05 \times 0.04}$$

$$= 0.651 \text{ hours} = 39.06 \text{ min.}$$

$$\text{Maximum area that can be irrigated} = \frac{Q}{I} = \frac{0.0072}{0.05} = 0.144 \text{ ha}$$

PROBLEMS

1. Discuss in brief various methods of surface irrigation.
2. What do you understand by contour farming? Compare it with wild flooding method.
3. Write a note on sub-surface irrigation, stating clearly the conditions under which this method is suitable.
4. Describe border strip method of irrigation. Derive the expression for the time required to cover a given area by this method, for a given rate of discharge and the rate of infiltration of water in the soil.
5. Write a note on sprinkler method of irrigation.
6. Compare surface irrigation with sub-surface irrigation.

Water Requirements of Crops

3.1. FUNCTIONS OF IRRIGATION WATER

The functions of soil moisture in plant growth are very important. Water and nutrients are the two most important requirements of the crop. Following are the main functions of irrigation water :

- (1) It acts as a solvent for the nutrients. Water forms the solution of the nutrients, and this solution is absorbed by the roots. Thus, water acts as the nutrient carrier.
- (2) The irrigation water supplies moisture which is essential for the life of bacteria beneficial to the plant growth.
- (3) Irrigation water supplies moisture which is essential for the chemical action within the plant leading to its growth.
- (4) Some salts present in soil react to produce nourishing food products only in the presence of water.
- (5) Water cools the soil and the atmosphere, and thus makes more favourable environment for healthy plant growth.
- (6) Irrigation water, with controlled supplies, washes out or dilutes salts in the soil.
- (7) It reduces the hazard of soil piping.
- (8) It softens the tillage pans.

3.2. QUALITY OF IRRIGATION WATER

A good irrigation water is the one which performs the above mentioned functions without any side effects which retard the plant growth. Irrigation water may be said to be unsatisfactory for its intended use if it contains (1) chemicals toxic to plants or the persons using plant as food, (2) chemicals which react with the soil to produce unsatisfactory moisture characteristics and (3) bacteria injurious to persons or animals eating plants irrigated with the water.

Impurities in irrigation water

The quality of irrigation water depends upon various types of impurities present in water, the following being the prominent ones :

1. Concentration of sediments in water
2. Total concentration of soluble salts (known as TDS)
3. Proportion of sodium ions to other cations

4. Concentration of toxic elements such as boron concentration
5. Concentration of bicarbonate in relation to the concentration of calcium and magnesium.
6. Bacterial concentration.

The effect of concentration of sediments transported by irrigation water depends upon the type of soil of irrigation fields. If the field contains sandy soil, its fertility is improved by the deposition of sediments carried by irrigation water. However, if the sediment has been derived from eroded areas, it may reduce the fertility and decrease the permeability of soil. High concentration of salts in water may be harmful in the long run. Similarly, if water contains high concentration of sodium ions, the irrigation soils become plastic and sticky when wet and become prone to form clods and they crust on drying. Similarly high percentage of toxic elements may be harmful to certain plants such as nuts, citrus fruits and deciduous plants. High concentration of bicarbonate ions may result in precipitation of calcium and magnesium bicarbonates from the soil solution, increasing the relative proportion of sodium ions and causing sodium hazards. Bacterial contamination may not be directly harmful to the plants as such, but it may be injurious to persons or animals eating these plants.

Classification of irrigation water

Irrigation water may be classified on the following basis :

- (i) Classification based on total concentration of soluble salts
- (ii) Classification based on sodium concentration
- (iii) Classification

based on electrical conductivity (EC), total salt content (TDS), sodium concentration (ESP), boron concentration etc.

1. Classification based on total concentration of soluble salts

Irrigation water may contain various types of salts such as sodium, calcium, magnesium and potassium etc. A high concentration of these salts may prove to be injurious to the crops. The salt content of irrigation water is usually expressed by one of the following ways :

- (i) Parts per million (ppm) or milligram per litre (mg/l), both units being equal.

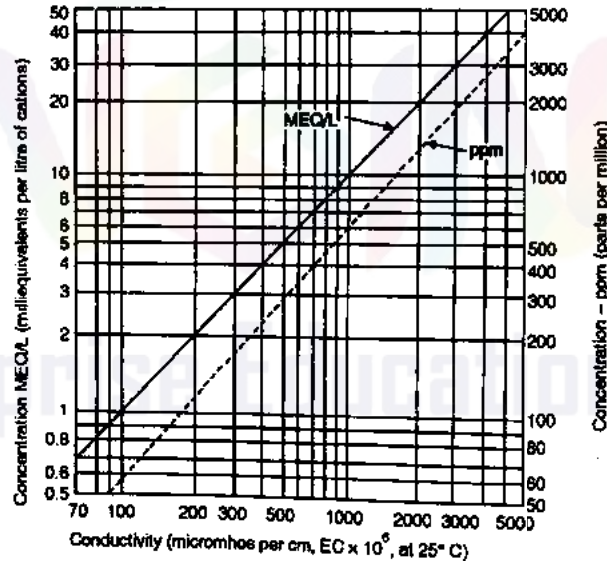


FIG. 3.1. RELATION BETWEEN CONCENTRATION AND ELECTRICAL CONDUCTIVITY.

- (ii) Milli equivalents per litre (MEQ/L).

- (iii) Electrical conductivity, expressed in micromhos per centimetre ($EC \times 10^6$).

Fig. 3.1 shows the relationship between the three.

Salts of sodium, calcium, magnesium and potassium may prove to be injurious in irrigation water. In excessive quantity, these salts reduce the osmotic activity of the plants, thus preventing the absorption of nutrients from the soil. In addition, they may have indirect chemical effects on the metabolism of the plant and may reduce soil permeability. The salinity concentration of soil solution (C_s) can be determined from the following formula :

$$C_s = \frac{C.Q}{Q - (C_u - P_{eff})} \quad \dots(3.1 a)$$

where C = concentration of salt in irrigation water

Q = total quantity of water applied to the soil

C_u = consumptive use of water

P_{eff} = useful rainfall.

Based on the salt concentration or electrical conductivity, U.S.D.A. classifies irrigation water in four types : Low conductivity water (C1), Medium conductivity water (C2), High conductivity water (C3) and very high conductivity water (C4) as indicated in Table 3.1.

TABLE 3.1 CLASSIFICATION OF IRRIGATION WATER BASED ON SALT CONCENTRATION

S.N.	Types of water	Suitability for irrigation
1.	Low salinity water (C1) Conductivity between 100 to 250 micro-mhos/cm at 25°C.	Suitable for all types of crops and all kinds of soils. Permissible under normal irrigation practices except in soil of extremely low permeability.
2.	Medium salinity water (C2) Conductivity between 250 to 270 micro-mhos/cm at 25°C.	Can be used, if a moderate amount of leaching occurs. Normal salt tolerant plants can be grown without much salinity control.
3.	High salinity water (C3) Conductivity between 750 to 2250 micro-mhos/cm at 25°C.	Unsuitable for soil with restricted drainage. Only high-salt tolerant plants can be grown.
4.	Very high salinity (C4) Conductivity more than 2250 micro-mhos/cm at 25°C.	Unsuitable for irrigation.

2. Classification Based on Sodium Concentration : Irrigation water having a higher sodium percentage will, after some time, give rise to a soil having a large percentage of replaceable sodium in the colloid. Such a soil is often known as *black alkali*. The percentage of sodium, also known as exchangeable sodium percentage (ESP), is determined from the following equation:

$$\text{Percentage sodium} = \text{ESP} = \frac{100 \text{ Na}^+}{\text{Ca}^{++} + \text{Mg}^{++} + \text{Na}^+ + \text{K}^+} \quad \dots(3.1 b)$$

If the percentage of sodium is more, the aggregation of soil grains break down, and the soil becomes less permeable and of proper tilth. Even on sandy soils with good drainage, water of 85% sodium or higher (given by equation 3.1 b) are likely

to make soils impermeable after prolonged use. With constant irrigation with high sodium water, the soil becomes plastic and sticky when wet and forms clods and crust on drying.

Irrigation water is classified on sodium concentration, on the basis of a factor called *Sodium-Absorption Ratio* (SAR), given by the following equation :

$$SAR = \frac{Na^+}{\sqrt{\frac{Ca^{++} + Mg^{++}}{2}}} \quad \dots(3.1c)$$

Based on SAR value, water is classified into four types : (i) *Low sodium water* (S1), (ii) *Medium sodium water*, (S2), (iii) *High sodium water* (S3), (iv) *Very high sodium water* (S4). The characteristics of the four types of irrigation water are given in Table 3.2.

TABLE 3.2. CLASSIFICATION OF IRRIGATION WATER BASED ON SODIUM ABSORPTION RATIO (SAR)

S.No.	Type of water	Suitability
1.	Low sodium water (S1) SAR : 0 to 10	Suitable for all types of crops and all types of soils, except for those crops which are highly sensitive to sodium.
2.	Medium sodium water (S2) SAR : 10 to 18	Suitable for coarse textured of organic soil with good permeability. Relatively unsuitable in fine textured soils.
3.	High sodium water (S3) SAR : 18 to 26	Harmful for almost all types of soils. Requires good drainage, High leaching, gypsum addition.
4.	Very high sodium water (S4) SAR above 26	Unsuitable for irrigation.

3. Classification based on EC, TDS, Sodium concentration, boron concentration etc : Table 3.3 gives an alternative classification of irrigation water based on the following four factors : (i) electrical conductivity, (ii) total salt concentration, (iii) % sodium (given by Eq. 3.1a) and (iv) Boron concentration (ppm).

TABLE 3.3. CLASSIFICATION BASED ON EC, TDS, ESP AND BORON CONCENTRATION : STANDARDS OF IRRIGATION WATER

Water class	Electrical conductivity (EC $\times 10^3$)	Total salt content TDS (ppm)	Exchangeable sodium percentage (ESP) (%)	Boron concentration (ppm)	Chloride concentration (ppm)	Sulphate concentration (ppm)
1.	0 - 1000	0 - 700	60	0.0 to 0.5	0 - 142	0 - 192
2.	1000 - 3000	700 - 2000	60 - 75	0.5 - 2.0	142 - 355	192 - 480
3.	over 3000	over 2000	over 75	over 2.0	over 355	over 480

Boron Concentration : The boron content of water is of great importance for many crops. Sugarcane may tolerate large concentration of boron in the irrigation water while other crops such as beans, ground-nuts etc. are very sensitive to an excess of boron. Boron concentration of more than 2 ppm is harmful to most of the crops.

Class 1 waters are considered to be excellent to good for most plants. Class 2 waters are considered as good to injurious, probably harmful to more sensitive crops. Class 3 waters are considered unsuitable under most conditions.

Example 3.1. An irrigation water has the following characteristics :

1. Concentration of Na^+ : 24 milli-equivalents per litre
 2. Concentration of Ca^{++} : 3.6 milli-equivalents per litre.
 3. Concentration of Mg^{++} : 2 milli-equivalents per litre.
 4. Electrical conductivity : 180 micro-mhos at 25° C
- (a) Classify the above irrigation water.
(b) Explain the problems that may arise due to use of such water in fine textured soils
(c) Suggest the remedial measures to reduce these problems.

Solution :

$$(a) \quad SAR = \frac{Na^+}{\sqrt{\frac{Ca^{++} + Mg^{++}}{2}}} = \frac{24}{\sqrt{\frac{3.6 + 2}{2}}} = 14.34$$

From Table 3.2, we find that for SAR = 14.34, the water falls into the S2 class (medium sodium water)

Also, EC = 180 micro-mhos at 25° C. Hence from Table 3.1, the water falls into C1 class (low salinity water)

Hence the water falls into C1-S2 class

(b) From Table 3.2, we observe that such water is not suitable for fine textured soils, as it may cause the following problems :

1. Soil becomes impermeable after prolonged use.
2. There is reduction in proper tilth of soil.
3. Soil becomes plastic and sticky when wet.
4. Soil forms clods and crusts on drying.

5. The pH of soil increases towards that of alkaline soil, tending to become black alkali after prolonged use.

(c) The sodium hazards of irrigation water can be reduced by addition of powdered gypsum ($CaSO_4$) either to water or to soil.

3.3. TYPES OF SOILS

Soil furnishes the following for the plant life :

- (i) Anchor for its roots.
- (ii) Water for its transpiration.
- (iii) Minerals for its nutrition.
- (iv) Oxygen for its metabolism.

Soils may be classified in the following five ways :

(I) *Classification according to age of formation :*

In this class, soils are of three types :

- (a) *Youthful* : Youthful soil is fully pervious.
- (b) *Mature* : Mature soil has low permeability.
- (c) *Senile* : A senile soil has little or no productivity. The soil becomes hard and of very low permeability.

(II) Classification according to geological process of formation :

(a) **Residual soils** : They are formed by disintegration of rock in place under various actions.

(b) **Alluvial soils** : They are formed by deposition by water borne material.

(c) **Eolian soils** : They are formed by deposition by wind action.

(d) **Colluvial soils** : They are formed by deposition by rainwash below foot hills.

(e) **Glacial soils** : They are formed by transportation and deposition by glaciers.

(f) **Volcanic ash** : They are ash deposits due to volcanic eruptions.

(g) **Soils of aggradation** : These are accumulating soils.

(h) **Soils of degradation** : These are the soils which are continuously zoning out.

(i) **Pan, clay pan** : These are impervious hard layers formed due to compaction or cementation by CaCO_3 , Fe_2O_3 , silica, etc.

(III) Classification according to content of calcium, aluminium and iron salts :

(a) **Ped-o-cal** : Soil rich in calcium carbonate.

(b) **Ped-al-fer** : Soil rich in aluminium or iron salts.

(c) **Humus** : Soil rich in organic salts.

IV Classification based on EC, ESP and pH : Saline and alkaline soils

Based on electrical conductivity (EC), exchangeable sodium percentage (ESP) and pH value soils are classed as *saline alkaline* or *saline-alkaline* as indicated in Table 3.4.

TABLE 3.4. CLASSIFICATION OF SALINE AND ALKALINE SOILS

S.N.	Classification	Electrical conductivity (EC) in micro-mhos/cm	ESP	pH value
1.	Saline soil (white alkali)	> 4000	< 15	≤ 8.5
2.	Alkaline soil (non saline alkali or sodic soil or black alkali)	< 4000	> 15	8.5 to 10.0
3.	Saline - alkaline soil	> 4000	> 15	< 8.5

Such soils are formed from normal non-saline soils through the accumulation of salts applied from irrigation water or by upward moving ground water or by a combination of both these effects. These salts usually appear in the form of a white efflorescent crust on the surface of the soil. These soils are not suitable for cultivation, unless these are reclaimed by adopting suitable methods.

(V) Soil classification on regional basis :

(a) Regions of autogeneous soil development.

(b) Regions of balanced decomposition.

(c) Regions of excessive denudation.

(d) Regions of eroded colian accumulation.

(VI) Soil classification on the basis of particle size : Textural classification

A composite soil can be classified on the basis of particle sizes, depending upon the percentage of sand, silt and clay content. Soil texture refers to composition of the

soil reflected by the particle size, shape and gradation. According to the international soil classification, the range of particles for each type of soil is given in Table 3.5.

TABLE 3.5 INTERNATIONAL SOIL CLASSIFICATION

Type of soil	Range of particle diameter (mm)
1. Coarse sand	2 to 0.2
2. Fine sand	0.2 to 0.02
3. Silt	0.02 to 0.002
4. Clay	< 0.002

Fig. 3.2 gives the *textural classification* chart for following twelve main *textural* classes of soils given by United States Department of Agriculture (USDA):

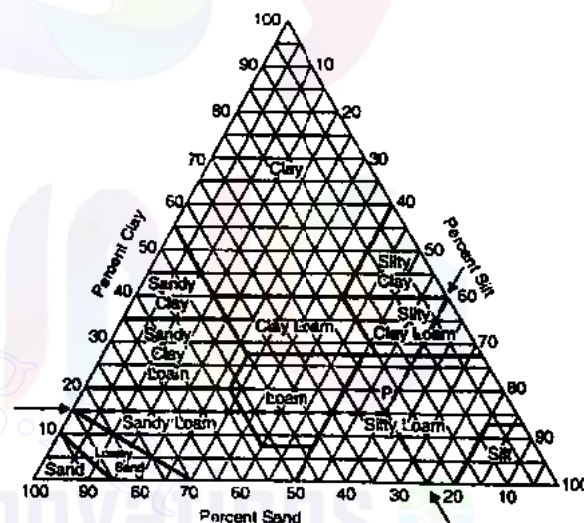
1. Sand	5. Loamy sand	9. Sandy clay
2. Silt	6. Sandy loam	10. Clay loam
3. Clay	7. Silty loam	11. Silty clay loam
4. Loam	8. Sandy clay loam	12. Silty clay

For example : Let the percent of sand, silt and clay in a soil sample be 25%, 60% and 15% respectively, determined as per Table 3.5. In Fig. 3.2, lines are drawn according to these percentages, parallel to the arrows indicated in the figure. We find that point of intersection (P) of these three lines (shown dotted) falls in the sector of silty loam. Hence the soil is *silty loam* as per USDA classification system.

Indian soils : Following are some of the principal Indian soils:

(1) **Alluvial soil** : Alluvial soils are formed by deposition under water. They are found in the Indo-Gangetic plains. Alluvial soils are very fertile. They absorb fair percentage of rainfall and act as a good source of ground water reservoir.

(2) **Red soil** : Red soil is a residual soil which has been left in place as a result of decay of the underlying parent rock, concealing the parent rock below the

**FIG. 3.2 SOIL TEXTURAL CLASSIFICATION CHART BY USDA**

ground surface. Red soils are light textured, porous and friable. They have a varying fertility and have low content of soluble salts.

(3) **Laterite soil** : Laterite soils are also residual soils. They are usually porous and well drained, but lack common nutrients.

(4) **Black soil** . The texture of black soils varies from clay to loam. They develop cracks on account of drying by heat or sun. It has generally good drainage, but where heavy type of clay called chopan, forms the substrata, the soil is likely to be water logged. The soil is quite fertile if adequate rainfall is available.

(5) **Clayey soil** : They are potentially rich soils but lacks drainage.

(6) **Sandy soil** : These are found in deserts of some parts of Punjab and greater part of Rajasthan. The soil has high pH value, low organic content and free drainage.

(7) **Humus**.

(8) **Peats and marshes**.

Suitability of Soil for Irrigation

The soil should be carefully studied with regard to the following:

(a) Size of soil particles.	(b) Compactness.
(c) Depth.	(d) Organic matter content.
(e) Position of water table.	

All the above aspects influence the depth of available water that the irrigation can store in the root zone of soil in a single application of water and hence influence the required frequency of watering. The soil conditions essential for good plant growth—originally worked out by Morgan and his associates in 1938 are given below:

- (1) Suitability for the agricultural implements required for most efficient production.
- (2) Effective resistance to destructive soil erosion or soil depletion under cropping system.
- (3) Adequate moisture storage to meet the water requirements of crops under normal rainfall or irrigation.
- (4) Adequate aeration to a suitable depth to permit the development of a favourable root system for the mature plant.
- (5) Available plant nutrients sufficient for profitable yields.
- (6) Freedom from adverse chemical conditions such as harmful concentration of soluble constituents and from other special soil conditions that favour the development of organisms parasitic to the crop.

3.4. PREPARATION OF LAND FOR IRRIGATION

The land hitherto uncultivated should be properly prepared before irrigation water is applied upon it. This can be done as follows :

- (i) Removal of thick jungle, bushes etc., from the raw land. The roots of the trees should be extracted and burnt. The land should thereafter be properly deamed.
- (ii) The land should be made level. High patches should be scraped and depression filled. Unless this is done, water will fill the depression and duty may be too low.

(iii) The land should be provided with regular slope in the direction of falling gradient.

(iv) The land should be divided into suitable plots by small levees according to the method of irrigation to be practiced.

(v) Permanent supply ditches and water courses should be excavated at regular spacings which facilitate proper distribution of the water to the entire field.

(vi) A drain ditch which carries the waste water should also be excavated.

(vii) Proper drainage measures should be adopted where the danger of water logging may become eminent after the introduction of canal irrigation.

3.5. CLASSES AND AVAILABILITY OF SOIL WATER

Water present in the soil may be classified under three heads (Fig. 3.3) :

1. Hygroscopic water,
2. Capillary water,
3. Gravitational water.

When an oven-dried sample is kept open in the atmosphere, it absorbs some amount of water from the atmosphere. This is known as *hygroscopic water*, and is not capable of movement by the gravity or capillary forces.

Capillary water is that part in excess of hygroscopic water which exists in the pore space of the soil by molecular attraction.

Gravitational water is that part in excess of hygroscopic and capillary water which will move out of the soil if favourable drainage is provided.

Water may also be classified as *unavailable*, *available* and *superfluous*. This classification is based on the availability of soil water to plants.

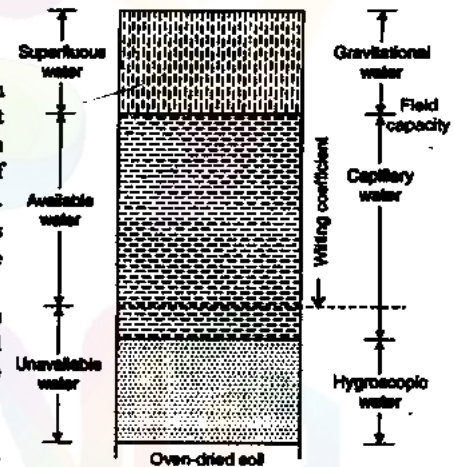


FIG. 3.3 CLASSES OF SOIL WATER.

Soil Moisture tension : The force per unit area that must be exerted in order to extract water from the soil is known as *soil moisture tension* and is expressed in terms of atmosphere (atm). It is also known as *Capillary potential*, *Capillary tension* or *force of suction*. Soil moisture tension is inversely proportional to moisture content of a soil of given texture and structure. It is measured in the laboratory with the help of various instruments such as *centrifuge*, *tensiometer* etc.

Soil moisture stress : Soil moisture stress is defined as the sum of the soil moisture tension and osmotic pressure of soil solution. Osmotic pressure is the increase in the force (or tension) caused by the salts present in the soil solution. The growth of plants is a function of both soil moisture tension as well as the osmotic pressure, and hence is a function of soil moisture stress. The osmotic pressure of the soil solution should be maintained as low as possible by *controlled leaching*, thus maintaining the

soil moisture stress at the root zone in a range that will provide adequate moisture to the roots of the plants and is discussed below.

Soil moisture constants : Various soil moisture constants are described below.

1. Saturation Capacity

This can also be called as *maximum moisture holding capacity* or *total capacity* and is the amount of water required to fill all the pore spaces between soil particles by replacing all air held in pore spaces. It is the upper limit of possible moisture content. When the porosity of a soil is known, the saturation capacity can be expressed as equivalent cm of water per metre of soil depth. So, if the porosity is 50 % by volume, the moisture in each meter of saturated soil is equivalent to depth of 50 cm the field surface.

2. Field Capacity

The field capacity is the moisture content of the soil after free drainage has removed most of the gravity water. The concept of field capacity is extremely useful in arriving at the amount of water available in the soil for plant use. Most of the gravitational water drains through the soil before it can be used consumptively by plants.

3. Permanent Wilting Point

Permanent wilting point or the *wilting coefficient* is that water content at which plants can no longer extract sufficient water from the soil for its growth. The permanent wilting point is at the lower end of the available moisture range. If the plant does not get sufficient water to meet its needs, it will wilt permanently. A plant is considered to be permanently wilted when it will not regain its turgidity even after being placed in a saturated atmosphere where little or no consumptive water use occurs. However, it will regain its turgidity if water is added to the soil.

The permanent wilting point depends upon the rate of water used by the plant, the depth of the root zone and the water holding capacity of the soil. Permanent wilting point is higher in a hot climate than in a cool climate. At the permanent wilting point, the soil moisture tension may range between 7 to 40 atm, depending upon rate of consumptive use, type of crop, soil texture, salt content of soil etc. However, recent studies indicate that the wilting point is closely indicated by the moisture retained against a tension of 15 atm. At the permanent wilting point, the films of water around the soil particles are held so tightly that the plant roots cannot extract enough moisture at sufficient rate to satisfy transpiration requirements of the plant, resulting in wilting of the plant. As an approximation, the permanent wilting percentage can be estimated by dividing the field capacity by a factor varying from 2.0 to 2.4, depending upon the amount of silt in the soil. For most of the soils, wilting coefficient is about 150 % of the hygroscopic water.

Temporary Wilting

Temporary wilting may sometimes take place during hot windy day, but the plant will recover in the cooler portion of the day. No addition of water is required. Thus temporary wilting may take place during the hot summer day, even when soil moisture is higher than the wilting coefficient, because of increased transpiration rates.

Ultimate Wilting

Ultimate wilting is slightly different from permanent wilting. When ultimate wilting occurs, the plant will not regain its turgidity even after the addition of sufficient water to the soil and the plant will die. The soil moisture tension at ultimate wilting point is as high as 80 atm. The *ultimate wilting point* occurs at the hygroscopic water content. Hence the ultimate wilting point is also known as *hygroscopic coefficient*. The *ultimate wilting point* or the *hygroscopic coefficient* is about 2/3 of the permanent wilting point.

4. Available Moisture

The difference in water content of the soil between field capacity and permanent wilting point is known as available water or available moisture.

5. Readily Available Moisture

It is that portion of the available moisture that is most easily extracted by plants, and is approximately 75 % of the available moisture.

The above mentioned soil-moisture constants for different types of soils are shown below in a tabular form (Table 3.6) :

TABLE 3.6

Types of Soil	Wilting Coefficient	Field Capacity	Available water
Sand	4	9	5
Sandy loam	6	14	8
Loam	10	22	12
Clay loam	13	27	14
Silty clay	15	31	16
Clay	17	35	18

6. Moisture Equivalent

This is an artificial moisture property of the soil and is used as an index of the natural properties. It is the percentage of moisture retained in a small sample of wet soil 1 cm deep when subjected to a centrifugal force 1000 times as great as gravity, usually for a period of 30 minutes. Moisture equivalent is used as a single factor to which the properties of soil can be related within reasonable limits. The moisture equivalent roughly equals field capacity for a medium textured soil. The relation between these are as follows :

$$\begin{aligned}\text{Moisture equivalent} &= \text{Field capacity} \\ &= 1.8 \text{ to } 2 \text{ Permanent wilting point} \\ &= 2.7 \text{ Hygroscopic coefficient.}\end{aligned}$$

7. Soil-Moisture Deficiency

Soil-moisture deficiency or field moisture deficiency is the water required to bring the soil moisture content of the soil to its field capacity.

Depth of water stored in root zone and available to plants :

In order to estimate the depth of water stored in the root zone of soil containing water upto field capacity, let d be the depth of root zone (in metres) and F_c be the field capacity (expressed as ratio).

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Let γ_d = dry unit weight of soil.
 γ_w = unit weight of water.

Consider unit area (1 sq. metre) of soil area. Then

$$F_c = \frac{\text{Wt. of water retained in unit area}}{\text{Wt. of soil of unit area}} = \frac{\text{Wt. of water retained in unit area}}{(\gamma_d \times 1 \times d)}$$

$$\therefore \text{Wt. of water retained in unit area} = F_c \cdot \gamma_d \cdot d$$

$$\therefore \text{Depth of water stored (in depth } d) = \frac{F_c \cdot \gamma_d \cdot d}{\gamma_w} \text{ metres} \quad \dots(3.2)$$

A part of this depth of water will be available for evapo-transpiration.

Available moisture depth (d_w) is given by

$$d_w = \frac{\gamma_d \cdot d}{\gamma_w} [\text{Field capacity} - \text{Wilting coefficient}] \quad \dots(3.2 a)$$

$$d_w = S_g \cdot d [F_c - w_c] \quad \dots(3.2 b)$$

where S_g = apparent specific gravity of soil = γ_d / γ_w

F_c = field capacity, expressed as ratio

w_c = wilting coefficient, expressed as ratio

Example 3.2. The root zone of an irrigation soil has dry weight of 15 kN/m^3 and a field capacity of 30%. The root zone depth of a certain crop, having permanent wilting percentage of 8% is 0.8 m. Determine (a) depth of moisture in the root zone at field capacity (b) depth of moisture in the root zone at permanent wilting point, and (c) depth of water available.

Solution

(a) Depth of water in root zone at field capacity, per metre depth of soil

$$= \frac{\gamma_d}{\gamma_w} \times F_c = \frac{15}{9.81} \times 0.3 = 0.459 \text{ m/m} = 459 \text{ mm/m}$$

(b) Depth of water in root zone at permanent wilting point, per m depth of soil:

$$= \frac{\gamma_d}{\gamma_w} \times \text{permanent wilting point} = \frac{15}{9.81} \times 0.08 = 0.122 \text{ m/m} = 122 \text{ mm/m}$$

(c) Depth of water available in root zone

$$= \frac{\gamma_d \cdot d}{\gamma_w} [F_c - \text{permanent wilting point}]$$

$$= \frac{15}{9.81} \times 0.8 [0.3 - 0.08] = 0.269 \text{ m} = 269 \text{ mm}$$

Example 3.3. The field capacity of a certain soil is 15% and the moisture content of the soil before irrigation is 8%. Determine the depth upto which the soil profile will be wetted with an application of 60 mm of water. Take the dry unit weight soil as 15.3 kN/m^3 .

$$\text{Solution : } \gamma_d / \gamma_w = 15.3 / 9.81 = 1.56$$

$$\text{Depth of water applied} = \frac{\gamma_d \cdot d}{\gamma_w} [\text{Field capacity} - \text{moisture content before irrigation}]$$

$$\therefore 60 = 1.56 d [0.15 - 0.08]$$

$$\text{From which } d \approx 550 \text{ mm}$$

Example 3.4. The field capacity of a certain soil is 20% and its apparent specific gravity is 1.6. Before applying irrigation water, a wet sample of soil was taken and its mass was found as 150 gm. The same sample weighed as 136 gm after oven drying. Determine the depth of water that must be applied to irrigate the soil to a depth of 0.9 m.

$$\text{Solution : } \gamma_d / \gamma_w = 1.6$$

$$\text{Moisture content before irrigation} = \frac{150 - 136}{136} \times 100 = 10.29\%$$

$\therefore$ Depth of water required to bring the moisture up to field capacity.

$$= S_g \cdot d [F_c - \text{moisture content before irrigation}]$$

$$= 1.6 \times 0.9 [0.2 - 0.1029] \approx 0.14 \text{ m} = 140 \text{ mm}$$

Example 3.5. Find the field capacity of a soil for the following data :

Root zone depth = 2 m

Existing water content = 5%

Dry density of soil = 15 kN/m^3

Water applied to the soil = 500 m^3

Water loss due to evaporation and deep percolation = 10%

Area of plot = 1000 sq.metres.

Solution.

$$\text{Total water applied} = 500 \text{ m}^3$$

$$\text{Loss of water} = 10\%$$

$$\therefore \text{Water used in the soil} = 500 \times \frac{90}{100} = 450 \text{ m}^3$$

$$\text{Weight of water used} = 450 \times 9.81 = 4414.5 \text{ kN}$$

$$\text{Total dry weight of the soil} = 1000 \times 2 \times 15 = 30000 \text{ kN}$$

$$\therefore \% \text{ of water added} = \frac{4414.5}{30000} \times 100 = 14.72\%$$

$$\therefore \text{New water content} = 5\% + 14.72\% = 19.72\%$$

$$\therefore \text{Field capacity} = 19.72\%$$

Example 3.6. A loam soil has field capacity of 22% and wilting coefficient of 10%. The dry unit weight of soil is 15 kN/m^3 . If the root zone depth is 70 cm, determine the storage capacity of the soil. Irrigation water is applied when moisture content falls to 14%. If the water application efficiency is 75%, determine the water depth required to be applied in the field.

Solution :

$$\text{Given } \gamma_d = 15 \text{ kN/m}^3. \text{ Also, } \gamma_w = 9.81 \text{ kN/m}^3$$

Maximum storage capacity = Available moisture

$$= \frac{\gamma_d \cdot d}{\gamma_w} [\text{Field capacity} - \text{Wilting coefficient}]$$

$$= \frac{15 \times 0.70}{9.31} [0.22 - 0.10] \text{ m} = 0.128 \text{ m} = 12.8 \text{ cm}$$

Depth of irrigation water

$$= \frac{\gamma_d \cdot d}{\gamma_w} [\text{Field capacity} - w] \text{ where } w = 14\%$$

$$= \frac{1.5 \times 0.7}{9.81} [0.22 - 0.14] \text{ m} = 0.086 \text{ m} = 86 \text{ mm}$$

Field irrigation requirement

$$= \frac{86}{0.75} = 115 \text{ mm.}$$

3.6. LIMITING SOIL MOISTURE CONDITIONS

It is essential to maintain *readily available water* in the soil if crops are to make satisfactory growth. The plant growth may be retarded if the soil-moisture is either deficient or excessive. If the soil moisture is only slightly more than the wilting coefficient, the plant must expend extra energy to obtain it and the plant will not grow healthy. Similarly, excessive flooding fills the soil pores with water, thus driving out air. Since air is essential to satisfactory plant growth, excessive water supply retards plant growth. The *optimum moisture percentage* is thus that moisture corresponding to which optimum growth of plant takes place, as shown in Fig. 3.4.

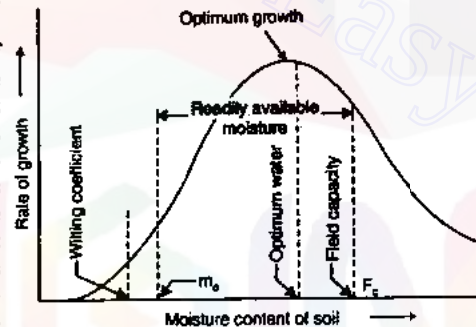


FIG. 3.4. LIMITING SOIL MOISTURE CONDITIONS.

3.7. DEPTH AND FREQUENCY OF IRRIGATION

As explained earlier, *available moisture* is the moisture between wilting point and the field capacity. The *readily available moisture* is that moisture which is easily extracted by the plants, and is approximately 75 % of the available moisture. At any time, therefore, the moisture content in the soil should be between the field capacity and the lower limit (m_0), of the readily available moisture, as shown in Fig. 3.5.

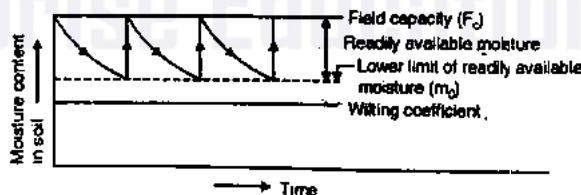


FIG. 3.5. FREQUENCY OF IRRIGATION

Thus, m_0 is the maximum level upto which the soil moisture may be allowed to be depleted in the root zone without fall in the yield.

When watering is done, the amount of water supplied should be such that the water content is equal to the field capacity. Water will be gradually utilized consumptively by plants after the water application, and the soil moisture will start falling. When the water content in the soil reaches the value m_0 , fresh doses of irrigation may be done so that water content is again raised to the field capacity of the soil.

The frequency of irrigation is controlled by the amount of available water contained in the root zone of the soil and the consumptive use rate. If d is the root zone depth (metres), F_c is the field capacity and m_0 is the lower limit of readily available moisture content, the depth of water d_w to be given during each watering is found from the following expression,

$$d_w = \frac{\gamma_d}{\gamma_w} \cdot d [F_c - m_0] \text{ metres} \quad \dots(3.3)$$

(where γ_d is the dry unit weight of soil and γ_w is the unit weight of water).

Both F_c and m_0 are the moisture contents to be expressed as ratio.

If C_u is the daily consumptive use rate, frequency of watering f_w is given by

$$f_w = \frac{d_w}{C_u} \text{ days} \quad \dots(3.4)$$

Time required to irrigate a certain area

Let t be the time required to apply the desired water depth d_w to bring the water level in the soil from m_0 to the field capacity F_c , over an irrigation field of area A . If q is the discharge in the field channel, in cumecs, we have

$$t = \frac{A \cdot d_w}{q} \text{ seconds} \quad \dots(3.5)$$

where A is the area in square metres and d_w is the depth of water to be applied in metres. If, however, area A is expressed in hectares, we have

$$t = \frac{d_w}{q} \cdot A \times 10^4 \text{ seconds} = \frac{2.778 A d_w}{q} \text{ hours} \quad \dots(3.5 a)$$

Example 3.7. After how many days will you supply water to soil (clay loam) in order to ensure efficient irrigation of the given crop, if

- Field capacity of soil = 27%
- Permanent wilting point = 14%
- Dry density of soil = 15 kN/m^3
- Effective depth of root zone = 75 cm
- Daily consumptive use of water for the given crop = 11 mm.

Solution. Available moisture = Field capacity - permanent wilting point
 $= 27 - 14 = 13\%$

Let the readily available moisture be 80% of the available moisture.

$\therefore$ Readily available moisture = $13 \times 0.8 = 10.4\%$

$m_0 = 27 - 10.4 = 16.6\%$

Hence when irrigation water is applied, moisture is raised from 16.6% to 27%.

∴ Depth of water stored in root zone, during each watering

$$= \frac{Y_d \cdot d}{Y_w} \text{ [Field capacity} - m_0] = \frac{Y_d}{Y_w} d [0.27 - 0.166] \text{ metres}$$

$$= \frac{1.5 \times 0.75}{9.81} \times 0.104 = 0.119 \text{ m} = 11.9 \text{ cm}$$

Thus, depth of water available for evapo-transpiration = 11.9 cm

Daily consumptive use of water = 1.1 cm

$$\therefore \text{Watering frequency} = \frac{11.9}{1.1} = 10.82 \text{ days} \approx 10 \text{ days}$$

Hence, water should be applied after every 10 days.

Example 3.8. An irrigation field 40 m wide 250 m long has soil which has apparent specific gravity equal to 1.56 and field capacity equal to 22%. The depth of root zone is 0.6 m. If the irrigation is started when 70% of the available moisture has been used, compute (a) net depth of irrigation water required, and (b) time required to irrigate the field if the discharge in the field channel is 20 litres per second.

Solution : (a) Given : $F_c = 22\% = 0.22$; $S_g = 1.56$; $d = 0.6 \text{ m}$

Moisture content before start of irrigation, $m = 0.3 \times 0.22 = 0.066$

$$\therefore d_w = S_g d [F_c - m] = 1.56 \times 0.6 [0.22 - 0.066] = 0.144 \text{ m} = 144 \text{ mm}$$

(b) Discharge $q = 20 \text{ lps} = 20 \times 10^{-3} \times 3600 \text{ m}^3 / \text{hour} = 72 \text{ m}^3 / \text{hour}$

$$\text{Hence from Eq. 3.5, } t = \frac{Ad_w}{q} = \frac{40 \times 250 \times 0.144}{72} = 20 \text{ hours}$$

3.8. PRINCIPAL CROPS AND CROP SEASONS

Crops can be classified in the following ways :

1. Agricultural classification
2. Classification based on crop seasons
3. Classification based on irrigation requirement
1. Agricultural classification

This consists of the following types of crops.

(i) **Field crops :** Such as wheat, rice, maize, barley, oats, great millet, spiked millet, gram, pulses etc.

(ii) **Commercial crops :** Such as sugar cane, cotton, tobacco, hemp, sugar beat etc.

(iii) **Oil seed crops :** Such as mustard, ground nut, sesame, linseed, castor etc.

(iv) **Horticulture crops :** Consisting of various fruit crops, various vegetable crops and flower crops

(v) **Plantation crops :** Such as tea, coffee, cacao, coconut, rubber etc.

(vi) **Forage crops :** Such as fodder, grass etc.

(vii) **Miscellaneous crops :** Such as medicinal crops, aromatic crops, sericulture crops, condiments and spices.

2. Classification based on crop seasons

Based on crop season, crops are classified as follows.

(i) **Rabi crops or Winter Crops :** These crops are sown in autumn (or October) and are harvested in spring (or March). Various crops that fall under this category are : gram, wheat, barley, peas, mustard, tobacco, linseed, potato etc.

(ii) **Kharif crops or monsoon crops :** These crops are sown by the beginning of the southwest monsoon and are harvested in autumn. These consist of rice, maize, spiked millet, great millet, pulses, ground nut etc.

(iii) **Perennial crops :** These are the crops that require water for irrigation throughout the year. Examples of perennial crops are : sugar cane, fruits, vegetables etc.

(iv) **Eight months crops :** These crops, such as cotton, require irrigation water for 8 months.

3. Classification based on irrigation requirements : Based on irrigation requirements, crops can be classified as (i) dry crops, (ii) wet crops and (iii) garden crops. Dry crops are the one which do not require water for irrigation; only rain water is sufficient for their growth. Wet crops are those which cannot grow without irrigation. Garden crops require irrigation throughout the year.

Principal crops of India

Table 3.7 gives the principal crops of India, along with crop season and average depth of water required (i.e. delta).

TABLE 3.7. PRINCIPAL CROPS OF INDIA

Crop	Sowing time	Harvesting time	Average Delta (cm)
(A) Kharif Crops			
Rice	June-July	Oct.-Nov.	120
Makai (Maize)	June-July	Sept.-Oct.	45
Bajra (Spiked Millet)	June-Aug.	Sept.-Oct.	30
Jowar (Great Millet)	June-July	Oct.-Nov.	30
Pulses (Mung, Moth, Urd)	June-July	Nov.-Dec.	30
Ground Nut	May		45
(B) Rabi Crops			
Gram	Sept.-Oct.	March-April	30
Wheat	Oct.-Nov.	March-April	40
Barley	Oct.-Nov.	March-April	45
Peas	Oct.-Nov.	March-April	50
Mustard	Oct.	Feb.-March	45
Tobacco	Feb.-March	June	60
Potato	Oct.	Feb.	75
(C) Eight Months Crops			
Cotton	May-June	Dec.-April	45
(D) Perennial Crops			
Sugar-cane	Feb.-March	Dec. to March	90

Crop Ratio. It is the ratio of the area irrigated in Rabi season to the area irrigated in Kharif season. Since the water requirements of Kharif crops are two to three times more than that required for rabi, the ratio is so kept that almost uniform discharge is required for this purpose.

Overlap Allowance : It may happen sometimes that the crop of some season may overlap some period of the next crop season. When such overlapping takes place, the crops of both the seasons require water simultaneously. Thus the overlap allowance is the extra discharge for this purpose.

3.9. DUTY AND DELTA

Duty represents the irrigating capacity of a unit of water. It is the relation between the area of a crop irrigated and the quantity of irrigation water required during the entire period of the growth of that crop. For example, if 3 cumecs of water supply is required for a crop sown in an area of 5100 hectares, the duty of irrigation water will be $5100/3 = 1700$ hectares/cumec, and the discharge of 3 cumecs will be required throughout the base period.

Delta is the total depth of water required by a crop during the entire period the crop is in the field and is denoted by the symbol Δ . For example, if a crop requires about 12 waterings at an interval of 10 days, and a water depth of 10 cm in every watering, then the delta for that crop will be $12 \times 10 = 120$ cm = 1.20 metres. If the area under that crop is A hectares, the total quantity will be $1.20 \times A = 1.2 A$ hectare-metres in a period of 120 days.

Crop period : Crop period is the time, in days, that a crop takes from the instant of its sowing to that of its harvesting.

Base period : Base period for a crop refers to the whole period of cultivation from the time when irrigation water is first issued for preparation of the ground for planting the crop, to its last watering before harvesting.

Ways of Reckoning Duty

The duty of water is reckoned in the following four ways :

(i) **By the number of hectares that one cumec of water can irrigate during the base period, i.e. 1700 hectares/cumec.**

(ii) **By total depth of water (or Delta), i.e. 1.20 metres.**

(iii) **By number of hectares that can be irrigated by a million cubic metre of stored water.** This system is used for tank irrigation.

(iv) **By the number of hectare metres expended per hectare irrigated.** This is also used in tank irrigation.

For a precise statement of duty by the first method, which is quite common in canal irrigation system, it is necessary to state the following along with the duty figures :

- the base period, and
- the place of measurement of duty.

Thus duty of water for a certain crop is 1700 hectares/cumec at the field, for a base period of 120 days.

Variation of Duty with the Place of its Measurement

The duty varies with the place of its measurement, because of continuous conveyance losses as the water flows. The duty of water goes on increasing as the water flows. For example, let C (Fig. 3.6) be the head of the field, B be the head of the water course or the field channel, and A be the head of the distributary. Let the area of the field be 1700 hectares, and let 1 cumec water be required to be delivered at point C , for the growth of the crop. Thus, the duty at the head of the field will be 1700 hectares/cumec. Assuming the conveyance losses between B and C to be 0.1 cumecs (say), the discharge required at B will be 1.1 cumecs, and hence duty of water measured at B will be $1700/1.1 = 1545$ hectares/cumec only. Again, if the losses between A to B are taken to be equal to 0.2 cumec, the discharge required at the head of the distributary will be $1.1 + 0.2 = 1.3$ cumecs, i.e. if 1.3 cumecs are discharged at A , then 1 cumec will reach at the head of the field. Hence the duty of water at A will be $1700/1.3 = 1308$ hectares/cumec only. Thus, duty at the head of the water course (at B) is lesser than the duty at the head of the field, and is greater than the duty at the head of the distributary. The duty at the head of the water-course is called the **outlet duty**. Thus measurements of duty are taken at four points noted below:

- At the head of main canal — known as **Gross Quantity**.
- At the head of a branch canal — known as **Lateral Quantity**.
- At the outlet of a canal — known as **Outlet Factor**.
- At the head of land to be irrigated — known as **Net Quantity**.

Relation Between Duty and Delta

Let D = duty in hectares/cumec
 Δ = total depth of water supplied (in metres)
 B = base period in days.

(i) If we take a field of area D hectares, water supplied to the field corresponding to the water depth Δ metres will be $= \Delta \times D$ hectare-metres
 $= D \times \Delta \times 10^4$ cubic-metres. ... (1)

(ii) Again for the same field of D hectares, one cumec of water is required to flow during the entire base period. Hence, water supplied to this field
 $= (1) \times (B \times 24 \times 60 \times 60) \text{ m}^3$... (2)

Equating Equations (1) and (2), we get

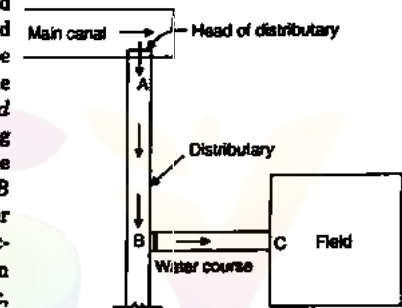


FIG. 3.6

$$D \times \Delta \times 10^4 = B \times 24 \times 60 \times 60$$

$$\text{or } \Delta = \frac{B \times 24 \times 60 \times 60}{D \times 10^4} = 8.64 \frac{B}{D} \text{ metres} \quad \dots(3.6)$$

Note 1 hectare = 10^4 sq. metres
1 cumec-day = 8.64 hectare-metres.

High and Low Duty

Duty is being referred to as being high or low according to the number of hectares/cumec irrigated is large or small.

Gross Commanded Area (G.C.A.)

An area is usually divided into a number of watersheds and drainage valleys. The canal usually runs on the watershed and water can flow from it on both sides, due to gravitational action only up to drainage boundaries. Thus in a particular area lying under the canal system, the irrigation can be done only upto the drainage boundaries. The gross commanded area is thus the total area lying between drainage boundaries which can be commanded or irrigated by a canal system.

Culturable Commanded Area (C.C.A.)

The gross commanded area also contains unfertile barren land, alkaline soil, local ponds, villages and other areas as habitation. These areas are known as unculturable areas. The remaining area on which crops can be grown satisfactorily is known as culturable commanded area (C.C.A.).

Thus $G.C.A. = C.C.A. + \text{Unculturable area.}$

The culturable commanded area can further be classified as culturable cultivated area and culturable uncultivated area.

Culturable Cultivated Area

It is the area in which crop is grown at a particular time or crop season.

Culturable Uncultivated Area

It is that area in which crop is not sown in a particular season. Such area is kept under no cultivation due to the following reasons:

1. To increase the fertility of the soil which has been reduced due to intense cultivation.
2. To provide pasture land for animals.
3. The crop to be sown in that land has a different crop season.
4. To protect the land from the possible danger of water-logging.

Intensity of Irrigation

Intensity of irrigation is defined as the percentage of culturable commanded area proposed to be irrigated during either a crop season or during a year. For example, if C.C.A. of an irrigation field is 120 hectares, out of which 90 hectares of the land is cultivated during kharif season and 60 hectares of the land is cultivated during rabi season, the intensity of irrigation during Kharif season will be $= (90/120) 100 = 75\%$ and the intensity of irrigation during rabi crop will be $(60/120) \times 100 = 50\%$. However, the

yearly intensity of irrigation will be equal to the sum of the two, i.e. equal to $75 + 50 = 120\%$. Thus yearly intensity of irrigation can be more than 100%.

3.10. FACTORS AFFECTING DUTY

The duty of water of canal system depends upon a variety of the factors. The principal factors are :

1. Methods and system of irrigation ;
2. Mode of applying water to the crops ;
3. Method of cultivation ;
4. Time and frequency of tilling ;
5. Type of the crop ;
6. Base period of the crop ;
7. Climatic conditions of the area;
8. Quality of water ;
9. Method of assessment of irrigation method ;
10. Canal conditions ;
11. Character of soil and sub-soil of the canal ;
12. Character of soil and sub-soil of the irrigation fields.

1. Methods and Systems of Irrigation

In the perennial irrigation system, soil is continuously kept moist, and hence water required for initial saturation is less. Also, due to the shallow depth of the water table, deep percolation losses are less. In the inundation irrigation, there is wasteful use of water. Hence, the perennial irrigation system has more duty than the inundation irrigation. The flow irrigation system has lower duty due to the conveyance losses in the network of the canals, while the lift irrigation system has higher duty because the commanded area of each well is very near to it. Tank irrigation gives high duty due to rigid control.

2. Mode of Applying Water

The flood irrigation system has lesser duty than the furrow system. Sub-irrigation system gives still higher duty. The ring basin irrigation and uncontrolled flooding give less duty.

3. Method of Cultivation

If the land is properly ploughed and made quite loose before irrigating, the soil will have high water retention capacity in its unsaturated zone. Thus, the number of waterings can be reduced, increasing the duty. The old and conventional methods of cultivation gives less duty in comparison to the modern methods.

4. Time and frequency of Tilling

Frequency of cultivation reduces the loss of moisture through weeds. Soil structure affects the plant growth to a very great extent. A good structure (i.e. the good arrangement of soil particles in relation to one another) is called good *tilth* of soils. When the soil is in good *tilth*, evaporation losses from the surface of soil is less, soil becomes properly aerated, and hence the yield of crop is also better.

5. Type of the crop

The duty varies from crop to crop.

6. Base Period of the Crop

If the base period of the crop is more, the amount of water required will be high, hence duty will be low and *vice-versa*.

7. Climatic Conditions of the Area

The climatic conditions which affect the duty are : (i) temperature, (ii) wind, (iii) humidity, and (iv) rainfall. Due to high temperature and wind, evaporation losses will be more, and duty will be less. A humid atmosphere reduces the losses. Rainfall during the crop-period will reduce the irrigation-water requirement, and the duty will thus be higher. In this context there are two popular terms : (i) *Duty inclusive of rainfall*, i.e. duty be considered by taking rainfall into account ; (ii) *Duty exclusive of rainfall*, that is the duty has been considered by not considering the rainfall in the area.

8. Quality of water

If the harmful salt-content and alkali content of the water is more, water will have to be applied liberally so that the salts are leached off. This will, in turn, reduce the duty. More fertilizing matter in water will cause less consumption of water, and increase duty.

9. Method of Assessment of water

Volumetric method of assessment always leads to a higher duty. This is because the farmer will use water economically. If, however, the method of assessment is based on the area under cultivation, the farmer will have a tendency to use more water, and the duty will be low.

10. Canal Conditions

In an earthen canal, the seepage and percolation losses will be high, resulting in the low duty. If, however, the canal is lined, the losses will be less, and duty will be high. If the high canal is so aligned that the irrigated areas are concentrated along it, the duty will be higher. The dispersion of the irrigated areas with respect to the canal tends to reduce the duty.

11. Character of Soil and Sub-soil of the Canal

If the canal is unlined, and if it flows through coarse grained, permeable soils, the seepage and percolation losses will be high. If the canal flows through fine grained soil, such losses will be less, and hence the duty will be higher.

12. Character of Soil and Sub-soil of the Irrigation Field

If the soil and sub-soil of the field is coarse-grained, percolation losses will be high. However, if there is hard pan at depth 1 to 2 metre below surface, the percolation loss reduces. The duty is also affected by the topography of the land. If the field is not level, the lower portions get more water, while higher portions may remain drier. In order to supply water to the higher places, more water will be used, and the duty will be reduced.

3.11. METHODS OF IMPROVING DUTY

When once the various factors affecting duty are properly understood, the duty can be improved by making those factors less effective which tend to reduce the duty.

1. Suitable method of applying water to the crops should be used.
2. The land should be properly ploughed and levelled before sowing the crop. It should be given good tilth.
3. The land should be cultivated frequently, since frequent cultivation reduces loss of moisture specially when the ground water is within capillary reach of ground surface.
4. The canals should be lined. This reduces seepage and percolation losses. Also, water can be conveyed quickly, thus reducing evaporation losses.
5. Parallel canals should be constructed. If there are two canals running side by side, the F.S.L. will be lowered, and the losses will thus be reduced.
6. The idle length of the canal should be reduced.
7. The alignment of the canal either in sandy soil or in fissured rock should be avoided.
8. The canal should be so aligned that the areas to be cultivated are concentrated along it.
9. The source of supply should be such that it gives good quality of water.
10. The rotation of crops must be practiced.
11. Volumetric method of assessment should be used.
12. The farmers must be trained in the proper use of water, so that they apply correct quantity of water at correct timing.
13. The land should be redistributed to the farmers so that they get only as much land as they are capable of managing it.
14. Research stations should be established in various localities to study the soil, the seed and conservation of moisture. The problems concerning the economical use of water should be studied at research stations.
15. The canal administrative staff should be efficient, responsible and honest. The operation of canal system should be such that the farmers both at the head of the canal as well as at the tail end get water as and when they need it.

3.12. SOME DEFINITIONS**1. Kor period and Kor depth**

We have already discussed the base period. The distribution of water during the period is however, not uniform. Crops require maximum water during first watering after the crops have grown few centimetres. During the subsequent watering the quantity of water needed by crops gradually decreases and is least when crop gains maturity. The first watering is known as *kor watering*, and the depth applied is known as *kor depth*. The portion of the base period in which *kor watering* is needed is known as *kor period*. While designing the capacity of a channel, *kor water* must be taken into account since discharge in the canal has to be maximum during this time.

2. **Outlet Factor** : It is defined as the duty at the outlet.

3. **Time Factor**

The time factor of a canal is the ratio of the number of days the canal has actually run to the number of days of irrigation period. For example, if the number of days of irrigation period = 12, and the canal has actually run for 5 days, the time factor will be = $5/12$. A day has a period of 24 hours (i.e. it includes the night).

4. **Capacity Factor**

This is the ratio of the mean supply (discharge) to the full supply of a canal.

5. **Cumec Day**

The quantity of water flowing for one day at the rate of 1 cumec is known as a cumec-day. It is equal to 8.64 hectare-metres.

6. **Paleo**

It is the first watering before sowing the crop. This is done in order to add sufficient moisture to the unsaturated zone of the soil and is required for the initial growth of the crop.

7. **Full Supply Coefficient**

It is defined as the area estimated to be irrigated during the base period divided by the design full supply discharge of the channel at its head during maximum demand. This is also known as *duty on capacity*.

8. **Nominal Duty**

In some states, cultivators have to submit an application for water required for their land under irrigation, to the department in advance of every season. After a permit is granted, they have to pay full amount for the water irrespective of the area they actually irrigate. Nominal duty is, therefore, the ratio of the area of which the permit has been granted for the period divided by the mean supply for the base period.

9. **Open Discharge**

It is the ratio of the number of cumec-days to the number of days the canal has actually been used for irrigation.

10. **Root Zone Depth**

Root zone depth is the maximum depth of soil strata in which the crop spreads its root system and derives water from the soil.

Example 3.9. Find the delta for a crop if the duty for a base period of 110 days is 1400 hectares/cumec.

Solution : Given : $B = 110$ days and $D = 1400$ hectares/cumec

$$\Delta = 8.64 \frac{B}{D} = \frac{8.64 \times 110}{1400} \text{ m} = 0.68 \text{ m} = 68 \text{ cm}$$

Example 3.10. A crop requires a total depth of 92 cm of water for a base period of 120 days. Find the duty of water.

Solution : Given : $B = 120$ days and $\Delta = 92 \text{ cm} = 0.92 \text{ m}$

$$D = \frac{8.64 B}{\Delta} \text{ hectares/cumec} = \frac{8.64 \times 120}{0.92} = 1127 \text{ hectares/cumec.}$$

Example 3.11. An irrigation canal has gross commanded area of 80,000 hectares out of which 85% is culturable irrigable. The intensity of irrigation for Kharif season is 30% and for Rabi season 60%. Find the discharge required at the head of the canal if the duty at its head is 800 hectares/cumecs for Kharif season and 1700 hectares cumec for rabi season.

Solution

Culturable irrigable area = $80000 \times 0.85 = 68000$ hectares

Area under Kharif season = $68000 \times 0.30 = 20400$ hectares.

Area under Rabi season = $68000 \times 0.60 = 40800$ hectares

The ratio of area irrigated in Rabi season to that irrigation in kharif season is known as *crop ratio*. In this example crop ratio is 2. The crop-ratio is so selected that the discharge in the canal during both the seasons may be uniform.

Water required at the head of the canal to irrigate the land under Kharif season
 $= \frac{20400}{800} \text{ cumecs} = 25.5 \text{ cumecs.}$

Water required at the head of canal to irrigate the land under Rabi season
 $= \frac{40800}{1700} \text{ cumecs} = 24 \text{ cumecs}$

Thus the water requirement during Kharif is more than that in Rabi. Hence, the canal must be designed to carry a discharge of 25.5 cumecs. Actually, the canal must be designed to carry the *peak discharge* and not the average discharge calculated based on the average duty figure. The peak requirements may be taken to be equal to 20% to 25% higher than the average requirement, or calculated from the kor period and kor depth of water, if available.

Example 3.12. A water course has culturable commanded area of 2600 hectares, out of which the intensities of irrigation for perennial sugar-cane and rice crops are 20% and 40% respectively. The duty for these crops at the head of water course are 750 hectares/cumec and 1800 hectares/cumec respectively. Find the discharge required at the head of water course if the peak demand is 120% of the average requirement.

Solution

Area under sugar-cane = $2600 \times 0.2 = 520$ hectares.

Area under rice = $2600 \times 0.4 = 1040$ hectares.

Water required for sugar-cane = $\frac{520}{750} = 0.694 \text{ cumec}$

Water required for rice = $\frac{1040}{1800} = 0.578 \text{ cumec.}$

Since sugar-cane is perennial crop, it will require water through-out the year. Hence, the water course must carry a total discharge of $(0.694 + 0.578) = 1.272$ cumecs. The design discharge, to meet the peak demand, will be $1.272 \times 1.2 = 1.53$ cumecs.

Example 3.13. The left branch canal carrying a discharge of 20 cumecs has culturable commanded area of 20,000 hectares. The intensity of Rabi crop is 80 percent,

and the base period is 120 days. The right branch canal carrying discharge of 8 cumecs has culturable commanded area of 12,000 hectares, intensity of irrigation of Rabi crop is 50 percent, and the base period is 120 days. Compare the efficiencies of the two canal systems.

Solution

(a) For the left canal

$$\text{Area under Rabi crop} = 20000 \times 0.8 = 16000 \text{ hectares}$$

$$\text{Discharge} = 20 \text{ cumecs}$$

$$\therefore \text{Duty} = \frac{16000}{20} = 800 \text{ hectares/cumec}$$

(b) For the right canal

$$\text{Area under Rabi crop} = 12000 \times 0.50 = 6000 \text{ hectares}$$

$$\text{Discharge} = 8 \text{ cumecs}$$

$$\therefore \text{Duty} = \frac{6000}{8} = 750 \text{ hectares/cumec.}$$

Since the left canal system has higher duty, it is more efficient.

Example 3.14. A water course has a culturable commanded area of 1200 hectares. The intensity of irrigation for crop A is 40% and for B is 35%, both the crops being Rabi crops. Crop A has a kor period of 20 days and crop B has kor period of 15 days. Calculate the discharge of the water course if the kor depth for crop A is 10 cm and for B it is 16 cm.

Solution : (a) For crop A

$$\text{Area under irrigation} = 1200 \times 0.40 = 480 \text{ hectares}$$

$$\text{Kor period} = b = 20 \text{ days}$$

$$\text{Kor depth} = \delta = 10 \text{ cm} = 0.1 \text{ m}$$

$$\therefore \text{Duty} = \frac{8.64(b)}{\delta} = \frac{8.64 \times 20}{0.1} = 1728 \text{ hectares/cumec}$$

$$\begin{aligned} \text{Hence discharge required} &= \frac{\text{Area under irrigation}}{\text{Outlet factor}} \\ &= \frac{480}{1728} = 0.278 \text{ cumec} \end{aligned}$$

(b) For crop B

$$\text{Area under irrigation} = 1200 \times 0.35 = 420 \text{ hectares}$$

$$\text{Kor period} = b = 15 \text{ days}$$

$$\text{Kor depth} = \delta = 16 \text{ cm} = 0.16 \text{ m}$$

$$\begin{aligned} \therefore \text{Duty or outlet factor} &= \frac{8.64(b)}{\delta} \text{ hectares/cumec} \\ &= \frac{8.64 \times 15}{0.16} = 810 \text{ hectares/cumec} \end{aligned}$$

$$\text{Hence discharge required} = \frac{420}{810} = 0.519 \text{ cumec}$$

Thus, the design discharge of water course

$$= 0.278 + 0.519 = 0.797 \approx 0.8 \text{ cumec.}$$

Example 3.15. A water course commands an irrigated area of 600 hectares. The intensity of irrigation of rice in this area is 60%. The transplantation of rice crop takes 12 days, and total depth of water required by the crop is 50 cm on the field during the transplantation period. During the transplantation period, the useful rain falling on the field is 10 cm. Find the duty of irrigation water for the crop on the field during transplantation, at the head of the field and also at the head of the distributory, assuming losses of water to be 20% in the water course. Also, calculate the discharge required in the water course.

Solution

The rice seed is generally germinated in separate seed beds and the young plants from seed beds are transplanted, being thrust by hand into previously prepared land. The preparation of land for rice cultivation usually consists of flooding the land till water stands several centimetres in depth over it and ploughing it when in this state, so as to thoroughly puddle the surface soil, reducing it to a soft slush. The operation of preparing a plot and transplanting it takes about 10 to 15 days and a very large quantity of water, varying in different soils and localities from about 30 to 60 cm on the field is required for the whole operation.

$$\text{Now} \quad \Delta = \frac{8.64 B}{D}$$

$$\text{Here} \quad B = \text{transplantation period} = 12 \text{ days}$$

$$\Delta = \text{depth of irrigation water actually applied in the field} \\ = 50 - 10 = 40 \text{ cm} = 0.4 \text{ m}$$

$$D = \text{duty of irrigation water, on the field in hectares/cumec.}$$

$$\therefore D = \frac{8.64 B}{\Delta} = \frac{8.64 \times 12}{0.40} = 259.2 \text{ hectares/cumec. (on the field)}$$

Since the losses in the canal are 20%, 1 cumec of water discharge at the head of the water course will be reduced to 0.80 cumec at the head of the field, and hence will irrigate $259.2 \times 0.8 = 207.36$ hectares only. Hence, the duty of water at the head of the water course will be 207.36 hectares/cumec.

$$\text{Now total area under rice plantation} = 600 \times 0.6 = 360 \text{ hectares}$$

$$\therefore \text{Discharge at the head of water course} = \frac{360}{207.36} \approx 1.74 \text{ cumecs.}$$

Example 3.16. Table below gives the necessary data about the crop, their duty and the area under each crop, commanded by a canal taking off from a storage tank. Taking a time factor for the canal to be 13/20, calculate the discharge required at the head to the canal. If the capacity factor is 0.8, determine the design discharge.

IRRIGATION AND WATER POWER ENGINEERING

Crop	Base period (days)	Area (hectares)	Duty at the head of the canal (hectares/cumec)
Sugar-cane	320	850	580
Overlap for sugar-cane in hot weather	90	120	580
Wheat (Rabi)	120	600	1600
Bajri (Monsoon)	120	500	2000
Vegetable (Hot weather)	120	360	600

Solution**Discharge required for crops**

$$\text{Discharge for sugar-cane} = \frac{850}{580} = 1.466 \text{ cumecs}$$

$$\text{Discharge for overlap sugar-cane} = \frac{120}{580} = 0.207 \text{ cumec}$$

$$\text{Discharge for wheat} = \frac{600}{1600} = 0.375 \text{ cumec}$$

$$\text{Discharge for Bajri} = \frac{500}{2000} = 0.25 \text{ cumec}$$

$$\text{Discharge for vegetables} = \frac{360}{600} = 0.60 \text{ cumec}$$

Since sugar-cane has a base period of 320 days, it will require water in Rabi, Monsoon and Hot weather.

$$\therefore \text{Discharge required in Rabi} = 1.466 + 0.375 = 1.841 \text{ cumecs}$$

$$\text{Discharge required in Monsoon} = 1.466 + 0.25 = 1.716 \text{ cumecs}$$

$$\text{Discharge required in Hot Weather} = 1.466 + 0.207 + 0.6 = 2.273 \text{ cumecs}$$

Thus, the maximum demand of 2.273 cumecs is in the hot weather. The time factor is the ratio of the number of days the canal has actually run to the number of days the canal was supposed to run. Here, time factor is 13/20.

$\therefore$ Full supply discharge at the head to canal will be

$$= 2.273 \times \frac{20}{13} = 3.497 \text{ cumecs}$$

Design discharge = Full supply discharge/capacity factor $3.497/0.8 = 4.372 \text{ cumecs}$

Example 3.17. The base period, intensity of irrigation and duty of various crops under a canal system are given in the Table below. Find the reservoir capacity if the canal losses are 20% and reservoir losses are 12%.

Crop	Base period (days)	Duty at the field (hectares/cumec)	Area under the crop (hectares)
Wheat	120	1800	4800
Sugar-cane	360	800	6600
Cotton	200	1400	2400
Rice	120	900	3200
Vegetables	120	700	1400

WATER REQUIREMENTS OF CROPS

Solution**(i) Wheat**

$$\text{Discharge required} = \frac{4800}{1800} \text{ cumecs}$$

$$\therefore \text{Volume of water required} = \frac{4800}{1800} \times 120 = 320 \text{ cumec-days}$$

(ii) Sugar-cane

$$\text{Discharge required} = \frac{5600}{800} = 7 \text{ cumecs}$$

$$\therefore \text{Volume of water required} = 7 \times 360 = 2520 \text{ cumec-days}$$

(iii) Cotton

$$\text{Discharge required} = \frac{2400}{1400} \text{ cumecs}$$

$$\therefore \text{Volume of water required} = \frac{2400}{1400} \times 200 = 343 \text{ cumec-days}$$

(iv) Rice

$$\text{Discharge required} = \frac{3200}{900} \text{ cumecs}$$

$$\therefore \text{Volume of water required} = \frac{3200}{900} \times 120 = 427 \text{ cumec-days}$$

(v) Vegetables

$$\text{Discharge required} = \frac{1400}{700} = 2 \text{ cumecs}$$

$$\therefore \text{Volume of water required} = 2 \times 120 = 240 \text{ cumec-days}$$

Hence, total volume of water required on the field for all crops

$$= 320 + 2520 + 343 + 427 + 240$$

$$= 3850 \text{ cumec-days}$$

$$\text{Now 1 cumec-day} = 1 \text{ cumec flowing for the whole day} = (1 \times 24 \times 60 \times 60) \text{ m}^3$$

$$1 \text{ hectare-metre} = 1 \times 10^4 \text{ m}^3$$

$$\text{Hence 1 cumec-day} = \frac{(1 \times 24 \times 60 \times 60)}{1 \times 10^4} = 8.64 \text{ hectare-metres}$$

$$\therefore \text{Total volume of water required on the field}$$

$$= 3850 \times 8.64 \text{ hectare-metres} = 33264 \text{hec-m}$$

Since losses in the canal system are 20% the volume of water required at the head of the canal

$$= 33264 \times \frac{100}{80} = 41580 \text{hec-m}$$

Allowing 12% reservoir losses, the storage capacity of the reservoir

$$= 41580 \times \frac{100}{88} = 47250 \text{ hectare-metres}$$

Alternative solution

The above example can also be solved by first finding the Delta for each crop and then multiplying the area by Delta to get the total volume of water needed.

$$\Sigma V = \Sigma (\Delta \times A)$$

The calculations are done in tabular form :

Crop (1)	Base period (days) (2)	Duty of the field hectare / cumec (3)	Δ in m $= \frac{8.64 B}{D}$ (4)	Area A in hectares (5)	Volume ($\Delta \times A$) hectare-m (6) = (4) $\times$ (5)
Wheat	120	1800	0.576	4800	2764.8
Sugar-cane	360	800	3.888	5600	21772.8
Cotton	200	1400	1.234	2400	2962.3
Rice	120	900	1.152	3200	3686.4
Vegetable	120	700	1.481	1400	2073.6
Cumulative Volume					33259.9 hect-m

Say, 33260 hect-m as calculated previously.

$$\therefore \text{Capacity of reservoir} = \frac{33260}{0.8 \times 0.88} = 47245 \text{ hectare-metres.}$$

3.13. CONSUMPTIVE USE OF WATER (EVAPO-TRANSPIRATION)

Evapo-transpiration or consumptive use of water by a crop is the depth of water consumed by evaporation and transpiration during crop growth, including water consumed by accompanying weed growth. Water deposited by dew or rainfall, and subsequently evaporating without entering the plant system is part of consumptive use. When the consumptive use of the crop is known, the water use of large units can be calculated.

Evaporation

Evaporation is the transfer of water from the liquid to the vapour state. The rate of evaporation from water surface is proportional to the difference between the vapour pressure at the surface and the vapor pressure in the overlaying air (Dalton's law). When irrigation water is applied by flooding methods, large amounts of water are lost by direct evaporation from soil surface without having passed through the roots, stems and leaves of the plants.

Transpiration and Transpiration Ratio

Transpiration is the process by which plants dissipate water from the surface of their leaves, stalks and trunks in the process of growth. As much as 99% of total water received by a plant through its roots is lost to the atmosphere by this process. Transpiration is associated with photosynthesis of plants; it is therefore, a process of day light hours. Water from soil is taken up by plant roots through a membrane by a process called osmosis. Green cells, called chloroplasts in plant leaf prepare food in presence of sun light and CO₂ and leave water through tiny openings called stomata the density of which may vary from 8000 to 12000 per sq. cm. The transpiration ratio is the ratio of the weight of water transpired by the plant during its growth to the

weight of dry matter produced by the plant exclusive of roots. The average values of transpiration ratio for wheat and rice are 560 and 680 respectively.

Factors Affecting Consumptive Use of Water

1. Evaporation, which depends upon humidity.
2. Mean monthly temperature.
3. Growing season of crop and cropping pattern.
4. Monthly precipitation in the area.
5. Irrigation depth or the depth of water applied for irrigation.
6. Wind velocity in the locality.
7. Soil and topography.
8. Irrigation practices and methods of irrigation.

Potential Evapo-transpiration (PET) and Actual Evapo-transpiration (AET)

We know that evapo-transpiration is the total loss of water from farm land as

evaporation and from plants grown on it as transpiration. If sufficient moisture is always available to completely meet the needs of the plants, the resulting evapo-transpiration is called potential evapo-transpiration (PET). The real evapo-transpiration occurring in a specific situation is called actual evapo-transpiration (AET). At the moisture content in the soil corresponding to field capacity (F), the water supply to the plant is adequate and hence AET will be equal to PET, or in other words, the ratio AET/PET will be equal to 1. With the reduction in the available moisture in the soil, the ratio AET/PET decreases and finally AET will be zero at wilting point. A relation between AET/PET and available moisture can be developed for different types of soils on the basis of experimental results. Fig. 3.7 shows two such curves for sandy soil and clayey soil. For the same AET/PET ratio, sandy soil has more available moisture than clayey soil. In other words, for the same percent of available moisture, the ratio, AET/PET will be less for sandy soil than for clayey soil.

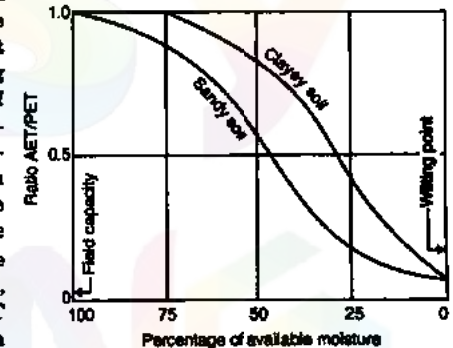


FIG. 3.7. VARIATION OF AET/PET WITH AVAILABLE MOISTURE

A relation between AET/PET and available moisture can be developed for different types of soils on the basis of experimental results. Fig. 3.7 shows two such curves for sandy soil and clayey soil. For the same AET/PET ratio, sandy soil has more available moisture than clayey soil. In other words, for the same percent of available moisture, the ratio, AET/PET will be less for sandy soil than for clayey soil.

(A) DIRECT MEASUREMENT OF CONSUMPTIVE USE

The consumptive use or evapo-transpiration can be measured by five principal methods :

1. Tank and Lysimeter methods.
2. Field experimental plots.
3. Soil moisture studies.

4. Integration method.
5. Inflow and outflow studies for large area.
1. Tank and Lysimeter Method

Tanks are containers set flush with the ground level having an area of 10 m² and 3 m deep. Larger the size of the tank greater is the resemblance to root development. The tank is filled with soil of the field and crop is grown in it. Consumptive use is determined by measuring the quantity of water required to maintain constant moisture conditions within the tank for satisfactory proper growth of the crop. In *Lysimeters*, the bottom is pervious. Consumptive use is the difference of water applied and that draining through pervious bottom and collected in a pan.

2. Field Experimental Plots

This method is more dependable than the tank and lysimeter method. In this method, irrigation water is applied to the selected field experimental plots in such a way that there is neither runoff nor deep percolation. Yield obtained from different fields are plotted against the total water used, and, as basis for arriving at the consumptive use, those yields are selected which appear to be most profitable. It is seen from observations that for every type of crop, the yield increases rapidly with an increase of water used to a certain point, and then decreases with further increase in water. At the 'break in the curve', the amount of water used is considered as the *consumptive use*.

3. Soil Moisture Studies

This method is specially suited to those areas where soil is fairly uniform and ground water is deep enough so that it does not affect the fluctuations in soil moisture within the root zone of the soil. Soil moisture measurements are done before and after each irrigation. The quantity of water extracted per day from soil is computed for each period. A curve is drawn by plotting the rate of use against time and from this curve, *seasonal use* can be determined.

4. Integration Method

In this method, it is necessary to know the division of total area under irrigation crops, natural vegetation, water surface area and bare land area. The integration method is summation of the products of (i) unit consumptive use for each crop times its area, (ii) unit consumptive use of native vegetation times its area, (iii) water surface evaporation times the water surface area, and (iv) evaporation from bare land times its area. Thus, in this method, annual consumptive use for the whole of the area is found, in hectare-metre units.

5. Inflow-Outflow Studies for Large Areas

In this method also, annual consumptive use is found for large areas. If U is the *valley consumptive use*, its value is given by

$$U = (I + P) + (G_1 - G_2) - R \quad \dots(3.7)$$

where, U = Valley consumptive use (in hectare-metre)

I = Total inflow during 12-months year

P = Yearly precipitation on valley floor

G_1 = Ground storage at the beginning of the year

WATER REQUIREMENTS OF CROPS

G_2 = Ground storage at the end of the year
 R = Yearly outflow.

All the above volumes are measured in hectare-metres.

(B) CONSUMPTIVE USE DETERMINATION BY USE OF EQUATIONS

The following are some of the commonly used methods for computation of consumptive use:

1. Blaney-Criddle method.
2. Penman method.
3. Hargreaves class A pan evaporation method.
1. Blaney-Criddle method

Blaney and Criddle (1962) proposed an empirical relation which is largely used by irrigation engineers. Blaney Criddle equation expresses potential evapo-transpiration (consumptive use) in terms of temperature and day time hours. If c_u is *monthly consumptive use*, its value is given by

$$c_u = k \cdot f \quad \dots(3.8 a)$$

where c_u = monthly consumptive use in cm.

k = monthly crop coefficient to be determined from experimental data

f = monthly consumptive use factor given by

$$f = (0.0457 t_m + 0.8128) p \pm \frac{p}{40} [1.8 t + 32] \quad \dots(3.9)$$

t_m = mean monthly temperature in °C

p = monthly percentage of hours of bright sunshine in the year (Table 3.9)

If C_u is the *seasonal consumptive use*, its value is given by

$$C_u = k \Sigma f \quad \dots(3.8 b)$$

However, if the crop coefficient (or crop factor) also varies from month to month, the *total consumptive use* or *seasonal consumptive use* is given by

$$C_u = \Sigma k f \quad \dots(3.8)$$

The value of k can be found from charts for different crops available at various agriculture research stations. However, in absence of these charts, the value of k can be taken from Table 3.8.

TABLE 3.8. MONTHLY CROP COEFFICIENT FACTOR k
 (FOR USE IN BLANEY-CRIDDLE METHOD)

Crop	Rice	Wheat	Maize	Sugarcane	Cotton	Potato	Corn	Vegetables	
								Light	Dense
Value of k	1.1	0.65	0.65	0.9	0.65	0.7	0.75	0.80	1.20
Range of k	0.85 - 1.3	0.5 - 0.75	0.5 - 0.75	0.8 - 1.0	0.5 - 0.9	0.65 - 0.75	0.65 - 0.85	0.7 - 1.0	1.1 - 1.4

TABLE 3.9
MONTHLY DAYTIME (SUNSHINE) PERCENTAGE HOURS (p)

Latitude Degrees	Jan.	Feb.	Mar.	April	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
North												
0	8.50	7.66	8.49	8.21	8.50	8.22	8.50	8.49	8.21	8.50	8.22	8.50
6	8.32	7.57	8.47	8.29	8.65	8.41	8.67	8.60	8.23	8.42	8.07	8.30
10	8.13	7.47	8.45	8.27	8.81	8.60	8.86	8.71	8.25	8.34	7.91	8.10
15	7.94	7.38	8.43	8.44	8.98	8.80	9.06	8.83	8.28	8.26	7.75	7.88
20	7.74	7.25	8.41	8.52	9.15	9.00	9.25	8.96	8.30	8.18	7.58	7.66
25	7.53	7.14	8.39	8.61	9.33	9.23	9.45	9.09	8.32	8.09	7.40	7.42
30	7.30	7.03	8.38	8.72	9.53	9.49	9.67	9.22	8.33	7.99	7.19	7.15
32	7.20	6.97	8.37	8.76	9.62	9.59	9.77	9.27	8.34	7.95	7.11	7.05
34	7.10	6.91	8.36	8.80	9.72	9.70	9.88	9.33	8.36	7.90	7.02	6.92
36	6.99	6.85	8.35	8.85	9.82	9.82	9.99	9.40	8.37	7.85	6.92	6.79
38	6.87	6.79	8.34	8.90	9.92	9.95	10.10	9.47	8.38	7.80	6.82	6.68
40	6.76	6.72	8.33	8.95	10.02	10.08	10.22	9.54	8.39	7.75	6.72	6.52
42	6.63	6.65	8.31	9.00	10.14	10.22	10.35	9.62	8.40	7.69	6.62	6.37
44	6.49	6.58	8.30	9.06	10.26	10.38	10.49	9.70	8.41	7.63	6.49	6.21
46	6.34	6.50	8.29	9.12	10.39	10.54	10.64	9.79	8.42	7.57	6.36	6.04
48	6.17	6.41	8.27	9.18	10.53	10.71	10.80	9.89	8.44	7.51	6.23	5.86
50	5.98	6.30	8.24	9.24	10.68	10.91	10.99	10.00	8.49	7.45	6.10	5.65
South												
0	8.50	7.66	8.49	8.21	8.50	8.22	8.50	8.49	8.21	8.50	8.22	8.50
5	8.68	7.76	8.51	8.15	8.34	8.05	8.33	8.38	8.19	8.56	8.37	8.68
10	8.86	7.87	8.53	8.09	8.18	7.86	8.14	8.27	8.17	8.62	8.53	8.88
15	9.05	7.98	8.55	8.02	8.02	7.65	7.95	8.15	8.15	8.68	8.70	9.10
20	9.24	8.09	8.57	7.94	7.86	7.43	7.76	8.03	8.13	8.76	8.87	9.33
25	9.49	8.21	8.60	7.84	7.66	7.20	7.54	7.90	8.11	8.96	9.04	9.58
30	9.70	8.33	8.62	7.73	7.45	6.96	7.31	7.76	8.07	8.97	9.24	9.85

2. Penman Method

Penman developed a theoretical formula based on principles of both energy budget and mass-transfer approaches to compute potential evapo-transpiration in the following form

$$E_t = \frac{A \cdot H + \alpha E_a}{A + \alpha} \quad \dots(3.10)$$

WATER REQUIREMENTS OF CROPS

where E_t = Evapo-transpiration, mm/day

α = psychrometric constant = 0.49 mm Hg/°C

A = Slope of the curve between saturated vapor pressure (e_s) and temperature at mean air temperature (Table 3.10)

E_a = drying power of air which includes wind velocity and saturation deficit and is given by the relation.

$$E_a = 0.002187 (160 + u_2) (e_s - e_a) \quad \dots(3.11)$$

u_2 = mean wind speed in km/day measured 2 m above the ground

e_s = Saturation vapour pressure at mean air temperature, in mm Hg (Table 3.10)

e_a = actual vapour pressure in the air, in mm of Hg = $e_s \cdot R_H$ where R_H is the relative humidity (%)

H = daily net radiation in mm of H_2O , given by the relation

$$H = R_A (1 - r) (0.29 \cos \phi + 0.55 n/N) - \sigma T_a^4 (0.56 - 0.092 \sqrt{e_a}) (0.10 + 0.9 n/N) \quad \dots(3.12)$$

R_A = mean monthly extra-terrestrial radiation in mm H_2O /day (Table 3.11)

ϕ = latitude of the place where E_t is being computed.

r = reflection coefficient of the surface

= 0.15 to 0.25 for close crops

= 0.05 to 0.45 for barren land

= 0.05 for water surface

n = actual duration of bright sunshine which is a function of latitude of the place, and is an observed data at the place. Alternatively, the value of n can be taken equal to p given by Table 3.9.

N = maximum possible hours of bright sunshine available at different locations (Table 3.12)

σ = Stefan-Boltzman constant = 2.01×10^{-8} mm/day.

T = mean air temperature in $K = 273 + ^\circ C$

The wind speed measured at any other height z can be reduced to 2 m height by the relation (known as 1/7th power law).

$$u_z = u \left(\frac{2}{z} \right)^{1/7} = u \left(\frac{2}{z} \right)^{0.143} \quad \dots(3.13)$$

Thus, measuring n , e_a and u_2 at the place, and knowing all other relevant data from Tables 3.10, 3.11 and 3.12, the value of E_t can be computed from Eq. 3.10 given by Penman.

TABLE 3.10. SATURATION VAPOUR PRESSURE OF WATER

Temp. °C	Saturation Vapour pressure e_s (mm Hg)	Slope of plot between (1) and (2)	Temp. °C	Saturation vapour pressure e_s (mm Hg)	Slope of plot between (1) and (2)
(1)	(2)	(3)	(1)	(2)	(3)
0	4.58	0.30	25	23.76	1.4
5.0	6.54	0.45	27.5	27.59	1.61
7.5	7.78	0.54	30	31.81	1.85
10.0	9.21	0.60	32.5	36.68	2.07
12.5	10.87	0.71	35	42.81	2.35
15.0	12.79	0.80	37.5	48.36	2.62
17.5	15.00	0.95	40	55.32	2.95
20.0	17.54	1.05	42.5	62.18	3.25
22.5	20.44	1.24	45	71.2	3.65

TABLE 3.11. MEAN MONTHLY SOLAR RADIATION (R_A) INCIDENT ON EARTH'S OUTER SPACE (EXTRA-TERRESTRIAL RADIATION) IN mm OF EVAPORABLE WATER PER DAY (AFTER CRIDDLE)

Month	Northern Hemisphere										Southern Hemisphere									
	90°	80°	70°	60°	50°	40°	30°	20°	10°	0°	10°	20°	30°	40°	50°	60°	70°	80°	90°	
Jan.	...	...	...	1.3	3.6	6.0	8.5	10.8	12.8	14.5	15.8	16.8	17.3	17.3	17.1	16.6	16.5	17.3	17.6	
Feb.	...	...	...	1.1	3.5	5.9	8.3	10.5	12.3	13.9	15.0	15.7	16.0	15.8	15.2	14.1	12.7	11.2	10.5	
Mar.	...	1.8	4.3	6.8	9.1	11.0	12.7	13.9	14.8	15.2	15.1	14.6	13.6	12.2	10.5	8.4	6.1	3.8	1.9	
Apr.	7.9	7.8	9.1	11.1	12.7	13.9	14.8	15.2	15.2	14.7	13.8	12.5	10.8	8.8	6.6	4.3	1.9	...	...	
May	14.2	14.6	13.6	14.6	15.4	15.9	16.0	15.7	15.0	13.9	12.4	10.7	8.7	6.4	4.1	1.9	0.1	...	...	
June	18.1	17.8	17.0	16.5	16.7	16.7	16.5	15.8	14.8	13.4	11.6	9.6	7.4	5.1	2.8	0.8	...	...	...	
July	16.8	16.5	15.8	15.7	16.1	16.3	16.2	15.7	14.8	13.5	11.9	10.0	7.8	5.6	3.3	1.2	...	...	...	
Aug.	11.2	10.6	11.4	12.7	13.9	14.8	15.3	15.3	15.0	14.2	13.0	11.5	9.6	7.5	5.2	2.9	0.8	...	...	
Sept.	2.6	4.0	6.8	8.5	10.5	12.2	13.5	14.4	14.9	14.9	14.4	13.5	12.1	10.5	8.5	5.2	3.8	3.3	...	
Oct.	...	0.2	2.4	4.7	7.1	9.3	11.3	12.9	14.1	15.0	15.3	15.3	14.8	13.8	12.5	10.7	8.8	7.1	7.0	
Nov.	...	...	0.1	1.9	4.3	6.7	9.1	11.2	13.1	14.6	15.7	16.4	16.7	16.5	16.0	15.2	14.5	14.0	15.3	
Dec.	...	...	...	0.9	3.0	5.5	7.9	10.3	12.4	14.3	15.8	16.9	17.6	17.8	17.8	17.5	18.1	18.9	19.3	

TABLE 3.12. MAXIMUM POSSIBLE HOURS OF BRIGHT SUNSHINE IN HOURS (N)

Month	North Latitude in Degrees					
	0°	10°	20°	30°	40°	50°
Jan.	12.1	11.6	11.1	10.4	9.6	8.6
Feb.	12.1	11.8	11.5	11.1	10.7	10.1
March	12.1	12.1	12.0	12.0	11.9	11.8
April	12.1	12.4	12.6	12.9	13.2	13.8
May	12.1	12.6	13.1	13.7	14.4	15.4
June	12.1	12.7	13.3	14.1	15.0	16.4
July	12.1	12.6	13.2	13.9	14.7	16.0
Aug.	12.1	12.4	12.8	13.2	13.8	14.5
Sept.	12.1	11.9	12.3	12.4	12.5	12.7
Oct.	12.1	11.9	11.7	11.5	11.2	10.8
Nov.	12.1	11.7	11.2	10.6	10.0	9.4
Dec.	12.1	11.5	10.9	10.2	9.1	8.1

3. Hargreaves Class A Pan Evaporation Method

This method is very much used in India, apart from its use in Israel, Philippines and U.S.A. According to this method, the consumptive use (C_u) or Evapo-transpiration (E_t) is given by

$$E_t \text{ or } C_u = K E_p \quad \dots(3.14)$$

where K = Consumptive use coefficient, which is different for different crops, and for each crop, it depends upon many environmental factors. It is determined experimentally. Table 3.14 gives the value of K for some Indian crops.

E_p = Class A pan evaporation, which can be determined by the following Christiansen formula :

$$E = 0.459 R_A \cdot C_t \cdot C_w \cdot C_h \cdot C_e \cdot C_z \quad (\text{cm}) \quad \dots(3.15)$$

where R_A = Extra-terrestrial radiation (cm), to be determined from Table 3.10.

C_t = Coefficient for temperature, to be determined from the expression :

$$C_t = 0.393 + 0.02796 T_c + 0.0001189 T_c^2 \quad \dots(3.16)$$

(T_c being mean temp. in °C)

C_w = Coefficient for wind velocity, given by

$$C_w = 0.708 + 0.0034 W - 0.0000038 W^2 \quad \dots(3.17)$$

(W being the mean wind velocity in km/day at 0.5 m above ground surface)

C_h = Coefficient for relative humidity given by

$$C_h = 1.250 - 0.0087 H + 0.75 \times 10^{-4} H^2 - 0.83 \times 10^{-8} H^4 \quad \dots(3.18)$$

(H being the mean % relative humidity at noon or average relative humidity for 11 and 18 hours.

C_s = Coefficient for percent of possible sunshine and is given by :

$$C_s = 0.542 + 0.008 S - 0.78 \times 10^{-4} S^2 + 0.62 \times 10^{-8} S^3 \quad \dots(3.19)$$

(S = mean percentage of bright sun-shine hours taken as 100 n/N)

$$C_e = \text{Coefficient of elevation given by } C_e = 0.97 + 0.00984 E \quad \dots(3.20)$$

(E being the elevation in 100 metres).

Values of Hargreaves monthly consumptive use coefficient K : The value of K depends upon (i) type of crop (ii) place or location of irrigation field, and (iii) stage of the growth of the crop or % of crop growing season. At a given place, the value of K is different for different crops. Similarly, for the same crops, the value of K is different for different places. Lastly, for the same crop, at a given place, the value of K depends upon its stage of growth at a given time. In absence of local figures for specific crops, the data given in Table 3.13, recommended by Hargreaves, can be used. In the table, various crops have been divided into the following groups:

Group A : Potato, Cotton, Maize, Bean, Peas, Jowar, Beat

Group B : Tomato, Olive, Plumes, and some delicious fruits

Group C : Onions, Grapes, Melons, Carrots, Hops

Group D : Wheat, Barley, Celery and other grass type plants

Group E : Peas, Plantain, Orchard crops etc.

Group F : Oranges, Fruits, Citrus crops

Group G : Sugarcane, Alfalfa etc.

Paddy (Rice)

Table 3.14 gives the value of K for same Indian crops.

TABLE 3.13. HARGREAVES AVERAGE VALUE OF K

% of crop growing season	Value of consumptive use coefficient K							
	Group A	Group B	Group C	Group D	Group E	Group F	Group G	Rice
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
0	0.20	0.15	0.12	0.08	0.90	0.60	0.50	0.80
5	0.20	0.15	0.12	0.08	0.90	0.60	0.55	0.90
10	0.36	0.27	0.22	0.15	0.90	0.60	0.60	0.95
15	0.50	0.38	0.30	0.19	0.90	0.60	0.65	1.00
20	0.64	0.48	0.38	0.27	0.90	0.60	0.70	1.05
25	0.75	0.56	0.45	0.33	0.90	0.60	0.75	1.10
30	0.84	0.63	0.50	0.40	0.90	0.60	0.80	1.14
35	0.92	0.69	0.55	0.46	0.90	0.60	0.85	1.17
40	0.97	0.73	0.58	0.52	0.90	0.60	0.90	1.21
45	0.99	0.74	0.60	0.58	0.90	0.60	0.95	1.25
50	1.00	0.75	0.60	0.65	0.90	0.60	1.00	1.30
55	1.00	0.75	0.60	0.71	0.90	0.60	1.00	1.30
60	0.99	0.74	0.60	0.77	0.90	0.60	1.00	1.30
65	0.96	0.72	0.58	0.82	0.90	0.60	0.95	1.25
70	0.91	0.68	0.55	0.88	0.90	0.60	0.90	1.20
75	0.85	0.64	0.51	0.90	0.90	0.60	0.85	1.15
80	0.75	0.56	0.45	0.90	0.90	0.60	0.80	1.10
85	0.60	0.45	0.36	0.80	0.90	0.60	0.75	1.00
90	0.46	0.35	0.28	0.70	0.90	0.60	0.70	0.90
95	0.28	0.21	0.17	0.60	0.90	0.60	0.55	0.80
100	0.20	0.20	0.17	0.60	0.90	0.60	0.50	0.70

TABLE 3.14. VALUE OF K FOR SOME INDIAN CROPS
(TO BE USED IN THE EQUATION $E_t = K E_p$)

% of crop growing season	Wheat (Ludhiana)	Wheat (Poona)	Cotton (Poona)	Maize (Ludhiana)
0	0.14	0.30	0.22	0.40
5	0.17	0.40	0.22	0.42
10	0.23	0.51	0.23	0.47
15	0.33	0.62	0.24	0.54
20	0.45	0.73	0.26	0.63
25	0.60	0.84	0.35	0.75
30	0.72	0.92	0.58	0.85
35	0.81	0.96	0.80	0.96
40	0.88	1.10	0.95	1.04
45	0.90	1.10	1.03	1.07
50	0.91	1.00	1.08	1.09
55	0.90	0.91	1.08	1.10
60	0.89	0.80	1.07	1.11
65	0.86	0.65	1.05	1.10
70	0.83	0.51	1.00	1.07
75	0.80	0.40	0.93	1.04
80	0.76	0.30	0.85	1.00
85	0.71	0.20	0.73	0.97
90	0.65	0.12	0.62	0.89
95	0.58	0.10	0.50	0.81
100	0.51	0.10	0.40	0.70
Seasonal Value of K	0.61	0.61	0.68	0.86

3.14. IRRIGATION EFFICIENCIES

Efficient use of irrigation water is an obligation of each user as well as of the planners. Even under the best method of irrigation, not all the water applied during an irrigation is stored in the root zone. In general, efficiency is the ratio of water output to the water input and is expressed as percentage. The objective of efficiency concepts is to show when improvements can be made which will result in more efficient irrigation. The following are the various types of irrigation efficiencies : (i) water conveyance efficiency, (ii) water application efficiency, (iii) water use efficiency, (iv) water storage efficiency, (v) water distribution efficiency and (vi) consumptive use efficiency.

1. Water Conveyance Efficiency (η_c)

This takes into account the conveyance or transit losses and is determined from the following expression :

$$\eta_c = \frac{W_f}{W_r} \times 100 \quad \dots(3.21)$$

where η_c = water conveyance efficiency

W_f = water delivered to the farm or irrigation plot

W_r = water supplied or diverted from the river or reservoir.

2. Water Application Efficiency (η_a)

The water application efficiency is the ratio of the quantity of water stored into the root zone of the crops to the quantity of water delivered to the field. This focuses the attention of the suitability of the method of application of water to the crops. It is determined from the following expression :

$$\eta_a = \frac{W_s}{W_f} \times 100 \quad \dots(3.22)$$

where η_a = water application efficiency

W_s = Water stored in the root zone during the irrigation

W_f = water delivered to the farm.

The common sources of loss of irrigation water during water application are (i) surface run off R_f from the farm and (ii) deep percolation D_f below the farm root-zone soil. Hence

$$W_f = W_s + R_f + D_f \quad \dots(3.23)$$

and

$$\eta_a = \frac{W_f - (R_f + D_f)}{W_f} \times 100 \quad \dots[3.23(a)]$$

In a well designed surface irrigation system, the water application efficiency should be atleast 60% ; in the sprinkler irrigation system this efficiency is about 75%. The common factors which are responsible for low water application efficiency are : (i) irregular land surfaces, (ii) shallow soils underlain by gravels of light permeability, (iii) either very small or excessively large irrigation streams, (iv) non-attendance of water during irrigation, (v) long irrigation runs, (vi) wrong irrigation methods, (vii) improper preparation of land, (viii) compact impervious soil, (ix) steep slopes of land surfaces and (x) excessive single application.

3. Water Use Efficiency (η_u)

It is the ratio of water beneficially used, including leaching water, to the quantity of water delivered, and is determined from the following expression :

$$\eta_u = \frac{W_u}{W_d} \times 100 \quad \dots(3.24)$$

where η_u = water use efficiency

W_u = water used beneficially or consumptively

W_d = water delivered.

4. Water Storage Efficiency (η_s)

The concept of water storage efficiency gives an insight to how completely the required water has been stored in the root zone during irrigation. It is determined from the following expression:

$$\eta_s = \frac{W_s}{W_n} \times 100 \quad \dots(3.25)$$

where η_s = water storage efficiency

W_s = water stored in the root zone during irrigation

W_n = water needed in the root zone prior to irrigation
= (Field capacity - Available moisture).

5. Water Distribution Efficiency (η_d)

Water distribution efficiency evaluates the degree to which water is uniformly distributed throughout the root zone. Uneven distribution has many undesirable results. The more uniformly the water is distributed, the better will be the crop response. It is determined from the following expression :

$$\eta_d = 100 \left[1 - \frac{y}{d} \right] \quad \dots(3.26)$$

where η_d = water distribution efficiency

y = average numerical deviation in depth of water stored from average depth stored during irrigation.

d = average depth of water stored during irrigation.

The efficiency provides a measure for comparing various systems or methods of water application, i.e. sprinkler compared to surface, one sprinkler system compared to the other system or one surface method compared to other surface method.

6. Consumptive Use Efficiency (η_{cu})

$$\text{It is given by } \eta_{cu} = \frac{W_{cu}}{W_d} \times 100 \quad \dots(3.27)$$

where W_{cu} or C_u = normal consumptive use of water

W_d = net amount of water depleted from root zone of soil.

The efficiency, therefore, evaluates the loss of water by deep percolation and by excessive surface evaporation following an irrigation.

(C) DETERMINATION OF IRRIGATION REQUIREMENTS OF CROPS

In order to determine the irrigation requirements of certain crop, during its base period, the following terms are required :

(i) Effective Rainfall (R_e)

Effective rainfall is that part of the precipitation falling during the growing period of a crop that is available to meet the evapo-transpiration needs of the crop.

(ii) Consumptive Irrigation Requirement (CIR)

Consumptive irrigation requirement is defined as the amount of irrigation water that is required to meet the evapo-transpiration needs of the crop during its full growth. Therefore,

$$CIR = C_u - R_e \quad \dots(3.28)$$

where C_u is the consumptive use of water.

(iii) Net Irrigation Requirement (NIR)

Net irrigation requirement is defined as the amount of irrigation water required at the plot to meet the evapo-transpiration needs of water as well as other needs such as leaching etc. Thus

$$NIR = C_u - R_e + \text{water lost in deep percolation for the purpose of leaching etc.} \quad \dots(3.29)$$

(iv) Field Irrigation Requirement (FIR)

Field irrigation requirement is the amount of water required to meet 'net irrigation requirements' plus the water lost in percolation in the field water courses, field channels and in field applications of water. If η_c is water application efficiency, we have

$$FIR = \frac{NIR}{\eta_c} \quad \dots(3.30)$$

(v) Gross Irrigation Requirement (GIR)

Gross irrigation requirement is the sum of water required to satisfy the field irrigation requirement and the water lost as conveyance losses in distributaries upto the field. If η_c is the water conveyance efficiency, we have

$$GIR = \frac{FIR}{\eta_c} \quad \dots(3.31)$$

Determination of FIR and GIR using Hargreaves method :

In order to determine the field irrigation requirement (FIR) or gross irrigation requirement (GIR) of a certain crop, it is essential to have the knowledge of monthly or periodical pan evaporation data as well as the knowledge of pan evaporation coefficient K . Table 3.15 shows sample calculations for wheat at a certain location. Following is the procedure for calculations :

TABLE 3.15. DETERMINATION OF IRRIGATION REQUIREMENTS OF LUDEHANA WHEAT
PERIOD OF GROWTH : 3rd NOV. TO 12th MARCH (131 DAYS)

Interval (1)	No. of days upto mid point of interval (2)	% of growing season = col. (2) $\times 100/131$ (3)	E_p (cm) (4)	K (5)	$C_u = K E_p$ (cm) (6)	R_e (cm) (7)	$NIR = C_u - R_e$ (cm) (8)	$FIR = \frac{NIR}{\eta_u}$ (cm) (9)	$GIR = \frac{FIR}{\eta_c}$ (cm) (10)
Nov. 3-30	14	11	15.8	0.30	4.7	-	4.7	6.9	8.5
Dec. 1-31	44	33	13.1	0.77	10.1	0.8	9.3	13.7	17.1
Jan. 1-31	75	57	12.8	0.90	11.5	0.6	10.9	16.0	20.0
Feb. 1-29	105	80	15.0	0.76	11.4	-	11.4	16.8	21.0
March 1-12	125	95	16.2	0.58	9.4	-	9.4	13.8	17.3
					$\Sigma = 47.1$		$\Sigma = 45.7$	$\Sigma = 67.2$	$\Sigma = 84.0$

Column 1: This denotes the monthly periods of growth.

Column 2: This denotes the number of days up to the mid point of the interval.

Column 3: This denotes the mid-point interval of growth (column 2) expressed as percentage of the total period of growth.

Column 4: This gives the pan evaporation data (E_p) for the various periods of growth of the crop.

Column 5: This gives the pan evaporation coefficient K for different % of growing season. Its values may be taken from available tables (such as Tables 3.13 or 3.14).

Column 6: This gives the monthly values of consumptive use C_u determined from the Eq. $C_u = K E_p$, in cm of water.

Column 7: This gives the data about the effective rainfall (R_e) during various periods.

Column 8: This gives the net irrigation requirement. If no additional water is required for leaching or other operations requiring deep percolation of water, consumptive irrigation requirement (CIR) and net irrigation requirements will be equal. Hence $NIR = C_u - R_e$.

Column 9: This gives the field irrigation requirement, by dividing NIR by water application efficiency. In the Table, η_u has been assumed to be 0.68.

Column 10: This gives the gross irrigation requirement (GIR) by dividing FIR by water conveyance efficiency η_c . In the Table η_c has been assumed to be 0.80.

Example 3.18. Table below gives the details for a certain crop. Using Blaney-Criddle equation and a crop factor $k = 0.75$, determine the following (i) consumptive use, (ii) consumptive irrigation requirement, (iii) field irrigation requirement, if water application efficiency is 0.7. The latitude of the place is 30° N.

Month (1)	Monthly temp (A_m) $^\circ\text{C}$ (2)	Monthly % of day time hours of the year (3)	Useful Rain fall (cm) (4)
Nov.	19.0	7.19	-
Dec.	16.0	7.15	1.2
Jan.	12.5	7.30	0.8
Feb.	13.0	7.03	-

Solution

The data of column 3 of the Table are taken from the sunshine table (Table 3.9). The monthly temperature and useful rainfall data are average values for the last 10 years. The consumptive use is computed from the Blaney-Criddle equation:

$$C_u = k \Sigma f$$

where $f = \frac{P}{40} [1.8t + 32]$

Computations are done in the tabular form below :

Month (1)	$^\circ\text{C}$ (2)	$p\%$ (3)	f (4)
Nov.	19.0	7.19	11.9
Dec.	16.0	7.15	10.9
Jan.	12.5	7.30	9.9
Feb.	13.0	7.03	9.7
			$\Sigma f = 42.4$

$$C_u = k \Sigma f = 0.75 \times 42.4 = 31.8 \text{ cm}$$

$$R_e = 1.2 + 0.8 = 2.0 \text{ cm.}$$

$$\text{CIR} = C_u - R_e = 31.8 - 2.0 = 29.8 \text{ cm.}$$

$$\text{NIR} = \text{CIR} \text{ (since no water is used for deep percolation)}$$

$$\text{FIR} = \frac{\text{NIR}}{\eta_u} = \frac{29.8}{0.7} = 42.6 \text{ cm.}$$

Example 3.19. Compute the consumptive use of rice for the month of January by using Penman's formula, for the following data available at the field.

1. Latitude of place : 20°N
2. Mean monthly temperature : 15°C
3. Relative humidity in January : 50%
4. Elevation of area : 250 m
5. Wind velocity at 2 m height : 25 km/day

Solution

From Table 3.10, we obtain the following for mean temperature $t_m = 15^\circ\text{C}$

(i) Saturation vapour pressure, $e_s = 12.79$ mm Hg

(ii) Slope of the curve between e_s and t_m , $A = 0.80$ mm per $^\circ\text{C}$

From Table 3.11, $R_A = 10.8$ mm of H_2O per day at latitude 20°N , in Jan.

From Table 3.12, $N = 11.1$ h for latitude 20° , in Jan.

From Table 3.9, monthly percentage of day time hours, $n (=p) = 7.74$ h for Jan at latitude of 20°N

$$\frac{n}{N} = \frac{7.74}{11.1} = 0.697$$

$$e_a = e_s - R_{H1} = 12.79 \times 0.5 = 6.395 \text{ mm Hg}$$

$$E_a = 0.002187 (160 + u_2) (e_s - e_a)$$

$$= 0.002187 (160 + 25) (12.79 - 6.395) = 2.59 \text{ mm/day}$$

Rice is a close crop for which the reflection coefficient r may be taken equal to 0.2. Take $\alpha = 0.49$ mm Hg/ $^\circ\text{C}$ and $\sigma = 2.01 \times 10^{-8}$ mm/day

Also, $T_a =$ mean air temp. in $^\circ\text{K} = 273 + 20 = 293^\circ\text{K}$

$$\begin{aligned} H &= R_A (1 - r) (0.29 \cos \phi + 0.55 n/N) - \sigma T_a^4 (0.56 - 0.092 \sqrt{e_a}) (0.10 + 0.9 n/N) \\ &= 10.8 (1 - 0.2) (0.29 \cos 20^\circ + 0.55 \times 0.697) - 2.01 \times 10^{-8} (293)^4 \times \\ &\quad (0.56 - 0.092 \sqrt{6.395}) \times (0.10 + 0.9 \times 0.697) \end{aligned}$$

$$= 5.67 - 3.53 = 2.14 \text{ mm of } \text{H}_2\text{O/day}$$

$$E_t = \frac{A H + \alpha E_a}{A + \alpha} = \frac{0.8 \times 2.14 + 0.49 \times 2.59}{0.8 + 0.49} = 2.31 \text{ mm/day}$$

$$= 2.31 \times 31 \text{ mm/month} = 71.6 \text{ mm of water for January}$$

3.15. SOIL FERTILITY

A soil is fertile when it contains :

- Ample supplies of organic materials.
- The source of nitrogen.
- Sufficient soluble compounds of the mineral elements needed for the growth of food plants.

The above requirements coupled with satisfactory surface topography, physical properties, subsoil drainage and application of adequate water at proper intervals leads to the production of rich crops.

Maintenance of soil fertility

Soil fertility can be maintained by the following :

- By use of proper cultivation methods.
- By spreading all the farmyard manure which supply organic materials as well as fertilising elements.
- Planting of different crops in rotation on different areas.

(iv) Application of carefully selected commercial fertilizers.

(v) Ploughing under or disking-in of crop, plants or native vegetation or green manures adds organic matter and fertilising elements. Deep ploughing is undesirable.

(vi) *Stubble-mulch farming* : It is a recently developed method of crop tillage that leaves the stubble and other residues of the preceding crop on or close to the soil surface to provide a surface mulch. The mulch tends to conserve moisture and soil fertility. It reduces compaction of the surface soil by rainfall, decreases surface run-off and largely prevent soil erosion by wind.

(vii) Suitable procedures for reducing or controlling erosions may constitute important role in maintaining soil fertility.

(viii) Irrigation with silty water.

(ix) Soils of acidic nature may show improvement with the application of calcium oxide to the extent of 1000 kg per hectare. Shortage of lime leads to dark colour of leaves and its excess may lead to their scorching.

3.16. CROP ROTATION

The term rotation of crops is somewhat self explanatory. It implies that nature of the crop sown in a particular field is changed year after year.

Necessity for rotation

The necessity for rotation arises from the fact that soil gradually loses its fertility if the same crop is sown every year and the field has to be allowed to lie fallow in order to regain its fertility. The effect of fallow is obtained by rotation of crops.

All crops require similar type of nutrients for their growth but all of them do not take in the same quantities or proportions; some crop favours certain plant nutrients and takes them more than the others. Thus if a particular crop is grown year after year, the soil gets deficient in the plant food favoured by that crop. If different crops are to be raised there would certainly be more balanced feeding and soil deficient in one particular type of nutrient is allowed to recoupe.

Crop diseases and insect pests will multiply at an alarming rate, if the same crop is to be grown continuously. Rotation will check this disease.

A leguminous crop (such as gram) if introduced in rotation will increase nitrogen content of the soil thus increasing its fertility.

There could be deep rooted crops and shallow rooted crops in rotation and so they shall draw their food from different depths of soil. The soil will thus be better utilised. Rotation of crops in the long run is beneficial to the farmer and soil both as there would be a rotation of cash crops, fodder crops and soil renovating crops.

The following rotations have been found useful :

(i) Introduction of groundnut in the usual cotton jowar rotation as per the experiments in Central Province, Hyderabad.

(ii) In Northern India, cotton is known to have improved when it follows berseem on fields manured with phosphate.

(iii) In Madras, Bajra when rotated with cotton has proved to be superior to Jawar being rotated with cotton.

(iv) Cotton gives good result after gram or fallow.

(v) Wheat does well after-sugar-cane and gowara.

In general the following rotation of the crops give good results :

1. Wheat-great millet-gram
2. Rice-gram
3. Cotton-wheat-gram
4. Cotton-wheat-sugarcane
5. Cotton-great millet-gram.

The reason for gram being one in various rotation combinations is its being leguminous crop giving nitrogen to the field.

3.17. ASSESSMENT OF IRRIGATION WATER

The water which has been supplied for irrigation to the farmers is at the Government expenses. Some nominal charges must, therefore, be levied on the farmers for using this water. The fixation of such charges is known as *assessment* of irrigation water. The returns on the capital investment is based upon the estimate of such water charges. A knowledge of the same is, therefore, very essential to the engineers.

The charges must be levied on the farmers for the following reasons :

(i) To recover the cost of construction of the project by which it has been possible to supply water.

(ii) To recover the maintenance cost of the various works and staff for certain improvement.

(iii) To collect some revenue for the nation.

(iv) To check the cultivators against uneconomical and careless use of water.

Methods of Assessment

There are five methods of assessment :

- (i) Assessment on area basis or crop rate basis
- (ii) Volumetric assessment
- (iii) Assessment on seasonal basis
- (iv) Composite rate basis
- (v) Permanent assessment

1. Assessment on Area Basis or Crop Rate Basis

Under this system of assessment of irrigation, area sowing crops is recorded by a 'patrol' both at the time of sowing and maturity. At the end of the crop period a 'demand statement' for each irrigator is prepared. The measurements are checked by various officials including Zilladar, Deputy Collector, Sub-Divisional Officer and the Divisional Canal Officer. Checking of irrigated area is an important part of Canal Officers' duty at all levels upto Divisional Officer. For failure of crops for reasons beyond the control of the irrigator or for shortage of water supply, remission may be given to the farmers.

Special charges are levied in addition to the usual charges under the Canal Act for the following :

- (a) Unauthorised irrigation comprising use of unauthorised water course.
- (b) Cuts in channel banks
- (c) Damaging the outlets
- (d) Opening outlets at unauthorised time
- (e) Irrigation out of sanctioned turn
- (f) Wasting water.

Special charges may go up 6 times the normal charges.

Almost all the canal irrigated lands in India are assessed for water on the area basis and crop rate basis. The rates for different crop have been fixed on the basis of

(1) Cash value for crop and hence its paying capacity.

(2) Water requirements of the crop.

(3) Time of demand for water *vis-a-vis* the available supplies in a river.

For illustration, the following information for a few principle crops is given (Table 3.16).

TABLE 3.16

S.No.	Crops	Yield in kg/hectare	Water rates in Rs./hectare	
			Punjab	U.P.
1.	Rice	320	Rs. 49	Rs. 79
2.	Sugarcane	1200	Rs. 62	Rs. 149
3.	Cotton	260	Rs. 34	Rs. 30
4.	Wheat	560	Rs. 32	Rs. 90
5.	Maize	560	Rs. 32	Rs. 22

The system of assessment on the basis of area has a number of disadvantages, notably amongst them are as follows :

1. The cultivators near the head reaches of the canal use more water and deprive the area near the tail, of their required share of water.

2. Since the charges are not levied on the basis of actual quantity of water used, cultivators make a wasteful use of water.

2. Volumetric Assessment

The method involves levying charges on actual volume of water supplied. This will be an ideal system under which the irrigator has an incentive for economic use of water and shortfalls in supplies can be distinctly located. It is, however, not a very practical method for India due to the necessity of installation of numerous water meters on irrigation outlets scattered all over the channel and difficult to maintain, besides a heavy initial cost.

In addition to the cost, the method has the following short comings :

1. A whole time staff is needed to keep records of the meters.
2. The meters may not work on small heads at the outlet.
3. Silt and debris may obstruct the flow through the metered outlet.

4. The cultivators may pilfer water by unauthorised methods.

5. The internal distribution of volumetric supplies and apportionment of dues between petty shareholders may be difficult.

The only possible place for such assessment is tube-well, where water is sold out on a volumetric basis by litres per rupee or by units rate of electric power consumed, as a tube well operator is there to keep the full record. The only other feasible proposal is to sell out water on volumetric basis to co-operative societies or panchayats of irrigation who should be responsible both for internal distribution as well as collection and payment of canal dues for the same.

3. Assessment on Seasonal Basis

In this method the rate of assessment is based on the type of crop grown in a particular tract during certain crop season. This method is not in vogue in India.

4. Composite Rate Basis

Composite rate is a combination of water charges and the land revenue. Some states prefer to collect both the revenues together and hence the name.

5. Permanent Assessment or Betterment Levy Basis

The cultivators in a particular area may have their own source of water supply under ordinary conditions and may not use water from a canal which has been aligned in that area only to protect it in drought period. The farmers of that area are levied at a fixed rate every year due to such provisions. Such a levy is known as *betterment levy*. During the draught periods, the farmer will be authorised to use water from such a canal without paying extra charges above the betterment levy.

Internal Distribution of Water

Internal distribution of water is carried out on a 'wara bandi' system fixed by the Divisional Canal Officer. Under this system each shareholder is given his turn in hours according to the area, out of seven days in a week. In the absence of 'wara bandi', the distribution is done by cultivators themselves.

Efficiency of Distribution

In arid and fertile areas, the value of canal water is really great and efficiency of equitable distribution has to be ensured both in the interest of irrigation as well as for economical and efficient use of water. The main disturbing factor in the equitable distribution is the upsetting of channel regime by silting, which has to be specially set right by remodelling. The efficiency of the actual performance of the water utilisation on outlets is kept under constant watch by the following methods :

1. Watching the gauges and noting changes in levels, if any by silting and over-drawing.
2. Actual discharge observation of the non-modular outlets by portable flumes.
3. Watching monthly data calculations or depth of water utilised per hectare for the channel and final crop delta as compared to previous figures.
4. By plotting efficiency diagrams for the outlets on a channel showing authorised and actual irrigation percentages which bring prominently to notice shortfalls and excess and the urgency of the steps to be taken for rectification of defects. Shortfalls may be due to main distribution requiring rectification by remodelling.

5. In the absence of a dependable metering head of the channel its percentage of the normal discharge table must be checked every month and at a few more sites along the length for a channel.

3.18. EXAMPLES FROM COMPETITIVE EXAMINATIONS

Example 3.20. Wheat is to be grown in field having a field capacity equal to 27% and the permanent wilting point is 13%. Find the storage capacity in 80 cm depth of soil, if the dry unit weight of the soil is 1.5 g/cc. If the irrigation water is to be supplied when the average soil moisture falls to 18%, find the water depth required to be supplied to field if the field application efficiency is 80%. What is the amount of water needed at the canal outlet if the water lost in the water course and the field channel is 15% of the out let discharge ? (Civil Services Exam. 1980)

Solution

(a) Maximum storage capacity = available moisture

$$= \frac{\gamma_d \cdot d}{\gamma_w} [F_c - w_c] = \frac{1.5 \times 80}{1} [0.27 - 0.13] = 16.8 \text{ cm}$$

(b) Depth of irrigation water $\frac{\gamma_d \cdot d}{\gamma_w} [F_c - m_a] = \frac{1.5 \times 80}{1} [0.27 - 0.18] = 10.8 \text{ cm}$

(c) Field irrigation requirement = $\frac{10.8}{0.8} = 13.5 \text{ cm}$

(d) Amount of water needed at the canal outlet = $\frac{13.5}{0.85} = 15.9 \text{ cm}$

Example 3.21. A canal takes off a reservoir to irrigate the areas as shown in the Table. 40% of the water required for irrigation is assumed to be available directly from precipitation. Channel conveyance losses are 15%. Reservoir losses are 10%. What would be the capacity of the reservoir needed ? (The reservoirs to be filled only once a year).

Crop	Base period (days)	Duty at the field (ha/cumec)	Area under crop (ha)
Wheat	120	1800	500
Sugarcane	320	800	600
Rice	120	900	300
Cotton	200	1400	1200
Bajra	100	1200	500

(Civil Services Exam. 1980)

Solution : The required computations are done in a tabular form below.

Crop	Base period B (days)	Duty at field (D) ha/cumec	$\Delta = 8.64 \frac{B}{D}$ (m)	Area A (ha)	Volume = $\Delta \times A$ ha-m
(1)	(2)	(3)	(4)	(5)	(6) = (4) × (5)
Wheat	120	1800	0.576	500	288.0
Sugarcane	320	800	3.456	600	2073.6
Rice	120	900	1.162	300	348.6
Cotton	200	1400	1.234	1200	1481.1
Bajra	100	1200	0.72	500	360.0
				EV	= 4543.3

However, 40% of the required water is available from precipitation.

$$\therefore \text{Capacity of reservoir} = \frac{0.6 \times 4548.3}{0.85 \times 0.9} \triangleq 3568 \text{ ha-m}$$

Example 3.22. The gross commanded area for a distributory is 10000 hectares, 75% of which can be irrigated. The intensity of irrigation for Rabi season is 60% and that for Kharif season is 30%. If the average duty at the head of the distributory is 2500 hectares per cumec for Rabi season and 1000 hectares per cumec for Kharif season, determine the discharge required at the head of the distributory from average demand consideration.

(Civil Services Exam. 1988)

Solution

Gross commanded area = 10000 hectares

Culturable commanded area = $0.75 \times 10000 = 7500$ hectares.

Area under irrigation in Rabi season = $7500 \times 0.6 = 4500$ hectares

Area under irrigation in Kharif season = $7500 \times 0.3 = 2250$ hectares.

$$\therefore \text{Discharge for Rabi} = \frac{4500}{2500} = 1.8 \text{ cumec}$$

$$\text{Discharge for Kharif} = \frac{2250}{1000} = 2.25 \text{ cumec.}$$

$\therefore$ Required discharge at the head of distributory = 2.25 cumecs

Example 3.23. Work out the irrigation schedule based on the soil moisture concept, given the following information. Also extract the data on the total depth of irrigation water required and the respective dates of irrigation water supply.

- The crop is grown in a appropriate soil with no restrictive layers within the top 1.5 m depth of soil.
- Normal root zone depth of the crop is 1.2 m
- Bulk density of soil is 1.35
- Field capacity is 18% and permanent wilting point is 7%.
- Moisture level in the soil is to be maintained at not less than one third of available retention. Irrigation will then be done over a duration of 2 days at a uniform rate of supply and at a uniform rate of advance to fully and just compensate for the depletion.
- No extra water is ever required for leaching.
- Sowing is done on 1 November when the soil moisture is left just at field capacity in the entire root zone.
- For the crop, at the location, the average evapo-transpiration rates are:

1 Nov - 30 Nov.	: 1.1 mm/day
1 Dec. - 31 Dec.	: 1.7 mm/day
1 Jan. - 31 Jan.	: 2.4 mm/day
1 Feb. - 28 Feb.	: 1.5 mm/day
1 March - 25 March	: 3.5 mm/day
- Harvesting is done on or after 26 March.
- There is expected an effective rainfall of 24 mm during 4 January and 19 January, both days inclusive, with uniform intensity.

- By the end of the crop growth season, only the minimum water needed to be left unused in the root zone. (Engg. services Exam. 1990)

Solution

Field capacity, $F_c = 18\%$

Wilting coefficient, $w_c = 7\%$

$$\therefore \text{Available moisture} = 18 - 7 = 11\%$$

$$\therefore m_0 = w_c + \frac{1}{3} \text{ available moisture} = 7 + \frac{1}{3} \times 11 = 10.67\%$$

$$\therefore d_w = \frac{1.4 \cdot d}{\gamma_w} [F_c - m_0] = S_g \cdot d [F_c - m_0]$$

Note that in the above question, bulk density has been erroneously given as 1.35. Actually, it is the apparent density or apparent specific gravity S_g .

$$\therefore d_w = 1.35 \times 1.2 [0.18 - 0.1067] \triangleq 0.1187 \text{ m} = 11.87 \text{ cm}$$

Hence irrigation water will be supplied as and when this 11.87 cm depth of stored moisture gets consumed by evapo-transpiration.

(a) *Irrigation requirements from 1 Nov. to 3 Jan. (when rains start)*

Depth of water consumed by evapo-transpiration during this period
 $= 1.1 \times 30$ (i.e. between 1 Nov. - 30 Nov.) $+ 1.7 \times 31$ (i.e. between 1 Dec - 31 Dec.)
 $+ 2.4 \times 3$ (i.e. between 1 Jan - 3 Jan) $= 92.9 \text{ mm} = 9.29 \text{ cm}$... (i)

This is less than $d_w = 11.87 \text{ cm}$. Hence no irrigation water is required during

this period.

(b) *Irrigation requirements after 3rd Jan.*

$$\text{Rainfall intensity between 4 Jan to 19 Jan} = \frac{24}{16} = 1.5 \text{ mm/day.}$$

Consumptive use during this period = 2.4 mm/day.

$\therefore$ Amount of water consumed from soil during this period of 4 Jan. to 19 Jan.

$$(16 \text{ days}) = (2.4 - 1.5) \times 16 = 14.4 \text{ mm} = 1.44 \text{ cm} \quad \dots (ii)$$

Hence total water withdrawn from soil from 1 Nov. to 19 Jan.

$$= 9.29 + 1.44 = 10.73 \text{ cm}$$

$\therefore$ Balance left in soil, to be used by crop before next irrigation.

$$= 11.87 - 10.73 = 1.14 \text{ cm} = 11.4 \text{ mm.}$$

Rate of evapo-transpiration for the remaining period in Jan = 2.4 mm/day

Hence the above water is consumed in $11.4/2.4 = 4.75 \text{ day} \triangleq 4 \text{ days (say)}$

Hence next irrigation will be needed after 4 days of stoppage of rainfall, i.e. on

24th Jan.

The irrigation water is supplied for 2 days (i.e. for 24th and 25th Jan). Hence depth of water required in the first irrigation = $10.73 + 4 \times 0.24 + 2 \times 0.24 = 12.17 \text{ cm}$

(c) Irrigation requirements from 26 Jan. to 25 March

Water consumed between 26 Jan - 31 Jan. = $2.4 \times 6 = 14.4$ mm

Water consumed between 1 Feb - 28 Feb. = $1.5 \times 28 = 42.0$ mm

Water consumed between 1 March - 25 March = $3.5 \times 25 = 87.5$ mm

Total 143.9 mm

This is greater than $d_w = 118.7$ mm

Hence next water supply is started after x days of March, where

$$x = \frac{118.7 - (14.4 + 42)}{3.5} = 17.8 \text{ days (i.e. 17 days)}$$

Hence second irrigation is required on 18th March.

Required water depth = $143.9 - 118.7 = 25.2$ mm

Hence the irrigation schedule is as follows.

(i) First watering on 29 and 30 Jan = 12.17 cm

(ii) Second watering on 18th March = 2.52 cm

Example 3.24. A certain crop is grown in an area of 3000 hectares which is fed by a canal system. The data pertaining to irrigation are as follows :

Field capacity of soil = 26%

Optimum moisture = 12%

Permanent wilting point = 10%

Effective depth of root zone = 80 cm

Relative density of soil = 1.4

If the frequency of irrigation is 10 days and the overall efficiency is 23%, find
(i) the daily consumptive use (ii) The water discharge in $\text{m}^3/\text{sec.}$ required in the canal feeding the area.
(Civil Services Exam. 1990)

Solution

$$d_w = S_e \cdot d [F_c - m_0] = 1.4 \times 80 [0.26 - 0.12] = 15.68 \text{ cm}$$

Frequency of irrigation = 10 days.

$$\therefore \text{Daily consumptive use} = \frac{15.68}{10} = 1.568 \text{ cm} = 15.68 \text{ mm}$$

$$\text{Total water required in 10 days} = A \times d_w = 3000 \times 10^4 \times \frac{15.68}{100} = 4704000 \text{ m}^3$$

$$\text{Discharge in the canal} = \frac{4704000}{10 \times 24 \times 3600} \approx 5.45 \text{ cumecs}$$

Example 3.25. The consumptive use requirements of a crop are 0.2 cm per day for days 1 to 15, 0.3 cm per day for days 16 to 40 ; 0.5 cm/day for days 41 to 50 and 0.1 cm/day for days 51 to 55. Effective rainfall of 3.5 cm, distributed uniformly during the 36th and 45th days (both inclusive) is predicted. Compute the total quantity

of water (in cu. m) to be delivered to a 50 hectares plot for the whole crop season with a pre-sowing requirement of 5 cm of water.
(Gate 1991)

Solution : Water requirements per hectare:

$$(i) \quad 1 \text{ to } 15 \text{ days i.e. } 15 \text{ days @ } 0.2 \text{ cm/day} = \frac{15 \times 0.2}{100} \times 10^4 = 300 \text{ m}^3$$

$$(ii) \quad 16 \text{ to } 40 \text{ days i.e. } 25 \text{ days @ } 0.3 \text{ cm/day} = \frac{25 \times 0.3}{100} \times 10^4 = 750 \text{ m}^3$$

$$(iii) \quad 41 \text{ to } 50 \text{ days i.e. } 10 \text{ days @ } 0.5 \text{ cm/day} = \frac{10 \times 0.5}{100} \times 10^4 = 500 \text{ m}^3$$

$$(iv) \quad 51 \text{ to } 55 \text{ days i.e. } 5 \text{ days @ } 0.1 \text{ cm/day} = \frac{5 \times 0.1}{100} \times 10^4 = 50 \text{ m}^3$$

$$\text{Pre-sowing requirement} = \frac{5}{100} \times 10^4 = 500 \text{ m}^3$$

Total = 2100 m^3

Effective rainfall/hectare during 36th and 45th day (i.e. 10 days)

$$= \frac{3.5}{100} \times 10^4 = 350 \text{ m}^3$$

Net water quantity to be delivered per hectare = $2100 - 350 = 1750 \text{ m}^3$

$\therefore$ Quantity for water to be delivered for 50 hectares

$$= 50 \times 1750 = 87500 \text{ m}^3$$

Example 3.26. The following data pertains to the healthy growth of a crop.

(i) Field capacity of soil = 30%

(ii) Permanent wilting percentage = 11%

(iii) Density of soil = 1300 kg/m^3

(iv) Effective depth of root zone = 700 mm

(v) Daily consumptive use of water for the given crop = 12 mm

For healthy growth moisture content must not fall below 25% of the water holding capacity between the field capacity and the permanent wilting point. Determine the watering interval in days.
(Engg. Services Exam. 1982)

Solution

Water holding capacity above wilting point = $F_c - w_c = 30 - 11 = 19\%$

$\therefore$ Optimum moisture $m_0 = 30 - 0.75 \times 19 = 15.75\%$

$$d_w = \frac{\gamma_d \cdot d}{\gamma_w} [F_c - m_0] = \frac{1300}{1000} \times 700 [0.30 - 0.1575] = 129.88 \text{ mm}$$

Daily consumptive use = 12 mm

$$\therefore \text{Watering interval} = \frac{129.88}{12} = 10.80 \approx 10 \text{ days}$$

Example 3.27. A water course commands an irrigated area 1000 hectares. The intensity of irrigation of rice in this area is 70%. The transplantation of rice, crop takes 15 days and during transplantation period, total depth of water required by the crop on the field is 500 mm. During the transplantation period, the useful rain falling on the field is 120 mm. Find the duty of irrigation water for crop on the field during transplantation at the head of the field and also at the head of the water course, assuming loss of water to be 20% in the water course. Also, calculate the discharge required in the water course. (Engg. services Exam. 1993)

Solution : Area under irrigation = $1000 \times 0.7 = 700$ hectares

Effective depth of water required during transplantation period
 $= 500 - 120 = 380 \text{ mm} = 0.38 \text{ m}$

$\therefore$ Duty of water at the field = $\frac{8.64 \text{ b}}{\delta} = 8.64 \times \frac{15}{0.38} = 341 \text{hec/cumec.}$

Duty at the head of water course = $341 \times \frac{80}{100} = 272.8 \text{ hec/cumec}$

Discharge required in water, course = $\frac{700}{272.8} \approx 2.57 \text{ cumecs}$

Example 3.28. The base period, intensity of irrigation and duty of water for various crops under the canal system are given. Determine the reservoir capacity if the culturable commands area is 4000 hectares, canal losses are 25% and reservoir losses are 15%.

Crop	Base period (days)	Duty at field (hec/cumec)	Intensity of irrigation
Wheat	120	1800	20%
Sugar cane	360	1700	20%
Cotton	180	1400	10%
Rice	120	800	15%
Vegetable	120	700	15%

(Engg. Services Exam. 1994)

Solution: The computations are done in the Tabular form below. Note that Quantity of water = Q (in cumecs) $\times B$ (days) = $8.64 \times 10^6 Q B \text{ m}^3$.

Crop	Base period (days)	Intensity of irrigation	Area under crop (hectares)	Duty (hec/cumec)	Discharge required (cumecs)	Quantity of water (m^3)
(1)	(2)	(3)	(4)	(5)	(6) = (4)/(5)	(7) = 8.64 $\times 10^6$ $\times$ (6) $\times$ (3)
Wheat	120	20	8000	1800	4.444	46.08×10^6
Sugar cane	360	20	8000	1700	4.706	146.38×10^6
Cotton	180	10	4000	1400	2.857	44.41×10^6
Rice	120	15	6000	800	7.5	77.76×10^6
Vegetable	120	15	6000	700	8.571	88.81×10^6
Total						403.52×10^6

$\therefore$ Volume of required at head of canal = $\frac{403.52 \times 10^6}{0.75} = 538.03 \times 10^6 \text{ m}^3$

$\therefore$ Storage capacity = $\frac{538.03 \times 10^6}{0.85} \approx 633 \times 10^6 \text{ m}^3$

PROBLEMS

1. Discuss in brief the functions of irrigation water. What are the standards of irrigation water ?

2. Describe with the help of a diagram various forms of soil moisture. What do you understand by the available moisture ?

3. Write notes on the following :

(i) Saturation capacity, (ii) Field capacity, (iii) Wilting point, (iv) Optimum water.

4. What do you understand by evapo-transpiration ? How is it determined ?

5. Explain the terms 'duty' and 'delta'. Derive a relationship between the two.

6. What are the factors affecting duty ? How can duty be improved ?

7. Define the following :

G.C.A., C.C.A., Kor depth, Kor period, outlet factor, capacity factor, nominal duty, open discharge, Rabi and Kharif crops.

8. What do you understand by crop rotation ? What are its advantages ?

9. Discuss various methods of assessment of irrigation water.

10. A field channel has culturable commanded area of 2000 hectares. The intensity of irrigation for gram is 80 % and for wheat is 50 %. Gram has a kor period of 18 days and kor depth of 12 cm, while wheat has a kor period of 15 days and a kor depth of 15 cm. Calculate the discharge of the field channel.

[Ans. 1.624 cumecs]

11. The discharge available from a tube-well is $120 \text{ m}^3/\text{hour}$. Assuming 3200 hours of working for a tube-well in a year, estimate the culturable area that this tube-well can command. The intensity of irrigation is 50 % and the average depth of Rabi and Kharif crops is 48 cm.

[Ans. 160 hectares]

12. A water course command an irrigated area of 800 hectares. The intensity of irrigation of rice in this area is 50 %. The transplantation of rice crop takes 15 days and total depth of water required by the crop is 60 cm on the field during the transplantation period, given that the rain falling on the field during this period is 15 cm. Find the duty of irrigation water for the crop on the field during transplantation, at the head of the distributory, assuming losses of water to be 20 % in the water course. Also calculate the discharge required in the water course.

[Ans. (i) 217 hec/cumec on the field.

(ii) 172.8 hec/cumec on the head of W.C.

(iii) $Q = 2.32$ cumecs]

13. The left canal of a tank irrigation scheme carries a discharge of 10 cumecs and has a culturable commanded area of 800 hectares. The intensity of Rabi crop is 70 %

IRRIGATION AND WATER POWER ENGINEERING

96

and the base period is 110 days. The right canal of the scheme carries a discharge of 24 cumecs and has a culturable commanded area of 15000 hectares. The intensity of Rabi crop is 80 % and the base period is 110 days. Compare the efficiency of the two canal systems.

[Ans. Left canal more efficient]

14. What do you understand by the following terms :
Crop ratio, overlap allowance, capacity factor and full supply coefficient.

SECTION-II

WATER RESOURCES ENGINEERING

Chapters :

4. HYDROLOGY
5. GROUND WATER : WELL IRRIGATION
6. RESERVOIR PLANNING
7. DAMS-I : GENERAL
8. DAMS-II : GRAVITY DAMS
9. DAMS-III : ARCH AND BUTTRESS DAMS
10. DAMS-IV : EARTH AND ROCKFILL DAMS
11. DAMS-V : SPILLWAYS
12. DIVERSION HEADWORKS

Hydrology

4.1. HISTORY OF HYDROLOGY

Water is the prime necessity for all forms of life. Human civilisation has progressed from early era with utilisation of available water resources. Civilisation flourished wherever water was available, and it collapsed wherever water resources depleted. It is learnt from Vedic literature that ground water was abundantly utilised for use. Archaeological excavations at Mohan Jodaro reveal that people of Indus Valley (3000 B.C.) had good knowledge of ground water development through wells. Later, surface water resources available through rivers, lakes, ponds were developed. People of Vedic period had knowledge of the hydrologic cycle. As more and more water was needed, people learnt the principles of hydrology by experience and consequently, reservoirs were built to store rain water for use in the periods of dry season. Chanakya (300 B.C.) described the use of raingauge in his book 'Arthashastra'. Apart from Indus Valley, people of Nile Valley also had the knowledge of hydrology. A.K.Biswas in his book 'History of Hydrology' describes these developments in a greater detail. According to him : "It is interesting to know that the river Nile was dammed in 300 B.C. to divert its water in near by land. Around the same time a method was devised to record the fluctuations in the river Nile. A well developed water supply and drainage system existed in Indus Valley during 2750 B.C". Varahamihira's 'Brihatsamhita' (505-587 A.D) contains the descriptions of raingauges, wind vane and procedures for prediction of rainfall.

The Europeans started using the knowledge of hydrologic cycle much later—some time around 1500 A.D. Chow in his book 'Hand Book of Applied Hydrology (1964) classifies the history of hydrology into the following eight periods :

1. Prior to 1400 AD	: Period of speculation
2. 1400 to 1600	: Period of observation
3. 1600 to 1700	: Period of measurement
4. 1700 — 1800	: Period of experimentation
5. 1800 — 1900	: Period of modernization
6. 1900 — 1930	: Period of empiricism
7. 1930 — 1950	: Period of rationalisation.
8. 1950 — todate	: Period of theorization

In India, the period of empiricism may be taken to have started some time in the middle of nineteenth century and continued upto first quarter of twentieth century,

(99)

During this period, development of empirical formulae, tables and curves took place through investigators like Binnie, Barlow, Beale and Whiting, Strange, Ryves, Dickens, Inglis, Lacey, Nawab Jung Bahadur, Kanwar Sain etc.

4.2. THE HYDROLOGIC CYCLE

An irrigation engineer is not only concerned with the collection and distribution of water for irrigation, but it is also essential for him to know about the occurrence, distribution and movement of water on the earth. *Hydrology* is the science which deals with the occurrence, distribution and movement of water on the earth, including that in the atmosphere and below the surface of the earth. Water occurs in the atmosphere in the form of vapour, on the surface as water, snow or ice and below the surface as ground water occupying all the voids within a geologic stratum.

Except for the deep ground water, the total water supply of earth is in constant circulation from earth to atmosphere, and back to the earth. The earth's water circulatory system is known as the *hydrologic cycle*. Hydrologic cycle is the process of transfer of moisture from the atmosphere to the earth in the form of precipitation, conveyance of the precipitated water by streams and rivers to ocean and lakes etc., and evaporation of water back to the atmosphere. Fig. 4.1 illustrates, diagrammatically, the complete hydrologic cycle.

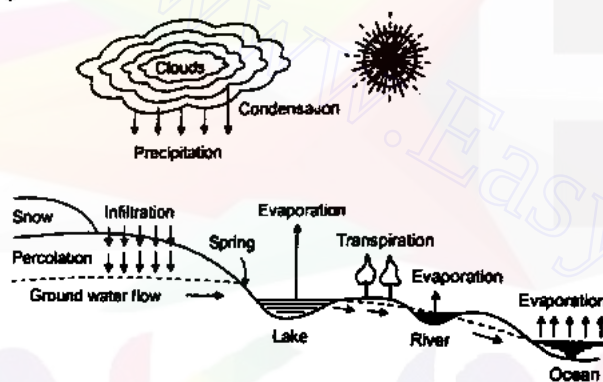


FIG. 4.1 THE HYDROLOGIC CYCLE.

The hydrologic cycle consists of the following processes :

1. Evaporation and Transpiration (E)

The water from the surfaces of ocean, rivers, lakes and also from the moist soil evaporates. The vapours are carried over the land by air in the form of clouds. *Transpiration* is the process of water being lost from the leaves of the plants from their pores. Thus, the total evaporation (E), inclusive of the transpiration consists of:

- Surface evaporation,
- Water surface evaporation: (a) From river surface; (b) From oceans,
- Evaporation from plants and leaves (transpiration), and
- Atmospheric evaporation.

2. Precipitation (P)

Precipitation may be defined as the fall of moisture from the atmosphere to the earth surface in any form. Precipitation may be two forms :

- Liquid precipitation* : i.e. rainfall.

- Frozen precipitation* : This consists of :

- Snow
- Hail
- Sleet
- Freezing rain

3. Run Off (R)

Run off is that portion of precipitation that is not evaporated. When moisture falls to the earth's surface as precipitation, a part of it is evaporated from the water surface, soil and vegetation and through transpiration by plants, and the remainder precipitation is available as run off which ultimately runs to the ocean through surface or sub-surface streams. Thus run off may be classified as follows :

(1) *Surface run off*. Water flows over the land and is first to reach the streams and rivers, which ultimately discharge the water to the sea.

(2) *Inter-flow or sub-surface run off*. A portion of precipitation infiltrates into surface soil and depending upon the geology of the basins, runs as sub-surface runoff and reaches the streams and rivers.

(3) *Ground water flow or base flow*. It is that portion of precipitation, which after infiltration, percolates down and joins the ground water reservoir which is ultimately connected to the ocean.

Thus, the hydrologic cycle may be expressed by the following simplified equation:

$$\text{PRECIPITATION} = \text{EVAPORATION} + \text{RUN OFF}$$

$$(P) = (E) + (R)$$

provided adjustment is made for the moisture held in storage at the beginning and at the end of the period.

4.3. METEOROLOGICAL DATA :

The hydrology of a place depends upon (i) climate (ii) topography and (iii) geology.

Climatic factors of importance are :

- Precipitation and its mode of occurrence
- Humidity
- Temperature
- Radiation
- Wind velocity

The *climate* is largely dependent on the geographical position of the place on earth's surface. *Topography* is important in its effect on precipitation and the occurrence of lakes, marshland and high and low rates of runoff. *Geology* of a place influences its topography, and it also affects the ground water zone.

HUMIDITY

Humidity is the state of the atmosphere in relation to the amount of water vapour it contains. Air easily absorbs moisture in the form of *water vapour*. The amount of moisture absorbed depends on the temperature – the higher the temperature, the more vapour the air can hold as shown in Fig. 4.2. Water vapour present in the atmosphere is very important to the hydrologist since the atmospheric water is the source of precipitation and it materially controls the rate of evaporation from land and water surfaces. The water vapour exerts a *partial pressure* (known as vapour pressure) which is usually measured in either *bars* (1 bar = 100 kN/m², 1 millibar = 10² N/m²) or mm height of column of mercury (1 mm Hg = 1.33 millibar)

METEOROLOGICAL OBSERVATION ARRAY : Now-a-days, a *meteorological observation array* available in the form of a *compact instrument* for making meteorological observations at regular, short time interval. It consists of the following :

- (1) On left lower arm : Instrument for measurement of net radiation.
- (2) On right lower arm : wet and dry bulb thermometers
- (3) On left lower : wind direction
- (4) On right upper arm : wind run
- (5) On top : solar and sky radiation
- (6) In the middle distance a rain gauge with anti-splash screen.

These instruments record automatically on magnetic tape, the various hydrological data collected by the array.

4.4. HYDROLOGICAL DATA

We have seen in §4.2 that the main components of the hydrologic cycle are (i) precipitation (ii) evaporation, including transpiration, and (iii) runoff, including direct runoff, infiltration, deep percolation and ground water runoff (effluent stream). Apart from information about the above components, information about climate, meteorology and geology is also required. The following *hydrological data* are required :

1. **Weather and climate records** : Data about temperature, humidity, radiation, wind etc, since these directly affect hydrological parameters.

2. **Precipitation data** : The study of precipitation forms a major portion of the subject of hydro-meteorology. The precipitation data helps in predicting runoff volume and its peak. It is also helpful in estimating the *water-budget-equation* for the basin. It gives indication to the further potential for expansion of water needs and plan accordingly.

3. **Stream flow data** : This helps in the planning of reservoirs, design of spillways, bridges, culverts, levees, and water power development and installation. The data is utilized for determination of maximum (peak) flood, flood magnitude, flood frequency, reservoir elevations, water budget etc.

4. **Evaporation and transpiration data** : This data is required for determining (i) water budget for the river basin, (ii) reservoir capacity for water resources development and (iii) evaporation suppression technique.

5. **Infiltration characteristics of the area** : This data helps in determining rainfall excess and run off computation.

6. **Ground water characteristics** : This helps in estimation and location of ground water reservoir, for ground water development.

7. **Physical and Geological characteristics** : This helps in the determination of runoff pattern and silt load movement. *Indian Meteorological Department (IMD)* is largely responsible for the collection for meteorological data, along with some State Government agencies. The Central Water Commission (CWC) monitors the stream flow data of major rivers.

4.5. THE WATER BUDGET

(a) **Global Water Budget** : Water is the most important natural resource which is vital for all forms of life on the earth. Water is a crucial element in development

planning. The total quantity of water in the World is roughly 1360 million cubic kilometers ($M km^3$; $1 M km^3 = 10^{12} m^3$) out of which about 97.2 % is held up in seas and oceans while about 2.1 % is frozen in ice caps, and about 0.31% is available as deep ground water. Thus about 99.6% of total water is of no use to man. Table 4.1 gives the inventory of earth's water.

TABLE 4.1. ESTIMATED EARTH'S WATER INVENTORY

Location	Water Volume, $M km^3 (= 1 \times 10^{12} m^3)$	% of Total Water
1. Fresh water (lakes)	0.125	0.621
2. Rivers	0.00125	
3. Soil moisture	0.065	
4. Ground water (a) < 0.8 km deep (b) > 0.8 km deep	4.125 4.125	
5. Saline lakes and inland seas	0.105	0.008
6. Atmosphere	0.013	0.001
7. Polar ice caps, glaciers and snow	29.2	2.1
8. Seas and snow	1320.0	97.25
Total	1360 $M km^3 (= 1.360 \times 10^{12} m^3)$	100.00

(b) **India's Water Budget** : India has a geographical area of nearly 3.3 million square kilometers. Normal annual rainfall varies from 100 mm in Western Rajasthan to over 11000 mm at Cherrapunji in Meghalaya. The annual average rainfall over the country is of the order of 1170 mm depth which is nearly $4000 km^3$. Of the total precipitation, a part goes towards increasing ground water storage, a part is lost as evapo-transpiration and the remaining appears as surface water.

The average flow in the river system of the country has been estimated to be $1880 km^3$. The recent estimates made by Central Water Commission (CWC) indicate that the water resources utilisable through surface structures is about $690 km^3$ only, which is about 36% of the total. The preliminary estimates of the Central Ground Water Board indicate that the utilisable ground water is about $450 km^3$. According to Nag and Kathpatia, total moisture in atmosphere has been estimated as 770 $M ha-m$ during June to September and 340 $M ha-m$ in the rest of the period, out of which about 25 to 30 % moisture comes back as rainfall over the sub-continent.

4.6. PRECIPITATION

To the hydrologist, precipitation is the general term for all forms of moisture emanating from the clouds and falling to the ground. The following are the essential requirements for precipitation to occur :

1. Some mechanism is required to cool the air sufficiently to cause condensation and droplet growth.

2. Condensation nuclei are also necessary for formation of droplets. They are usually present in the atmosphere in adequate quantities.

3. Large scale cooling is essential for significant amount of precipitation. This is achieved by lifting of air. Thus a meteorological phenomenon of lifting of air masses

is essential to result precipitation.

4.6.1. Types of Precipitation. Precipitation is often classified according to the factors responsible for lifting. Broadly speaking, there are four types of precipitation:

- (1) Cyclonic precipitation. (2) Convective precipitation.
- (3) Orographic precipitation. (4) Precipitation due to turbulent ascent.

1. Cyclonic Precipitation : Cyclonic precipitation results from lifting of air masses converging into low pressure area of cyclone. The cyclonic precipitation may be divided into (a) frontal precipitation, and (b) non-frontal precipitation.

(a) Frontal Precipitation. A border region between two adjacent air masses having different characteristics such as temperature and humidity is called a *front*. When a flow of warm and moist air mass from the south meets cold air mass of polar region, the cold air being heavier, under run the warm air flow in the form of flat wedge forcing the warm air aloft. The lifted warm air mass cools down at high altitudes, causing precipitation. A front may be warm front or cold front depending upon whether there is active or passive ascent of warm air mass over cold air mass.

(b) Non-frontal Precipitation. In the case of non-frontal precipitation, the moist warm air mass is stationary and the moving cold air mass meets it. Thus, due to lightness of the warm air there is passive ascent of warm air over cold air owing to the active under cutting. When the lifted warm air cools down at higher altitude precipitation occurs.

2. Convective Precipitation : Convective precipitation is caused by natural rising of warmer lighter air in colder, denser surroundings. The difference in temperature may result from unequal heating at the surface, unequal cooling at the top of the air layer, or mechanical lifting when air is forced to pass over a denser colder air masses. Convective precipitation is spotty and its intensity may vary from light showers to cloud bursts.

3. Orographic Precipitation : Orographic precipitation is due to the lifting of warm moisture laden air masses due to topographic barriers (such as mountains). As it reaches higher elevation, it comes in contact with cold air and precipitation occurs. The zone to the other side of the mountain will be the zone of rain shadow area where the rainfall may not occur, as shown in Fig. 4.3. All the precipitation we have in Himalayan region is because of the orographic ascent of air masses, rich in moisture content because of their long travel over oceans.

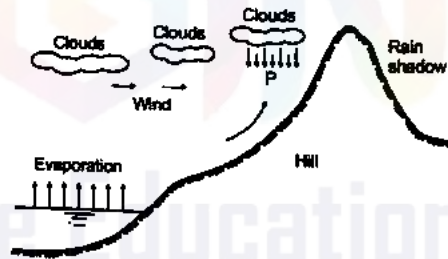


FIG. 4.3. OROGRAPHIC PRECIPITATION

4. Precipitation Due to Turbulent Ascent : Air mass is forced to rise up due to greater friction of earth surface after its travel over ocean. The air mass rises up because of increased turbulence and friction, when it ultimately condenses and precipitation occurs. Winter rainfall in Madras State is mainly due to this process.

In nature, the effects of these various types of cooling are often inter-related and the resulting precipitation cannot be identified as being of any one type.

4.6.2. Forms of Precipitation

The various forms of precipitation are : Drizzle, rain, glaze, sleet, snow, snow flakes and hail.

Drizzle : When the size of water droplets is under 0.5 mm, and its intensity is < 1 mm per hour. Because of the lightness, the droplets appear to be floating in air.

Rain : When the size of the drops is more than 0.5 mm. The upper size of water drop is generally 6.25 mm, as drops greater than this tend to break up as they fall through the air.

Glaze : When the drizzle or rain freezes as it comes in contact with cold objects, it is known as glaze.

Sleet : It is frozen rain drops cooled to the ice stage while falling through air at sub-freezing temperature.

Snow : Precipitation in the form of ice crystals resulting from sublimations (i.e. water vapour changed directly to ice).

Snowflakes : Number of ice crystals fused together form snow flakes.

Hail : Hail is lumps or bulbs of ice over 5 mm diameter formed by alternate freezing or melting as they are carried up and down in highly turbulent air currents.

4.6.3. Rainfall in India

The major rainfall season in India is from June to October. India lies in the tropical belt (i.e. North of Equator) and has the following four distinct weather periods:

- (i) Monsoon period (ii) Post-monsoon period
- (iii) Winter rainfall period and, (iv) Summer rainfall period.

(i) Monsoon period (June to October) : A major part of India (excluding Jammu Kashmir and South Eastern parts) gets more than 75% of rainfall due to monsoon winds which originate in the Indian ocean and proceed to the sub-continent. Fig. 4.4 gives the dates of onset (shown by solid lines) and dates of withdrawals (shown by dotted lines) of monsoons in different parts of India. The monsoon winds advancing from south-western direction breaks up into two distinct branches : (i) Bay of Bengal branch and (ii) Arabian sea branch. The Arabian sea branch touches Kerala while the Bay of Bengal branch reaches the north-eastern states of India, practically at the same time (i.e. 1st June) and causes heavy rainfall. The Arabian sea branch moves northwards causing heavy precipitation in western ghats while the Bay of Bengal branch moves westward, causing rain in West bengal, Orissa,

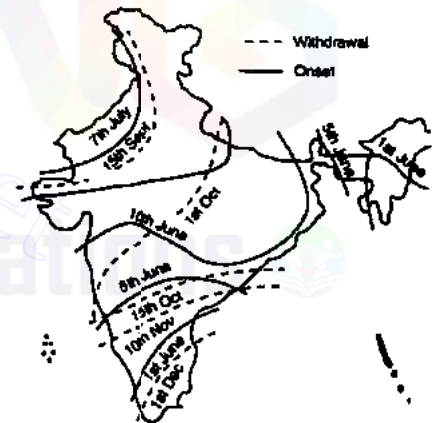


FIG. 4.4. DATES OF ONSET AND WITHDRAWAL OF MONSOON RAINFALL IN INDIA.

Bihar, U.P. and Delhi. Central India, Punjab, Haryana and Himachal Pradesh get rain from the Arabian sea branch. Both the branches form a low pressure region, known as *monsoon trough* extending from Bay of Bengal to Rajasthan. The total rainfall in various parts of India vary widely — 200-400 cm in north-eastern states, 200-400 cm in western ghat regions, 120-160 cm in Orissa and West Bengal and 100-120 cm in Punjab, U.P. and Haryana region.

(ii) **Post monsoon period (Oct-Nov.)** : The south-western monsoon retreats in October and November and at the same time north-eastern wind becomes active, causing rain. In addition, tropical cyclones are formed in both Bay of Bengal as well as Arabian sea, resulting in intense rainfall that cause heavy loss of life, specially in the south eastern states of Indian Peninsula. Light rain is also caused in the south eastern part of south peninsula, due to winds blowing over Bay of Bengal and touching the south eastern part.

(iii) **Winter rainfall period (December-February)** : Eastern parts of southern India get winter rainfall due to low pressure pockets created in Bay of Bengal. During the same period, western disturbances of extra-tropical origin travel eastward, causing precipitation in the form of rain and showers in northern region of north India, including Jammu and Kashmir.

(iv) **Summer rainfall period (March-May)** : Summer rainfall, also called *pre-monsoon showers*, occurs in parts of eastern coast and north-eastern states, due to localized convective currents. Pre-monsoon rainfall also takes place in West Bengal, Orissa and Andhra Pradesh, due to *cyclonic storms*, caused due to low pressure pockets developed over Bay of Bengal.

4.7. MEASUREMENT OF RAIN FALL

Rainfall is the source of all water used for irrigation purposes and, therefore, a knowledge of its amount, character, seasons or periods and the effects produced by it is of prime importance to all whose duty is to design, carry out, improve, or maintain irrigation works. The amount of precipitation is expressed as the depth in centimetres (or inches) which falls on a level surface, and is measured by *rain-gauge*. The following are the main types of rain-gauges used for measurement of rainfall :

1. **Non-automatic rain-gauge**. This is also known as non-recording rain-gauge. Symon's rain-gauge is the instrument prescribed by use at all Government rain-gauge stations throughout India.

2. **Automatic rain-gauge**. These are integrating type recording rain-gauges and are of three types :

- (i) Weighing bucket rain-gauge.
- (ii) Tipping bucket rain-gauge.
- (iii) Float type rain-gauge.

1. **Symon's Rain-gauge** : Symon's rain-gauge is most common type of non-automatic rain-gauge, and is used by Meteorological Department of Government of India. As shown in Fig. 4.5, it consists of cylindrical vessel 127 mm (or 5") in diameter with a base enlarged to 210 mm (or 8") diameter. The top section is a funnel provided with circular brass rim exactly 127 mm (5") internal diameter. The funnel shank is inserted in the neck of a receiving bottle which is 75 to 100 mm (3 to 4") diameter. A receiving

bottle of rain-gauge has a capacity of about 75 to 100 mm of rainfall and as during a heavy rainfall this quantity is frequently exceeded, the rain should be measured 3 or 4 times in a day on day of heavy rainfall lest the receiver fill should overflow. A cylindrical graduated measuring glass is furnished with each instrument, which reads to 0.2 mm. The rainfall should be estimated to the nearest 0.1 mm.

The rain-gauge is set up in a concrete block 60 cm × 60 cm × 60 cm (2' × 2' × 2'), as shown in Fig. 4.5. The rim should be 305 mm (12") above the surface of the ground. The following important points should be kept in mind while selecting the site for a rain-gauge station :

1. The site where a rain-gauge is set up should be an open place.
2. The distance between the rain-gauge and the nearest object should be at least twice the height of the object. In no case should it be nearer to the obstruction than 30 metres.
3. The rain-gauge should never be situated on the side or top of a hill if a suitable site on a level ground can be found.
4. In the hills, where it is difficult to find level space, the site for the gauge should be chosen where it is best shielded from high winds, and where the wind does not cause eddies.
5. A fence, if erected to protect the gauge from cattle etc. should be so located that distance of the fence is not less than twice its height.

2. Weighing Bucket Type Rain-gauge

Self recording gauges are used to determine rates of rainfall over short periods of time. The most common type of self-recording gauge is the *weighing bucket type*, as shown in Fig. 4.6.

The weighing bucket rain-gauge essentially consists of a receiver bucket supported by a spring

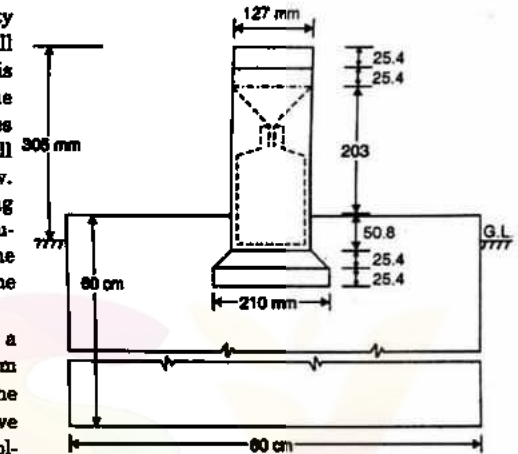


FIG. 4.5 SYMON'S RAIN-GAUGE (ADOPTED BY METEOROLOGICAL DEPT. OF GOVT. OF INDIA)

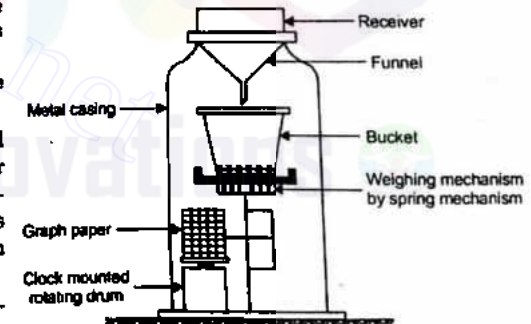


FIG. 4.6. WEIGHING BUCKET TYPE RAIN-GAUGE.

or lever balance or any other weighing mechanism. The movement of the bucket due to its increasing weight is transmitted to a pen which traces the record on a clock-driven chart.

3. Tipping Bucket Type Rain-gauge

A Steven's tipping bucket type rain-gauge consists of 300 mm diameter sharp edge receiver (Fig. 4.7). At the end of the receiver is provided a funnel. A pair of buckets are pivoted under the funnel in such a way that when one bucket receives 0.25 mm (0.01 inch) of precipitation it tips, discharging its contents into a container bringing the other bucket under the funnel. Tipping of the bucket completes an electric circuit causing the movement of pen to mark on clock driven revolving drum which carries a record sheet. The electric pulses generated due to the tipping of the buckets is recorded at the control room far away from the rain gauge station.

4. Float (or Syphon) Type Rain-gauge

The working of a float or syphon type rain-gauge is similar to the weighing bucket type gauge. A funnel receives the rain water which is collected in a rectangular container. A float is provided at the bottom of the container. The float is raised as the water level rises in the container, its movement being recorded by a pen moving on a recording drum actuated by a clock work. When the water level in the container rises so that the float touches the top, the syphon comes into operation, and releases the water; thus all the water in the box is drained out.

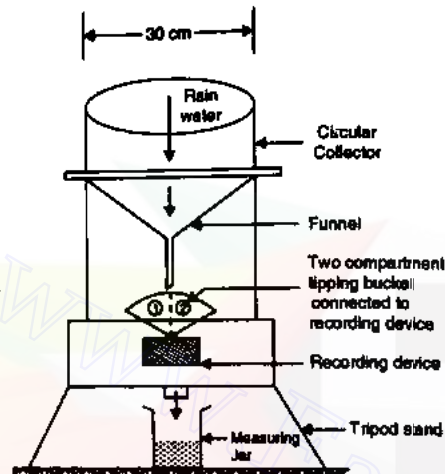


FIG. 4.7. TIPPING BUCKET TYPE RAIN-GAUGE.

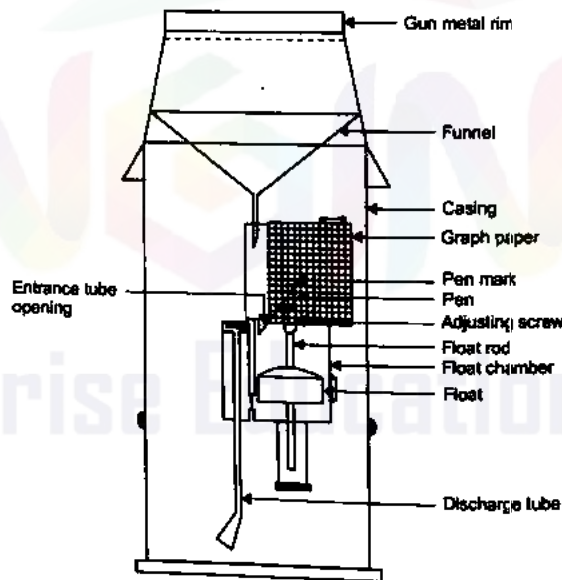


FIG. 4.8. FLOAT TYPE AUTOMATIC RAIN-GAUGE.

Advantages of Recording Rain-gauges

Following are the advantages of recording type rain-gauge over the non-recording type :

1. The rainfall is recorded automatically and therefore, there is no necessity of any attendant.
2. The recording rain-gauge also gives the intensity of rainfall at any time while the non-recording gauge gives the total rainfall in any particular interval of time.
3. As no attendant is required such rain-gauge can be installed in far-off places also.
4. Possibility of human error is obviated.

Disadvantages

1. It is costly in comparison with non-recording gauge.
2. Fault may develop in electrical or mechanical mechanism or recording the rainfall.

Sources of Errors in Recording the Measurements

1. Mistakes in reading the scale of the gauge.
2. Some amount of water will be displaced by the measuring stick of water which may creep up the stick; this may increase the error by 1 %.
3. Dents in the collector rim may change its receiving area.
4. Funnel and inside surface require about 2.5 mm of rain to get moistened when the gauge is initially dry. This may amount to the extent of 25 mm per year in some areas.
5. There is some loss resulting from rainfall splash from the collector.
6. The most serious error is the deficiency of measurements due to wind. Vertical acceleration of air forces upwards over a gauge gives an upward acceleration to precipitation about to enter the gauge and results in deficient catch.
7. Inclination of gauge may cause lesser collection. A 10% inclination gives about 1.5 % low catch.

Some more Errors Associated with Tipping Bucket

8. There is difficulty in the measurement of jogs on a record paper due to their close vicinity during heavy precipitation.
9. When the intensity is not the same as at the time of calibration of the jog, the record may show higher or lower intensity according to the variation in precipitation.
10. Tipping of the buckets may be affected due to rusting or accumulation of dirt at the pivot.
11. No rainfall is recorded during tipping of the bucket.

Microwave Radar for Measurement of Rainfall

A microwave radar (1 to 20 cm wavelength) can be used to obtain, roughly, information on rainfall distribution within its scanning area. The image on the radar screen can be interpreted as an approximate indication of rainfall intensity, since the

amount of reflected energy is dependent on the raindrop size and the distance from transmitter and the drop size is roughly correlated with rain intensity. A calibration may also be made from actual rain-gauge measurements in the area scanned by the radar.

Totaliser : A *totaliser*, used mainly for measuring rainfall for hydrologic purposes, consists of a large container with a funnel like opening which can hold rain catch of large quantity. It is mostly used to measure rainfall at a remote place, at a large interval of time, say one week, one fortnight or a month. To record snow as precipitation at a remote location, a storage gauge or stand pipe gauge is used. Such a stand pipe or storage gauge is normally of 305 mm in diameter, with enough height, with an antifreeze chemical (such as calcium chloride solution) to melt the snow. Since precipitation is measured at intervals, a thin layer of oil is spread to prevent evaporation.

4.8. RAINGAUGE NETWORK

It is absolutely essential to design a proper network of raingauges in a given catchment (water shed) to collect the necessary precipitation data. The *raingauge density* or *network density* is defined as the ratio of total area of catchment to the total number of gauges in the catchment. A question that frequently arises concerns the *number* and *type* of gauges that are necessary to ensure accurate assessment of a catchment's rainfall. The World Meteorological Organisation (WMO) has laid down the following norms for *minimum network density* (Table 4.2)

TABLE 4.2

Region	Description	Network density	
		Minimum	Tolerable
I	Flat region of temperate, mediterranean and tropical zones	1 gauge for 600 to 900 km ²	1 gauge for 900 to 3000 km ²
II	Mountainous areas of temperate, mediterranean and tropical zones	1 gauge for 100 to 250 km ²	1 gauge for 250 to 1000 km ²
III	Arid and Polar zones	1 gauge for 1500 to 10000 km ²	—

Ten percent of these gauges should be of recording type to enable the determination of rainfall intensities.

IS Code Recommendations

IS : 4987-1968 has recommended the following densities :

- One gauge per 520 km² in plain areas, with denser network for the areas lying in the path of low pressure systems.
- One gauge per 260 to 390 km² in regions with average elevation of 1000 m above mean sea level.
- One gauge per 130 km² in predominantly hilly regions with heavy rainfall, higher density being preferred wherever possible.

It is also recommended that 10% of the gauges are of recording type. The network arrived at based on the above recommendations should be so located that all the gauges will have more or less equal Thiessen weights.

The optimum No. of rain gauge stations (N), as per ISI is given by

$$N = \left[\frac{C_v}{p} \right]^2 \quad \dots(4.3)$$

where C_v = coefficient of variation of the rainfall values of existing stations.

$$= \frac{s_x}{\bar{x}} \times 100 \quad \dots(4.4)$$

s_x = standard deviation

$\bar{x}$ = mean of rainfall values of existing stations

p = desired degree of error in estimating mean rainfall.

Both C_v and p should be expressed as percentage. If $N < n$, where n is the number of existing stations, the existing network estimates the average depth of rainfall with an error less than allowable value p and no more gauges are required. If, however, $N > n$, the number of additional rain gauge stations will be $(N - n)$, and these should be distributed in different zones (caused by isohyets) in proportion to their areas, i.e. depending upon the spatial distribution of the existing raingauge stations and the variability of the rainfall over the basin.

Saturated network

In important projects, rainfall has to be estimated with greater accuracy. Hence p will be small and thus N will be large. The net work of raingauge stations should be so set that any addition of raingauge stations will not appreciably alter the average depth of rainfall estimated. Such a network is known as *saturated net work*.

Example 4.1. A catchment has five raingauge stations. In a year, the annual rainfall recorded by the gauges are 78.8 cm, 90.2 cm, 98.6 cm, 102.4 cm and 70.4 cm. For a 6 % error in the estimation of the mean rainfall, determine the additional number of gauges needed.

Solution

$$\bar{x} = \frac{1}{n} \sum x_i = \frac{78.8 + 90.2 + 98.6 + 102.4 + 70.4}{5} = 88.08 \text{ cm.}$$

The standard deviation of the rainfall is given by

$$s_x^2 = \frac{\sum (x_i - \bar{x})^2}{n - 1} = \frac{(78.8 - 88.08)^2 + (90.2 - 88.08)^2 + (98.6 - 88.08)^2 + (102.4 - 88.08)^2 + (70.4 - 88.08)^2}{(5 - 1)} = 179.732$$

$$s_x = 13.41$$

$$\text{Hence } C_v = \frac{s_x}{\bar{x}} \times 100 = \frac{13.41}{88.08} \times 100 = 15.22$$

$$N = \left(\frac{C_v}{p} \right)^2 = \left(\frac{15.22}{6} \right)^2 = 6.43 \approx 7$$

Additional number of stations = $7 - 5 = 2$

4.9. ESTIMATION OF MISSING RAINFALL DATA

Sometimes, it may not be possible to measure the rainfall at a particular measuring station due to absence of the observer or instrument failure or any other reason. In that case, the prediction of the missing data can be made with the help of available data of nearby measuring stations, using the following prominent methods :

1. Arithmetic mean method
2. Normal ratio method
3. Inverse distance method by U.S. Weather Service.

1. Arithmetic mean method : According to this method, the missing rainfall P_x of the station X is computed by simple arithmetic average of the rainfall at the nearby stations (known as *index stations*) in the following form :

$$P_x = \sum_{i=1}^n P_i / n = \frac{1}{n} (P_1 + P_2 + \dots + P_n) \quad \dots(4.5)$$

where n = number of index stations.

The above method is used only under the following conditions :

- (i) The normal annual rainfall of the missing station is within 10% of the normal annual rainfall of the index stations.
- (ii) Data of at least three index stations should be available.
- (iii) The index stations should be evenly spaced around the missing station and should be as close as possible.

2. Normal ratio method : In this method, the rainfall (P_i) of the surrounding index stations are weighed by the ratio of normal annual rainfalls by using the following equation:

$$P_x = \frac{1}{n} \left[P_1 \frac{N_x}{N_1} + P_2 \frac{N_x}{N_2} + \dots + P_n \frac{N_x}{N_n} \right] = \frac{N_x}{n} \left[\frac{P_1}{N_1} + \frac{P_2}{N_2} + \dots + \frac{P_n}{N_n} \right] \quad \dots(4.6)$$

where $N_1, N_2, \dots, N_n$ = normal annual rainfall of index stations.

N_x = normal annual rainfall of missing station

n = number of index stations.

The above method is used when the normal annual precipitation of the index stations differ more than 10% of the missing station. Here also, the data of atleast three index stations should be available, and all these index stations should be evenly spaced.

3. Inverse distance method (U.S. Weather Service method)

In this method a set of rectangular co-ordinate axes are passed through the missing rain gauge station so that its co-ordinates are (0,0). The co-ordinates (x_i, y_i) of each index station, surrounding the missing station are found. The weightage (W_i) of each index station is represented by the inverse of the square of its distance from the missing station, and is given by

$$W_i = \frac{1}{D_i^2} = \frac{1}{x_i^2 + y_i^2} \quad \dots(4.7a)$$

The missing rainfall data of the station X is then computed from the following

$$\text{equation : } P_x = \frac{\sum_{i=1}^n P_i W_i}{\sum_{i=1}^n W_i} \quad \dots(4.7)$$

The above method gives good results and is therefore the most acceptable method for scientific analysis. However, the limitation of the method is that it estimates missing rainfall between the highest and the lowest values of the index stations.

Example 4.2. A watershed has four rain gauge stations, A, B, C and D. During a storm, rain gauge station A was inoperative, while stations B, C and D, surrounding station A, recorded rainfall of 48 mm, 51 mm and 45 mm respectively. Estimate the missing storm precipitation of station A, using arithmetic mean method.

Solution

$$\text{From Eq. 4.5} \quad P_A = \frac{1}{3} (P_B + P_C + P_D) = \frac{1}{3} (48 + 51 + 45) = 48 \text{ mm}$$

Example 4.3. A precipitation station X was inoperative for some time during which a storm occurred. The storm totals at three stations A, B and C surrounding X , were respectively 6.60, 4.80 and 3.70 cm. The normal annual precipitation amounts at stations X, A, B and C are respectively 65.6, 72.6, 51.8 and 38.2 cm. Estimate the storm precipitation for station X.

Solution : If N_X, N_A, N_B and N_C are the average annual precipitation amounts at X, A, B and C and P_A, P_B and P_C are the storm totals of stations A, B and C surrounding X, the storm precipitation P at station X is given by.

$$P_X = \frac{1}{3} \left[\frac{N_X}{N_A} \times P_A + \frac{N_X}{N_B} \times P_B + \frac{N_X}{N_C} \times P_C \right] \quad \dots(1)$$

$$P_X = \frac{1}{3} \left[\frac{65.6}{72.6} \times 6.6 + \frac{65.6}{51.8} \times 4.80 + \frac{65.6}{38.2} \times 3.70 \right] = \frac{1}{3} (18.33) = 6.11 \text{ cm}$$

Example 4.4. In a river basin a station A was inoperative during a storm, while stations B, C, D and E, surrounding A were in operation, registering 74 mm, 88 mm, 71 mm and 80 mm of precipitation. The co-ordinates of B, C, D and E are (9,6), (12, -9), (-11, -6) and (-7, 7) respectively, with co-ordinates of A as (0,0). Estimate the missing storm precipitation of station A by the inverse distance method.

Solution:

$$D_b^2 = (9)^2 + (6)^2 = 117 ; W_b = 1/D_b^2 = 1/117 = 8.547 \times 10^{-3}$$

$$D_c^2 = (12)^2 + (-9)^2 = 225 ; W_c = 1/D_c^2 = 1/225 = 4.444 \times 10^{-3}$$

$$D_d^2 = (-11)^2 + (-6)^2 = 157 ; W_d = 1/D_d^2 = 1/157 = 6.369 \times 10^{-3}$$

$$D_e^2 = (-7)^2 + (7)^2 = 98 ; W_e = 1/D_e^2 = 1/98 = 10.204 \times 10^{-3}$$

$$\therefore P_x = \frac{\sum P_i W_i}{\sum W_i} = \frac{74 \times 8.547 + 88 \times 4.444 + 71 \times 6.369 + 80 \times 10.204}{8.547 + 4.444 + 6.369 + 10.204} = 77.5 \text{ mm}$$

4.10. COMPUTATION OF AVERAGE RAINFALL OVER A BASIN

In order to compute the average rainfall over a basin or catchment area, the rainfall is measured at a number of rain-gauge stations suitably located in the area. The network density of rain-gauge depend upon the uses for which the rainfall data is intended. A network should be so planned as to have a representative picture of the areal distribution of rainfall. There should be no concentration of gauges in heavy rainfall areas at the expense of dry areas, or *vice versa*.

If a basin or catchment area contains more than one rain-gauge station, the computation of average precipitation or rainfall may be done by the following methods:

1. Arithmetic average method.
2. Thiessen polygon method.
3. Isohyetal method.
4. Grid point method

1. Arithmetic Average Method

If the rainfall is uniformly distributed on its areal pattern, the simplest method of estimating average rainfall is to compute the arithmetic average of the recorded rainfall values at various stations. Thus, if $P_1, P_2, P_3, \dots, P_n$ etc., are the precipitation or rainfall values measured at n gauge stations, we have

$$P_{av} = \frac{P_1 + P_2 + \dots + P_n}{n} = \frac{1}{n} \sum_{i=1}^n P_i \quad \dots(4.8)$$

The procedure is illustrated in Table 4.3.

TABLE 4.3. ARITHMETIC AVERAGE METHOD

Station No.	Precipitation (mm)	Average Precipitation
1	12.6	
2	18.8	
3	14.8	
4	10.4	
5	16.2	
Sum	$\Sigma P = 72.8$ mm	$P_{av} = \frac{72.8}{5} = 14.6$ mm

2. Thiessen Polygon Method

The arithmetic average method is most approximate method since rainfall varies in intensity and duration from place to place. Hence rainfall recorded by each rain-gauge station should be weighed according to the area it is assumed to represent.

Thiessen polygon method is a more common method of weighing the rain-gauge observations according to the area. Thiessen polygon method is also called *weighted mean method* and is more accurate than the arithmetic average method.

Procedure (Fig. 4.9)

1. Join the adjacent rain-gauge stations, A, B, C, D, etc., by straight lines.
2. Construct the perpendicular bisectors of each of these lines.
3. A Thiessen network is thus constructed. The polygon formed by the perpendicular bisectors around a station encloses an area which is everywhere closer to that station than to any other station. Find the area of each of these polygons shown hatched in Fig. 4.9.

4. Multiply the area of each Thiessen polygon by the rain-gauge value of the enclosed station.

5. Find the total area (ΣA) of the basin.

6. Compute the average precipitation or rainfall from the equation :

$$P_{av} = \frac{A_1 P_1 + A_2 P_2 + \dots + A_n P_n}{A_1 + A_2 + \dots + A_n} = \frac{\Sigma (A \times P)}{\Sigma A} \quad \dots(4.9)$$

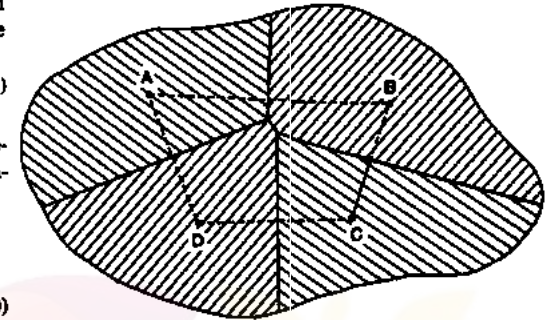


FIG. 4.9 THIESSEN POLYGON METHOD.

$$P_{av} = P_1 W_1 + P_2 W_2 + \dots + P_n W_n \quad \dots(4.9 a)$$

where $W_1, W_2, \dots, W_n$ are Thiessen weights computed as $W_1 = A_1 / \Sigma A$,

$W_2 = A_2 / \Sigma A, \dots, W_n = A_n / \Sigma A$, such that $W_1 + W_2 + \dots + W_n = 1.00$

The calculations are illustrated in a Table below :

TABLE 4.4. THIESSEN POLYGON METHOD

Rain gauge station	Area of Thiessen Polygon (A)	Precipitation (P)	A × P
A	45 sq. km	30.8 mm	1386
B	38 sq. km	34.6 mm	1315
C	30 sq. km	32.5 mm	978
D	40 sq. km	24.5 mm	974
Sum	153 sq. km		4663

$$P_{av} = \frac{\Sigma A \times P}{\Sigma A} = \frac{4663}{153} = 30.5 \text{ mm}$$

Defects of Thiessen polygon method

1. A new Thiessen diagram is required if there is an addition or deletion of rain gauge station for the basin.
2. No account is taken of the orographic features of the catchment.
3. The assumption of linear variation of precipitation between two stations is not desirable since the precipitation is influenced by a large number of meteorological and catchment characteristics.
4. The method does not consider topographic features and other barriers.
3. Isohyetal Method

The basic assumption in the Thiessen Polygon method is that a rain-gauge station best represents the area which is close to it. However, this may not always be valid,

specially when the rainfall is controlled by topography or results from intense convection. The Isohyetal method is the most elaborate and accurate in such conditions.

An *Isohyet* is a line, on a rainfall map of the basin, joining places of equal rainfall readings. An *isohyetal map* showing contours of equal rainfall presents a more accurate picture of the rainfall distribution over the basin.

Procedure

1. From the rainfall values recorded at various rain-gauge stations, prepare the isohyetal map for the storm causing the rainfall over the area.

2. Measure the areas enclosed between successive isohyets with the help of planimeter.

3. Multiply each of these areas by the average rainfall between the isohyets.

4. The average rainfall is then computed from the expression :

$$P_{av} = \frac{\sum A \times \frac{P_1 + P_2}{2}}{\sum A} \quad \dots(4.10)$$

The calculations are illustrated in a Table below :

TABLE 4.5. ISOHYETAL METHOD

Isohyets (cm)	Area between Isohyets (A) (Sq. km.)	Average precipitation $\frac{1}{2}(P_1 + P_2)$	Product $A \times \frac{(P_1 + P_2)}{2}$
9			
	22	9.5	209
10			
	80	10.5	840
11			
	105	11.5	1208
12			
	98	12.5	1225
13			
	78	13.5	1063
14			
	16	14.5	232
15			
Sum	399		4767

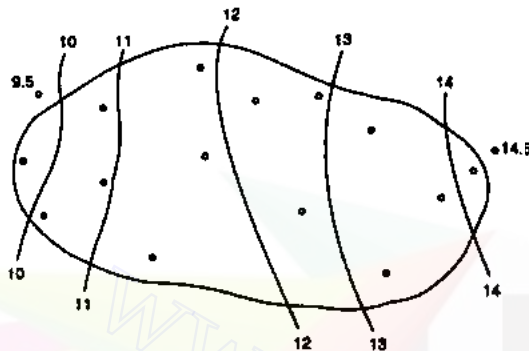


FIG. 4.10. ISOHYETAL METHOD.

$$P_{av} = \frac{\sum A \times \frac{P_1 + P_2}{2}}{\sum A} = \frac{4767}{399} = 11.92 \text{ cm.}$$

Isohyetal method gives better results than the previous two methods, provided isohyets are drawn by an experienced hydrologist with due consideration of the topographical and other influences. However, the major demerit of the method is that a separate isohyetal map is required to be drawn for each storm. If monthly rainfall is to be computed by the isohyetal method, then for each month of each year, one set of isohyetal map is to be drawn. Due to this, the work is increased drastically. However, since this method takes into account the topographical features, it is largely used to compute the average precipitation during storms for flood studies of various projects.

4. Grid point method

In this method, all the grid stations are marked on the map of the basin, along with the depth of precipitation. On this map, drawn to a suitable scale, a uniform rectangular grid at some suitable spacing is super-imposed. At the corners of the grid points, precipitation is estimated suitably by considering the weather and storm characteristics of the basin. The average precipitation of the four grid corners is then multiplied by the area of the grid, to give the average precipitation volume of the grid. The sum of all such products divided by the total area of the basin will give the average precipitation for the basin.

Mean Annual rainfall

Mean annual rainfall is usually calculated by taking the simple average of total rainfall of several consecutive years. Great variations are recorded in the annual rainfall at the same station, and a rainfall record for considerable number of years is necessary to get a reliable estimate of mean annual rainfall at any place. The greater is the number of the years considered, the more correct will be the mean value so obtained. From the study of rainfall records of a large number of different places extending over many years, it has been found that 35 years rainfall records are required for a reliable estimate of the mean annual rainfall of any place. Sir Alexander Binnie has shown that if the available rainfall records are for a period lesser than 35 years, there will be error in the computed mean rainfall, as given by Table 4.6.

TABLE 4.6

Period in years	5	10	15	20	25	30	35
Percentage error in mean rainfall	14.9	8.2	4.8	3.3	2.8	2.3	1.8

Example 4.5. Fig. 4.11 shows the map of Cauvery basin with rainfall observations in cm of water marked at various rain gauge stations. Compute the average rainfall by (1) Arithmetic average method (2) Isohyetal method and (3) Thiessen Polygon method.

Solution

(i) **Arithmetic Average Method:** It is clear from Fig. 4.11 that stations having gauge

readings as 53, 81 and 63 are located outside the basin. Hence they are not included in taking the arithmetical average.

$$\therefore \text{Arithmetic mean} = \frac{1}{9} (58 + 61 + 69 + 56 + 84 + 86 + 69 + 79 + 71) = 70.34 \text{ cm.}$$

$$\therefore \text{Average rainfall} = 70.34 \text{ cm.}$$

(ii) **Isohyetal method:** The isohyets are drawn, just as contours, as shown in Fig. 4.11. Area between adjacent isohyets are determined by a planimeter. The calculations are shown in Table below

TABLE 4.7 (a)

Isohyet (cm)	Area enclosed between Isohyets (A) cm ²	Average Precipitation $\frac{1}{2} (P_1 + P_2)$	Product = $A \times \frac{(P_1 + P_2)}{2}$
86			
	0.43	86.5	36.80
85			
	5.20	82.5	429.00
80			
	4.00	77.5	310.00
75			
	5.04	72.5	365.40
70			
	5.85	67.5	395.00
65			
	4.53	62.5	283.00
60			
	4.08	57.5	235.00
55			
	1.27	52.5	66.80
50			
Sum	30.41		2121.00

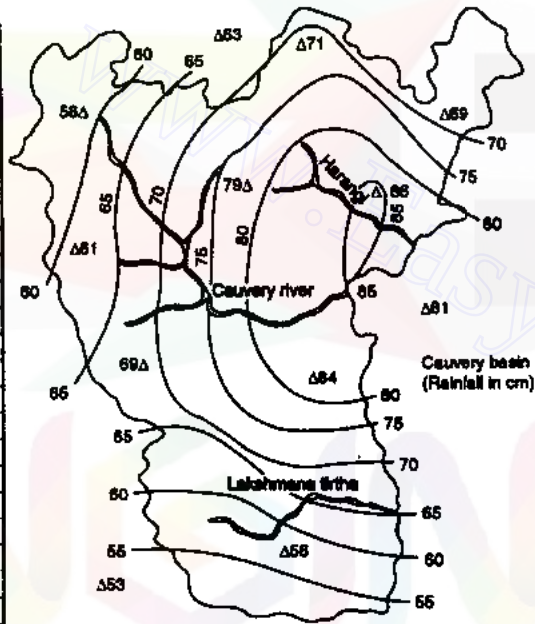


FIG. 4.11.

$$P_{av} = \frac{\sum A \frac{(P_1 + P_2)}{2}}{\sum A} = \frac{2121.0}{30.41} \approx 69.7 \text{ cm}$$

(iii) **Thiessen Polygon Method:** The polygons are drawn as explained in § 4.10 and are shown in Fig. 4.12. Area of each polygon is determined by planimeter. The calculations are shown in Table below.

TABLE 4.7 (b)

Area of Thiessen Polygon A (cm ²)	Observed precipitation P (cm)	Product A × P
3.26	58	189.0
0.39	63	24.6
1.61	71	114.2
2.04	69	140.8
2.48	66	211.5
0.84	61	68.0
3.91	84	328.5
5.09	56	285.0
0.41	53	21.7
3.94	69	272.0
2.06	61	125.7
4.40	79	347.5
Sum	30.41	2324.3

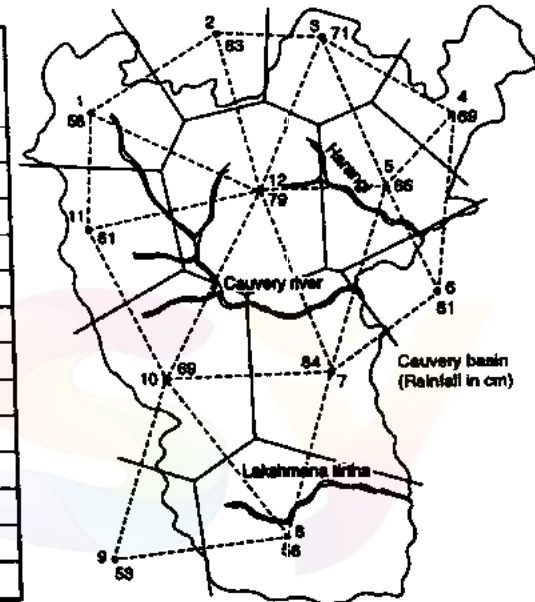


FIG. 4.12

$$\text{Hence } P_{av} = \frac{\sum AP}{\sum A} = \frac{2324.3}{30.41} = 76.4 \text{ cm.}$$

4.11. PRESENTATION OF RAINFALL DATA

There are three methods of presentation of rainfall data collected through measurements at a given station :

- Hyetograph method
- Mass curve of rainfall method
- Point rainfall method

1. Hyetograph Method

A hyetograph is a bar graph showing the intensity of rainfall with time as shown in Fig. 4.13. The hyetograph can be prepared either from the mass curve of rainfall, or directly from the data obtained from automatic raingauges.

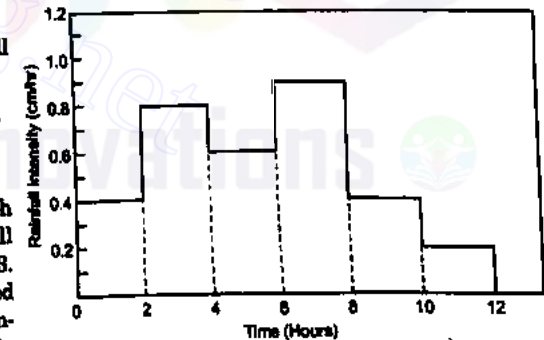


FIG. 4.13. HYETOGRAPH OF STORM

2. Mass Curve of Rainfall

A mass curve of rainfall is a plot of cumulative depth of rainfall against time, as shown in Fig. 4.14. The steepness of the curve indicates the intensity of rainfall. A horizontal portion of the curve indicates that there was no rainfall during that period.

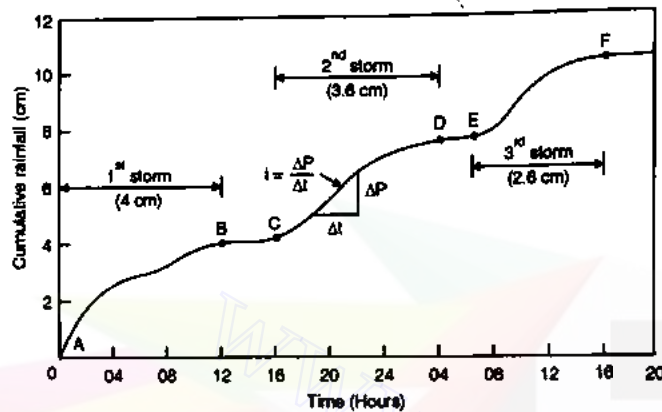


FIG. 4.14. MASS CURVE OF RAINFALL

Thus, in Fig. 4.14, A to B is the first period of rainfall. There was no rainfall from B to C, and therefore the curve is horizontal. C to D shows the second period of rainfall, while E to F shows third period of storm. The mass curve of rainfall is a rising curve. The intensity of rainfall during any period is given by

$$i = \frac{\Delta P}{\Delta t} \quad \dots(4.11)$$

A mass curve of rainfall is very useful in extracting the information on the duration and magnitude of a storm. The mass curve of a design storm can be obtained by maximising the mass curves of the severe storms in the basin.

3. Point Rainfall Method : The rainfall data of a station is known as *point rainfall* or *station rainfall*. This data can be presented as daily, weekly, monthly, seasonal or annual values. The point rainfall data is presented graphically as plots of magnitudes vs chronological time in the form of a bar diagram. Point precipitation data are used collectively to estimate the areal variability of rainfall. It is also used in deriving the intensity-duration-frequency curves.

4.12. INTERPRETATION OF RAINFALL DATA

Generally, the precipitation information may be required under the following headings:

(i) Intensity (i)	(ii) Duration (t)
(iii) Frequency (f)	(iv) Areal extent

Intensity of rainfall : The intensity of rainfall (i) is defined as the rate at which rainfall occurs, and is expressed as cm/h or mm/h. The non-recording type raingauge, however, measures/records only rainfall depth (P) in a day, or in a duration (t) of the rainfall. In that case, the intensity of rainfall is given by

$$i = \frac{P}{t}$$

in which i is the rainfall intensity, P is the rainfall amount and t is the duration of rainfall. However, in the case of recording type raingauge, the duration and depth of rainfall both are recorded simultaneously in the form of *mass curve* on the raingauge recording chart.

The intensity of rainfall is classified into the following three types :

(i) Light intensity rainfall	: 2.5 mm/h
(ii) Moderate intensity rainfall	: 2.5 to 7.5 mm/h
(iii) Heavy intensity	: > 7.5 mm/h

Frequency of rain fall : The frequency of rainfall of a *specified period* is determined assuming that the rainfall is a random variable and the mathematical theory of probability is applicable. The design of hydraulic structures, such as flood control structures, soil conservation structures, waste-water systems, drains and culverts etc. is based on the probability of the occurrence of extreme rainfall and therefore the run off.

(a) Frequency of Point Precipitation : The *frequency* of a rainfall is the number of time that a given magnitude of rainfall may occur in a given period. The study of the *probability* of occurrence of a particular extreme rainfall (such as 24-h maximum rainfall) is of extreme importance to the determination of design flood. This is determined with the help of *frequency analysis*. For such an analysis, the point rainfall data at a place are arranged in a chronological order to constitute a *time series*. For example, let us assume that we have maximum 24-h rainfall data for 30 years, recorded at a station. The highest value of 24-h rainfall depth in the set occurs once in 30 years, or it's frequency of occurrence is 30 years. Again, the second highest 24-h rainfall depth occurs twice in the record of 30 years, and hence it has a frequency of 15 years, and so on for other data. The *recurrence interval* is the interval in years for occurrence of the event of the same magnitude and is the reciprocal of the frequency.

Thus the *recurrence interval*, also known as *return period*, is given by

$$T = \frac{1}{P_n} \quad \dots(4.12)$$

where P_n is the probability of occurrence.

For the frequency analysis, various events are arranged in descending order of magnitude, and each event is assigned an order number m , with $m = 1$ for the first entry, $m = 2$ for the second entry and so on, till the last entry (event) has $m = N =$ number of years of record.

The probability P_n of an event being equalled or exceeded is given by the following formulae :

$$(i) \text{ California formula : } P_n = \frac{m}{N} \text{ or } T = \frac{N}{m} \quad \dots(4.13)$$

$$(ii) \text{ Hazen formula : } P_n = \frac{2m-1}{2N} \text{ or } T = \frac{2N}{2m-1} \quad \dots(4.14)$$

$$(ii) \text{ Weibull formula : } P_n = \frac{m}{N+1} \text{ or } T = \frac{N+1}{m} \quad \dots(4.15)$$

Out of these, Weibull formula is most widely used.

Fig. 4.15 shows a log log plot between rainfall and frequency. From the above plot, rainfall magnitude of specific duration for any recurrence interval can be estimated.

Frequency (f): The probability of occurrence of an event, expressed as a percentage is known as frequency (f). Thus

$$f = 100 P_m = \frac{1}{T} \times 100 \quad \dots(4.16)$$

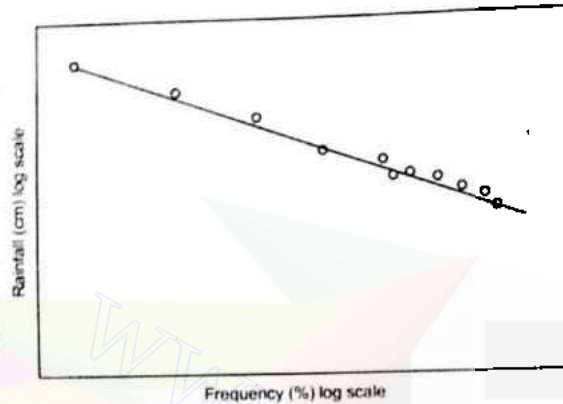


FIG. 4.15 RAINFALL FREQUENCY CURVE

We shall discuss here the following :

1. Intensity duration analysis
2. Intensity-duration-frequency relationship
3. Depth area relationship
4. Depth area duration relationship.

1. INTENSITY DURATION ANALYSIS

It has generally been observed that greater the intensity of rainfall, shorter is the length of time it continues. As the duration of storm increases, the maximum intensity of storm decreases. Sherman gave the following relation between intensity and duration :

$$i = \frac{a}{(t + b)^n} \quad \dots(4.17)$$

where i = intensity of rainfall (cm/hr)
 t = time in min. (time duration)

a, b, n = constants to be determined for the area.

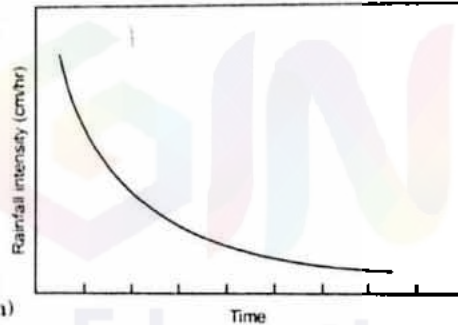


FIG. 4.16 INTENSITY DURATION CURVE

The curve (Fig. 4.16) represented by Eq. 4.17 will be hyperbolic. Paulhus suggests that if rainfall is plotted against duration in a log-log scale, the World's greatest recorded rainfalls lie on or just under a straight line whose equation is

$$R = 16.6 D^{0.475} \quad \dots(4.18)$$

where R = rainfall (inches), D = duration (hours)

The World's highest recorded intensities are of the order of 40 mm in a min., 200 mm in 20 min., and 26 m in a year.

2. INTENSITY DURATION FREQUENCY RELATIONSHIP

It is observed that a storm of any given duration will have larger intensity if its return period is large. This means that for storm of given duration, storms of higher intensity in that duration are rarer than storms of smaller intensity. The relationship between intensity, duration and return period can be expressed by the following

$$i = \frac{K T^x}{(t + b)^n} \quad \dots(4.19)$$

where x, b and n = constants for the catchment
 T = return period (years)

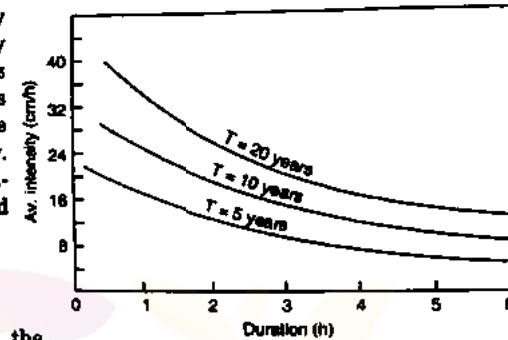


FIG. 4.17. INTENSITY DURATION FREQUENCY CURVES

Typical values of the constants K, b, n and x (Eq. 4.19) are given in Table 4.8. These values are based on studies conducted by the Central Soil and Water Conservation Research and Training Institute, Dehradun.

TABLE 4.8. TYPICAL VALUES OF CONSTANTS K, b, n and x

S.N.	Place in India	Values of constants			
		K	b	n	x
1.	Bellary	8.16	0.5	0.694	0.972
2.	Bhopal	6.93	0.5	0.189	0.878
3.	Chandigarh	5.82	0.4	0.160	0.750
4.	Nagpur	11.45	1.25	0.156	1.032
5.	Raipur	4.68	0.15	0.139	0.928

Sherman gave the following relations :

$$i = \frac{K T^x}{t^n} \quad \dots(4.20)$$

$$\text{and, } i = \frac{40 T^{0.27}}{(t + T)^{0.7}} \quad \dots(4.21)$$

Fig. 4.17 shows the plot between i, T and t , expressed by Eq. 4.20. These relationships will be represented by straight line when plotted on log log scale.

Depth duration frequency curves

When intensity of rainfall, given by Eq. 4.19, is converted into the depth of rainfall (i.e. depth = $i \times t$), and results

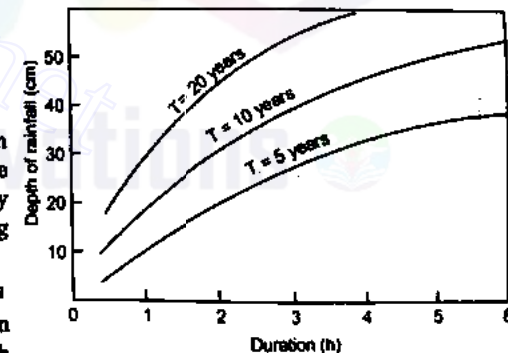


FIG. 4.18. DEPTH-DURATION-FREQUENCY CURVES

are plotted, we get depth duration frequency curves of the form shown in Fig. 4.18.

3. DEPTH AREA RELATIONSHIP

The areal distribution characteristics of a storm of given duration is reflected in the depth area relationships and depth area curves. Precipitation rarely occurs uniformly over the whole area. For a rainfall of a given duration, the average depth of rainfall decreases from the *maximum* (or highest) value as the considered area increases. The relationship is expressed by the following equation :

$$\bar{P} = P_n \varepsilon^{-kA^n} \quad \dots(4.22)$$

where P_n = highest amount of rainfall observed at the storm centre

A = area (km^2)

$\bar{P}$ = average depth in cm, over the area

k, n = constants

The above relationship is shown in Fig. 4.19

On the basis of 42 severest storms in North India, Dhar and Bhattacharya (1975) gave the following values of k and n for storms of different duration (Table 4.9).

TABLE 4.9. TYPICAL VALUES OF k and n

Duration of storm	k	n
1 day	8.258×10^{-4}	0.6614
2 days	9.877×10^{-4}	0.6306
3 days	1.746×10^{-3}	0.5961

4. DEPTH AREA DURATION CURVES

Rainfall rarely occurs uniformly over the whole of the catchment. Variations in total depth of rainfall and intensity occur from the centres to the peripheries of storms. The knowledge of maximum depth of rainfall occurring on areas of various sizes for storms of different durations is of great importance in many hydrological design problems. The development of relationship between maximum depth, area and duration for a region is known as DAD analysis. DAD analysis is performed to determine the maximum amounts of precipitation of various durations over areas of various sizes.

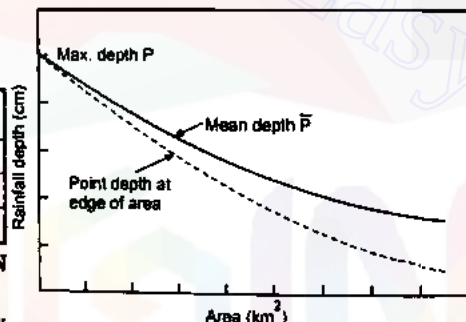


FIG. 4.19. DEPTH AREA CURVES OF RAINFALL

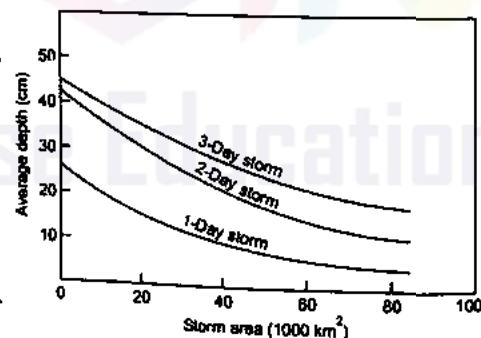


FIG. 4.20. DAD CURVES

DOUBLE MASS CURVE OF RAINFALL

Double mass curve is a technique used for checking the consistency of a record. The trend in the rainfall records at a station may slightly change after some years due to change in the environment of a station, or tampering of the instrument or shift in the observation practices. The change due to meteorological factors will equally affect all stations in the test and thus will not cause a lack of consistency created by the external effects.

A double mass curve is a graph plotted between the accumulated annual rainfall at a given station X (test station) versus accumulated annual values of the average of group of base stations, for various consecutive time periods, as shown in Fig. 4.21. A break in the slope of the resulting plot indicates a change in the precipitation regime of station X . The precipitation values at the station X , beyond the period of break are corrected by the following relationship :

$$P_{xc} = P_x \cdot \frac{a}{b} \quad \dots(4.23)$$

where P_{xc} = corrected precipitation at the test station X , at any time period t

P_x = recorded precipitation at the test station X , at time period t .

Example 4.6. The annual rainfall at station X and the average of annual rainfalls at 10 surrounding base stations in cm are given below for a period of 16 years, starting from 1970.

Year	Rainfall at X (cm)	10-Station Average (cm)	Year	Rainfall at X (cm)	10-Station average (cm)
1970	95	75	1978	75	75
1971	75	65	1979	65	65
1972	90	70	1980	70	90
1973	85	65	1981	87	66
1974	95	75	1982	67	68
1975	90	70	1983	62	67
1976	100	70	1984	55	62
1977	90	70	1985	69	77

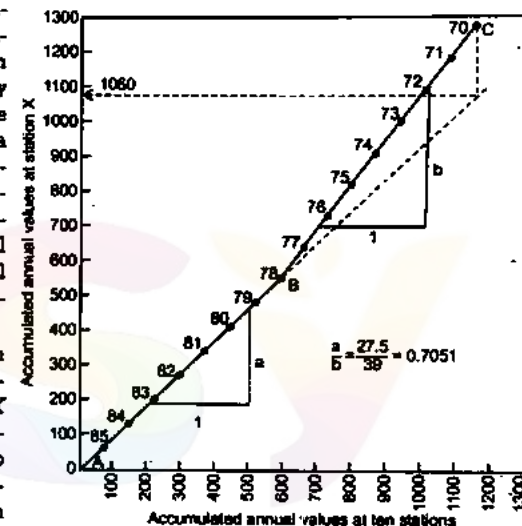


FIG. 4.21. DOUBLE MASS CURVE OF RAINFALL

The precipitation values at the station X , beyond the period of break are corrected by the following relationship :

- (i) Check whether the data at station X is consistent.
- (ii) In which year a change in regime is indicated ?
- (iii) Compute the adjusted annual rainfalls at X for the affected period.

Solution

Computations for cumulative rainfalls at X and at average of 10 surrounding stations are arranged in tabular form as under, starting from data of most recent year.

Year	Annual rainfall at X (cm)	Cumulative rainfall at X (cm)	Ao. of annual at 10 base stations (cm)	Cumulative rainfall of base stations (cm)
1985	69	69	77	77
1984	55	124	82	139
1983	62	186	67	206
1982	67	253	68	274
1981	87	340	86	360
1980	70	410	90	450
1979	65	475	65	515
1978	75	550	75	590
1977	90	640	70	660
1976	100	740	70	730
1975	90	830	70	800
1974	95	925	75	875
1973	85	1010	65	940
1972	90	1100	70	1010
1971	75	1175	55	1065
1970	95	1270	75	1140

The data are plotted as shown in Fig. 4.21

From Fig. 4.21, we observe that the graph is not having uniform slope. There is break at point B. Hence record at X is not consistent. Thus change in the regime is indicated in 1978 corresponding to point B.

From Fig. 4.21, we observe that $\frac{a}{b} = 0.7051$

Hence data beyond point B are to be corrected in the ratio a/b . Adjusted cumulative in 1970, at station X = 1060 cm.

$$P_x = P_r \cdot \frac{a}{b} = 0.7051 \cdot P_r$$

The corrected rainfall data at X are tabulated as under :

Year	Observed rainfall at X ($= P_x$)	Adjusted rainfall at X $P_x = 0.7051 P_r$
1970	95	67
1971	75	53
1972	90	63
1973	85	60
1974	95	67
1975	90	63
1976	100	71
1977	90	63

4.13. PROBABLE MAXIMUM PRECIPITATION (PMP)**(a) Probable maximum precipitation**

For the safe design of a hydraulic structure, even a 50 year or 100 year precipitation may be inadequate due to unusual meteorological combinations resulting in a very high precipitation. The *probable maximum precipitation*, commonly known as PMP, for a region is the precipitation resulting from the most critical meteorological combinations that are considered probable of occurrence. It can be defined as that rainfall over a basin which would produce the flood flow with virtually no risk of being exceeded. For the determination or estimation of PMP, the knowledge of an experienced hydro-meteorologist is required. Over the past 40 years, there has been an increasing use of the science of hydro-meteorology (i.e. science of meteorology in combination with hydrology), by engineers concerned with water resources and flood control. Hydrometeorology, has been largely developed in USA. The PMP can be estimated by maximising the different parameters like humidity, temperature, wind velocity and other connected meteorological factors of an observed severe storm over the basin. There are two approaches for determining PMP :

- (i) Meteorological approach
- (ii) Statistical approach

As has been pointed out in § 4.12, World's greatest observed point rainfall lie on or just under a straight line whose equation is

$$R = 16.6 D^{0.475} \quad \dots(4.18)$$

where R is the rainfall in inches and D is the duration in hours.

Extreme point rainfalls recorded in the British Isles, extracted from more than half a million station years of data, lie close to a line of equation

$$R = 106 D^{0.46} \quad \dots(4.24)$$

where R is in mm and D in hours.

From the statistical studies, PMP can also be estimated from the following equation

$$PMP = \bar{P} + K\sigma \quad \dots(4.25)$$

Where $\bar{P}$ = mean of annual maximum rainfall series

σ = standard deviation of the series

K = frequency factor, which is usually in the neighbourhood of 15

Maximum and minimum rainfall

The magnitudes of maximum rainfall and minimum rainfall within specified time period can be determined by the use of frequency formula given by Hazen :

$$T = \frac{2N}{2m-1} = \frac{N}{m-0.5} \quad \dots(4.26)$$

where T = recurrence interval within which the event is either equal to or greater than the specified amount.

and m = rank number assigned to the event.

Procedure for determination of maximum rainfall

1. Arrange the rainfall data in *descending* order and assign rank number (m) to each rainfall event, the total number of event being equal to N .
2. Compute recurrence interval (T) for each event, using Eq. 4.26.
3. Plot a graph between recurrence interval (T) as abscissa and corresponding rainfall as ordinate.
4. Determine the expected *maximum* rainfall magnitude for any desired value of T from this graph.

See Example 4.13 for illustration.

Procedure for determination of minimum rainfall

1. Arrange the rainfall data in *ascending* order and assign rank number (m) to each rainfall event, the total number of events being equal to N .
2. Compute recurrence interval (T) for each event, using Eq. 4.26.
3. Plot a graph between recurrence interval (T) as abscissa and corresponding rainfall as ordinate.
4. Determine the expected *minimum* rainfall magnitude for any desired value of T from this graph.

See Example 4.13 for illustration.

Isopluvial Maps : We have seen earlier that the standard IDF curves indicate extreme rainfall intensities of different duration for different frequencies (T). As an alternative, IMD (Indian Meteorological Department) has standardised *isopluvial maps*, indicating extreme point rainfall values of different durations and for different frequencies over the whole country. An *isopluvial line* is a line of *equal rainfall*, each for a particular duration and a particular frequency. The isopluvial maps for rainfall of duration 15 minutes, 30 min., 45 min, 1 hr., 3 hr, 6 hr, 9 hr, 12 hr, 15 hr, etc. with recurrence intervals of 2, 5, 10, 25, 50 years have been published by IMD as well as by CWC. From these maps, the total rainfall depth at a particular location, for different durations and return periods can be determined.

Example 4.7. The ordinates (in mm) of a rainfall mass curve for a storm, which commenced at 06.30 hours, recorded by self recording rain gauge at 15 minutes interval are as under:

0, 12.4, 22.1, 35.1, 52.7, 63.7, 81.9, 109.2, 123.5, 132.6, 143.3, 146 and 146.
Construct the hyetograph of this storm for a uniform interval of 15 minutes.

Solution

The computations are done in tabular form below, which is self explanatory. The corresponding rainfall hyetograph is shown in Fig. 4.22.

Time	Cumulative rainfall (mm)	Rainfall in successive 15 min. interval (mm)	Rainfall intensity (mm/hr)
06.30	0	-	-
06.45	12.4	12.4	49.6
07.00	22.1	9.7	38.8
07.15	35.1	13.0	52.0
07.30	52.7	17.6	70.4
07.45	63.7	11.0	44.0
08.00	81.9	18.2	72.8
08.15	109.2	27.3	109.2
08.30	123.5	14.3	57.2
08.45	132.6	9.1	36.4
09.00	143.3	10.7	42.8
09.15	146.0	2.7	10.8
09.30	146.0	0.0	0.0

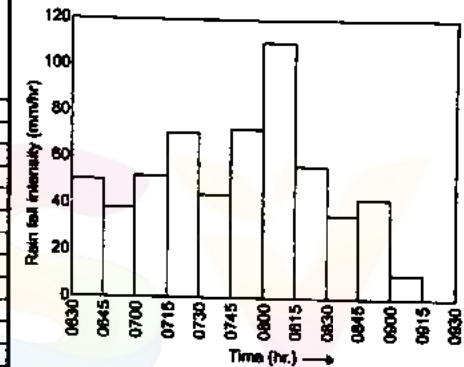


FIG. 4.22. RAINFALL HYETOGRAPH.

Example 4.8. For the storm of Example 4.7, compute the maximum rainfall intensities for durations of 15, 30, 45, 60, 90 and 120 minutes and plot the intensity duration graph.

Solution : The computations are done in Tabular form below.

Time	Cumulative Rainfall (mm)	Rainfall in any possible time interval of					
		15 min.	30 min.	45 min.	60 min.	90 min.	120 min.
06.30	0						
06.45	12.4	12.4					
07.00	22.1	9.7	22.1				
07.15	35.1	13.0	22.7	35.1			
07.30	52.7	17.6	30.6	40.3	52.7		
07.45	63.7	11.0	28.6	41.8	51.8		
08.00	81.9	18.2	29.2	48.8	59.8	81.9	
08.15	109.2	27.3	45.5	56.5	74.1	86.8	
08.30	123.5	14.3	41.6	59.8	70.8	101.4	123.5
08.45	132.6	9.1	23.4	50.7	68.9	97.5	120.2
09.00	143.3	10.7	19.3	34.1	61.4	90.6	121.2
09.15	146.0	2.7	10.4	22.5	36.8	82.3	110.9
09.30	146.0	0.0	2.7	13.4	22.5	64.1	88.9

In the above table, the bold figures denote the maximum rainfall depth for the respective durations. Thus, the maximum rainfall depth of 15 min, 30 min, 45 min, 60 min, 90 min and 120 min. durations are respectively 27.3, 45.5, 59.8, 74.1, 101.4 and 123.5 mm.

- $\therefore$ Max. intensity for 15 min. duration
 $= i_{15} = 27.3/0.25 = 109.2 \text{ mm/h}$
 Max. intensity for 30 min. duration
 $= i_{30} = 45.5/0.5 = 91 \text{ mm/h}$
 Max. intensity for 45 min. duration
 $= i_{45} = 59.8/0.75 = 79.7 \text{ mm/h}$
 Max. intensity for 60 min. duration
 $= i_{60} = 74.1/1 = 74.1 \text{ mm/h}$
 Max. intensity for 90 min. duration
 $= i_{90} = 101.4/1.5 = 67.6 \text{ mm/h}$
 Max. intensity for 120 min. duration
 $= i_{120} = 123.5/2 = 61.75 \text{ mm/h}$

Fig. 4.23 shows the max. intensity duration curve for the given storm.

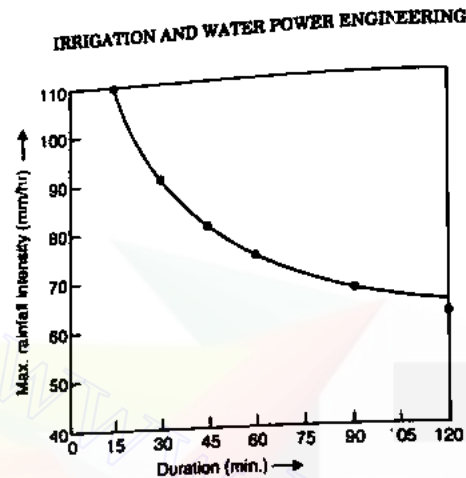


FIG. 4.23. INTENSITY DURATION CURVE

Example 4.9. The values of annual precipitation at a rain-gauge station in mm per year, in chronological sequence from 1972 to 1981 are indicated below : 475; 377; 731; 1066; 361; 305; 926; 628; 409; 236; 337 and 853.

Estimate the value of precipitation which has a recurrence interval of 5 years, using the probability or statistical method.

Solution

Total No. of years for which record is available = $N = 12$ years.

Recurrence interval = $T = 5$ years.

$\therefore$ Ranking of storm = $m = N/T = 12/6 = 2$

Hence we should pick up the 2nd severest storm. From the records, we find that the most severe storm is 1066 mm and the next severe storm is 926 mm. Hence the value of precipitation, which has a recurrence interval of six years is 926 mm.

Example 4.10. Using depth-area curve, estimate the average depth of precipitation that may be expected over an area of 1800 sq. km due to a storm of 15th July 1992, lasting for 24 hours, assuming the storm to be located at the centre of the area. The isohyetal map for the storm gave the following areas enclosed between different isohyets:

Isohyet (mm)	25	24	23	22	21	20	19	18	17	16
Enclosed area (sq. km)	407	1008	1522	1909	2216	2460	2651	2782	2910	2936

Solution : In order to plot the depth-area curve, let us first compute the equivalent-uniform depths (EUD) of rainfall (i.e. mean depth of rainfall) over increasing area of the basin. The computations are done in a tabular form below.

Isohyet (mm)	Area enclosed between isohyets and boundary of basin (km ²)	Net incremental area between isohyets (km ²)	Average rainfall on incremental area (mm)	Rainfall volume on incremental area (mm. km ²)	Cumulative volume (mm. km ²)	Equivalent uniform depth EUD (mm)
(1)	(2)	(3)	(4)	(5) = (3) × (4)	(6) = Σ(5)	(7) = (6)/(5)
25	407	407	25.5 (avg)	10378	10378	25.50
24	1008	601	24.5	14724	25102	24.90
23	1522	514	23.5	12079	37181	24.43
22	1909	387	22.5	8707	45888	24.04
21	2216	307	21.5	6600	52488	23.89
20	2460	244	20.5	5002	57490	23.37
19	2651	191	19.5	3724	61214	23.09
18	2782	131	18.5	2424	63638	22.87
17	2910	128	17.5	2240	65878	22.64
16	2936	26	16.5	429	66307	22.58

In the above Table, cols. (1) and (2) correspond to the given data. Column (3) gives the net incremental area between the isohyets, while col.(4) gives the average rainfall on the incremental areas. Column (5) gives the rainfall-volume over incremental areas and is equal to cols. (3) × (4). Col. (6) gives the cumulative volume upto the isohyets under consideration. Lastly, the EUD of Col. (7) is obtained by dividing Col. (6) by Col. (2).

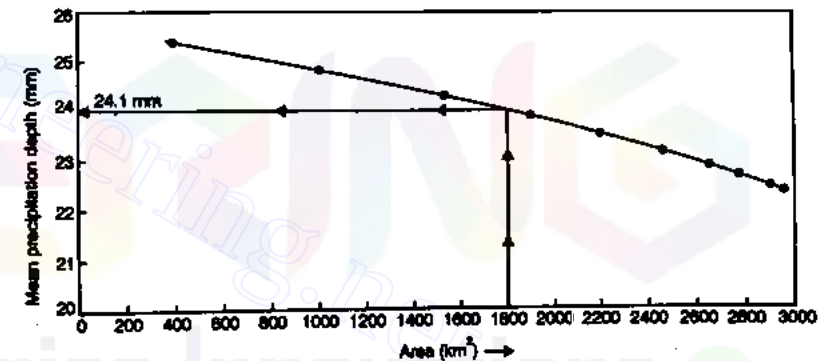


FIG. 4.24. DEPTH-AREA CURVE FOR 1 DAY RAINFALL.

Fig. 4.24 shows the plot between mean depth of rainfall and area enclosed between the isohyets and boundary of the basin. From the above depth-area curve, we obtain average depth of precipitation = 24.1 mm for an area of 1800 sq. km.

Example 4.11. Table below gives the recorded annual 24 h maximum rainfall data at a certain location, for a period of 20 years. Prepare the rainfall frequency curve and determine the following :

- (a) 24 h max. rainfall with return period of 8, 15 and 25 years recurrence interval.
 (b) 24 h max. rainfall with 40%, 24% and 8% probability.

What would be the probability of a rainfall of magnitude equal to or exceeding 100 mm occurring in 24 h at the location ?

Year	Rainfall (mm)	Year	Rainfall (mm)	Year	Rainfall (mm)	Year	Rainfall (mm)
1981	116	1986	64	1991	66	1996	95
1982	126	1987	102	1992	76	1997	108
1983	142	1988	92	1993	86	1998	71
1984	88	1989	78	1994	68	1999	80
1985	82	1990	73	1995	69	2000	65

Solution :

To start with, the values of rainfall are arranged in the decreasing order with their corresponding years as shown in the Table below. Column (3) of the table shows the ranking (m) of the storm. The probability of occurrence is computed from Weibull formula (Eq. 4.15) and entered in col. (4), while the recurrence interval ($T = 1/p_{ro}$) is entered in col. (5) of the Table. Here, $N = 20$.

Year	Rainfall mm	Ranking (m) of the storm	Probability (%) $p_{ro} = \frac{m}{N+1} \times 100$	Recurrence interval (years) $T = 1/p_{ro} = 100/p_{ro}$
(1)	(2)	(3)	(4)	(5)
1983	142	1	4.76	21.0
1982	126	2	9.52	10.5
1981	116	3	14.29	7.0
1997	108	4	19.05	5.25
1987	102	5	23.81	4.2
1996	95	6	28.57	3.5
1988	92	7	33.33	3.0
1984	88	8	38.10	2.63
1993	86	9	42.86	2.33
1985	82	10	47.62	2.1
1999	80	11	52.38	1.91
1989	78	12	57.14	1.75
1992	76	13	61.90	1.62
1990	73	14	66.67	1.5
1998	71	15	71.43	1.4
1995	69	16	76.19	1.31
1994	68	17	80.95	1.24
1991	66	18	85.71	1.17
2000	65	19	90.48	1.11
1984	64	20	95.23	1.05

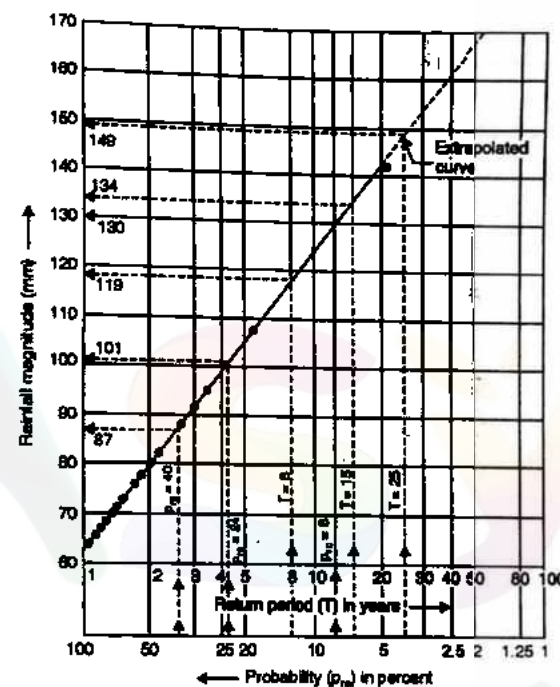


FIG. 4.25. FREQUENCY CURVE

The rainfall frequency curve is prepared by plotting the annual rainfall and corresponding probabilities on semi-log graph paper, shown in Fig. 4.25. Using this curve, the values of rainfall at the desired recurrence intervals and desired probabilities can be read out. The values so obtained are given in the Table below.

T (years)	Rainfall (mm)	p_{ro} (%)	Rainfall (mm)
8	119	40	87
15	134	24	101
25 (extrapolated)	149	8	130

Also, for rainfall = 100 mm, we get $T = 4$ years and $p_{ro} = 25\%$

Example 4.12. IDF Curves : Rainfall data, for rains of duration of 5 min., 10 min., 20 min., 30 min., 60 min., 90 min. and 120 min., at a certain station, available for a period of three decades were analysed and ten worst storms of each of these durations are shown in their decreasing order in Table below. Plot the IDF curves for storms of return periods of 10, 2, and 1 years. Use California formula.

5 min. duration		10 min. duration		20 min. duration		30 min. duration		60 min. duration		90 min. duration		120 min. duration	
Year	Rain (mm)	Year	Rain (mm)	Year	Rain (mm)	Year	Rain (mm)	Year	Rain (mm)	Year	Rain (mm)	Year	Rain (mm)
1955	10.6	1955	14.7	1955	19.3	1955	20.8	1955	25.5	1962	29.0	1962	34.7
1967	9.4	1962	12.8	1962	16.8	1962	18.6	1962	22.8	1951	28.1	1962	30.7
1962	9.1	1962	11.4	1962	15.2	1962	16.3	1951	20.2	1951	25.2	1951	27.3
1981	9.0	1951	10.8	1968	13.8	1968	14.6	1973	17.2	1968	21.3	1968	24.8
1976	8.2	1972	10.3	1972	13.2	1972	14.2	1966	16.6	1972	19.4	1972	21.4
1973	7.7	1981	9.8	1981	12.4	1978	13.2	1961	15.8	1961	17.7	1964	19.2
1978	6.3	1976	9.5	1976	12.0	1981	12.6	1977	14.8	1977	16.5	1961	18.1
1951	5.6	1978	8.3	1978	11.2	1976	12.1	1981	14.2	1964	16.0	1978	17.6
1962	4.5	1965	6.4	1965	10.0	1965	11.4	1956	13.5	1974	15.8	1974	17.1
1956	3.5	1959	6.2	1959	8.9	1959	10.0	1962	13.2	1956	15.0	1956	16.5

Solution : The return period for each event by the relation $T = N/m$, where $N = 10$, in the present case and tabulate the results as shown below.

in the present case and tabulate the results as shown below:								
Rank (m)	Rainfall during							$T = N/m$
	5 min. duration	10 min. duration	20 min. duration	30 min. duration	60 min. duration	90 min. duration	120 min. duration	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
1	10.6	14.7	19.3	20.8	25.5	29.0	34.7	10
2	9.4	12.8	16.8	18.6	22.8	28.1	30.7	5
3	9.1	11.4	15.2	16.3	20.2	25.2	27.3	3.33
4	9.0	10.8	13.8	14.6	17.2	21.3	24.8	2.5
5	8.2	10.3	13.2	14.2	16.6	19.4	21.4	2.0
6	7.7	9.8	12.4	13.2	15.8	17.7	19.2	1.67
7	6.3	9.5	12.0	12.6	14.8	16.5	18.1	1.43
8	5.6	8.3	11.2	12.1	14.2	16.0	17.6	1.25
9	4.5	6.4	10.0	11.4	13.5	15.8	17.1	1.11
10	3.5	6.2	8.9	10.0	13.2	15.0	16.5	1.0

The next step would be to compute the average intensity of rainfall, during each duration/storm, for the required return periods of 10, 2 and 1 years, and tabulate the results as shown below.

Duration t (min.)	P (mm)	$i = \frac{P}{t} \times 60$ (mm/hr.)	Duration t (min.)	P (mm)	$i = \frac{P}{t} \times 100$ (mm/hr.)	Duration t (min.)	P (mm)	$i = \frac{P}{t} \times 60$ (mm/hr.)
(a) T = 10 years			(b) T = 2 years			(c) T = 1 year		
5	10.6	127.2	5	8.2	98.4	5	3.5	42
10	14.7	88.2	10	10.3	61.8	10	6.2	37.2
20	19.3	57.9	20	13.2	39.6	20	8.9	26.7
30	20.8	41.6	30	14.2	28.4	30	10.0	20.0
60	25.5	25.5	60	16.6	16.6	60	13.2	13.2
90	29.0	19.3	90	19.4	12.9	90	15.0	10.0
120	34.7	17.4	120	21.4	10.7	120	16.5	8.3

Fig. 4.26 shows the intensity duration frequency (IDF) curves for $T = 10, 2$ and 1 years.

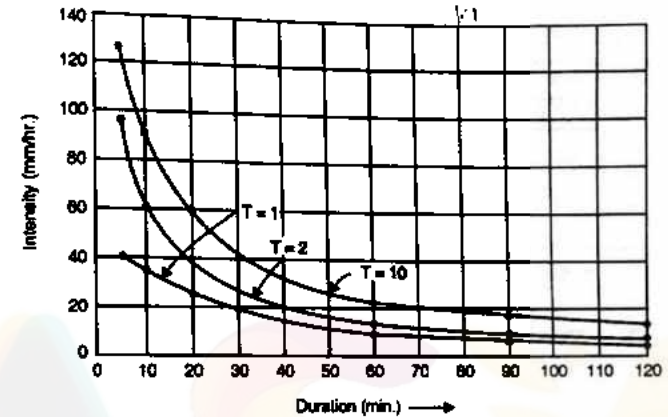


FIG. 4.26. IDF CURVES.

Example 4.13. Maximum and minimum rainfall : The rainfall data recorded at a rain gauge station are given below.

Year	Rainfall (cm)	Year	Rainfall (cm)	Year	Rainfall (cm)	Year	rainfall (cm)
1951	61	1956	30	1961	54	1966	82
1952	50	1957	46	1962	70	1967	78
1953	75	1958	52	1963	34	1968	58
1954	32	1959	72	1964	63	1969	56
1955	36	1960	40	1965	68	1970	65

Compute the values of maximum and minimum rainfall values at 15 years recurrence interval.

Solution : Here, $N = 20$

(a) Computation of maximum rainfall

For computation of maximum rainfall, arrange the rainfall data in descending order and assign rank number to each rainfall event. Using Hazen's formula, compute recurrence interval (T) for each event. Finally plot a curve between T (on x-axis) and rainfall depth (on y-axis), as shown in Fig. 4.27.

From this curve, we obtain max. rainfall of 79 cm for $T = 15$ years.

(b) Computation of minimum rainfall

For computation of minimum rainfall, arrange the rainfall data in ascending order and assign rank number to each rainfall event. Using Hazen's formula, compute recurrence interval T for each event. Finally, plot a curve between T (on x-axis) and rainfall depth (on y-axis), as shown in Fig. 4.28. From this curve, we obtain minimum rainfall of 31 cm for $T = 15$ years.

COMPUTATION OF MAXIMUM RAINFALL

Rainfall (cm)	Rank number (m)	$T = \frac{N}{m - 0.5}$
82	1	40
78	2	13.33
75	3	8
72	4	5.71
70	5	4.44
68	6	3.64
65	7	3.08
63	8	2.67
61	9	2.35
58	10	2.11
56	11	1.90
54	12	1.74
52	13	1.60
50	14	1.48
48	15	1.38
40	16	1.29
36	17	1.21
34	18	1.14
32	19	1.08
30	20	1.03

COMPUTATION OF MINIMUM RAINFALL

Rainfall (cm)	Rank number (m)	$T = \frac{N}{m - 0.5}$
30	1	40
32	2	13.33
34	3	8
36	4	5.71
40	5	4.44
46	6	3.64
50	7	3.08
52	8	2.67
54	9	2.35
56	10	2.11
58	11	1.90
61	12	1.74
63	13	1.60
65	14	1.48
66	15	1.38
70	16	1.29
72	17	1.21
75	18	1.14
78	19	1.08
82	20	1.03

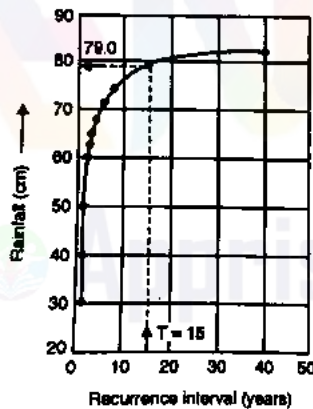


FIG. 4.27

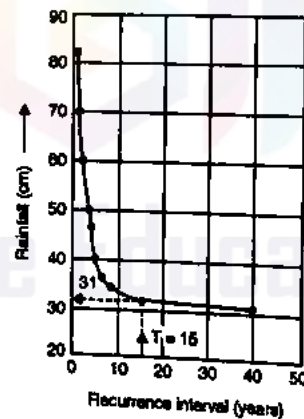


FIG. 4.28

4.14. LOSSES OR ABSTRACTIONS FROM PRECIPITATION

When precipitation takes place on land surface, whole of it is not available as runoff because of losses that take place during or after the precipitation. Important losses consist of (i) interception (ii) evaporation (iii) transpiration, (iv) infiltration, (v) depression storage, and (vi) watershed leakage. Out of these, evaporation, transpiration and infiltration are the major losses.

Thus, Precipitation - Surface runoff = Total loss.

where Total loss = Interception + Evaporation + Transpiration + Infiltration + Depression storage + Watershed leakage.

Interception : Interception may be defined as that amount of precipitation water which is intercepted by vegetative foliage, buildings and other objects lying over the land surface. Interception does not reach the land surface, but is returned back to the atmosphere by evaporation. This *interception storage* is the amount of precipitated water which wets the initially dried surface of the object lying above the ground surface. Various factors that affect interception are (i) storm factor (ii) plant-factor, (iii) season of the year, and (iv) prevailing wind.

Depression storage : A catchment area generally has many depressions of shallow depth and of varying size and shape. When precipitation takes place, water runs towards these depressions and fill them before actual overland flow or run off towards a stream takes place. The depression storage is therefore defined as the water retained in these depressions/ditches. The size of depression may vary from micro to large capacity. These depressions form miniature reservoirs detaining water temporarily. The water stored in these depressions partly evaporates and partly infiltrates into the ground to meet the ground water reservoir, if any. The following relationship may be used for computing the depression storage.

$$V_{ds} = K [1 - e^{-P_e/K}] \quad \dots(4.27)$$

where V_{ds} = Volume of water stored in surface depression.

P_e = Rainfall excess (volume of rainfall in excess of infiltration and interception).

K = Depression storage capacity of the basin.

The above relationship gives an exponentially decaying curve and attains value of K when the ratio P_e/K increases.

Following are the factors that affect depression storage :

(i) Land form, (ii) Soil characteristics (iii) topography, (iv) antecedent precipitation index, and (v) man made disturbance, like terrace farming etc.

Watershed leakage : Adjacent basins are separated by ridge lines, so that rainfall falling over a basin flows towards the drainage lines (i.e. streams etc.) of the basin. Watershed leakage may be defined as flow of water from one basin to another basin, or from one basin to the sea through major

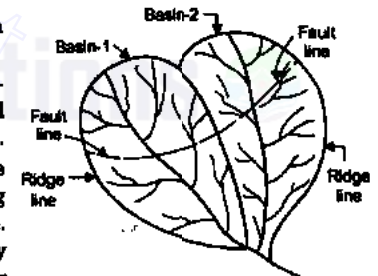


FIG. 4.29. WATERSHED LEAKAGE.

faults, fissures or other geographical features (Fig. 4.29). Due to these faults/fissures, underground hydraulic conduits so formed convey the discharge falling over a part of the catchment.

4.15. EVAPORATION

Evaporation and evapo-transpiration are the two most important phases of hydrologic cycle which redistribute the heat energy between surfaces and atmosphere. Evaporation is the process in which liquid changes to gaseous state at the free surface, below the boiling point through the transfer of heat energy. It is a continuous natural process by which a substance changes from liquid to gaseous state. The main source of evaporation is the solar radiation. For arid regions, the loss due to evaporation may be as high as 90% of the annual precipitation.

The rate of evaporation is dependent on the following factors :

- (i) Vapour pressure at the water surface and air above.
- (ii) Air and water temperatures
- (iii) Solar radiation
- (iv) Wind speed
- (v) Atmospheric pressure
- (vi) Nature and size of evaporating surface
- and (vii) Quality of water.

Dalton's law of evaporation

From the Dalton's law of evaporation, the rate of evaporation (E) can be related to vapour pressure as under :

$$E = C(e_s - e_a) + (a + b v)(e_s - e_a) \quad \dots(4.28)$$

where E = evaporation loss (mm/day)

e_s = saturation vapour pressure at water surface temperature in millibar.

e_a = actual vapour pressure of the air above.

C = a constant whose value depends upon various factors such as barometric pressure, wind velocity, humidity etc.

$$= a + b v \quad \dots(4.28 a)$$

a, b = constants

v = wind speed in km/h

For evaporation to continue, e_a should be less than e_s . Evaporation will be high if $(e_s - e_a)$ is large. Evaporation will be zero when $e_a = e_s$.

Factors affecting evaporation losses

Evaporation losses depend upon (i) Nature of evaporating surface (ii) Area of water surface (iii) Depth of water in the water body (iv) humidity (v) wind velocity, (vi) temperature (vii) atmospheric pressure and (viii) quality of water.

(i) **Nature of evaporating surface** : Different evaporating surfaces like soil, barren land, forest area, houses and lakes affect evaporation to the extent they have the potential. Black cotton soils help to evaporate the soil water faster than red soil because such soils have the potential to absorb incoming radiation more effectively. Evaporation from wet soil is faster and it reduces gradually as the soil becomes drier.

(ii) **Area of water surface** : The evaporation loss directly depends upon the area of the water surface — greater the area, greater will be the water loss due to evaporation.

(iii) **Depth of water in water body** : Deep water bodies evaporate slower than shallow water bodies in summer while in winter season, they evaporate faster.

(iv) **Humidity** : Evaporation is inversely proportional to humidity. If the humidity in the atmosphere is more, evaporation will be less.

(v) **Wind velocity** : Wind removes the overlying vapour from an evaporating body thereby increasing the rate of evaporation. However, high wind speed may not necessarily remove water vapour from a small water body. There is thus a relation between the wind speed and size of the water bodies or evaporating surface.

(vi) **Temperature of air** : Increase in air temperature increases the evaporation rate though not always proportionately. For the same temperature, colder months have less evaporation than summer months due to combined effect of other environmental parameters.

(vii) **Atmospheric pressure** : As per Dalton's law, evaporation will be less if the atmospheric pressure (e_a) is more. Thus, at higher altitudes, evaporation loss is more while in deep valleys, evaporation is less.

(viii) **Quality of water** : The presence of dissolved salts in water reduces the saturation vapour pressure (e_s) of water which consequently reduces the rate of evaporation. For example, the Bay of Bengal has salt concentration of 3.15%, and its evaporation rate is nearly 3% less than the evaporation from fresh water.

Estimation of evaporation from surfaces of water bodies : The following are prominent methods of estimation of evaporation loss from surfaces of large water bodies :

- (i) Measurement using evaporation pans
- (ii) Use of empirical equations.
- (iii) Water budget method
- (iv) Energy budget method

1. Measurement by evaporation pans : The most reliable method for the estimation of evaporation from large water bodies is that by measurements from evaporation pans.

Fig. 4.30 shows one such evaporation pan recommended by IS : 5973-1970. This evaporimeter is a modified version of US Weather Bureau Class-A pan. The pan, 1.22 m in diameter and 0.255 m deep, is made of 0.9 mm thick copper sheet with hexagonal wire netting of galvanised iron mesh covering it to protect water from birds, and also to make the water temperature more

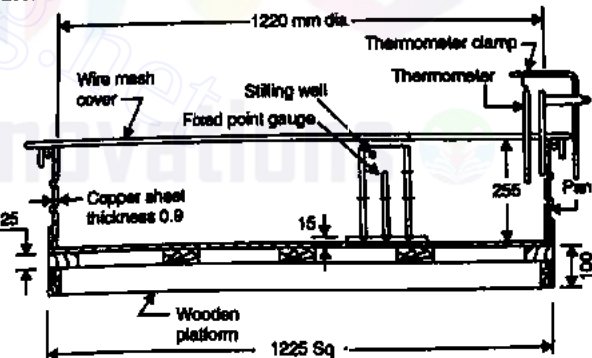


FIG. 4.30. INDIAN STANDARD EVAPORATION PAN

uniform during day and night. The pan placed over a square wooden platform of 1.225 m width and 10 cm height, so that circulation of air is possible all around the pan. This also helps to thermally insulate the pan completely from ground. Water level in the pan is recorded by a point gauge arrangement placed inside a stilling basin. Measurement is taken atleast once a day by adding water to the pan by a calibrated cylindrical glass jar to bring the water level to the previous position. This gives directly the evaporation depth over the time lapse. Similarly, if there is rainfall exceeding the depth of evaporation, then water is taken out of the pan in the same way by measuring jar and knowing the depth of rainfall from the rain gauge, the evaporation depth is found by subtraction.

The evaporation recorded by a pan differs from that of a lake or reservoir due to the following reasons :

- Depth of exposure of pan above ground.
- Colour of the pan
- Height of the rim.
- Heat storage and heat transfer capacity with respect to reservoir
- Pan diameter
- Variation in vapour pressure, wind speed and water temperature.

Due to these reasons, the lake evaporation will be less than the pan evaporation, the two being related as under :

$$\text{Lake evaporation} = \text{Pan coefficient} \times \text{Pan evaporation}$$

The pan coefficient for the Indian Standard evaporimeter is around 0.8.

2. Use of empirical equations

There are a large number of empirical equations for estimating the evaporation loss. However, we shall consider here the following two popular formulae :

- Meyer's formula.
- Rohwer's formula.

(i) Meyer's formula (1915)

$$E = K_m (e_s - e_a) \left[1 + \frac{V_9}{16} \right] \quad \dots(4.29)$$

where E = evaporation from the water body, in mm/day.

e_s = saturation vapour pressure at the water-surface temperature in mm of mercury.

e_a = actual vapour pressure of overlying air at the specified height (i.e. 9 m here), in mm of mercury

V_9 = monthly mean wind velocity in km/h at a height of 9 m above the ground

K_m = coefficient accounting for various other factors

= 0.36 for large deep waters.

= 0.50 for small shallow waters.

(iii) Rohwer's formula (1931)

$$E = 0.771 (1.465 - 0.000732 P_a) (0.44 + 0.0733 V_{0.6}) (e_s - e_a) \quad \dots(4.30)$$

where E , e_s and e_a have the same meanings as in Meyer's formula.

$V_{0.6}$ = mean wind velocity in km/h at 0.6 m above the ground level.

Thus, Rohwer's formula takes into account a correction for the effect of pressure, in addition to the wind speed effect. It is known that in the lower part of the atmosphere, upto a height of about 500 m above the ground level, the wind velocity is assumed to follow the $1/7$ power law as under :

$$V_h = C h^{1/7} \quad \text{or} \quad \frac{V_{h1}}{V_{h2}} = \left(\frac{h_1}{h_2} \right)^{1/7} \Delta \left(\frac{h_1}{h_2} \right)^{0.143} \quad \dots(4.31)$$

where V_h is the wind velocity at height h above the ground.

3. Water budget or water balance method : This method balances all the incoming, outgoing and stored water in a lake or reservoir over a period of time, using the following equation :

$$\Sigma \text{ Inflow} - \Sigma \text{ Outflow} = \text{Change in storage} + \text{Evaporation loss}$$

$$\text{or} \quad E = \Sigma I - \Sigma O \pm \Delta S \quad \dots(4.32 a)$$

The above equation can be generalised as under, taking all the factors of inflow and outflow :

$$E = (P + I_s + I_g) - (O_s + O_g + T) \pm \Delta S \quad \dots(4.32)$$

where P = precipitation

I_s = surface water inflow

I_g = ground water inflow

O_s = surface water outflow

O_g = ground water, outflow

T = transpiration loss, which may be neglected.

ΔS = change in storage.

The above method does not give accurate results because it is very difficult to measure I_g and O_g for a lake/reservoir.

4. Energy budget or energy balance method

This method uses the conservation of energy by incorporating all the incoming, outgoing and stored energy of a lake/reservoir in the following from :

$$H_n = H_a + H_s + H_e + H_i + H_r \quad \dots(4.33)$$

where H_n = net heat energy received by water surface = $H_c (1 - r) - H_b$

$H_c (1 - r)$ = incoming solar radiation into a surface of reflection coefficient (albedo) r

The value of r for water surface approximates 0.05. For newly laid snow, $r = 0.90$.

H_b = back radiation (long wave) from the water body.

H_s = sensible heat transfer from water surface to air

H_e = heat energy used up in evaporation = $\rho_w L_e E$,

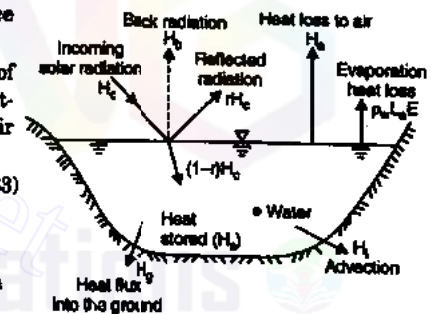


FIG. 4.31. ENERGY BALANCE IN A WATER BODY

where ρ_w = density of water, L_a = latent heat of evaporation.
 H_g = heat flux into the ground, H_s = heat stored in water body
 H_i = net heat conducted out of the system by water flow (advected energy)

In the above equation, all the energy terms are in calories per square mm per day. All the terms, except H_a can either be measured or evaluated. For small time periods, the terms H_s and H_i can be neglected. However, the sensible heat term H_a can be estimated by using Bowen's ratio (β) by the following expression :

$$\beta = \frac{H_a}{\rho_w L_a E} = 6.1 \times 10^4 P_a \frac{T_w - T_a}{e_s - e_a} \quad \dots(4.34)$$

where T_w = temperature of water surface in $^{\circ}\text{C}$
 T_a = temperature of air in $^{\circ}\text{C}$
 P_a = atmospheric pressure in mm of mercury

Combining Eqs. 4.33 and 4.34, we obtain the following equation for evaporation E :

$$E = \frac{H_n - H_g - H_s - H_i}{\rho_w L_a (1 + \beta)} \quad \dots(4.35)$$

The above method gives satisfactory results, with errors of the order of about 5%, when applied to periods less than a week.

Transpiration and Evapo-transpiration : Transpiration is the process in which water is lost through the living plants, during the respiration process, and back to the atmosphere. *Evapo-transpiration* is a combined term of evaporation and transpiration, defined as the total loss of water through evaporation and transpiration from the plants.

For detailed discussion, reference may be made to Chapter 3.

Example 4.14. The following Table gives data from a Class-A pan. Assuming a pan coefficient of 0.8, calculate the daily lake evaporation and average evaporation for one week.

Date	1-7-90	2-7-90	3-7-90	4-7-90	5-7-90	6-7-90	7-7-90
Rainfall (mm)	0	6	8	12	9	5	0
Water added (mm)	+ 12	+ 5	+ 2	- 3	+ 1	+ 6	+ 11

Solution : The computations are arranged in a tabular form below, which is self explanatory.

Date	Water added or taken out (mm)	Rainfall (mm)	Pan evaporation (mm)	Lake evaporation (mm)
(1)	(2)	(3)	(4) = (2) + (3)	(5) = 0.8 × (4)
1-7-90	+ 12	0	12	9.6
2-7-90	+ 5	6	11	8.8
3-7-90	+ 2	8	10	8.0
4-7-90	- 3	12	9	7.2
5-7-90	+ 1	9	10	8.0
6-7-90	+ 6	5	11	8.8
7-7-90	+ 11	0	11	8.8
Sum				59.2

$$7 \text{ day's average evaporation} = \frac{\Sigma E}{7} = \frac{59.2}{7} \triangleq 8.46 \text{ mm}$$

Example 4.15. A reservoir with average surface spread of 4.8 km^2 in the first week of November has the water surface temperature of 30°C and relative humidity of 40%. Wind velocity measured at 3.0 m above the ground is 18 km/h . The mean barometer reading is 760 mm of Hg . Calculate the average evaporation loss from the reservoir in mm/day and the total depth and volume of evaporation loss in the first week of November. Use both Meyer's equation as well as Rohwer's equation. Take saturation vapour pressure at 30°C as 31.81 mm of Hg .

Solution : (a) Using Meyer's formula

$$E = K_m (e_s - e_a) \left[1 + \frac{V_s}{16} \right]$$

Given : $e_s = 31.81 \text{ mm Hg}$. Relative humidity $R_H = 40\% = 0.4$
 $e_a = e_s \cdot R_H = 31.81 \times 0.4 = 12.724 \text{ mm Hg}$.

$$V_s = V_z \left(\frac{9}{3} \right)^{1/7} = 18 \times 1.1699 = 21.06 \text{ km/h}$$

$K_m = 0.36$ for large, deep waters

$$E = 0.36 (31.81 - 12.724) \left[1 + \frac{21.06}{16} \right] = 15.91 \text{ mm /day}$$

$\therefore$ Total depth of evaporation in one week = $7 \times 15.91 = 111.4 \text{ mm}$

$$\begin{aligned} \text{Total volume of water evaporated} &= (111.4 \times 4.8 \times 10^6) 10^{-3} \\ &= 0.5347 \times 10^6 \text{ m}^3 = 53.47 \text{ hectare-m} \end{aligned}$$

(b) Using Rohwer's formula

$$E = 0.771 (1.465 - 0.000732 P_a) (0.44 + 0.0733 V_{0.6}) (e_s - e_a)$$

where $e_s = 31.81 \text{ mm Hg}$ and $e_a = 12.724 \text{ mm Hg}$, as earlier.

$P_a = 760 \text{ mm Hg}$.

$$V_{0.6} = \left(\frac{0.6}{2} \right)^{1/7} \times 18 = 15.16 \text{ km/h}$$

$$\begin{aligned} E &= 0.771 (1.465 - 0.000732 \times 760) (0.44 + 0.0733 \times 15.16) (31.81 - 12.724) \\ &= 20.74 \text{ mm/day} \end{aligned}$$

$\therefore$ Total evaporation for one week = $20.74 \times 7 \triangleq 145.2 \text{ mm}$

$$\begin{aligned} \therefore \text{Total volume of water evaporated} &= (145.2 \times 4.8 \times 10^6) 10^{-3} = 0.697 \times 10^6 \text{ m}^3 \\ &= 69.7 \text{ hectare-m} \end{aligned}$$

Thus, we find that Rohwer's formula gives about 30% higher results than Meyer's formula.

4.16. INFILTRATION

Infiltration may be defined as the downward movement of water from soil surface, into the soil mass through the pores of the soil. When rain-water falls on the ground, a small part of it is initially absorbed by the top thin layer of soil so as to replenish the soil-moisture deficiency, and thereafter, excess water infiltrates downwards to join

ground water. In simple terms, infiltration is the entry or passage of water into the soil through soil surface. Once water enters into the soil, the process of transmission of water in the soil, known as percolation, takes place, thus removing the water from near the surface to down below. Infiltration and percolation are directly inter-related. When percolation stops, infiltration also stops. The term infiltration was first used by Horton (1935) to describe the phenomenon of soaking water into the soil or absorbed by the soil. He defined it as the entry of water into soil through the surface layer of the soil, vertically. Infiltration is a major process continuously affecting the magnitude, timing and distribution of surface run off at any measured outlet of a basin.

Infiltration capacity

The capacity of any soil to absorb water from rain falling continuously at an excessive rate goes on decreasing with time, until a minimum rate to infiltrate is reached (Fig. 4.32). At any instant, the infiltration capacity (f_0) of a soil is the maximum rate at which water will enter the soil in a given condition. The infiltration rate at any instant is the rate at which water actually enters the soil during a storm, and is equal to the infiltration capacity (f_0) or the rainfall rate, whichever is less.

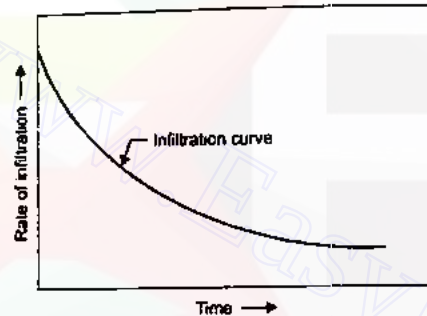


FIG. 4.32. INFILTRATION CAPACITY CURVE

Effects of infiltration : The following are the effects of infiltration

1. It reduces the magnitude of the flood
2. It delays the time of arrival of water to the channel
3. It recharges the ground water reservoir.
4. It reduces soil erosion
5. It fills the soil pores to its field capacity, thus making water available to plants.
6. It sustains green vegetation cover on the ground surface and thus helps in reducing dust storms.

Factors affecting Infiltration

Following are various factors, both meteorological as well as the characteristics of soil media, on which infiltration rate and infiltration capacity depends.

1. Condition of entry surface : Vegetation cover versus bare land

If the area is covered by grass, vegetation and bushy plants, infiltration capacity will be more. On the contrary if the soil surface is bare, the impact of rain drops falling on the surface will cause in-washing of finer particles of the soil and will clog the surface, thus resulting in the retardation of infiltration. The vegetation cover also provides a layer of decaying organic matter, which promotes activities of burrowing insects etc. which in turn produce spongy permeable soil structure resulting in increased infiltration. In addition to this, the vegetation cover removes soil moisture through transpiration thus tending to reduce infiltration during initial periods of rain.

2. Permeability/percolation characteristics of soil formation

The infiltration will continue only when percolation continues. The infiltrated water must be transmitted down by the force of gravity and capillary action. Percolation, however, depends upon several factors such as soil and its composition, permeability, porosity, stratification, presence of organic matter and presence of salts.

3. Antecedent moisture conditions in soil

Infiltration rate will depend on initial moisture conditions of soil. When the soil moisture is high, the infiltration rate (f_i) will be low. When water falls on a dry surface, the upper surface becomes wet while the lower parts of the soil remain comparatively dry initially. This results in a large difference of capillary potential, due to which a down ward force will act on the water, in addition to the normal force of gravity. This will result in rapid infiltration. For the second storm in succession, the soil will have lesser rate of infiltration than the first maiden storm of the season.

4. Temperature

The viscosity of water changes with temperature. The flow of water within the body of the soil is laminar, and the flow is directly related to viscosity. In summer, therefore, infiltration will be higher due to less viscous water, in comparison to winter. In sub zero temperature, crystallisation of water in the pores blocks the passage, thus reducing or even stopping the infiltration.

5. Intensity and duration of rainfall

When the precipitation takes place with heavy intensity, the impact of water causes mechanical compaction and in-wash of fine particles, resulting in faster decrease in the rate of infiltration. However, rainfall of lesser intensity result in higher infiltration rate. The rainfall with higher duration will result in lower infiltration in comparison to the same quantity of rain falling as a number of isolated storms.

6. Movement of man and animals

When there is heavy movement of man and/or animals, the soil gets compacted, resulting in reduction in the infiltration rate.

7. Change due to human activities

Cultivation of barren land, by growing crops and grass cover, results in increased rate of infiltration. On the other hand, construction of roads, houses, factories, play grounds, including over-grazing of pastures results in reduction in infiltration capacity.

8. Quality of water

Silts and other impurities present in incoming water result in retardation of infiltration rate due to clogging of soil pores. The salts present in water affect the viscosity of water and may react with soil to form complexes that reduce the porosity of soil.

9. Presence of ground water table

Presence of ground water table reduces infiltration. For infiltration to continue, the position of ground water table should not be very close.

10. Size and characteristics of soil particles : The infiltration rate is directly proportional to the grain size/diameter, for granular soils. However, if the soil has swelling minerals like illite and montmorillonite, the infiltration rate will reduce drastically.

11. Catchment parameters

It is well known now, from exhaustive studies, that the infiltration capacity decreases with increase in drainage density (i.e. length of all the stream of a basin divided by its catchment area). Similarly, the infiltration capacity (cm/hour) decreases with increase in the runoff rate ($\text{m}^3/\text{sq.km}$). Similarly, the infiltration capacity (cm/h) decrease with increase in sediment yield rate ($\text{m}^3/\text{sq.km}$)

Infiltration capacity curve (I.C. curve)

Infiltration capacity curve for a given soil formation graphically represents the variation of infiltration capacity (f_i) with time during or a little after rainfall. As stated earlier, the infiltration rate is maximum (f_0) at the beginning of the rainfall and thereafter, infiltration rate decreases exponentially with time.

Horton's equation

Horton gave the following equation for finding infiltration rate (f_i) at any time period (t)

$$f_i = f_c + (f_0 - f_c) e^{-kt} \quad \dots(4.36)$$

where f_i = infiltration rate at any time t
 f_c = constant infiltration rate at time $t = T$ (say)
 f_0 = Infiltration rate in the beginning ($t = 0$)
 k = a constant which depends on the soil and vegetation.

Solution of Horton's equation

Transposing and taking log on both sides.

$$\log_{10} (f_i - f_c) = \log_{10} (f_0 - f_c) - kt \log_{10} e$$

$$\text{From which } t = -\frac{1}{k \log_{10} e} [\log_{10} (f_i - f_c) - \log_{10} (f_0 - f_c)]$$

$$\text{or } t = -\frac{1}{k \log_{10} e} \log_{10} (f_i - f_c) + \frac{1}{k \log_{10} e} \log_{10} (f_0 - f_c) \quad \dots(4.36 \text{ a})$$

The above equation is the equation of a straight line of the form $y = mx + c$ where $y = t$, $m = -\frac{1}{k \log_{10} e}$,

$$x = \log_{10} (f_i - f_c) \text{ and } c = \frac{1}{k \log_{10} e} \log_{10} (f_0 - f_c)$$

The slope of the line is $-\frac{1}{k \log_{10} e}$, where the negative sign shows that as t increases, f_i decreases and hence $(f_i - f_c)$ and $\log_{10} (f_i - f_c)$ also decreases as is clear from the straight line of Fig 4.34. The value of slope can be measured by knowing the values of f_i at two timings t_1 and t_2 and drawing a straight line through these two points. Knowing the slope

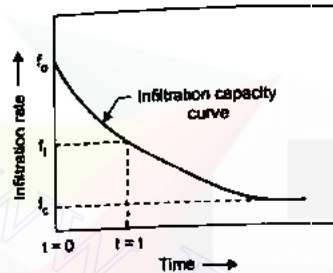


FIG. 4.33. INFILTRATION CAPACITY CURVE

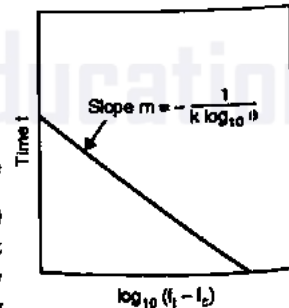


FIG. 4.34

m of the line, the value of constant k is known. Thus, the equation of the curve can be written easily in terms of known parameters k and f_c .

Field Measurement of infiltration rate

Infiltration in the field can be measured with the help of two types of *infiltrimeters*: (a) single cylindrical or single tube infiltrimeters and (a) concentric double cylindrical or double ring infiltrimeters.

Single tube infiltrimeter : It consists of a hollow metal cylinder of 30 cm diameter and 60 cm length with both ends open. The cylinder is driven in the ground such that 10 cm of it projects above the ground. The cylinder is filled with water, such that a head of 7 cm within the infiltrimeter is maintained above ground level. Due to infiltration of water, the water level in the cylinder will go on decreasing. Water is added to the cylinder, through graduated jar or burette, so as to maintain constant level. The volume of water added over a predetermined time interval gives the infiltration rate for that time interval. The observations are continued till almost uniform infiltration rate is obtained, which may take about 3 to 6 hours, depending upon the type of soil. A plot of time in abscissa against rate of water added in mm/h gives the infiltration capacity curve for the area. The major draw back of the single tube infiltrimeter is that infiltrated water percolates laterally at the bottom of the ring as shown in Fig. 4.35. Hence it does not truly represents the area through which infiltration takes place.

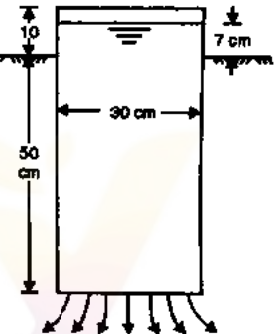


FIG. 4.35. SINGLE TUBE INFILTRIMETER

Double tube (or double ring) infiltrimeter

The above defect of lateral percolation of water is rectified in the double tube infiltrimeter which consists of two concentric hollow rings (or cylinders) driven into the soil uniformly without any tilt and disturbing the soil, to the least depth of 15 cm. The diameter of the rings may vary from 25 to 60 cm (Fig. 4.36). Water is applied in both the inner and outer rings to maintain a constant depth of about 5 cm. Water is replenished after the level falls by about 1 cm. The water depth in the inner and outer rings should be kept the same during the observation period. However, the measurement includes the recording of volume of water added into the *inner compartment*, to maintain the constant water level and the corresponding elapsed time. As the purpose of the outer ring is to suppress the lateral percolation of water from the inner ring, the water added to it need not be measured though water is added to it to maintain the same depth as the inner ring. Observations are continued till constant infiltration rate is observed.

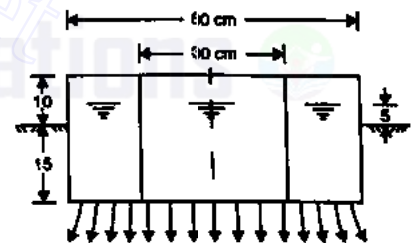


FIG. 4.36. DOUBLE RING INFILTRIMETER

Example 4.16. The following observations were taken from a double tube infiltrimeter with

inside ring diameter of 30 cm. Plot the infiltration capacity curve and find the constant rate of infiltration that the experimental field have towards the end. Also, compute the average infiltration rate for the first 10 and first 30 minutes.

Time (min.)	0	2	5	10	20	30	45	60	80	100	120	150	180
Cumulative volume (cm ³)	0	200	470	840	1406	1840	2245	2510	2745	2885	2990	3130	3270

$$\text{Solution : Area of infiltrometer} = \frac{\pi}{4} (30)^2 = 706.86 \text{ cm}^2$$

The computations are arranged in a tabular form, which is self explanatory

Time (min.)	Time increment Δt (hours)	Cumulative vol. of water added (V) (cm ³)	Cumulative infiltration $F = V/A$ (cm)	Incremental infiltration ΔF (cm)	Infiltration rate $f_i = \Delta F / \Delta t$ (cm/h)
0	0	0	0		
2	0.0333	200	0.283	0.283	8.50
5	0.0500	470	0.665	0.382	7.64
10	0.0833	840	1.188	0.523	6.28
20	0.1667	1406	1.988	0.800	4.80
30	0.1667	1840	2.603	0.615	3.69
45	0.2500	2245	3.176	0.573	2.29
60	0.2500	2510	3.551	0.375	1.50
80	0.3333	2745	3.883	0.332	1.00
100	0.3333	2885	4.081	0.198	0.59
120	0.3333	2990	4.230	0.149	0.45
150	0.5000	3130	4.428	0.198	0.40
180	0.5000	3270	4.626	0.198	0.40

Fig. 4.37 shows the graph between rate of infiltration (f_i) in cm/hr as ordinate vs. time in minutes as abscissa. From the graph as well as the above Table, constant rate of infiltration, $f_c = 0.40$ cm/hr, which is also the minimum infiltration capacity.

Also, average rate of infiltration for the first 10 minutes

$$= \frac{1.188}{(10/60)} = 7.13 \text{ cm/hr}$$

Average rate of infiltration for the first 30 minutes

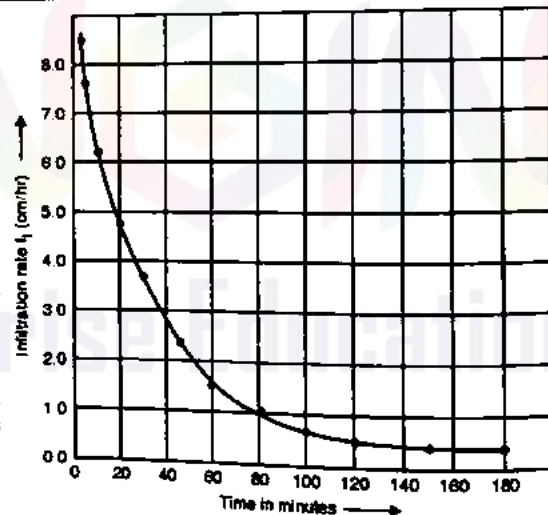
$$= \frac{2.603}{(30/60)} = 5.21 \text{ cm/hr.}$$


FIG. 4.37. INFILTRATION CURVE

4.17. RUN-OFF

The term runoff is used for water that is on the 'run' or in a flowing state, in contrast to the water held in depression storage and water evaporated in the atmosphere. The run off of a catchment area in any specified period is the total quantity of water draining into a stream or into a reservoir in that period. This can be expressed as (i) centimetres of water over a catchment, or (ii) the total water in cubic-metre or hectare-metres for given catchment. Runoff is broadly classified into three types

1. Surface runoff
 2. sub-surface runoff
 3. Base flow.
- Direct runoff

and

As discussed earlier, the rainfall is disposed off in the following manner :

1. Basin Recharge.
2. Direct run-off (or simply run-off).
3. Percolation down to ground water.
4. Evaporation and transpiration.

1. **Basin Recharge** : It consists of :

- (i) Rain intercepted by leaves and stems of vegetation.
- (ii) Water held up in surface depressions, commonly known as the *depression storage*.
- (iii) Soil moisture held as capillary water in pore spaces of soil or as hygroscopic water absorbed on the surface of soil particles.

2. Direct Run-off

Direct run-off is that water which reaches the stream shortly after it falls as rain. Direct run-off consists of :

- (i) Overland flow or surface run-off.
- (ii) Sub-surface runoff or interflow (influent stream).

Overland flow is that portion of water which travels across the ground surface to the nearest stream. However, if the soil is permeable, water percolates into it, and when it become saturated, flows laterally in the surface soil to a stream channel. The essential condition for *interflow* or *influent stream* is that the surface soil is permeable, but the sub-soil is relatively impermeable so that water does not percolate deep to meet the ground water.

3. Percolation Down to Ground Water (Base Flow)

If the sub-soil is also permeable, water percolates deep downwards to meet the ground water. Much of the low water flow of rivers is derived from the ground water. Stream channels which are below the ground water table are called *effluent streams*.

Schematic representation of runoff process

Fig. 4.38 shows the schematic representation of process of runoff resulting from precipitation over a basin.

Fig. 4.39 shows the forms of runoff in the hydrologic cycle.

Some basic terms used in hydrology

1. **Basin** : A basin, also called a *drainage basin*, *catchment* or *watershed*, is the area bounded by the highest contour called ridge line from where precipitated water

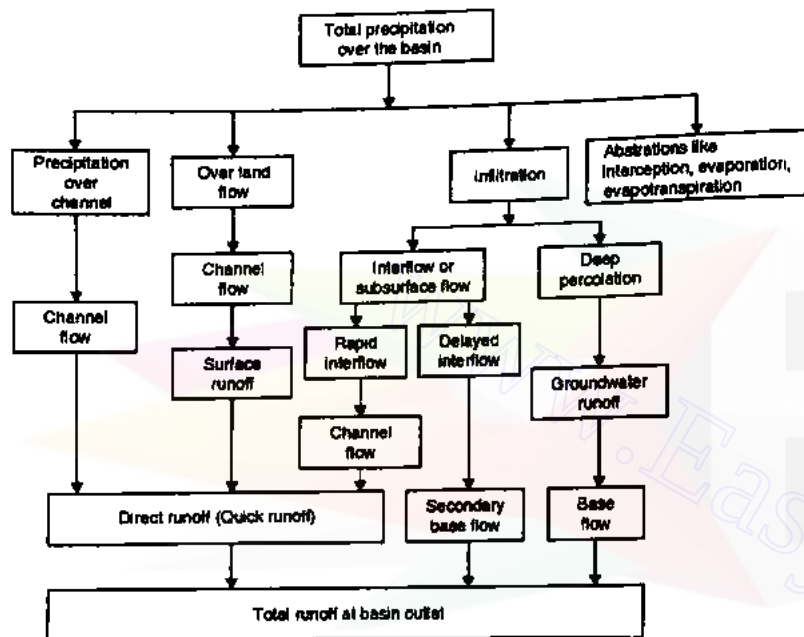


FIG. 4.38. SCHEMATIC REPRESENTATION OF RUNOFF PROCESS RESULTING FROM PRECIPITATION OVER THE BASIN

is collected by surface and subsurface flows and drained out through a natural stream or river. Adjacent basins are divided by ridge lines. The total area of a basin is known as the catchment area.

2. Stream : Stream is a natural flow channel towards which the water from a basin flows. The stream drains out the water so collected, into a larger water body.

3. Overland flow : Overland flow is that portion of rain water which flows over the land surface in the form of sheet of water to join the nearest stream.

4. Surface runoff : Surface runoff is the part of the precipitation and other drainage water of a basin which moves over the natural land surface and then through a network of channels of gradually large sections.

5. Stream flow : Stream flow or the *total runoff* consists of surface flow, subsurface flow, ground water flow (or base flow) and the flow due to precipitation falling directly in the stream.

6. Drainage density : Drainage density (D_d) is defined as the ratio of the total length of all streams of the catchment divided by its area.

Thus,
$$D_d = \frac{L_s}{A} \quad \dots(4.37)$$

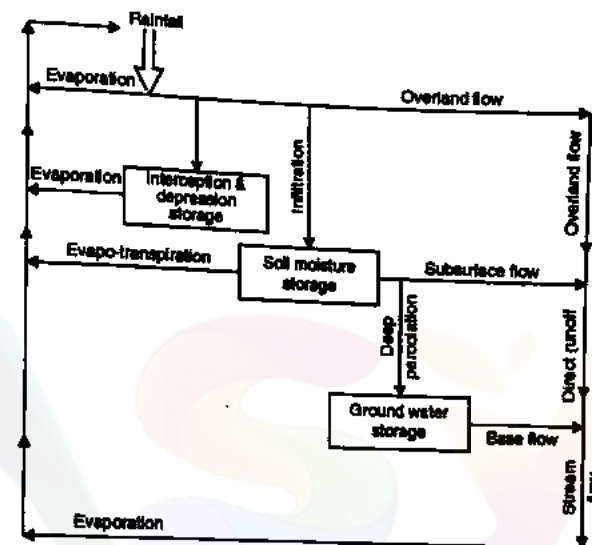


FIG. 4.39. FORMS OF RUNOFF IN THE HYDROLOGIC CYCLE

where L_s = total length of all streams in a basin
and A = drainage area of basin.

The drainage density indicates the drainage efficiency of the basin — the higher the value, the quicker the runoff.

7. Stream density : Stream density (D_s) is defined as the number (N_s) of the streams of given order per sq. km of the catchment area.

Thus,
$$D_s = \frac{N_s}{A} \quad \dots(4.38)$$

8. Centroid of the basin : It is the point of weighted centre of a watershed.

9. Form factor : Horton (1932) defined form factor (F_f) as the ratio of the watershed area (A) to the square of its length (L_b^2). It is also defined as the ratio of average width (W_b) of the basin to its length (L_b) measured between the stream outlet to the most remote point on the basin.

The average width (W_b) is obtained by dividing the watershed area (A) with the axial length of the same. Thus,

$$F_f = \frac{W_b}{L_b} = \frac{A/L_b}{L_b} = \frac{A}{L_b^2} \quad \dots(4.39)$$

10. Compactness coefficient (C_c) : Strahler (1964) defined it as the ratio of the perimeter of the basin to the circumference of a circle whose area is the area of the basin.

$$\text{Thus, } C_r = \frac{P}{\sqrt{4\pi A}} = \frac{0.2821P}{\sqrt{A}} \geq 1 \quad \dots(4.40)$$

where P = perimeter of the basin in km.

A = area of the basin in sq. km.

11. Elongation ratio (E_r) : Elongation ratio, introduced by Schuman (1956) is the ratio of the diameter of a circle of the same area of the basin to the maximum length of the basin.

$$\text{Thus } E_r = \frac{1}{L} \sqrt{\frac{A}{0.786}} \leq 1 \quad \dots(4.41)$$

12. Circularity ratio (C_r) : Circularity ratio, proposed by Millar (1956) is the ratio of the area of the basin to the area of a circle having the same perimeter as the basin.

$$\text{Thus, } C_r = 12.57 \frac{A}{P^2} \leq 1 \quad \dots(4.42)$$

4.18. FACTORS AFFECTING RUN-OFF

The principal factors affecting the flow from a catchment area are :

1. Precipitation characteristics.
2. Shape and size of the catchment.
3. Topography.
4. Geological characteristics.
5. Meteorological characteristics.
6. Character of the catchment surface.
7. Storage characteristics.

1. Precipitation Characteristics

This is most important factor on which run-off depends. Important precipitation characteristics are : (i) intensity (ii) duration (iii) aerial distribution (iv) direction of storm movement, (v) form of precipitation and (vi) evapo-transpiration. The more the rainfall, more will be the run-off. Run-off depends on the type of the storm causing precipitation, and also upon its duration. Run-off also increases with the intensity of rainfall. Run-off also increases with the extent of the storm over the catchment. If the rainfall intensity is very less, and it rains as light showers, much of the water will be lost in infiltration and evaporation etc., and the run-off will be less. Greater evapo-transpiration will result in lesser runoff.

Similarly, if the precipitation is in the form of snow, or if water freezes as it falls, it will be retained in the catchment till temperature increases. Thus run-off will be less.

2. Shape and Size of the Catchment

The run-off from a catchment also depends upon the size, shape and location of the catchment. More intense rainfalls are generally distributed over a relatively smaller area ; a stream collecting water from a small catchment area is likely to give greater run-off intensity per unit area. In the case of a very big catchment, uniform rain seldom falls over the entire area, with the result that only very few tributaries of the stream feed water to main stream during a particular storm. Thus run-off intensity of larger stream, per unit catchment area is lesser.

In the case of a fan or sector shaped catchment [Fig. 4.40 (a)] all the tributaries are approximately of the same size. Such catchment gives greater run-off since the peak flood from the tributaries is likely to reach the main stream approximately at the same time. In the case of a fern leaf catchment [Fig. 4.40 (b)], the tributaries are generally of different lengths, and meet the main stream at the regular intervals. In such a narrow catchment, the peak flood intensity is reduced since discharges are likely to be distributed over a long period of time.

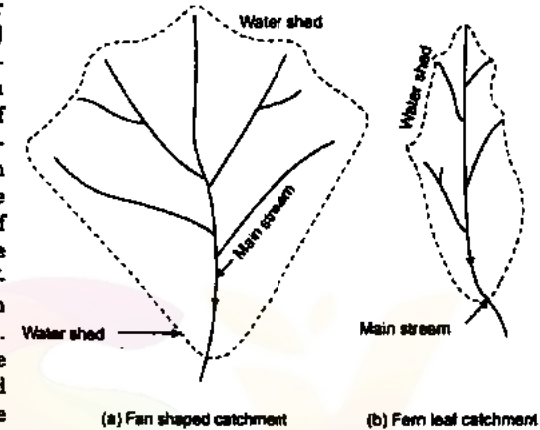


FIG. 4.40. EFFECT OF SHAPE OF THE CATCHMENT

3. Topography of Catchment

The run-off depends upon whether the surface of the catchment is smooth or rugged. If the surface slope is steep, water will flow quickly, and absorption and evaporation losses will be less, resulting in greater run-off. If the catchment is mountainous, and is on the windward side of the mountains, the intensity of rainfall will be more, and hence run-off will also be more.

4. Orientation of watershed : The orientation of watershed affects the evaporation and transpiration loss by influencing the amount of heat received from the sun. The North and South orientation of watershed affects the melting time of collected snow and hence the runoff. Similarly, in mountainous watershed, the windward side of the mountain receives comparatively higher intense rainfall than the leeward side.

5. Geological Characteristics of basin

Geological characteristics of the catchment area is an important factor affecting the run-off. These include the type of surface soil and sub-soil, type of rock and their permeability characteristics. If the soil and sub-soil is pervious, seepage will be more and this in turn reduces the peak flood. If the surface is rocky, the absorption will be practically nil, and run-off will be more. However, if the rocks have fissures, or if they are porous or have lava tunnels etc., most of the water will be lost by way of seepage, and surface run-off will be very small. However, if this seeping water has the outlet in the stream of the catchment, most of the water flows back to the stream during its period of low flow.

6. Meteorological Characteristics

Temperature, wind and humidity also affect the run-off. If the temperature is low and the ground is saturated and frozen, it gives rise to greater run-off. However, if the whole of the stream freezes, the peak floods will be reduced. On the other

hand, high temperature and greater wind velocity give rise to greater evaporation loss and reduce the run-off. The peak flood depends upon the direction of movement of the storm causing rainfall with relation to the direction of the stream. If the storm centre moves against the direction of the stream, the run-off from various points along the catchment reaches the stream at different timings, with the result that the peak flow is reduced. On the other hand, if the storm centre moves in the direction of flow of the stream, and practically with the same velocity, the run-off tends to reach at the same time resulting in a higher peak flow.

7. Character of the Catchment Surface

The run-off also depends upon the surface conditions — whether the surface is drained or undrained, natural or cultivated, and whether it is bare or covered with vegetation etc. If the surface has no natural drainage, absorption loss will be more. If more area of a catchment is cultivated, surface run-off will be less. The presence of vegetal cover reduces the run-off specially during smaller storms. However, for a bigger storm after the vegetal cover gets saturated, water runs from the surface freely as if there was no vegetation.

8. Storage Characteristics of the Catchment

The artificial storage such as dams, weirs etc. and natural storage such as lakes, ponds etc. tend to reduce the peak flow. They also give rise to greater evaporation losses. If the tributaries of a river discharge into a lake, and the river takes off from the lake, the peak floods in the river will be reduced. The storage into the pervious surface soil and sub-soil tends to reduce the peak flow and tend to regularise the flow of a stream.

4.19. COMPUTATION OF RUN-OFF

The run-off from a catchment can be computed daily, monthly or yearly. Following are some of the methods of computing the run-off :

- By linear or exponential regression
- By empirical equations and Tables.
- By Infiltration method.
- By Unit Hydrograph.
- By Rational method.

The unit hydrograph method and the rational method have been discussed in subsequent articles.

(a) **Runoff by linear or exponential regression** : Because of several factors affecting runoff resulting from a given rainfall, the relationship between these two is quite complex. However we may use *straight-line regression* for small and medium sized catchments and exponential form of regression for large catchments.

1. **Straight line regression between P and R** : The equation of linear regression line between observed values of R and P is

$$R = aP + b \quad \dots(4.43)$$

where a and b are constants representing abstractions. The values of a and b are given by the following equations :

$$a = \frac{N(\Sigma P \cdot R) - (\Sigma P)(\Sigma R)}{N(\Sigma P^2) - (\Sigma P)^2} \quad \dots(4.43 \ a)$$

and

$$b = \frac{\Sigma R - a \Sigma P}{N} \quad \dots(4.43 \ b)$$

and coefficient of correlation, $r = \frac{N(\Sigma PR) - (\Sigma P)(\Sigma R)}{\sqrt{[N(\Sigma P^2) - (\Sigma P)^2] \times [N(\Sigma R^2) - (\Sigma R)^2]}} \quad \dots(4.43 \ c)$

2. **Exponential regression between P and R** : For large catchments, it is advantageous to use the following exponential relationship:

$$R = \beta P^m \quad \dots(4.44)$$

where β and m are constants. Taking logarithms of both the sides, we get
 $\log_e R = m \log_e P + \log_e \beta$

which is a linear form, from which the values of coefficients β and m can be computed, as explained above, from observed sets of values of P and R .

(b) RUN-OFF BY FORMULAE AND TABLES

1. Run-off Coefficient :

The run-off and the rainfall can be inter-related by run-off coefficient, by the expression.

$$R = kP$$

where R = run-off in cm; P = rainfall in cm; k = run-off coefficient.

The run-off coefficient naturally depends upon all the factors which affect the run-off. This method is used only for small water control project, and should be avoided for the analysis of major storms. The customary values of k are given below :

Area	k
Urban residentials : Single Houses	0.3
Garden apartments	0.5
Commercial and Industrial	0.9
Forested areas, depending on soil	0.05 - 0.2
Parks, farm land, pasture	0.05 - 0.3
Asphalt or concrete pavement	0.85

Barlow's Table. T.G. Barlow (1912) carried out studies of catchments mostly under 140 sq km in the United Provinces (U.P.) and gave the following values of k (in percentage) for various classified catchments.

TABLE 4.10. BARLOW'S TABLE

Class	Description of Catchment	Run-off percent
A	Flat, cultivated and black cotton soils	10
B	Flat partly cultivated : various soils	15
C	Average	20
D	Hills and plains with cultivation	35
E	Very hilly and steep, with hardly any cultivation	45

The above values of run-off percentages are for average monsoons. These are to be multiplied by the following coefficients (Table 4.11) according to the nature of season.

TABLE 4.11. BARLOW'S COEFFICIENT

Nature of season	Class of Catchment				
	A	B	C	D	E
(1) Light rain (upto 25 mm/day); no heavy downpour	0.7	0.8	0.8	0.8	0.8
(2) Average or varying rainfall (25-75 mm/day); no continuous downpour	1.0	1.0	1.0	1.0	1.0
(3) Continuous downpour greater than 75 mm/day	1.5	1.5	1.6	1.7	1.8

2. **Strange's Tables and Curves.** W.L. Strange (1928) gave Tables and curves for run-off resulting from rainfall in the plains of South India. The following Tables and graphs are for the Konkan region of Maharashtra. Strange's tables and curves give run-off for daily rainfall, and take into account three types of catchments (i.e. good, average and bad), and three surface conditions, (i.e. dry, damp and wet) prior to the rain.

TABLE 4.12. DAILY RUN-OFF ACCORDING TO STRANGE

Note. For good and bad catchment, add or deduct upto 25% of yield.

Daily Rainfall (mm)	Run-off percentage and yield when the original state of ground was					
	Dry		Damp		Wet	
	%	Yield (mm)	%	Yield (mm)	%	Yield (mm)
6.25	...	...	...	...	8	0.6
12.5	...	...	6	0.75	12	1.5
18.75	...	...	8	1.50	16	3.0
25	3	0.75	11	2.75	18	4.5
31.25	5	1.56	14	4.37	22	6.88
37.5	6	2.25	16	6.00	25	9.37
43.75	8	3.5	19	8.31	30	13.1
50	10	5.0	22	11.00	34	17.0
62.5	15	9.37	20	18.15	43	25.9
75	20	15.00	37	27.75	55	41.25
100	30	30.00	50	50.00	70	70.00

3. **Ingilis's Formula.** : C.C. Ingilis gave the following formulae, derived from data collected from 37 catchments in the Bombay Presidency :

For Ghat areas (western ghats)

$$R = 0.85 P - 30.5 \quad (\text{when } R \text{ and } P \text{ are in cm}) \quad \dots(4.45)$$

For Non-ghat areas (Plain regions)

$$R = 0.00394 P^2 - 0.0701 P \quad \text{where } R \text{ and } P \text{ are in cm.} \quad \dots(4.46)$$

HYDROLOGY

4. Lacey's Formula

$$R = \frac{P}{1 + \frac{304.8 F}{PS}} \quad \dots(4.47)$$

(where P and R are in cm)

where S = Catchment factor
and F = Monsoon duration factor.

Corresponding to the five classes of catchments, defined by Barlow, Lacey gave the following values of the catchment factor (S).

TABLE 4.13

Class of catchment	Values of S
A	0.25
B	0.60
C	1.00
D	1.70
E	3.45

Lacey also divided the monsoon into three classes, depending upon its duration and gave the following values of monsoon duration factors :

TABLE 4.14

No.	Class of monsoon	Monsoon Factor (F)
1.	Very Short	0.50
2.	Standard length	1.00
3.	Very Long	1.50

5. Khosla's formula (1960)

$$R_m = P_m - 0.48 T_m \quad \dots(4.48)$$

where R_m = monthly runoff in cm.

where T = mean temperature in $^{\circ}\text{C}$ on the entire catchment.

P_m = monthly precipitation in cm.

The temperature introduced in the formula takes into account various factors affecting losses by evaporation, transpiration, sunshine and wind velocity.

6. ICAR Formula (For Nilgiri Hills)

$$R = 1.511 (P^{1.44}) (T_m)^{-1.34} (A^{-0.0013}) \quad \dots(4.49)$$

where

A = Watershed area km^2

P and R = annual rainfall in cm

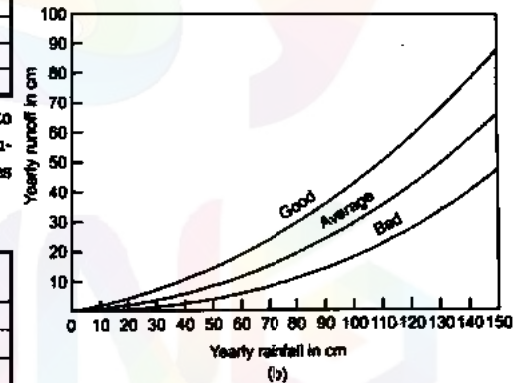
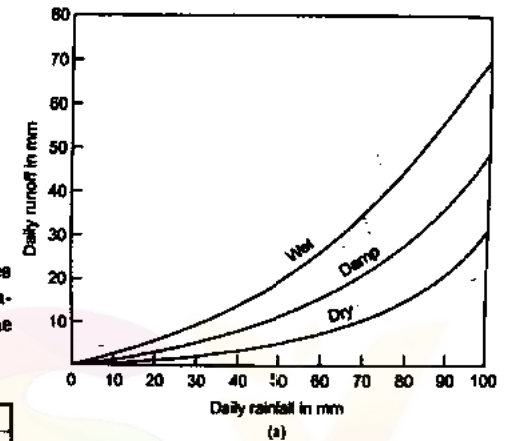


FIG. 4.41 STRANGE'S RUN-OFF CURVES.

7. Dhir, Ahuja and Majumdar formulae

$$\begin{aligned}
 R &= 120P - 4945 \text{ (for Chambal, Rajasthan)} & \dots(4.50 \ a) \\
 R &= 435P - 17200 \text{ (for Tapi, Gujrat)} & \dots(4.50 \ b) \\
 R &= 13400P - 575000 \text{ (for Damodar Valley, West Bengal)} & \dots(4.50 \ c) \\
 R &= 34.6P - 1510 \text{ (for Tawa, M.P.)} & \dots(4.50 \ d) \\
 R &= 13.18P + 86.5 \text{ (for Machakund, A.P.)} & \dots(4.50 \ e)
 \end{aligned}$$

8. UP Irrigation Research Institute formula

$$\begin{aligned}
 R &= 5.45 P^{0.5} \text{ (for Ganga, Haridwar)} & \dots(4.51 \ a) \\
 R &= 0.35 P^{1.1} \text{ (for Yamuna at Tajewala)} & \dots(4.51 \ b) \\
 R &= 2.7 P^{0.8} \text{ (for Sarda at Bana Basa)} & \dots(4.51 \ c)
 \end{aligned}$$

(C) RUN-OFF BY INFILTRATION METHOD

Infiltration is defined as the movement of water through the soil surface and into the soil. The capacity of any soil to absorb water from rainfall falling continuously at an excessive rate goes on decreasing with time, until a minimum rate to infiltration is reached. At any instant, the *infiltration capacity* of a soil is the maximum rate at which water will enter the soil in a given condition. The *infiltration rate* is the rate at which water actually enters the soil during a storm, and is equal to the infiltration capacity or the rainfall rate, whichever is less. The *infiltration capacity* of a soil can be determined experimentally by subjecting an experimental plot to rainfall rates in excess of infiltration capacity, and by measuring the surface run-off (Fig. 4.42). For small areas having uniform infiltration characteristics, the run-off volume can be estimated by subtracting infiltration from the design rainfall.

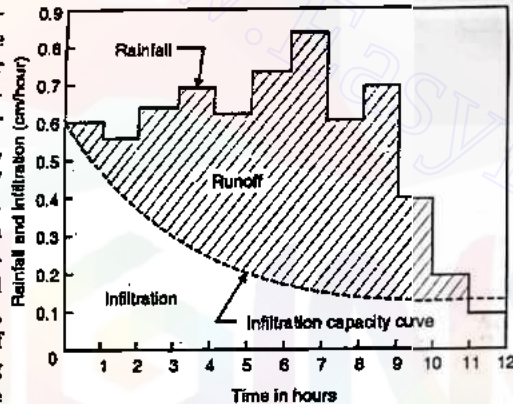


FIG. 4.42. INFILTRATION CAPACITY CURVE AND RUN-OFF

Infiltration Index. Infiltration index is the average rate of loss such that the volume of rainfall in excess of that rate will be equal to the direct run-off. Estimate of run-off volume from large areas having heterogeneous infiltration loss and rainfall characteristics, are made by use of *infiltration indices*.

There are two types of infiltration indices :

(i) *Average infiltration rate or W-index* and (ii) *Φ-index*.

The *W-index* is calculated from the expression :

$$W_i = \frac{P - R - S_R}{t_r} \text{ cm/hr} \quad \dots(4.52)$$

where t_r = duration of rainfall in hours, S_R = surface retention.

The Φ -index is defined as the average rate of rainfall above which the rainfall volume equal the run-off volume. Alternatively, it is defined as the average rate of loss such that the volume of rainfall in excess of that rate will be equal to volume of direct runoff. The runoff volume, above Φ -index is usually known as *rainfall excess* or *effective rainfall*.

$$\text{Thus, } \Phi_i = \frac{P - R}{t_r} \quad \dots(4.53)$$

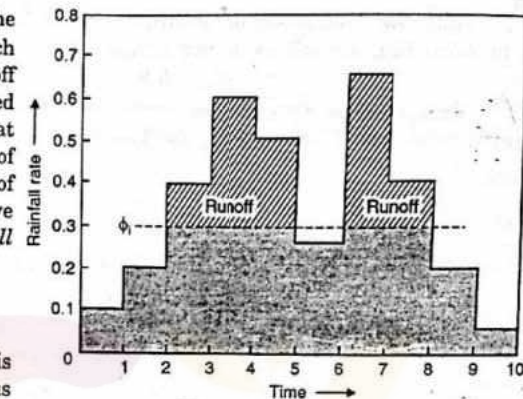
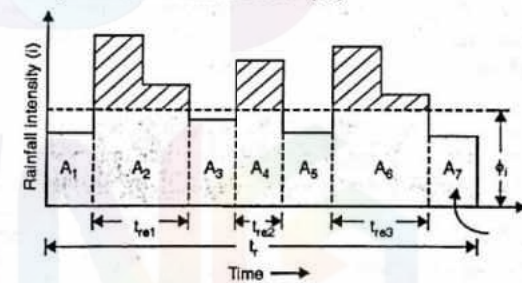
Thus, we find that *W-index* is essentially equal to Φ -index minus average rate of retention by interception and depression storage, and therefore, $W_i < \Phi_i$. Again refer Fig. 4.44, from which

$$\Phi_i = \frac{\text{Total infiltration during periods of rainfall excess}}{\text{Total period of rainfall excess } (t_{re})}$$

or $\Phi_i = \frac{A_2 + A_4 + A_6}{t_{re1} + t_{re2} + t_{re3}}$
where A_2, A_4, A_6 = infiltration during periods of rainfall excess
 $t_{re1}, t_{re2}, t_{re3}$ = individual periods of rainfall excess
and W_i

$$= \frac{A_1 + A_2 + A_3 + A_4 + A_5 + A_6 + A_7}{t_r}$$

where t_r = total period of rainfall

FIG. 4.43. Φ -INDEX AND RUN-OFF.FIG. 4.44. DETERMINATION OF Φ AND W_i

However, if there is uniform rain as shown in Fig. 4.45, Φ_i and W_i would be equal.

For flood forecasting, approximate index values must be derived by correlation with those factors which determine the index at any time. In such an approach, there seems to be no advantage over the method discussed earlier, in which the run-off and rainfall are correlated. However, the infiltration index can be used to estimate the run-off coefficient (k) from the relation,

$$k = \frac{i - W_i}{i} \quad \dots(4.54)$$

where i = rainfall intensity (cm/hr).

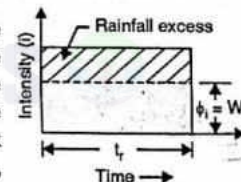


FIG. 4.45

Thus, the information on infiltration can be used to estimate the run-off coefficient k in computing run-off as a percentage of rainfall, by the relation :

$$R = kP$$

Central Water Commission (CWC) gave the following equation, on the basis of rainfall and runoff co-relations, for the determination of Φ -index.

$$R = \alpha (i)^{1.2}$$

$$\text{and } \Phi_i = \frac{i - R}{24} \quad \dots(4.55)$$

where R = runoff in cm from in 24 h rainfall.

i = rainfall intensity in cm/day

α = a coefficient which depends upon the soil type as given in Table 4.15.

TABLE 4.15. VALUE OF COEFFICIENT α

S.N.	Soil Type	α
1.	Sandy soil and sandy loam	0.17 to 0.75
2.	Coastal alluvium and silty loam	0.25 to 0.34
3.	Red soils, clayey loam, grey and brown alluvium	0.42
4.	Black-cotton and clayey soils	0.42 to 0.46
5.	Hilly soils	0.46 to 0.5

If sufficient data is not available, Φ -index value may be assumed equal to 0.1 cm/h for the purpose of estimating maximum floods.

Effective rainfall hyetograph (ERH)

As seen earlier, a hyetograph is a plot of the intensity of rainfall against the time interval, and it is represented as a bar chart. If this hyetograph is pruned by deducting losses (i.e. initial loss and infiltration losses), the resulting hyetograph is known as *effective rainfall hyetograph (ERH)*, or *hyetograph of rainfall excess or supra rainfall*.

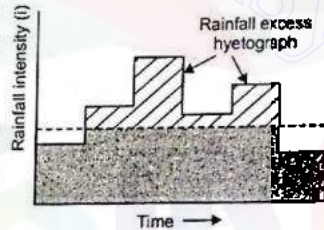


FIG. 4.46. EFFECTIVE RAINFALL HYETOGRAPH (ERH)

Example 4.17. A watershed has the following data :

1. Area of watershed : 82 km²
2. Distance between the outlet and further most point : 12.6 km
3. Total length of channels of various orders : 440 km
4. Elevation difference between outlet and further most point : 656 m

Determine (a) drainage density (b) form factor (c) channel slope and (d) average over land flow length.

Solution

(a) Drainage density, $D_d = \frac{\text{Total channel length}}{\text{Area of basin}} = \frac{440}{82} = 5.37 \text{ km/km}^2$

(b) Form factor, $F_f = \frac{A}{L_b^2} = \frac{82}{(12.6)^2} = 0.517$

(c) Channel slope $S = \frac{\Delta H}{L_b} = \frac{656}{12.6 \times 1000} = \frac{1}{19.21}$

(d) Average overland flow length, $L_o = \frac{1}{2 D_d} = \frac{1}{2 \times 5.37} = 0.093 \text{ km} = 93 \text{ m}$

Example 4.18. The rainfall rates for successive 30-minutes intervals upto 4 hours are given below. If the surface runoff is 3.6 cm, determine Φ and W indices.

Time (minutes)	0	30	60	90	120	150	180	210	240
Rainfall intensity (cm/h)	0	1.3	2.8	4.1	3.9	2.8	2.0	1.8	0.9

Solution : (a) *Computation of Φ -index*

Let i = rate of rain in cm/h and Φ -index be expressed in cm/h.

Then total $R = \Sigma (i - \Phi_i) t$ where t is the time in hours.

Assuming Φ_i to be greater than 1.3 cm/h, but less than 1.8 cm/h we have

$$3.6 = [(2.8 - \Phi_i) + (4.1 - \Phi_i) + (3.9 - \Phi_i) + (2.8 - \Phi_i) + (2.0 - \Phi_i) + (1.8 - \Phi_i)] \frac{30}{60}$$

From which we get $\Phi_i = 1.7 \text{ cm/h}$

Since this is greater than 1.3 and less than 1.8 cm/h, the above computations are correct.

(b) *Computation of W -index*

$$W_i = \frac{P - R - S_R}{t_r}$$

Here $P = 1.3 + 2.8 + 4.1 + 3.9 + 2.8 + 2.0 + 1.8 + 0.9 \frac{30}{60} = 9.8 \text{ cm}$

$R = 3.6 \text{ cm}$; $S_R = 0$ and $t_r = 4 \text{ hours}$

$$\therefore W_i = \frac{9.8 - 3.6 - 0}{4} = 1.55 \text{ cm/h}$$

Example 4.19. Table below gives the time distribution of a rainfall lasting for nine hours. If the direct runoff is 9.3 cm, determine the Φ -index of the storm and time of rainfall excess.

Time from start (h)	1	2	3	4	5	6	7	8	9
Incremental rainfall in each hour (cm)	0.7	1.4	2.4	3.7	2.9	2.6	1.7	0.8	0.5

Solution

Total rainfall = 0.7 + 1.4 + 2.4 + 3.7 + 2.9 + 2.6 + 1.7 + 0.8 + 0.5 = 16.7 cm

Total runoff = 9.3 cm

$\therefore$ Total infiltration = 16.7 - 9.3 = 7.4 cm

First trial : Let the time of rainfall excess (t_r) be 9 hours

Then $\Phi_i = \frac{7.4}{9} = 0.822 \text{ cm/hour.}$

This value of Φ_i makes rainfalls of first, eighth and ninth hour ineffective, as these magnitudes are less than 0.822 cm/hour.

Second trial : Hence assume $t_{re} = 6$ hours in the second trial.

For this revised period, infiltration = $(16.7 - 9.3) - (0.7 + 0.8 + 0.5) = 5.4$ cm

$$\therefore \Phi_i = \frac{5.4}{6} = 0.9 \text{ cm/hour.}$$

This value of Φ_i is satisfactory, since it gives $t_{re} = 6$ hours. Let us verify the rainfall excess (i.e. direct runoff) for these values.

Time from start (h)	1	2	3	4	5	6	7	8	9
Rainfall excess = $P - \Phi_i$	0	0.5	1.5	2.8	2.0	1.7	0.8	0	0

From the above table, rainfall excess (i.e. direct runoff) = 9.3 cm

Hence $\Phi_i = 0.9$ cm/hour and $t_{re} = 6$ hours

Example 4.20. Table below gives the observed values of yearly precipitation (P) and corresponding yearly runoff (R) for a catchment, for a period of 16 years. Develop a linear correlation equation between P and R .

Year	1970	1971	1972	1973	1974	1975	1976	1977
P (cm)	72.2	70.1	73.3	42.5	81.3	50.6	52.9	69.4
R (cm)	24.1	22.7	25.6	11.3	28.4	12.7	13.4	15.7
Year	1978	1979	1980	1981	1982	1983	1984	1985
P (cm)	60.3	64.3	68.8	56.7	77.2	40.5	44.1	65.5
R (cm)	16.2	17.7	19.2	14.9	25.4	10.6	11.7	17.9

Solution: Here, $N = 16$. Let the regression equation be $R = aP + b$.

Values of a and b are given by Eqs. 4.43.

The computations required for determination of a and b are done in a tabular form below.

S.N. of year	P (cm)	P^2 (cm ²)	R (cm)	R^2 (cm ²)	PR (cm ²)
1	72.2	5212.8	24.1	580.8	1740.0
2	70.1	4914.0	22.7	515.3	1591.5
3	73.3	5372.9	25.6	655.4	1876.5
4	42.5	1806.3	11.3	127.7	480.3
5	81.3	6609.7	28.4	806.6	2308.9
6	50.6	2560.4	12.7	161.3	642.6
7	52.9	2798.4	13.4	179.6	708.9
8	59.4	3528.4	15.7	246.5	932.6
9	60.3	3636.1	16.2	262.4	976.9
10	64.3	4134.5	17.7	313.3	1138.1
11	68.8	4733.4	19.2	368.6	1321.0
12	56.7	3214.9	14.9	222.1	844.8
13	77.2	5959.8	25.4	645.2	1960.9
14	40.5	1640.3	10.6	112.4	429.3
15	44.1	1944.8	11.7	136.9	516.0
16	65.5	4290.2	17.9	320.4	1172.5
Σ	979.7	62356.9	287.5	5654.5	18640.4

From the Table, we have $\Sigma P = 979.7$; $(\Sigma P)^2 = (979.7)^2 = 959812$; $\Sigma P^2 = 62356.9$

$$\Sigma R = 287.5; (\Sigma R)^2 = (287.5)^2 = 82656; \Sigma R^2 = 5654.5$$

$$\Sigma PR = 18640.4$$

$$a = \frac{N(\Sigma PR) - (\Sigma P)(\Sigma R)}{N(\Sigma P^2) - (\Sigma P)^2} = \frac{16 \times 18640.4 - (979.7) \times (287.5)}{16 \times 62356.9 - 959812} = 0.4376$$

$$b = \frac{\Sigma R - a\Sigma P}{N} = \frac{287.5 - 0.4376 \times 979.7}{16} = -8.826$$

Hence the equation is $R = 0.4376P - 8.826$

Example 4.21. For a catchment of 8.6 km² area, the mass curve of rainfall of 4 hours storm is given below. Determine the effective rainfall hyetograph and the volume of direct runoff from the catchment due to the above storm, taking Φ -index for the catchment as 0.8 cm/hour.

Time from start of storm (h)	0	0.5	1	0.5	2	2.5	3	3.5	4
Accumulated rainfall (cm)	0	0.4	1.1	2.3	3.8	4.8	5.6	6.2	6.7

Solution : For any time interval Δt , effective rainfall $P_e = P - \Phi_i \Delta t$, or = 0, whichever is larger. For the data given above, $\Delta t = 0.5$ hours. Hence $\Phi_i \Delta t = 0.8 \times 0.5 = 0.4$ cm. The computations for ordinates of effective rainfall hyetograph are done in a tabular form below. Column (7) gives the ordinates of effective rainfall hyetograph (ERH). Fig. 4.47 gives the ERH of the storm.

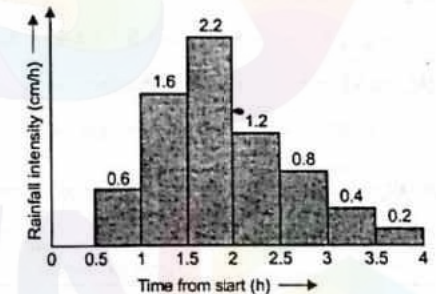


FIG. 4.47. EFFECTIVE RAINFALL HYETOGRAPH

Time from start (h)	Time interval Δt (hours)	Accumulated rainfall (cm)	Depth of rainfall in Δt (cm)	$\Phi_i \cdot \Delta t$ (cm)	Effective rainfall P_e (cm)	Intensity of effective rainfall (cm/h)
(1)	(2)	(3)	(4)	(5)	(6) = (4) - (5)	(7) = (6) / Δt
0	-	0	-	-	-	-
0.5	0.5	0.4	0.4	0.4	0	0
1.0	0.5	1.1	0.7	0.4	0.3	0.6
1.5	0.5	2.3	1.2	0.4	0.8	1.6
2.0	0.5	3.8	1.5	0.4	1.1	2.2
2.5	0.5	4.8	1.0	0.4	0.6	1.2
3.0	0.5	5.6	0.8	0.4	0.4	0.8
3.5	0.5	6.2	0.6	0.4	0.2	0.4
4.0	0.5	6.7	0.5	0.4	0.1	0.2

Direct runoff due to storm = Total effective rainfall = $(\Sigma \text{ordinates of ERH}) \cdot \Delta t$

$$= (0.6 + 1.6 + 2.2 + 1.2 + 0.8 + 0.4 + 0.2) \times 0.5 = 3.5 \text{ cm}$$

$$\text{Volume of direct runoff} = \frac{3.5}{100} \times 8.6 (1000)^2 = 30.1 \times 10^6 \text{ m}^3$$

Example 4.22. The rate of rainfall for successive 30 minutes periods of a 4-hour storm are as follows :

3.5, 6.5, 8.5, 7.8, 6.4, 4.0, 4.0, 6.0 cm/hr

Taking a value of Φ -index as 4.5 cm/hour compute the following :

(a) Total rainfall (b) Total rainfall excess, and (c) W_i .

Solution : The rainfall hyetograph is shown in Fig. 4.48 along with Φ -index

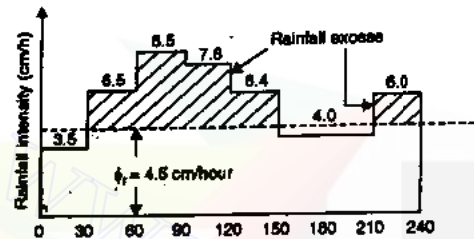


FIG. 4.48

(a) Total rainfall,

$$P = (3.5 + 6.5 + 8.5 + 7.8 + 6.4 + 4.0 + 4.0 + 6.0) \times \frac{30}{60} = 23.35 \text{ cm}$$

(b) Total rainfall excess = Hatched area

$$= [(6.5 - 4.5) + (8.5 - 4.5) + (7.8 - 4.5) + (6.4 - 4.5) + (6.0 - 4.5)] \times \frac{30}{60} = 6.35 \text{ cm}$$

$$(c) W_i = \frac{P - R}{t_r} = \frac{23.35 - 6.35}{4} = 4.25 \text{ cm/hour}$$

Example 4.23. A catchment area of 40 km² has one recording gauge. During a storm, the following mass curve of rainfall was recorded.

Time from start of rainfall (h)	0	2	4	6	8	10	12	14
Accumulated rainfall (mm)	0	8	22	74	92	105	114	120

If the volume of runoff due to the storm measured is $2.0 \times 10^6 \text{ m}^3$, estimate the Φ -index of the catchment

Solution

$$\text{Total run off volume} = 2.0 \times 10^6 \text{ m}^3, \text{ Catchment area} = 40 \times (1000)^2 \text{ m}^2$$

$$\therefore \text{Total runoff depth, } R = \frac{2.0 \times 10^6}{40 \times 10^6} = 0.05 \text{ m} = 50 \text{ mm}$$

$$\text{Total rainfall, } P = 120 \text{ mm}$$

$$\therefore \text{Total losses} = P - R = 120 - 50 = 70 \text{ mm}$$

$$\text{Duration of rainfall, } t_r = 14 \text{ hours.}$$

$$\therefore W_i = \frac{P - R}{t_r} = \frac{120 - 50}{14} = 5 \text{ mm/h}$$

It is to be noted that Φ_i will be more than this. Hence infiltration loss in every 2 hour interval will be more than 2×5 or more than 10 mm. Table below gives the

incremental rainfalls in each 2 hour interval. From the column (3) of the table, we find that there is no rainfall excess in 3 intervals of time, out of 7 time intervals of 2 hours each. Hence time of effective rainfall, $t_e = 4 \times 2 = 8$ hours only.

Time from the start of storm (h)	Accumulated rainfall (mm)	Incremental rainfall during each interval (mm)
(1)	(2)	(3)
0	0	-
2	8	8*
4	22	14
6	74	52
8	92	18
10	105	13
12	114	9*
14	120	6*

$$\text{Total rainfall excess} = (P - R) - (8 + 9 + 6) = 70 - 23 = 47 \text{ mm}$$

$$\therefore \Phi_i = \frac{47}{8} \approx 5.88 \text{ mm/h}$$

Example 4.24. A 12-hour storm rainfall with the following depths in cm occurred over a basin :

2.0, 2.5, 7.6, 3.8, 10.6, 5.0, 7.0, 10.0, 6.4, 3.8, 1.4 and 1.4.

The surface run-off resulting from the above storm is equivalent to 25.5 cm of depth over the basin. Determine the average infiltration index for the basin.

Solution : Total rainfall in 12 hours

$$= 2.0 + 2.5 + 7.6 + 3.8 + 10.6 + 5.0 + 7.0 + 10.0 + 6.4 + 3.8 + 1.4 + 1.4 = 61.5 \text{ cm}$$

$$\text{Total run-off in 12 hours} = 25.5 \text{ cm}$$

$$\therefore \text{Total infiltration in 12 hours} = 61.5 - 25.5 = 36 \text{ cm}$$

$$\therefore \text{Average infiltration} = \frac{36}{12} = 3.0 \text{ cm/hour.}$$

It will be observed from the above data that rainfall is less than average infiltration in 1st, 2nd, 11th and 12th hour. Hence during these hours, rate of infiltration will be equal to the rainfall. In the central period, the rainfall is more than infiltration.

Hence if f is the average rate of infiltration during the central 8 hours, we have

$$8f + 2.0 + 2.5 + 1.4 + 1.4 = 36$$

or

$$8f = 36 - 7.3 = 28.7$$

$$\therefore f = \frac{28.7}{8} \approx 3.6 \text{ cm/hour.}$$

$\therefore$ Average rate to infiltration = 3.6 cm/hour.

Example 4.25. From the storm data given in the previous example, determine the average depth of hourly rainfall excess for a basin of area of 120 hectares. The basin consists of areas A_1 , A_2 and A_3 having average infiltration indices given below:

Area designation	A_1	A_2	A_3
Area (hectares)	20	40	60
Infiltration Index (Φ) cm/hour	7.6	3.8	1.0

Solution

The run-off coming out from the individual areas A_1 , A_2 and A_3 will have to be spread over the entire basin and hence its depth will be reduced. The calculations are arranged in the tabular form below.

Time (h)	Rainfall (cm)	Rain-fall excess from			Average rain-fall excess on basin from			Total hourly rainfall excess in cm/hour over the entire basin
		Area A_1 ($\Phi_1 = 7.6$)	Area A_2 ($\Phi_2 = 3.8$)	Area A_3 ($\Phi_3 = 1.0$)	$A_1 = 20$ (5) $\times \frac{20}{180}$	$A_2 = 40$ (4) $\times \frac{40}{180}$	$A_3 = 60$ (5) $\times \frac{60}{180}$	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
1	2.0	-	-	1.0	-	-	0.50	0.50
2	2.5	-	-	1.5	-	-	0.75	0.75
3	7.6	-	3.8	6.6	-	1.27	3.30	4.57
4	3.8	-	-	2.8	-	-	1.40	1.40
5	10.6	3.0	6.8	9.6	0.50	2.27	4.80	7.57
6	6.0	-	1.2	4.0	-	0.40	2.00	2.40
7	7.0	-	3.2	6.0	-	1.07	3.00	4.07
8	10.0	2.4	6.2	9.0	0.40	2.07	4.60	6.97
9	6.4	-	2.6	5.4	-	0.87	2.70	3.57
10	3.8	-	-	2.8	-	-	1.40	1.40
11	1.4	-	-	0.4	-	-	0.20	0.20
12	1.4	-	-	0.4	-	-	0.20	0.20

4.20. HYDROGRAPH ANALYSIS

A *hydrograph* is a graph showing variations of discharge with time, at a particular point of a stream. It shows the time distribution of total runoff at the point of measurement.

As the runoff includes the contributions from surface run off, subsurface runoff (or inter flow) and ground water runoff (or base flow), the hydrograph can be regarded as an integral expression of the physiographic and climatic characteristics that govern the relations between rainfall and runoff. The factors affecting the hydrograph at a place being complex and inter-related, a basin may not produce two exact hydrographs with two similar precipitations as input, nor can two basins of the same drainage

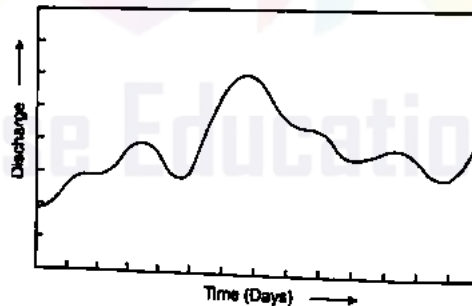


FIG. 4.49. STORM HYDROGRAPH

area produce the same flood hydrographs with similar precipitation. Since a hydrograph is generated from runoff due to precipitation resulting from either an isolated storm or a series of consecutive storms, (known as complex storm) it is also known as a *storm hydrograph*. The hydrograph resulting from an isolated storm will be a single peaked hydrograph, while there will be a multi-peaked hydrograph (Fig. 4.49) resulting from a complex storm.

Components of a single peaked hydrograph

Fig. 4.50 shows a single peaked hydrograph resulting from an isolated storm. The rainfall hystograph, along with infiltration curve, is shown on the top of the diagram.

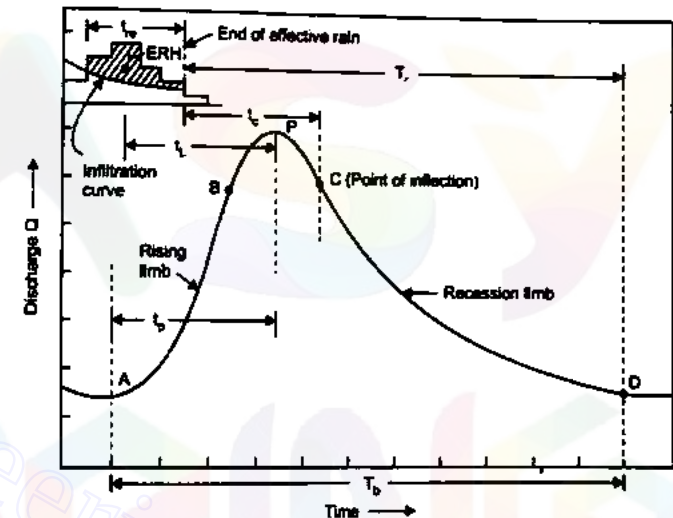


FIG. 4.50. COMPONENTS OF A SINGLE PEAKED HYDROGRAPH

There are three essential components of a single peaked hydrograph resulting from an isolated storm :

- The rising limb (AB)
- The peak or crest element (BPC)
- The recession limb (CD)

(i) Rising limb

The rising limb AB (Fig. 4.50) is the ascending portion of the hydrograph corresponding the increase of discharge due to gradual formation of storage in the channels existing in the area and also over the watershed surface. The rising limb is also known as the *concentration curve*. In the early period of a storm, there are initial losses as well as high infiltration losses and hence the discharge rises slowly. As the time increases, the initial losses stop and the infiltration rate also decreases. Due to this, more and more discharge from the distant parts reach the basin outlet (i.e. the point of measurement). Point A is the starting point while B is the point of inflection. The shape of the

rising limb is controlled by the characteristics of basin and duration, intensity and uniformity of the rain. The initial portion of the rising curve is concave due to the following reasons:

1. Greater losses due to interception, surface detention, evaporation and infiltration during initial period of rainfall.
2. Heavier concentration of area between adjacent isochrones is laid between middle and upper part of the watershed.

(ii) **The peak or crest segment (BPC)**

The peak or crest segment includes the part of the hydrograph from the inflection point (B) on the rising limb to an inflection point (C) on the recession limb. Crest segment is one of the most important component of the hydrograph because it indicates the peak flow rate. The peak represents the arrival of flow at the outlet from all parts of the basin. For large catchment, the peak flow (point P) occurs after the cessation of rainfall. The time interval between centre of mass of rainfall to the peak (designated as t_1) is controlled by the characteristics of both the basin as well as the storm. The peak may be sharp or flat depending upon the rainfall-catchment characteristics. Multiple peaked complex hydrographs in a basin can occur when two or more storms occur in close succession. The point of inflection represent the change point of contribution of the catchment to the channel system. The end of the crest segment, corresponding to the inflection point C, marks the time when direct runoff from the overland flow (excess rainfall) into the stream outlet ceases. The point of inflection (C) represents the condition of maximum detention storage (i.e. depth of water built-up over the land surface).

(iii) **The recession limb (CD)**

The recession limb extends from the point of inflection (point C) of the crest segment to the point D, the point of commencement of the natural ground water flow. The recession limb indicates the storage contribution from surface storage (which includes both the surface detention and channel storage), interflow and ground water flow. Since the depletion of this detention storage takes place after the cessation of rainfall, the shape of the recession curve depends only on the basin characteristics and is independent of rainstorm characteristics. The recession curve can be considered as a rate of discharge resulting from the draining off process. The equation of the recession curve may be expressed in the following form (Barnes, 1940) :

$$Q_t = Q_{r0} K_r^t \quad \dots(4.56)$$

where Q_t = discharge at any time interval t (days) after the point of inflection C
 Q_{r0} = discharge at the point of inflection C (at $t \approx 0$)

K_r = recession constant with a value less than unity.

Barnes observed that the above equation holds good for the lower portion of the curve where the contribution is mainly from ground water. For the upper portion of the curve where the contribution of discharge is from all the three phases (i.e. surface storage, subsurface flow and ground water flow) the recession constant K_r is composed of all the three components, and is given by

HYDROLOGY

where $K_r = K_{rs} \cdot K_{ri} \cdot K_{rg}$... (4.57)
 where K_{rs} = recession constant for surface storage
 (value varying from 0.08 to 0.4)

K_{ri} = recession constant for interflow (value varying from 0.55 to 0.9)
 and K_{rg} = recession constant for ground water flow (value varying from 0.9 to 1)

Time parameters used in hydrograph analysis

1. **Effective time duration (t_e)**: It is the net duration of precipitation during which rainfall rates are more than infiltration rates.

2. **Lag time or basin lag (t_L)**: It is the time interval between the centre of mass of net rainfall (or rainfall excess) and centre of mass of runoff hydrograph. More commonly, it is also taken as the time lapse between the centre of mass of effective rainfall and the peak (P) of the hydrograph, as marked in Fig. 4.50.

3. **Time to peak (t_p)**: It is the time interval between the starting of the rising limb (A) to the peak (P) of the hydrograph, as shown in Fig. 4.50.

4. **Time of concentration (t_c)**: The time of concentration (t_c) is defined in two ways. In terms of physical characteristics of watershed, it is defined as the travel time of a water particle from the hydraulically most remote point in the basin to the outflow location. However, based on rainfall and hydrograph characteristics, it is taken as the time from the end of the net rainfall to the point of inflection (C) of the falling limb of the hydrograph. It is estimated by the following formula by Kirpich (1940)

$$t_c = \frac{0.000323 L^{0.77}}{S^{0.385}} \quad \dots(4.58)$$

where t_c = time of concentration, in hours

L = maximum length of travel of water in metres.

S = slope of the channel = H/L

H = difference in elevation between the remote point in the channel and the outlet point.

5. **Recession time (T_r)**: It is the duration of direct runoff after the end of the effective rainfall duration. Thus, it is the difference in time between the end of effective rainfall and lowest point (D) of the recession limb.

6. **Time base of hydrograph (T_b)**: It is the time from which direct runoff begins to the time when it stops. In other words, it is the time from the beginning point (A) to the end point (D) of the direct runoff.

Separation of base flow

There are three popular methods of separation of base flow and direct runoff in a single peaked storm hydrograph shown in Fig. 4.50. The point A on the hydrograph (Fig. 4.50), which represents the beginning of the direct runoff, can be easily identified in view of sharp change in the run off rate at that point. But, the point D which marks the end of the direct runoff can not be easily located. However, an empirical equation for the time interval N (days) from the peak (P) to the point D is :

$$N = 0.827 A^{0.2} \quad \dots(4.59)$$

where A = area of drainage basin in km^2

Another empirical equation to determine N is given by

$$N = a t_p \quad \dots(4.60)$$

where t_p = time to peak = time interval between beginning point A and peak P .
 a = constant whose value may be taken between 2 to 4 for small basins and 4 for large basins.

Thus the two points where the direct runoff begins (i.e. point A) and ceases (i.e. point D) are identified.

Method 1 : Straight line method

Fig. 4.51 (a) shows the two points A and D which mark the beginning and end of direct runoff. In the straight line method, points A and D are joined by a straight line AD . This is the simplest method of separation.

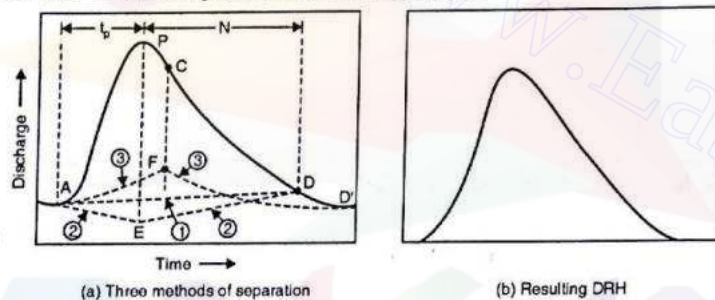


FIG. 4.51.

Method 2 : Two lines method

This is most widely used method in which the base flow is separated from direct runoff by two lines. The first line AE is obtained by extending the base flow curve, existing prior to the commencement of the surface runoff, till it intersects a vertical line, drawn from the peak (P) in a point E . Point E (so obtained) and point D , located earlier, are joined by a straight line. The segments marked by two line AE and ED demarcate the base flow and direct runoff.

Method 3 : Curves extension method

This method consists of extending the base flow curve on recession side backward till it intersects a vertical line through the point of inflection (C), in point F . Thus, in Fig. 4.51 (a), $D'F$ is the curve extended backwards. Points A and F are then joined by an arbitrary smooth curve. Thus the two segments marked by two curves AF and FD' demarcate the base flow and direct runoff. This method is preferred where the stream and ground water table are hydraulically connected and flow from ground water storage (the contribution of which may be significant) reaches the stream quickly.

The surface runoff hydrograph obtained after the base flow separation is known as direct runoff hydrograph (DRH), such as the one shown in Fig. 4.51 (b).

Actual base flow or depletion curve

The base flow or depletion curve AED (or AD , or AFD') determined above (Fig. 4.51 a) are, theoretical and may not represent actual conditions at the site. Actual contribution of ground water is shown by dotted line in Fig. 4.52 (b). As shown in Fig. 4.52 (b), when the water level before the storm is low, there is a flow from ground water to the stream. But as the water level in the stream is raised due to storm, there is a flow from the stream to the ground water. As the flood level in the stream recedes, the ground water again starts contributing to the stream. Thus, AED denotes the ground water line only if there were no flood. However, since the ground water contribution is small percentage of total flow, it is usually considered justified that the flow above the line AED is due to direct runoff.

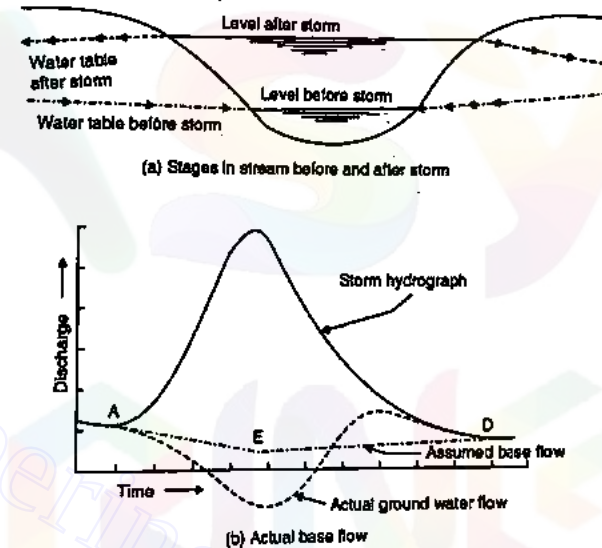


FIG. 4.52. ACTUAL BASE FLOW OR DEPLETION CURVE.

4.21. COMPUTATION OF DIRECT RUN-OFF OR RAIN FALL EXCESS FROM STORM HYDROGRAPH.

Procedure (Refer Table 4.16).

1. Find the ordinates of storm hydrograph, representing total discharge Q at a given time interval, say t hours.
2. Separate the ground water flow. Find the ordinates of the base flow at the same time interval.
3. Find the ordinates of direct run-off by subtracting the ordinates of base flow from total discharge ordinates.
4. The direct run-off in depth of water (in cm) is found from the expression

$$\text{Direct run-off} = \frac{\text{Total volume of direct run-off}}{\text{Area of basin}} = \left[\frac{(\Sigma O) \times t \times 60 \times 60}{A \times 10^6} \right] \times 100$$

$$= 0.36 \frac{(\Sigma O) \times t}{A} \text{ cm.} \quad \dots(4.61)$$

where t = Time interval between successive ordinates in hours.

A = Area of basin in sq. km.

Q = Discharge in cumecs.

ΣO = Sum of discharge ordinates (direct run-off) in cumecs.

TABLE 4.16

Date and Time (1)	Ordinate of hydrograph : Total Q (cumecs) (2)	Base flow (cumecs) (3)	Direct run-off (O) (4)
14-8-65			
0500	14	14	0
0800	25	12	13
1100	51	11	40
1400	65	10	55
1700	54	11	43
2000	28	13	15
2300	14	14	0
			$\Sigma O = 166$

For the example of Table 4.16, if area of the basin is 30 sq. km,

$$\text{Direct run-off} = 0.36 \frac{(\Sigma O) \times t}{A} = 0.36 \frac{166 \times 3}{30} = 5.98 \text{ cm}$$

4.22. UNIT HYDROGRAPH (UH)

A unit hydrograph is a hydrograph representing 1 cm (or 1 inch) of run off from a rainfall of some unit duration and specific areal distribution. Unit duration refers to the duration of a run-off producing rainfall-excess, that results in a unit hydrograph. For example, if a unit hydrograph results from a 3 hours unit rainfall duration, it is known as a 3-hours unit hydrograph, meaning thereby a hydrograph produced by surface run-off from a storm lasting for 3 hours and yielding rainfall excess of 1 cm. The unit duration may vary from one day for large catchments to a few hours for a small catchments. Experience has shown that best unit duration is about one fourth of the basin lag, the time from the centre of mass of net rainfall to the peak of the hydrograph. Sherman (1942) suggested the following values of unit time duration (Table 4.17)

TABLE 4.17. SUGGESTED VALUES OF UNIT TIME DURATION

Area of Basin	Unit time duration (hours)
> 2500 km ² (1000 mile ²)	12 to 24 (preferably 12)
250 - 2500 km ² (100 - 1000 mile ²)	6, 8 or 12
50 - 250 km ² (20 mile ²)	2
Small areas	One third to one fourth of the time of concentration.

The basis of the unit hydrograph concept, first given by L.K.Sherman (1932) is that if two identical rain storms could occur over a drainage basin with identical conditions prior to the rain, the hydrographs of run-off from the two storms would be expected to be the same. The basic premise implies that the rainfall excess of n centimetres within the selected unit duration will give a run-off hydrograph having ordinates n times as great as those of unit hydrograph of the same unit duration.

Assumptions of Unit Hydrograph Theory. The assumptions of unit hydrograph theory must be properly understood before applying it to actual problems, as these assumptions impose certain limitations which must be carefully noted. The various assumptions are given below.

1. The effective rainfall is uniformly distributed within its duration of specified period of time.

2. The effective rainfall is uniformly distributed throughout the whole area of the drainage basin.

3. The base or time duration of the hydrograph of direct run-off due to an effective rainfall of unit duration is constant.

4. The ordinates of direct run-off of common base-time are directly proportional to the total amount of direct run-off represented by each hydrograph.

5. For a given drainage basin the hydrograph of run-off due to a given period of rainfall reflects all the combined physical characteristics of the basin.

CONSTRUCTION OF UNIT HYDROGRAPH

The following are the steps for constructing a unit hydrograph of some unit duration from a storm hydrograph of the same unit duration (Fig. 4.53 and Table 4.18):

1. From the past records, select some unit period of intense rainfall duration corresponding to an isolated storm uniformly distributed over the area.

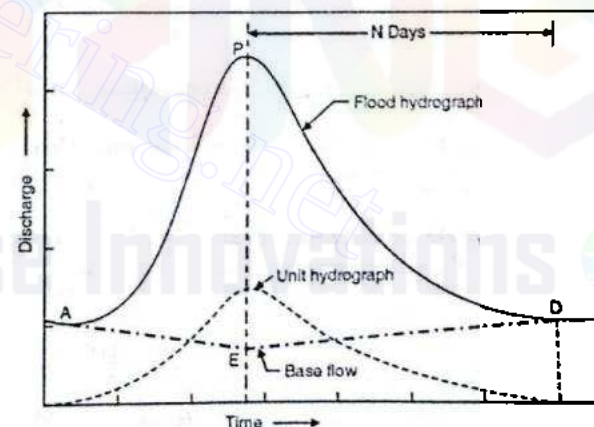


FIG. 4.53. CONSTRUCTION OF UNIT HYDROGRAPH

TABLE 4.18

Date	Hour	Total Q (cumecs)	Base flow (cumecs)	Ordinates of direct run-off (cumecs)	Ordinates of unit hydrograph
(1)	(2)	(3)	(4)	(5)	(6) = $\frac{(5)}{D.R.Q. \text{ in cm.}}$
12 Aug.	0600	6	6.0	0	0
	0800	8	6.0	2.0	0.08
	1000	10	6.5	4.5	0.18
	1200	16	5.0	11.0	0.47
	1400	28	4.5	23.5	0.99
	1600	42	4.0	38.0	1.61
	1800	60	3.5	56.5	2.39
	2000	80	3.0	77.0	3.25
	2200	110	2.5	107.5	4.54
	2400	100	2.5	97.5	4.11
13 Aug.	0200	90	2.5	87.5	3.70
	0400	80	3.0	77.0	3.25
	0600	68	3.0	65.0	2.74
	0800	56	3.5	52.5	2.22
	1000	45	3.5	41.5	1.75
	1200	35	4.0	31.0	1.31
	1400	28	4.0	22.0	0.92
	1600	18	4.5	13.5	0.57
	1800	11	4.5	6.5	0.27
	2000	9	5.5	4.0	0.17
	2200	8	5.5	2.5	0.11
	2400	7	5.5	1.5	0.06
	0200	6	6.0	0	0
				$\Sigma O = 822$	
14 Aug.					

2. From the past records of the river discharge for that storm, plot the storm hydrograph for some days before and after the period of rainfall of that unit duration.

3. By the method indicated in § 4.20, separate the ground water flow (base flow) from the direct run-off.

4. Subtracting the ordinates of base flow from the total ordinates, find the ordinates of direct run-off.

5. Calculate direct run-off n (in centimetres) by the expression :

$$\text{Direct run-off, } n = 0.36 \frac{(\Sigma O) \times t}{A} \text{ cm} \quad \dots(4.61)$$

where ΣO = Sum of the discharge ordinates (direct run-off) in cumecs.

t = Time interval between successive ordinates in hours.

A = Area of drainage basin in sq. km.

6. Calculate the ordinates of unit hydrograph by the relation :

$$\text{Ordinate of unit hydrograph} = \frac{\text{Ordinate of direct run-off}}{\text{Direct run-off } n \text{ in cm.}}$$

The calculations may be arranged in a tabular form shown in Table 4.18. From Table 4.18, $\Sigma O = 822$.

Area of basin = 25 sq. km.

$$\text{Direct run-off} = 0.36 \frac{(\Sigma O) \times t}{A} = \frac{0.36 \times 822 \times 2}{25} = 23.7 \text{ cm.}$$

APPLICATION OF THE UNIT HYDROGRAPH TO THE CONSTRUCTION OF A FLOOD HYDROGRAPH RESULTING FROM RAINFALL OF UNIT DURATION

The unit hydrograph can be used to construct a flood hydrograph resulting from rainfall of the same unit duration for which the unit hydrograph is available. The number of unit hydrographs for a given drainage basin are theoretically infinite since there may be one for every possible duration of rainfall and every possible distribution pattern in a basin. Practically only a limited number of unit hydrographs can be used for a given basin. The unit hydrograph selected for computing the flood hydrograph should be corresponding to the storm of like duration and pattern. However, a tolerance of as much as 25 % of the unit-hydrograph duration can ordinarily be accepted without much serious error. Thus a 6-hour unit hydrograph may be applied for storms of 4.5 to 7.5 hours duration. Alternatively, unit hydrograph having a similar range in duration may be averaged to obtain an average unit hydrograph.

Procedure. The calculations for the storm hydrograph corresponding to n cm of rainfall-excess (or direct runoff) are done in the tabular form below (Table 4.19).

Direct run-off ordinates = (ordinates of unit hydrograph) $\times n$ cm.

TABLE 4.19 ($n = 8$ cm)

Date	Hour	Ordinates of unit hydrograph (cumecs)	Ordinates of direct runoff (4) = (3) $\times n$	Base flow (cumecs)	Total discharge ordinates (6) = (4) + (5)
(1)	(2)	(3)	(4)	(5)	(6) = (4) + (5)
22 August	0600	0.00	0	4.0	4
	0800	0.12	0.96	3.5	4.46
	1000	0.35	2.80	3.0	5.80
	1200	0.83	7.04	2.5	9.54
	1400	1.50	12.00	2.0	14.00
	1600	2.80	22.40	1.5	23.90
	1800	2.00	16.00	1.8	17.80
	2000	1.85	14.80	2.1	16.90
	2200	1.53	12.24	2.4	14.64
	0000	1.26	10.08	2.7	12.78
	0200	0.84	6.72	3.0	9.72
	0400	0.50	4.00	3.3	7.30
	0600	0.35	2.80	3.6	6.40
	0800	0.12	0.96	3.8	4.76
	1000	0.00	0.0	4.0	4.0

APPLICATION OF UNIT HYDROGRAPH TO CONSTRUCTION OF A FLOOD HYDROGRAPH RESULTING FROM TWO OR MORE PERIODS OF RAINFALL

A unit hydrograph of some specific unit duration can also be utilised for construction of flood hydrograph resulting from the rainfall lasting for a long duration. The essential condition, however, is that the storm pattern should be the same as that for the unit

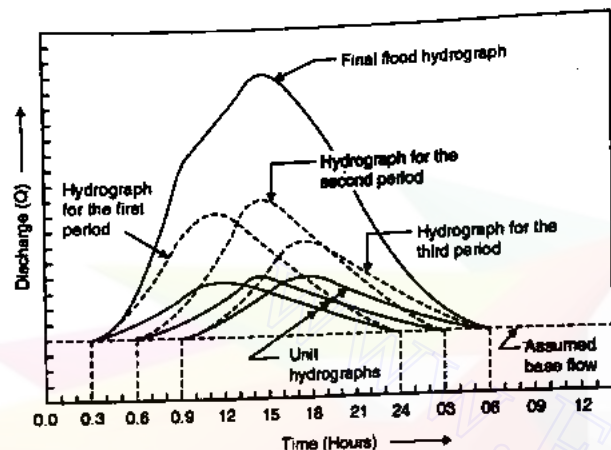


FIG. 4.54 FLOOD HYDROGRAPH RESULTING FROM A STORM OF LONGER DURATION

hydrograph. As an example, let us say that a 3-hours unit-hydrograph is available, and it is required to compute the flood hydrograph resulting from a rainfall lasting for 9 hours with variable intensities of rainfall — the intensity rates having n_1 cm/3-hours for the first period of 3 hours, n_2 cm/3-hours for the second 3 hours, and n_3 cm/3-hour for the last 3 hours. The storm is divided into 3 parts, and the flood hydrograph of each part is computed separately and added. The hydrographs for the second part of the storm starts 3 hours later than that for the first part. Similarly, the hydrograph for the third part of the storm starts 6 hours later than that for the first, or 3 hours later than that for the second part (Fig. 4.54). See Example 4.28 for illustration.

4.23. S-HYDROGRAPH (SUMMATION HYDROGRAPH)

S-hydrograph or S-curve is a hydrograph that is produced by a continuous effective rainfall at a constant rate for indefinite period. It is a continuous rising curve, in the form of letter S, till equilibrium is reached. At the time of equilibrium, it will represent a constant rate of continuous effective rainfall, say R_0 cm per hour. At the time of equilibrium, the S-curve will represent a run-off discharge given by

$$Q_0 = \frac{(A \times 100 \times 100) R_0}{100 \times 3600} = \frac{A R_0}{36} \text{ cumecs} \quad \dots(4.62)$$

where A is the catchment area in hectares.

If the catchment area A is in square kilometres, the discharge represented by S-curve, at the time of equilibrium is given by

$$Q_0 = \frac{(A \times 1000 \times 1000) R_0}{100 \times 3600} = 2.778 A R_0 \text{ (cumecs)} \quad \dots(4.63)$$

The S-hydrograph or S-curve is constructed by adding together a number of unit hydrographs of unit time duration (T_0) spaced at a unit time duration T_0 (i.e. duration of effective rainfall). This is illustrated in Table 4.20 and Fig. 4.56.

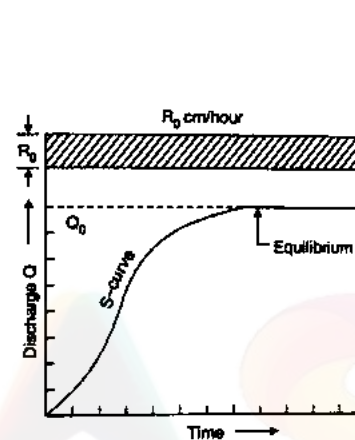


FIG. 4.55 S-HYDROGRAPH

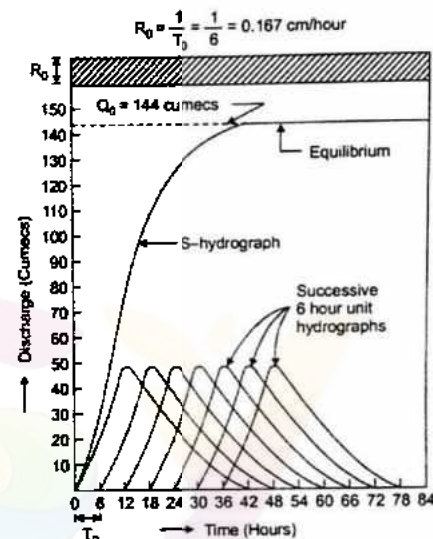


FIG. 4.56 DERIVATION OF S-HYDROGRAPH FROM UNIT HYDROGRAPH

TABLE 4.20. COMPUTATIONS FOR S-HYDROGRAPH

Time	Ordinate of unit hydrograph	Ordinates of successive unit hydrographs						Ordinate of S-hydrograph
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
00	0	—	—	—	—	—	—	0
06	9	—	—	—	—	—	—	9
12	20	0	—	—	—	—	—	20
18	35	9	—	—	—	—	—	44
24	49	20	0	—	—	—	—	69
30	43	35	9	—	—	—	—	87
36	35	49	20	0	—	—	—	104
42	28	43	35	9	—	—	—	115
48	22	35	49	20	0	—	—	126
54	17	28	43	35	9	—	—	132
60	12	22	35	49	20	0	—	138
66	9	17	28	43	35	9	—	141
72	6	12	22	35	49	20	0	144
78	3	9	17	28	43	35	9	144
84	0	6	12	22	35	49	20	144

Let us have a 6-hour unit hydrograph and construct a S-curve from it. Here $T_0 = 6$ hours. Column 1 of Table 4.20 shows the time intervals while column 2 shows the ordinates of a 6-hour unit hydrograph for a drainage basin having area $A = 311$

sq. km. Columns 3, 4, 5, 6, 7, 8 show the ordinates of successive unit hydrographs, each shifted by the unit time interval $= T_0 = 6$ hours. Column 9 shows the horizontal summation of the ordinates of the original unit hydrograph (column 2) and various successive unit hydrographs. Evidently, column 9 gives the ordinates of S-hydrograph. The summation is continued upto the time equal to the base of the unit hydrograph minus one unit duration ($T_0 = 6$ hrs.) to reach the equilibrium rate of discharge. It should be noted that the ordinates of S-hydrograph approach the rate corresponding to the effective rainfall (R_0) at the time of equilibrium. For the present case, the equilibrium discharge is given by

$$Q_0 = 2.778 AR_0 \quad \dots(4.63)$$

where $R_0 =$ constant rate of continuous effective rainfall $= \frac{1}{T_0}$ cm/hour $= \frac{1}{6}$ cm/hour

$A =$ catchment area in sq. km $= 311$ sq. km

$$Q_0 = \frac{2.778 \times 311}{6} = 144 \text{ cumecs.}$$

This rate is reached in Table 4.20 at 36 hours (= base period - T_0 hours).

Alternative Method

An alternative simple method to compute ordinates of S-hydrograph is illustrated in Table 4.21 for the same data. Column 1 shows Line No., Column 2 shows time, while column 3 shows the ordinates of unit hydrograph. Column 5 shows the ordinates of S-hydrograph (to be computed) in which the values in the first two lines (upto $T_0 = 6$ hours) are the same as the corresponding values of unit hydrograph in column (3). These two values of Lines 1 and 2 of column (5) are entered in column 4 (offset ordinates) at an offset $= T_0 = 6$ hours against line 3 and 4 respectively.

TABLE 4.21. COMPUTATIONS FOR S-HYDROGRAPH (ALTERNATIVE METHOD)

Line No. (1)	Time (hours) (2)	Ordinates of 6-hours unit hydrograph (3)	Off-set ordinates (4)	Ordinates of S-hydrograph ($R_0 = 1/6$ cm/hr.) (5)
1	00	0	-	0
2	03	9	-	9
3	06	20	0	20
4	09	35	9	44
5	12	49	20	69
6	15	43	44	87
7	18	35	69	104
8	21	28	87	115
9	24	22	104	126
10	27	17	115	132
11	30	12	126	138
12	33	9	132	141
13	36	6	138	144
14	39	3	141	144
15	42	0	144	144

Now lines 3 and 4 of columns (3) and (4) are respectively added horizontally, and entered in lines 3 and 4 of column (5), thus giving values of ordinates as $20 + 0 = 20$ and $35 + 9 = 44$ cumecs. These two values of lines 3 and 4 of column (5) are shifted to lines 5 and 6 respectively of column 4. Now lines 5 and 6 of columns (3) and (4) are respectively added horizontally and entered in lines 5 and 6 respectively, of column 5 thus getting ordinates of 69 and 87. This process of shifting or offsetting the ordinates of column (5) by time interval T_0 , entering into column (4) and adding with corresponding ordinates of column (3) to get new ordinates for column (5) is continued, till the base of the unit hydrograph is reached, or till the equilibrium discharge is reached. Sometimes, the computed ordinates of S-curve may not fall along a smooth curve. In that case, smoothening is carried out and the corresponding new ordinates along smoothened curves are found.

It should be noted that the S-hydrograph so derived from a 6 hours unit hydrograph represents constant $R_0 = 1/T_0 = 1/6$ cm/hour. The technique of constructing a S-hydrograph is very useful in deriving a unit hydrograph of shorter or longer unit duration from a given unit hydrograph of same unit duration. This is explained in the next article.

4.24. CONSTRUCTION OF UNIT HYDROGRAPH OF DIFFERENT UNIT DURATION FROM A UNIT HYDROGRAPH OF SOME GIVEN UNIT DURATION

(a) Construction of Longer Period Unit hydrograph from a Given Unit Hydrograph of Shorter Unit period

Let it be required to derive a unit hydrograph of unit duration t_0 hours, from a unit hydrograph of unit duration T_0 where $t_0 > T_0$. If t_0 is an integral multiple of T_0 this can be done by the simple principle of superposition.

Let it be required to derive a unit hydrograph of 6 hour unit duration ($t_0 = 6$ hours) from a given unit hydrograph of 2 hours duration ($T_0 = 2$ hours). This can be obtained by taking the sum of the ordinates of the three unit hydrographs of 2 hours duration, each lagging from the other by 2-hours, and dividing the sum by 3. This is illustrated in Fig. 4.57, in which it is clear that the peak discharge (Q_m) of the new hydrograph is less than Q_m of the original hydrograph, and also that the peak occurs later. If, however, t_0 is not an integral multiple of T_0 , the method of S-hydrograph is used, as described below.

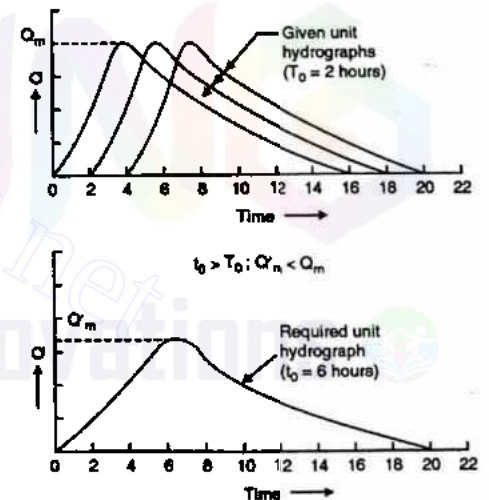


FIG. 4.57

(b) Construction of Shorter or Longer Period Unit Hydrograph from a Given Unit Hydrograph

Let it be required to obtain a unit hydrograph of unit period t_0 from a given unit hydrograph of unit period T_0 , where t_0 can be either greater or smaller than T_0 . For this, the S-hydrograph method described below may be used.

From the given unit hydrograph of unit period T_0 , S-curve can be derived, as discussed in § 4.23. This S-curve will represent a constant effective rainfall of $R_0 = 1/T_0$ cm/hour. Fig. 4.58 (a) shows the S-curve so obtained. An offset curve is then drawn by advancing or offsetting the position of original S-curve for a period equal to the desired unit period t_0 hours. The difference between the ordinates of original S-curve and offset S-curve, divided by $R_0 t_0$ will give the ordinate of the desired unit hydrograph [Fig. 4.58 (b)].

Thus, at any time period t , if the difference between the ordinates of the two S-curves is Δy , then the ordinate of the desired unit hydrograph of t_0 unit period

$$= \frac{\Delta y}{R_0 t_0} = \frac{\Delta y}{(1/T_0) t_0} = \Delta y \cdot \frac{T_0}{t_0}$$

Thus, ordinates at various time intervals can be computed and the desired unit hydrograph may be obtained.

For numerical illustration, let it be required to obtain unit hydrograph of 3 hours unit duration from the hydrograph of 6 hours unit duration given in Table 4.21. Column (2) of Table 4.22 gives the ordinates of the given hydrograph of 6 hours unit duration. Column (4) gives the ordinates of S-hydrograph, derived from the given unit hydrograph. This S-hydrograph represents $R_0 = 1/T_0 = 1/6$ cm/hour.

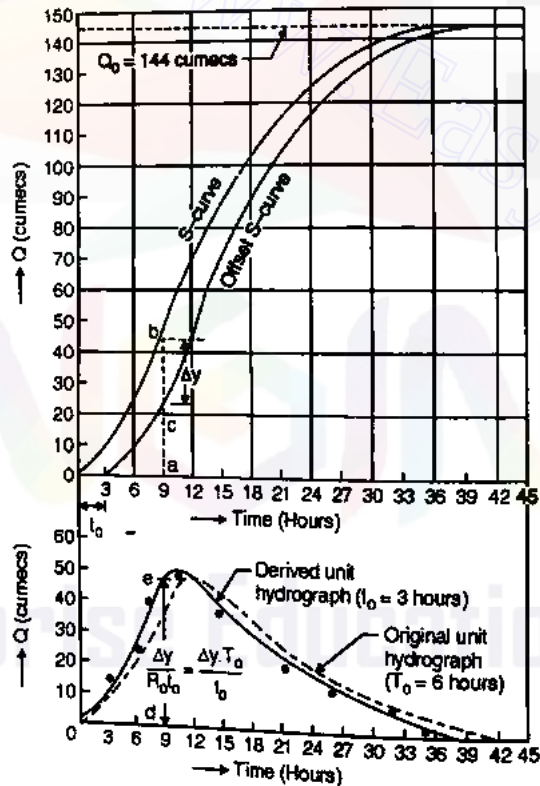


FIG. 4.58

TABLE 4.22. DERIVATION OF UNIT HYDROGRAPH OF $t_0 (= 3$ HOURS) UNIT DURATION FROM THE ONE OF $T_0 (= 6$ HOURS) UNIT DURATION

Time (1)	Ordinate of unit hydro- graph (2)	Offset ordinate (3)	Ordinate of S-curve (4)	Ordinate of offset S-curve (5)	Δy (cumecs) (6)	$\Delta y \cdot \frac{T_0}{t_0}$ $= 2 \Delta y$ (7)	Ordinate of 3 hr. hydrograph smoothened (8)
0	0	—	0	—	0	0	0
0.8	9	—	9	—	9	18	18
0.6	20	—	20	—	9	22	23
0.8	35	—	44	—	20	48	47
1.2	49	20	69	—	44	50	49
1.5	43	44	87	—	69	36	40
1.8	35	69	104	—	87	34	32
2.1	28	87	115	—	104	22	25
2.4	22	104	126	—	115	22	19
2.7	17	115	132	—	126	6	14
3.0	12	126	138	—	132	6	10
3.3	9	132	141	—	138	6	7
3.6	6	138	144	—	141	6	4
3.9	3	141	144	—	144	0	0
4.2	0	144	144	—	144	0	0
					$\Sigma 0$	288	288

Column (5) gives the ordinates of the offset S-curve, shifted by $t_0 = 3$ hours. Column (6) gives the difference Δy of the columns (4) and (5) of the two S-curves. Column (7) gives the ordinates of the required unit graph of 3 hours duration. In general, the ordinates of the unit hydrograph so obtained does not fall on a smooth curve. In that case smoothening has to be carried out [Fig. 4.58(b)]. Column 8 shows the new values of ordinates along the smoothened curve. In each case $\Sigma 0$ should be the same. As a check, $\Sigma 0 = 288$.

$$R = 0.36 \frac{\Sigma 0}{A} \times t = \frac{0.36 \times 288}{311} \times 3 = 1 \text{ cm.}$$

4.25. DERIVATION OF UNIT HYDROGRAPH FROM COMPLEX STORMS

The simple approach for the determination of unit hydrograph discussed in § 4.20 is not always applicable, as an ideal storm assumed in the derivation is seldom available in practice. For the case of complex storms, the following procedure is adopted to derive the unit hydrograph.

Procedure (Fig. 4.59)

(1) Let the direct run-off ordinates of the given complex hydrograph be represented by $Q_1, Q_2, Q_3, \dots, Q_n$ in successive periods during the storm.

(2) Let the ordinates of the unit hydrograph be designated by $U_1, U_2, U_3, \dots, U_m$ at the successive time intervals.

(3) For the purpose of illustration, we shall assume that the whole period of complex storm has three storms, each of 'effective duration' equal to 'unit duration' for which unit hydrograph is to be drawn. On to the hyetographs of rainfall, superimpose the infiltration rate curves and find the 'rainfall excess' rates. Let R_1, R_2 , and R_3 be the respective rainfall-excesses for the three storms. Thus, for the whole duration,

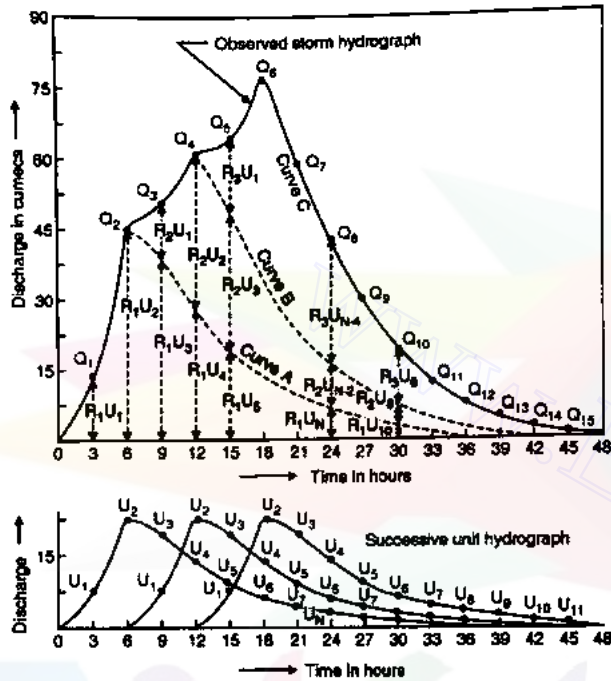


FIG. 4.59

there will be three similar unit hydrographs, each operating at a time lag equal to the 'unit duration'.

(4) In Fig. 4.59, the curve with thick line is the observed storm hydrograph. We observe that :

$$\text{First ordinate } Q_1 = R_1 U_1 \quad \dots(1)$$

Knowing R_1 and Q_1 , U_1 is known.

$$\text{Second ordinate } Q_2 = R_1 U_2 \quad \dots(2)$$

Knowing R_1 and Q_2 , U_2 is known.

$$\text{Third ordinate } Q_3 = R_1 U_3 + R_2 U_1 \quad \dots(3)$$

Knowing R_1 , R_2 , Q_3 , U_1 and U_2 , U_3 is known.

$$\text{Fourth ordinate } Q_4 = R_1 U_4 + R_2 U_2 \quad \dots(4)$$

Hence U_4 is known.

$$\text{Fifth ordinate } Q_5 = R_1 U_5 + R_2 U_3 + R_3 U_1 \quad \dots(5)$$

Hence U_5 is known.

$$N \text{th ordinate } Q_N = R_1 U_N + R_2 U_{N-2} + R_3 U_{N-4} \quad \dots(N)$$

where N is less than m

Thus the ordinates $U_1, U_2, U_3, \dots, U_m$ of the unit hydrograph are known.

In Fig. 4.59, ordinates of curve A are equal to $R_1 \times$ successive ordinates of the first unit hydrograph. Similarly, ordinates of curve B = (successive ordinates of curve A) + ($R_2 \times$ successive ordinates of second unit hydrograph).

And, ordinates of curve C = (successive ordinates of curves A + B) + $R_3 \times$ successive ordinates of Third Unit hydrograph). See example 4.29 for illustration.

Example 4.26. The following table gives the observed total runoff hydrograph for a drainage basin of 92 km². The ordinates of the rainfall mass curve, which produced the above runoff are also given. Derive the unit hydrograph.

Date and time	Runoff (m ³ /s)	Date and time	Runoff (m ³ /s)
20-7-60		21-7-60	
06	10.6	02	58.2
08	9.7	04	48.0
10	107.8	06	38.2
12	175.6	08	29.4
14	183.9	10	20.2
16	150.3	12	14.0
18	126.2	14	10.2
20	106.9	16	10.4
22	90.0	18	
24	72.8	20	

Date	Time (h)	Cumulative rainfall (cm)
20-7-60		
	06	0
	08	2
	10	4.8
	12	10.8
	14	18.8
	16	21.4
	18	21.4

Solution : Fig. 4.60 (a) shows the storm hydrograph while Fig. 4.60 (b) shows the rainfall hyetograph, each constructed adopting a time interval $\Delta t = 2$ hours. The point of rise (A) is easily located at the end of 08 hours of 20th July. The time distance

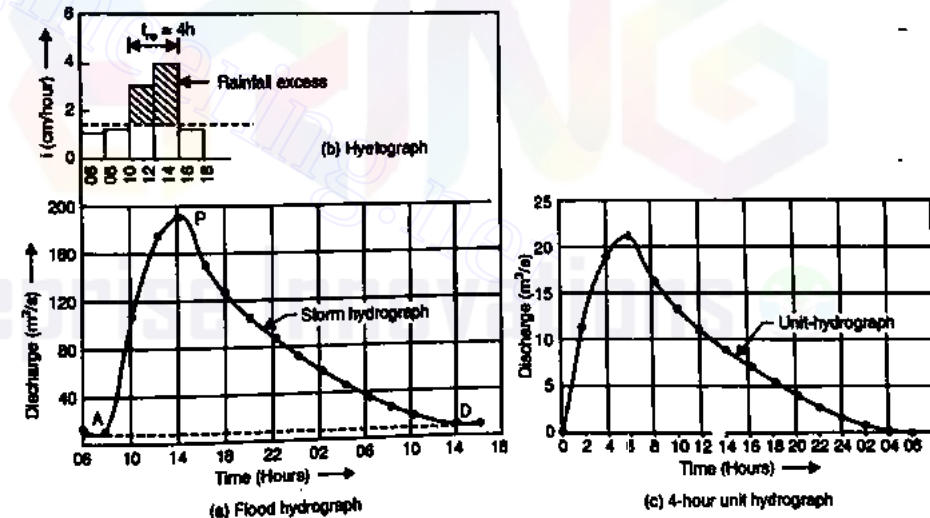


FIG. 4.60

N for the location of point D is given by

$$N = 0.827 (A)^{0.2} = 0.827 (92)^{0.2} = 2.043 \text{ days } \pm 49 \text{ hours}$$

From the plotted hydrograph, we observe that this value of N is too large. Hence the end point (D) of direct runoff is arbitrarily fixed at 14 hours of 21st July. The base line AD is thus obtained by joining points A and D .

The ordinates of direct-runoff are now computed by subtracting the base flow from total runoff, as shown in Table below.

Date	Time (hours)	Total runoff (m ³ /s)	Base flow (m ³ /s)	Direct runoff ordinate (O) (m ³ /s)	Ordinate of unit hydrograph (m ³ /s)	Time of start of direct-runoff (h)
(1)	(2)	(3)	(4)	(5) = (3) - (4)	(6) = (5)/ n	(7)
20-7-60	06	10.6	10.6	0		
	08	9.7	9.7	0		
	10	107.8	9.73	98.07	11.51	00
	12	175.6	9.77	165.83	19.46	02
	14	193.9	9.80	184.10	21.61	04
	16	150.3	9.83	140.47	16.49	06
	18	126.2	9.87	116.33	13.65	08
	20	106.9	9.90	97.00	11.38	10
21-7-70	22	90.0	9.93	80.07	9.40	12
	24	72.8	9.97	62.83	7.37	14
	02	58.2	10.00	48.20	5.66	16
	04	48.0	10.03	37.97	4.46	18
	06	36.2	10.07	26.13	3.07	20
	08	28.4	10.10	18.30	2.15	22
	10	20.2	10.13	10.07	1.18	24
	12	14.0	10.16	3.84	0.45	26
	14	10.2	10.20	0.00	0.00	28
	16	10.4	10.40	0.00		
				Σ	1089.21	

$$n = \frac{0.36 \Delta t \Sigma O}{A} = \frac{0.36 \times 2 (1089.21)}{92} = 8.52 \text{ cm}$$

Hence the ordinates of unit hydrograph (column 6) are obtained by dividing the ordinates of direct runoff (column 5) by $n = 8.52$.

In order to determine unit duration, let us first determine the value of infiltration index Φ_i . The rainfall intensities during the successive 2-hr periods are $\frac{2-0}{2} = 1.0 \text{ cm/hr}$, $\frac{4.6-2.0}{2} = 1.3 \text{ cm/hr}$, $\frac{10.8-4.6}{2} = 3.1 \text{ cm/hr}$, $\frac{18.8-10.8}{2} = 4.0 \text{ cm/hr}$, $\frac{21.4-18.8}{2} = 1.3 \text{ cm/hr}$ and $\frac{21.4-21.4}{2} = 0 \text{ cm/hr}$. To start with, let us assume that $\Phi_i > 1.3 \text{ cm/hr}$. The total rainfall excess or effective rainfall (P_e) is then given by

$$P_e = [(3.1 - \Phi_i) + (4.0 - \Phi_i)] 2$$

This should be equal to depth (n) of direct runoff. Hence we have

$$[(3.1 - \Phi_i) + (4.0 - \Phi_i)] 2 = n = 8.52 \text{ cm which gives } \Phi_i = 1.42 \text{ cm/hr.}$$

Since this greater than 1.3 cm/hr, no further trials are required. The line $\Phi_i = 1.42 \text{ cm/hr}$ is drawn on in Fig. 4.60 (b), to get the effective rainfall hyetograph, from which we get effective rainfall duration (t_{re}) equal to 4 hours. Hence the unit time duration is 4 hours and the resulting unit hydrograph is a 4-hr unit hydrograph.

Example 4.27. Given below are the observed flows from a storm of 6 hours duration on a stream with a drainage area of 316 sq. km.

Time	Flow (in cumecs)	Time	Flow (in cumecs)
June 1, 0.0 A.M.	17.0	June 3, 0.0 A.M.	53.8
6.0 A.M.	113.2	6.0 A.M.	42.5
Noon	254.5	Noon	31.1
6.0 A.M.	198.0	6.0 P.M.	22.6
June 2, 0.0 A.M.	150.0	June 4, 0.0 A.M.	17.0
6.0 A.M.	113.2		
Noon	87.7		
6.0 P.M.	67.9		

Assuming constant base flow of 17.0 cumecs, derive and plot a 6 hours unit hydrograph. How many cm of rainfall excess does the above storm hydrograph represent?

Solution : The example has been solved in the following tabular form below

Time (1)	Ordinates of storm hydrograph (cumecs) (2)	Base flow (cumecs) (3)	Ordinates of direct run-off (4) = (2) - (3)	Ordinates of unit hydrograph (5) = (4)/6.477
June 1, 0.0 A.M.	17.0	17	0	0
6.0 A.M.	113.2	17	96.2	14.9
Noon	254.5	17	237.5	36.6
6.0 A.M.	198.0	17	181.0	28.0
June 2, 0.0 A.M.	150.0	17	133.0	20.5
6.0 A.M.	113.2	17	96.2	14.9
Noon	87.7	17	70.7	10.9
6.0 P.M.	67.9	17	50.9	7.86
June 3, 0.0 A.M.	53.8	17	36.8	5.69
6.0 A.M.	42.5	17	25.5	3.94
Noon	31.1	17	14.1	2.18
6.0 P.M.	22.6	17	5.6	0.87
June 4, 0.0 A.M.	17.0	17	0	0
Sum			947.5	

$$\text{Rainfall excess} = \frac{947.5 \times 60 \times 60 \times 6 \times 10^6}{316 \times 10^6 \times 10^6} = 6.477 \text{ cm.}$$

Alternatively from Eq. 4.61 :

$$\text{Direct run-off} = 0.36 \frac{\Sigma O \times t}{A} = 0.36 \frac{947.5 \times 6}{316} = 6.477 \text{ cm.}$$

Example 4.28. Find the ordinates of a storm hydrograph resulting from a 3 hour storm with rainfall of 2, 6.75 and 3.75 cm during subsequent 3 hours intervals. The ordinates of unit 3-hour hydrograph are given in the following table :

Hours	03	06	09	12	15	18	21	24	03	06	09	12	15	18	21	24
Ordinates of unit hydrograph (cumecs)	0	110	365	500	390	310	250	235	175	130	95	65	40	22	10	0

Assume an initial loss of 5 mm, infiltration index of 2.5 mm/hour and base flow of 10 cumecs.

Solution

- (i) Rainfall excess during the first three hours
 $= 20 - (2.5 \times 3) - 5 = 7.5 \text{ mm} = 0.75 \text{ cm.}$
- (ii) Rainfall excess during the second three hours
 $= 67.5 - (3 \times 2.5) = 60 \text{ mm} = 6 \text{ cm.}$
- (iii) Rainfall excess during the last three hours
 $= 37.5 - (3 \times 2.5) = 30 \text{ mm} = 3 \text{ cm}$

Rainfall excess as ratio of unit rainfall of 1 cm during the subsequent 3 hours intervals are 0.75, 6 and 3. The example has been solved in a tabular form below. The computations of run-off due to 0.75 cm rainfall excess will start from 03 hours. The computations of run-off due to 6 cm rainfall excess will start from 06 hours. Lastly, the computations of run-off due to 3 cm rainfall excess will start from 09 hours.

Time in hours	Ordinates of 3-hour unit hydrograph (cumecs)	Rainfall excess cm/2 hours	Surface run-off from rain fall excess during successive unit periods (cumecs)				Base flow (cumecs)	Total discharge (cumecs)
			0.75	6.0	3.0	Sub total		
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
03	0	0.75	0			0	10	10.0
06	110	6.00	82.5	0		82.5	10	92.5
09	365	3.0	274.0	660	0	934.0	10	944.0
12	500		375.0	2190	330	2895.0	10	2905.0
15	390		295.5	3000	1045	4337.5	10	4347.5
18	310		232.5	2340	1500	4072.5	10	4082.5
21	250		187.5	1860	1170	3217.5	10	3227.5
24	235		176.0	1500	930	2606.0	10	2616.0
03	175		131.5	1410	750	2291.5	10	2301.5
06	130		97.5	1050	705	1852.5	10	1862.5
09	95		71.3	780	525	1376.3	10	1386.0
12	65		48.6	570	390	1008.6	10	1018.6
15	40		30.0	390	285	705.0	10	715.0
18	22		16.5	240	195	451.5	10	461.5
21	10		7.5	132	120	259.5	10	269.5
24	0		0	60	66	126.0	10	136.0
30				0	30	30.0	10	40.0
06					0	0	10	100.0

Example 4.29. The following direct run-off hydrograph resulted from three successive 6 hours periods of rainfall having run-off estimated as 2, 4 and 3 cm respectively :

Time (hr)	0	3	6	9	12	15
18	21	24	27	30	33	36
Flow (cumecs)	0	21	80	82	189	123
	184	87	55.5	25.25	9	6

Using the method of complex hydrographs, derive and plot a 6 hour unit hydrograph for the basin having area = 103.4 sq. km.

Solution

Let the unit hydrograph have ordinates of $U_0, U_3, U_6, U_9, U_{12}, U_{15}, U_{18}, U_{21}, U_{24}$, at 3 hours intervals. The direct run-off hydrograph will be obtained by multiplying the unit hydrograph values with 2, 4 and 3 cm respectively with successive hydrographs having six hours lag.

$$\text{Then } U_0 = 0$$

$$2U_3 = 21$$

$$2U_6 + 4U_0 = 80$$

$$2U_9 + 4U_3 = 82$$

$$2U_{12} + 4U_6 + 3U_0 = 189$$

$$2U_{15} + 4U_9 + 3U_3 = 123$$

$$2U_{18} + 4U_{12} + 3U_6 = 184$$

$$2U_{21} + 4U_{15} + 3U_9 = 87$$

$$2U_{24} + 4U_{18} + 3U_{12} = 55.5$$

$$\text{or } U_0 = 0$$

$$\text{or } U_3 = 10.5 \text{ cumecs}$$

$$\text{or } U_6 = 40 \text{ cumecs}$$

$$\text{or } U_9 = 20 \text{ cumecs}$$

$$\text{or } U_{12} = 14.5 \text{ cumecs}$$

$$\text{or } U_{15} = 5.75 \text{ cumecs}$$

$$\text{or } U_{18} = 3.0 \text{ cumecs}$$

$$\text{or } U_{21} = 2.0 \text{ cumecs}$$

$$\text{or } U_{24} = 0.0 \text{ cumecs}$$

Therefore, ordinates of unit hydrograph are as follows :

Time hour	0	3	6	9	12	15	18	21	24	Total
Flow cumecs	0	10.5	40	20	14.5	5.75	3.0	2.0	0	95.75

The ordinates of unit hydrograph will be correct if it gives a direct run-off of 1 cm.

$$\text{Direct run-off} = \frac{95.75 \times 3 \times 3600}{103.4 \times 10^4} \approx 1 \text{ cm.}$$

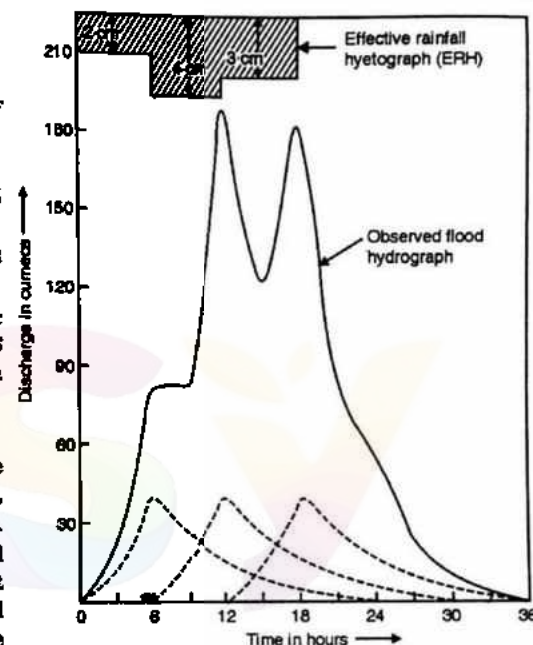


FIG. 4.61

The successive unit hydrographs will have the same ordinates but will be shifted laterally by 6 hours as shown in Fig. 4.61.

Example 4.30. The hourly ordinates of a two-hour unit hydrograph are given below. Derive a 6-hours unit hydrograph for the same catchment.

Time (hours)	00	01	02	03	04	05	06	07	08	09	10	11	12	13	14	15
Discharge (cumecs)	0.0	1.0	2.7	5.0	8.0	9.8	9.0	7.5	6.3	5.0	4.0	2.9	2.1	1.3	0.5	0.0

Solution Given $T_0 = 2$ hours ; $t_0 = 6$ hours.

Since t_0 is an integral multiple of T_0 (i.e. $t_0 = 3T_0$), the desired hydrograph can be obtained by taking the sum of the ordinates of three unit hydrographs of 2-hours duration, each lagging from the other by 2-hours, and dividing the sum by 3. The computations are arranged in table below.

Time	Given 2 hours unit hydrograph	Offset unit hydrograph	Offset unit hydrograph	Sum	Ordinates of 6 hr unit hydrograph	Ordinate of 6 hr moderated hydrograph
00	0.0	-	-	0.0	0.00	0.0
01	1.0	-	-	1.0	0.33	0.3
02	2.7	0	-	2.7	0.90	0.9
03	5.0	1.0	-	6.0	2.00	2.0
04	8.0	2.7	0	10.7	3.57	3.6
05	9.8	5.0	1.0	15.8	5.27	5.3
06	9.0	8.0	2.7	19.7	6.57	6.6
07	7.5	9.8	5.0	22.3	7.43	7.4
08	6.3	9.0	8.0	23.3	7.77	7.8
09	5.0	7.5	9.8	22.3	7.43	7.4
10	4.0	6.3	9.0	19.3	6.43	6.4
11	2.9	5.0	7.5	15.4	5.13	5.1
12	2.1	4.0	6.3	12.4	4.13	4.1
13	1.3	2.9	5.0	9.2	3.07	3.1
14	0.5	2.1	4.0	6.6	2.20	2.2
15	0.0	1.3	2.9	4.2	1.40	1.4
16		0.5	2.1	2.6	0.87	0.9
17		0.0	1.3	1.3	0.48	0.4
18			0.5	0.5	0.17	0.2
19			0.0	0.0	0.00	0.0
$\Sigma 0$	65.1				65.10	65.1

It should be clearly noted that $\Sigma 0$ of the 6-hours unit hydrograph so obtained should be the same as $\Sigma 0$ of the original 2-hours unit hydrograph, so that each of the unit hydrographs represent direct runoff 1 cm. Also, we note that while the peak discharge of the 2-hours unit hydrograph was 9.8 cumecs occurring at 05 hours, the peak discharge of the 6-hours unit hydrograph is 7.8 cumecs, occurring at 08 hours.

Example 4.31. Given the 6 hour unit hydrograph of Table 4.22 (column 2). Derive a 9 hour unit hydrograph for the same catchment.

Solution: The method has already been illustrated in § 4.24, using S-hydrograph. Column 2 of Table below gives the given 6 hours unit hydrograph while column (4)

gives the ordinates of S-curve derived from it. Column (5) gives the ordinates of the offset S-curve, shifted by $t_0 = 9$ hours. Column (6) gives the difference Δy between the ordinates of the two S-curves. The ordinates of the 9 hours unit hydrograph are given by

$$O = \Delta y \frac{T_0}{t_0} = \Delta y \frac{6}{9} = \frac{2}{3} \Delta y$$

Column (8) gives the ordinates of the moderated 9 hour unit hydrograph.

Time (hours)	Ordinates of 6 hour unit hydrograph (cumecs)	Offset ordinates	Ordinates of S-hydrograph (cumecs)	Ordinates of offset S-curve	Δy (cumecs)	$O = \Delta y \frac{T_0}{t_0} = \frac{2}{3} \Delta y$ (cumecs)	Ordinates of moderated 9 hour unit hydrograph
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
00	0	-	0	-	0	0.0	0.0
03	9	-	9	-	9	6.0	5.5
06	20	0	20	-	20	13.33	15.0
09	35	9	44	0	44	29.33	28.5
12	48	20	69	9	60	40.0	40.0
15	43	44	87	20	67	44.67	45.0
18	35	69	104	44	60	40.0	40.0
21	28	87	115	69	46	30.67	31.5
24	22	104	126	87	39	26.0	25.0
27	17	115	132	104	28	18.67	19.0
30	12	126	136	115	23	15.33	14.5
33	8	132	141	126	15	10.0	10.5
36	6	136	144	132	12	8.0	7.9
39	3	141	144	136	8	4.0	4.3
42	0	144	144	141	3	2.0	2.2
45			144	144	0	0.0	0.0
$\Sigma 0$	288					288.0	288.0

4.26. DISTRIBUTION GRAPH

Distribution graph was introduced by Bernard (1935), and hence it is also known as 'Bernard's distribution graph'. It represents the unit hydrograph in the form of percentage of total runoff occurring in successive periods of equal intervals of unit period. The unit period or unit interval is taken equal to the duration of rainfall excess. The ordinates of the distribution graph are indicated at successive such unit intervals. Since the discharge represented by a unit hydrograph is directly proportional to net rain, the percentage in unit time will remain constant whatever be the net rain. The percentage are drawn as rectangular bars or steps against successive unit times (Fig. 4.62).

To draw a distribution graph (Fig. 4.62b) corresponding to a given unit hydrograph (Fig. 4.62a), the base period of unit hydrograph is divided into a number of unit time periods, each equal to the unit duration (T_0) of unit hydrograph. The volume of runoff in each period of unit hydrograph is expressed as a percentage of the total volume of runoff represented by the entire unit hydrograph. Thus, in Fig. 4.62 (a) we observe that

$$\frac{\text{Shaded area of U.H.}}{\text{Total area of U.H.}} = 0.29 = 29\%$$

Hence the ordinate of distribution graph for the unit time period (T_0) will be 29%. The use of the distribution graph to generate a direct runoff hydrograph is exactly the same as that of a unit hydrograph. The area under the curve and under the step lines are the same, and hence in deriving unit hydrograph from distribution percentages, a smooth line must be drawn through steps to give equal areas. The distribution graph is therefore less precise than the unit hydrograph. However, it is much better suited for Collin's iterative process of deriving unit hydrograph from complex storms. The distribution graph is also useful in comparing runoff characteristics of different catchments.

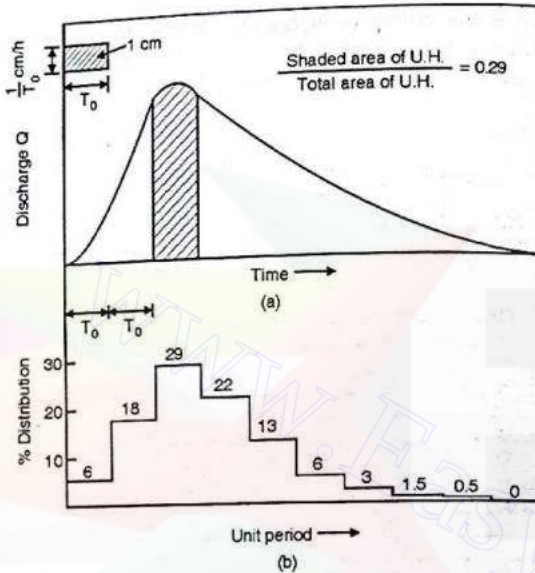
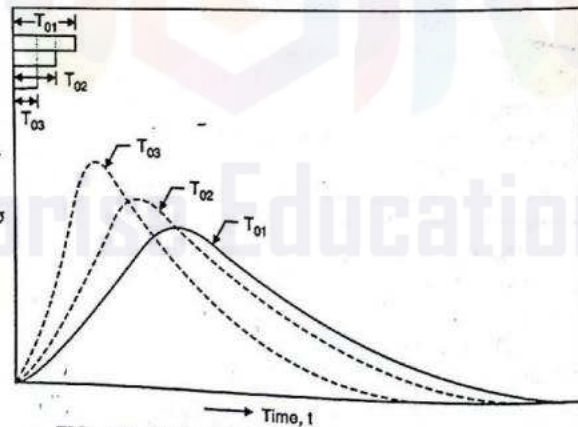


FIG. 4.62. DERIVATION OF DISTRIBUTION GRAPH

4.27. INSTANTANEOUS UNIT HYDROGRAPH

We have seen earlier that each unit hydrograph representing 1 cm of direct runoff, is for some unit duration T_0 . For a catchment, there can be a number of unit hydrographs corresponding to various values of unit duration T_0 . To obtain the runoff hydrograph resulting from a storm of varying duration and varying intensities, it is preferable to have a unit hydrograph of very short unit duration. Theoretically, the shortest unit duration in zero. If the duration of rainfall excess becomes infinitesimally small, the resulting unit hydrograph is called *instantaneous unit hydrograph*. Thus IUH is a fictitious, conceptual unit hydrograph which represents surface runoff from the catchment due to an

FIG. 4.63. UNIT HYDROGRAPHS OF DIFFERENT UNIT DURATIONS (T_0)

instantaneous precipitation of rainfall excess volume of 1 cm. The IUH is designated as $u(t, 0)$ or simply as $u(t)$.

IUH can be easily derived from S-curve hydrograph obtained from an available unit hydrograph of duration T_0 . If two S-curves are drawn at a time lag of t_0 , the ordinate of unit hydrograph of t_0 hour unit duration at any time t is given by

$$u(t, t_0) = \frac{T_0}{t_0} [S_t - S_{t-t_0}] \quad \dots(4.64)$$

where $u(t, t_0)$ = ordinate of unit hydrograph of unit duration t_0

T_0 = unit duration of unit hydrograph from which S-curve has been obtained

S_t = ordinate of S-curve at any time t

S_{t-t_0} = ordinate of shift S-curve, shifted by t_0

If t_0 is taken as Δt , the ordinates of resulting unit hydrograph of Δt unit duration

$$\text{is given by } u(t, \Delta t) = \frac{T_0}{\Delta t} [S_t^{T_0} - S_{t-\Delta t}^{T_0}] \quad \text{or } u(t, \Delta t) = T_0 \frac{\Delta S_t^{T_0}}{\Delta t} \quad \dots(4.65)$$

where $S_t^{T_0}$ is the S-curve ordinate derived from unit hydrograph of T_0 unit duration.

In the limit $\Delta t \rightarrow 0$, we get the IUH, given by

$$\text{Lt}_{\Delta t \rightarrow 0} u(t, \Delta t) = T_0 \frac{d S_t^{T_0}}{d t}$$

or

$$u(t) = T_0 \frac{d S_t^{T_0}}{d t} = \frac{1}{R_0} \frac{d S_t^{T_0}}{d t} \quad \dots(4.66)$$

Hence $\left\{ \begin{array}{l} \text{The ordinate of} \\ \text{IUH at any time } t \end{array} \right\} = T_0 \left\{ \begin{array}{l} \text{The slope of S-curve derived} \\ \text{from } T_0 \text{ hour unit hydrograph at time } t \end{array} \right\}$

In the above expression, R_0 is the intensity of rainfall excess, given by $R_0 = 1/T_0$. If $R_0 = 1$ cm, we get

$$u(t) = \frac{d S_t}{d t} \quad \dots(4.67)$$

where S_t is the ordinate of S-curve of intensity 1 cm/hour. Thus, the ordinate of IUH at any time t is the slope of S-curve of intensity 1 cm/hour.

An IUH, designated by $u(t)$ is a single peaked hydrograph with a finite base width. It has the following properties :

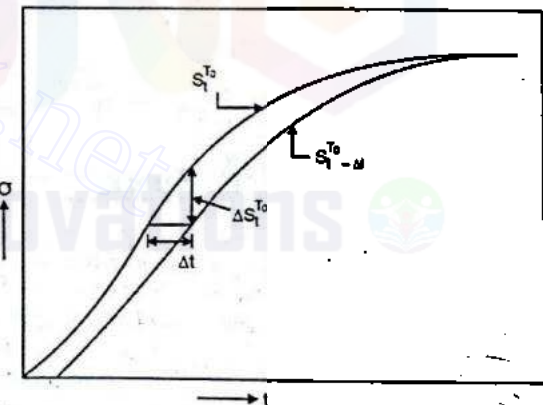


FIG. 4.64. THE INSTANTANEOUS UNIT HYDROGRAPH AS S-CURVE DERIVATIVE

1. $0 \leq u(t) < \infty$ a positive value at $t > 0$
2. $u(t) = 0$ at $t \leq 0$
3. $u(t) \rightarrow 0$ as $t \rightarrow \infty$
4. $\int_0^{\infty} u(t) dt = \text{unit depth over catchment}$

and 5. (Time to peak) < (time to centroid of curve)

It is interesting to note that IUH is a unique demonstration of a particular catchment's response to rain, independent of duration, just as unit hydrograph is its response to rain of a particular unit duration. IUH is not time dependant. It is a graphical expression of the integration of all the parameters of the catchment, such as length, shape, slope etc that control such a response.

The IUH can be developed either directly from the observed data or by adopting conceptual models.

When once IUH is available for a catchment, unit hydrographs of various unit durations can be easily derived. Let us derive unit hydrograph $u(t, T_0)$ of T_0 unit duration. We already know that

$$u(t, T_0) = S_t - S_{t-T_0}$$

$$\text{or } u(t, T_0) = \frac{1}{T_0} \left[\int_0^t u(z) dz - \int_0^{t-T_0} u(z) dz \right]$$

$$\text{or } u(t, T_0) = \frac{1}{T_0} \int_{t-T_0}^t u(z) dz \quad \dots(4.68)$$

Hence

$$\left[\text{Ordinate of U.H. of } T_0 \text{ at any time } t \right] = \frac{1}{T_0} \left[\text{area of IUH in the limits between } (t-T_0) \text{ and } t \right]$$

If IUH is assumed to be linear between $(t-T_0)$ and t , the Eq. 4.68 reduces to

$$u(t, T_0) = u\left(t - \frac{T_0}{2}\right) \quad \dots(4.69)$$

Thus, the ordinate of T_0 unit hydrograph at any time t is the average of IUH for $(t-T_0)$ hours and that of t hour.

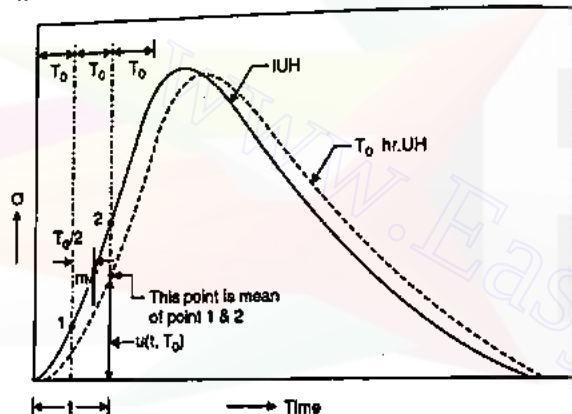


FIG. 4.65. DERIVATION OF T_0 HR. UNIT HYDROGRAPH FROM IUH

From Fig. 4.65, it is clear that if IUH is divided into T_0 hour time interval, and if the averages of the ordinates at the beginning and the end of each interval are plotted at the end of the interval, we get ordinate of T_0 unit hydrograph.

$$\text{Thus, } u(t, T_0) = \frac{1}{2} [u(t) + u(t - T_0)] \quad \dots(4.70)$$

4.28. SYNTHETIC UNIT HYDROGRAPH

We have seen that if the rainfall and runoff records are available, unit hydrographs can be derived. However, in the case of *ungauged rivers*, these data are not available. In some other cases, the data available may be scanty. For such catchments, unit hydrographs are derived by relating the selected basin characteristics to the unit hydrograph shape. The resulting hydrograph, derived from basin characteristic relationships is known as a *synthetic unit hydrograph*. There are several methods to derive synthetic unit hydrograph, but the one given by Snyder (1938), is widely used, and will therefore be discussed here.

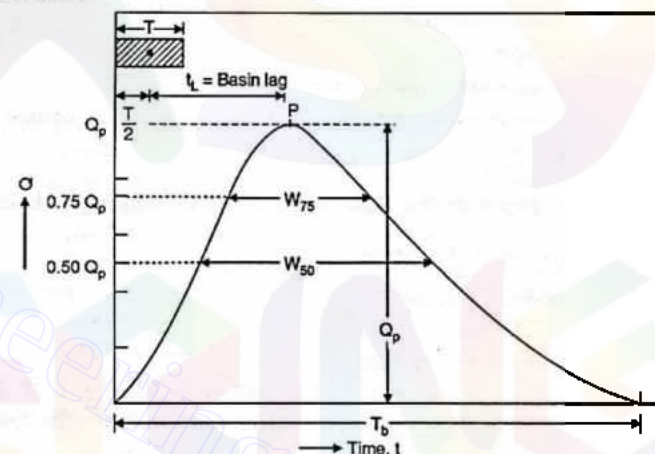


FIG. 4.66. PARAMETERS OF SYNTHETIC UNIT HYDROGRAPH

Snyder's Method

Snyder studied the data of catchments of Appalachian Highlands of Eastern United States, and then developed empirical equations for synthetic hydrograph.

Snyder selected three parameters of unit hydrograph : (i) base width T_b , (ii) peak discharge Q_p and (iii) basin lag t_L , as marked in Fig 4.66. The equations given by him take into account catchment area, shape of basin, topography, channel slopes, stream density and channel storage. He eliminated all these parameters except the first two, by including them in a single coefficient C_t . He dealt with the size and shape of the catchment by measuring the length of the main stream channel, by proposing the following equation for basin lag :

$$t_L = C_t (L_{ca} \cdot L)^{0.3} \quad \dots(4.71)$$

where t_L = basin lag in hours, (basin lag is the time between mass of centre of unit rainfall of T hour duration and runoff peak flow)

C_t = a coefficient depending upon units and drainage basin characteristics. The value of C_t varies between 1.35 and 1.65 for the Appalachian High-lands Catchments studied by Synder.

L = distance from station to catchment boundary measured along the main-stream, in km.

L_{ca} = distance along the mainstream from gauging station to a point opposite the centroid of the watershed, in km.

The peak discharge of Q_p unit hydrograph of standard duration T is given by

$$Q_p = \frac{2.778 C_p \cdot A}{t_L} \quad \dots(4.72)$$

where Q_p = peak discharge (m^3/sec)

A = Catchment area (km^2)

C_p = a regional constant, ranging from 0.56 to 0.69

Synder adopted the standard duration (T) hours of effective rainfall given by

$$T = \frac{2}{11} t_L \quad \dots(4.73)$$

The duration of surface runoff or the base length T_b of unit hydrograph is given by

$$T_b = 3 + 3 \left(\frac{t_L}{24} \right) \text{ (days)} \quad \dots(4.74a)$$

When T_b is expressed in hours,

$$T_b = 72 + 3 t_L \quad \dots(4.74)$$

(where both T_b and t_L are in hours)

It is found that Eq. 4.74 gives unreasonably long base periods for small catchments. Some investigators recommend that a base period equal to five times the time to peak should be taken :

$$T_b = 5 \left(\frac{T}{2} + t_L \right) = 5 \left(\frac{t_L}{11} + t_L \right)$$

$$\text{or } T_b = 5.455 t_L \quad \dots(4.75)$$

Alternatively, assuming a triangular shape of unit hydrograph,

$$T_b = \frac{5.556}{Q_p} \quad \dots(4.75a)$$

Sketching of unit hydrographs becomes easier by adopting the following recommendations given for U.S. catchments, by US Army Corps of Engineers, for widths of unit hydrographs at 50 and 75% of the peak discharge :

$$W_{50} = \frac{5.87}{Q_p} \quad \dots[4.76(a)]$$

$$\text{and } W_{75} = \frac{W_{50}}{1.75} \quad \dots[4.76(b)]$$

where W_{50} = width of unit hydrograph in hours, at 50% peak discharge

W_{75} = width of unit hydrograph, in hours, at 75% peak discharge

$$q_p = \frac{Q_p}{A} = \text{peak discharge per unit area.}$$

Usually, one third of this width is kept before the unit hydrograph peak and two thirds after the peak.

If synthetic unit hydrograph of any other duration T' is required, then the modified basin lag is

$$T'_L = T_L + \frac{T' - T}{4} \quad \dots[4.77(a)]$$

$$\text{Also, } Q'_p = \frac{2.778 A C_p}{t'_L} \quad \dots[4.77(b)]$$

4.29. PEAK FLOW DETERMINATION

The extreme hydrologic events are *floods* and *droughts*. In general qualitative terms, these refer to periods of unusually high and low water supplies. A flood is an unusually high stage of a river due to runoff from rainfall and/or melting of snow in quantities too great to be confined in the normal water surface elevations of the river or stream. In the peak flow determination, reference is made to the following three types of floods :

(i) **Standard Project Flood (SPF)** : It is the flood that would result from the most severe combination of meteorological and hydrological factors that are reasonably applicable to the basin. However, extreme rare combinations are excluded.

(ii) **Maximum Probable Flood (MPF)** : This includes the extreme rare combination also, excluded in SPF. Thus, it is defined as extreme flood that is physically possible in a region as a result of severest combination including extreme rare combination of meteorological and hydrological factors. It is observed that SPF is about 80% of MPF.

(iii) **Design Flood** : A design flood is the flood discharge adopted for design of a hydraulic structure after careful considerations of hydrologic and economic factors. A design flood in most of the cases may be less than MPF. There may be several design flood values adopted for various components of a project. For example, design flood used for spillway will be more than the design flood used for cofferdam of the same project.

The peak flood discharge in a stream may be determined by the following methods.

1. By physical indication of past floods
2. By empirical flood formulae
3. By unit hydrograph
4. By flood frequency analysis
5. By enveloping curves
6. By rational formula

PHYSICAL INDICATION OF PAST FLOODS

Ancient monuments, etc., situated on river banks always bear past floods marks. Old persons in the villages situated on the bank of the river may be contacted to know the maximum water level attained in the past 35 years. The cross-section of the river may be plotted and the water line corresponding to the highest flood can be drawn on it. From such a cross-section, the water-flow-area, wetted perimeter and hydraulic mean depth can be calculated. By longitudinal sectioning with the help of levelling slightly to the upstream and downstream of the site where cross-section has been plotted, the longitudinal slope of the bed of the river can be determined. Assuming this to be the same as the hydraulic slope during the past flood, the mean velocity of flow can be computed by Chezy's or any other suitable hydraulic formula. This velocity can be multiplied with the probable area of water section at the time of past flood to calculate the flood discharge. This procedure should be repeated at several villages or water marks, to get consistent results.

4.30. EMPIRICAL FORMULAE FOR FLOOD DISCHARGE

Some of the empirical formulae for estimating the flood discharge are given below. Most of these are in the form :

$$Q = CA^n$$

where : Q = flood discharge
 A = catchment area
 n = flood index
 C = flood coefficient.

Both C and n depend upon various factors, such as (i) size, shape and location of catchment, (ii) topography of the catchment, and (iii) intensity and duration of rainfall, and distribution pattern of the storm over the basin.

1. Dicken's Formula

$$Q = CA^{3/4} \quad \dots(4.78)$$

where Q = discharge in cumecs
 A = area of basin in sq.km

The constants C depend upon the catchment and may be obtained from Table 4.23.

2. Ryve's Formula

For Madras catchments

$$Q = CA^{2/3} \quad \dots(4.79)$$

(where Q is in cumecs and A in sq. km)

Values of C may be obtained from Table 4.24.

3. Inglis formula

Inglis formula is applicable for catchments of former Bombay Presidency.

$$Q = \frac{123A}{\sqrt{A+10.4}} \approx 123A^{1/2} \quad \dots(4.80)$$

TABLE 4.23

Region	C
Northern India	11.4
Central India	13.9 — 19.5
Western India	22.2 — 25

TABLE 4.24

Location of the catchment	C
1. Areas within 24 km from the coast.	6.75
2. Areas within 24 km to 161 km from the coast.	8.45
3. Limited areas near hills.	10.1

4. Nawab Jang Bahadur Formula

For catchments of Old Hyderabad state :

$$Q = CA^{(0.993 - \frac{1}{14} \log A)} \quad \dots(4.81)$$

where C varies from 48 to 60.

5. Fanning's Formula

For American catchments :

$$Q = CA^{5/8} \quad \dots(4.82)$$

where average value of C may be taken equal to 2.54.

6. Creager's Formula

Applicable for American catchments : Expressed in F.P.S. units,

$$Q_1 = 46 C_1 A_1^{(0.8904 A_1^{-0.048})} \quad \dots(4.83)$$

The constant varies from 30 to 100.

7. Fuller's Formula

The formula takes into account the flood frequency also; expressed in metric units,

$$Q_{max} = CA^{0.8} (1 + 0.8 \log T) (1 + 2.67 A^{-0.3}) \quad \dots(4.84)$$

where T = Number of years after which such a flood is to recur.

Q = Maximum flood (in cumecs) during any part of the day that could occur in T -years.

A = Area of drainage basin, in sq. km

C = Constant, varying from 0.185 to 1.3.

4.31. FLOOD FREQUENCY STUDIES

Since the exact sequence of stream flow for future years cannot be predicted, probability concepts must be used to study the probable variations in flow so that the design can be completed on the basis of a calculated risk.

Flood Frequency. Flood frequency denotes the likelihood of flood being equalled or exceeded. A 10% frequency means that the flood has 10 out of 100 chances of being equalled or exceeded.

Recurrence Interval. Recurrence interval denotes the number of years in which a flood can be expected once. It is the period of time between the equalling or exceeding of a specific flood. This is usually denoted by a symbol T .

Return period : It is the average recurrence interval for a certain event or flood.

Probability of occurrence (P) : The probability of an event being equalled or exceeded in any one year is the probability of its occurrence.

The probability (P) of occurrence of a flood having a recurrence interval of T years in any year or the probability of exceedance is

$$P = \frac{1}{T}$$

The probability that it will not occur in a year, is known as *probability of non-exceedance* (q) and is given by
 $q = 1 - P$

Frequency (f) : The probability of occurrence of an event expressed as a percent is known as frequency (f). Thus $f = 100 P = \frac{100}{T}$

Frequency studies for flood

Frequency studies interpret a past record of events to predict the future probabilities of occurrence. It is based on the assumption that combination of the numerous factors which produce floods are a matter of pure chance and therefore are subject to analysis according to mathematical theory of probability. There are two methods of compiling flood peak data :

- (i) Annual duration series (ii) Partial duration series

In the *annual series*, the largest flood observed in each water year only is taken. It ignores the second and lower order events of each year which may sometimes exceed many of the annual maximum. In the *partial duration series* all flood events above a selected base value are included. The base is usually so chosen that not more than three or four events are included for each year. If the extreme flood are of primary concern, wherein the flood magnitude with exceedance probability of 0.5 or less are estimated, the annual series is used. When estimates of very frequent events with return periods of less than 5 years are required, (such as in design of coffer dams, urban drainage etc.) the partial series is preferable to annual series. However, for spillway design flood, the annual series is preferable since the flood should not be exceeded in the dams life time, say 100 years.

Annual flood series

Annual flood series consist of the values of annual maximum flood from a given catchment area, for large number of successive years. The data of the series are arranged in the decreasing order of magnitude. The probability (P) of each event being equalled to or exceeded (known as *plotting position*) is computed from one of the following *plotting position formulae*:

1. *California method* (1923) :

$$P = \frac{m}{N} ; T = \frac{N}{m}$$

2. *Allen Hazen method* (1939) :

$$P = \frac{2m-1}{2N} ; T = \frac{2N}{2m-1}$$

3. *Weibull method* (1939) :

$$P = \frac{m}{N+1} \text{ or } T = \frac{N+1}{m}$$

4. Gumbel's Method

$$P = \frac{m+C-1}{N} \text{ or } T = \frac{N}{m+C-1}$$

where C is known as Gumbel's correction. The correction depends upon $\frac{m}{N}$ ratio and can be found from Table 4.25.

TABLE 4.25. GUMBEL'S CORRECTION

$\frac{m}{N}$	1	0.9	0.8	0.7	0.6	0.5	0.4	0.3	0.2	0.1	0.08	0.04
C	1	0.96	0.88	0.845	0.78	0.73	0.66	0.59	0.52	0.4	0.38	0.28

The exceedance probability of the event (such as flood discharge etc.) obtained by the use of an empirical formula (such as Weibull formula) is called the *plotting position*. Weibull equation given above is the most popular *plotting position formula*.

Binomial Distribution

The *binomial distribution* given below can be used to find the probability of occurrence of the event r times in n successive years.

$$P_{r,n} = {}^nC_r P^r q^{n-r} = \frac{n!}{(n-r)! r!} P^r q^{n-r} \quad \dots(4.85)$$

where $P_{r,n}$ = probability of flood of given magnitude and exceedance probability P , occurring r times in n successive years,

and $q = 1 - P$

Thus, the probability of flood of given magnitude occurring 3 times in n successive years is

$$P_{3,n} = \frac{n!}{(n-3)! 3!} P^3 q^{n-3}$$

Also,

$$P_{2,n} = \frac{n!}{(n-2)! 2!} P^2 q^{n-2}$$

and

$$P_{1,n} = \frac{n!}{(n-1)! 1!} P^1 q^{n-1} = n P q^{n-1}$$

The probability of the flood of given magnitude not occurring at all (i.e. $r=0$) in n successive years is

$$P_{0,n} = q^n = (1-P)^n$$

The probability of the flood of given magnitude occurring at least once in n successive years is called risk (R_A) and is given by

$$R_{A,n} = 1 - q^n = 1 - (1-P)^n$$

Probability Plotting

The annual flood peak can be plotted against the return period either on (i) ordinary probability plot, or on (ii) logarithmic probability plot.

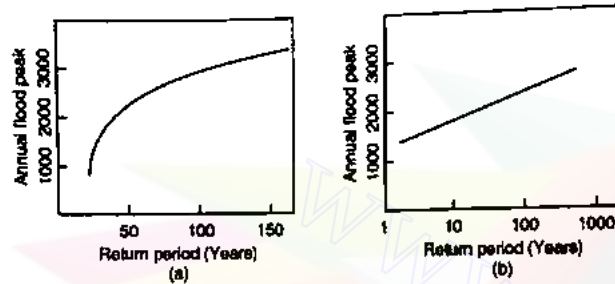


FIG. 4.67. PROBABILITY PLOTS

In the ordinary probability plot (Fig. 4.67 a) both the annual peak flood as well as return periods are plotted on an algebraic scale. In the logarithmic probability plot, (Fig. 4.67 b) annual flood peak is plotted on arithmetic scale while return period is plotted on log scale, giving rise to a straight line plotting.

Frequency distribution functions

According to Chow (1951), most frequency distribution functions can be expressed by the following general equation :

$$x_T = \bar{x} + K \sigma_x \quad \dots(4.86)$$

where x_T = Value of the variate X (such as flood discharge, max. rainfall depth etc.) of a random hydrologic series with return period T

$\bar{x}$ = mean of the variate

σ_x = standard deviation of the variate

K = Frequency factor depending upon the return period (T) and the assumed frequency distribution.

Following are some of the commonly used frequency distribution functions :

- Gumbel's extreme value distribution
- Log-Pearson Type III distribution, and
- Log-normal distribution.

We shall, however, discuss only the first type of distribution in this Chapter.

Gumbel's extreme value distribution

This method is useful for obtaining values of flood discharges for a high recurrence interval. According to Gumbel (1941), the probability of occurrence of an event equal to or larger than a value x_0 is given by

$$P(X \geq x_0) = 1 - e^{-e^{-x}} \quad \dots(4.87)$$

In the above equation, y is a dimensionless variable, given by the expression

$$y = \alpha(x - \bar{x})$$

where $\alpha = 1.2825/\sigma_x$ and $a = \bar{x} - 0.45006 \sigma_x$

$$\text{Hence } y = \frac{1.2825(x - \bar{x})}{\sigma_x} + 0.577 \quad \dots(4.88)$$

Transposing Eq. 4.87, we get

$$y_T = -\ln[-\ln(1 - P)] \quad \dots(4.87 a)$$

If y_T = value of y for a given T = reduced variate and noting that $T = 1/P$,

$$\text{we have } y_T = -\left[\ln \cdot \ln \frac{T}{T-1} \right] \quad \dots(4.89)$$

$$\text{or } y_T = -\left[0.834 + 2.303 \log \log \frac{T}{T-1} \right] \quad \dots(4.89 a)$$

Again, re-arranging Eq. 4.88, the value x_T for the variate X with a return period T is given by

$$x_T = \bar{x} + K \sigma_x \quad \dots(4.90)$$

$$\text{where } K = \frac{y_T - 0.577}{1.2825} \quad \dots(4.90 a)$$

Eqs. 4.90 are the basic Gumbel's equations which are applicable to an infinite series (i.e. where $N \rightarrow \infty$). In actual practice, N is finite and hence Eq. 4.90 is modified as under :

$$x_T = \bar{x} + K_T \sigma_{n-1} \quad \dots(4.91)$$

where σ_{n-1} = standard deviation of the sample of size $N = \sqrt{\frac{\sum (x - \bar{x})^2}{N-1}}$

$$K_T = \text{Modified frequency factor} = \frac{y_T - \bar{y}_n}{S_n} \quad \dots(4.91 a)$$

y_T = reduced variate given by Eq. 4.89.

$\bar{y}_n$ = reduced mean, a function of sample size N , the values of which are given in Table 4.21 (a). Note that when $N \rightarrow \infty$, $\bar{y}_n \rightarrow 0.577$

S_n = reduced standard deviation, a function of sample size N , the values of which are given in Table 4.21 (b). Note that when $N \rightarrow \infty$, $S_n \rightarrow 1.2825$

Procedure

Step 1 : For the given annual flood discharge data of size N , find mean ($\bar{x}$) and standard deviation (σ_{n-1})

Step 2 : From Table 4.21 (a) find reduced mean $\bar{y}_n$ for the given value of N . For example if $N=33$, we get $\bar{y}_n = 0.5388$ from Table 4.21 (a).

Step 3 : From Table 4.21 (b), find reduced standard deviation S_n for the given value of N . For example, if $N=33$, we get $S_n = 1.1226$.

TABLE 4.26 (a) VALUES OF GUMBEL'S REDUCED MEAN $\bar{y}_n$ FOR SAMPLE SIZE N

$N \rightarrow$	0	1	2	3	4	5	6	7	8	9
10	0.4952	0.4996	0.5036	0.5070	0.5100	0.5128	0.5167	0.5181	0.5202	0.5220
20	0.5236	0.5262	0.5288	0.5283	0.5296	0.5309	0.5320	0.5332	0.5343	0.5353
30	0.5362	0.5371	0.5380	0.5388	0.5396	0.5402	0.5410	0.5418	0.5424	0.5430
40	0.5436	0.5442	0.5448	0.5453	0.5458	0.5463	0.5468	0.5473	0.5477	0.5481
50	0.5485	0.5489	0.5493	0.5497	0.5501	0.5504	0.5508	0.5511	0.5515	0.5518
60	0.5521	0.5524	0.5527	0.5530	0.5533	0.5535	0.5538	0.5540	0.5543	0.5546
70	0.5548	0.5550	0.5552	0.5555	0.5557	0.5559	0.5561	0.5563	0.5565	0.5567
80	0.5569	0.5570	0.5572	0.5574	0.5575	0.5578	0.5580	0.5581	0.5583	0.5585
90	0.5586	0.5587	0.5589	0.5591	0.5592	0.5593	0.5595	0.5596	0.5598	0.5599
100	0.5600									
∞	0.577									

TABLE 4.26 (b) VALUES OF GUMBEL'S REDUCED STANDARD DEVIATION S_n FOR SAMPLE SIZE N

N	0	1	2	3	4	5	6	7	8	9
10	0.9496	0.9676	0.9833	0.9971	1.0096	1.0206	1.0316	1.0411	1.0493	1.0565
20	1.0628	1.0696	1.0764	1.0811	1.0864	1.0915	1.0961	1.1004	1.1047	1.1086
30	1.1124	1.1169	1.1193	1.1226	1.1255	1.1286	1.1313	1.1339	1.1363	1.1388
40	1.1413	1.1436	1.1458	1.1480	1.1499	1.1519	1.1538	1.1557	1.1574	1.1590
50	1.1607	1.1623	1.1638	1.1653	1.1667	1.1681	1.1696	1.1708	1.1721	1.1734
60	1.1747	1.1769	1.1770	1.1782	1.1793	1.1803	1.1814	1.1824	1.1834	1.1844
70	1.1854	1.1863	1.1873	1.1881	1.1890	1.1898	1.1906	1.1915	1.1923	1.1930
80	1.1938	1.1945	1.1953	1.1959	1.1967	1.1973	1.1980	1.1987	1.1994	1.2001
90	1.2007	1.2013	1.2020	1.2026	1.2032	1.2038	1.2044	1.2049	1.2055	1.2061
100	1.2065									
∞	1.2825									

Step 4 : For the given value of recurrence interval T , determine y_T from Eq. 4.89.

Step 5 : Knowing y_T , $\bar{y}_n$ and S_n , calculate the value of K_T from Eq. 4.91 (a).

Step 6 : Lastly, compute the value of x_T (i.e. the desired flood discharge at recurrence interval T) from Eq. 4.91 in which $\bar{x}$, K_T and σ_{n-1} are known.

See example 4.34 for illustration.

Note 1 : Gumbel's distribution has the property which gives $T=2.33$ years for the average of the annual series, when N is very large. Hence the value of the flood with $T=2.33$ years is called *mean annual flood*.

Note 2 : In the above mentioned procedure, a series of steps are required to compute the value of K_T for given sample size (N) and recurrence interval or frequency (T). However, Table 4.27 gives the computed values of K_T for given set of values of N and T .

TABLE 4.27. VALUES OF FREQUENCY FACTOR K_T FOR GUMBEL'S METHOD

Sample size (N) in years	Frequency (T) in years									
	5	10	15	20	25	30	40	50	75	100
15	0.967	1.703	2.117	2.410	2.632	2.823	3.321	3.501	3.721	4.005
20	0.919	1.626	2.023	2.302	2.517	2.690	3.179	3.352	3.563	3.836
25	0.888	1.575	1.963	2.235	2.444	2.614	3.068	3.257	3.463	3.729
30	0.866	1.541	1.922	2.188	2.393	2.560	3.018	3.191	3.393	3.653
35	0.851	1.516	1.891	2.152	2.354	2.520	2.979	3.142	3.341	3.598
40	0.838	1.495	1.866	2.128	2.326	2.489	2.943	3.104	3.301	3.554
45	0.829	1.478	1.847	2.104	2.303	2.464	2.913	3.078	3.268	3.520
50	0.820	1.466	1.831	2.086	2.283	2.443	2.889	3.048	3.241	3.491
55	0.813	1.456	1.818	2.071	2.267	2.426	2.869	3.027	3.219	3.467
60	0.807	1.446	1.806	2.059	2.253	2.411	2.852	3.008	3.200	3.446
65	0.801	1.437	1.796	2.048	2.243	2.398	2.837	2.992	3.183	3.429
70	0.797	1.430	1.788	2.038	2.230	2.387	2.824	2.979	3.169	3.413
75	0.792	1.423	1.780	2.029	2.220	2.377	2.812	2.967	3.155	3.400
80	0.788	1.417	1.773	2.020	2.212	2.368	2.802	2.956	3.145	3.387
85	0.785	1.413	1.767	2.013	2.205	2.361	2.793	2.946	3.135	3.376
90	0.782	1.409	1.762	2.007	2.196	2.353	2.785	2.938	3.125	3.367
95	0.780	1.405	1.757	2.002	2.193	2.347	2.777	2.930	3.116	3.357
100	0.779	1.401	1.752	1.993	2.187	2.341	2.770	2.922	3.109	3.349

Plot of x_T versus T

In order to obtain the variation of x_T with T , the values of x_T are computed for some return periods T ($< N$) using Gumbel's formula. These pairs of values of x_T and T are plotted on semi-log, log-log or Gumbel's probability paper. If these are plotted on Gumbel's probability paper, it will give rise to a straight line. This straight line will pass through the *mandatory point* corresponding to the pair of values of $T=2.33$ years and $x_T=\bar{x}$ (Fig. 4.68).

Construction of Extreme value probability paper (Gumbel's probability paper)

The above mentioned probability paper, which plots Gumbel's extreme value distribution as a straight line can be easily constructed as follows :

Step 1 : Construct the arithmetic scale of y_T values, say from -2 to $+6$ on the abscissa

Step 2 : Compute the value of y_T for some selected values of T (say, 2, 5, 10, 50, 100, 500 etc.), using Eqs. 4.89 (a) or 4.89 as under :

$$y_T = - \left[\ln \ln \frac{T}{T-1} \right] = - \left[0.834 + 2.303 \log \log \frac{T}{T-1} \right]$$

Step 3 : Mark off these positions on the abscissa, thus giving rise to the T -scale or probability scale for recurrence interval T

It is to be noted that the variate x_T (i.e. flood discharge for the present case) is generally plotted on arithmetic scale on the Gumbel's paper.

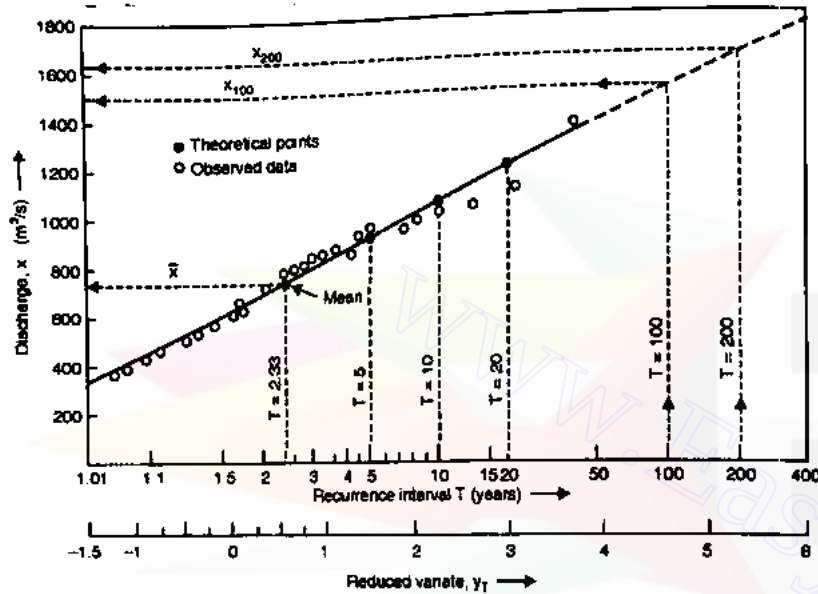


FIG. 4.68. FLOOD PROBABILITY PLOT ON GUMBEL'S PAPER

Test for observed flood data

The Gumbel's straight line mentioned above may be obtained by plotting two or three pairs of values of x_T and T , though the line will pass through the mandatory point $T = 2.33$ years and $x_T = \bar{x}$. In order to confirm whether the observed data follows Gumbel's distribution or not, the following procedure is followed.

Step 1 : Arrange the observed data (about flood) in descending order of magnitude and assign each the order m , the total number of observations being N .

Step 2 : Find the return periods (plotting positions) for each discharge, using Weibull formula : $T = (N + 1)/m$

Step 3 : Plot these pairs of values of discharge and the corresponding return periods, on the Gumbel's paper.

Step 4 : If a good fit of the observed data with the theoretical variation line is seen, it will be concluded that Gumbel's distribution is applicable to the given series.

The values of x_T (i.e. flood discharge) for T greater than N can be determined by extrapolating Gumbel's straight line (x_T vs T).

Selection of design return period : Risk and Reliability

In the terminology of probability theory, $1/T$ indicates the *probability* with which T year flood may be equalled or exceeded in any one year. Hence if it is desired to select a design flood which is not likely to occur during the life period of the structure, it becomes necessary to use a return period greater than the estimated useful life.

Let us take the case of a weir which is designed for T years flood and its useful life is n years. The probability that the design flood is equalled or exceeded, and hence the probability that the weir may fail in any year is $1/T$. In other words, the probability that the structure does not fail in any year is $\left(1 - \frac{1}{T}\right)$. Assuming that the annual flood peaks are independent events, the probability that the weir does not fail in the next n years is $\left(1 - \frac{1}{T}\right)^n$.

Hence the risk R_{sk} in the design, which is the probability that the weir may fail in any one of the next n years, is given by

$$R_{sk} = 1 - \left(1 - \frac{1}{T}\right)^n = 1 - (1 - P)^n \quad \dots(4.92)$$

The *reliability* R_{el} is defined as

$$R_{el} = 1 - R_{sk} = \left(1 - \frac{1}{T}\right)^n = (1 - P)^n \quad \dots(4.92 a)$$

The above equations can also be used to determine the required return period of the design flood for a given risk and given life period n . For example, if the weir with life period of 50 years is designed for a 50 years flood, the risk of failure is

$$R_{sk} = 1 - \left(1 - \frac{1}{50}\right)^{50} = 0.636 \text{ or } 63.6\%$$

If the weir is designed for a 100 year flood,

$$R_{sk} = 1 - \left(1 - \frac{1}{100}\right)^{50} = 0.605 \text{ or } 60.5\%$$

However, if we want to reduce the risk to 0.10 (or 10%) we have

$$0.10 = 1 - \left(1 - \frac{1}{T}\right)^{50} \text{ which gives } T = 475 \text{ years.}$$

4.32. FLOOD DISCHARGE BY RATIONAL FORMULA

Amongst various types of empirical relations, *rational formula* is the most rational method of calculating peak discharge for *small catchments*. In this method, it is assumed that the maximum flood flow is produced by a certain rainfall intensity which lasts for a time equal to or greater than the period of *concentration time* (t_c). The maximum rate of runoff from the watershed appears when the entire area contributes at the basin outlet. The runoff gradually increases from zero to peak when the rainfall duration reaches the time of concentration (t_c). If the rainfall continues beyond t_c , the runoff will be constant and at the peak value. The peak value of runoff is given by

$$Q_p = F_p \cdot C i A \quad \dots(4.93)$$

where C = runoff coefficient representing a ratio of runoff to rainfall

A = Catchment area.

i = rainfall intensity

F_p = a factor which permits the expression of terms Q_p , A and i in consistent units

Let Q_p = discharge in cubic metres per second (cumecs)
 A = catchment area in km^2

i = intensity of rainfall in cm/hour

In order to find corresponding value of factor F_u , we have

$$Q_p = (10^6 A) (C) \left(\frac{i}{100 \times 3600} \right) = \frac{C i A}{0.36} = 2.778 C i A \quad \dots(4.94)$$

Hence factor $F_u = 2.778$

If, however, i is expressed in mm/hour, we have

$$Q_p = (10^6 A) (C) \left(\frac{i}{1000 \times 3600} \right) = \frac{C i A}{3.6} = 0.2778 C i A \quad \dots(4.95)$$

Runoff coefficient C

Runoff coefficient is a highly critical element that serves the purpose of converting the average rainfall rate of a particular recurrence interval to the peak runoff intensity of the same frequency. Its magnitude depends upon the following factors (i) antecedent moisture conditions (ii) ground slope (iii) ground cover (iv) depression storage (v) soil moisture, (vi) shape of drainage area (vii) overland flow velocity, (viii) intensity of rainfall (ix) geology of catchment, etc. In spite of this, its value is generally considered fixed for any drainage area, depending only on the surface type. This simplistic approach is a major cause of criticism for the rational method. Some typical values of C are indicated in Table 4.28.

TABLE 4.28. TYPICAL VALUES OF RUNOFF COEFFICIENT C

Type of area	Value of C	Type of area	Value of C
(a) Urban area			
(a) Lawns :			
(i) Sandy soil, flat 2%	0.05 - 0.10	(d) Industrial area	
(ii) Sandy-soil, average 2.7%	0.1 - 0.15	(i) Unimproved	0.1 - 0.3
(iii) Sandy-soil, steep < 7%	0.15 - 0.2	(ii) Light areas	0.5 - 0.8
(iv) Heavy soil, flat	0.13 - 0.18	(iii) Heavy areas	0.6 - 0.9
(v) Heavy soil, average	0.18 - 0.22	(iv) Rail yards	0.2 - 0.4
(vi) Heavy soil, steep	0.25 - 0.35	(e) Streets	0.7 - 0.95
(b) Residential area			
(B) Agricultural area			
(a) Flat			
(i) Single House area	0.3 - 0.5	(i) Light clay, cultivated	0.50
(ii) Multi-units attached	0.6 - 0.75	(ii) Light clay, wood land	0.40
(iii) Suburban	0.25 - 0.4	(iii) Sandy loam : cultivated	0.7
(c) Business area			
	0.5 - 0.95	(iv) Sand loam : wood land	0.1
(b) Hilly			
		(i) Light clay ; cultivated	0.7
		(ii) Light clay ; wood land	0.6
		(iii) Sandy loam ; Cultivated	0.4
		(iv) Sandy loam ; wood land	0.3

If a watershed of total area A is non-homogeneous, having component sub-areas having different values of C , a weighted runoff coefficient (C_w) is computed from the following equation :

$$C_w = \frac{C_1 A_1 + C_2 A_2 + \dots + C_n A_n}{A_1 + A_2 + \dots + A_n} = \frac{\sum_{j=1}^n C_j A_j}{A} \quad \dots(4.96)$$

where

$A_1, A_2, A_3, \dots, A_n$ = areas of sub-zones

$C_1, C_2, C_3, \dots, C_n$ = runoff coefficients for the corresponding sub-zones.

Rainfall intensity (i)

The rainfall intensity (i) corresponding to a duration t , and the desired probability of exceedance P (or return period T) is given by Eq. 4.19 indicating the relationship between intensity, duration and return period.

$$i = \frac{K T^x}{(t + b)^n} \quad \dots(4.19)$$

where i = intensity of rainfall (cm/h).

Typical values of constants K , x , b and n are given in Table 4.8.

In the above equation, t is the time of concentration (in hours) which can be found from the following formula by Kirpich (1940).

$$t = t_c = 0.000323 L^{0.77} S^{-0.385} \quad \dots(4.58)$$

4.33. METHOD OF ENVELOPING CURVES

In this method, areas having similar topographical features and climatic conditions are grouped together, the available data regarding discharges are compiled along with

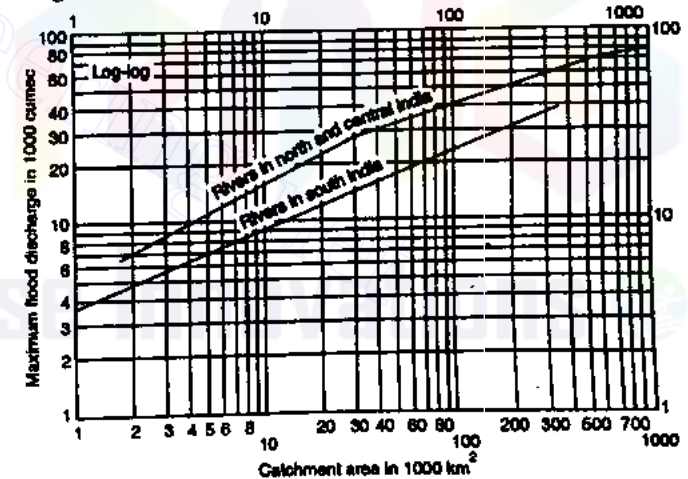


FIG. 4.69. ENVELOPING CURVES FOR INDIAN RIVERS

their catchment areas, the peak flood discharges are plotted against the drainage areas, their catchment areas, the peak flood discharges are plotted against the drainage areas, Kanwar and a curve is then drawn to cover or envelope the highest plotted points. Kanwar and Sain and Karpov (1967) have presented two enveloping curves, one for Northern and Central Indian rivers and the other for the Southern Indian rivers, as shown in Fig. 4.69.

McIlurath (1951) gave the following equation for the enveloping curve of maximum floods (Q_{mp}), based on maximum recorded flood throughout the World :

$$Q_{mp} = \frac{3025 A}{(278 + A)^{0.78}} \quad \dots(4.97)$$

where Q_{mp} is in m^3/s and A is in km^2 .

Limitations of Rational Formula

1. The formula gives good results only for small catchments, having area upto $50 km^2$.
2. It is applicable only if the duration of rainfall is equal to or more than time of concentration (t_c).
3. Rainfall intensity (i) should be constant over the entire catchment, during the time of concentration.
4. It assumes constant value of C for a given area, for all storms, which is not reasonable.
5. If a plot is made between Q_p and i , a straight line is obtained with zero intercept. Nature does not follow such a linear relationship.

Due to the above limitations, the rational formula is generally used to estimate the peak flood (Q_p) in the design of urban drainage system, storm sewers, design of small culverts, and bridges etc.

Example 4.32. The following Table gives flood data for 16 years recorded at the Bhakra Dam site on Sutlej river :

Year	Discharge (cumecs)	year	Discharge (cumecs)
1937	3110	1944	2290
1938	5800	1945	2380
1939	3690	1946	3810
1940	1723	1947	7800
1941	3630	1948	4525
1942	6800	1949	3254
1943	5260	1950	4980
		1951	9200

Find out the recurrence interval for the flood of various magnitudes by the following methods : (i) California method, (ii) Hazen's method, and (iii) Gumbel's method.

Solution : The recurrence interval has been calculated by the three methods in the tabular form below. The value of C for Gumbel's method has been taken from Table 4.20. Here $N=15$.

Order No. (n)	Peak yearly discharges arranged in decreasing order (cumecs)	Recurrence Interval		
		California Method $T = \frac{N}{m}$	Hazen's method $T = \frac{2N}{m-1}$	Gumbel's method $T = \frac{N}{m+C-1}$
1	9200	15.0	30	50
2	7800	7.50	10	10.41
3	6800	5.00	6	5.95
4	5800	3.75	4.23	4.21
5	5260	3.00	3.23	3.25
6	4980	2.50	2.73	2.65
7	4525	2.14	2.39	2.24
8	3810	1.87	2.09	1.94
9	3630	1.66	1.82	1.71
10	3250	1.50	1.56	1.53
11	3110	1.36	1.42	1.39
12	3090	1.25	1.30	1.26
13	2380	1.15	1.20	1.17
14	2290	1.07	1.11	1.08
15	1723	1.00	1.03	1.00

Example 4.33. For a river valley project, the following results were obtained from flood frequency analysis using Gumbel's method :

Return period T (years)	Peak flood (m^3/s)
40	27000
80	31000

Estimate the flood magnitude with a return period of 240 years.

Solution

From Gumbel's equation (Eq. 4.91)

$$x_T = \bar{x} + K_T \sigma_{n-1}$$

Hence

$$x_{80} = \bar{x} + K_{80} \sigma_{n-1} \text{ and } x_{40} = \bar{x} + K_{40} \sigma_{n-1}$$

$$\therefore (K_{80} - K_{40}) \sigma_{n-1} = x_{80} - x_{40} = 31000 - 27000 = 4000 \quad \dots(1)$$

$$\text{But } K_T = \frac{y_T - \bar{y}_n}{S_n} = \frac{y_T - \bar{y}_n}{S_n}, \text{ where } \bar{y}_n \text{ and } S_n \text{ are constants for given series}$$

$$\therefore (y_{80} - y_{40}) \frac{\sigma_{n-1}}{S_n} = 4000 \quad \dots(2)$$

$$\text{From Eq. 4.89, } y_T = - \left[\ln \cdot \ln \frac{T}{T-1} \right]$$

$$\therefore y_{80} = - [\ln \cdot \ln (80/79)] = 4.37574$$

and

$$y_{40} = - [\ln \cdot \ln (40/39)] = 3.67625$$

$$\text{Hence from (2), } \frac{\sigma_{n-1}}{S_n} = \frac{4000}{4.37574 - 3.67625} = 5718.5 \quad \dots(3)$$

$$\text{For } T = 240 \text{ years, } y_{240} = - [\ln \cdot \ln (240/239)] = 5.47855$$

Now $(y_{240} - y_{80}) \frac{\sigma_n - 1}{S_n} = x_{240} - x_{80}$ or $(5.47855 - 4.37574) \times 5718.5 = x_{240} - 31000$

From which $x_{240} \approx 37300 \text{ m}^3/\text{s}$

Example 4.34. The annual maximum recorded floods for a river, for a period of 40 years from 1961 to 2000 are given below. Verify whether the Gumbel extreme-value distribution fits the recorded values. Estimate the flood discharge with a recurrence interval of 100 years, and 200 years by graphical interpolation as well as theoretical computations.

Year	1961	1962	1963	1964	1965	1966	1967	1968	1969	1970
Max. flood (m^3/s)	530	835	1030	570	380	1330	950	710	700	585
Year	1971	1972	1973	1974	1975	1976	1977	1978	1979	1980
Max. flood (m^3/s)	940	840	670	795	810	485	925	980	708	420
Year	1981	1982	1983	1984	1985	1986	1987	1988	1989	1990
Max. flood (m^3/s)	550	975	1095	620	595	853	465	855	625	505
Year	1991	1992	1993	1994	1995	1996	1997	1998	1999	2000
Max. flood (m^3/s)	390	500	705	460	605	495	945	756	610	825

Solution : The given flood data is entered in column (2) of the Table below, in descending order, while column (1) shows the rank of each discharge. The plotting position recurrence interval (T_p) for each discharge is computed from Weibull equation: $T_p = (N+1)/m$ where $N=40$ in the present case.

Order No. (m)	Discharge x (m^3/s)	T_p (years)	$(x - \bar{x})^2$	Order No. (m)	Discharge x (m^3/s)	T_p (years)	$(x - \bar{x})^2$
1	1330	41	377549	21	700	1.95	242
2	1095	20.5	143982	22	670	1.86	2075
3	1030	13.67	98879	23	625	1.78	8199
4	980	10.25	69934	24	620	1.71	9130
5	975	8.2	67314	25	610	1.64	11140
6	950	6.83	54967	26	605	1.58	12221
7	945	5.86	52647	27	595	1.52	14532
8	940	5.13	50378	28	585	1.46	17043
9	925	4.56	43869	29	570	1.41	21185
10	855	4.10	19446	30	550	1.37	27407
11	853	3.73	18893	31	530	1.32	34429
12	840	3.42	15488	32	505	1.28	44331
13	835	3.15	14268	33	500	1.24	48462
14	825	2.93	11979	34	495	1.21	48642
15	810	2.73	8921	35	485	1.17	53153
16	795	2.56	6312	36	465	1.14	62775
17	756	2.41	1636	37	460	1.11	65306
18	710	2.28	31	38	420	1.08	87850
19	708	2.16	57	39	390	1.05	106983
20	705	2.05	111	40	380	1.03	112594
				Σ	28622		1840860

$\Sigma x = 28622$ and hence $\bar{x} = 28622/40 \approx 715.6 \text{ m}^3/\text{s}$

$\Sigma (x - \bar{x})^2 = 1840860$; hence $\sigma_{n-1} = \sqrt{1840860/39} = 217.3$

Here

$N = 40$ years. Hence from Tables 4.21 (a) and (b), we get $\bar{y}_n = 0.5436$ and $S_n = 1.1413$

Choosing $N = 10$, years, we get

$$y_T = y_{10} = -[\ln \cdot \ln (10/9)] = 2.25037$$

$$K_T = K_{10} = \frac{y_T - \bar{y}_n}{S_n} = \frac{2.25037 - 0.5436}{1.1413} = 1.4955$$

$$x_T = x_{10} = \bar{x} + K_T \sigma_{n-1} = 715.6 + 1.4955 \times 217.3 \approx 1041 \text{ m}^3/\text{s}$$

Choosing $N = 20$ years, we get

$$y_T = y_{20} = -[\ln \cdot \ln (20/19)] = 2.9702$$

$$K_T = K_{20} = \frac{y_T - \bar{y}_n}{S_n} = \frac{2.9702 - 0.5436}{1.1413} = 2.1262$$

$$x_T = x_{20} = \bar{x} + K_T \sigma_{n-1} = 715.6 + 2.1262 \times 217.3 \approx 1178 \text{ m}^3/\text{s}$$

Finally, choosing $N = 5$ years, we have

$$y_T = y_5 = -[\ln \cdot \ln (5/4)] = 1.4999$$

$$K_T = K_5 = \frac{y_T - \bar{y}_n}{S_n} = \frac{1.4999 - 0.5436}{1.1413} = 0.8379$$

$$x_T = x_5 = \bar{x} + K_T \sigma_{n-1} = 715.6 + 0.8379 \times 217.3 \approx 898 \text{ m}^3/\text{s}$$

Thus we have following (theoretical) three sets of values

T (years)	Theoretical values of x_T
5	898
10	1041
20	1178

These values are plotted on Gumbel's probability paper, as shown in Fig. 4.68. It is seen that this theoretical line will also pass through mandatory point corresponding to $x_T = \bar{x} = 715.6$ and $T = 2.33$ years.

All these four points are shown by solid dots. On the same paper, the observed sets of values (x_T, T) are plotted, as shown by hollow dots in Fig. 4.68. It is seen that the data of the given series fit well with Gumbel's extreme value distribution function.

From the same graph, we get following values by extrapolation :

When $T = 100$, $x_{100} \approx 1500 \text{ m}^3/\text{s}$

When $T = 200$, $x_{200} \approx 1620 \text{ m}^3/\text{s}$

However, these values can also be computed theoretically as under :

When $T = 100$ years

$$y_T = y_{100} = -[\ln \cdot \ln (100/99)] = 4.6001$$

$$K_T = K_{100} = \frac{y_T - \bar{y}_n}{S_n} = \frac{4.6001 - 0.5436}{1.1413} = 3.5543$$

$$x_T = x_{100} = \bar{x} + K_T \sigma_{n-1} = 715.6 + 3.5543 \times 217.3 \pm 1488 \text{ m}^3/\text{s}$$

When $T = 200$ years

$$y_T = y_{200} = -(\ln \ln (200/199)) = 5.2958$$

$$K_T = (5.2958 - 0.5436)/1.1413 = 4.1639$$

$$x_T = x_{200} = 715.6 + 4.1639 \times 217.3 \pm 1620 \text{ m}^3/\text{s}$$

Example 4.35. From the analysis of available data on annual flood peaks of a stream for a period of 40 years, the 50 year and 100 year floods have been estimated to be $878 \text{ m}^3/\text{s}$ and $970 \text{ m}^3/\text{s}$. Using Gumbel's method, estimate the 200 year flood for the stream.

Solution

For $N = 40$ years, we get the following values from Table 4.21 (a) and (b).

$$y_n = 0.5436 \text{ and } S_n = 1.1413$$

These are thus constants for the series.

Now

$$y_{50} = -(\ln \ln (50/49)) = 3.90194$$

and

$$K_{50} = \frac{y_{50} - \bar{y}_n}{S_n} = \frac{3.90194 - 0.5436}{1.1413} = 2.9426$$

Similarly,

$$y_{100} = -(\ln \ln (100/99)) = 4.60015$$

and

$$K_{100} = \frac{y_{100} - \bar{y}_n}{S_n} = \frac{4.60015 - 0.5436}{1.1413} = 3.5543$$

From the general equation $x_T = \bar{x} + K_T \sigma_{n-1}$, we have

$$x_{50} = 878 = \bar{x} + 2.9426 \sigma_{n-1} \quad \dots(1)$$

and

$$x_{100} = 970 = \bar{x} + 3.5543 \sigma_{n-1} \quad \dots(2)$$

Solving the two simultaneous equations (1) and (2), we get

$$\bar{x} = 435.4 \text{ m}^3/\text{s} \text{ and } \sigma_{n-1} = 150.4 \text{ m}^3/\text{s}$$

Hence, we have

$$x_T = 435.4 + K_T (150.4) \quad \dots(3)$$

Again,

$$y_{200} = -(\ln \ln (200/199)) = 5.29581$$

$$K_{200} = \frac{y_{200} - \bar{y}_n}{S_n} = \frac{5.29581 - 0.5436}{1.1413} = 4.16386$$

Hence

$$x_{200} = 435.4 + 4.16386 (150.4) \pm 1063 \text{ m}^3/\text{s}$$

Example 4.36. A coffer dam is designed for a 30 year flood and constructed. If it takes 6 years to complete the construction of main dam, what is the risk that in the design of coffer dam would have reduced the risk to 10%? What return period

Solution The risk of failure is given by Eq. 4.92.

$$R_{dt} = 1 - \left(1 - \frac{1}{T}\right)^n, \text{ where } T = 30 \text{ years and } n = 6 \text{ years.}$$

$$\therefore R_{dt} = 1 - \left(1 - \frac{1}{30}\right)^6 = 0.184 \text{ or } 18.4\%$$

If risk is reduced to 10%, we have

$$0.1 = 1 - \left(1 - \frac{1}{T}\right)^6$$

which gives $T = 57.45 \pm 58$ years (say)

Example 4.37. A flood of a certain magnitude has a return period of 40 years. Determine : (a) the probability of exceedance, (b) probability of the flood of magnitude equal to or greater than the given magnitude occurring

(i) at least once in 10 successive years.

(ii) two times in 10 successive years, and

(iii) once in 10 successive years

Solution : Given : $T = 40$ years.

$$(a) \text{ Exceedance probability } = P = \frac{1}{T} = \frac{1}{40} = 0.025$$

(b) Probability of equaling or exceeding at least once in n successive years in given by Eq. 4.92.

$$R_{dt} = 1 - (1 - P)^n \text{ where } n = 10 \text{ here}$$

$$\therefore R_{dt} = 1 - (1 - 0.025)^{10} = 0.224$$

Probability of occurrence twice in n years is given by Eq. 4.85

$$P_{2,n} = \frac{n!}{(n-2)!2!} P^2 q^{n-2} \text{ where } q = 1 - P = 1 - 0.025 = 0.975 \text{ and } n = 10$$

$$\therefore P_{2,10} = \frac{10!}{8!2!} (0.025)^2 (0.975)^8 = \frac{8! \times 9 \times 10}{8! \times 2 \times 1} (0.025)^2 (0.975)^8 = 0.0229$$

Probability of exceedance once in $n (= 10)$ years is

$$P_{1,n} = n \cdot P \cdot q^{n-1} \text{ where } n = 10 \text{ years}$$

$$\therefore P_{1,10} = 10 (0.025) (0.975)^{10-1} = 0.199$$

Example 4.38. A small water shed consists of 3.2 km^2 of cultivated area with $C = 0.22$, 4.8 km^2 under forest with $C = 0.12$ and 1.8 km^2 under grass cover with $C = 0.32$. The water course, 2.4 km in length has a fall of 30 m . The intensity-frequency-duration relation for the area may be expressed by the following relation :

$$i = \frac{78 T^{0.22}}{(t + 12)^{0.45}}$$

where i is in cm/h , T is in years and t is in minutes. Estimate the peak rate of runoff for a 30 years frequency, using rational formula.

Solution : Slope of water course, $S = \Delta H/L = 30/2400 = 1/80$

$$t = t_c = 0.000323 L^{0.77} S^{-0.385} = 0.000323 (2400)^{0.77} \times \left(\frac{1}{80}\right)^{-0.385}$$

$$= 0.6993 \text{ hours} \triangleq 42 \text{ minutes}$$

$$i = \frac{78 T^{0.22}}{(t + 12)^{0.45}} = \frac{78 (30)^{0.22}}{(42 + 12)^{0.45}} = 27.38 \text{ cm/h}$$

$$Q_p = 2.778 C i A = 2.778 i \Sigma C \times A$$

$$= 2.778 \times 27.38 (3.2 \times 0.22 + 4.8 \times 0.12 + 1.8 \times 0.32) \triangleq 141.2 \text{ m}^3/\text{s}$$

Example 4.39. A small watershed has an area of 2.4 km^2 . The slope of the catchment is $1/200$ and the maximum length of travel of water is 1.8 km . The maximum depth of rainfall with a 30-year return period is given in the following Table

Duration (min)	5	10	15	20	25	30	40	50	60
Depth of rainfall (cm)	2.5	3.8	4.8	5.9	6.7	7.4	8.4	8.7	9.2

Determine the peak flow rate for a return period of 30 years if the average runoff coefficient for the watershed is 0.25.

Solution :

The time of concentration is given by Kirpich formula :

$$t_c = 0.000323 L^{0.77} S^{-0.385}$$

$$= 0.000323 (1800)^{0.77} \left(\frac{1}{200}\right)^{-0.385} = 0.7974 \text{ hours} = 47.84 \text{ min.}$$

For a duration ($t = t_c$) of 47.84 min., the maximum depth of rainfall can be determined from the above table by linear interpolation and is equal to

$$8.4 + \frac{8.7 - 8.4}{10} \times 7.84 \triangleq 8.64 \text{ cm}$$

Hence average intensity $i_{av} = \frac{8.64}{47.84} \times 60 = 10.83 \text{ cm/h}$

Hence $Q_p = 2.778 C i A = 2.778 \times 0.25 (10.83) \times 2.4 = 18.05 \text{ m}^3/\text{s}$

4.34. EXAMPLES FROM COMPETITIVE EXAMINATIONS

Example 4.40. A precipitation station X was inoperative for some time during which a storm occurred. At three stations A, B and C surrounding X, the total precipitation recorded during this storm are 75, 58 and 47 mm respectively. The normal annual precipitation amounts at stations X, A, B and C are 757, 826, 618 and 482 mm respectively. Estimate the storm precipitation for station X.

(Engg. services Exam. 1993)

Solution : This problem is similar to Example 4.3.

$$P_x = \frac{N_x}{3} \left[\frac{P_A}{N_A} + \frac{P_B}{N_B} + \frac{P_C}{N_C} \right] = \frac{757}{3} \left[\frac{75}{826} + \frac{58}{618} + \frac{47}{482} \right] \triangleq 71.2 \text{ mm}$$

Example 4.41. In a basin, six rain gauge stations are reporting rainfall in a year. The annual rainfall recorded by the gauges are :

Station	A	B	C	D	E	F
Rainfall (cm)	41	51	32	55	50	68

For 80% error in estimation of mean rainfall, calculate the optimum no. of rain gauge stations in the basin.

(Civil Services Exam. 1995)

Solution : This problem is similar to Example 4.1.

From Eq. 4.3, $N = \left[\frac{C_v}{p} \right]^2$

where $C_v = \frac{s_x}{\bar{x}} \times 100$; $\bar{x} = \frac{\Sigma x_i}{n}$ and $s_x = \sqrt{\frac{\Sigma (x_i - \bar{x})^2}{n-1}}$; $n = 6$

Here $\bar{x} = \frac{41 + 51 + 32 + 55 + 50 + 68}{6} = 49.5 \text{ cm}$

$$s_x = \left[\frac{(41 - 49.5)^2 + (51 - 49.5)^2 + (32 - 49.5)^2 + (55 - 49.5)^2 + (50 - 49.5)^2 + (68 - 49.5)^2}{5} \right]^{1/2}$$

$$= 12.28$$

$$C_v = \frac{s_x}{\bar{x}} \times 100 = \frac{12.28}{49.5} \times 100 = 24.80$$

$$p = 8\%$$

$$N = \left(\frac{C_v}{p} \right)^2 = \left(\frac{24.80}{8} \right)^2 = 9.61 \triangleq 10$$

Example 4.42. Find the mean precipitation for the area sketched in Fig. 4.70 by Thiessen's method. The area is composed of a square plus an equilateral triangular plot of side 4 kilometers. Rainfall readings in centimeters at the various station are also given in the Fig. (Engg Services Exam. 1968)

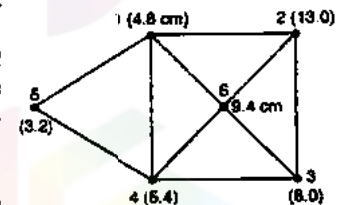


FIG. 4.70

Solution

Let the stations 1,2,3,4,5,6 be named as station A,B,C,D,E and F respectively, for convenience. Let the length of the sides of square ABCD be $a (= 4 \text{ km})$. Then the length of each side of the equilateral triangular plot will be also a . Now for the triangular plot, draw perpendicular bisectors Aa, Dd and Ee, so that they meet in point g. Similarly, draw the perpendicular bisectors eb, bc, cf and fe of the lines FA, FB, FC and FD respectively, shown in Fig. 4.71.

Evidently, station F (or station 6) will be fed by the rectangular area, bcfe, where length of its side, say bc will be equal to $\frac{1}{2} AC = \frac{1}{2} \sqrt{2} a = a/\sqrt{2}$.

$$\text{Hence area } bcfe = A_s = \frac{a}{\sqrt{2}} \times \frac{a}{\sqrt{2}} = \frac{a^2}{2} = \frac{(4)^2}{2} = 8 \text{ km}^2 \quad \dots(i)$$

Then each of the corner stations, say station A, will be fed by triangular area Abe and sectorial area Adge.

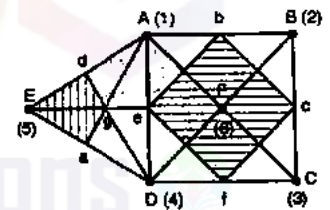


FIG. 4.71

Triangular area $A_{be} = \frac{1}{2} \cdot \frac{a}{\sqrt{2}} \cdot \left(\frac{\sqrt{2}a}{4} \right) = \frac{a^2}{8}$

Sectorial area $A_{de} = \frac{1}{3}$ area of triangle $ADE = \frac{1}{3} \times \frac{1}{2} a \times \frac{\sqrt{3}}{2} a = \frac{a^2}{4\sqrt{3}}$

Hence station A will be fed by area = area A_{be} + area A_{de}

$$= \frac{a^2}{8} + \frac{a^2}{4\sqrt{3}} = \frac{(4)^2}{8} + \frac{(4)^2}{4\sqrt{3}} = 4.3094 \text{ km}^2$$

Hence, $A_1 = A_4 = 4.3094 \text{ km}^2$... (ii)

Also station E will be fed by sectorial area $A_{de} = \text{area } A_{de} = \frac{a^2}{4\sqrt{3}}$

Hence $A_5 = \frac{a^2}{4\sqrt{3}} = \frac{(4)^2}{4\sqrt{3}} = 2.3094 \text{ km}^2$... (iii)

Station B will be fed by area $bcb = \text{area } A_{be} = \frac{a^2}{8} = \frac{(4)^2}{8} = 2 \text{ km}^2$

$\therefore A_2 = A_3 = 2 \text{ km}^2$... (iv)

Lastly, station F will be fed by area $ebcf = \left(\frac{a}{\sqrt{2}} \right)^2$

$\therefore A_6 = \frac{(4)^2}{2} = 8 \text{ km}^2$

Hence $P_{av} = \frac{P_1 A_1 + P_2 A_2 + \dots + P_6 A_6}{A_1 + A_2 + \dots + A_6}$

$$= \frac{4.8 \times 4.3094 + 13.0 \times 2 + 8.0 \times 2 + 5.4 \times 4.3094 + 3.2 \times 2.3094 + 9.4 \times 8}{4.3094 + 2 + 2 + 4.3094 + 2.3094 + 8}$$

$$= 7.36 \text{ cm}$$

Example 4.43. For a drainage basin of 640 km^2 , isohyets based on a storm event yield the following data :

Isohyet interval (cm)	14-12	12-10	10-8	8-6	6-4	4-2	2-0
Inter isohyetal area (km^2)	90	140	125	140	85	40	20

Estimate the average depth of precipitation over the basin.

(Engg. Services Exam. 2002)

Solution : $P_{av} = \frac{\sum A \times \left[\frac{P_1 + P_2}{2} \right]}{\sum A}$... (4)

$\therefore P_{av} = \frac{[(90 \times 13) + (140 \times 11) + (125 \times 9) + (140 \times 7) + (85 \times 5) + (40 \times 3) + (20 \times 1)]}{640}$

$$\triangleq 8.41 \text{ cm}$$

Example 4.44. The information available from a Isohyetal map of 1100 sq. km basin is as follows :

Zone	Area (km^2)	Rain gauge station	Normal annual rainfall (cm)
I	85	A	120
II	290	B	95
III	395	C	96
		D	60
		E	65
IV	230	F	70
V	85	G	45
		H	21

How many additional rain gauge stations will be required if the desired limit of error in the mean value of rainfall is not to exceed 10 cm? Suggest how you propose to distribute these stations. What factors will you consider in locating the additional rain gauge stations between different isohyets. (Engg Services Exam 1994)

Solution : This problem is similar to Example 4.1.

Optimum No. of rain gauge stations is given by Eq. 4.4.

$$N = \left[\frac{C_v}{p} \right]^2 \quad \text{where } C_v = \frac{s_x}{\bar{x}} \times 100$$

where s_x = standard deviation $\sqrt{\frac{\sum (x_i - \bar{x})^2}{n-1}}$, where $n=8$ here.

$$\bar{x} = \frac{1}{8} [120 + 95 + 96 + 60 + 65 + 70 + 45 + 21] = 71.5 \text{ cm.}$$

$$s_x = \left[\frac{(120-71.5)^2 + (95-71.5)^2 + (96-71.5)^2 + (60-71.5)^2 + (65-71.5)^2 + (70-71.5)^2 + (45-71.5)^2 + (21-71.5)^2}{7} \right]^{1/2} = 31.47$$

$$C_v = \frac{s_x}{\bar{x}} \times 100 = \frac{31.47}{71.5} \times 100 = 44.02$$

$$p = \frac{10}{\bar{x}} \times 100 = \frac{10}{71.5} \times 100 = 13.99\%$$

$$N = \left(\frac{C_v}{p} \right)^2 = \left(\frac{44.02}{13.99} \right)^2 = 9.91 \triangleq 10$$

Hence two more rain gauge station are required. One of these should be in zone IV and the other should be located between zones III and IV.

Example 4.45. The mass curve of precipitation resulted from the storm of 14th Aug 1983 gave the following values.

Hour-min	Accumulated depth at the end of periods (in mm)	Hour-min	Accumulated depth at the end of periods (in mm)
22.00	0.0 (Beginning of storm)	22.30	84.0
22.05	10.2	22.35	71.6
22.10	20.8	22.40	78.8
22.15	33.0	22.45	85.4
22.20	47.3	22.50	91.4 (End of storm)
22.25	55.8		

From the above storm, construct hyetograph and draw the maximum intensity-duration curve.

Solution : This problem is similar to Example 4.8.

First of all, let us calculate the rainfall intensities, using uniform time interval of 5 minutes, in a tabular form below, which is self explanatory.

Time (hr. min)	Accumulated rainfall (cm)	Rainfall in successive 5 min. interval (cm)	Rainfall intensity cm/hr.
(1)	(2)	(3)	(4) = (3) $\times$ 60/5
22.00	0.00		
22.05	1.02	1.02	12.24
22.10	2.08	1.06	12.72
22.15	3.30	1.22	14.64
22.20	4.72	1.42	17.04
22.25	5.58	0.86	10.32
22.30	6.40	0.82	9.84
22.35	7.16	0.76	9.12
22.40	7.88	0.72	8.64
22.45	8.54	0.66	7.92
22.50	9.14	0.60	7.20

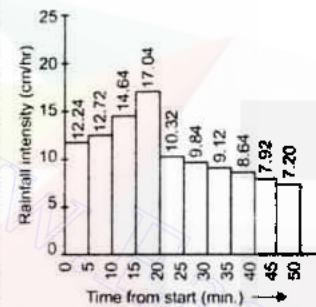


FIG. 4.72

Fig 4.72 shows the rainfall hyetograph.

Next let us find the maximum rainfall values of rainfall durations of 5 min, 10 min, 15 min, 20 min, ...50 min. For this, the computations are done in tabular form below. In this table, the rainfall values of 5 min time interval of rain will be the same as computed in column (3) of the table above. However, for interval of 10 min. the rainfall values will be obtained by adding the rainfall values of two successive durations of 5 minutes each. Similarly, for interval of 15 min. the rainfall values will be obtained by adding the rainfall values of three successive durations of 5 minutes each, and so on. Thus, the rainfall values of various times intervals of 5 min, 10 min, 15 min 50 min will be obtained, as tabulated in columns (3), (4), (5)(12) respectively, of the table. For each of these time interval, the maximum rainfall value will be selected, as shown in bold letters in the Table.

Time (hr. min.)	Accumulated rainfall (cm)	Rainfall in any possible time interval equal to									
		5 min	10 min	15 min	20 min	25 min	30 min	35 min	40 min	45 min	50 min
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
22.00	0.00										
22.05	1.02	1.02									
22.10	2.08	1.06	2.08								
22.15	3.30	1.22	2.28	3.30							
22.20	4.72	1.42	2.64	3.70	4.72						
22.25	5.58	0.86	2.28	3.50	4.56	5.58					
22.30	6.40	0.82	1.68	3.10	4.32	5.38	6.40				
22.35	7.16	0.76	1.58	2.44	3.86	5.08	6.14	7.16			
22.40	7.88	0.72	1.48	2.30	3.16	4.58	5.80	6.86	7.88		
22.45	8.54	0.66	1.38	2.14	2.96	3.82	5.24	6.46	7.52	8.54	
22.50	9.14	0.60	1.26	1.98	2.74	3.56	4.42	5.84	7.06	8.12	9.14

After having found the maximum rainfall during each of the possible rainfall durations, we compute the max. intensity of rainfall for each of these durations, in Tabular form below. Finally, we plot a graph between rainfall intensity (col.3) and the rainfall durations (col. 1), as shown in Fig. 4.73.

Duration (min.)	Max. rainfall (cm)	Max. rainfall intensity (cm/hr.)
(1)	(2)	(3) = (2) $\times$ 60/5
05	1.42	17.04
10	2.64	15.84
15	3.70	14.80
20	4.72	14.16
25	5.58	13.39
30	6.40	12.80
35	7.16	12.27
40	7.88	11.82
45	8.54	11.39
50	9.14	10.97

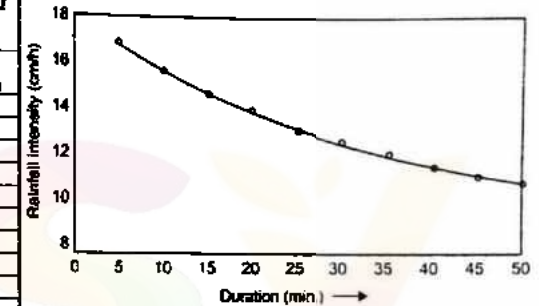


FIG. 4.73

Example 4.46. Compute and draw the storm hyetograph and the intensity duration curve for the following storm (of a given frequency) on a drainage basin :

Duration (minute)	Accumulated Precipitation (cm)
0	-
30	5.0
60	7.5
90	8.5
120	9.0

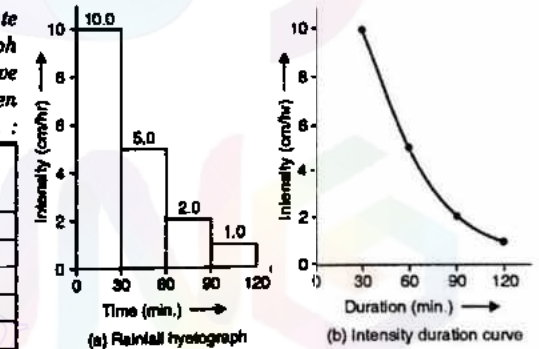


FIG. 4.74

(Engg. Services Exam. 1998)

Solution

The computations are done in a tabular form below :

Time (min.)	Accumulated Rainfall (cm)	Rainfall in successive 30 min. interval (cm)	Rainfall intensity (cm/hr)
(1)	(2)	(3)	(4) = (3) $\times$ 60/30
0	0.0		
30	5.0	5.0	10.0
60	7.5	2.5	5.0
90	8.5	1.0	2.0
120	9.0	0.5	1.0

The rainfall hyetograph is shown in Fig. 4.74 (a) while the intensity-duration curve is shown in Fig. 4.74 (b).

Example 4.47. Estimate the average depth of precipitation, from depth area curve, that may be expected over an area of 2400 sq. km, due to the storm of 27th Sept. 1978, lasting for 24 hours, assuming the storm centre to be located at the centre of the area. The Isohyetal map for the storm gave the areas enclosed between different isohyets as follows :

Isohyet in mm	21	20	19	18	17	16	15	14	13	12
Enclosed area in sq. km	543	1345	2030	2545	2955	3280	3535	3710	3890	3915

Hence, determine the depth of rainfall due to the storm, that may be expected to be recorded by a rain gauge placed at the storm centre.

(Engg. Services Exam, 1986)

Solution : This problem is exactly similar to Example 4.10. The computations for equivalent uniform depth (EUD) are done in a tabular form below :

Isohyet (mm)	Area enclosed between isohyet and boundary of basin (km ²)	Net incremental area between isohyets (km ²)	Average rainfall on incremental area (mm)	Rainfall volume on incremental area (mm. km ²)	Cumulative volume (mm. km ²)	EUD (mm)
(1)	(2)	(3)	(4)	(5) = (3) × (4)	(6) = Σ (5)	(7) = (6) / (2)
21	543	543	21.5 (avg)	11676	11676	21.50
20	1345	802	20.5	16441	28116	20.90
19	2030	686	19.5	13357	41473	20.43
18	2545	515	18.5	9528	51001	20.04
17	2955	410	17.5	7175	58176	19.69
16	3280	325	16.5	5362	63538	19.37
15	3535	256	15.5	3968	67491	19.09
14	3710	175	14.5	2537	70028	18.88
13	3890	170	13.5	2295	72323	18.64
12	3915	25	12.5	312	72761	18.59

Fig. 4.75 shows the plot between mean depth of rainfall (EUD) and area enclosed between the isohyets and the boundary of the basin. From the above depth-area curve, we obtain average depth of precipitation = 20.15 mm for an area of 2400 sq. km.

Example 4.48. At the beginning of a certain week, the depth of water in an evaporation pan, 1.2 metres diameter, was 7.75 cm. During the week, the rainfall was 3.8 cm, and 2.5 cm of water was removed from the pan to keep the depth of water in it within a fixed

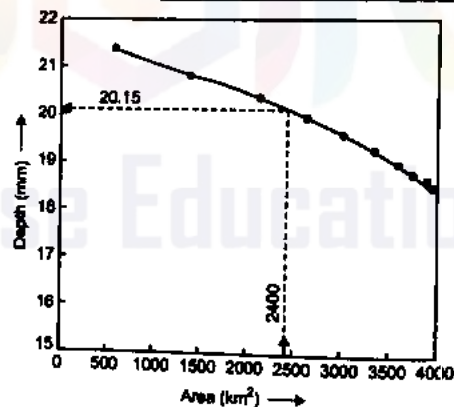


FIG. 4.75

range. At the end of the week, the gauge indicated a depth of 8.32 cm of water in the pan. Using a suitable evaporation pan coefficient, estimate the evaporation during the week from the surface of reservoir under similar atmospheric conditions.

(Engg. Services Exam. 1970)

Solution : Given :

Initial depth of water in the pan = 7.75 cm

Rainfall during the week = 3.80 cm

Depth of water removed = 2.50 cm

∴ Net addition of water in the pan = 3.8 - 2.50 = 1.30 cm

If there were no loss, depth at the end of week = 7.75 + 1.30 = 9.05 cm

Actual depth at the end of week = 8.32 cm

∴ Evaporation loss from the pan = 9.05 - 8.32 = 0.73 cm

Let the Pan coefficient = 0.7

∴ Evaporation loss from reservoir = 0.7 × 0.73 = 0.51 cm during the week.

Example 4.49. A 12 hour storm rainfall has the following depth in cm for each hour occurring over a basin :

1.8, 2.6, 7.8, 3.9, 10.6, 5.4, 7.8, 9.2, 6.5, 4.4, 1.8 and 1.6

The surface runoff resulting from the above storm is found to be 24.4 cm depth over the basin. Determine the average infiltration index for the basin.

(Gate : 1992)

Solution :

This problem is similar to Example 4.19.

Total $P = 1.8 + 2.6 + 7.8 + 3.9 + 10.6 + 5.4 + 7.8 + 9.2 + 6.5 + 4.4 + 1.8 + 1.6$

= 63.4 cm

$t = 12$ hours

Total surface runoff = 24.4 cm (given)

$$W_i = \frac{P - R}{t} = \frac{63.4 - 24.4}{12} = 3.25 \text{ cm/hour.}$$

Now Φ_i will be greater than W_i , i.e. greater than 3.25 cm/hour

Let us assume period of effective rainfall, t_e to be equal to 7 hours (i.e. $\Phi_i \geq 3.9$), we

get

$$\Phi_i = \frac{(63.4 - 24.4) - (1.8 + 2.6 + 3.9 + 1.8 + 1.6)}{7} = 3.9 \text{ cm/hour}$$

Check : This value of Φ_i is satisfactory since it gives $t_e = 7$ hours as assumed.

Let us verify the rainfall excess (i.e. direct runoff) for these values

Time from start (h)	1	2	3	4	5	6	7	8	9	10	11	12	sum
Rainfall excess = $P - \Phi_i$	0	0	3.9	0	6.7	1.5	3.9	5.3	2.6	0.5	0	0	24.4

From the above Table, we get $R =$ rainfall excess = 24.4 cm.

Hence $\Phi_i = 3.9 \text{ cm/hour}$

Example 4.50. A storm with 15.0 cm precipitation produced a direct runoff of 8.7 cms. The time distribution of the storm is as follows :

Time from start (hours)	1	2	3	4	5	6	7	8
Incremental rainfall in each hour (cm)	0.6	1.35	2.25	3.45	2.70	2.40	1.50	0.75

(Civil Services Exam. 1987)

Solution : This problem is similar to Example 4.23.

The given data about rainfall in each hour also represent rainfall intensity in cm/hr. Fig. 4.76 shows the rainfall hyetograph for total rainfall duration $t_r = 8$ h.

Total $P = 15$ cm

Total direct runoff, $R = 8.7$ cm

$$W_i = \frac{P - R}{t_r} = \frac{15 - 8.7}{8} = 0.7875 \text{ cm/h}$$

Since Φ_i will be more than W_i and hence more than 0.7875 cm/h, we find that the first hour and the last hour of the rainfall will be ineffective in producing rainfall excess. Hence effective rainfall duration, t_{re} will be $= 8 - 2 = 6$ hours.

$$\Phi_i = \frac{(15 - 8.7) - 0.6 - 0.75}{6} = 0.825 \text{ cm/hr.}$$

Example 4.51. Assuming the initial infiltration rate of 10 mm/h, final infiltration rate of 5 mm/h and the constant value (describing the rate of decay of the difference between the initial and final infiltration rates) as 0.95 h^{-1} , calculate the total infiltration depth for a storm lasting 6 h.

Solution

The equation of the infiltration curve is

$$f_t = f_c + (f_0 - f_c)e^{-kt} \quad \dots(4.36)$$

Given : $f_c = 5$ mm/h, $f_0 = 10$ mm/h and $k = 0.95 / \text{h}$

Hence substituting the values in Eq. 4.36, we get

$$f_t = 5 + (10 - 5)e^{-0.95t} = 5 + 5e^{-0.95t} \quad \dots(1)$$

Integrating the above equation for t between 0 hour to 6 hours, we get

$$\begin{aligned} \text{Total infiltration} &= \int_0^6 f_t dt = \int_0^6 (5 + 5e^{-0.95t}) dt = 5 \left[t \right]_0^6 + 5 \left[\frac{e^{-0.95t}}{-0.95} \right]_0^6 \\ &= 5 [6 - 0] - \frac{5}{0.95} [e^{-0.95 \times 6} - e^0] \\ &= 30 - \frac{5}{0.95} [3.346 \times 10^{-3} - 1] = 35.25 \text{ mm} \end{aligned}$$

Example 4.52. The infiltration capacities of an area at different intervals of time are indicated below. Find an equation for the infiltration capacity in the exponential form.

Time in hours	0	0.25	0.50	0.75	1.00	1.25	1.50	1.75	2.00
Infiltration capacity (f) in cm/hr.	10.4	5.8	3.2	2.1	1.5	1.2	1.1	1.0	1.0

(Engg Services Exam. 1970)

Solution

Let the equation of the infiltration curve be :

$$f_t = f_c + (f_0 - f_c)e^{-kt} \quad \dots(4.36)$$

where f_c = constant infiltration rate ≈ 1.0 cm/hr

f_0 = infiltration rate at the beginning = 10.4 cm/hr.

k = constant to be determined.

Hence

$$f_t = 1.0 + (10.4 - 1.0)e^{-kt}$$

or

$$f_t = 1.0 + 9.4e^{-kt} \quad \dots(1)$$

In order to get the value of k , let us make a plot between t on simple algebraic scale on the ordinate and $\log_{10}(f_t - f_c)$ on the abscissa.

t (hours)	0	0.25	0.50	0.75	1.0	1.25	1.50	1.75	2.00
f_t (cm/hr)	10.4	5.8	3.2	2.1	1.5	1.2	1.1	1.0	1.0
$(f_t - f_c)$	9.4	4.8	2.2	1.1	0.5	0.2	0.1	0.0	0.0
$\log_{10}(f_t - f_c)$	+ 0.973	+ 0.683	+ 0.342	+ 0.041	- 0.301	- 0.699	- 1.00		

Fig. 4.77 shows the straight line plot between t and $\log(f_t - f_c)$.

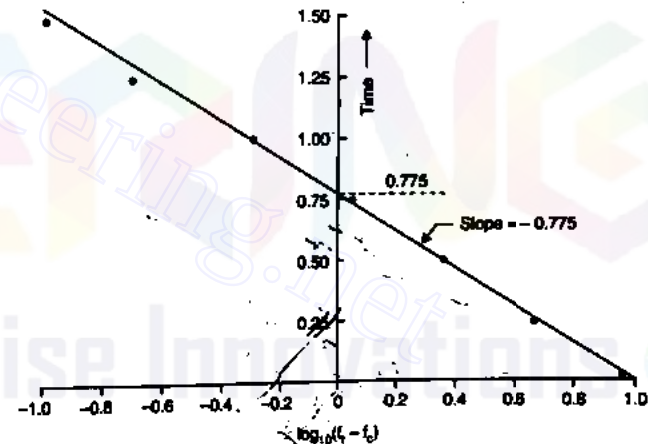


FIG. 4.77

$$\text{Slope of the line} = \frac{0.775}{1} = -0.775$$

Hence

$$-\frac{1}{k \log_{10} e} = -0.775$$

or $k \log_{10} 2.7183 = \frac{1}{0.775} = 1.2903$

From which $k = \frac{1.2903}{0.43429} \approx 2.97$

Hence the desired equation is

$$f_t = 1.0 + 0.4e^{-2.97t}$$

Example 4.53. The following is the set of observed data for successive 15 minutes period of 105 minutes storm in a catchment.

Duration (min.)	Rainfall (cm/hour)
15	2.0
30	2.0
45	8.0
60	7.0
75	1.25
90	1.25
105	4.5

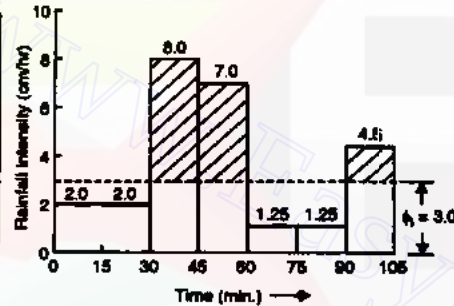


FIG. 4.78

If the value of Φ -index is 3.0 cm/hr, estimate the net runoff, the total rainfall and the value of W -index.

(Engg. Services Exam. 1996)

Solution

Fig. 4.78 shows the rainfall hyetograph, Φ -index and rainfall excess hyetograph situated above the Φ -line.

Total rainfall $P = (2 \times 30 + 8 \times 15 + 7 \times 15 + 1.25 \times 30 + 4.5 \times 15) / 60 = 6.5$ cm

Net runoff $R =$ rainfall excess (shown hatched in Fig. 4.78)

$$= [(8.0 - 3.0) \times 15 + (7.0 - 3.0) \times 15 + (4.5 - 3.0) \times 15] / 60 = 2.625 \text{ cm}$$

$$W_i = \frac{P - R}{t_r} = \frac{6.5 - 2.625}{(105/60)} = 2.214 \text{ cm/hour}$$

Example 4.54. An isolated 3 hour storm occurred over an area of 120 ha. as below :

Partial area of catchment (ha)	Φ -index (cm/hr)	Rainfall (cm)		
		1st hour	2nd hour	3rd hour
36	0.90	0.6	2.4	1.3
18	1.10	0.9	2.1	1.5
66	0.50	1.0	2.0	0.8

What is the total rainfall on the catchment in this storm? Estimate the runoff from the catchment. If the Φ -index were to remain at the same value, what runoff would be produced by a uniform rainfall of 3.3 cm in 3 hours uniformly spread over the catchment?

(Civil Services Exam. 1989)

Solution

Total rainfall on 36 hectares = 0.6 + 2.4 + 1.3 = 4.3 cm

Total rainfall on 18 hectares = 0.9 + 2.1 + 1.5 = 4.5 cm

Total rainfall on 66 hectares = 1.0 + 2.0 + 0.9 = 3.9 cm

$$\therefore \text{Total rainfall on the entire catchment} = \frac{\sum P \times A}{\sum A} = \frac{4.3 \times 36 + 4.5 \times 18 + 3.9 \times 66}{120} = 4.11 \text{ cm}$$

Runoff from the catchment is computed in a tabular form below

Area (ha)	Runoff = Rainfall - Φ_i (cm)			Total (cm)
	(1)	(2)	(3)	
36	0	1.5	0.4	1.9
18	-	1.0	0.4	1.4
66	0.5	1.5	0.4	2.4

$$\therefore \text{Runoff from the entire catchment} = \frac{36 \times 1.9 + 18 \times 1.4 + 66 \times 2.4}{120} = 2.1 \text{ cm}$$

$$\text{Average } \Phi_i \text{ for the entire catchment} = \frac{0.9 \times 36 + 1.10 \times 18 + 0.5 \times 66}{120} = 0.71 \text{ cm/hr}$$

Total uniform rainfall = 3.3 cm in 3 hours = 1.1 cm/hr.

$$\therefore \text{Total runoff} = (1.1 - 0.71) \times 3 = 1.17 \text{ cm}$$

Example 4.55. Given below are the monthly rainfall, P and the corresponding runoff values R , for a period of 10 months for a catchment. Develop a correlation between R and P .
(Engg. Services Exam. 2003)

Month	P (cm)	R (cm)
1	4	0.2
2	22	7.1
3	28	10.9
4	15	4.0
5	12	3.0
6	8	1.3
7	4	0.4
8	16	4.1
9	10	2.0
10	5	0.3

Solution : This problem is similar to Example 4.20.

Let the correlation equation be

$$R = aP + b \quad \dots(4.43)$$

where constants a and b are given by Eqs. 4.43 (a) and (b) respectively. The computations are done in a Tabular form below :

S.N. of month	P	P ²	R	R ²	PR
1	4	16	0.2	0.04	0.8
2	22	484	7.1	50.41	156.2
3	28	784	10.9	118.81	305.2
4	15	225	4.0	16.00	60
5	12	144	3.0	9.00	36
6	8	64	1.3	1.69	10.4
7	4	16	0.4	0.16	1.6
8	16	256	4.1	16.81	65.6
9	10	100	2.0	4.00	20
10	6	36	0.3	0.09	1.8
Σ	123	2083	33.3	217.01	653.2

From the Table, $\Sigma P = 123$; $(\Sigma P)^2 = 15129$; $\Sigma P^2 = 2083$; $N = 10$
 $\Sigma R = 33.3$, $(\Sigma R)^2 = 1108.9$; $\Sigma R^2 = 217.01$; $\Sigma PR = 653.2$

$$a = \frac{N(\Sigma PR) - (\Sigma P)(\Sigma R)}{N(\Sigma P^2) - (\Sigma P)^2} = \frac{10 \times 653.2 - 123 \times 33.3}{10 \times 2083 - 15129} = 0.4273$$

and
$$b = \frac{\Sigma R - a \Sigma P}{N} = \frac{33.3 - 0.4273 \times 123}{10} = -1.9259$$

Hence the correlation equation is

$$R = 0.4273P - 1.9259$$

Example 4.56. A peak of a flood hydrograph due to a six-hour storm is 470 m³/sec. The average depth of rainfall is 8.0 cms. Assume an infiltration loss of 0.25 cm/hour and a constant base flow of 15 m³/sec. Estimate the peak discharge of 6 hour unit hydrograph for this catchment. (Civil Services Exam. 1996)

Solution

Peak discharge of flood hydrograph = 470 m³/sec

Base flow = 15 m³/sec

Peak discharge of direct runoff hydrograph = 470 - 15 = 455 m³/sec

Average depth of rainfall = 8.0 cm.

Rainfall excess = 8.0 - 0.25 × 6 = 6.5 cm

Peak discharge of unit hydrograph = $\frac{455}{6.5} = 70$ m³/sec

Example 4.57. The ordinates of a 4-hour unit hydrograph of catchment are given below :

Time (hours)	0	4	8	12	16	20	24	28	32	36	40	44
Ordinate of 4h UH (m ³ /sec)	0	20	60	180	120	90	70	50	30	20	10	0

Derive the flood hydrograph in the catchment due to the storm given below :

Time from start of storm (h)	0	4	8	12
Accumulated rainfall (cm)	0	5.0	6.8	8.8

The Φ -index for the catchment can be assumed to be 0.25 cm/hour and a constant base flow of 20 cm³/sec. is appropriate. (Civil services Exam 1991)

Solution : This problem is similar to Example 4.28.

	(1st 4 hr)	(2nd 4 hour)	(3rd 4 hour)
Time from start of storm	0	4	8
Accumulated rainfall (cm)	0	5.0	5.8
Rainfall in each period (cm)		5.0	0.8
Loss in each period = $4 \times \Phi$		1.0	1.0
Effective rainfall		4.0	0

The computation for the ordinates of flood hydrograph are in a Tabular form below which is self explanatory

Time (hours)	Ordinate of 4-h UH (m ³ /sec)	Surface runoff from rainfall excess during successive unit periods				Base flow	Ordinate of flood hydrograph
		$u_1 = 4.0$	$u_2 = 0.0$	$u_3 = 2.0$	Sub-total		
(1)	(2)	(3) = (2) × 4.0	(4) = (2) × 0.0	(5) = (2) × 2.0	(6)	(7)	(8) = (6) + (7)
0	0	0			0	20	20
4	20	80	0		80	20	100
8	60	240	0	0	240	20	260
12	180	600	0	40	340	20	660
16	120	480	0	120	300	20	620
20	90	360	0	300	560	20	680
24	70	280	0	240	520	20	540
28	50	200	0	180	380	20	400
32	30	120	0	140	260	20	280
36	20	80	0	100	180	20	200
40	10	40	0	60	100	20	120
44	0	0	0	40	40	20	60
48			0	20	20	20	40
52				0	0	20	20

Example 4.58. On a catchment area of 200 sq. km, rainfalls of 7.5 cm 2.0 cm and 5.0 cm occurred on three consecutive days. The Φ -index was 2.5 cm/day. The distribution graph percentages of surface runoff which extended over six days for every rainfall of such magnitudes are 5, 15, 40, 25, 10 and 5. Determine the ordinates of the discharge hydrograph and determine the peak discharge. Neglect base flow. (Engg Services Exam. 1983)

Solution:

The runoff, measured as depth on three consecutive days will be equal to rainfall excess on these three days, with the values as follows :

Runoff on first day = 7.5 - 2.5 = 5.0 cm

Runoff on second day = 2.0 - 2.5 = 0.0 cm

Runoff on second day

Runoff on third day = $5.0 - 2.5 = 2.5$ cm

Runoff, measured as depth of water per day, spread over the whole catchment area can be converted into discharge as follows :

$$1 \text{ cm runoff per day over } 200 \text{ km}^2 \text{ area} = \frac{1}{100} (200 \times 10^6) \text{ m}^3/\text{day}$$

$$= \frac{(2 \times 10^6)}{24 \times 3600} \text{ m}^3/\text{sec} = 23.148 \text{ m}^3/\text{sec}.$$

However, the discharge pattern is governed by distribution graph percentages, contributed by effective rain of each day, spread over 6 days from its occurrence. Since there is effective rainfall on three consecutive days, these discharge contributions by each day rainfall excess are added to give total discharge. The computations are arranged in Tabular form below which is self explanatory.

Time interval in days	Rainfall excess (cm)	Distribution graph %	Distributed runoff for rainfall excess of			Total runoff	
			5 cm	0 cm	2.5 cm	As depth (cm)	As discharge (cumecs)
(1)	(2)	(3)	(4)	(5)	(6)	(7) = (4) + (5) + (6)	(8) = 23.148 × (7)
0-1	5.0	5	0.25			0.25	5.787
1-2	0.0	15	0.75	0		0.75	17.361
2-3	2.5	40	2.00	0	0.125	2.125	49.190
3-4		25	1.25	0	0.375	1.625	37.616
4-5		10	0.50	0	1.00	1.500	34.722
5-6		5	0.25	0	0.825	0.875	20.255
6-7			0	0	0.25	0.25	5.787
7-8				0	0.125	0.125	2.894
8-9					0	0	0
Total	7.5	100	5.0	0	2.5	7.5	173.612

Check : Total runoff (R) = $\frac{0.36 \Sigma O \times t}{A} = \frac{0.36 \times 173.612 \times 24}{200} = 7.5 \text{ cm}$

Example 4.59. The ordinates of a flood hydrograph, resulting from two successive storms each of 1 cm rainfall excess and 6 hour duration, are tabulated below. Find a 6 hour unit hydrograph.

Time (hours)	Ordinate of 6 hr flood hydrograph (m ³ /s)	Time (hours)	Ordinate of 6 hr flood hydrograph (m ³ /s)
0	10	42	126
6	30	48	92
12	90	54	62
18	220	60	40
24	280	66	20
30	220	72	10
36	166		

(Engg. Services Exam. 1997)

Solution :

If R_1 and R_2 are the rainfall excesses, we have $R_1 = 1$ cm and $R_2 = 1$ cm. Assuming a base flow of $10 \text{ m}^3/\text{s}$, the ordinates $Q_1, Q_2, \dots, Q_{12}$ of the direct runoff hydrograph will be as given in Table below.

Let $U_1, U_2, U_3, \dots, U_{12}$ be the ordinates of the 6 hours unit hydrograph. The computations are done in tabular form below.

Time (hours)	Ordinate of 6 hr. flood hydrograph (m ³ /s)	Base flow (m ³ /s)	Ordinate of direct runoff	Equations for direct runoff hydrograph	Values put in the equation	Ordinate of 6 hr. unit hydrograph
0	10	10	0 (Q_1)	$Q_1 = R_1 U_1$	$0 = 1 \times U_1$	$U_1 = 0$
6	30	10	20 (Q_2)	$Q_2 = R_1 U_2 + R_2 U_1$	$20 = 1 \times U_2 + 1 \times 0$	$U_2 = 20$
12	90	10	80 (Q_3)	$Q_3 = R_1 U_3 + R_2 U_2$	$80 = 1 \times U_3 + 1 \times 20$	$U_3 = 60$
18	220	10	210 (Q_4)	$Q_4 = R_1 U_4 + R_2 U_3$	$210 = 1 \times U_4 + 1 \times 60$	$U_4 = 150$
24	280	10	270 (Q_5)	$Q_5 = R_1 U_5 + R_2 U_4$	$270 = 1 \times U_5 + 1 \times 150$	$U_5 = 120$
30	220	10	210 (Q_6)	$Q_6 = R_1 U_6 + R_2 U_5$	$210 = 1 \times U_6 + 1 \times 120$	$U_6 = 90$
36	166	10	156 (Q_7)	$Q_7 = R_1 U_7 + R_2 U_6$	$156 = 1 \times U_7 + 1 \times 90$	$U_7 = 60$
42	126	10	116 (Q_8)	$Q_8 = R_1 U_8 + R_2 U_7$	$116 = 1 \times U_8 + 1 \times 60$	$U_8 = 50$
48	92	10	82 (Q_9)	$Q_9 = R_1 U_9 + R_2 U_8$	$82 = 1 \times U_9 + 1 \times 50$	$U_9 = 32$
54	62	10	52 (Q_{10})	$Q_{10} = R_1 U_{10} + R_2 U_9$	$52 = 1 \times U_{10} + 1 \times 32$	$U_{10} = 20$
60	40	10	30 (Q_{11})	$Q_{11} = R_1 U_{11} + R_2 U_{10}$	$30 = 1 \times U_{11} + 1 \times 20$	$U_{11} = 10$
66	20	10	10 (Q_{12})	$Q_{12} = R_1 U_{12} + R_2 U_{11}$	$10 = 1 \times U_{12} + 1 \times 10$	$U_{12} = 0$
72	10	10	10 (Q_{13})	$Q_{13} = R_1 U_{13} + R_2 U_{12}$	$10 = 1 \times U_{13} + 1 \times 10$	$U_{13} = 0$

Example 4.60. The following are the ordinates for a flood hydrograph resulting from an isolated storm of 6 hours duration.

Time (hr)	0	12	24	36	48	60	72	84	96
Ordinate of flood hydrograph (cumecs)	5	15	40	80	60	50	25	15	5

Determine the ordinates of 1 cm-6 hour unit hydrograph if catchment area is 450 sq. km .

(Engg. Services Exam. 2001)

Solution

Let us assume a constant base flow of 5 cumecs. The ordinates of direct runoff hydrograph are obtained by subtracting base flow (= 5 cumecs) from each ordinate

of the given flood hydrograph, as shown in the Table below.

Time (hours)	0	12	24	36	48	60	72	84	96
Ordinates of flood hydrograph	5	15	40	80	60	50	25	15	5
Ordinate of direct runoff	0	10	35	75	55	45	20	10	0
Ordinate of unit hydrograph	0	4.17	14.58	31.25	22.92	18.75	8.33	4.17	0

The direct runoff is given by

$$\text{Direct Runoff} = n = 0.36 \frac{(\Sigma O) \times t}{A} \text{ cm}$$

$$\Sigma O = 0 + 10 + 35 + 75 + 55 + 45 + 20 + 10 + 0 = 250 \text{ cumecs}$$

$$t = \text{time interval between ordinates, in hours,} = 12 \text{ hours}$$

$$A = \text{catchment area in sq. km} = 450 \text{ sq. km}$$

$$\therefore \text{Direct runoff } n = \frac{0.36 \times 250 \times 12}{450} = 2.4 \text{ cm}$$

Hence the ordinates of 6-hour unit hydrograph are obtained by dividing the ordinates of direct runoff by $n = 2.4$, as shown in the Table above.

Example 4.61. The ordinates of 8-hour unit hydrograph for a drainage basin are given below.

Time in hours	Ordinates of 8-hr unit hydrograph	Time in hours	Ordinate of 8-hr unit hydrograph	Time in hours	Ordinate of 8-hr unit hydrograph
0	0.0	32	231.0	64	14.0
4	5.5	36	165.0	68	9.5
8	13.5	40	112.0	72	6.6
12	26.5	44	79.0	76	4.0
16	45.0	48	57.0	80	2.0
20	82.0	52	42.0	84	1.0
24	162.0	56	31.0	88	0
28	240.0	60	22.0		

Obtain 24-hour unit hydrograph by tabulation method

(Engg. Services Exam. 1988)

Solution

When 3 unit hydrographs, each of 8 hour duration, are added together placed at 8 hour lag successively from one another, then we will get the ordinates of a 24

hr. surface runoff hydrograph, containing 3 cm of direct runoff. Hence the ordinates of a 24-hour unit hydrograph are obtained by dividing each of the above ordinates by 3. The computations are done in a tabular form below.

Time (hours)	1st 8-hr unit hydrograph	2nd 8-hr unit hydrograph	3rd 8-hr unit hydrograph	Total 24 hr hydrograph of 3 cm runoff	Ordinates of 24-h unit hydrograph
(1)	(2)	(3)	(4)	(5) = (2) + (3) + (4)	(6) = (5)/3
0	0.0			0	0
4	5.5			5.5	1.8
8	13.5	0.0		13.5	4.5
12	26.5	5.5		32.0	10.7
16	45.0	13.5	0	58.5	19.5
20	82.0	26.5	5.5	114.0	38.0
24	162.0	45.0	13.5	220.5	73.5
28	240.0	82.0	26.5	348.5	116.2
32	231.0	162.0	45.0	438	146.0
36	165.0	240.0	82.0	487	162.3
40	112.0	231.0	162.0	505	168.3
44	79.0	165.0	240.0	484	161.3
48	57.0	112.0	231.0	400	133.3
52	42.0	79.0	165.0	286	95.3
56	31.0	57.0	112.0	200	66.7
60	22.0	42.0	79.0	143	47.7
64	14.0	31.0	57.0	102	34.0
68	9.5	22.0	42.0	73.5	24.5
72	6.6	14.0	31.0	51.6	17.2
76	4.0	9.5	22.0	35.5	11.8
80	2.0	6.6	14.0	22.6	7.5
84	1.0	4.0	9.5	14.5	4.8
88	0.0	2.0	6.6	8.6	2.9
92		1.0	4.0	5.0	1.7
96		0.0	2.0	2.0	0.7
100			1.0	1.0	0.3
104			0.0	0.0	0.0

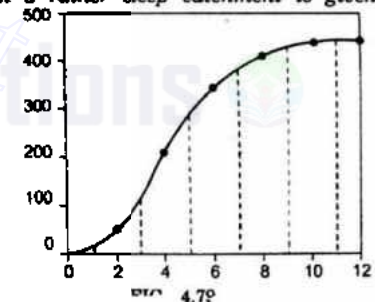
Example 4.62. A 2 hr. unit hydrograph in a rather steep catchment is given below :

Time hr	0	2	4	6	8	10	12
Discharge $100 \text{ m}^3/\text{s}$	0	0.54	1.75	1.27	0.58	0.25	0

Compute the 1 hr. unit hydrograph for the catchment

(Engg. services Exam. 1989)

Solution : For the derivation of 1 hr ($= t_0$) unit hydrograph from the unit hydrograph of $T_0 (= 2 \text{ hr})$ unit duration, we will follow the method of S-curve discussed in §4.24. Note that



ordinates of S-curve, obtained in col.(4) are at 2 hours ($=T_0$) interval. In order to obtain the ordinate of curve at 1 hours ($=t_0$) interval,

S-curves is plotted and the its offsets are read off at 1 hours interval and entered in column (4). The S-curve is shown plotted in Fig. 4.79.

Time (in hr)	Ordinate of 2 hr unit hydrograph (cumecs)	Offset ordinate (cumecs)	Ordinate of S-curve (cumecs)	Ordinate of offset S-curve (cumecs)	Δy (cumecs)	Ordinate of 1 hr unit hydrograph $0 = \Delta y \times \frac{T_0}{t_0} = 2 \Delta y$
(1)	(2)	(3)	(4)	(5)	(6) = (4) - (5)	(7)
0	0		0	-	0	0
1			25	0	25	50
2	54	0	54	25	29	58
3			120	54	66	132
4	175	54	229	120	109	218 (peak)
5			300	229	71	142
6	127	229	356	300	56	112
7			390	356	34	68
8	58	356	414	390	24	48
9			430	414	16	32
10	25	414	439	430	9	18
11			439	439	0	0
12	0	439	439	439	0	0

Example 4.63. The regression analysis of a 30-year flood data at a point on a river yielded sample mean $\bar{x} = 1200 \text{ m}^3/\text{s}$ and standard deviation $s_x = 650 \text{ m}^3/\text{sec}$. For what discharge would you design the structure at this point to 96% assurance that the structure would not fail in the next 50 years? Use Gumbel's method. The value of mean and standard deviation of the reduced variate for $n = 30$ are 0.53622 and 1.11238 respectively.

(Engg. Services Exam. 2000)

Solution

Given : Assurance = 96%, are hence risk $R_{50} = 100 - 96 = 4\% = 0.04$

$$\sigma_{n-1} = s_x = 650 \text{ m}^3/\text{sec}$$

But from Eq. 4.92, $R_{50} = 1 - \left(1 - \frac{1}{T}\right)^n$, where $n = 50$ years

$$\therefore 0.04 = 1 - \left(1 - \frac{1}{T}\right)^{50} \text{ or } 1 - \frac{1}{T} = (0.96)^{1/50}$$

From which we get $T \approx 975.3$ years

For $T = 975.3$ years, the reduced variate y_T is given by

$$y_T = -[\ln \cdot \ln (975.3/974.3)] = 6.88184$$

$$K_T = \frac{y_T - \bar{y}_n}{S_n} = \frac{6.88184 - 0.53622}{1.11238} = 5.70454$$

HYDROLOGY

Hence the design flood $x_T = \bar{x} + K_T \sigma_{n-1}$

or

$$x_T = 1200 + 5.70454 \times 650 \approx 4908 \text{ m}^3/\text{sec}$$

Example 4.64. Flood frequency computations for a flash river at a point 50 km upstream of a bund site indicated the following:

Return period; T (years)	50	100
Peak flood; m^3/sec	20800	22150

Estimate the flood magnitude in the river with a return period of 500 years through use of Gumbel's method.

(Engg. Services Exam. 2003)

Solution :

This problem is similar to the one solved at Example 4.63.

From Gumbel's equation (Eq. 4.91).

$$(K_{100} - K_{50}) \sigma_{n-1} = x_{100} - x_{50} = 22150 - 20800 = 1350 \text{ m}^3/\text{s} \quad \dots(1)$$

Now $K_T = \frac{y_T - \bar{y}_n}{S_n} = \frac{y_T}{S_n} - \frac{\bar{y}_n}{S_n}$ where $\bar{y}_n$ and S_n are constants for a given series.

$$\therefore \frac{y_{100} - y_{50}}{S_n} = (K_{100} - K_{50}) = \frac{1350}{\sigma_{n-1}}$$

$$\text{or } (y_{100} - y_{50}) \frac{\sigma_{n-1}}{S_n} = 1350 \quad \dots(2)$$

$$\text{But } y_{100} = -[\ln \cdot \ln (100/99)] = 4.60015 \text{ and } y_{50} = -[\ln \cdot \ln (50/49)] = 3.90194$$

$$\therefore \frac{\sigma_{n-1}}{S_n} = \frac{1350}{4.60015 - 3.90194} \approx 2220 \quad \dots(3)$$

$$\text{For } T = 500 \text{ years, } y_{500} = -[\ln \cdot \ln (500/499)] = 6.21361$$

$$\text{Now } (y_{500} - y_{100}) \frac{\sigma_{n-1}}{S_n} = x_{500} - x_{100}$$

$$\text{or } (6.21361 - 4.60015) \times 2220 = x_{500} - 22150$$

$$\text{From which } x_{500} \approx 25732 \text{ m}^3/\text{s}$$

Example 4.65. In a frequency analysis of rainfall, based on 15 years of data of 10 minutes storm, the following values were obtained

Arithmetic mean of data = 1.65 cm

Standard deviation = 0.45 cm

Find, using Gumbel's extreme distribution, the recurrence interval of a storm of 10 minutes duration and a depth equal to 3 cm. (Civil Services Exam. 1996)

Solution Given : $\bar{x} = 1.65 \text{ cm}$; $\sigma_x = 0.45 \text{ cm}$ and $x = 3 \text{ cm}$

$$\text{From Eq. 4.88, } y = \frac{1.2825(x - \bar{x})}{\sigma_x} + 0.577$$

$$\text{or } y = \frac{1.2825(3 - 1.65)}{0.45} + 0.577 = 4.4245$$

From Eq. 4.89,

$$\ln \cdot \ln \frac{T}{T-1} = -y_T = -4.4245$$

$$\ln \frac{T}{T-1} = 0.01198 \quad \text{or} \quad \frac{T}{T-1} = 1.01205$$

From which $T = 83.8$ years

Example 4.86. A large sample of peak floods was available for a river. Flood frequency computations using Gumbel's method yield the following results :

Return Period (years)	Peak flood m^3/sec
50	30800
100	36300

Estimate the magnitude of a flood for this river with a return period of 200 years.

(Engg. Services Exam 1997)

Solution : This problem is similar to the one solved in Example 4.33. From Gumbel's equation (Eq. 4.91) :

$$(K_{100} - K_{50}) \sigma_{n-1} = x_{100} - x_{50} = 36300 - 30800 = 5500 \quad \dots(1)$$

$$\text{Now } K_T = \frac{y_T - \bar{y}_n}{S_n} = \frac{y_T}{S_n} - \frac{\bar{y}_n}{S_n} \quad \text{where } \bar{y}_n \text{ and } S_n \text{ are constants for a series.}$$

$$\therefore \frac{(y_{100} - y_{50})}{S_n} = (K_{100} - K_{50}) = \frac{5500}{\sigma_{n-1}} \quad \dots(2)$$

$$\text{or } (y_{100} - y_{50}) \frac{\sigma_{n-1}}{S_n} = 5500$$

$$\text{But } y_{100} = -[\ln \cdot \ln (100/99)] = 4.60015$$

$$\text{and } y_{50} = -[\ln \cdot \ln (50/49)] = 3.90194$$

$$\therefore \frac{\sigma_{n-1}}{S_n} = \frac{5500}{y_{100} - y_{50}} = \frac{5500}{4.60015 - 3.90194} = 7877.3 \quad \dots(3)$$

$$\text{For } T = 200 \text{ years, } y_{200} = -[\ln \cdot \ln (200/199)] = 5.29581$$

$$\text{Now } (y_{200} - y_{100}) \frac{\sigma_{n-1}}{S_n} = x_{200} - x_{100}$$

$$\text{or } (5.29581 - 4.60015) \times 7877.3 = x_{200} - 36300$$

$$\text{From which } x_{200} \triangleq 41780 m^3/sec$$

Example 4.87. For a data of maximum recorded flood of a river, the mean and standard deviations are $4200 m^3/s$ and $1705 m^3/s$ respectively. Using Gumbel's extreme value distribution, estimate the return period of a design flood of $9950 m^3/s$. Assume an infinite sample size.

Solution : For an infinite sample size (i.e. infinite series)

$$x_T = \bar{x} + K \sigma_x \quad \dots(4.90)$$

$$\text{For the present case, } \bar{x} = 4200 m^3/s; \sigma_x = 1705 m^3/s \text{ and } x_T = 9950 m^3/s$$

$$9550 = 4200 + K \times 1705$$

$$\text{From which } K = 3.13783$$

Hence from Eq. 4.90 (a)

$$y_T = 1.2825 K + 0.577 = 1.2825 \times 3.13783 + 0.577 = 4.6013$$

But

$$y_T = -\left[\ln \cdot \ln \frac{T}{T-1} \right] \text{ from Eq. 4.89.}$$

$$\therefore \ln \cdot \ln \frac{T}{T-1} = -y_T = -4.6013$$

or

$$\ln \frac{T}{T-1} = e^{-4.6013} = 0.01004 \quad \text{or} \quad \frac{T}{T-1} = 1.01009$$

$$\text{which gives } T \triangleq 101 \text{ years}$$

Example 4.88. For a river, the estimated flood peaks for two return periods by the use of Gumbel's method are given below:

Return period (years)	Peak flood (m^3/s)
100	485
50	445

What flood discharge in this river will have a return period of 1000 years. (Civil Services Exam. 1990)

Solution :

This problem is similar to the one solved at Example 4.33.

From Gumbel's Equation (Eq. 4.91).

$$(K_{100} - K_{50}) \sigma_{n-1} = x_{100} - x_{50} = 485 - 445 = 40 m^3/s \quad \dots(1)$$

$$\text{Now } K_T = \frac{y_T - \bar{y}_n}{S_n} = \frac{y_T}{S_n} - \frac{\bar{y}_n}{S_n}$$

where $\bar{y}_n$ and S_n are constant for a given series

$$\therefore \frac{(y_{100} - y_{50})}{S_n} = (K_{100} - K_{50}) = \frac{40}{\sigma_{n-1}}$$

$$\text{or } (y_{100} - y_{50}) \frac{\sigma_{n-1}}{S_n} = 40 \quad \dots(2)$$

$$\text{But } y_{100} = -[\ln \cdot \ln (100/99)] = 4.60015 \text{ and } y_{50} = -[\ln \cdot \ln (50/49)] = 3.90194$$

$$\therefore \frac{\sigma_{n-1}}{S_n} = \frac{40}{4.60015 - 3.90194} = 57.29 \quad \dots(3)$$

$$\text{For } T = 1000 \text{ years, } y_{1000} = -[\ln \cdot \ln (1000/999)] = 6.90726$$

$$\text{Now } (y_{1000} - y_{100}) \frac{\sigma_{n-1}}{S_n} = x_{1000} - x_{100}$$

$$\text{or } (6.90726 - 4.60015) \times 57.29 = x_{1000} - 485$$

$$\text{From which } x_{1000} \triangleq 617.2 m^3/s$$

Example 4.89. A flood of a certain magnitude has a return period of 25 years.
(a) what is its probability of exceedance? (b) what is the probability that this flood may occur in the next 12 years. (Engg. Services Exam. 1983)

Solution : Given : $T = 25$ years.

(a) Probability of exceedance, $P = \frac{1}{T} = \frac{1}{25} = 0.04$

(b) Probability of non-exceedance in next n successive years is

$$y_{0..n} = q^n = (1 - P)^n = (1 - 0.04)^{12} = 0.6127$$

$\therefore$ Probability of occurrence at least once in next 12 years is

$$R_{12} = 1 - (1 - P)^n = 1 - 0.6127 = 0.3873$$

PROBLEMS

1. Explain with the help of a diagram the hydrologic cycle.
2. What do you understand by precipitation? Explain various types of precipitation.
3. Explain any one type of automatic rain gauge.
4. Describe various methods of computing average rainfall over a basin.
5. What is run-off? What are the factors that affect the run-off from a catchment area?
6. What are the methods of computing run-off from a catchment area? Give various formulae stating clearly the area for which each is applicable.
7. Explain various methods of determining flood discharge in a stream.
8. Give various flood discharge formulae applicable for Indian catchments.
9. What is a hydrograph? Draw a single peaked hydrograph and explain its components.
10. Explain the method of determining direct run-off from a given storm hydrograph.
11. What do you understand by unit hydrograph? How is it derived? Explain its use in construction of flood hydrograph resulting from two or more periods of rainfall.
12. Find out the ordinates of a storm hydrograph resulting from a 3 hour storm with rainfall of 3, 4.5 and 1.5 cm during subsequent 3 hours intervals. The ordinates of unit hydrograph are given in the Table below :

Hours	0	03	06	09	12	15	18	21	24	03	6	09	12
Ordinates of unit hydrograph (cumecs).	0	80	200	350	450	350	260	190	130	80	45	20	0

Assume an initial loss of 5 mm, infiltration index of 5mm/hour and base flow of 20 cumecs.

Ans. 20, 110, 490, 970, 1520, 1720, 1330, 990, 720, 490, 306, 175, 80 and 20.

13. What do you understand by infiltration index? How do you determine it?

Ground Water : Well Irrigation

5.1. INTRODUCTION

Ground water hydrology is the science of occurrence, distribution and movement of water below the surface of earth. The largest available source of fresh water lies underground. The total ground water potential is estimated to be one third the capacity of oceans.

The main source of ground water is precipitation. A portion of rain falling on the earth's surface infiltrates into ground, travels down and, when checked by impervious layer to travel further down, forms ground water. The ground water reservoir consists of water held in voids within a geologic stratum. Other sources of ground water include water from deep in the earth which is carried upward in intrusive rocks and water which is trapped in sedimentary rocks during their formation. The quantities of such water are small and they are often so highly mineralised as to be unsuited for use. Water bearing formations of the earth's crust act as conduits for transmission and as reservoirs of storage of ground water.

The discharge from ground water occurs in two ways : (1) natural way, (2) artificial way. The natural discharge occurs as flow in lakes, reservoirs, rivers, oceans and springs. Pumpage from wells constitutes the major artificial discharge from ground water.

5.2. SOME DEFINITIONS

Aquifer. Aquifers are the permeable formations having structures which permit appreciable quantity of water to move through them under ordinary field conditions. Thus these are the geologic formations in which ground water occurs (i.e., sands and gravels).

Aquiclude. Aquicludes are the impermeable formations which contain water but are not capable of transmitting and supplying a significant quantity (e.g., clays).

Aquifuge. Aquifuge is an impermeable formation which neither contains water nor transmits any water.

Probably, 90 percent of all developed aquifers consist of unconsolidated rocks, chiefly gravel and sand. Sands are composed of macroscopic particles that are rounded (bulky) or angular in shape. They drain easily, do not swell, possess insignificant capillary potential and when dry exhibit no shrinkage. Clays on the other hand, are composed of microscopic particles of plate like shape. They are highly impervious, exhibit considerable

Example 4.88. A flood of a certain magnitude has a return period of 25 years.
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Solution : Given : $T = 25$ years.

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$\therefore$ Probability of occurrence at least once in next 12 years is

$$R_{ab} = 1 - (1 - P)^n = 1 - 0.6127 = 0.3873$$

PROBLEMS

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Assume an initial loss of 6 mm, infiltration index of 5mm/hour and base flow of 20 cumecs.

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swelling, possess a high capillary potential and demonstrate considerable volume reduction upon drying.

Porosity. Porosity (n) is defined as the ratio of the volume of openings or pores (or voids) V_v in the material to its total volume V and is expressed as percentage:

$$n = \frac{V_v}{V} \times 100$$

For a cubical array of spheres [Fig. 5.1 (a)]

$$V = d^3, V_s = \frac{\pi}{6} d^3$$

$$V_v = V - V_s = d^3 \left(1 - \frac{\pi}{6}\right)$$

$$\therefore n = \frac{d^3 \left(1 - \frac{\pi}{6}\right)}{d^3} \times 100 = 47.6\% \quad \dots(11)$$

For a rhombohedral packing [Fig. 5.1 (b)], it can be shown that

$$n = \left(1 - \frac{\sqrt{2}}{6} \pi\right) 100 = 26\%$$

Values of the porosity of some soils in their natural form are given below : (Terzaghi and Pack) :

Soil	Porosity (%)
Uniform sand, loose	46
Uniform sand, dense	34
Glacial till, very mixed-grained	20
Soft glacial clay	56
Stiff glacial clay	37
Soft very organic clay	75
Soft bentonite	84

Specific Yield. The capacity of a formation to contain water is measured by porosity. However, a high porosity does not indicate that an aquifer will yield large volumes of water to a well. The only water which can be obtained from the aquifer is that which will flow by gravity. The specific yield of an aquifer is defined as the ratio expressed as a percentage, of the volume of water which after being saturated, can be drained by gravity to its own volume.

Thus, Specific yield = $\frac{\text{Volume of water drained by gravity}}{\text{Total volume}}$

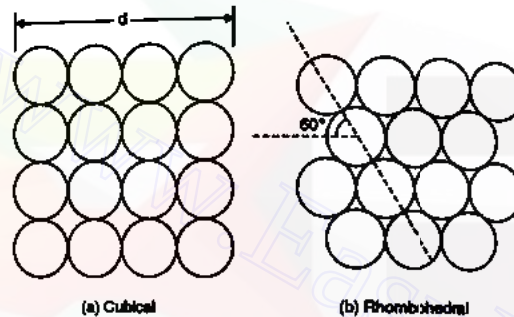


FIG. 5.1.

or

$$S_y = \frac{w_y}{V} \times 100$$

where w_y = volume of water drained by gravity.

Specific yield is an indication of water yielding capacity of an unconfined aquifer.

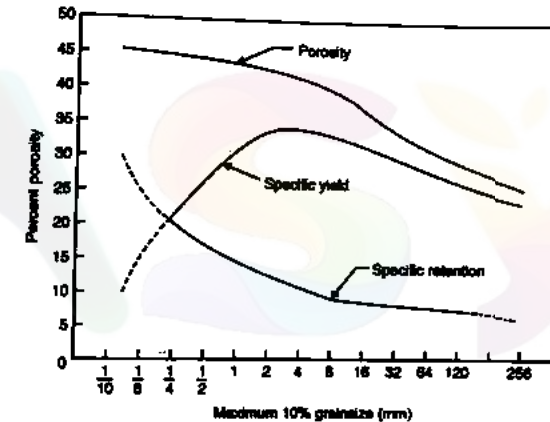


FIG. 5.2 POROSITY, SPECIFIC YIELD AND SPECIFIC RETENTION VARIATION WITH GRAIN SIZE (AFTER ECKIS).

Specific Retention. Specific yield is always less than porosity since some water will be retained in the aquifer by molecular and surface tension forces. The specific retention (S_r) of an aquifer is the ratio, expressed as a percentage, of the volume of water it will retain after saturation against the force of gravity to its own volume. Thus,

$$\text{Specific retention } S_r = \frac{w_r}{V} \times 100$$

where w_r = volume of water retained.

$$\text{Porosity } n = \frac{V_v}{V} \times 100 = \frac{w}{V} \times 100$$

where w = volume of water = V_v in a saturated aquifer = $w_y + w_r$.

$$\text{Hence } n = S_y + S_r.$$

The value of specific yield depends upon grain size, shape and distribution of pores, and compaction of stratum. Fig. 5.2 shows the variations of porosity, specific yield and specific retention of soil with grain size.

5.2. DIVISIONS OF SUB-SURFACE WATER

The sub-surface water, as indicated in Fig. 5.3 can be divided into following zones:

(a) Zone of Aeration

Consisting of :

- (i) Soil water zone : Soil water.
- (ii) Intermediate zone : Pellicular and gravitational water.
- (iii) Capillary zone : Capillary water.

(b) Zone of Saturation

Ground water fills all the interstices in the saturated zone.

Water Table

The saturated zone is bounded at the top by either a limiting surface of saturation or overlying (confining) impermeable strata, and extends down to underlying impermeable strata. In the absence of the confining impermeable layer, the static level of water in wells penetrating the zone of saturation is called the water table. The water table is the surface of a water body which is constantly adjusting itself towards an equilibrium condition. If there were no recharge to or outflow from the ground water in basin, the water table would eventually become horizontal.

5.4. TYPES OF AQUIFERS

Aquifers are mainly of two types:

1. Unconfined aquifer.
2. Confined aquifer (artesian aquifer).

Unconfined Aquifer

Unconfined aquifer, or water table aquifer is the one in which a water table serves as the upper surface of the zone of saturation. It is also sometimes known as the free, phreatic or non-artesian aquifer. In such an aquifer, the water table varies in undulating form and in slope. Rises and falls in the water table corresponds to changes in the volume of water in storage within unconfined aquifer.

Confined Aquifer or Artesian Aquifer

Confined aquifer or artesian aquifer is the one in which ground water is confined under pressure greater than atmospheric by over-lying, relatively impermeable strata. Artesian aquifers are analogous to pipelines. The static pressure at a point within the artesian aquifer is equivalent to the elevation of the water table in the recharge

FIG. 5.3 DIVISIONS OF SUB-SURFACE WATER.

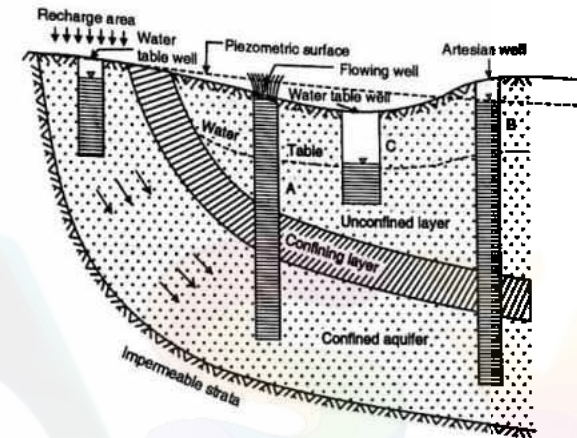
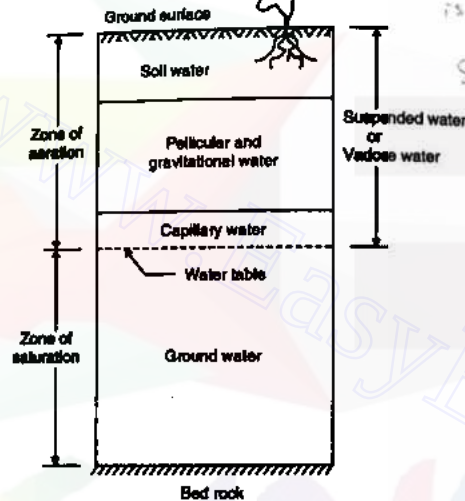


FIG. 5.4 CONFINED AND UNCONFINED AQUIFERS.

area less the loss in head through the aquifer to the point under consideration. In a well penetrating such an aquifer, the water level will rise to the level of the local static pressure or artesian head. Water enters a confined aquifer in an area where the confining bed rises to the surface or ends underground and the aquifer becomes unconfined. Artesian aquifers usually have relatively small recharge areas as compared with unconfined aquifers. When water is withdrawn from an artesian well, a local depression of the piezometric surface results. This decrease in pressure permits a slight expansion of the water and in some cases a compaction of the aquifer.

Flowing Well and Artesian Well

When a well penetrates a confined aquifer, water rises in the well to the level of local static pressure or artesian head. If this artesian pressure is sufficient to raise the water above the ground level, a flowing well occurs, such as well A in Fig. 5.4. If, however, the water level in such a well is below the ground level, but is above the local water table it is known as the artesian well such as well B in Fig. 5.4.

Perched Aquifer

Perched aquifer (Fig. 5.5) is a special type of unconfined aquifer, and occurs where a ground water body is separated from the main ground water by a relatively imper-

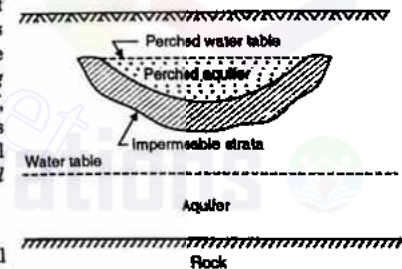


FIG. 5.5 PERCHED AQUIFER.

meable stratum of small aerial extent and by the zone of aeration above the main body of ground water.

5.5. STORAGE COEFFICIENT

The water yielding capacity of a confined aquifer can be expressed in terms of its storage coefficient. Storage coefficient is defined as the volume of water that an aquifer releases from or takes into storage per unit surface area of aquifer per unit change in the component of head normal to that surface.

Let us consider a vertical column of unit area (one metre $\times$ one metre) extending through a confined aquifer (Fig. 5.6). Then, the storage coefficient, S is the volume of water, in cubic metres, released from the aquifer when the piezometric surface declines by one metre. In most of the confined aquifers, the value of storage coefficient ranges between 0.00005 to 0.005. Its value can be determined from pumping out tests on wells penetrating fully into confined aquifer.

In an unconfined aquifer, when the water table is lowered by one metre, the water from one metre height of the vertical column of unit area drains freely under gravity. Thus, storage coefficient for an unconfined aquifer corresponds to its specific yield.

Coefficient of Permeability (k)

The coefficient of permeability is defined as the velocity of flow which will occur through the total cross-sectional area of the soil (or aquifer) under a unit hydraulic gradient. Some typical values of the coefficient of permeability are given in Table 5.1.

TABLE 5.1

Soil type	Coefficient of permeability (cm/sec)
Clean gravel	1.0 and greater
Clean sand (coarse)	1.0-0.01
Sand (medium)	0.01-0.005
Fine sand	0.05-0.001
Silty sand	0.002-0.0001
Silt	0.0005-0.00001
Clay	0.000001 and smaller

Coefficient of Transmissibility (T)

Coefficient of transmissibility is defined as the rate of flow of water (in m^3/day or gallons/day) through vertical strip of aquifer of unit width (1 m or 1 ft) and extending the full saturation height under unit hydraulic gradient, at a temperature of 60° F.

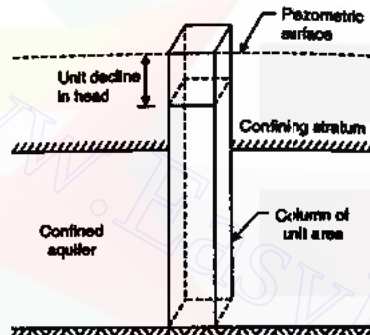


FIG. 5.6 STORAGE COEFFICIENT.

Thus, the coefficient of transmissibility T equals to the field coefficient of permeability multiplied by the aquifer thickness (B) :

$$T = Bk$$

5.6. WELL HYDRAULICS

Darcy's Law. The percolation of water through soil was first studied by Darcy (1856) who demonstrated experimentally that for laminar flow conditions in a saturated soil, the rate of flow, or the discharge per unit time is proportional to the hydraulic gradient, and it could be expressed as follows :

$$Q = k i A \quad \dots(5.1)$$

or

$$v = \frac{Q}{A} = k i \quad \dots(5.2)$$

where Q = rate of flow

i = hydraulic gradient

k = Darcy's coefficient of permeability

A = total cross-sectional area of soil mass perpendicular to the direction of flow

v = flow velocity.

Eq. 5.2 demonstrates the linear dependency between the hydraulic gradient and the discharge velocity. However, it in no way describes the state of affairs within an individual pore. Strictly speaking, Darcy's law represents the statistical macroscopic equivalent of the Navier-Stokes equation of motion for the viscous flow of ground water.

Darcy's law is valid only for laminar flow. Because of very small pore dimensions in fine grained soils, a laminar flow should exist, but in course grained soils turbulent flow may be expected under certain conditions. It has been borne out by experiments that the limits of validity of Darcy's law may be fixed with respect to particle size, velocity of flow and hydraulic gradient. Fancher, Lewis and Barnes demonstrated that flow through sands remains laminar and the Darcy's law valid so long as the Reynold's number expressed in the form below is equal to or less than unity :

$$\frac{\rho v d}{\mu} \leq 1 \quad \dots(5.3)$$

where ρ = mass density

μ = dynamic viscosity

d = diameter or particle size

v = velocity of flow

For the ground water flow occurring in nature, the law is generally within its validity limits. But in rock aquifers, in unconsolidated aquifers with steep hydraulic gradients, or in those containing large diameter solution openings, Darcy's law may not be applicable. Also, the flow in the immediate vicinity of wells have steep hydraulic gradient and the Darcy's law is not applicable in the immediate vicinity of the well.

STEADY RADIAL FLOW TO A WELL : DUPUIT'S THEORY

When a well is penetrated into an extensive homogeneous aquifer, the water table initially remains horizontal in the well. When the well is pumped, water is removed

from the aquifer and the water table or piezometric surface, depending upon the type of the aquifer, is lowered resulting in a circular depression in the water table or the piezometric surface. This depression is called the *cone of depression* or the *drawdown curve*. At any point, away from the well, the *drawdown* is the vertical distance by which the water table or the piezometric surface is lowered. The analysis of such radial flow towards a well was originally proposed by Dupuit in 1863 and later modified by Thiem (1906). For the sake of analysis, we shall take two cases :

(1) well in unconfined aquifer, and (2) well fully penetrating a confined aquifer.

1. Unconfined Aquifer.

Fig. 5.7 shows a well penetrating an unconfined or free aquifer to its full depth.

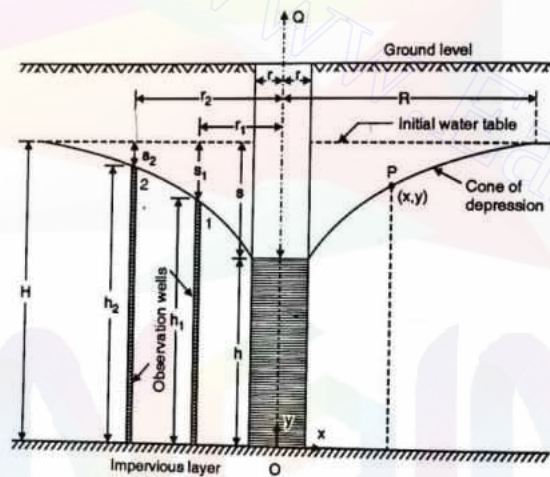


FIG. 5.7 UNCONFINED AQUIFER

Let r = radius of the well
 H = thickness of the aquifer, measured from the impermeable layer to the initial level of water table.
 s = drawdown at the well
 h = depth of water in the well measured above impermeable layer.
 Considering the origin of co-ordinates at a point O at the centre of the well at its bottom, let the co-ordinates of any point P on the drawdown curve be (x, y) . Then, from Darcy's law

$$Q = k A_x i_x$$

where A_x = area of cross-section of the saturated part of aquifer at P
 $= (2\pi x) \times (y) = 2\pi xy$

$$i_x = \text{hydraulic gradient at } P = \frac{dy}{dx}$$

Hence $Q = k(2\pi xy) \frac{dy}{dx}$

$$Q \frac{dx}{x} = 2\pi k y dy.$$

Integrating between the limits (R, r) for x and (H, h) for y we get

$$Q \int_r^R \frac{dx}{x} = 2\pi k \int_h^H y dy$$

$$Q [\log_e x]_r^R = 2\pi k \left[\frac{y^2}{2} \right]_h^H$$

$$\text{From which } Q = \frac{\pi k (H^2 - h^2)}{\log_e \frac{R}{r}} = \frac{1.36 k (H^2 - h^2)}{\log_{10} \frac{R}{r}} \quad \dots(5.4)$$

If k (coefficient of permeability) is expressed in cubic metre per day per square metre ($\text{m}^3/\text{day}/\text{m}^2$) of the area of sub-soil, the above expression for discharge will directly be in cubic metre per day (m^3/day) units. If, however, k is in gallons per day per sq. foot of area of sub-soil, the discharge will be in gallons per day.

In the above expression, R , commonly known as *radius of zero drawdown*, is the radius measured from the centre of the well to a point where the drawdown curve meets the original water table tangentially. In practice, the selection of the radius of influence R is approximate and arbitrary but the variation in Q is small for a wide range of R . Suggested values of R fall in the range of 150 to 300 metres.

Alternatively, R may be computed from the following approximate expression given by Sichardt :

$$R = 3000 s \sqrt{k}$$

where R and s are in metres, and k is in m/sec .

If there are two observation wells at radial distance r_1 and r_2 ($r_2 > r_1$) and if the depths of water in them are h_1 and h_2 respectively, Eq. 5.4 can also be expressed in the form :

$$Q = \frac{\pi k (h_2^2 - h_1^2)}{\log_e \frac{r_2}{r_1}} \quad \dots[5.4(a)]$$

$$\text{or } Q = \frac{1.36 k (h_2^2 - h_1^2)}{\log_{10} \frac{r_2}{r_1}} \quad \dots[5.4(b)]$$

If the drawdown (s) is measured at the well face, we have
 $s = H - h$

and

$$H = s + h \text{ or } H + h = (s + 2h)$$

Then, from Eq. 5.4,

$$Q = \frac{\pi k (H - h) (H + h)}{\log_e \frac{R}{r}} = \frac{\pi k s (s + 2h)}{\log_e \frac{R}{r}}$$

$$Q = \frac{\pi k s (s + 2L)}{\log_e \frac{R}{r}} = \frac{2\pi k s (L + s/2)}{\log_e \frac{R}{r}}$$

where $h = L = \text{Length of the strainer}$

$$Q = \frac{2.72 k s (L + s/2)}{\log_{10} \frac{R}{r}} \quad \dots (5.5)$$

Assumptions and Limitations of Dupuit's Theory

Dupuit's theory of flow for unconfined aquifer is based on the following assumptions :

1. The velocity of flow is proportional to the tangent of the hydraulic gradient instead of sine.
2. The flow is horizontal and uniform everywhere in the vertical section.
3. Aquifer is homogeneous, isotropic and of infinite aerial extent.
4. The well penetrates and receives water from the entire thickness of the aquifer.
5. The coefficient of transmissibility is constant at all places and at all times.
6. Natural ground water regime affecting an aquifer remains constant with time.
7. Flow is laminar and Darcy's law is applicable.

Out of these, assumptions (1), (2) and (7) are of particular importance. The flow is not horizontal, especially near the well. Also, the piezometric surface attains greater slope as it approaches the well boundary, with the result that assumption 1 is an approximation. Due to these reasons, the parabolic form of piezometric surface computed from the Dupuit's theory deviates from the observed surface. This deviation is large at the well face, resulting in the formation of seepage face. In addition to these, the velocity near the well increases and the flow no longer remains laminar. Thus, Darcy's law equation is not valid near the well face.

2. Confined Aquifer.

Fig. 5.8 shows a well fully penetrating a confined or artesian aquifer. Let (x, y) be the co-ordinates, of any point P on the drawdown curve, measured with respect to the origin O . Then, from Darcy's law, flow crossing a vertical plane through P is given by

$$Q = k i_x A_x$$

where $A_x = \text{cross-sectional area of flow, measured at } P = 2\pi x b$
 $b = \text{thickness of confined aquifer}$

$i_x = \text{hydraulic gradient at } P = \frac{dy}{dx}$

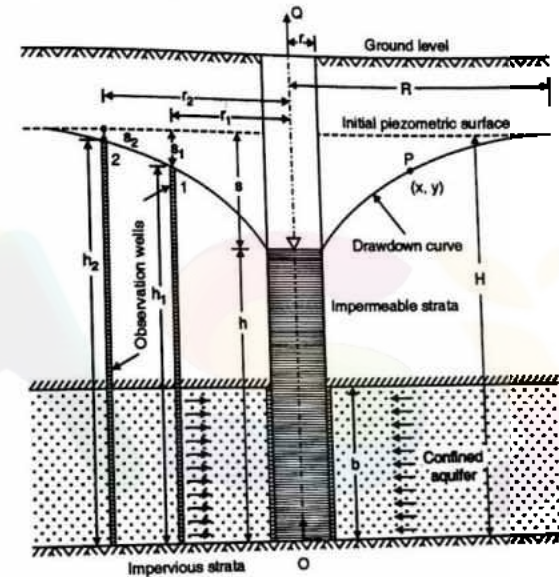


FIG. 5.8 CONFINED AQUIFER.

$$Q = k \left(\frac{dy}{dx} \right) (2\pi x b)$$

$$Q \frac{dx}{x} = 2\pi k b dy$$

Integrating between the limits (R, r) for x and (H, h) for y , we get

$$Q \int_r^R \frac{dx}{x} = 2\pi k b \int_h^H dy$$

$$Q [\log_e x]_r^R = 2\pi k b [y]_h^H$$

$$Q = \frac{2\pi k b (H - h)}{\log_e \frac{R}{r}} = \frac{2.72 b k (H - h)}{\log_{10} \frac{R}{r}} = \frac{2\pi b k s}{\log_e \frac{R}{r}} = \frac{2.72 b k s}{\log_{10} \frac{R}{r}} \quad \dots [5.6(a)]$$

From which

$$Q = \frac{2.72 T \cdot s}{\log_{10} \frac{R}{r}} \quad \dots [5.6(b)]$$

or
 where $T = \text{coefficient of transmissibility} = bk$

s = drawdown at the well

Eq. 5.6(a) is known as the *equilibrium equation* or *Thiem equation*.

If h_1 and h_2 are the measured depth of water in two observation wells situated radially at distances r_1 and r_2 respectively, the above equations can also be expressed as

$$Q = \frac{2.72 bk (h_2 - h_1)}{\log_{10} \frac{r_2}{r_1}} = \frac{2.72 T (h_2 - h_1)}{\log_{10} \frac{r_2}{r_1}} \quad \dots[5.6(d)]$$

If h_x is the depth at any radial distance x , the discharge can be written as

$$Q = \frac{2 \pi kh (h_x - h)}{\log_e \frac{r_x}{r}}$$

Equating this to Eq. 5.6(a), we get

$$h_x - h = (H - h) \frac{\log_e \frac{r_x}{r}}{\log_e \frac{R}{r}} \quad \dots(5.7)$$

This shows that head varies linearly with the logarithm of distance regardless of rate of discharge.

5.7. DETERMINATION OF AQUIFER CONSTANT T

(a) **Confined aquifer.** The aquifer constant T (coefficient of transmissibility) can be determined by a pumping out test, and by observing drawdown at various observation wells. Pumping must continue at a uniform rate for a sufficient time to approach a steady state condition for which equilibrium Eq. 5.6 is applicable. Steady state condition is the one in which the drawdown changes negligibly with time. Refer Fig. 5.8.

Let s_1 = drawdown in well 1 = $(H - h_1)$

s_2 = drawdown in well 2 = $(H - h_2)$.

$$\therefore h_2 - h_1 = (H - s_2) - (H - s_1) = s_1 - s_2$$

Then from Eq. 5.6 (d),

$$Q = \frac{2.72 T (h_2 - h_1)}{\log_{10} \frac{r_2}{r_1}} = \frac{2.72 T (s_1 - s_2)}{\log_{10} \frac{r_2}{r_1}}$$

$$\therefore T = \frac{Q}{2.72 (s_1 - s_2)} \log_{10} \frac{r_2}{r_1} \quad \dots(5.8)$$

Choosing $r_2 = 10 r_1$, we find $\log_{10} \frac{r_2}{r_1} = 1$.

$$\text{Hence } T = \frac{Q}{2.72 (s_1 - s_2)} = \frac{Q}{2.72 \Delta s} \quad \dots(5.9)$$

where Δs = difference in drawdowns at the two wells so selected that $r_2 = 10 r_1$.

The method, therefore, consists in observing drawdowns $s_1, s_2, \dots, s_x$ at certain observation wells distant $r_1, r_2, \dots, r_x$ etc., and plotting a graph between s_x as ordinate

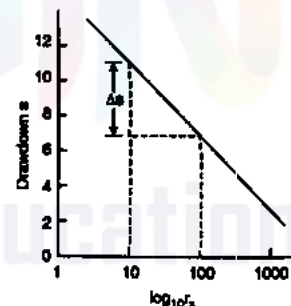


FIG. 5.9

and $\log_{10} r_x$ as abscissa, thus getting a straight line as shown in Fig. 5.9. From the graph, Δs can be obtained for one log cycle of distance and can be substituted in Eq. 5.9 to get T .

(b) **Unconfined Aquifer.** The above method of determination of T can also be extended to unconfined aquifer. Refer Fig. 5.7.

$$h_1 = H - s_1 ; \quad h_2 = H - s_2$$

$$\therefore h_2^2 - h_1^2 = (H - s_2)^2 - (H - s_1)^2$$

$$= 2H \left[\left(s_1 - \frac{s_1^2}{2H} \right) - \left(s_2 - \frac{s_2^2}{2H} \right) \right] = 2H (s_1' - s_2')$$

where s_1' and s_2' are *modified drawdowns* given by

$$s_1' = s_1 - \frac{s_1^2}{2H}$$

$$s_2' = s_2 - \frac{s_2^2}{2H}$$

and

Now from Eq. [5.4(a)],

$$Q = \frac{\pi k (h_2^2 - h_1^2)}{\log_e \frac{r_2}{r_1}} = \frac{1.36 k (h_2^2 - h_1^2)}{\log_{10} \frac{r_2}{r_1}}$$

Substituting the value of $(h_2^2 - h_1^2)$ from above, we get

$$Q = \frac{1.36 k [2H (s_1' - s_2')]}{\log_{10} \frac{r_2}{r_1}} = \frac{2.72 kH (s_1' - s_2')}{\log_{10} \frac{r_2}{r_1}}$$

Since H = aquifer thickness, we have $kH = T$

$$Q = \frac{2.72 T (s_1' - s_2')}{\log_{10} \frac{r_2}{r_1}}$$

...(5.10)

From which

$$T = \frac{Q}{2.72 (s_1' - s_2')} \log_{10} \frac{r_2}{r_1}$$

It should be noted that Eq. 5.8 and 5.10 are identical. Choosing $r_2 = 10 r_1$ as before, we get

$$T = \frac{Q}{2.72 \Delta s'} \quad \dots(5.11)$$

Thus the observed drawdowns are corrected or modified, and a graph is plotted between the modified drawdowns (s') and logarithm of distances of observation wells from the discharge well. Measuring $\Delta s'$ for one log cycle of distance, we get T from Eq. 5.11.

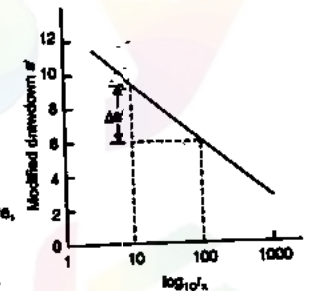


FIG. 5.10

5.8. CHARACTERISTIC WELL LOSSES : SPECIFIC CAPACITY OF WELL

Well Loss. When water is pumped out of a well, the total drawdown caused includes not only that of the logarithmic drawdown curve at the well face, but also drawdown caused by flow through well screen and axial movement within the well. The latter drawdown is called *well loss*. Since turbulent flow generally occurs near the well face, this loss may be taken to be proportional to Q^n .

Rewriting Eq. 5.4(a) and adding well loss CQ^n to it, we get

$$s = (H - h) = \frac{Q}{2\pi kb} \log_e \frac{R}{r} + CQ^n$$

where the constant C is governed by several factors such as well radius, construction and condition of the well. Rewriting the above, we get

$$s = BQ + CQ^n \quad \dots(5.12)$$

where $B = \frac{\log_e \frac{R}{r}}{2\pi kb}$; BQ = aquifer loss and CQ^n = well loss.

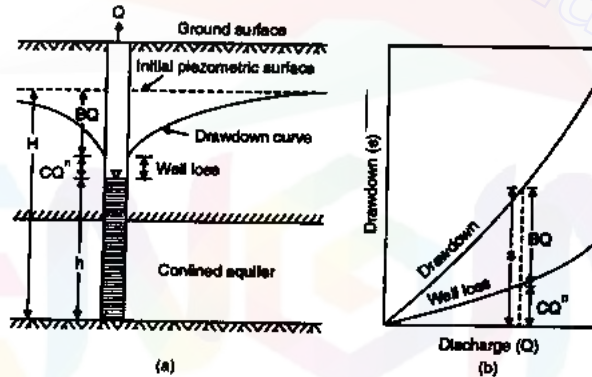


FIG. 5.11. WELL LOSS IN CONFINED AQUIFER.

Fig. 5.11(a) illustrates that the total drawdown consists of the sum of the aquifer loss (BQ) and the well loss (CQ^n). Fig. 5.11(b) shows the variations of drawdown and well loss with the discharge. For a screen which is not clogged or encrusted and whose size is compatible to the surrounding porous media, the portion of the well loss caused by water entering the well is small in comparison with the portion resulting from axial movement inside the well to the pump intake.

Specific Capacity.

The specific capacity of a well is the measure of the effectiveness of the well, and is defined as the yield of the well per unit drawdown. Thus, if s = drawdown and Q = well discharge or the yield.

Then, specific capacity = $\frac{Q}{s}$

For a confined aquifer, from Eq. 5.12, we have $s = BQ + CQ^n$

$$\text{Hence specific capacity} = \frac{Q}{BQ + CQ^n} = \frac{1}{B + CQ^{n-1}} \quad \dots(5.3)$$

This shows that the specific capacity of a well is not constant, but decreases as discharge increases.

5.9. INTERFERENCE AMONG WELLS

When two wells, situated near to each other, are discharging, their drawdown curves intersect within their radius of zero drawdown. Thus, though the total discharge is increased, the discharge in individual well is decreased due to interference.

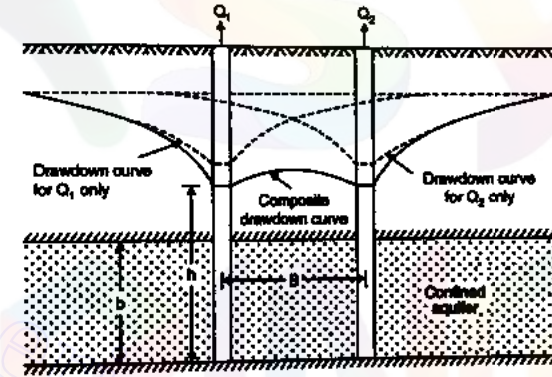


FIG. 5.12 INTERFERENCE BETWEEN TWO WELLS

Fig. 5.12 shows interference between two wells. If the two wells are a distance B apart, and have the same diameter and drawdown and discharge over the same period of time, it can be shown with the help of method of complex variables, that the discharge through each well is given by

$$Q_1 = Q_2 = \frac{2\pi kb(H-h)}{\log_e \frac{R^2}{rB}} \quad \dots(5.14)$$

where R is the radius of area of influence ($R \gg B$).

If there were only one well, then the discharge, under the same drawdown, would have been, from Eq. [5.6(a)],

$$Q = \frac{2\pi kb(H-h)}{\log_e \frac{R}{r}} \quad \dots[5.6(a)]$$

Since $R \gg B, \frac{R^2}{rB} > \frac{R}{r}$
Hence $Q > Q_1$.

Thus, discharge in each well decreases due to the interference will decrease.

Similarly, if there are three wells forming an equilateral triangle a distance B on a side, and if all the three wells have the same characteristics,

$$Q_1 = Q_2 = Q_3 = \frac{2\pi kb(H-h)}{\log_e \frac{R^3}{rB^2}} \quad \dots(5.15)$$

5.10. FULLY PENETRATING ARTESIAN GRAVITY WELL

Sometimes, in an artesian well (i.e., a well in a confined aquifer) high pumping rates may lower the water at the well face to a level below the top confined aquifer, as shown in Fig. 5.13. In such a case, the flow pattern close to the well is similar to that for a gravity well (i.e., a well in unconfined aquifer) whereas at distances farther from the well the flow is artesian. This type of well is known as a *combined artesian gravity well*.

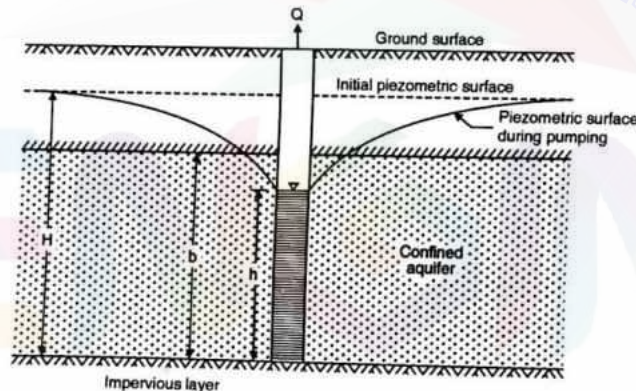


FIG. 5.13 ARTESIAN GRAVITY WELL.

Fig. 5.13 shows an artesian gravity well. The flow from such a well can be computed from the following expression developed by Muskat :

$$Q = \frac{\pi k (2bH - b^2 - h^2)}{\log_e \frac{R}{r}} \quad \dots[5.16(a)]$$

5.11. PARTIALLY PENETRATING ARTESIAN WELL

A partially penetrating artesian well is the one in which the well screen does not penetrate to the full depth of the confined aquifer. The pattern of flow in the

aquifer in the vicinity of such a well deviates from that for a fully penetrating well. In practice, we often encounter such wells that extend only part way through the water bearing strata.

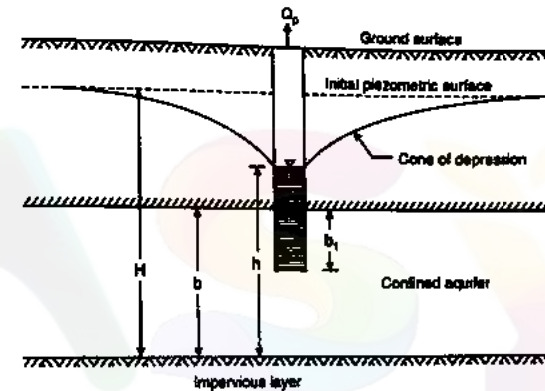


FIG. 5.14. PARTIALLY PENETRATING ARTESIAN WELL.

Fig. 5.14 shows a partially penetrating artesian well in which the strainer length b_1 is less than the aquifer thickness b . The discharge Q_p from such a well can be computed from the following equation :

$$Q_p = \frac{2\pi kb(H-h)}{\log_e \frac{R}{r}} \quad G = Q_p \quad \dots[5.16(b)]$$

where Q_p = discharge for partially penetrating well.

Q = discharge for a fully penetrating well for the same drawdown $(H-h)$.

G = correction factor for partial penetration = $\frac{Q_p}{Q}$.

A reasonable estimation of the correction factor G can be obtained from the following expression developed by Kozeny :

$$G = \frac{b_1}{b} \left(1 + 7 \sqrt{\frac{r}{2b_1}} \cos \frac{\pi b_1}{2b} \right) \quad \dots[5.17]$$

5.12. SPHERICAL FLOW IN A WELL

Fig. 5.15 shows a special case of partially penetrating well where the well just penetrates the top surface of a semi-infinite porous medium. Here $b_1 = 0$ and equation 5.16 does not apply because the flow towards the well is purely spherical. Thus discharge Q_s from such a well can be computed from the expression

$$Q_s = 2\pi k r (H-h) \quad \dots(5.18)$$

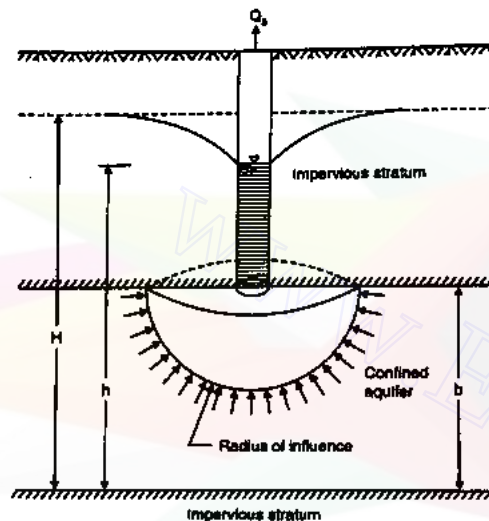


FIG. 5.15 SPHERICAL FLOW IN A WELL.

For the case of simple radial flow in a fully penetrating well the discharge Q is given by equation 5.6(a)

$$Q = \frac{2\pi k b (H - h)}{\log_e \frac{R}{r}} \quad \dots [5.6(a)]$$

$$\frac{Q_s}{Q} = \frac{r}{b} \log_e \frac{R}{r} = 2.303 \frac{r}{b} \log_{10} \frac{R}{r} \quad \dots (5.19)$$

As a numerical example,

Let $r = 8 \text{ cm} = 0.8 \text{ m}$. Hence $\frac{R}{r} = 1000$

$b = \text{thickness of aquifer} = 16 \text{ m}$.

$$\frac{Q_s}{Q} = 2.303 \times \frac{0.08}{10} \log_{10} 1000 \approx \frac{1}{30}$$

This shows that the spherical flow is very much less efficient than the radial flow.

5.13. TUBE WELLS

A water well is a hole or shaft, usually vertical, excavated in the ground for bringing ground water to the surface. Well can be mainly divided into two classes:

1. Dug wells or open wells.
2. Bored or drilled wells or tube wells.

An open well is comparatively of bigger diameter and is suitable for discharge upto 0.005 cumec. This is because the cross-sectional area of flow is less in the open well, and the water can be withdrawn safely only at the critical velocity for the soil. A tube well is a long pipe sunk into the ground with a strainer which allows water to pass through but prevents sand from coming in. Because of the strainer, high velocity of flow can be permitted without danger of soil particles being carried away with water. Also because of the radial flow towards the well, the cross-sectional area of flow is more. Due to the increased velocity and more cross-sectional area of flow a tube well, though much less in diameter than an open well, gives discharge many times more than the open well.

Types of Tube Wells. The tube well may be of three types :

1. Strainer well. 2. Cavity well. 3. Slotted well.

(1) Strainer Type Tube Well

The strainer well is the most common and widely used well. In common term, the word "tube well" refers to the strainer type of tube well. In this type of well, a strainer, which is a special type of wire mesh, is wrapped round the main tube of the well. The main pipe contains bigger holes or slots than the openings of the strainer. The total area of the openings of the tube is kept equal to the openings of the strainer so that the velocity of flow does not change. Due to fineness of the openings of the strainer, a higher operational velocity of water can be permitted. Little annular space is left between the strainer and the pipe so that the open area of pipe perforations is not reduced. Thus mesh size of the strainer is generally kept equal to D_{60} to D_{70} of the surrounding soil.

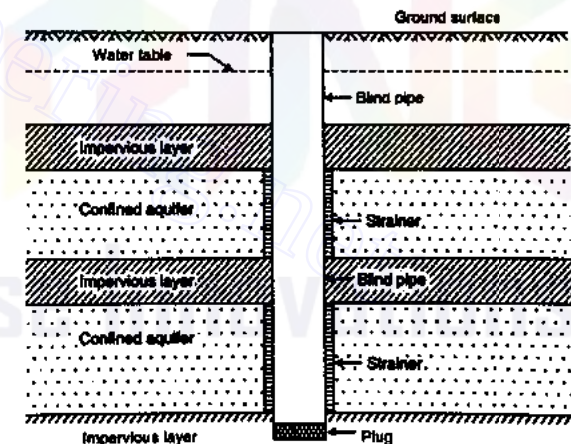


FIG. 6.18. STRAINER TYPE TUBE WELL.

A strainer well may draw water either from an unconfined aquifer of unlimited extent, or from one or more confined aquifer layers. The strainers are provided only in that length of the pipe where it crosses the aquifer. The pipe in the aquifer portion is kept perforated. In the rest of the portion, plain or blind pipe is provided. At the bottom, a short pipe is provided to permit settlement of any sand if passed through the strainer. The well is generally plugged at the bottom.

Abyssinian tube well is a special type of strainer tube well, in which the diameter of pipe is kept 38 mm and the strainer is provided only for a length of about 1.2 m to 1.5 m.

Design Criteria for Strainer Type Tube Well

The following points should be noted for the design of a strainer type of tube well :

- (1) The wire screen should not be in contact with the slotted tubes otherwise a large part of the opening will be covered by the wire screen.
- (2) The area of the opening in the wire screen should be equal to the area of waterway in the perforated or slotted tube to permit no change in the velocity of incoming water particles. If there is any change, sand will be deposited in the annular space.
- (3) The design should permit less velocity through soil than the exit critical velocity which is 1.25 cm/sec.
- (4) The discharging velocity should vary from 1 to 2 m/sec.
- (5) The total surface area of tube well should be more than three times the area of perforations.
- (6) The material of the strainer (wire screen) should be such that it can withstand the strain of sinking and at the same time can give maximum amount of water way consistent with fineness of openings necessary to prevent entry of sand.
- (7) The material should preferably be of one metal. A bimetal construction poses a danger of electrolytic action in water, which may ultimately lead to deposition of salt and choking of openings.
- (8) The material should be sturdy and withstand rough handling.

Following metals and alloys have been found useful for strainers. They are corrosion-resistive :

- (i) Zinc-free-brass or cupro-nickel alloy.
- (ii) Stainless steel.
- (iii) Low carbon steel.
- (iv) High copper-alloy.

Type of Strainers. Following are some of the common types of strainers used in tube well :

- (i) Cook strainer.
- (ii) Tej strainer.
- (iii) Brownlie strainer.

- (iv) Ashford strainer.
- (v) Leggett strainer.
- (vi) Phoenix strainer.
- (vii) Layne and Bowler strainer.

(i) **Cook strainer.** This is a very costly strainer of American patent. It is made up of solid drawn brass tube slotted with wedge-shaped horizontal slots. The slots are made with a slot cutting machine from inside the tube. The slots are wide inside and narrower outside, as shown in Fig. 5.17. The gauge of slots depend on the coarseness of sand and varies from 0.15 to 0.4 mm.

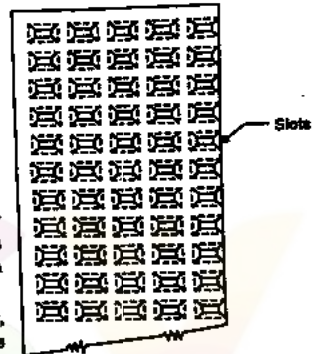


FIG. 5.17. COOK STRAINER.

(ii) **Tej strainer.** It is similar to Cook strainer, but is manufactured in India. It consists of brass tube constructed of a brass sheet bent round to form the tube, the vertical joint being brazed. The slots are cut in the sheet before it is bent. The strainer is generally manufactured from 7.5 cm diameter upwards, and is made in 2½ meter lengths. The individual lengths of the strainer are then joined together by means of screwed collars of brass.

(iii) **Brownlie strainer.** The Brownlie strainer is made of a polygonal convoluted steel plate having perforations. A wire mesh surrounds the steel tube, as shown in Fig. 5.18. The mesh consists of heavy parallel copper wires woven with copper ribbons. Since the wire mesh is slightly away from the perforated tube, it is known as the best type of strainer.

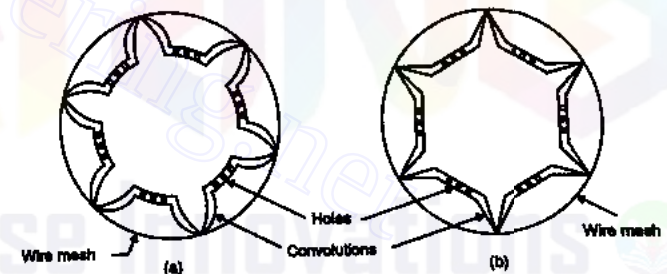


FIG. 5.18 BROWNIE STRAINER.

(iv) **Ashford strainer.** This is very delicate strainer and consists of perforated tube with a wire round it over which a wire mesh is soldered. The wire keeps the mesh away from the tube. The wire mesh is protected and strengthened by wire net around it, as shown in Fig. 5.19.

(v) **Leggett strainer.** It is an expensive type of strainer in which a cleaning device is provided. The cleaning device is in the shape of cutters which can be turned in the slits. The cutters are operated from the top (ground surface). These cutters clean the strainer clogged by the solid matter.

(vi) **Phoenix strainer.** It is a mild steel tube in which the openings are made by cutting slits from inside. The tube is cadmium plated to keep it free from danger of choking and corrosion caused by chemical action.

(vii) **Layne and Bowler strainer.** It is a robust type strainer manufactured in America. It consists of wedge-shaped steel wire wound to a suitable pitch round a slotted or perforated steel or wrought iron pipe. The joints of the strainer pipes are made by screwed collars.

Chocking of Strainers. The strainer of a tube well may get choked due to two actions : (a) Mechanical action, (b) Chemical action.

(a) **Mechanical chocking.** Mechanical chocking may result from the chocking of slits with sand and other particles. This may, however, be prevented by providing such slits which expand inwards. The pulsating action of the centrifugal pump may also remove the chocking. To safeguard against chocking, proper screening or shrouding should be provided. Another method of eliminating chocking is to permit inflow velocity lesser than the critical.

(b) **Chemical chocking.** The strainer may be choked due to chemical action of salts present in water. The chemical action may also deteriorate a strainer by corrosion. If calcium bicarbonate present in water exceeds by an amount of 15 parts per million part of water, carbon dioxide is released when pressure is reduced due to pumping and calcium carbonate is precipitated on the strainer. The cumulative action of such precipitation reduces the yield. The chemical chocking by deposition of carbonates is reduced by providing large slit area and having low inflow velocity. Similarly sodium salts may attack mild steel and cast iron strainer pipes causing chocking. Sodium bicarbonate may attack copper to certain extent, though brass is not readily attacked.

(2) Cavity Type Tube Well

This is a special type of tube well in which water is not drawn through the strainer, but it is drawn through the bottom of the well where a cavity is formed. The tube well pipe penetrates a strong clay layer which acts as a strong roof. Thus, a cavity tube well is similar to a deep well. However, a deep well draws from the first aquifer below the *mota* while a cavity well need not do so. The essential condition for a cavity tube well to function efficiently is to have confined aquifer of good specific yield, and the aquifer should have a strong impervious material above it. In the initial stage of pumping with the help of a centrifugal pump or an air lift pump, fine sand comes with water and consequently a hollow or cavity is formed. As the spherical

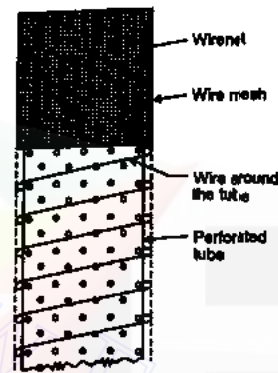


FIG. 5.19 ASHFORD STRAINER.

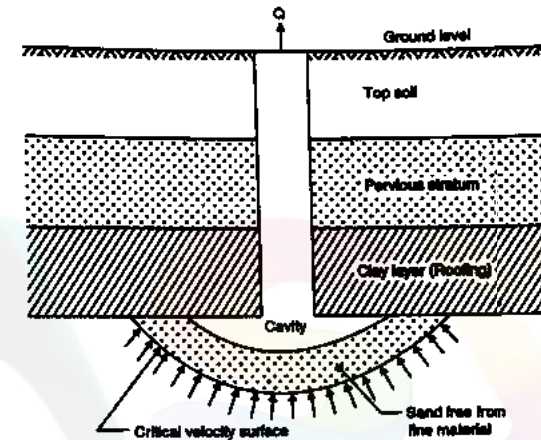


FIG 5.20. CAVITY TUBE WELL.

surface area of cavity increases outwards, the radial critical velocity decreases, and the sand particles stop entering the well. At this stage, an equilibrium in the cavity formation is established and clean water continues to enter the well on further pumping at the same constant discharge. After the formation of the cavity, the velocity of entry of water at the bottom of the pipe is lesser than the critical.

The essential difference between a strainer tube well and a cavity tube well lies in the fact that in the former case, due to the provision of a strainer, the flow is *radial*, while in the later case the flow is *spherical*. In the strainer tube well, the area of flow is increased by increasing the length of the strainer pipe (in an unconfined aquifer of infinite extent) while in the case of cavity type, the area of flow is increased by enlarging the size of the cavity. The cavity, formed due to a particular discharge Q_1 , increases if an increased discharge Q_2 is pumped out.

(3) Slotted Type Tube Well

A slotted tube well is resorted to under two circumstances : (1) *sufficient* depth of water bearing stratum is not available even upto a depth of 75 to 100 m, so that strainer type tube well cannot be used and (2) Suitable strong roof is not available so that a cavity well cannot be formed. In such a circumstance, a slotted well is used and it is made to penetrate to some depth in the water bearing strata. A slotted tube well essentially consists of a slotted tube penetrating the confined aquifer. The size of slots may be 25 mm × 3 mm at 10 to 12 mm spacings. In order to prevent the fine particles entering the pipe, it is shrouded with a mixture of gravel and bajri (coarse sand).

First of all, a casing pipe of 36 cm diameter is lowered, and soil is excavated out, and the water bearing strata is penetrated by a depth of about 5 m length. The perforated pipe, sometimes known as the education pipe of 15 cm diameter is then lowered, the slotted portion being only 5 m long and the rest of the length being of plain pipe. Gravel is then poured from the top, upto about 3 to 4 m higher than the top level of perforated portion of the pipe. The casing pipe is then withdrawn 5 cm at a time and the well is developed with the help of compressed air pumped into the education pipe. Finally, when the casing pipe is fully withdrawn, the annular space between the casing pipe and the education pipe is suitably plugged. By developing the well with the help of compressed air, the sand surrounding the gravel filter is freed of finer particles, and the chances of getting the filter clogged are reduced. Due to the provision of gravel shrouding, a larger area of radial flow is obtained.

There are two essential differences between a strainer tube well and a slotted tube well : (1) in the strainer tube well, the strainer pipes are surrounded by wire mesh to prevent the fine particles from entering the well, while in the slotted tube well the gravel shrouding serves this purpose, (2) a strainer tube well may have several alternative lengths of strainer pipes and plain pipes, while a slotted tube well has the slotted pipe length only at its bottom. Thus a strainer tube well draws water from several aquifers sandwiched between impervious layers, while a slotted tube well draws water only from one pervious stratum which has sufficient water bearing capacity.

5.14. METHODS FOR DRILLING TUBE WELLS

For installing a tube well in the ground, so as to penetrate the required stratum, a hole, slightly of larger diameter than the diameter of the strainer pipe, is bored. The most common methods for boring a well are :

1. Wash boring or water jet boring method.
2. Cable tool method (also known as percussion or standard method).
3. Hydraulic rotary method.
4. Reverse rotary method.

1. Wash Boring or Water-jet Boring Method

This method is suitable at places where the well is to be sunk into formations consisting of gravel, sand, clay or other soft deposits. The boring is done by cutting action of a downward directed stream of water. The outer casing in first erected in position in a suitable pit dug at the surface. A jet pipe with a nozzle is then lowered

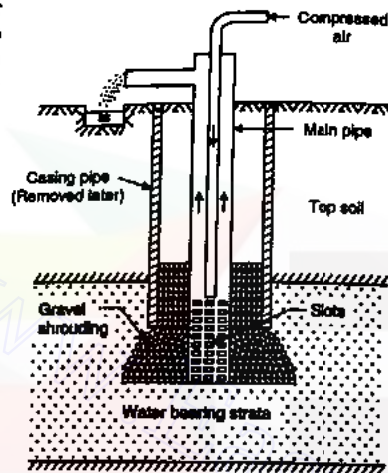


FIG. 5.21. SLOTTED TYPE TUBE WELL.

in the casing tube, and water under pressure is forced through it. The dislodged soil particles and broken pieces form slurry with water and are lifted up through the annular space between the casing and jet pipe by the returning water in the upward direction. The casing pipe having shoes at the bottom is kept rotated slowly, and is thus lowered. In penetrating clays and hard pans, various types of jetting drill bits are fitted to the drill pipe which is raised and lowered sharply, causing the bit to shatter the formation. When the casing pipe has penetrated to a sufficient depth into the aquifer, the well pipe attached with screen etc. is lowered in the casing pipe. The outer casing is then pulled. Sometimes a self jetting well point is used. In this, the casing pipe is not used but, instead a tube of brass screen ending into a jetting nozzle to the main well pipe (Fig. 5.22). As the jetting action progresses, the well pipe goes on sinking. An annular space round the well pipe is automatically created due to the upward motion of water carrying the dislodged particles. When the well pipe has been sunk to the desired depth, jetting is stopped, and the annular space is packed with gravel.

2. Cable Tool Method (Percussion Method)

Cable tool method, also sometimes known as the percussion method or Standard method, is used for drilling deep wells through consolidated rock materials. In this method, a standard well drilling rig consists of a multiline hoist, a walking beam and an engine—all assembled and mounted on a truck for easy portability. A string of percussion tools consists of (Fig. 5.23) a rope socket, a set of jars, a drill stem and a drilling bit—the total weight of these amounting to several thousand kilograms.

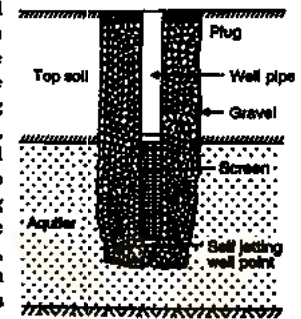


FIG. 5.22. SELF-JETTING WELL POINT.

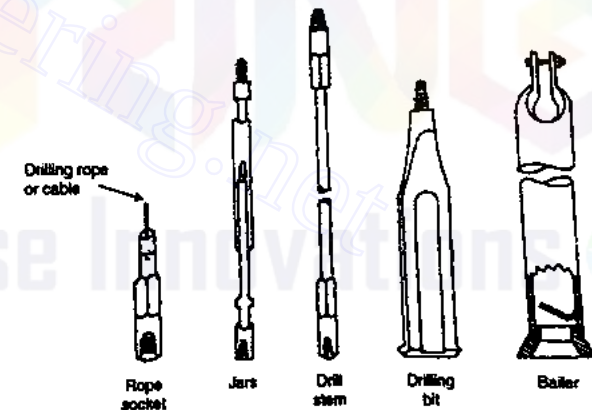


FIG. 5.23. PERCUSSION DRILLING TOOLS.

The drilling bits are manufactured in 1 to 3 meter lengths, and may weigh upto 1500 kg. A pit is dug at the site where the well is to be drilled. A casing pipe, with a drive shoe is inserted in the pit. The string of drilling tools is inserted in the first length of the casing pipe. Drilling is then accomplished by regular lifting and dropping of the string of tools mechanically. During drilling, the tools make 40 to 60 strokes per minute, ranging from 40 cm to 1 m in length. The Drilling line is kept continuously rotated so that the drilling bit will form a round hole. After the bit has cut 1 to 1½ metre through the formation, the string of tools are taken out and a bailer (Fig. 5.23) is inserted in the hole to remove the drill cutting. The bailer essentially consists of a pipe like section with a valve at the bottom. When the bailer is inserted in the hole, the valve is automatically opened by the upward movement of the cuttings. The valve, however, prevents the cuttings from moving in the downward direction and thus escaping during lifting. When the bailer is full, it is lifted up to the surface and emptied. The length of the bailer varies with its diameter, and may range from 3 to 13 metres. When the cuttings have been taken out, the string of tools are again inserted and blows given to break the formation by impact. If no water is encountered in the hole, water is added from the surface to form a paste with the cuttings. The casing is driven down by motion of the tools striking the top of the casing, protected by a drive head, sinks the casing. The individual subsequent lengths of casing are joined by threaded or welded joints.

In soft and fissured rock formations, manual labour may be used (Fig. 5.24).

In this case, the boring is done with the help of 'sludger' or 'sand pump'. The sludger, almost similar to the bailer, is a steel pipe 2 to 4 metres long, having a cutting shoe of hard steel riveted to its bottom. A flap valve at its lower end permits the entry of the cut material. The sludger is inserted in the casing pipe and is worked up and down by means of a rope the other end of which passes over a pulley fixed centrally to a tripod. A platform is attached to the upper end of the casing pipe, and the weight placed on the platform drives the casing pipe slowly into the hole. When the sludger is full with the cutting paste, it is taken out and emptied. The process goes on till the required level is reached. However, if the rock formations are hard, the sludger is unable to cut it and a string of drilling bits, shown in Fig. 5.23, is inserted and operated by an engine.

In both the methods, the record of the material collected by bailer or sludger is kept. A bore log is then plotted to know the depths of various formations. The well pipe with strainers at determined levels of aquifer is then lowered to the desired

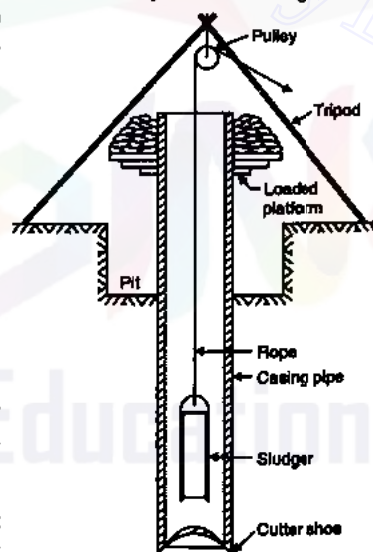


FIG. 5.24. PERCUSSION BORING BY MANUAL LABOUR.

depth. The well is then shrouded and the casing pipe is taken out when the shrouding is done in steps. Wells by cable tool method have been drilled in diameters up to 30 cm and to depths as great as 600 metres.

8. Hydraulic Rotary Method

Hydraulic rotary method, sometimes known as rotary boring method is used for drilling large bores in unconsolidated strata. This is fastest method and has been used for wells upto 45 cm diameter (upto 150 cm with a reamer), and for depth over 163 m. Oil wells over 7000 metre deep have been drilled by this method.

In this method, the boring is done with the help of a drilling bit attached at the end of a string of hollow pipe (Fig. 5.25). A mixture of clay and water, known as *drilling mud*, is continuously circulated through the drill shaft in the hole. Material loosened by the bit is carried upward in the hole by the rising mud. Ordinarily no casing is required since the drilling mud forms a clay lining and supports the walls of the hole.

The drill bits have hollow shanks and one or more centrally located orifices for jetting the mud into the bottom of the hole. The drill rod made of heavy pipe, carries the drill bit at one end and is screwed to a square section known as *kelly*. A rotating table, which fits closely the kelly, rotates the drill rod which slides downward as the hole deepens. The rising drilling mud carrying rock fragments is taken to the settling basin where the cuttings settle. The mud is recirculated to the hole. To maintain the required consistency, clay and water is added to the circulating mud from time to time. A complete boring record is maintained to know the type of formations at various depths. When desired level is reached, the drill rod, etc. are taken out and well pipe containing strainers pipes at appropriate locations (opposite aquifers) is lowered. Since the well walls are coated with clay, it should be washed to get more discharge. Back washing is done by lowering the drill pipe and bit in the well-pipe, and forcing water containing calgon (sodiumhexa- meta phosphate). Calgon has the property of dispersing clay colloids. A collar of the size of well pipe, is attached to the drill rod just above the bit. This forces the water through the strainer causing washing action on the clay wall. At the same time, the drill rod is plunged up and down causing surging action. When washing in the bottom is done, the bit is raised through some distance and the operation is repeated.

4. Reverse Rotary Method

Reverse rotary method, similar to the hydraulic rotary method, is very much used in Europe. In this method, the cutting are removed by a suction pipe. A large

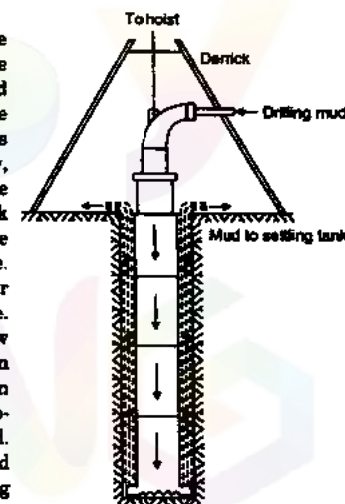


FIG. 5.25. HYDRAULIC ROTARY METHOD.

capacity centrifugal pump is used for this purpose. A mixture of water and fine grained material is circulated in the hole. The procedure is essentially a suction dredging method. The walls of the hole during drilling are supported by hydrostatic pressure acting against the film of fine-grained material deposited on the walls by the drilling water. The method of recirculation of drilling water containing fine grained particles and cleaning the well after inserting the well-pipe is similar to that of hydraulic rotary method.

6.15. WELL SHROUDING AND WELL DEVELOPMENT

(a) Well Shrouding

Well shrouding is a process of interposing coarse material such as gravel and coarse sand between the well-pipe (strainer pipe) and the aquifer soil to prevent finer particles of soil coming in contact with the strainer and choking it. This is essential in sandy and unconsolidated formations of aquifer. This is also essential in slotted type tube well where a strainer is not used. Such a tube well is also sometimes known as a *gravel-packed well*. The shrouding increases the effective well diameter, acts as a strainer to keep fine material out of the well, and protects the well-pipe from caving of surrounding formations. A gravel packed well has a greater specific capacity than one of the same diameter not shrouded by gravel. A minimum thickness of 40 cm gravel pack is necessary to make it effective. The proper grain size distribution of the shrouding material depends upon the mechanical analysis of the aquifer and upon the perforation or screen slot size.

The amount of shrouding material per 30 cm length of the casing pipe can be calculated accurately before hand. In the beginning, the shrouding material for 60 cm lengths is shovelled in from the top between the tube-pipe and casing. The casing is raised by 30 cm with the help of jack. Then the quantity for each 30 cm is added and the casing pipe is withdrawn 30 cm at a time till the strainer is covered. The shrouding material is sometimes placed through small pipes or pilot holes around the tube-pipe which feed the material down into position. A further refinement is the use of bladeless pumps for pumping the shrouding material in place.

(b) Well Development

Well development is the process of removing fine material from the aquifer formation surrounding the strainer pipe, and is aimed at (i) increasing the specific capacity of the well, (ii) preventing sand flowing in, and (iii) obtaining maximum economic well life. The actual yield of the well can be known only after well development. Thus, it also helps in determining the requiring characteristics of the pump and power unit to be installed. Depending upon formation characteristics of the aquifer, a well may be developed by one of the following methods :

1. Development by pumping.
2. Development by surging.
3. Development by compressed air.
4. Development by back washing.
5. Development by dry ice.

1. Development by Pumping

In this method a variable speed pump is used. The method is based on the principle that irregular and non-continuous pumping agitates the fine material surrounding the well so that it can be carried into the well and pumped out. Initially, the pump is started with a very low discharge. The fine particles start coming. This low speed is maintained till clear water comes. The discharge is then increased in steps until maximum discharge or well capacity is reached. The pump is then stopped and levels permitted to increase till it comes to normal. The pump is then again started and the procedure repeated, till no fine particles come.

2. Development by Surging

In this method, surging effect is created by up and down movement of a hollow surge block or a bailer. Calgon (sodium hexametaphosphate) is added to water so that it acts as dispersing agent for fine grained particles. When the surge block is moved up it sucks water in. When it is moved down, it forces water-calgon solution back in the formation. Further upward motion brings with it fine material. The surge block is connected to a string of hollow pipe from which the water charged with fine particles is pumped out continuously. The procedure is repeated by increasing the speed of surging till clear water comes out.

3. Development by Compressed Air

In this method, the development is done with the help of an air compressor, a discharge pipe and air pipe. The air pipe is put into the discharge pipe and is lowered into the well tube, till the assembly reaches near the bottom of the strainer-pipe section. The lower end of the air pipe is kept emerging out of the discharge pipe by a small length. The air entry to the air pipe is first closed and, the compressor is then started till a pressure of 6 to 10 kg/cm² is built up. The air is then suddenly made to enter the pipe, at this pressure, with the help of suitable quick-opening valve. This sudden entry of air into well creates a powerful surge within the well causing loosening of fine material surrounding the perforations. When ice pressure decreases, water enters the well bringing the loosened particles with it. The continuous air injection creates an air lift pump, and the water carrying fine particles is pumped out. The process is repeated till clear water comes. The pipe assembly is then lifted up, and the surging is again created. This operation is repeated at interval along the screen section till the well is fully developed.

4. Development by Back Washing

In this method, in addition to the compressor, a discharge pipe and an air pipe, and additional small air pipe is used. The well is sealed at its top so that it becomes air tight. The discharge pipe and air pipe assembly is lowered in the well, as in the previous method, but the end of the air pipe is kept inside the discharge pipe. A small air pipe is fitted at the top of the air-tight cover, and is provided with a three-way cock. With the help of the three way cock, air can be admitted to the well either through the long air pipe (but inside the discharge pipe) or through the small air pipe fitted at the top. Air is first made to enter the long air pipe. This forces air and water out of the well through the discharge pipe. When clear water comes, the

valve is closed, and water level is allowed to increase in the well. The valve is then turned to the other-side so that air enters through the small air pipe. This *back washes* the water from the well through the discharge pipe and at the same time agitates the fine particles surrounding the well. Calgon is often added to water. When air starts escaping from the discharge pipe, the valve is turned so that air enters the long air pipe, so that assembly works at an air-lift pump and the water is pumped out. The procedure is repeated till clear water comes and the well is fully developed.

5 Development by Dry Ice (Solid Sodium Dioxide)

In this method, well is developed with the help of two chemicals : hydrochloric acid and solid sodium dioxide (known as dry ice). First of all hydrochloric acid is poured into the well. The well is capped at the top and compressed air is forced into the well. The pressure of the compressed air forces the chemical into the formation. The cap is then removed and blocks of dry ice are dropped into the well. The sublimation releases gaseous carbon dioxide, and a high pressure of this gas is built up in the well. On releasing the pressure the muddy water is forced up in the form of a jet and is automatically thrown out of the well. Explosion of mud and water extending 40 metres into the air from a well in Utah (U.S.A.) was observed when the well was developed with dry ice.

5.16. OPEN WELL

As stated earlier, an open well is essentially of a bigger diameter than that of tube well, and derives its water only from one pervious stratum. Since a tube well, in general may derive water from more than one aquifer formation, it has greater depth than an open well. The economically feasible depth of an open well is limited to 30 metres below the ground surface. In a lined open well, the entry of water is from the bottom and *not* from the sides.

An open well is classified as :

- (i) Shallow well.
- (ii) Deep well.

The nomenclature of shallow and deep well has nothing to do with the actual depth of the well. A *deep well* is a well which is supported on a *mota* layer and

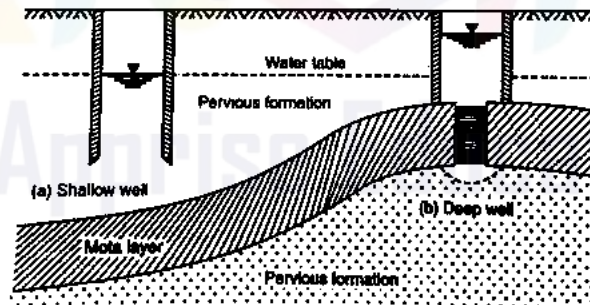


FIG. 5.26. SHALLOW AND DEEP WELLS.

draws its water supply through a hole bored in it from the pervious formation below the *mota* layer. A *shallow well*, on the other hand, penetrates the pervious stratum only and draws its water supply through it. The term *mota* layer also sometimes known as *matbarwa* or *nagasan*, refers to a layer of clay, cemented sand, kankar or any other hard material. The *mota* layer gives structural support to the open well, and is found throughout the Indo-Gangetic plain. The *mota* layer may either be continuous or may be localised sand and may be found in different thicknesses and depths at different places.

Fig. 5.26(a) shows a shallow well which derives water from the pervious stratum, and does not rest on a *mota* layer. Fig. 5.26(b) shows a deep well resting on a continuous *mota* layer. Fig. 5.27 shows a deep well resting on a localised *mota* layer and deriving its water from the second pervious stratum. Actually, a shallow well can be deeper than deep well. However, since a shallow well draws water from the first pervious stratum (i.e., the top formation), the water in it is liable to be contaminated by rain water percolating in the vicinity, and may take with it mineral organic matter such as decomposing animals and plants. The water in a deep well is not liable to get such impurities and infection. Also, the pervious formation, below a *mota* layer, normally has greater water content and specific yield. Hence discharge from a deep well is generally more than a shallow well.

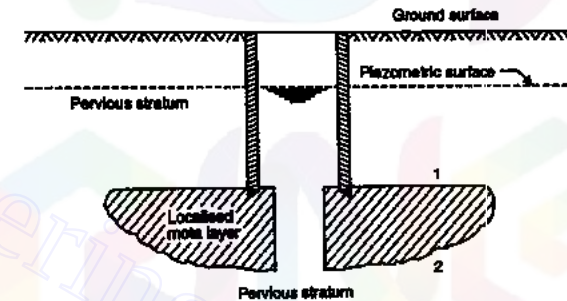


FIG. 5.27. DEEP WELL ON LOCALISED MOTA LAYER.

The open well may further be classified as :

- (i) Kachha well or unlined well.
- (ii) Well with impervious lining.
- (iii) Well with pervious lining.

Kachha Well . A kachha well is a temporary well of a very shallow depth. It is suitable only in hard formations the walls of which can stand vertically. They are suitable only when the water table is very near the ground surface. Such well often collapse after some time and are dangerous.

Well with Impervious or Pucca Lining. This is the most common type of open well, and is suitable for all types of formations. Once constructed, it becomes a permanent source of water supply. Impervious lining for an open well in sandy formations is most essential to give structural stability to the well.

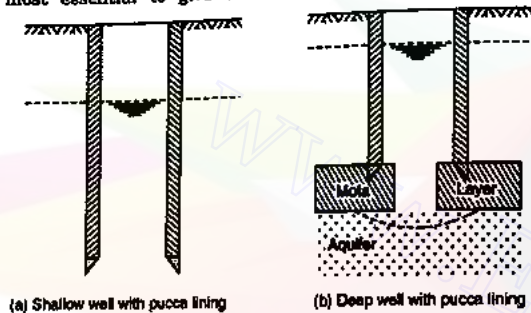


FIG. 5.28. WELLS WITH THE PUCCA LINING

The thickness of impervious lining (steining) varies from 30 to 60 cm and may be either in brick masonry or in stone masonry. The linings carry well curbs under them. Well curbs may be constructed of either wood, iron or reinforced concrete. In a pucca well, the flow is not radial. Water enters only from the bottom and, after a virtual cavity has been formed at the bottom, the flow is spherical.

Well with Pervious Lining. Such types of wells are suitable in coarse formations. The lining consists of dry bricks or stones with no mortar or binding material. Due to this, water enters from the sides, and the flow is, therefore, radial. Such wells are generally plugged at the bottom. If there is no plug at the bottom the flow is a combination of radial and spherical pattern.

If a well with pervious lining is constructed in a sandy formation, brick ballast upto 20 mm size and gravel is placed behind the lining to prevent sand coming into the well along with the flowing water.

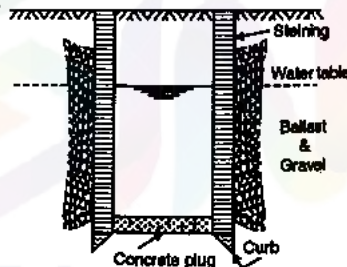


FIG. 5.29. WELL WITH PERVIOUS LINING.

5.17. YIELD OF AN OPEN WELL

The hydraulics of a lined open well is essentially different from that of a tube well. The yield from an open well can be found by the following two tests :

- (1) Constant level pumping test.
- (2) Recuperation test.

(1) CONSTANT LEVEL PUMPING TEST

In this test, a pump with suitable regulating arrangement is used. The water level is depressed by an amount h (say) known as the *depression head*. The speed

of the pump is so adjusted that whatever water enters the well under this depression head is pumped out and a constant water level is maintained in the well. The amount of water pumped out is measured with the help of a V-notch or any other arrangement, in a given amount of time for which the pump speed was regulated to a constant value. The quantity pumped out in one hour gives the yield of the well per hour.

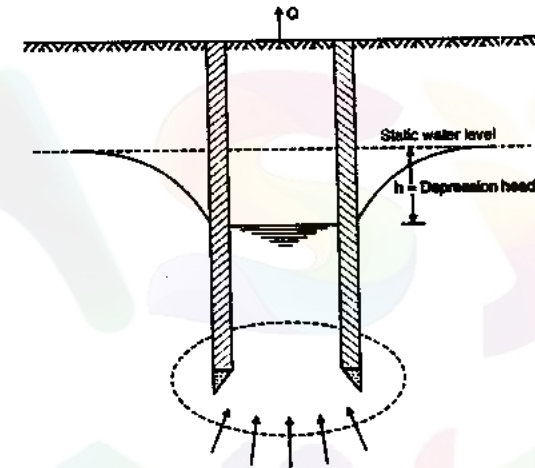


FIG. 5.30 CONSTANT LEVEL PUMPING TEST.

The formula for discharge, in cumecs from an open well with impervious lining may be written as :

$$Q = A \times v$$

or

$$Q = A \times C \times h \text{ cumecs} \quad \dots(5.20)$$

where Q = discharge, in cubic metres per second

A = cross-sectional area of flow into the well at its base, in m^2

v = mean velocity of water percolating into the well, in metres/sec

h = depression head in metres

C = percolation intensity coefficient. This is a constant of the formation around the well. Its value is greater for coarser soil and smaller for finer soils (m/sec under unit head).

The above formula can also be derived from Darcy's law as under :

$$Q = k \cdot A \cdot i = k \cdot A \cdot \frac{h}{L} = \frac{k}{L} \cdot A \cdot h$$

or

$$Q = C \cdot A \cdot h \quad \dots(5.20)$$

Due to cavity formation, the area A is taken to be equal to $4/3$ times the actual cross-sectional area of the bottom of the well.

From the above expression, it is clear that the discharge increases with the percolation head h . However, the percolation head cannot be increased beyond a certain critical value because otherwise the percolation velocity will be exceeded and the soil particles will be disturbed and dislodged. The critical value of h at which the velocity is critical is known as the *critical depression head*. Normally, the depression head is kept equal to $1/3$ of the critical head; such a head is known as the *working head*.

Maximum yield or critical yield, therefore, will be obtained corresponding to the critical depression head. The yield under the working head is known as the *maximum safe yield*. From a pumping test, we can find the maximum safe yield.

(2) RECUPERATION TEST

Though the constant level pumping test gives an accurate value of the safe yield of an open well, it is sometimes very difficult to regulate the pump in such a way that constant level is maintained in the well. In such a circumstance, a recuperation test is resorted to. In the recuperation test, water level is depressed to any level below the normal and the pumping is stopped. The time taken for the water to recuperate to the normal level is noted. From the data, the discharge from the well can be calculated as under (Fig. 5.31).

Let aa = static water level in the well, before the pumping started.

bb = water level in the well when the pumping stopped

h_1 = depression head in the well when the pumping stopped (metres)

cc = water level in the well at a time T after the pumping stopped

h_2 = depression head in the well at the time T after the pumping stopped (metres)

h = depression head in the well at a time t after the pumping stopped (metres)

dh = decrease in depression head in a time dt

t, T = time in hours.

Thus, in a time t , reckoned from the instant of stopping the pump, the water level recuperates by $(h_1 - h)$ metres. In a time dt after this, the head recuperates by a value dh metres.

Volume of water entering the well, when the head recuperates by dh is

$$dV = A dh \quad \dots(1)$$

where A = cross-sectional area of well at its bottom

Again, if Q is the rate of discharge in the well at the time t , under the depression head h , the volume of water entering the well in time t hours is given by

$$dV = Q dt$$

But

$$Q \propto h$$

or

$$Q = Kh$$

$$dV = K h dt \quad \dots(2)$$

$$\dots(3)$$

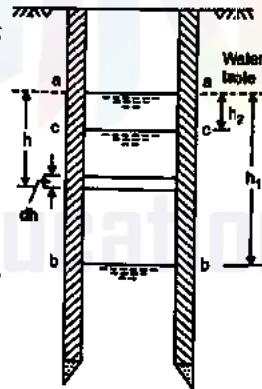


FIG. 5.31. RECUPERATION TEST.

where K is a constant depending upon the soil at the base of the well through which water enters.

Equating (1) and (3), we get

$$K h dt = - A dh \quad \dots(4)$$

The minus sign indicates that h decreases as time t increases. Integrating the above between the limits: $t = 0$ when $h = h_1$; $t = T$ when $h = h_2$.

$$\text{we get} \quad \frac{K}{A} \int_0^T dt = - \int_{h_1}^{h_2} \frac{dh}{h}$$

$$\text{or} \quad \frac{K}{A} \int_0^T dt = - \int_{h_1}^{h_2} \frac{dh}{h}$$

$$\text{From which} \quad \frac{K}{A} T = \left[\log_e h \right]_{h_2}^{h_1}$$

$$\therefore \frac{K}{A} = \frac{1}{T} \log_e \frac{h_1}{h_2} = \frac{2.303}{T} \log_{10} \frac{h_1}{h_2} \quad \dots(5.21)$$

Thus knowing the value of h_1 , h_2 and T from recuperation test, the quantity K/A can be calculated. K/A is known as the *specific yield or specific capacity* of an open well, in cubic meter per hour per sq meter of the area through which water percolates under one metre depression head. In the absence of the recuperation test, the following rough values of K/A specified by Marriot can be adopted:

Type of soil	K/A Cubic metres per hour, per sq metre of area under unit depression head
Clay	0.25
Fine sand	0.5
Coarse sand	1.00

Knowing the value of K/A by observation, the discharge from a well under a constant depression head H can be calculated as under:

$$Q = KH \quad \text{(from 2)}$$

$$\text{or} \quad Q = \left(\frac{K}{A} \right) A H \quad \dots(5.22)$$

$$\text{or} \quad Q = \frac{2.303}{T} \left(\log_{10} \frac{h_1}{h_2} \right) A H \text{ m}^3/\text{hour} \quad \dots(5.23)$$

It should be noted that the time T in above expression is in hours. If T is substituted in seconds, Q will be in cumecs. The discharge Q will be the maximum yield of the well if H corresponds to the maximum depression head. If H is the average depression head, Q will be the average yield.

5.18. METHODS OF LIFTING WATER

The various devices for lifting water can be divided into two heads:

- Indigenous methods.
- Mechanical methods.

(a) **Indigenous Methods.** They include :

1. Basket	2. Doon
3. Archimedian screw	4. Denki
5. Rati or Girmi	6. Windlass
7. Mote or Churus	8. Persian wheel.

(b) **Mechanical Methods.** They include:

9. Windmill	10. Hydraulic Ram
11. Pumps.	

It is assumed that the reader is familiar with the methods (1) to (10).

Pumps. The discharge from a shallow open well is usually less, and hence hand operated pitcher pumps, wind mill operated plunger pumps, turbine pumps, gear pumps or centrifugal pumps may be used. Though atmospheric pressure is equal to 13.6 m (34 ft) head of water, dissociation takes place much earlier. Hence, the suction lift in these pumps should not exceed 7.62 m (25 ft). For deep wells and tube wells, where the lift is more than 8 m, large capacity pumps are used. The factors to be considered for the selection of pump for such wells are : diameter and depth of well, depth of water table, drawdown, seasonal variability of ground water table, duration of pumping, capacity, initial and maintenance costs, Power required and water quality. The following common types of pumps are suitable for deep well and tube well operation:

1. Deep well turbine pump.
2. Deep well jet pump.
3. Air lift pump.

1. Deep Well Turbine Pump

This is the most common type of pump used in tube wells. Deep well turbines are available for diameters varying from 10 cm to 35 cm, and their usual lengths vary from 20 to 60 cm. For each 5 to 8 m lift, small diameter impellers or bowls are used in series. The power impellers are of centrifugal type and the upper impellers are of mixed flow type. The driving shaft is enclosed in a pipe and the space between the two pipes serves as an outlet for water.

2. Deep Well Jet Pump

Jet pumps are used only for small discharges and are suitable for 10 to 20 cm diameter wells and for lifts between 6 to 50 metres. In a jet pump, a rotor pumps water into a draft tube which has a valve at the lower end. Water rises up in the tube due to suction created by the jet. Some quantity of water is again utilised by the jet.

3. Air lift pump

In this type, a compressor is used to force air into a small diameter pipe, called the *air pipe*. The air pipe is placed in a bigger diameter discharge pipe, sometimes known as the *education pipe*. When air is forced into water, air-water mixture is formed whose specific gravity is below that of water. Due to this, water rises in the education pipe and is discharged out. Due to this, water rise in arranging air and education pipes are shown in Fig. 5.32.

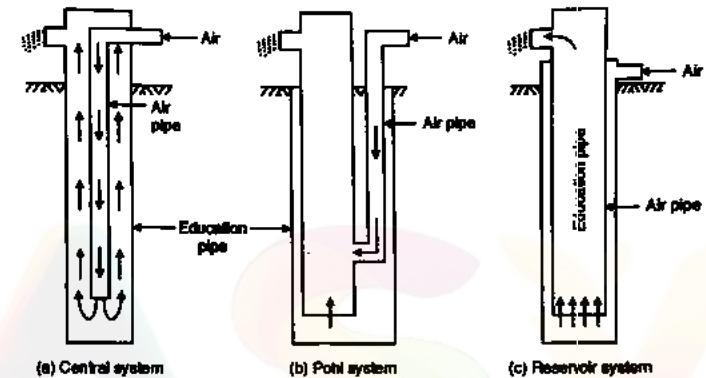


FIG. 5.32. SCHEMATIC DIAGRAMS OF AIR LIFT PUMP SYSTEMS.

5.19. ADVANTAGES AND DISADVANTAGES OF WELL IRRIGATION OVER CANAL IRRIGATION

Advantages

- (1) The well is under the direct control of the owner. Hence wells may be sunk and equipped as required.
- (2) Isolated area can be irrigated by a well. Wells may be sited to command any desired land.
- (3) The supply from a well can be maintained fairly constant. Water can be turned off at any moment, taking advantage of the rainfall.
- (4) Since a well is generally centrally located, the loss in transit is very much reduced. The duty of water in well irrigation is generally higher.
- (5) Volumetric assessment is possible.
- (6) Well irrigation is helpful in lowering the sub-soil water level and in draining off irrigated land which might become water logged. In canal irrigation, the chances of water logging is much more than in the well irrigation.
- (7) Unless drought continues for several years, well irrigation does not fail in drought seasons, while a canal may fail.
- (8) With the help of well irrigation, more than one crop in a year can be grown.
- (9) The well water, which is warmer in cold weather, and cooler in hot weather, is more agreeable to crops.
- (10) The cost of construction of a well is low and irrigation in a locality can be introduced in stages.

Disadvantages

- (1) Since water has to be lifted from the well, the working expenses are very high in comparison to the canal irrigation.

(2) Due to mechanical defects in the pump or due to interruption in electric supply, water may not be made available to the crop in right time.

(3) The well water is clear and free from silt. Clear water does not have the manuring value which silt in canal water provides.

(4) The tube well strainer is subject to progressive deterioration due to mechanical and chemical action. Thus, replacements are necessary after frequent interval of time.

(5) The maintenance of mechanical and electrical machinery also requires great care and more funds.

5.20. SELECTION OF SUITABLE SITE FOR A TUBE WELL

The site where a tube well is to be located must be carefully selected. Improper or careless selection of the site have resulted in tube wells having very low yield or without any yield during summer season. Examples of such tube wells are present in Mahi area where some tube wells did not discharge any water during summers and had to be shifted.

The following points should be considered for the selection of the site :

- (1) Tube well should be selected at a site where large underground reservoir exists. A judgement of such a place can be made on the following considerations:
 - (a) The place will be usually dark and green in summer also indicating a high water table.
 - (b) Certain types of worms and insects such as gnats hover in a column round the spot where there is likelihood of sub-soil water.
 - (c) By a study of geological strata by actual boring.
- (2) If the geological explorations indicate ridges and depressions of impermeable strata inside the ground, the tube well should be located where there is the valley (Fig. 5.33).

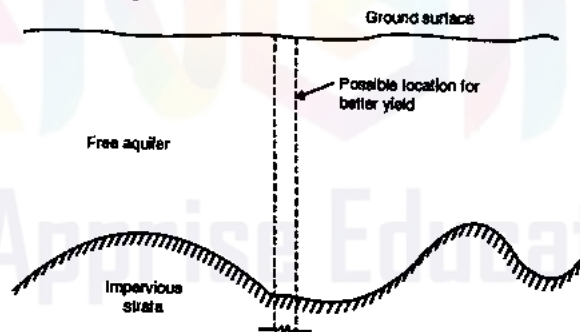


FIG. 5.33.

- (3) The area should have an access for the availability of a cheap electric supply so that motor driven pumps can be fitted with tube wells without an exorbitant cost. Diesel driven sets are not as efficient as those by electric power.

- (4) The area should have a well distributed and uniform demand for irrigation throughout the year. If the demand is not uniform, tube well will be idling for some time or has to work inefficiently and thus the cost for overall irrigation will increase.
- (5) The area around the tube well should have an intensive cultivation and the tube well should be located centrally so as to reduce and the length of guls and thereby transient losses. In any case the tube well should have a command.
- (6) The water available from the tube well should be tested to find out the irrigation quality of water. If the water is found to contain harmful salts, the site may have to be changed in spite of various advantages.

Section of a Tube Well. Fig. 5.34 shows the section of tube well with a centrifugal pump.

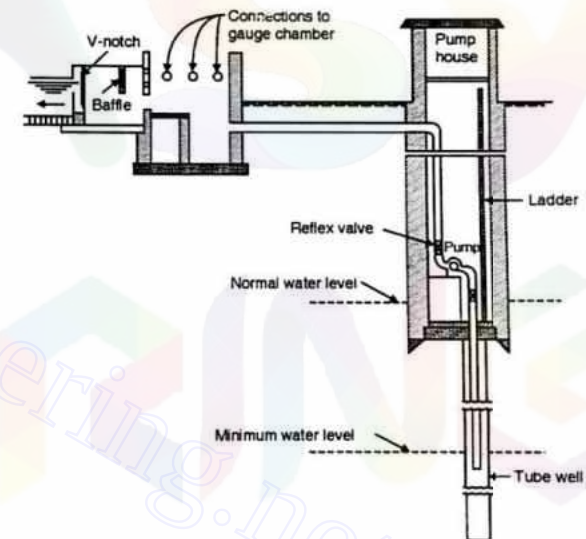


FIG. 5.34. SECTION OF A TUBE WELL.

ILLUSTRATIVE EXAMPLES

Example 5.1. design an open well in fine sand to give a discharge of 0.003 cumec when worked under a depression head of 2.5 metres.

Solution

Required discharge $Q = 0.003 \text{ cumec} = 0.003 \times 3600 = 10.8 \text{ m}^3 / \text{hour}$
From Eq. 5.22

$$Q = \left(\frac{K}{A} \right) A.H. \quad \dots(1)$$

For fine sand, $\frac{K}{A} = 0.5 \text{ m}^3/\text{hour per m}^2$ of area under unit depression head

$H = \text{depression head} = 2.5 \text{ m.}$

Substituting values in (1), we get

$$10.8 = 0.5 \times A \times 2.5$$

$$A = \frac{10.8}{0.5 \times 2.5} \text{ m}^2 = 8.64 \text{ m}^2$$

$$\text{Well diameter } d = \sqrt{\frac{4A}{\pi}} = \sqrt{\frac{4 \times 8.64}{\pi}} = 3.32 \text{ m (say 3.4 m)}$$

Alternative Solution

Another method to find the well diameter is to use the discharge formula Eq. 5.20 in terms of percolation coefficient. From Eq. 5.20,

$$Q = C.A.H. \quad \dots (2)$$

From Table 5.2, percolation or permeability constant for fine sand can be taken equal to 0.0075 cm/sec. (average)

$$C = 0.0075 \text{ cm/sec.} = 7.5 \times 10^{-5} \text{ m/sec.}$$

$$Q = 0.003 = 3 \times 10^{-3} \text{ m}^3/\text{sec.}$$

$$H = 2.5 \text{ m}$$

$A = \text{percolation area}$

$$A = \frac{Q}{CH} = \frac{3 \times 10^{-3}}{7.5 \times 10^{-5} \times 2.5} = 16 \text{ m}^2$$

If no cavity is formed at the bottom, $A = \frac{\pi}{4} d^2$

If full hemispherical cavity is formed, $A = \frac{\pi d^2}{2}$

Usually the area of cavity is between the two values

$$A = \frac{1}{2} \left(\frac{\pi}{4} d^2 + \frac{\pi d^2}{2} \right) = \frac{3}{8} \pi d^2 \approx \frac{1}{3} \pi d^2.$$

Thus the area of percolation at the well bottom may be taken equal to 4/3 times the cross-sectional area of well bottom (i.e., $\frac{\pi}{3} d^2 = \frac{4}{3} \times \frac{\pi}{4} d^2$)

$$\frac{\pi}{3} d^2 = 16$$

$$d = \sqrt{\frac{16 \times 3}{\pi}} = 3.9 \text{ m.}$$

Example 5.2: During a recuperation test, the water in an open well was depressed by pumping by 2.5 metres and it recuperated 1.8 metres in 80 minutes. Find (a) yield

from a well of 4 m diameter under a depression head of 3 metres, (b) the diameter of the well to yield 8 litres/second under a depression head of 2 metres.

Solution

From Eq. 5.21, the specific yield is given by

$$\frac{K}{A} = \frac{2.303}{T} \log_{10} \frac{h_1}{h_2}$$

where $T = \text{time in hours} = \frac{80}{60} \approx \frac{4}{3} \text{ hours}$

$$h_1 = 2.5 \text{ m}$$

$$h_2 = 2.5 - 1.8 = 0.7 \text{ m}$$

$$\frac{K}{A} = \frac{2.303 \times 3}{4} \log_{10} \frac{2.5}{0.7} = 0.955.$$

(a) Yield from the well of diameter 4 m :

$$A = \frac{\pi}{4} (4)^2 = 12.56 \text{ m}^2$$

$$Q = \left(\frac{K}{A} \right) A.H. = 0.955 \times 12.56 \times 3 = 36 \text{ m}^3/\text{hour} = 10 \text{ lit/sec.}$$

(b) Yield = 8 lit/sec = 28.8 m³/hour (since 1 lit/sec = 3.6 m³/hour)

$$Q = \left(\frac{K}{A} \right) A.H.$$

$$A = \frac{Q}{H} \times \left(\frac{A}{K} \right) = \frac{28.8}{2} \times \frac{1}{0.955} = 15.08 \text{ m}^2$$

$$d = 4.38 = 4.4 \text{ m (say).}$$

Example 5.3. A tube well of 30 cm diameter penetrates fully in an artesian aquifer. The strainer length is 15 m. Calculate the yield from the well under a drawdown of 3 m. The aquifer consists of sand of effective size of 0.2 mm having coefficient of permeability equal to 50 m/day. Assume radius of drawdown equal to 150 metres.

Solution

From Eq. 5.6(c), the discharge is given by

$$Q = \frac{2.72 b k s}{\log_{10} \frac{R}{r}}$$

where $b = \text{thickness of aquifer} = \text{length of strainer} = 15 \text{ m}$

$s = \text{drawdown} = 3 \text{ m}$

$k = \text{coefficient of permeability} = 50 \text{ m/day}$

$R = 150 \text{ m}$

$r = 15 \text{ cm} = 0.15 \text{ m}$

$$Q = \frac{2.72 \times 15 \times 50 \times 3}{\log_{10} \frac{150}{0.15}} \text{ m}^3/\text{day}$$

$$= 2040 \text{ m}^3/\text{day} = 85 \text{ m}^3/\text{hour} = 23.6 \text{ lit/sec.}$$

Example 5.4. A tube well penetrates fully an unconfined aquifer. Calculate the discharge from the tube well under the following conditions :

Diameter of the well	= 30 cm
Drawdown	= 2 m
Effective length of the strainer under the above drawdown	= 10 m
Coefficient of permeability of aquifer	= 0.05 cm/sec.
Radius of zero drawdown	= 300 m.

Solution

From Eq. 5.5, discharge is given by

$$Q = \frac{2.72 ks \left(L + \frac{s}{2} \right)}{\log_{10} \frac{R}{r}}$$

where $k = 0.05 \text{ cm/sec} = \frac{0.05}{100} \text{ m/sec} = 5 \times 10^{-4} \text{ m/sec}$

$s = \text{drawdown} = 2 \text{ m}$

$L = \text{effective length of strainer} = 10 \text{ m}$

$R = 300 \text{ m} ; r = 0.15 \text{ m}$

$$Q = \frac{2.72 \times 5 \times 10^{-4} \times 2(10 + 1)}{\log_{10} \frac{300}{0.15}} \text{ m}^3/\text{sec} = 9.06 \times 10^{-3} \text{ m}^3/\text{sec}$$

$$\triangleq 0.0091 \text{ cumec} = 9.06 \text{ lit/sec.}$$

Example 5.5. Design a tube well for the following data :

- (i) Yield required = 0.08 cumec
- (ii) Thickness of confined aquifer = 30 m
- (iii) Radius of circle of influence = 300 m
- (iv) Permeability coefficient = 60 m/day
- (v) Drawdown = 5 m.

Solution

From Eq. 5.6(c), we have

$$Q = \frac{2.72 b k s}{\log_{10} \frac{R}{r}}$$

Here $Q = 0.08 \text{ cumec} = 0.08 \text{ m}^3/\text{sec} ; b = 30 \text{ m} ; s = 5 \text{ m}$

$$k = 60 \text{ m/day} = \frac{60}{3600 \times 24} \text{ m/sec}$$

$$R = 300 ; r = ?$$

Hence
$$0.08 = \frac{2.72 \times 30 \times 5}{\log_{10} \frac{R}{r}} \times \frac{60}{3600 \times 24}$$

$$\log_{10} \frac{R}{r} = \frac{2.72 \times 30 \times 5 \times 60}{3600 \times 24 \times 0.08} = 3.5417$$

$$\frac{R}{r} = \text{antilog } 3.5417 = 3480$$

$$r = \frac{R}{3480} = \frac{3000}{3480} = 0.862 \text{ m.}$$

Hence adopt well of 9 cm diameter.

Example 5.6. An artesian tube well has a diameter of 20 cm. The thickness of aquifer is 30 m and its permeability is 38 m/day. Find its yield under a drawdown of 4 m at the well face. Use radius of influence as recommended by Sichardt.

Solution

From Eq. 5.6, we have

$$Q = \frac{2.72 b k s}{\log_{10} \frac{R}{r}}$$

Here $b = 30 \text{ m} ; s = 4 \text{ m} ; r = 0.10 \text{ m}$

$$k = 38 \text{ m/day} = \left(\frac{38}{24 \times 3600} \right) \text{ m/sec}$$

$$R = 3000 s \sqrt{k} = 3000 \times 4 \sqrt{\frac{38}{24 \times 3600}} = 245 \text{ m.}$$

$$Q = \frac{2.72 \times 30 \times 38 \times 4}{\log_{10} \frac{245}{0.1}} = 3467 \text{ m}^3/\text{day} = 144.5 \text{ m}^3/\text{hour} = 40.1 \text{ lit/sec (or 0.04 cumec)}$$

Example 5.7. A well penetrates fully a 10 m thick water bearing stratum of medium sand having coefficient of permeability of 0.005 m/sec. The well radius is 10 cm and is to be worked under a drawdown of 4 m at the well face. Calculate the discharge from the well. What will be the percentage increase in the discharge if the radius of the well is doubled ?

Take $R = 300 \text{ m}$ in each case.

Solution

From Eq. 5.6(c), we have

$$Q = \frac{2.72 b k s}{\log_{10} \frac{R}{r}} = \frac{2.72 \times 10 \times 0.005 \times 4}{\log_{10} \frac{300}{r}} = \frac{0.544}{\log_{10} \frac{300}{r}}$$

(i) In the first case $r = 10 \text{ cm} = 0.1 \text{ m}$.

$$Q_1 = \frac{0.544}{\log_{10} \frac{300}{0.1}} = \frac{0.544}{3.477} = 0.1565 \text{ cumec}$$

(ii) In the second case, $r = 20 \text{ cm} = 0.2 \text{ m}$.

$$Q_2 = \frac{0.544}{\log_{10} \frac{300}{0.2}} = \frac{0.544}{3.176} = 0.1713 \text{ cumec}$$

$$\therefore \% \text{ increase in discharge} = \frac{0.1713 - 0.1565}{0.1565} \times 100 = 9.5\%$$

Thus the size of the well has very little influence on the discharge. By doubling the well diameter, the discharge has increased by 9.5% only.

Example 5.8. In order to determine the field permeability of a free aquifer, pumping out test was performed and following observations were made :

Diameter of well = 20 cm

Discharge from the well = 240 m³/hour

R.L. of original water surface, before pumping started = 240.5 m

R.L. of water in the well at constant pumping = 235.6 m

R.L. of the impervious layer = 210 m

R.L. of water in observation well = 239.8 m.

Radial distance of observation well from the tube well = 50 m

Calculate k . Also calculate (i) the error in k if observations are not taken in the observation well, and radius of influence is assumed to be 300 m, (ii) actual radius of influence based on the observations of observation well.

Solution. (Refer Fig. 5.7)

(a) From Eq. 5.4 (b), we have

$$Q = \frac{1.36 k (h_2^2 - h_1^2)}{\log_{10} \frac{r_2}{r_1}}$$

Using suffix 1 for the well face, and suffix 2 for the observation well, we have

$$r_1 = 10 \text{ cm} = 0.1 \text{ m}; \quad r_2 = 50 \text{ m}$$

$$h_1 = 235.6 - 210 = 25.6 \text{ m}; \quad h_2 = 239.8 - 210 = 29.8 \text{ m}$$

$$\therefore (h_1 + h_2) = 29.8 + 25.6 = 55.4 \text{ m}; \quad (h_2 - h_1) = 29.8 - 25.6 = 4.2 \text{ m.}$$

$$Q = 240 \text{ m}^3/\text{hour.}$$

$$240 = \frac{1.36 \times k (55.4 \times 4.2)}{\log_{10} \frac{50}{0.1}}$$

$$\text{or} \quad k = \frac{240}{1.36 \times 55.4 \times 4.2} \log_{10} 500 = 2.047 \text{ m/hour} = 49.13 \text{ m/day} \dots (a)$$

(b) If k is computed on the basis of assumed radius of influence we have, from Eq. 5.4,

$$Q = \frac{1.36 k (H^2 - h^2)}{\log_{10} \frac{R}{r}}$$

Here

$$\frac{R}{r} = \frac{300}{0.1} = 3000$$

$$H = 240.5 - 210 = 30.5 \text{ m}; \quad h = 235.6 - 210 = 25.6 \text{ m}$$

$$H + h = 30.5 + 25.6 = 56.1 \text{ m}$$

$$H - h = 30.5 - 25.6 = 4.9 \text{ m.} \dots (b)$$

$$240 = \frac{1.36 k (56.1)(4.9)}{\log_{10} 3000}$$

$$\therefore k = \frac{240}{1.36 \times 56.1 \times 4.9} \log_{10} 3000 = 2.236 \text{ m/hour} = 53.67 \text{ m/day}$$

$$\therefore \% \text{ error} = \frac{53.67 - 49.15}{49.15} \times 100 = 9.1\%$$

(c) Calculation of actual radius of influence :

From Eq. 5.4,

$$Q = \frac{1.36 k (H^2 - h^2)}{\log_{10} \frac{R}{r}}$$

where

$$k = 2.047 \text{ m/hour}$$

[From (a)]

$$H + h = 56.1 \text{ m}$$

[From (b)]

$$H - h = 4.9 \text{ m}$$

$$Q = 240 \text{ m}^3/\text{hour.}$$

$$240 = \frac{1.36 \times 2.047 (56.1)(4.9)}{\log_{10} \frac{R}{r}}$$

$$\log_{10} \frac{R}{r} = \frac{1.36 \times 2.047 \times 56.1 \times 4.9}{240} = 3.186$$

$$\frac{R}{r} = 1544$$

$$R = 1544 \times r = 1544 \times 0.1$$

$$R \approx 154 \text{ m.}$$

or

Example 5.9. A tube well commands 20 hectares of land under rice. When the well is discharging at a constant rate, the water level in the well drops to 120.2 m. The level to the highest land is 129 m. The duty for rice is 800 hectares/cumec on the field. Taking the pump efficiency as 60 % calculate the input h.p. of the pump.

Solution

$$\text{Discharge required} = \frac{20}{800} = 0.025 \text{ cumec.}$$

∴ Weight of water lifted per second = $0.025 \times 1000 \text{ kg} = 25 \text{ kg}$.

Minimum static lift of the pump = $129 - 120.2 = 8.8 \text{ m}$

Taking 0.2 m head required for the water to flow from the well to the field, design lift = 9 m (say).

∴ Work done by pump in lifting the water = $9 \times 25 = 225 \text{ kg-m/sec}$.

∴ Now 1 h.p.(metric) = 75 kg-m/sec.

∴ Output h.p. of pump = $\frac{225}{75} = 3$

∴ Input h.p. pump = $\frac{3}{\eta \text{ of pump}} = \frac{3}{0.6} = 5 \text{ h.p.}$

Example 5.10. The discharge available from a tube well is $150 \text{ m}^3/\text{hour}$. Assuming 4000 hours of working for a tube well in a year, estimate the culturable area that this tube well can command. The intensity of irrigation is 45 % and average depth of Rabi and Kharif crop is 38 centimetres.

Solution

Volume of water pumped = $150 \times 4000 \text{ m}^3 = 60 \times 10^4 \text{ cubic metres}$.

Average depth of water = 38 cm = 0.38 m

∴ Area of irrigation = $\frac{60 \times 10^4}{0.38} = 1.58 \times 10^6 \text{ m}^2$

Culturable commanded area = $\frac{1.58 \times 10^6}{0.45} = 3.51 \times 10^6 \text{ m}^2 = 351 \text{ hectares}$.

Example 5.11. Two tube wells, each of 20 cm diameter are spaced at 100 m distance. Both the wells penetrate fully a confined aquifer of 12 m thickness. Calculate the discharge if only one well is discharging under a depression head of 3 m. What will be the percentage decrease in the discharge of the well if both the wells are discharging under the depression head of 3 m. Take radius of influence for each well equal to 250 metres, and coefficient of permeability of aquifer as 60 metres/day.

Solution. (Refer Fig. 5.12)

In the first case, when only one well is discharging, the discharge is given by Eq. 5.6 (c),

$$Q = \frac{2.72 b k s}{\log_{10} \frac{R}{r}}$$

where $b = 12 \text{ m}$; $s = 3 \text{ m}$; $k = 60 \text{ m/day}$

$$\log_{10} \frac{R}{r} = \log_{10} \frac{250}{0.1} = 3.398$$

$$Q = \frac{2.72 \times 12 \times 60 \times 3}{3.398} \text{ m}^3/\text{day} \\ = 1729 \text{ m}^3/\text{day} \approx 72.0 \text{ m}^3/\text{hour}.$$

When both the wells are discharging, the discharge from each well is given by Eq. 5.4.

$$Q_1 = Q_2 = \frac{2 \pi k b (H - h)}{\log_{10} \frac{R^2}{rB}} \\ = \frac{2.72 k b s}{\log_{10} \frac{R^2}{rB}} \\ = \frac{2.72 \times 60 \times 12 \times 3}{\log_{10} \left(\frac{250 \times 250}{0.1 \times 100} \right)} \\ = \frac{2.72 \times 60 \times 12 \times 3}{3.796} = 1548 \text{ m}^3/\text{day} \\ = 64.5 \text{ m}^3/\text{hour}.$$

∴ Percentage decrease in the discharge

$$= \frac{72 - 64.5}{72} \times 100 = 10.43 \%$$

Example 5.12. A gravity well has a diameter of 60 cm. The depth of water in the well is 40 metres before pumping is started. When pumping is being done at the rate of 2000 litres per minute, the drawdown in a well 10 metres away is 4 metres and in another well 20 metres away is 2 metres. Determine (a) radius of zero drawdown, (b) coefficient of permeability, (c) drawdown in the well, (d) specific capacity of the well, (e) maximum rate at which water can be pumped from the well.

Solution

(a) For a tube well in unconfined aquifer, we have from Eq. 5.4,

$$Q = \frac{1.36 k (H^2 - h^2)}{\log_{10} \frac{R}{r}}$$

Here, we have $H = 40$; At $r = 10 \text{ m}$, $h = 40 - 4 = 36 \text{ m}$.

At $r = 20 \text{ m}$, $h = 40 - 2 = 38 \text{ m}$.

Applying the above equation at these two locations, we have

$$\frac{40^2 - 36^2}{\log_{10} \frac{R}{10}} = \frac{(40^2 - 38^2)}{\log_{10} \frac{R}{20}} \\ \log_{10} \frac{R}{20} = \frac{(40^2 - 38^2)}{40^2 - 36^2} \log_{10} \frac{R}{10} = 0.5132 \log_{10} \frac{R}{10} \\ \log_{10} \frac{R}{20} = \log_{10} \left(\frac{R}{10} \right)^{0.5132}$$

or

or

or

$$\frac{R}{20} = \frac{R^{0.5132}}{3.2599}$$

$$R^{1-0.5132} = \frac{20}{3.2599} = 6.1352$$

$$R = (6.1352)^{\frac{1}{0.4868}} = (6.1352)^{2.0542}$$

$$R = 41.52 \text{ m}$$

(b) $Q = 2000 \text{ litres/min} = 2 \text{ m}^3/\text{min}$ (since $1 \text{ litres/min} = 0.001/\text{m}^3/\text{min}$).

$$R = 41.52 \text{ m}; H = 40 \text{ m},$$

$$h = 36 \text{ m}; r = 10 \text{ m}$$

$$Q = 2 = \frac{1.36 k (40^2 - 36^2)}{\log_{10} \frac{41.52}{10}}$$

$$k = \frac{2}{1.36 (40^2 - 36^2)} \log_{10} \frac{41.52}{10} = 0.003 \text{ m/min} = 4.31 \text{ m/day.}$$

(c) Depth of water in the well is given by

$$2 = \frac{1.36 \times 0.003 (40^2 - H_s^2)}{\log_{10} \frac{41.52}{0.30}}$$

$$1600 - H_s^2 = \frac{2}{1.36 \times 0.003} \times \log_{10} \frac{41.52}{0.30} = 1049.58.$$

$$H_s = 23.46 \text{ m}$$

Hence drawdown at the well $= 40 - 23.46 = 16.54 \text{ m}$.

(d) **Specific Capacity.** It is defined as the discharge per unit drawdown. Let it be designated by S_c . It is not constant, but decreases as the discharge increases. Let us assume that the yield is directly proportional to the drawdown or to the radius of zero-drawdown.

$$Q \propto R$$

or $Q = CR$ where C is a constant.

For the given data,

$$C = \frac{Q}{R} = \frac{2}{41.52} = 0.04817.$$

$$R = \frac{Q}{C} = \frac{Q}{0.04817} \text{ in general.}$$

Now corresponding to drawdown of 1 m, discharge $Q = S_c$.

$$\text{Hence radius of zero drawdown} = \frac{S_c}{C} = \frac{S}{0.04817}$$

$$\text{Hence } Q = S_c = \frac{1.36k (40^2 - 39^2)}{\log_{10} \frac{R}{r}} = \frac{1.36 \times 0.003 (40^2 - 39^2)}{\log_{10} \left(\frac{S_c}{0.04817 \times 0.3} \right)}$$

$$\text{or } S_c \log_{10} (61.2 S_c) = 0.3223.$$

Solving this by trial and error, we get

$$S_c = 0.266 \text{ m}^3/\text{min.}$$

Hence, specific capacity of the well is $0.266 \text{ m}^3/\text{min}/\text{m}$ depression head.(e) Maximum rate of discharge Q_m will be obtained when drawdown in the well is equal to H , i.e., when H_s is zero.

$$Q_m = \frac{1.36 \times 0.003 (40^2 - 0^2)}{\log_{10} \frac{Q_m}{0.04817 \times 0.3}}$$

$$\text{or } Q_m \log_{10} 69.2 Q_m = 6.528.$$

Solving this by trial and error, we get

$$Q_m = 2.85 \text{ m}^3/\text{min.}$$

5.21. OTHER SOURCES OF UNDER GROUND WATER

The ground water can be tapped with the help of the following sources :

1. Wells.
2. Springs.
3. Infiltration galleries.
4. Karez.

Out of these, well have been discussed at length in the previous articles. We shall now discuss in brief the other sources of ground water.

Springs

Ground water which flows out at the surface constitutes a spring. A pervious layer, sandwiched between two impervious layers, give rise to a natural spring. A spring indicates the outcropping of the water table. Springs are of the following types :

1. Stratum spring.
2. Valley springs.
3. Fault springs.
4. Springs from solution channels.
5. Artesian spring (geyser).
6. Deep seated spring.

Stratum spring is formed when the downward passage of ground water in a permeable deposit is hindered by an underlying impervious layer. The ground water emerges as a spring at the contact of the two formations, and hence such spring is also known as *contact spring* (Fig. 5.35). Another kind of stratum spring may arise at the junction of two formations where permeable beds dip beneath an impermeable cover. This may be called an *overflow spring*, since the ground water literally overflows at the edge at the impervious stratum (Fig. 5.36).

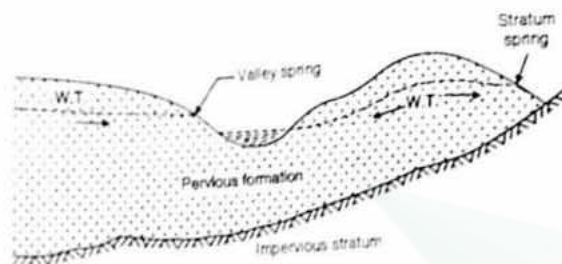


FIG. 5.35. SPRINGS

A **Valley spring** is due to the emergence of the water table where the topography falls below its level such as that shown in Fig. 5.35. Water flows out at the surface and feeds the surface stream.

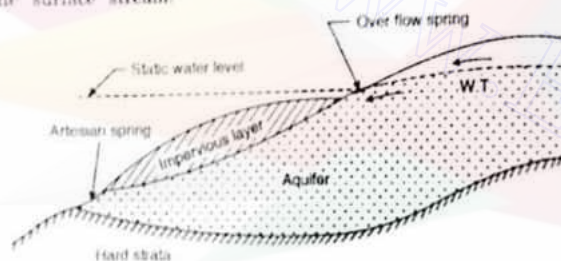


FIG. 5.36. SPRING

When a permeable formation is brought against an impermeable one by faulting, ground water in the permeable bed may escape along the fault line and appear at the surface as a **fault spring** (Fig. 5.37).

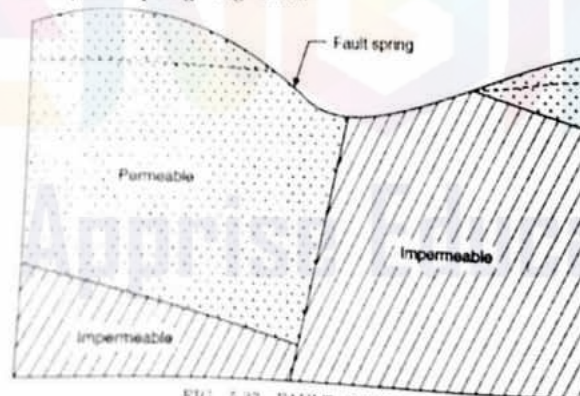


FIG. 5.37. FAULT SPRING.

In limestone district where there is much underground water in circulation, streams flow along bedding planes in channels enlarged by solutions, and may emerge as springs in valleys, to feed surface stream.

Artesian spring arise where water is confined in permeable rocks under artesian head beneath a cover of impervious layer (Fig. 5.36). If this cover is locally broken by fracturing, or if the artesian aquifer outcrops, the artesian water below emerges as an artesian spring. Artesian springs, also known as *geysers*, are of infrequent occurrence.

The **deep seated springs** are associated with vulcanicity and deep fractures, and yield juvenile water and not the rainwater.

Infiltration Galleries. It is a small tunnel in the form of pipes laid under the ground to tap up underground flow available at moderate depth. These tunnels could be of wood, stone or bricks. Their shape is generally circular, and they are covered

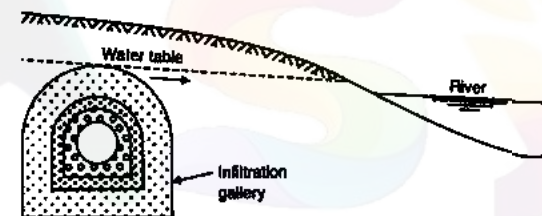


FIG. 5.38. INFILTRATION GALLERY.

by graded filters all round. The ground water can enter the tunnel through side and bottom. These galleries are often constructed parallel to a river bank to collect the inflow of ground water to river. Fig. 5.38 shows a typical infiltration gallery surrounded by a three layer graded filter consisting of :

- (i) 16 cm ballast (25 to 50 mm size) around the pipe,
- (ii) 16 cm large pebbles (12 to 24 mm) as intermediate layer and
- (iii) 24 cm fine pebbles (3 to 10 mm size) as the outer layer.

Karez

A Karez is an underground tunnel driven into the hill side to tap water from the underground spring. The tunnel has certain bed inclination. Water from the Karez

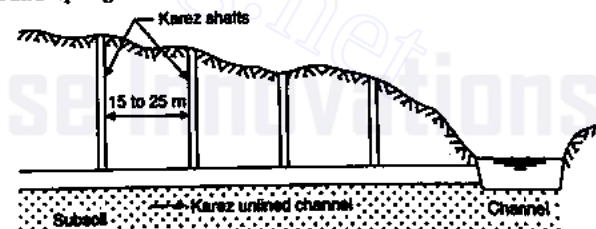


FIG. 5.39 KAREZ.

is taken in an open channel. Such tunnels are in use in Baluchistan and North West Frontier provinces (West Pakistan).

5.22. UNSTEADY FLOW TOWARDS WELLS*

The analysis of flow towards well discussed earlier is based on the assumption that steady state of flow is developed immediately after pumping is started. Actually, the cone of depression fluctuates with time. The gradual approach of cone of depression towards a steady state is produced primarily by the removal of water from storage as cone deepens. Hence, a storage coefficient comes into play. The storage coefficient S is a dimensionless constant of the aquifer and may be interpreted as the amount of water in storage released from storage from a column of aquifer of unit cross-section under a unit decline of head.

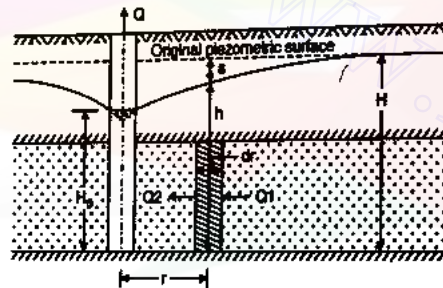


FIG. 5.40. UNSTEADY FLOW.

Equations developed for unsteady or transient well flow normally show how the drawdowns of the piezometric surface or water table is related to time of pumping the well. In Fig. 5.40, consider an annular cylinder of thickness dr and radius r . Due to unsteady flow conditions, water will be released from storage of this elementary cylinder. According to definition of storage coefficient S , the rate $\frac{\partial V}{\partial t}$ at which a certain volume V is released from storage of an aquifer of area A is given by

$$\frac{\partial V}{\partial t} = -\frac{\partial h}{\partial t} S \cdot A \quad \dots(1)$$

where V = volume of water released per horizontal area A of aquifer.
 h = height of piezometric surface or water table above lower boundary of aquifer.
 S = storage co-efficient.

A = area of the aquifer to which $\frac{\partial V}{\partial t}$ applies.
 t = time.

*Junior students may omit this article.

Since h decreases with time t , minus sign with $\partial h / \partial t$ has been used. For the elementary cylinder of the aquifer, $A = 2\pi r \cdot dr$.

$$\text{Hence} \quad \frac{\partial V}{\partial t} = -2\pi r \cdot dr \cdot S \frac{\partial h}{\partial t} \quad \dots(2)$$

Let Q_1 = discharge entering the outer face of the cylinder
 Q_2 = discharge leaving the inner face of the cylinder.
 $\therefore$ Change in discharge $\delta Q = Q_1 - Q_2$.

Since the rate of increase in Q can be expressed as $-\frac{\partial Q}{\partial r}$, the increase in discharge over the annular area $= -\frac{\partial Q}{\partial r} dr$. Substituting this for $\frac{\partial V}{\partial t}$ in (2), we get

$$-\frac{\partial Q}{\partial r} dr = -2\pi r \cdot dr \cdot S \frac{\partial h}{\partial t} \quad \text{or} \quad \frac{\partial Q}{\partial r} = 2\pi r S \frac{\partial h}{\partial t} \quad \dots(3)$$

For confined radial flow, we get from Darcy law,

$$Q = 2\pi r T \frac{\partial h}{\partial r}$$

$$\therefore \frac{\partial Q}{\partial r} = 2\pi T \frac{\partial}{\partial r} \left(r \frac{\partial h}{\partial r} \right) = 2\pi T \left(\frac{\partial h}{\partial r} + r \frac{\partial^2 h}{\partial r^2} \right) \quad \dots(4)$$

Substituting in (3), we get

$$2\pi T \left(\frac{\partial h}{\partial r} + r \frac{\partial^2 h}{\partial r^2} \right) = 2\pi r S \frac{\partial h}{\partial t}$$

$$\text{which simplifies to} \quad \frac{\partial^2 h}{\partial r^2} + \frac{1}{r} \frac{\partial h}{\partial r} = \frac{S}{T} \frac{\partial h}{\partial t} \quad \dots(5.24)$$

This is the basic equation for unsteady flow towards the well. Theis (1935) obtained a solution for this equation based on the analogy between ground water flow and heat conduction, by assuming that the well is replaced by a mathematical sink of constant strength such that $h = H$ before pumping begins and that $h \rightarrow H$, as $r \rightarrow \infty$ after pumping begins ($t \geq 0$). The solution is

$$H - h = \frac{Q}{4\pi T} \int_{u=\frac{r^2 S}{4Tt}}^{\infty} \frac{e^{-u}}{u} du \quad \dots[5.25(a)]$$

$$\text{or} \quad H - h = s = \frac{Q}{4\pi T} W(u) \quad \dots(5.25)$$

$$\text{where} \quad W(u) = \int_u^{\infty} \frac{e^{-u}}{u} du = \text{well function.}$$

Eq. 5.25 is known as the non-equilibrium equation or Theis equation. The integral in the above equation is a function of the lower limit, and is known as exponential integral. It can be expressed as a convergent series so that Eq. 5.25 can also be written as

$$s = \frac{Q}{4\pi T} \left[-0.5772 - \log_e u + u - \frac{u^2}{2 \cdot 2!} + \frac{u^3}{3 \cdot 3!} - \dots \right] \quad \dots [5.25(b)]$$

Wenzel tabulated the values of $W(u)$ for various values of u ranging from 10^{-15} to 9.9. Table 5.2 gives the values of $W(u)$ for u ranging from 10^{-15} to 9.

TABLE 5.2. VALUES OF WELL FUNCTION $W(u)$
($u = N \times 10^n$)

$N \downarrow \quad u \rightarrow$	$N \times 10^{-15}$	$N \times 10^{-14}$	$N \times 10^{-13}$	$N \times 10^{-12}$	$N \times 10^{-11}$	$N \times 10^{-10}$	$N \times 10^{-9}$	$N \times 10^{-8}$
1	33.96	31.66	29.36	27.05	24.75	22.45	20.15	17.84
2	33.27	30.97	28.66	26.36	24.06	21.76	19.45	17.15
3	32.86	30.56	28.26	25.96	23.65	21.35	19.05	16.75
4	32.57	30.27	27.97	25.67	23.36	21.06	18.76	16.46
5	32.35	30.05	27.75	25.44	23.14	20.84	18.54	16.23
6	31.17	29.87	27.50	22.26	22.96	20.66	18.35	16.05
7	32.02	29.71	27.40	25.11	22.81	20.50	18.20	15.90
8	31.88	29.58	27.28	24.97	22.67	20.37	18.07	15.76
9	31.76	29.46	27.26	24.86	22.55	20.25	17.95	15.65

$N \downarrow \quad u \rightarrow$	$N \times 10^{-7}$	$N \times 10^{-6}$	$N \times 10^{-5}$	$N \times 10^{-4}$	$N \times 10^{-3}$	$N \times 10^{-2}$	$N \times 10^{-1}$	N
1	15.54	13.24	10.94	8.633	8.332	4.038	1.823	0.2194
2	14.85	12.55	10.24	7.940	5.639	3.355	1.223	0.04891
3	14.44	12.14	9.837	7.535	5.235	2.959	0.906	0.01315
4	14.15	11.85	9.550	7.247	4.948	2.681	0.702	0.003779
5	13.93	11.68	9.326	7.024	4.726	2.468	0.560	0.001148
6	13.75	11.45	9.144	6.842	4.545	2.295	0.454	0.000360
7	13.60	11.29	8.990	6.688	4.392	2.151	0.374	0.000116
8	13.46	11.17	8.856	6.555	4.259	2.027	0.311	0.0000377
9	13.34	11.04	8.739	6.437	4.142	1.919	0.260	0.0000125

From the Table,

When $u = 5$, $W(u) = 0.001148$; and when $u = 5 \times 10^{-3}$, $W(u) = 4.726$.

The values of formation constant S and T can be found by measuring drawdowns in observation wells when the well under study is pumped at a constant rate of discharge Q . There are several methods of determining S and T , but we will discuss here following three methods:

- Theis method,
- Jacob's method,
- Chow's method.

(a) THEIS METHOD

Theis proposed a curve-fitting method for finding formation constant S and T from a pumping out test. From Eq. 5.25, we observe that

$$s = \left[\frac{Q}{4\pi T} \right] W(u) \text{ or } \log W(u) = \left[\log \frac{4\pi T}{Q} \right] + \log s \quad \dots [5.26(a)]$$

$$\text{and} \quad \frac{r^2}{t} = \left[\frac{4T}{S} \right] u \quad \text{or} \quad \log u = \left[\log \frac{S}{4T} \right] + \log \frac{r^2}{t} \quad \dots [5.26(b)]$$

If a constant withdrawal rate (Q) is maintained, the bracketed portions of the above two equations are constant for a given pumping test. It is to be noted that s is related to r^2/t in a manner that is similar to the relation of $W(u)$ to u . Hence, if a plot of data curve is made between s and r^2/t on logarithmic co-ordinates tracing paper [Fig. 5.41(a)] to the same scale as the type curve $W(u)$ versus u [Fig. 5.41(b)] the data curve will be similar to the type curve.

Procedure

- In the observation well situated at a radial distance r from the main well, observe s and t .
- Plot s versus r^2/t on a log-log tracing paper [Fig. 5.41(a)].
- Plot $W(u)$ versus u on log-log graph paper [Fig. 5.41(b)].
- Keep Fig. 5.41(a) on Fig. 5.41(b), and adjust in such a way that when the corresponding co-ordinate axes are held parallel, the data curve is oriented in position which represents the best fit of the field data to the type curve, [Fig. 5.41(c)].
- With both graph sheets at the best match position, an arbitrary point P is selected on the type curve and pricked through.
- The co-ordinates (a, b) and (a_1, b_1) of the match point are noted from both the curves.

Thus, the pair of values $[s, W(u)]$ and $(r^2/t, u)$ are known. The aquifer constants are then calculated from the relation

$$T = \frac{Q}{4\pi s} W(u) \quad \dots [5.27(a)]$$

$$\text{and} \quad S = \frac{4uT}{r^2/t} \quad \dots [5.27(b)]$$

(b) JACOB'S METHOD

Jacob suggested method which completely avoids curve fitting. He observed that when r is small and t is large, u may be small and hence all terms after 2nd term of the series expansion [Eq. 5.25(b)] of $W(u)$ may be neglected.

Thus $W(u) = -0.5772 - \log_e u$

$$\therefore s = \frac{Q}{4\pi T} \left[-0.5772 - \log_e u \right] \quad \text{or} \quad s = \frac{Q}{4\pi T} \left[\log_e \frac{4Tt}{r^2 S} - 0.5772 \right] \quad \dots [5.28]$$

For the same observation well, if s_1 and s_2 are the observed drawdowns at times t_1 and t_2 since pumping started, we have

$$\Delta s = s_2 - s_1 = \frac{2.303 Q}{4\pi T} \log_{10} \frac{t_2}{t_1} \quad \dots [5.29]$$

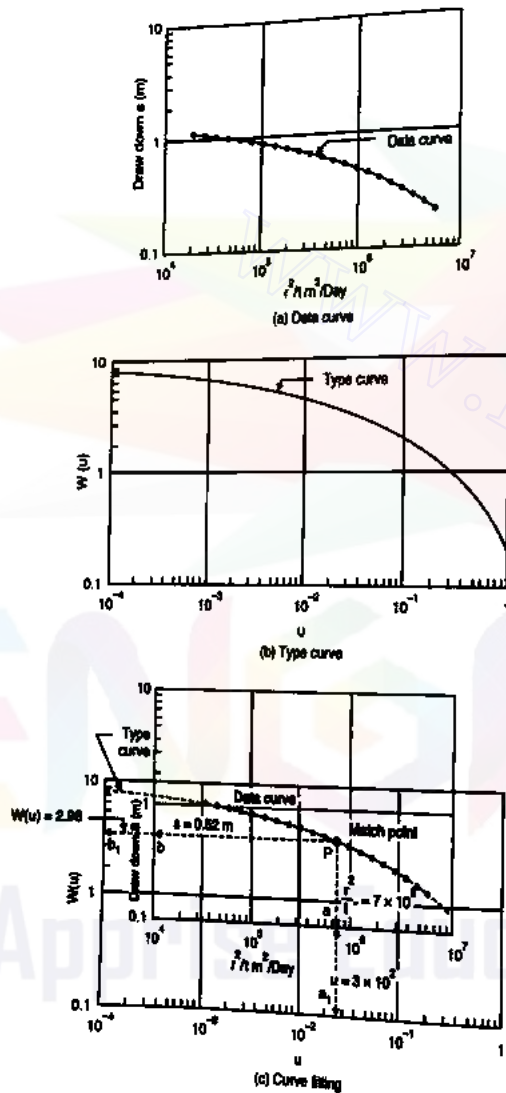


FIG. 5.41. THEIS METHOD.

If a plot is made between s and $\log_{10} t$ (Fig. 5.42) then for one log cycle of time difference (i.e., $t_2 = 10 t_1$), we get

$$\Delta s = \frac{2.303 Q}{4\pi T}$$

$$T = \frac{2.303 Q}{4\pi \Delta s}$$

... (5.30)

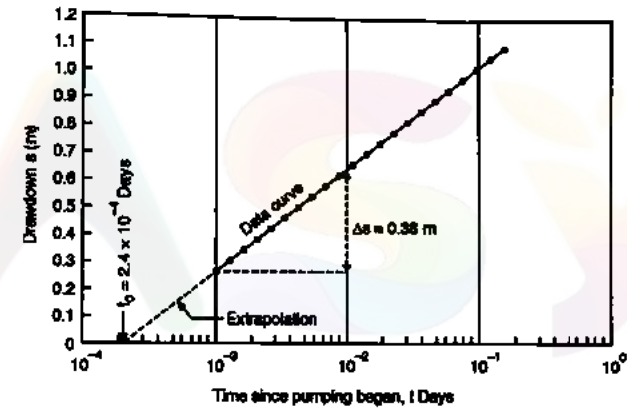


FIG. 5.42. JACOB'S METHOD.

Extrapolating the straight line of the curve to intersect with the zero drawdown axis permits the calculation for S . Let t_0 be the time (since pumping started), at which the drawdown in observation well is still zero, we have from Eq. 5.28 :

$$s = 0 = \frac{Q}{4\pi T} \left[\log_e \frac{4T t_0}{r^2 S} - 0.5772 \right]$$

or

$$\log_e \frac{4T t_0}{r^2 S} = 0.5772$$

or

$$\frac{4T t_0}{r^2 S} = e^{0.5772}$$

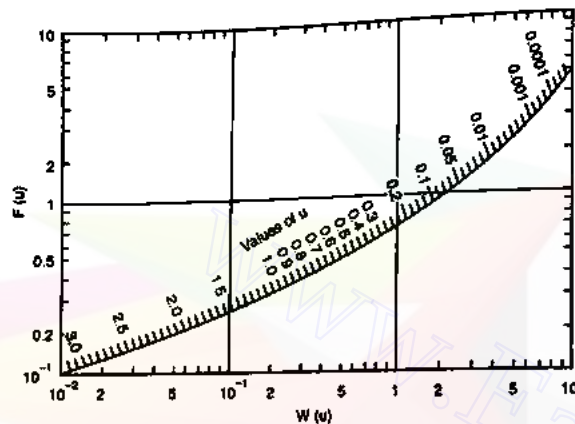
which gives

$$S = \frac{2.25 T t_0}{r^2} \quad \dots (5.31)$$

Thus storage-coefficient S can be calculated from Eq. 5.31. The straight line approximation for this method should be restricted to values of u less than about 0.01 to avoid large errors.

(c) CHOW'S METHOD

Chow gave a method of solution which avoids curve fitting. He introduced a function $F(u)$ given by the relation :

FIG. 5.43. FUNCTION $F(u)$.

$$F(u) = \frac{W(u) \cdot e^u}{2.303} \quad \dots(5.32)$$

The relation between $F(u)$, $W(u)$ and u is shown in Fig. 5.43 with the u scale on the curve. The function $F(u)$ is introduced to relate $W(u)$ and u to a certain combination of s and t . The procedure is as follows :

1. Select an observation well near the pumped well and observe the drawdown (s) at all times.
2. Plot a graph between s and $\log_{10} t$ as shown in Fig. 5.44. Join the points by a smooth curve.
3. On the plotted graph, choose an arbitrary point P and note its co-ordinates s and t .
4. Draw the tangent of curve at the chosen point P and determine the drawdown difference Δs per log cycle of time.
5. Compute the function $F(u)$ by the relation

$$F(u) = \frac{s}{\Delta s} \quad \dots(5.33)$$

6. Knowing $F(u)$ and using Fig. 5.43, find u and $W(u)$.
7. Compute S and T from the following relations :

$$T = \frac{Q}{4\pi s} W(u) \quad \text{and} \quad S = \frac{4uTt}{r^2}$$

Note. For small values of u , we have from Jacob's method :

$$\Delta s = \frac{2.303 Q}{4\pi T} \log_{10} \frac{t_2}{t_1} = \frac{2.303 Q}{4\pi T} \quad (\text{when } t_2 = 10 t_1)$$

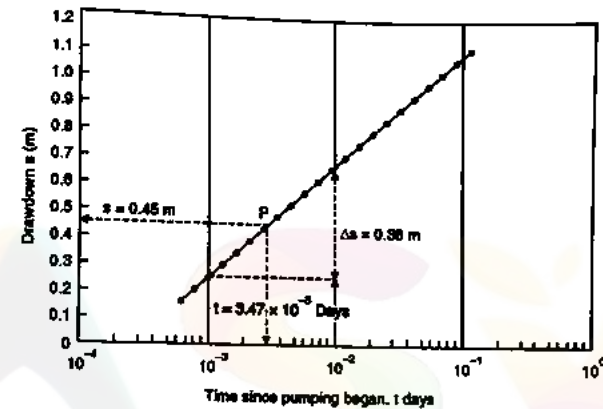


FIG. 5.44. CHOW'S METHOD.

Also

$$s = \frac{Q}{4\pi T} W(u)$$

$$F(u) = \frac{s}{\Delta s} = \frac{W(u)}{2.303} \quad \dots[5.32(a)]$$

For $F(u) < 2$, the above equation can safely be used. For $F(u) > 2$, u becomes large. Hence Eq. 5.32 should be used.

Example 5.12. Theis Method. A well fully penetrating a confined aquifer is pumped at a uniform rate of 2500 litres per minute. The drawdown in an observation well situated at 60 m away are given in Table 5.3. Using Theis method, determine the formation constants of the aquifer (Adapted from data from U.S. Geological Survey).

TABLE 5.3

Time since Pumping began t (min)	Drawdown s (m)	t (min)	s (m)	t (min)	s (m)
0	0.00	12	0.59	160	1.03
1.0	0.20	14	0.62	180	1.06
1.5	0.26	18	0.66	210	1.08
2.0	0.30	24	0.71	240	1.10
2.5	0.33	30	0.75		
3	0.36	40	0.80		
4	0.41	60	0.83		
5	0.45	80	0.86		
6	0.48	80	0.91		
8	0.53	100	0.95		
10	0.56	120	0.98		

Solution

The values of t , s and r^2/t are tabulated in Table 5.4. A plot is made between s and r^2/t as shown in Fig. 5.41 (a) on a log-log tracing paper. The type curve $[W(u)]$ versus u is shown in Fig. 5.41 (b), prepared from data of Table 5.2.

TABLE 5.4. COMPUTATION FOR DATA CURVE ($r = 60$ m)

Time t		r^2/t m ² /day	s (m)	Time t		r^2/t m ² /day	s (m)
min.	days			min.	days		
0	0	∞	0.0	18	12.5×10^{-2}	2.88×10^5	0.66
1.0	6.94×10^{-4}	5.18×10^6	0.20	24	1.67×10^{-2}	2.16×10^5	0.71
1.5	1.04×10^{-3}	3.46×10^6	0.26	30	2.08×10^{-2}	1.73×10^5	0.76
2.0	1.39×10^{-3}	2.59×10^6	0.30	40	2.78×10^{-2}	1.30×10^5	0.80
2.5	1.74×10^{-3}	2.07×10^6	0.33	50	3.47×10^{-2}	1.04×10^5	0.83
3.0	2.08×10^{-3}	1.73×10^6	0.36	60	4.17×10^{-2}	8.64×10^4	0.86
4	2.78×10^{-3}	1.30×10^6	0.41	90	5.56×10^{-2}	6.48×10^4	0.91
5	3.47×10^{-3}	1.04×10^6	0.45	100	6.94×10^{-2}	5.18×10^4	0.96
6	4.17×10^{-3}	8.64×10^5	0.48	120	8.33×10^{-2}	4.32×10^4	0.98
8	5.56×10^{-3}	6.48×10^5	0.53	150	1.04×10^{-1}	3.46×10^4	1.03
10	6.94×10^{-3}	5.18×10^5	0.56	180	1.25×10^{-1}	2.88×10^4	1.06
12	8.33×10^{-3}	4.32×10^5	0.59	210	1.46×10^{-1}	2.47×10^4	1.08
14	9.72×10^{-3}	3.70×10^5	0.62	240	1.67×10^{-1}	2.16×10^4	1.10

The data curve of Fig. 5.41(a) is superimposed on the type curve and properly oriented to get the best possible fit. A match point P is picked [Fig. 5.41(c)] and its co-ordinates are found as follows :

$$s = 0.52 \text{ m} ; W(u) = 2.96$$

$$\frac{r^2}{t} = 7 \times 10^5 ; u = 3 \times 10^{-2}$$

Discharge $Q = 2500 \text{ litres/min} = 2.5 \text{ m}^3/\text{min} = 3600 \text{ m}^3/\text{day}$

$$T = \frac{Q}{4\pi s} W(u) = \frac{3600}{4 \times \pi \times 0.52} \times 2.96 = 1631 \text{ m}^3/\text{day/m}$$

and $S = \frac{4uT}{r^2/t} = \frac{4 \times 3 \times 10^{-2}}{7 \times 10^5} \times 1631 = 0.00028.$

Example 5.13. Jacob's Method. Solve example 5.12 by Jacob's method.

Solution

Fig. 5.42 shows the plot between s and t . Most of the points lie on a straight line. The drawdown Δs for one log cycle of time difference comes out to be 0.38 m. Hence from Eq. 5.30,

$$T = \frac{2.303 Q}{4\pi \Delta s} = \frac{2.303 \times 3600}{4\pi \times 0.38} = 1738 \text{ m}^3/\text{day/m}.$$

Extending the straight line so as to cut the time axis, we get

$$t_0 = 2.4 \times 10^{-4} \text{ days}$$

$$S = \frac{2.25 T t_0}{r^2} = \frac{2.25 \times 1738 \times 2.4 \times 10^{-4}}{(60)^2} = 0.00028.$$

Example 5.14. Chow's Method. Solve Example 5.12 by Chow's method.

Solution

The plot between s and t is shown in Fig 5.44. Choose any point P whose co-ordinates are $s = 0.45$ m and $t = 3.47 \times 10^{-3}$ days. Since the curve is a straight line, the tangent to the curve at P will coincide with the straight line itself. For one log cycle of time, we get

$$F(u) = \frac{s}{\Delta s} = \frac{0.45}{0.38} = 1.184$$

From Fig. 5.34, corresponding to $F(u) = 1.184$, we get

$$W(u) = 2.7 \text{ and } u \approx 0.04$$

Alternatively, from Eq. 5.32(a)

$$W(u) = 2.303 F(u) = 2.303 \times 1.184 = 2.727,$$

and from Table 5.2, $u = 0.035$

Hence

$$T = \frac{Q}{4\pi s} W(u) = \frac{3600}{4\pi \times 0.45} \times 2.727 = 1738 \text{ m}^3/\text{day/m}$$

$$S = \frac{4uT}{r^2} = \frac{4 \times 0.035 \times 3.47 \times 10^{-3}}{3600} \times 1738 = 0.000234.$$

5.23. EXAMPLES FROM COMPETITIVE EXAMINATIONS

Example 5.15. A 30 cm diameter well penetrates 25 m below the static water table. After 24 hours of pumping @ 5400 litres/minute, the water level in a test well at 90 m is lowered by 0.53 m, and in a well 30 m away the drawdown is 1.11 m. (a) what is the transmissibility of the aquifer ? (b) Also determine the draw down in the main well.

(Engg. Services Exam. 1968)

Solution :

For the present case of well penetrating an unconfined aquifer, we have :

$$Q = \frac{\pi k (h_1^2 - h_2^2)}{\log_e r_2/r_1} \quad \dots(5.4)$$

Here, depth of static water level = $H = 25$ m

At $r = r_1 = 30$ m, $s_1 = 1.11$ m. Hence $h_1 = H - s_1 = 25 - 1.11 = 23.89$ m

Also, at $r_2 = 90$ m, $s_2 = 0.53$ m. Hence $h_2 = H - s_2 = 25 - 0.53 = 24.47$ m

$$Q = 5400 \text{ lpm} = 5400 \times 10^{-3}/60 \text{ m}^3/\text{s} = 0.09 \text{ m}^3/\text{s}.$$

$$k = \frac{Q \log_e r_2/r_1}{\pi (h_2^2 - h_1^2)} = \frac{0.09 \log_e (90/30)}{\pi [(24.47)^2 - (23.89)^2]} = 1.1221 \times 10^{-3} \text{ m/s}$$

$$\text{Hence } T = kH = 1.1221 \times 10^{-3} \times 25 = 0.0281 \text{ m}^2/\text{s}$$

Applying Eq. 5.4. at the well face, we have $r_1 = r_w$ and $h_1 = h_w$

$$Q = \frac{\pi k (h_2^2 - h_w^2)}{\log_e r_2/r_w} \text{ where } r_w = 0.3/2 = 0.15 \text{ m}$$

$$h_2^2 - h_w^2 = \frac{Q \log_e r_2/r_w}{\pi k} = \frac{0.09 \log_e (90/0.15)}{\pi \times 1.1221 \times 10^{-3}} = 163.317$$

$$\text{or } h_w^2 = h_2^2 - 163.317 = (24.47)^2 - 163.317 = 435.46$$

$$\text{or } h_w = 20.87 \text{ m}$$

Hence draw down at well face, $s_w = H - h_w = 25 - 20.87 = 4.13 \text{ m}$

Example 5.16. 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 homogenous aquifer, estimate the discharge at 18 metres drawdown. The distance from the well where the drawdown influences are not appreciable may be taken equal for both the cases.

(Engg. Services Exam. 1969)

Solution :

For an unconfined aquifer, we have from Eq. 5.4

$$Q = \frac{\pi k (H^2 - h^2)}{\log_e R/r}$$

In the first case, $Q = Q_1 = 250 \text{ lpm}$, $h_1 = H - s_1 = 100 - 12 = 88 \text{ m}$

$$\frac{\log_e R/r}{\pi k} = \frac{(H^2 - h_1^2)}{Q_1} = \frac{(100)^2 - (88)^2}{250} = 9.024 \text{ units.}$$

$$\text{In the second case, } Q = Q_2 = \frac{\pi k}{\log_e R/r} (H^2 - h_2^2) = \frac{[(100)^2 - (82)^2]}{9.024} = 363 \text{ lpm}$$

Example 5.17. A horizontal aquifer is of 10 m thickness and infinite areal extent with its top level 25 m below ground level. Static piezometric surface is 10 m below ground level. During steady rate of pumping from the well at 5000 m³/day, the steady drawdown at the well is 12 m. Permeability of aquifer formation is 48 m/day. Assume the radius of influence as 500 m. Find the effective well diameter, in metres.

(Gate, 1988)

Solution

For a confined aquifer, we have, from Eq. 5.6

$$Q = \frac{2\pi b k s}{\log_e R/r}$$

Here $Q = 5000 \text{ m}^3/\text{day}$; $b = 10 \text{ m}$; $k = 48 \text{ m/day}$; $R = 500 \text{ m}$; $s = 12 \text{ m}$

Substituting the values, we get

$$\log_e R/r = \frac{2\pi b k s}{Q} = \frac{2\pi (10 \times 48 \times 12)}{5000} = 7.2383$$

$$R/r = 1391.63$$

$$\therefore \text{Dia. of well} = D_w = 2r = \frac{2R}{1391.63} = \frac{2 \times 500}{1391.63} \approx 0.72 \text{ m}$$

Example 5.18. A well is located in a 30 m thick confined aquifer of permeability 35 m/day and storage coefficient of 0.004. If the well is pumped at the rate of 1500 litres per minutes, calculate the drawdown at a distance of 40 m from the well after 20 hours of pumping. (Civil Services Exam. 1991)

Solution

This is the case of unsteady flow towards a well in confined aquifer. The draw down (s) in an observation well at $r = 40 \text{ m}$ and $t = 20 \text{ hours} = 20 \times 3600 \text{ seconds}$ after pumping started is given by Jacob's equation

$$s = \frac{Q}{4\pi T} \left[\log_e \frac{4Tt}{r^2 S} - 0.5772 \right] \quad \dots(5.28)$$

where $Q = 1500 \text{ lpm} = 1500 \times 10^{-3}/60 \text{ m}^3/\text{s} = 0.025 \text{ m}^3/\text{s}$

$S = \text{storage coefficient} = 0.004$

$$T = bk = 30 \times 35 \text{ m}^2/\text{day} = \frac{30 \times 35}{60 \times 60 \times 24} \text{ m}^2/\text{s} = 0.01215 \text{ m}^2/\text{s}$$

$$t = 20 \text{ hours} = 20 \times 3600 \text{ seconds} = 72000 \text{ seconds}$$

Substituting the values in eq. 5.28, we get

$$s = \frac{0.025}{4\pi (0.01215)} \left[\log_e \frac{4 \times 0.01215 \times 72000}{(40)^2 \times 0.004} - 0.5772 \right] = 0.94 \text{ m}$$

Example 5.19. During a recuperation test, the water in an open well was depressed by pumping by 2.5 m and it recuperated 1.8 m in 80 minutes. Calculate the yield from a well of 4 m diameter under a depression head of 3 m.

(Engg Services Exam. 1993)

Solution : This is exactly the same as example 5.2.

For solution, refer Example 5.2, part (a).

Example 5.20. A 30 cm well completely penetrates an unconfined aquifer of depth 40 m. After a long period of pumping at a steady rate of 1500 lpm, the drawdown in two observation wells 25 m and 75 m from the pumping well were found to be 3.5 m and 2.0 m respectively. Determine the transmissibility of the aquifer. What is the drawdown at the pumping well? (Civil Services Exam. 1994)

Solution : From Eq. 5.4, we have

$$Q = \frac{\pi k (h_2^2 - h_1^2)}{\log_e r_2/r_1}$$

If s_1 and s_2 are the drawdowns in the two wells, and if $H (= 40 \text{ m})$ is the depth of water before pumping we have

$$h_1 = H - s_1 = 40 - 3.5 = 36.5 \text{ m and } h_2 = H - s_2 = 40 - 2 = 38 \text{ m}$$

$$\text{Also, } r_1 = 25 \text{ m, } r_2 = 75 \text{ m and } Q = 1500 \times 10^{-3} \times 60 = 0.025 \text{ m}^3/\text{s}.$$

Substituting the values, we get

$$k = \frac{Q \log_e r_2/r_1}{\pi (h_1^2 - h_2^2)} = \frac{0.025 \log_e 75/25}{\pi [(38)^2 - (36.5)^2]} = 7.82 \times 10^{-5} \text{ m/sec.}$$

$$T = bk = 40 \times 7.82 \times 10^{-5} = 3.128 \times 10^{-3} \text{ m}^2/\text{sec}$$

Again, applying Eq. 5.4 at the well face, making $h_1 = h_w$ and $r_2 = 0.15 \text{ m}$, we get

$$Q = \frac{\pi k (h_2^2 - h_w^2)}{\log_e r_2/r_w}$$

Substituting the values, we get

$$h_w^2 - h_2^2 = \frac{Q \log_e r_2/r_w}{\pi k} = \frac{0.025 \log_e (75/0.15)}{\pi (7.82 \times 10^{-5})} = 632.4$$

$$h_w^2 = h_2^2 - 632.4 = (38)^2 - 632.4 = 811.59$$

$$\text{From which } h_w = 28.48 \text{ m}$$

$$\therefore \text{Drawdown at well face, } s_w = H - h_w = 40 - 28.48 = 11.52 \text{ m}$$

Example 5.21. An aquifer of 20 m average thickness is overlain by an impermeable 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 drawdown in the test well.

(Engg. Services Exam. 1995)

Solution : The arrangements is shown in Fig. 5.45. From Eq. 5.6 (a)

$$Q = \frac{2.72 kb (h_2 - h_1)}{\log_{10} \frac{r_2}{r_1}}$$

$$h_2 = H - s_2 \text{ and } h_1 = H - s_1$$

$$\therefore h_2 - h_1 = (H - s_2) - (H - s_1) = s_1 - s_2$$

$$\text{Hence } Q = \frac{2.72 bk (s_1 - s_2)}{\log_{10} \frac{r_2}{r_1}} \quad \dots(5.6c)$$

Substituting the values, we get

$$k = \frac{Q \log_{10} r_2/r_1}{2.72 b (s_1 - s_2)} = \frac{0.1 \log_{10} (60/10)}{2.72 \times 20 (4 - 3)} = 1.43 \times 10^{-5} \text{ m/sec}$$

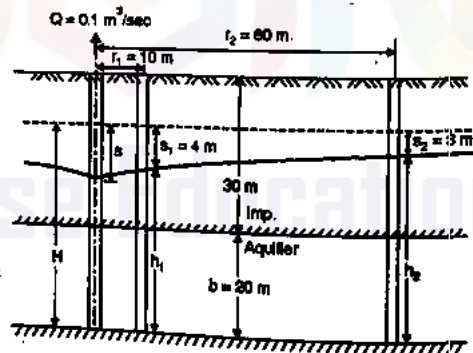


FIG. 5.45

Again, making $s_1 = s$ = draw down at well face and $r_1 = r$ = radius of well, Eq. 5.6 (c) reduces to the form

$$Q = \frac{2.72 bk (s - s_2)}{\log_{10} \frac{r_2}{r}}$$

$$s - s_2 = \frac{Q \log_{10} r_2/r}{2.72 bk} = \frac{0.1 \log_{10} (60/0.25)}{2.72 \times 20 \times 1.43 \times 10^{-5}} \approx 3.06$$

$$s = 3.06 + s_2 = 3.06 + 3 = 6.06 \text{ m}$$

Example 5.22. 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 well to yield 10 litres/sec. under a depression head of 2 m.

(Engg. Services Exam. 1996)

Solution : This problem is similar to Example 5.2.

The specific yield (K/A) of the well is given by Eq. 5.21

$$\frac{K}{A} = \frac{1}{T} \log_e \frac{h_1}{h_2}$$

where T = time in hours $\approx 90/60 = 1.5 \text{ h}$; $h_1 = 2.1 \text{ m}$; $h_2 = 2.1 - 1.6 = 0.5 \text{ m}$

$$\frac{K}{A} = \frac{1}{1.5} \log_e \frac{2.1}{0.5} = 0.9567$$

$$\text{Required yield, } Q = 10 \text{ litres/sec} = \frac{10 \times 3600}{1000} = 36 \text{ m}^3/\text{hour}$$

$$\text{Now } Q = \left(\frac{K}{A} \right) A H, \text{ where } H = 2 \text{ m}$$

$$\text{From which } A = \frac{Q}{(K/A) H} = \frac{36}{0.9567 \times 2} = 18.8147 \text{ m}^2$$

$$\therefore d = \left(\frac{4}{\pi} A \right)^{1/2} = \left(\frac{4}{\pi} \times 18.8147 \right)^{1/2} = 4.89 \approx 4.9 \text{ m}$$

Example 5.23. During a recuperation test, the water level in an open well was depressed by pumping upto 2.5 m. The water level was raised by 1 m within 60 minutes just after stopping the pumping. Determine the yield from the well of 2 m diameter when the depression head is 3 m.

(Engg. Services Exam. 2001)

Solution : From Eq. 5.21, we have

$$\frac{K}{A} = \frac{1}{T} \log_e \frac{h_1}{h_2}$$

Here $T = 60 \text{ min.} = 1 \text{ h}$; $h_1 = 2.5 \text{ m}$; $h_2 = 2.5 - 1 = 1.5 \text{ m}$

$$\frac{K}{A} = \frac{1}{1} \log_e \frac{2.5}{1.5} = 0.5108$$

$$\text{Now } Q = KH = \left(\frac{K}{A} \right) A H = 0.5108 \left(\frac{\pi}{4} \times 2^2 \right) \times 3 = 4.814 \text{ m}^3/\text{hr}$$

PROBLEMS

- Define the following terms :
Aquifer, aquiclude, specific yield, piezometric surface, water table, perched aquifer.
- Show, with the help of sketches, various types of wells.
- Explain the terms 'storage coefficient' and 'coefficient of transmissibility'.
- State and discuss assumptions and limitations of Dupuit's theory.
- Derive an expression for discharge from a well in unconfined aquifer. The well fully penetrates it.
- Derive an expression for discharge from a well fully penetrating a confined aquifer.
- Explain the method of determining the coefficient of transmissibility of a confined aquifer by pumping out test. How can this method be extended for unconfined aquifer?
- Write notes on the following :
(i) well loss, (ii) specific capacity of well, (iii) spherical flow in well, (iv) interference among wells.
- Describe various types of tube wells.
- Describe with the help of sketches, some of the common types of strainers used in tube wells.
- Explain the percussion method of drilling a tube well.
- What do you understand by well shrouding?
- Describe in brief various methods of developing a tube well.
- What do you understand by recuperation test? Derive the equations used in the test.
- Distinguish clearly between a shallow well and a deep well. How does a deep well differ from a tube well in confined aquifer?
- The following observations were recorded during a pumping out test on a tube well penetrating fully in a aquifer :
Well diameter : 25 cm
Discharge from the the well : $300 \text{ m}^3/\text{hour}$
R.L. of original water surface, before pumping started = 122.0 m
R.L. of water in the well at constant pumping = 117.1 m
R.L. of water in the observation well = 121.3 m
R.L. of impervious layer = 92.0 m
Radial distance of observation well from the tube well = 50 m
Determine : (a) the field permeability coefficient of the free aquifer and (b) radius of zero drawdown.
[Ans. (a) 60.7 m/day , (b) 167 m]
- A tube well commands 40 hectares of land under wheat. When the well is discharging at constant rate, the water level in the well drops to R.L. 160.2 m. The R.L. of highest land is 166.5 m. The duty for wheat is 1600 hectares/cumec. Taking the pump efficiency as 75% calculate the input H.P. of the pump.
[Ans. 2.89 H.P.]
- Design a tube well for the following data :
(i) Yield required = 0.2 cumec
(ii) Thickness of confined aquifer = 40 m

- Radius of circle of influence = 300 m
- Permeability coefficient = 80 m/day
- Drawdown = 6 m.

[Ans. 28 cm, or say 30 cm]

- During a recuperation test, the water in open well was depressed by pumping by 2 m and it recuperated 1.5 m in 1 hour. Estimate the yield from a well of 2 m diameter under a depression head of 2 m situated in the same area. Derive the expression you use.
[Ans. $8.7 \text{ m}^3/\text{hour}$]

- A tube well penetrates fully a 8 m thick water bearing stratum (confined) of medium sand having coefficient of permeability of 0.004 m/sec. The well radius is 15 cm and is to be worked under a drawdown of 8 m at the well face. Calculate the discharge from the well. What will be the percentage increase in the discharge if the radius of the well is doubled? Take radius of zero drawdown equal to 400 m in each.

[Ans. (i) $275 \text{ m}^3/\text{hour}$, (ii) 9.6%]

- Design an open well in fine sand to give a discharge of 0.005 cumec when worked under a depression head of 3 metres. Take the value of the specific yield for fine sand as $0.5 \text{ m}^3/\text{hour}$ per square metre of area, under unit depression head.

[Ans. Dia. 3.9 m]

Reservoir Planning

6.1. INTRODUCTION

Dams are constructed across the rivers and streams to create an artificial lake or reservoir behind it. Dams and reservoirs are the most important and expensive elements in multi-purpose river basin development. They require very careful planning, design, and operation. A number of problems arise in design, construction and operation of dams and reservoirs; for example, selection of site, the relative merits of different types of dams, storage capacity and optimum yield and co-ordinated use of storage for different purposes.

Storage works are constructed to serve many purposes, which include :

1. Storage and control of water for irrigation
2. Storage and diversion of water for domestic uses
3. Water supplies for industrial uses
4. Development of hydroelectric power
5. Increasing water depths for navigation
6. Storage space for flood control
7. Reclamation of low-lying lands
8. Debris control
9. Preservation and cultivation of useful aquatic life
10. Recreation.

Depending upon the purposes served, reservoirs may be classified as under :

- (i) Storage or conservation reservoirs
- (ii) Flood protection reservoirs
- (iii) Distribution reservoirs
- (iv) Multipurpose reservoirs.

(i) **Storage or Conservation Reservoir** : Storage reservoirs are primarily used for water supplies for irrigation, hydroelectric development, domestic and industrial supplies. A river does not carry the same quantity of water throughout the year, and may carry large quantities in the other part of the year. A storage reservoir is constructed to store the excess water during the period of large supplies, and release it gradually as and when it is needed.

(306)

RESERVOIR PLANNING

307

(ii) **Flood Control Reservoirs** : Flood control or flood protection reservoirs are those which store water during flood and release it gradually at a safe rate when the flood reduces. By the provision of artificial storage during the floods, flood damage downstream is reduced. There are two pillars in flood prevention planning : river bed stabilizing planning for stabilizing the river channel from the upper stream down to the estuary and flood disposition planning for causing the flood water to flow down without doing any damage. Flood control by means of dam is a part of the flood disposition planning covering the whole river system and, sometimes it shares part of river-bed stabilization planning.

Thus, in Fig. 6.1 ABC is the natural hydrograph at the dam site, having a maximum flood discharge Q_1 . By the construction of the dam, the natural hydrograph is moderated by the reservoir, as shown by the dotted lines (AB'C). Thus, the flood discharge is reduced from Q_1 to Q_2 . The area shown hatched represents the storage to be provided in the reservoir. The portion marked by dots represents the excess volume released later.

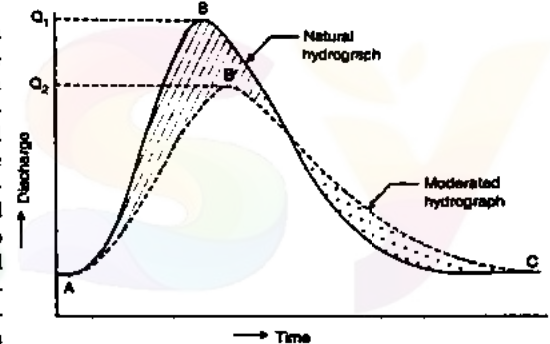


FIG. 6.1. HYDROGRAPH MODERATED BY THE RESERVOIR

(iii) **Distribution Reservoir** : A distribution reservoir is a small storage reservoir used for water supply in a city. A distribution reservoir accounts for the varying rate of water during the day. Such distribution reservoir permits the pumping plants and water-treatment works etc., to operate at a constant rate. The varying demand rate, exceeding to constant pumping rate, is met from the distribution reservoir.

(iv) **Multipurpose Reservoir** : A multipurpose reservoir is that which serves more than one purpose. For example a reservoir designed to protect the down stream area from floods, and to store water for irrigation and hydroelectric purposes is a multipurpose reservoir.

6.2. INVESTIGATIONS FOR RESERVOIR PLANNING

The following investigations are required for reservoir planning :

1. Engineering surveys.
2. Geological investigations.
3. Hydrological investigations.

1. ENGINEERING SURVEYS

The area at the dam site is surveyed in detail and a contour plan is prepared.

From the plan, the following physical characteristics are prepared :

- (a) Area-elevation curve.
- (b) Storage-elevation curve.
- (c) Map of the area to indicate the land property to be surveyed

(d) Suitable site selection for the dam.

Area-elevation and Storage-elevation Curves.

Fig. 6.2 shows a typical contour plan at the reservoir site. The hatched area shows the water spread area. The area A_1, A_2, A_3 enclosed by the successive contours can be determined with a planimeter. A close observation of Fig. 6.2 shows that as the value of a contour (i.e., elevation) increases, its area increases. A curve, such as curve AB shown in Fig. 6.3, may be drawn between elevation and area.

The reservoir capacity, or the volume of storage, corresponding to a given water level in the reservoir may be calculated either by trapezoidal formula or by prismoidal formula. Thus, if V is the storage volume and h is the contour interval, the formulae are:

$$(1) \quad V = \Sigma \frac{h}{2} (A_1 + A_2) \dots$$

(Trapezoidal formula)

$$= \left\{ \frac{A_1 + A_n}{2} + A_2 + A_3 \dots A_{n-1} \right\} \dots (6.1)$$

$$(2) \quad V = \Sigma \frac{h}{3} (A_1 + A_2 + \sqrt{A_1 A_2}) \dots \dots \dots \text{(Cone formula)} \dots (6.2)$$

$$(3) \quad V = \frac{h}{3} \{ (A_1 + A_n) + 4(A_2 + A_4 + \dots) + 2(A_3 + A_5 + \dots) \} \dots \text{(Prismoidal formula)} \dots (6.3)$$

where A_n is the area of the contour corresponding to the water surface elevation in the proposed reservoir. The volumes of storage corresponding to various water-surface elevation may be calculated and a curve, such as CD in Fig. 6.3, may be plotted between elevation and storage.

The contour plan (Fig. 6.2) also indicates the water spread corresponding to the reservoir elevation and enables us to determine the area under submergence and the compensation to be paid to the owners.

2. GEOLOGICAL INVESTIGATIONS

In almost all major civil engineering projects, geological advice is the most essential. Geological investigations cost very little in comparison to the total cost of the project; in a number of cases in recent years it has amounted to between 1/4 to 1 percent of the whole. This relatively small amount represents a valuable insurance against expensive difficulties, otherwise unforeseen, which might arise during construction but



FIG. 6.2. A TYPICAL CONTOUR PLAN.

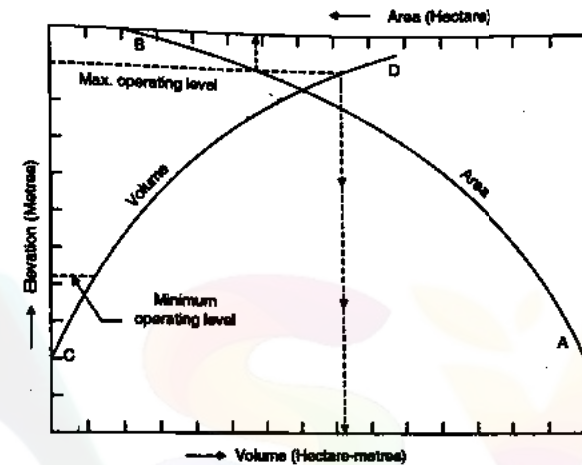


FIG. 6.3. ELEVATION AREA AND ELEVATION-STORAGE CURVES.

which it may be possible to predict from a study of the geological factors involved. Geological investigations are required to give detailed information about the following items :

1. Water tightness of reservoir basin.
2. Suitability of foundations for the dam.
3. Geological and structural features, such as folds, faults, fissures etc. of the rocks basin.
4. Type and depth of over burden (superficial deposits).
5. Location of permeable and soluble rocks, if any.
6. Ground water conditions in the region.
7. Location of quarry sites for materials required for the dam construction and quantities available from them.

The geology of the catchment area should also be studied, since it affects the proportions of run-off percolation. The special requirement for the geology of the reservoir site is that there should be no danger of serious leakage when the ground is under pressure from the full head of water in the reservoir. The geology of the dam site is important from the point of view of secure foundation for the dam. The nature of sub-surface geology should be explored by trial bores or various means of geophysical exploration.

3. HYDROLOGICAL INVESTIGATIONS

The hydrological investigation is a very important aspect of reservoir planning. The capacity of the irrigation canals and/or the installed capacity of the power houses will depend upon the available supplies from the reservoir. The hydrological investigations may be divided into two heads :

1. Study of run-off pattern at the proposed dam site, to determine the storage capacity corresponding to a given demand, and
2. Determination of the hydrograph of the worst flood, to determine the spillway capacity and design.

6.3. SELECTION OF SITE FOR A RESERVOIR

The final selection of site for a reservoir depends upon the following factors :

1. The geological condition of the *catchment area* should be such that percolation losses are minimum and maximum run-off is obtained.
2. The reservoir site should be such that quantity of leakage through it is a minimum. Reservoir site having the presence of highly permeable rocks reduce the water tightness of the reservoir. Rocks which are not likely to allow passage of water include shales and slates, schists, gneisses, and crystalline igneous rocks such as granite.
3. Suitable dam site must exist. The dam should be founded on sound watertight rock base, and percolation below the dam should be minimum. The cost of the dam is often a controlling factor in selection of a site.
4. The reservoir basin should have narrow opening in the valley so that the length of the dam is less.
5. The cost of real estate for the reservoir, including road, rail road, dwelling re-location etc. must be as less as possible.
6. The topography of the reservoir site should be such that it has adequate capacity without submerging excessive land and other properties.
7. The site should be such that a deep reservoir is formed. A deep reservoir is preferable to a shallow one because of (i) lower cost of land submerged per unit of capacity, (ii) less evaporation losses because of reduction in the water spread area, and (iii) less likelihood of weed growth.
8. The reservoir site should be such that it avoids or excludes water from those tributaries which carry a high percentage of silt in water.
9. The reservoir site should be such that the water stored in it is suitable for the purpose for which the project is undertaken. The soil and rock mass at the reservoir site must not contain any objectionable minerals and salts.

6.4. ZONES OF STORAGE IN A RESERVOIR

The following are various zones of storage in reservoir :

(1) Useful storage.	(2) Surcharge storage.
(3) Dead storage.	(4) Bank storage.
(5) Valley storage.	

The maximum level to which the water will rise in the reservoir during ordinary operation condition is called *normal pool level*. The normal pool level is corresponding to either the level of the spillway crest, or to the top level of the spillway gates. The level to which water rises during the design flood is known as the *maximum pool level*. The lowest elevation to which the water in the reservoir is to be drawn under ordinary operating conditions is known as the *minimum pool level*.

The volume of water stored between the normal pool level and the minimum pool level is known as the *useful storage*. The volume of water below the minimum pool level is known as the *dead storage* and is not useful under ordinary operating conditions. The volume of water stored between the normal pool level and the maximum level corresponding to a flood is called *surcharge storage*, and is usually uncontrolled. The terms *bank storage* and *valley storage* are referred to the volume of water stored in the pervious formations of the river banks and the soil above it. Such storage depends upon the geological conditions of river banks. The bank storage effectively increases the capacity of the reservoir above that indicated by the elevation-storage curve.

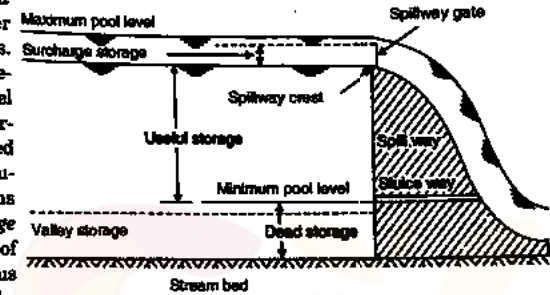


FIG. 6.4. ZONES OF STORAGE IN A RESERVOIR.

6.5. STORAGE CAPACITY AND YIELD

Yield. Yield is the amount of water that can be supplied from the reservoir in a specified interval of time. The interval of time chosen for the design varies from a day for small distribution reservoirs to a year for large conservation reservoirs. For example, if 25,000 cubic metres of water is supplied from a reservoir in one year, its yield is 25,000 cubic metres/year or 2.5 hectare-metres per year.

Safe yield or firm yield. The maximum quantity of water that can be guaranteed during a critical dry period is known as the safe yield or firm yield.

Secondary yield. Secondary yield is the quantity of water available in excess of safe yield during periods of high flood.

Average yield. The arithmetic average of the firm and the secondary yield over a long period of time is called average yield.

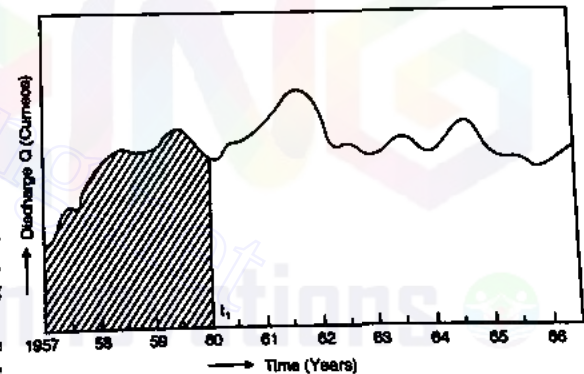


FIG. 6.5. FLOOD HYDROGRAPH OF INFLOW.

Mass inflow curve. The reservoir capacity corresponding to a specified yield is determined with the help of mass inflow curve and the demand curve. A mass inflow curve is a plot between the cumulative inflow in the reservoir with time.

Fig. 6.5 shows a flood hydrograph of inflow for several years. Fig. 6.6 shows a mass inflow curve prepared from the flood hydrograph of Fig. 6.5. Taking the starting year (i.e. 1957) as the base, the total quantity of water from 1957 to a time t_1 (say year 1960) that has flown through the river is the volume represented by the hatched area. In the mass inflow curve (Fig. 6.6), the corresponding ordinate at time t_1 (ordinate

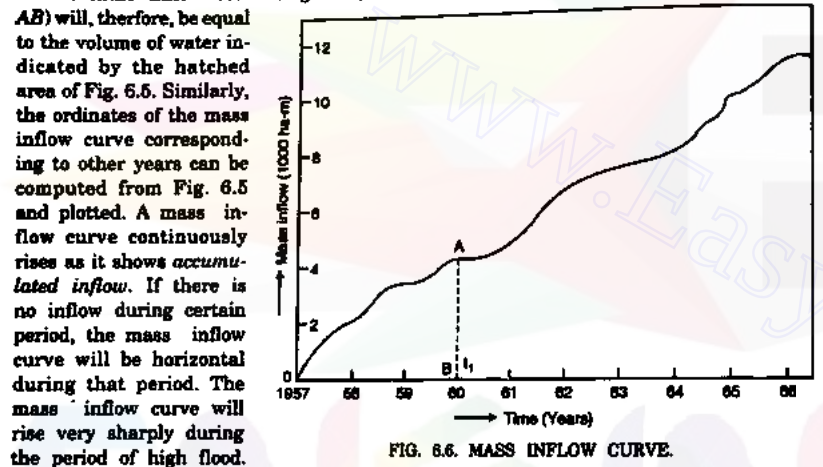


FIG. 6.6. MASS INFLOW CURVE.

Thus the steepness of the mass inflow curve shows the rate of inflow at that time interval. The hollows on the mass inflow curve show relatively dry periods.

Demand curve. A demand curve (Fig. 6.7) is a plot between accumulated demand with time. The demand curve representing a uniform rate of demand is a straight

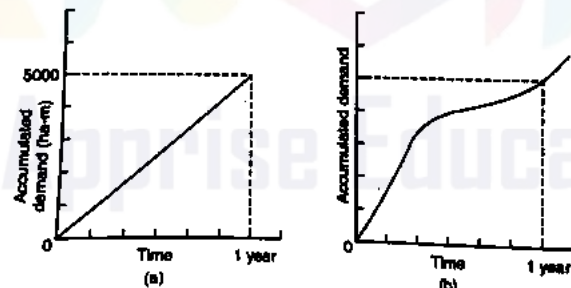


FIG. 6.7. DEMAND CURVES.

ine having the slope equal to the demand rate. A demand curve may be curved also, indicating variable rate of demand.

6.6. CALCULATION OF RESERVOIR CAPACITY FOR A SPECIFIED YIELD, FROM THE MASS INFLOW CURVE

Procedure (Fig. 6.8) :

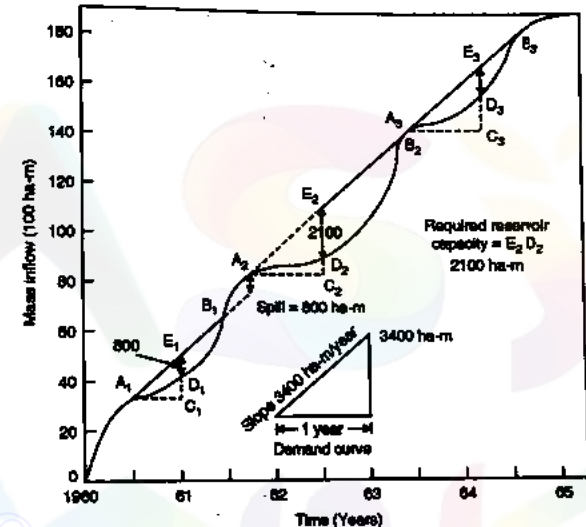


FIG. 6.8. DETERMINATION OF RESERVOIR CAPACITY.

1. From the flood hydrograph of inflow for several years, prepare the mass inflow curve. Also prepare the mass curve of demand on the same scale.
2. From the apices $A_1, A_2, A_3, \dots$ etc. of the mass inflow curve, draw tangents parallel to the demand curve.
3. Measure the maximum vertical intercepts $E_1D_1, E_2D_2, E_3D_3, \dots$ etc., between the tangent and the mass inflow curve. The vertical intercepts indicate the volume by which the inflow falls short of demand. For example, for a period corresponding to points A_1 and C_1 , C_1D_1 represents the net inflow while C_1E_1 represents the demand. Hence, the volume E_1D_1 has to be provided from the reservoir storage.
4. The biggest of the vertical ordinates amongst E_1D_1, E_2D_2, E_3D_3 etc. represent the required reservoir capacity.

It should be noted that the vertical distance between successive tangents represents water wasted over the spillway. The spillway must have sufficient capacity to discharge this flood volume.

Corresponding to the numerical figures indicated in Fig. 6.8, we observe that:

- (1) The required reservoir capacity is 2100 ha-m.
- (2) Assuming the reservoir to be full at A_1 it is depleted to $(2100 - 800) = 1300$ ha-m at D_1 and is again full at B_1 .
- (3) Assuming the reservoir to be full at A_2 it is empty at D_2 and is again full at B_2 .
- (4) The reservoir is full between B_1 and A_2 and the quantity of spill is 800 ha-m.

6.7. DETERMINATION OF SAFE YIELD FROM A RESERVOIR OF A GIVEN CAPACITY

The following is the procedure of determining the safe yield from a reservoir of a given storage capacity, with the help of a mass inflow curve :

- (1) Prepare the mass inflow curve. On the same diagram, draw straight lines, from a common origin, representing demands at various rates, say varying from 0 to 5000 ha-m per year.
- (2) From the apices A_1, A_2, A_3 etc. of the mass inflow curve, draw tangents in such a way that their maximum departure from the mass inflow curve does not exceed the specified reservoir capacity. Thus, in Fig. 6.9, the ordinates E_1D_1, E_2D_2, E_3D_3 , etc. are all equal to the reservoir capacity (say 1500 ha-m).

(3) Measure the slopes of each of these tangents. The slopes indicate the yield which can be attained in each year from the reservoir of given capacity. The slope of flattest demand line is the firm yield or safe yield.

6.8. SEDIMENT FLOW IN STREAMS : RESERVOIR SEDIMENTATION

All the rivers carry certain amount of silt eroded from the catchment area during heavy rains. The extent of erosion, and hence the silt load in the stream depends upon the following factors :

- (i) Nature of soil of the catchment area
- (ii) Topography of the catchment area
- (iii) Vegetation cover
- (iv) Intensity of rainfall.

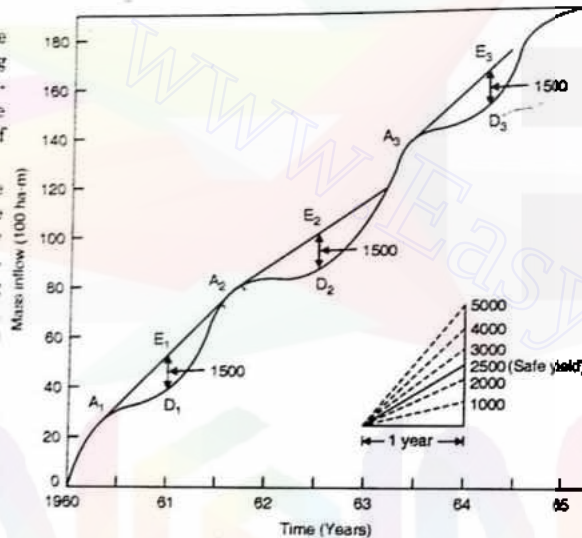


FIG. 6.9. DETERMINATION OF YIELD FROM RESERVOIR OF SPECIFIED CAPACITY

The nature of the soil of the catchment area is an important factor. If the soil is soft, there is always a possibility of *sheet erosion*. The tributaries collecting water of the catchment area containing hard soil carry lesser silt. Steep slopes give rise to high velocities and erode the surface soil easily. Similarly higher intensity of rainfall causes greater run-off and more erosion. If the catchment area has sufficient vegetation cover the higher velocities are checked and erosion is very much reduced. Area having poor or practically no vegetal cover are productive of more silt. The rivers or tributaries passing through such areas carry more silt load with it, causing quick silting of the reservoir.

The sediment transported by the river can be divided into two heads : (a) *Bed load*, and (b) *Suspended load*. The bed load is dragged along the bed of the stream. The suspended load is kept in suspension because of the vertical component of the

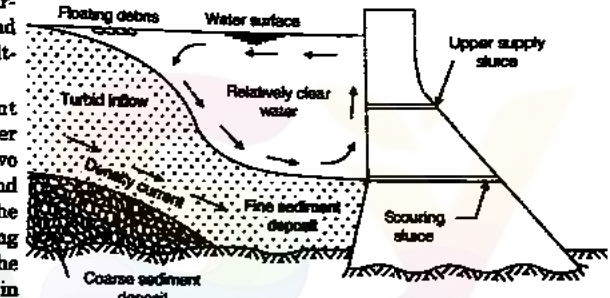


FIG. 6.10. RESERVOIR SEDIMENTATION.

eddies formed due to friction of flowing water against the bed. The bed load is generally much smaller — 10 to 15% of the suspended load. When the stream approaches the reservoir, the velocity is very much reduced. Due to this reduction, the coarser particles settle in the head reaches of the reservoir, while the finer particles are kept in suspension for sufficient time till they settle just to the upstream side of the dam, as shown in Fig. 6.10. Some fine particles may pass through sluice ways, turbines or spillways.

Density Currents : Density current may be defined as a gravity flow of fluid under another fluid of approximately equal density. In case of reservoirs the water stored is usually clear and the inflow during floods is generally muddy. The two fluids have, therefore, different densities and the heavy turbid water flows along the channel bottom towards the dam under the influence of gravity as shown in Fig. 6.10. This is known as density current. The rate of silting in case of reservoirs reduces if the density currents are vented by proper location and operation of outlet and sluice gates.

Measurement of Sediment load : The amount of silt or the sediment load carried by a stream is determined by taking the samples of water carrying silt, at various depths. The samples are then filtered and the sediment is removed and dried. The sediment load measured in the units of 'parts per million part of water' (ppm). There are no accurate devices to measure the bed load, which is estimated to about 15% of the suspended load.

Life of Reservoir : The ultimate destiny of a reservoir is to be filled with silt deposits. To allow for silting, a certain percentage of the total storage is usually left unutilised, and is called 'dead storage'. However, as the time passes on, more and more silting takes place and the 'live' or 'effective storage' is gradually reduced. The

useful life of reservoir is terminated when its capacity is reduced to 20 % of the designed capacity. The reservoir planning must, therefore, include the consideration of probable rate of silting so that the useful life of the reservoir may be determined.

The reservoir sedimentation is measured in terms of its *trap efficiency* (η). Trap efficiency of a reservoir is the percent of inflowing sediment which is retained in reservoir. Detailed observations show that the trap efficiency is a function of the ratio of reservoir capacity to the total inflow, i.e.

$$\eta = f\left(\frac{\text{Capacity}}{\text{Inflow}}\right)$$

Fig. 6.11 shows a plot between trap efficiency and capacity inflow ratio on the basis of the existing reservoirs. It is clear from the above curve that for a given inflow rate, the trap efficiency decreases with the reduction in reservoir capacity due to sediment deposit. Hence the rate of silting is higher in the initial stages, and it decreases as silting takes place. Thus the complete filling of a reservoir with silt may take a very long time.

At the same time, a small reservoir (having small capacity) on a large stream (having large inflow rates) has a very small (Capacity / Inflow) ratio. The trap efficiency for such a reservoir is extremely small and the stream passes most of its inflow so quickly that the finer sediments do not settle but are discharged downstream. On the other hand, a large reservoir constructed on small stream (having less inflow rates) has greater (Capacity / Inflow) ratio. Such a reservoir has a greater trap efficiency. Such a reservoir may retain water for several years and permit almost complete deposition of the sediment.

Procedure for Calculation of Life of a Reservoir :

- (1) The required reservoir capacity is determined from the considerations discussed in the previous article. Knowing the inflow rate calculate the (Capacity / Inflow) ratio and obtain the trap efficiency from the curve (Fig. 6.11) for the full capacity of the reservoir.
- (2) Divide the total capacity into any suitable interval, say 10%. Assuming that 10 % capacity has been reduced due to sediment deposit, find the trap efficiency for the reduced capacity (i.e., 90 % of the original) and the inflow ratio.
- (3) For this interval of 10 % capacity, find the average trap efficiency by taking the average of η found in steps (2) and (3).

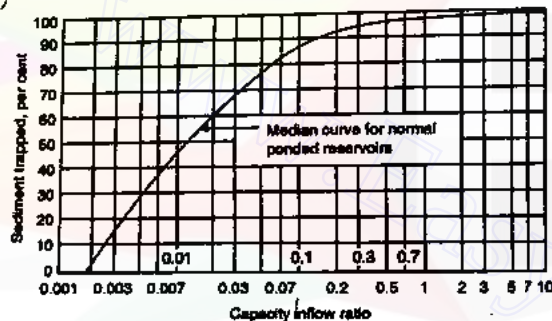


FIG. 6.11 TRAP EFFICIENCY OF RESERVOIRS

(4) Determines the sediment inflow rate by taking water samples and drying the sediment. Multiply the total annual sediment transported by the trap efficiency found in step (3). Convert this sediment deposited into hectare-metre (volumetric) units deposited in one year.

(5) Divide the volume interval (i.e., 10 % of the capacity) by the sediment deposited (determined in step 4) to get the number of years to fill this volume interval of 10 % capacity.

(6) Repeat this procedure for further intervals, i.e. at 80%, 70%, ..., 20% of the capacity. The total life of the reservoir will be equal to the total of the number of years required to fill each of the volume intervals.

The procedure is illustrated in Example 6.3.

6.9. RESERVOIR SEDIMENT CONTROL

Following are some of the methods used for the control of silting of reservoirs:

(1) Proper Selection of Reservoir Site

The silt transported in the system depends upon the nature of the catchment area. A stream collecting water from catchment area having soft or loose soil, and having steep slopes, may carry more sediment load. Hence, the reservoir site should be such that it excludes runoff from easily erodable catchment area. If a certain tributary of the main stream carries more silt, the dam should be constructed to the upstream of that tributary.

(2) Control of Sediment Inflow

This is a preventive measure. Small check dams may be constructed across those tributaries which carry more silt. Increase of vegetal cover over the catchment area also decreases the soil erosion and hence sediment inflow is reduced.

(3) Proper Designing and Reservoir Planning

The sediment trapped in the reservoir also depends upon the reservoir planning. As discussed earlier, a small reservoir on a big river has lesser trap efficiency. Hence, if the dam is constructed lower in the first instance, and is being raised in stages, the life of the reservoir will be very much increased. During the floods, the sediment carried by the stream is the maximum. Hence, sufficient outlets should be provided in the dam at various elevations, so that the floods can be discharged to the downstream without much silt deposit.

(4) Control of Sediment Deposit

The sediment deposit in the reservoir can be controlled by proper operation of the gates of scouring sluices and the head regulators of the canals etc. These should be so designed and operated that selective removal of silt is affected. During the floods, when sediment inflow is higher, the scouring sluices must be opened to discharge the silt downstream.

(5) Removal of Sediment Deposit

This is a curative measure. The under-sluices may be used to scour the deposited silt. However, it has been observed that when the scouring sluices are operated, they do not remove the bulk of the silt. A narrow channel is scooped along its line and

the rest of silt remains undisturbed. The silt deposit can also be removed by excavating and dredging processes, though this proposal is very costly. The best way, therefore, is to first disturb the deposited silt by mechanical means so that the silt is loosened and pushed towards the sluices, and then opening the sluices so that most of the silt is discharged downstream. Another useful way is to let most of the flood water pass through the scouring sluices with minimum detention in the reservoir. This will require strong structural components of the sluice gates.

(6) *Erosion Control in the Catchment Area*

Under this category fall the various methods of soil conservation, such as provision of control bunds, checking gully formation by providing small embankments, afforestation, regrassing and control of grazing etc. Provision of vegetation screen helps in reducing the 'sheet erosion'.

6.10. SINGLE PURPOSE FLOOD CONTROL RESERVOIRS

Flood control reservoirs are adopted to prevent loss of life and valuable property due to floods. Most often it is provided to protect the economic development in an area. There are two types of single purpose flood control reservoirs, namely :

- (1) Retarding Reservoirs. (2) Detention Basins.

1. Retarding Reservoirs

A retarding reservoir is the one in which the spillway and out-lets are not controlled by gates or valves. Instead, they moderate the flood peak by passing discharge downstream, depending upon reservoir elevation. It has uncontrolled outlets and maximum discharge through the outlets do not exceed the safe carrying capacity of downstream river during the highest flood.

Advantages of a Retarding Reservoir

(1) There are no gates at the outlets and hence the possibility of human error in reservoir operation is eliminated.

(2) The cost of expensive gate installation and operation is saved.

Disadvantages

A serious disadvantage of retarding reservoirs is that automatic regulation may cause coincidence of flood crest farther downstream where two or more channels taking off from retarding reservoirs join together.

An ideal location of such reservoirs is immediately above a city or above the area to be protected from flood furyosity.

2. Detention Basins

A flood control reservoir with gate installations at the spillway and outlets is termed as a detention basin. The gate installation provide more flexibility of operation and thus increase the usefulness of reservoir. Gated outlets permit the release of any amount of discharge upto maximum. The reservoir storage can, therefore, be utilized more effectively by passing maximum safe discharge in downstream channel at every stage of flood. Also, the reservoir can be emptied more rapidly during early floods to prepare it to receive the subsequent floods.

The disadvantages of detention basins arise out of the possibility of human error during operation and high initial cost to be incurred for gate installation and subsequent maintenance. But the advantage that the flood crest in the downstream channel can be regulated properly so as not to cause their coincidence far outweighs its disadvantages. Detention should be adopted where the protection of large areas is needed and where the area extends for some distance downstream of reservoir site.

Maximum capacity of reservoir should be equal to the design inflow minus safe release in the downstream channel. However, reservoir size should be fixed by an economic analysis. The economic analysis involves the determination of benefit and cost ratio for different capacities of the reservoir. The capacity which gives maximum benefit cost ratio — which should be greater than one — should be adopted for design.

6.11. MULTIPURPOSE RESERVOIRS

The term 'multipurpose reservoir' includes all reservoirs actually designed and operated to serve more than one function and that it excludes those whose design and operation are controlled by a single function, even though other benefits accrue as by-products.

There can be several purposes for which a reservoir may be made. These are listed in § 6.1. If some of these purposes are combined, there will be more effective utilisation of water and economical construction of a reservoir. Preferable combinations for a multipurpose reservoir are :

- (1) Reservoir for Irrigation and Power.
- (2) Reservoir for Irrigation, Power and Navigation.
- (3) Reservoir for Irrigation, Power and Water supply.
- (4) Reservoir for Recreation, Fisheries and Wild life.
- (5) Reservoir for Flood control and Water supply.
- (6) Reservoir for Power and Water supply.
- (7) Reservoir for Flood Control, Irrigation, Power and Water Supply.

This is the most common combination.

Planning of a Multipurpose Reservoir

Multipurpose planning involves substantially more than simple combination of single purpose elements, if effective utilisation of water resources has to be brought about. None of the possible uses of a reservoir are entirely compatible with any other use, although under many circumstances it is possible to bring them into good agreement. The unique feature in the multipurpose design, therefore, is an operation plan which is an effective compromise among the various uses.

There are two possible extremes in allocation of reservoir storage :

- (1) To assume that all storage is jointly used.
- (2) To assume that no storage is jointly used.

In the second case, storage requirements for all functions are pyramided to create a large total storage requirement which can be economically obtained only when unit cost of storage is constant or decreases as total storage increases. The other extreme results in maximum economy since the required storage is not greater than that necessary

for any one of the several purposes. Such a situation is relatively rare and the usual multipurpose reservoir is designed to fall somewhere between these two extremes.

For Indian conditions the usual combination is irrigation, hydroelectric power and flood control. Planning of such a reservoir requires a detailed analysis of past records of run-off and other hydrological data. The requirements of operation for such a reservoir are :

- (1) to provide storage for flood control either by permanent allocation of space exclusively for flood control or by seasonal allocation of space for flood storage.
- (2) to provide a steady discharge during April, May and June for 'Kharif' irrigation and a large discharge from mid-November to mid-February for 'Rabi' irrigation.
- (3) to provide a steady discharge for running of turbines to generate maximum firm power.

Supposing the reservoir is located on an ice fed river, it will have lowest water level, sometimes upto dead storage level during February and March. This is because of the low supplies in the river and heavy draw of water for Rabi irrigation during preceeding months. During the months of April, May and June, the supplies in the river steadily increases as the snow starts melting. Reservoir has to supply water for power and Kharif irrigation during these months. Therefore, depending upon inflow and outflow, the reservoir surface may increase or decrease slightly at a slow rate. This goes on and just before monsoon, a large reservoir capacity is available for dual purpose of conservation and flood control. During monsoon months demand for irrigation is small. There is only a constant release of water needed for power and inflow in the river is very large. The reservoir surface rises and it is designed to rise upto maximum conservation level at the end of normal monsoon period, say middle October. Even at this time some flood control storage is available between conservation level and maximum reservoir level. Actual data by which the reservoir should be allowed to be filled upto maximum conservation level depends upon run-off pattern of past years and many a time upon wise descretion of operating authorities, who have studied the behaviour of flood for many years. During the month of October and November, the supplies in the river go down and the outflow increases to meet the demands for 'Rabi' irrigation and, therefore, reservoir surface goes down. During the months from November to February, draw-down from reservoir is heavy and the reservoir surface goes down rapidly except for some winter freshets in December and January.

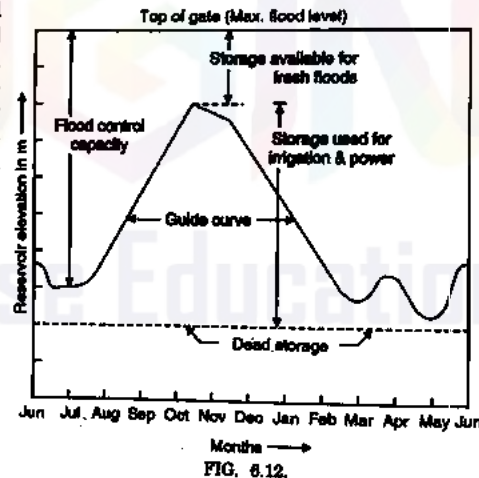


FIG. 6.12.

Seasonal allocation to flood control is done only where a definite flood season occurs such as the case in North India. However, a small portion need always be reserved for unusual floods.

Various features of the operation are usually expressed in the form of chart known as *rule curve*. The rule curve shows the amount of space to be left vacant on any date and dates during which the reservoir is to be used. Fig. 6.12 shows the operation plan for a reservoir intended to serve flood control, irrigation and power. The operation plan is usually a guide based on the analysis of past records to help the operators.

A good example of multipurpose reservoir is Damodar Valley Corporation (D.V.C.) on tributaries of river Damodar in Bihar and Bengal. It envisages 7 dams in Bihar and Bengal with a total of 490 sq km of reservoir area. It produces 200,000 kW of hydroelectric power, irrigates 0.4 million hectare of land, supplies 96 cusecs of drinking water, stores 60 ha-m of flood, besides doing navigation, soil conservation and eradication of malaria.

6.12. APPORTIONMENT OF TOTAL COST OF A MULTIPURPOSE RESERVOIR

Apportionment of total cost of a multipurpose reservoir means the sub-division of cost for various uses for which the reservoir is planned. Various methods of apportionment of total cost are as follows :

- (1) Equal apportionment method.
- (2) Use of facilities method.
- (3) Alternative justifiable expenditure method.
- (4) Remaining benefits method.

There is no really satisfactory method of cost allocation, which is equally applicable to all projects and which may yield unquestionable correct results. Any method of allocation must first set aside the *separable costs* which are chargeable to a single project function directly, such as the cost of a power-house structure and its equipments in case of a hydroelectric project, the cost of navigation lock in case of a navigation project and the cost of canal head works in case of irrigation project etc. The term *joint cost* used hereinafter, is the difference of total cost and separable cost.

1. Equal Apportionment Method

The joint cost of the project is equally divided into different purposes, irrespective of separate cost of benefit. The method is not based on convincing logic and is used only when data available are so meagre that other methods cannot be used.

2. Use of Facilities Method

The joint cost is divided in the ratio of storage volume provided for each purpose. The method is simple in use but the cost is not directly proportional to storage. In fact the cost per unit volume goes on decreasing as the storage increases and hence the cost of each unit storage is not the same.

3. Alternative Justifiable Expenditure Method : In this method, the joint cost is distributed in the ratio of difference between the separable cost and the estimated cost of a single-purpose project which would provide equivalent services and would itself be economically justifiable. The greatest difficulty lies in use of this method is the collection of data for estimation of alternative justifiable cost of a single purpose project.

4. Remaining Benefit Method

In this method, the joint cost is assumed to be distributed in accordance with the difference between separable cost and the estimated benefit of each function or its alternative justifiable cost whichever is lower. This is the best of all the methods and is sufficiently accurate, though here also it is difficult to find out the benefits or its alternative cost.

6.13. FLOOD ROUTING

It is the process of calculating water levels in reservoir, the storage quantities and outflow rates corresponding to a particular inflow hydrograph at various instants. Flood routing is carried out in a reservoir to determine what will be the maximum rise in its water surface and what will be the discharge in the downstream channel when particular flood passes through it. Flood routing is an important technique necessary for the complete solution of a flood control problem and for the satisfactory operation of a flood-prediction service.

Modification of a inflow flood hydrograph due to flood routing process is shown in Fig. 6.1. It can be seen therein that when a flood passes through the reservoir, its peak reduces, its hydrograph acquires a bigger time base and a great basin lag is introduced in it.

METHODS OF FLOOD ROUTING

A large number of methods are in use for flood routing process. Some of the important methods which have a more practical bearing are :

- (1) Calculus method
- (2) Step by step methods : (a) Graphical method (b) Trial and error method.

The calculus method is beyond the scope of present study.

Step by Step Methods

The relationship governing the inflow and outflow is essentially simple. The flood entering a given instant of time is partly stored in the reservoir and is partly let out by the spillway and other outlets. The process is, however, complicated by the fact that neither the flood inflow hydrograph nor the storage and outlet discharge can be expressed by simple algebraic equations. A step by step procedure is, therefore, followed which can be done either graphically or by trial and error.

(a) GRAPHICAL METHOD : Inflow-Storage-Discharge (ISD) Curves Method

The governing relation between inflow, outflow and change in storage is

$$\bar{I} - \bar{O} = \Delta s \quad \dots(1) \quad \dots(6.4)$$

where $\bar{I}$ is average inflow during a given time period ; $\bar{O}$ is the average outflow during given time period, and Δs is volume of water stored in the reservoir during the same period. Equation (1) can also be written as,

$$\frac{I_1 + I_2}{2} t - \frac{O_1 + O_2}{2} t = s_2 - s_1 \quad \dots(2) \quad \dots(6.5)$$

where I_1 and I_2 are the inflow rates, O_1 and O_2 are the outflow rates at the beginning and end of the time interval t and s_1 and s_2 are the storage volumes in the beginning

and end of the time interval. Equation (2) can be written as,

$$(I_1 + I_2) + \left(\frac{2s_1}{t} - O_1 \right) = \left(\frac{2s_2}{t} + O_2 \right) \quad \dots(3) \quad \dots(6.6)$$

Following graphs are required to proceed further with the analysis.

1. Inflow flood hydrograph from which ordinates I_1, I_2 etc. can be found at chosen interval of time period. This is shown in Fig. 6.13 (Graph I).
2. Elevation-outflow curve of the reservoir, as shown in Fig. 6.14 (Graph II).
3. Elevation-storage curve of the reservoir, as shown in Fig. 6.14 (Graph III).

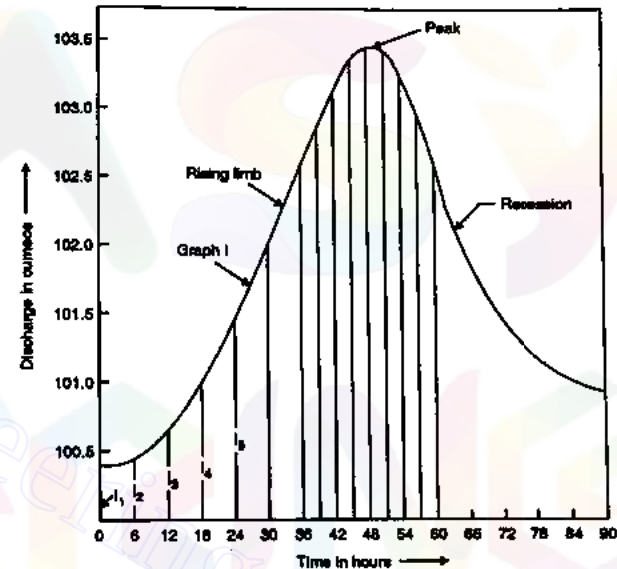


FIG. 6.13. INFLOW FLOOD HYDROGRAPH

Before starting the actual routing operations by the graphical method, the curves of $\left(\frac{2s}{t} \pm O \right)$ v/s outflow O are derived, as shown in Fig. 6.15. To do this, the values of s and O are taken, from Fig. 6.14, for various values of reservoir elevations. Knowing the corresponding values of s and O , values of $\left(\frac{2s}{t} \pm O \right)$ are computed and plotted against the corresponding outflow rates O (Fig. 6.15). In this, t is any chosen time interval.

Now, at the beginning of the routing period, both s_1 and O_1 are zero. Substituting these in Eq. (3), we get $(I_1 + I_2) = \frac{2s_2}{t} + O_2$.

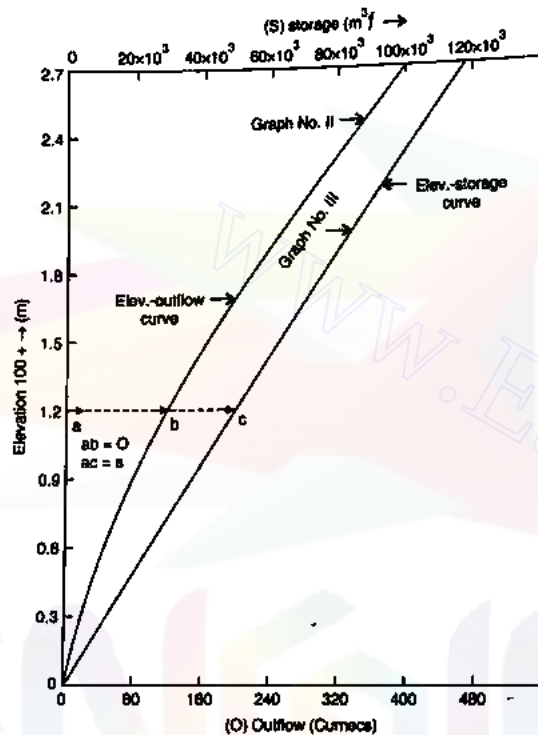


FIG. 6.14 ELEVATION OUTFLOW AND ELEVATION STORAGE CURVES.

For the first time interval t , the inflow rates I_1 and I_2 at its beginning and end are known from Fig. 6.13. Hence, $\left(\frac{2s_2}{t} + O_2\right)$ is known. In Fig. 6.15, enter this value of $\left(\frac{2s_2}{t} + O_2\right)$ and get O_2 . For this value of O_2 also find $\left(\frac{2s_2}{t} - O_2\right)$ from the same figure. Now applying Eq. (3) for the next time interval, we get

$$(I_2 + I_3) + \left(\frac{2s_2}{t} - O_2\right) = \left(\frac{2s_3}{t} + O_3\right)$$

In this equation, I_2 and I_3 are known from Fig. 6.13, and $\left(\frac{2s_2}{t} - O_2\right)$ is also known as explained above. Hence, $\left(\frac{2s_3}{t} + O_3\right)$ is known. Hence, from Fig. 6.15, find the value

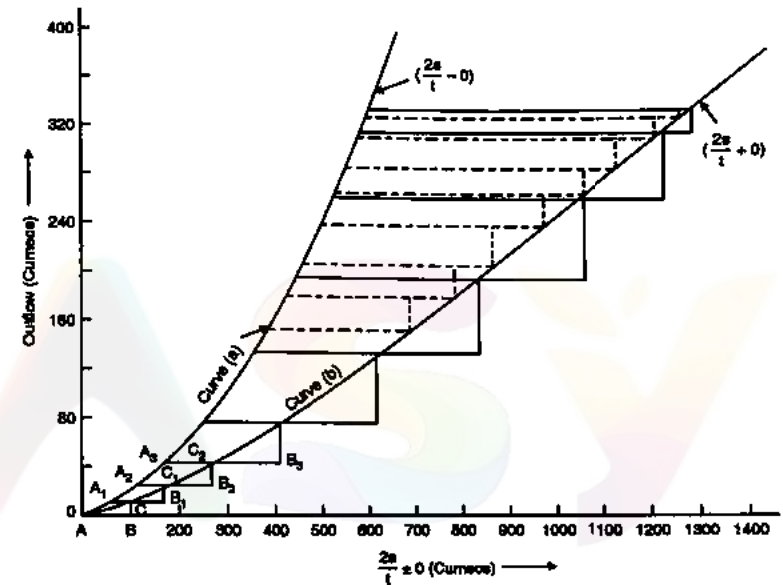


FIG. 6.15 FLOOD ROUTING BY GRAPHICAL METHOD.

of O_3 corresponding the value of $\left(\frac{2s_3}{t} + O_3\right)$. Also find $\left(\frac{2s_3}{t} - O_3\right)$ for this value of O_3 . Similarly, applying Eq. (3) for the next time interval, we get

$$(I_3 + I_4) + \left(\frac{2s_3}{t} - O_3\right) = \left(\frac{2s_4}{t} + O_4\right)$$

From which $\left(\frac{2s_4}{t} + O_4\right)$ is known and the process is repeated for the complete period of flood.

The graphical method can thus be summarised by the following steps:

- (i) Calculate the total inflow from Fig. 6.13 for the time interval t and enter it on Fig. 6.15 as the value AB .
- (ii) Draw vertical line from B to meet the curve of $\left(\frac{2s}{t} + O\right)$ in a point C . Point C gives the value of outflow at the end of interval.
- (iii) Draw a horizontal line through C to cut the curve of $\left(\frac{2s}{t} - O\right)$ in point A_1 .
- (iv) Calculate the total inflow during the next period from inflow flood hydrograph and mark and measure it as A_1CB_1 .

(v) Erect a perpendicular from B_1 to meet the curve of $\left(\frac{2s}{t} + O\right)$ in a point C_1 and repeat the same procedure as outlined in (iii) and (iv) until the entire flood is routed. The outflow discharge at any time interval is given by the total vertical ordinates. The biggest of these ordinates will give the value of the peak outflow rate for which the spillway is to be designed.

For example, the peak outflow rate obtained in Fig. 6.15 is 334 cumecs. When flood routing is continued, the vertical ordinates decrease, representing the receding flood.

(vi) The outflow discharge at various time intervals having been determined as described above, the reservoir water surface elevation for these can be determined from Fig. 6.14, Graph No. II.

Semi-graphical method : Instead of drawing the vertical ordinates and doing it purely graphically, the values can be tabulated, as illustrated in Table 6.4 of Example 6.1.

(b) Modified Puls' method

The modified Puls' method is simpler than the conventional graphical method (ISD method), since it uses lesser number of graphs for computations. The basic equation used for flood routing is the same i.e. Eq. 6.6 :

$$I_1 + I_2 + \left(\frac{2s_1}{t} - O_1\right) = \left(\frac{2s_2}{t} + O_2\right) \quad \dots(6.6)$$

Procedure

1. Choose a suitable time interval t and prepare a curve between elevation as ordinate and the function $\left(\frac{2s}{t} + O\right)$ as abscissa, as shown in Fig. 6.16.

2. At the beginning of the routing process, s_1 and O_1 are zero and hence all the terms on the L.H.S. of Eq. 6.6 are known. Hence compute the value of the function $\left(\frac{2s_2}{t} + O_2\right)$.

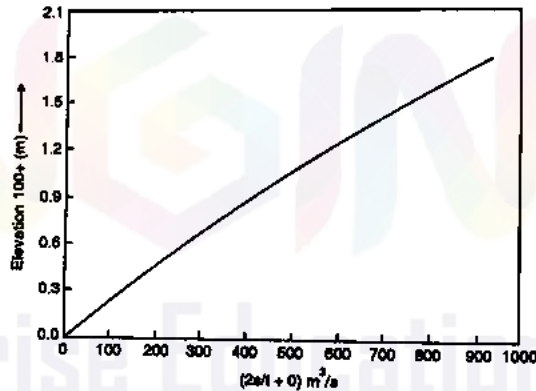


FIG. 6.16

3. For the value of function $\left(\frac{2s_2}{t} + O_2\right)$ determined in step 2 above, find the value of elevation from Fig. 6.16 and outflow rate O_2 from Fig. 6.14.

4. Compute the value of the function $\left(\frac{2s_2}{t} - O_2\right)$ from the known value of the function $\left(\frac{2s_1}{t} + O_1\right)$ and O_2 , using the following equation :

$$\left(\frac{2s_2}{t} - O_2\right) = \left(\frac{2s_1}{t} + O_1\right) - 2O_2 \quad \dots(6.7)$$

5. Next, compute the value of $\left(\frac{2s_3}{t} + O_3\right)$, using Eq. 6.6 modified as under

$$\left(\frac{2s_3}{t} + O_3\right) = (I_2 + I_3) + \left(\frac{2s_2}{t} - O_2\right)$$

In the above equation all the terms in the R.H.S. are completely known. Thus the value of $\left(\frac{2s_3}{t} + O_3\right)$ is known, corresponding to which elevation is found from Fig. 6.16 and O_3 is found from Fig. 6.14.

6. Repeat the above procedure till the entire inflow hydrograph is routed.

7. Determine the maximum water level and the maximum outflow rate from the values obtained in the above procedure.

(c) TRIAL AND ERROR METHOD

This is a more commonly used method. The steps required for flood routing are described below :

(i) Divide the inflow flood hydrograph of Fig. 6.13 into various small intervals of time.

(ii) Begin with the first period and calculate the total inflow during the interval by multiplying the average inflow rate at the beginning and end of the period with the time interval $\left(= \frac{I_1 + I_2}{2} t\right)$.

(iii) The water surface elevation at the beginning of flood is known. Assume a trial value of water surface elevation in the reservoir at the end of period t .

(iv) From the elevation-outflow curve [Fig. 6.14] find out the outflow at the beginning and end of period for the corresponding water surface elevations. The average value of outflow multiplied by the time interval represents the net outflow during the period $\left(= \frac{O_1 + O_2}{2} t\right)$.

(v) From the elevation-storage curve [Fig. 6.14] find out the storage at the beginning and end of the period as the corresponding water surface elevations are known. Find out their difference $(s_2 - s_1)$ which represents the amount of flood stored in the reservoir during the period.

(vi) Add the value of outflow obtained in step (iv) to the value of storage obtained in step (v). This value should be equal to the total inflow during the periods as calculated in step (ii). If this is so the assumed trial elevation is correct, otherwise another value

of trial elevation should be assumed and the trial repeated so as to bring the coincidence between the total inflow with the sum of outflow and storage.

(vii) Repeat steps (ii) to (vi) for other time intervals, till the flood is routed.

A typical example illustrating trial and error procedure follows.

Example 6.1. The following data (Table 6.1) pertains to an inflow flood hydrograph whose flows in 100 cumecs have been recorded at 6 hours interval starting from 0.00 hours on June 1, 1959 on a certain stream.

TABLE 6.1

0.42	0.45	0.57	0.88	1.47	2.10
2.72	3.40	3.50	3.38	3.14	2.88
2.63	2.40	1.98	1.70	1.43	1.20

This flood approaches a reservoir with uncontrolled spillway with elevation area and elevation outflow data as shown in Table 6.2.

TABLE 6.2

Elevation (m)	100.00	100.3	100.6	100.9	101.2	101.5	101.8	102.1	102.4	102.7
Area (ha)	405	412	420	425	428	436	445	453	460	469
Outflow (cumecs)	0	14.9	42.2	77.3	119	169	217	272	334	405

The water level just reaches the crest level (elevation 100.00 m) of the spillway at 4 hours on June 1, 1959.

Determine the maximum reservoir level and maximum discharge over the spillway. Draw inflow and routed hydrographs indicating the reduction in peak flow and peak lag introduced due to routing.

(a) **Solution by semi-graphical method**

1. Draw the inflow hydrograph as shown in Fig. 6.13 Graph-I.
2. Draw elevation-outflow curve, as shown in Fig. 6.14 Graph II.
3. Draw elevation storage curve, as shown in Fig. 6.14 Graph III. The starting elevation for this spillway is crest level. For plotting this curve, volume between different contours is found by cone formula, or by any other formula.

4. The values of $\frac{2s}{t} \pm O$ are calculated in the Table 6.3.

The values of column (7) and column (8) are plotted against column (2), as shown in Fig. 6.15.

The routing procedure is exactly the same as described under the graphical method. Instead of drawing the vertical ordinates and doing it purely graphically, the values can be tabulated in the following table (Table 6.4). The values of $\left(\frac{2s}{t} - O\right)$ and O are read each time from Fig. 6.15 for different values of $\frac{2s}{t} + O$. The tabulated values are given in Table 6.4. Assume a time interval of 6 hours.

TABLE 6.3

Elevation (m)	Outflow (cumecs)	Area enclosed by each contour (A) sq m $\times 10^4$	Storage between contours (a) = $\frac{H_2}{3} \times (A_1 + A_2 + \sqrt{A_1 A_2})$	Cumulative storage above spillway crest	$\frac{2s}{t}$ for $t = 6 \text{ hr} = \frac{s}{10800}$	$\frac{2s}{t} - O$	$\frac{2s}{t} + O$
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
100.0	0	405					
100.3	14.9	412	122.4	122.4	113.3	98.4	128.2
100.6	42.2	420	124.8	247.2	228.5	186.3	270.7
100.9	77.3	425	126.9	374.0	348.0	258.7	428.3
101.2	119	428	128.0	502.0	485.0	348.0	584.0
101.5	169	436	128.6	631.9	585.0	418.9	752.0
101.8	217	445	132.2	763.8	708.0	498.0	923.0
102.1	272	453	134.8	898.6	831.5	559.5	1103.5
102.4	334	460	137.0	1036.6	957.5	623.5	1291.5
102.7	405	469	139.4	1175.0	1088.0	683.0	1483.0

The method of drawing vertical ordinates and corresponding horizontal ordinates are shown in Fig. 6.15.

TABLE 6.4

Time (Starting from June 1, 1959)	Inflow cumecs I	$I_1 + I_2$	$\frac{2s}{t} - O$ Values read from curve s of Fig. 6.15	$\frac{2s}{t} + O$	Outflow O [Read from Fig. 6.15 for Col. (5)]	Reservoir Elevation [Read from Fig. 6.14 for Col. (6)]
(1)	(2)	(3)	(4)	(5)	(6)	(7)
hours 0	42					
6	45	87	0	87	10	100.20
12	57	102	60	162	24	100.39
18	58	145	122	267	42	100.58
24	147	235	185	420	74	100.86
30	210	357	288	623	130	101.26
36	272	482	362	844	194	101.85
42	240	612	455	1067	280	102.33
48	350	690	545	1235	316	102.31
54	388	668	605	1283	334*	102.40*
60	314	652	623	1275	328	102.37
66	286	602	620	1222	312	102.30
72	263	561	600	1181	286	102.18
78	240	509	575	1078	254	102.06
84	198	438	550	988	236	101.90
90	170	388	515	863	204	101.72
96	143	315	470	783	177	101.56
102	120	263	430	693	150	102.40

Results.

1. Maximum reservoir level, from column (7) = 102.4 m
2. Maximum discharge over spillway, from column (6) = 334 cumecs.
3. Reduction in peak discharge = 350 - 334 = 16 cumecs
(= Maximum inflow - Maximum outflow)
4. Peak lag = (Time for maximum inflow) - (Time for maximum outflow) = 54 - 48 = 6 hours.

The inflow flood hydrograph and outflow flood hydrograph have been plotted together in Fig. 6.17.

(b) Solution by Trial and Error Method

Make a table having 12 columns (Table 6.5). Choose a time interval of 6 hours in the beginning of flood and 3 hours during the period the flood is near its peak, and enter into column (1). Begin routing the flood from spillway crest onward and enter the reservoir elevation at the beginning of the period in column (2). Calculate the inflow flood volume from flood inflow hydrograph (plotted in Fig. 6.13) for the time interval under consideration and enter it into column (3). Find the spillway discharge for the reservoir elevation at the beginning of period from elevation outflow curve of Fig. 6.14 and enter it into column (4). Assume a trial reservoir elevation, and enter it into column (5). For this trial reservoir elevation, calculate the spillway discharge and enter into column (6). Find out the mean outflow rate at the beginning and end of period and enter it into column (7). Find the outflow volume from the mean outflow rate $\left(\frac{Q_1 + Q_2}{2} t \right)$ and enter into column (8). The outflow is less than inflow volume so long as the peak of flood has not been routed. Once the peak of flood has been routed, the outflow volume exceeds the inflow volume and the maximum condition may be taken to have occurred. From the storage elevation curve (Fig. 6.14) find the storage capacity between the reservoir elevation at the beginning

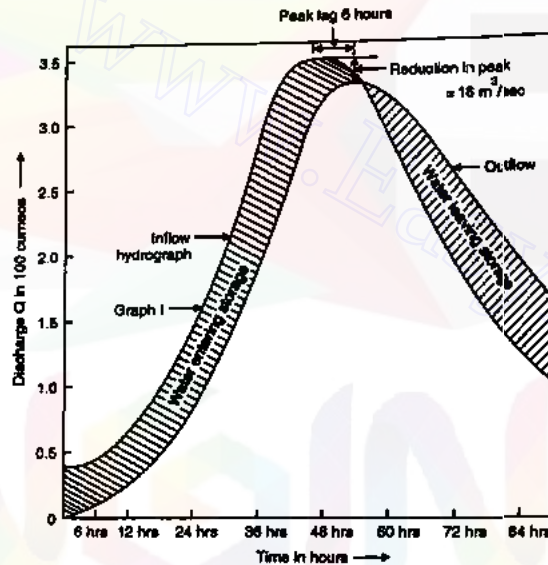


FIG. 6.17.

and end of period and enter it into column (9). Find out the sum of outflow volume (column(8)) and storage capacity (column(9)) and enter it into column (10). Find out the difference between column (9) and column (10) and enter it into column (11). This difference should be negligible, otherwise the trial should be repeated. Mention may be made about the trials made and maximum conditions for the outflow flood in the remarks column (12).

TABLE 6.5. FLOOD-ROUTING BY TRIAL AND ERROR METHOD

Time period hours beginning from June 1	Reservoir Elevation : beginning of period (metres)	Inflow Volume (million cubic metres)	Spillway discharge : beginning of period (cumecs)	Trial Reservoir Elevation : end of period (metres)	Spillway discharge : end of period (cumecs)	Mean outflow rate (cumecs)	Outflow Volume (million cubic metres)	Storage capacity between Reservoir Elevation in beginning and end of period (million cubic metres)	Outflow Volume + Storage capacity (million cubic metres)	Col. (9) - Col. (10)	Remarks
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
0-6	100.00	0.940	0	100.20	10	5.0	0.108	0.832	0.940	0.008	
6-12	100.20	1.100	10	100.39	24	17.0	0.367	0.740	1.107	-0.007	
12-18	100.39	1.565	24	100.58	42	33.0	0.712	0.850	1.562	0.008	
18-24	100.58	2.540	42	100.86	74	58.0	1.250	1.290	2.540	0.000	
24-30	100.86	3.680	74	101.26	130	102.0	2.200	1.665	3.685	0.005	
30-36	100.25	5.200	130	101.65	184	162.0	3.600	1.700	5.200	0.000	
36-39	101.65	3.300	184	101.83	224	209.0	2.360	1.046	3.305	0.005	
39-42	101.83	3.490	224	102.02	260	242.0	2.610	0.880	3.490	0.000	
42-45	102.02	3.725	260	102.165	284	272.0	2.940	0.780	3.720	0.006	
45-48	102.165	3.755	284	102.31	316	300.0	3.240	0.515	3.755	0.000	
48-51	102.31	3.750	316	102.355	325	320.5	3.480	0.290	3.750	0.000	
51-54	102.355	3.690	325	102.40*	334*	329.5	3.506	0.180	3.686	0.006	
54-57	102.40	3.508	334	102.385	331	332.5	3.580	-0.080	3.510	0.002	

*Maximum reservoir elevation = 102.40 m *Maximum discharge over spillway = 334 cumecs

Example 6.2. Table 6.6 gives the monthly inflows during the critical low water period at the site of a proposed dam, the corresponding monthly evaporation and precipitation at a nearby station and the estimated monthly demand for water. Prior water rights require the release of full natural flow or 10 ha-m per month, whichever is least. Assume that 30 percent of the rainfall on the land area to be flooded by the reservoir has reached the stream in the past. Using a net increased pool area of 500 hectares, find the required useful storage. Use a pan evaporation coefficient of 0.72.

TABLE 6.6

Months	Inflow at the proposed site (ha-m)	Pan evaporation (cm)	Precipitation (cm)	Demand (ha-m)
January	8.6	2.2	0.8	14.5
February	2.2	2.3	1.2	15.8
March	1.8	3.1	0.0	16.2
April	0.0	8.6	0.0	16.8
May	0.0	12.8	0.0	17.5
June	13.5	15.6	4.8	18.0
July	280.6	12.3	12.2	18.0
August	510.2	10.6	18.6	17.0
September	136.0	10.0	8.6	16.5
October	52.5	8.6	1.5	16.0
November	20.6	5.8	0.0	15.8
December	12.3	3.0	0.0	15.0

Solution. Computations are arranged in Table 6.7 in which the first five columns are the same as given in Table 6.6. Column 6 gives the downstream requirements due to water right flow and the entries in this column are equal to the river flow or 10 hectare metres whichever is the minimum.

TABLE 6.7

Month	Inflow (ha-m)	Pan Evap. (cm)	Precipitation (cm)	Demand (ha-m)	D/S Requirements (ha-m)	Evaporation $E = 3.6 \times \text{col. (3)}$ (ha-m)	Precipitation $P = 3.5 \times \text{col. (4)}$ (ha-m)	Adjusted Inflow $I = \text{col. (2)} - \text{col. (6)} + \text{col. (7)} + \text{col. (8)}$ (ha-m)	Water reqd. from storage $S = \text{col. (5)} - \text{col. (9)}$ (ha-m)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
January	8.6	2.2	0.8	14.5	8.6	7.92	2.8	-5.12	19.82
February	2.2	2.3	1.2	15.8	2.2	8.28	4.2	-4.08	19.88
March	1.8	3.1	0.0	16.2	1.8	11.16	0.0	-11.16	27.36
April	0.0	8.6	0.0	16.8	0.0	30.96	0.0	-30.96	47.76
May	0.0	12.8	0.0	17.5	0.0	46.08	0.0	-46.08	61.58
June	13.5	15.6	4.8	18.0	10.0	56.16	16.8	-35.86	51.86
July	280.6	12.3	12.2	18.0	10.0	44.28	42.7	+269.02	Nil
August	510.2	10.6	18.6	17.0	10.0	38.16	65.1	+527.14	Nil
September	136.0	10.0	8.6	16.5	10.0	36.00	30.1	+120.10	Nil
October	52.5	8.2	1.5	16.0	10.0	29.52	5.25	+18.23	Nil
November	20.6	5.8	0.0	15.8	10.0	20.88	0.0	-10.28	24.08
December	12.3	3.0	0.0	15.0	10.0	10.80	0.0	-8.50	21.50
Sum	1038.3	94.5	47.7	197.1	82.6	340.20	166.95	782.45	281.84

Column 7 gives the evaporation over the whole of the reservoir area. Its value E is calculated from the relation :

$$E = \frac{\text{Reservoir area} \times \text{Pan evaporation} \times \text{Pan evaporation coefficient}}{100}$$

$$= \frac{500 \times \text{Column (3)}}{100} \times 0.72 \text{ hectare-metres} = 3.6 \times \text{Column (3)} \text{ hectare-metres.}$$

Column 8 gives the precipitation in hectare-metres, falling over the reservoir area. Since 30 % of the precipitation is already reaching, and has been included in the inflow (column 2), only 70% of the precipitation has been included in the computation. Thus the precipitation P is calculated from the relation

$$P = \frac{\text{Reservoir area} \times \text{Column (4)}}{100} \times 0.7 = \frac{500 \times \text{Column (4)}}{100} \times 0.7 = 3.5 \times \text{Column (4)} \text{ hectare-metres.}$$

Column 9 gives the adjusted Inflow (I) computed from the relation

$$I = \text{Column (2)} - \text{Column (6)} + \text{Column (7)} + \text{Column (8)}$$

Column (10) gives the water required from storage, (i.e. S) and as computed from the following relation :

$$S = D - I$$

or Column (10) = Column (5) - Column (9).

In the above relation, only positive values are to be included. When column (9), i.e. adjusted inflow is more than column (5), zero values are to be written in column (10) indicating that no water is required from the storage since demand is much less than the adjusted inflow. The required storage is the sum of the monthly increments of demand in excess of stream flow. The required storage capacity in this case works out to be 281.64 ha-m.

Example 6.3. Table 6.8 gives the details about the average seasonal discharges of a river for 12 years. Determine the storage capacity required to maintain a flow of 475 cumecs throughout the year.

TABLE 6.8

Year	16th June to 30 Sept. (cumecs)	1st Oct-31st March (cumecs)	1st April -15th June (cumecs)
1960-61	1050	300	50
1961-62	3000	250	40
1962-63	3500	370	90
1963-64	2000	150	120
1964-65	1200	350	65
1965-66	1400	400	100
1966-67	3800	200	80
1967-68	3000	150	120
1968-69	700	210	50
1969-70	800	120	80
1970-71	2400	320	120
1971-72	3200	280	80

Solution. Let the periods from 16th June to 30th Sept., 1st Oct. to 31st March and 1st April to 15th June be designated as M , W and S respectively. The data for preparing mass inflow curve are tabulated in Table 6.9. The mass inflow curve is shown in Fig. 6.18.

TABLE 6.9. DATA FOR MASS-INFLOW CURVE

S.N.	Year	Season	No of days	Ave. discharge (cumecs)	Volume (million ha-m)	Cumulative Volume (million ha-m)
1.	1960-61	M	107	1050	0.9707	0.9707
		W	182	300	0.4717	1.4424
		S	76	50	0.0328	1.4752
2.	1961-62	M	107	3000	2.7734	4.2486
		W	182	250	0.3981	4.6417
		S	76	40	0.0263	4.6680
3.	1962-63	M	107	3500	3.2357	7.9037
		W	182	370	0.5818	8.4855
		S	76	90	0.0591	8.5436
4.	1963-64	M	107	2000	1.8490	10.3933
		W	182	150	0.2356	10.6293
		S	76	120	0.0788	10.7083
5.	1964-65	M	107	1200	1.1094	11.8177
		W	182	350	0.5504	12.3681
		S	76	65	0.0427	12.4108
6.	1965-66	M	107	1400	1.2943	13.7051
		W	182	400	0.6290	14.3341
		S	76	100	0.0657	14.3998
7.	1966-67	M	107	3600	3.3281	17.7279
		W	182	200	0.3145	18.0424
		S	76	80	0.0525	18.0949
8.	1967-68	M	107	3000	2.7734	20.8673
		W	182	150	0.2359	21.1032
		S	76	120	0.0788	21.1820
9.	1968-69	M	107	700	0.6441	21.8291
		W	182	210	0.3302	22.1593
		S	76	50	0.0328	22.1921
10.	1969-70	M	107	800	0.7396	22.9317
		W	182	120	0.1887	23.1204
		S	76	80	0.0525	23.1729
11.	1970-71	M	107	2400	2.2188	25.3917
		W	182	320	0.5032	25.8949
		S	76	120	0.0788	25.9737
12.	1971-72	M	107	3200	2.9583	28.9320
		W	182	280	0.4403	28.3723
		S	76	80	0.0525	29.4248

Yearly demand = $475 \times 365 \text{ cumec-days} = 475 \times 365 \times 8.64 = 1,497,960 \text{ ha-m}$
 $\triangleq 1.5 \text{ million ha-m.}$

In Fig. 6.18, T_1, T_2, T_3, T_4, T_5 , etc. are drawn tangent to the apexes, and parallel to the demand curve which has a slope of 1.5 million ha-m in 1 year. The ordinates $O_1, O_2, \dots, O_5$, indicate the deficiencies during the dry periods, assuming that the reservoir was full at the beginning of the period. The maximum of these ordinates (i.e. $O_5 = 1.6 \text{ m. ha-m}$) gives the desired reservoir capacity.

Fig. 6.19 shows the enlarged view of the curve from period 1967 to 1971 during which maximum storage is required. Line AB is drawn parallel to the demand curve

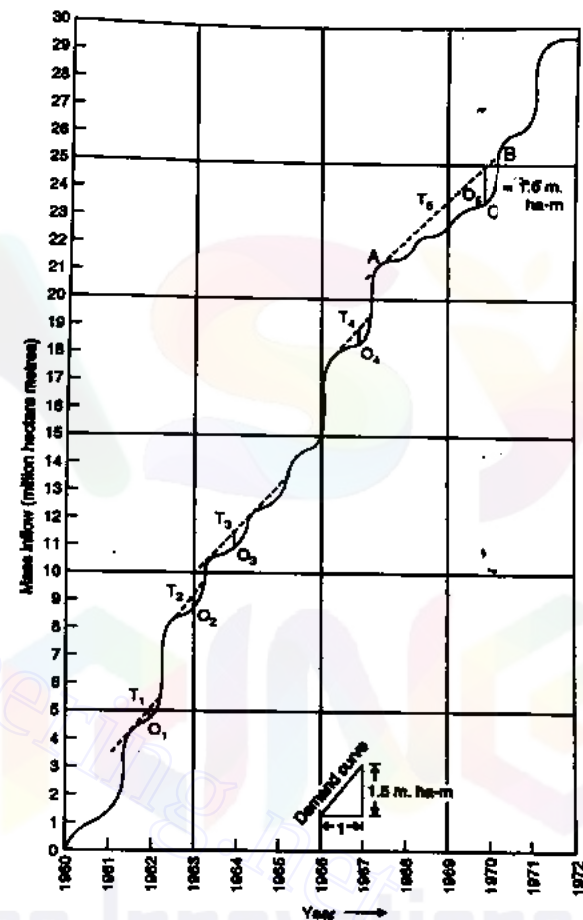


FIG. 6.18. DETERMINATION OF RESERVOIR CAPACITY.

and tangential to the mass inflow curves at point A. At point C, of curve, a storage capacity of 1.6 million hectare metres is required. It is essential that the demand line AB should meet the inflow curve at point B, so that reservoir becomes full at B, otherwise it will never be full. Similarly, if a line CD is drawn parallel to the demand curve, and tangential to the mass-inflow curve at C, then it should intersect the curve at D so that the reservoir becomes full at the start of the dry period.

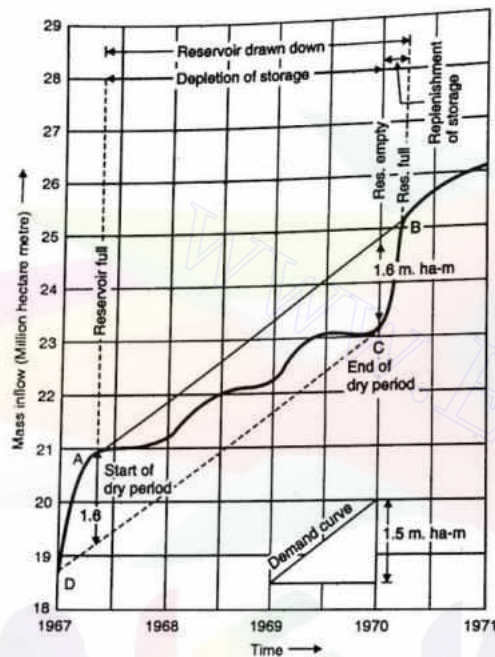


FIG. 6.19

Example 6.4. The following information is available regarding the relationship between trap efficiency and capacity-inflow ratio for a reservoir :

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 (η %)	87	93	95	95.5	96	96.5	97	97.2	97.3	97.5

Find the probable life of the reservoir with an initial reservoir capacity of 30 million cubic metres, if the annual flood inflow is 60 million cubic metres and the average annual sediment inflow is 3600,000 kN. Assume a specific weight of sediment equal to 12 kN/m³. The useful life of the reservoir will terminate when 80% of initial capacity is filled with sediment.

Solution

$$\text{Annual sediment inflow} = 3600,000 \text{ kN} = 3.6 \times 10^6 \text{ kN}.$$

$$\therefore \text{Volume of sediment inflow} = \frac{3.6 \times 10^6}{12} = 0.3 \text{ million cubic metres.}$$

$$\text{Initial reservoir capacity} = 30 \text{ m.c.m.}$$

$$\text{Annual flood inflow} = 60 \text{ m.c.m.}$$

$$\therefore \text{Initial capacity/inflow ratio} = 30/60 = 0.5.$$

This ratio goes on decreasing as the sedimentation takes place, and hence trap efficiency goes on decreasing. The computations for the life of the reservoir are done in Table 6.10. The volume interval chosen in the Table is 6 m.c.m. (i.e. 20% of initial capacity).

TABLE 6.10

Capacity		Capacity Inflow Ratio	Trap efficiency η	Average η for the interval	Annual Sediment, trapped, S_1 (m.c.m.)	Year to fill $\frac{S}{S_1}$
%	Volume $\times 10^6 \text{ m}^3$					
100	30	0.5	0.96	0.9675	0.2872	20.9
80	24	0.4	0.955	0.9625	0.2857	21.0
60	18	0.3	0.95	0.9400	0.2820	21.3
40	12	0.2	0.93	0.9000	0.2700	22.2
20	6	0.1	0.87			
Total						85.4 years.

Hence the useful life of the reservoir is ≈ 85 years.

Example 6.5. A small reservoir has a water spread area given by $A = (30 + 6h) \text{ M. m}^2$, when h is the head of water over spillway crest. The spillway discharge is given by $q = 300 h^{3/2}$ cumecs. The inflow hydrograph due to a certain storm is as under.

Time from start (hours)	0	3	6	9	12	15	18	21	24
Inflow (m^3/s)	60	480	900	470	270	160	110	80	60

Compute the maximum outflow discharge over the spillway and the corresponding maximum level of water above the spillway crest, if the reservoir level was at the spillway crest at the start.

Solution

Let us solve the problem analytically by proceeding step by step and using the basic routing equation. Let us assume that the head over the spillway crest at the end of each 3-hour interval be $h_1, h_2, h_3, \dots, h_n$ metres.

1. For first interval of 0 to 3 hours

Let h_1 be the head over the spillway crest at the end of the interval.

$$\text{Inflow : } I_1 = 60 \text{ cumecs ; } I_2 = 480 \text{ cumecs.}$$

$$\therefore \text{Total inflow} = \frac{1}{2}(I_1 + I_2)t = \frac{1}{2}(60 + 480) \times 3 \times 3600 = 2.916 \text{ M. m}^3$$

$$\text{Outflow : } O_1 = 0; O_2 = 300 h_1^{3/2}$$

$$\therefore \text{Total outflow} = \frac{1}{2}(O_1 + O_2)t = \frac{1}{2}(0 + 300 h_1^{3/2}) \times 3 \times 3600 = 1.62 h_1^{3/2} \text{ M. m}^3$$

$$\text{Storage : } s_1 = 0$$

$$\text{Water spread area at spillway crest} = 30 \text{ M. m}^2$$

$$\text{Water spread area at height } h_1 \text{ above crest} = 30 + 6 h_1$$

$$\therefore s_1 = \frac{1}{2} [30 + (30 + 6 h_1)] h_1 = (30 + 3 h_1) h_1$$

$$[\text{i.e. storage} = (\text{water spread area at } h_1/2) \times h_1]$$

Applying the basic equation for the first interval :

$$\text{Total inflow} = \text{Total outflow} + \text{Change in storage}$$

$$\therefore 2.916 = (1.62 h_1^{3/2}) + (30 + 3 h_1) h_1$$

$$\text{or } h_1^3 + 0.54 h_1^{3/2} + 10 h_1 - 0.972 = 0$$

Solving this by hit and trial, we get $h_1 = 0.0954 \text{ m}$

Note: In the first trial, neglecting h_1^3 and $h_1^{3/2}$ we get $h_1 = 0.972/10 = 0.0972 \text{ m}$.

Hence the value of h_1 for the next trial will be less than 0.0972 m.

2. For the second interval of 3 hours to 6 hours

Let the head over the crest be $h_2 \text{ m}$ at the end of the interval

$$\text{Total inflow} = \frac{1}{2}(480 + 900) \times 3 \times 3600 = 7.452 \text{ M. m}^3$$

$$\text{Total outflow} = \frac{1}{2} [300 (0.0954)^{3/2} + 300 h_2^{3/2}] \times 3 \times 3600 = 0.0477 + 1.62 h_2^{3/2}$$

$$\Delta s = [(30 + 3 h_2) h_2 - (30 + 3 \times 0.0954) \times 0.0954] = 30 h_2 + 3 h_2^2 - 2.8893$$

$$\text{Hence } 7.452 = (0.0477 + 1.62 h_2^{3/2}) + (30 h_2 + 3 h_2^2 - 2.8893)$$

$$\text{or } h_2^3 + 0.54 h_2^{3/2} + 10 h_2 - 8.4312 = 0$$

Solving this by hit and trial, we get $h_2 = 0.323 \text{ m}$

3. For the third interval of 6 hours to 9 hours:

Let the head over the spillway crest be $h_3 \text{ m}$ at the end of the interval.

$$\text{Total inflow} = \frac{1}{2} (900 + 470) \times 3 \times 3600 = 7.398 \text{ M. m}^3$$

$$\text{Total outflow} = \frac{1}{2} [300 (0.323)^{3/2} + 300 (h_3)^{3/2}] \times 3 \times 3600 = 0.2974 + 1.62 h_3^{3/2} \text{ M. m}^3$$

$$\Delta s = \left[(30 + 3 h_3) h_3 - (30 + 3 (0.323)) \times 0.323 \right] = 30 h_3 + 3 h_3^2 - 10.0030 \text{ M. m}^3$$

$$\therefore 7.398 = (0.2974 + 1.62 h_3^{3/2}) + (30 h_3 + 3 h_3^2 - 10.0030)$$

$$\text{or } h_3^3 + 0.54 h_3^{3/2} + 10 h_3 - 5.7012 = 0$$

Solving this by hit and trial, we get $h_3 = 0.522 \text{ m}$

4. For the fourth interval of 9 hours to 12 hours

Let the head over spill way crest be $h_4 \text{ m}$ at the end of the interval.

$$\text{Total inflow} = \frac{1}{2} (470 + 270) \times 3 \times 3600 = 3.996 \text{ M. m}^3$$

$$\text{Total outflow} = \frac{1}{2} [300 (0.522)^{3/2} + 300 h_4^{3/2}] \times 3 \times 3600 = 0.611 + 1.62 h_4^{3/2} \text{ M. m}^3$$

$$\Delta s = \left[(30 + 3 h_4) h_4 - (30 + 3 (0.522)) \times 0.522 \right] = 30 h_4 + 3 h_4^2 - 16.4775$$

$$\therefore 3.996 = (0.611 + 1.62 h_4^{3/2}) + (30 h_4 + 3 h_4^2 - 16.4775)$$

$$\text{or } h_4^3 + 0.54 h_4^{3/2} + 10 h_4 - 6.6208 = 0$$

Solving this by hit and trial, we get $h_4 = 0.601 \text{ m}$

5. For the fifth interval of 12 hours to 15 hours

Let the head over the spillway crest be $h_5 \text{ m}$ at the end of the interval

$$\text{Total inflow} = \frac{1}{2} (270 + 160) \times 3 \times 3600 = 2.322 \text{ M. m}^3$$

$$\text{Total outflow} = \frac{1}{2} [300 (0.601)^{3/2} + 300 h_5^{3/2}] \times 3 \times 3600 = 0.7548 + 1.62 h_5^{3/2} \text{ M. m}^3$$

$$\Delta s = \left[(30 + 3 h_5) h_5 - (30 + 3 (0.601)) \times 0.601 \right] = 30 h_5 + 3 h_5^2 - 19.1136 \text{ M. m}^3$$

$$\therefore 2.322 = (0.7548 + 1.62 h_5^{3/2}) + (30 h_5 + 3 h_5^2 - 19.1136)$$

$$\text{or } h_5^3 + 0.54 h_5^{3/2} + 10 h_5 - 6.8936 = 0$$

Solving this by hit and trial, we get $h_5 = 0.624 \text{ m}$

6. For the sixth interval of 15 hours to 18 hours

Let the head over the spillway crest be $h_6 \text{ m}$ at the end of the interval.

$$\text{Total inflow} = \frac{1}{2} (160 + 110) \times 3 \times 3600 = 1.458 \text{ M. m}^3$$

$$\text{Total outflow} = \frac{1}{2} [300 (0.624)^{3/2} + 300 h_6^{3/2}] \times 3 \times 3600 = 0.7985 + 1.62 h_6^{3/2} \text{ M. m}^3$$

$$\Delta s = \left[(30 + 3 h_6) h_6 - (30 + 3 (0.624)) \times 0.624 \right] = 30 h_6 + 3 h_6^2 - 19.8881$$

$$\therefore 1.458 = (0.7985 + 1.62 h_6^{3/2}) + (30 h_6 + 3 h_6^2 - 19.8881)$$

$$\text{or } h_6^3 + 0.54 h_6^{3/2} + 10 h_6 - 6.8492 = 0$$

Solving this by hit and trial, we get $h_6 = 0.620 \text{ m}$

Since $h_6 < h_5$, the head has started reducing.

$$\text{Hence } h_{\text{max}} = h_5 = 0.624 \text{ m}$$

Corresponding maximum outflow discharge = $300 (0.624)^{3/2} = 147.88 \text{ cumecs}$

Example 8.6. A multipurpose project has a total cost of 240 million rupees. For the data given below, calculate the allocations to each project purpose, by the following methods.

(a) Remaining benefits method (b) Alternate justifiable expenditure method.

Item	Flood control	Power generation	Irrigation
1. Separable costs	Rs. 33 million	Rs. 88 million	Rs. 72 million
2. Estimated benefits	Rs. 40 million	Rs. 138 million	Rs. 112 million
3. Alternate single purpose cost	Rs. 47 million	Rs. 104 million	Rs. 101 million

Solution : Total cost of project = 240 million = 240×10^6

Total separable cost = $(32 + 88 + 72)$ million = 192×10^6

∴ Total joint cost = $(240 - 192) 10^6 = 48 \times 10^6$

(a) Allocation of cost by remaining benefits method

The computation for allocation are done in the tabular form below.

(All cost in millions of rupees)

Line	Item	Flood control	Power generation	Irrigation	Total
1.	Benefits limited by alternate cost	40	104	101	245
2.	Remaining benefits	$40 - 32 = 8$	$104 - 88 = 16$	$101 - 72 = 29$	$245 - 192 = 53$
3.	Allocated joint cost	7.2453	14.4906	26.2641	48.0
4.	Total allocation	39.2453	102.4906	98.2641	240.0
5.	Total allocation (%)	16.352	42.705	40.943	100.0

In the table, line 1 gives the estimated benefits, or alternate single purpose cost, whichever is less. Thus the figures for flood control, power generation and irrigation are 40, 104 and 101 million rupees respectively, making a total of Rs 245 millions.

Line 2 gives the remaining benefits, which are obtained by subtracting the separate costs from the data of line 1.

Line 3 gives the allocated joint costs for each purpose, obtained as under :

Allocated joint cost = $\frac{\text{Total joint cost}}{\text{Total remaining benefits}} \times \text{remaining benefit of that purpose}$

Thus, allocated joint cost for flood control = $\frac{48}{53} \times 8 = 7.2453$ million

Allocated joint cost for power = $\frac{48}{53} \times 16 = 14.4906$ million

Allocated joint cost for irrigation = $\frac{48}{53} \times 29 = 26.2641$ million

Line 4 gives the total allocation, which is equal to the sum of separable cost and the allocated joint cost. Finally, line 5 gives the total allocation in percentage.

(b) Allocation of cost by alternate justifiable expenditure method

The computations for cost allocation are done in Tabular form below.

(All costs in millions of Rupees)

Line	Item	Flood control	Power generation	Irrigation	Total
1.	Alternate cost less separable cost	$47 - 32 = 15$	$104 - 88 = 16$	$101 - 72 = 29$	60
2.	Allocated joint cost	12	12.8	23.2	48.0
3.	Total allocation	$32 + 12 = 44$	$88 + 12.8 = 100.8$	$72 + 23.2 = 95.2$	240.0
4.	Total allocation (%)	18.333	42	39.667	100

Line 1 in the above table is obtained by subtracting separable cost from alternate single purpose cost, for each allocation.

Line 2 gives the allocated joint costs, obtained from the relation :

$$\text{Allocated joint cost} = \left[\frac{\text{Total joint cost}}{\text{Total alternate cost less separable cost}} \right] \times \left[\frac{\text{Alternate cost less separable cost of that purpose}}{\text{Total alternate cost less separable cost}} \right]$$

Thus, allocated joint cost for flood control = $\frac{48}{60} \times 15 = 12$ millions

Allocated joint cost for power = $\frac{48}{60} \times 16 = 12.8$ millions

Allocated joint cost for irrigation = $\frac{48}{60} \times 29 = 23.2$ millions

Line 3 gives the total allocation which is equal to the sum of separable cost and allocated joint cost for each purpose.

Line 4 gives the total allocation in percentage.

6.14. EXAMPLES FROM COMPETITIVE EXAMINATIONS

Example 6.7. The Muskingum method by McCarthy assumes the reach storage in a stream to be given by $S = K[xI + (1-x)O]$ where K is the storage constant. Also basic routing equation written for discrete time is

$$\frac{I_1 + I_2}{2} - \frac{O_1 + O_2}{2} t = S_2 - S_1$$

Derive from these Muskingum equation and incidentally determine the coefficients therein. What is the sum of coefficients? (Engg. Services Exam. 1986)

Solution

Given the reach storage equation as under :

$$S = K[xI + (1-x)O] \quad \dots(1)$$

Also, the basic routing equation is

$$\frac{I_1 + I_2}{2} t - \frac{O_1 + O_2}{2} t = S_2 - S_1 \quad \dots(2)$$

Substituting the following values of S_1 and S_2 from (1) into (2), we have

$$S_1 = K[xI_1 + (1-x)O_1] \text{ and } S_2 = K[xI_2 + (1-x)O_2]$$

$$\therefore \frac{I_1 + I_2}{2} t - \frac{O_1 + O_2}{2} t = K[xI_2 + (1-x)O_2] - K[xI_1 + (1-x)O_1]$$

$$\text{or } \frac{I_1 + I_2}{2} t + K[xI_1 + (1-x)O_1] = \frac{O_1 + O_2}{2} t + K[xI_2 + (1-x)O_2]$$

$$\text{or } (I_1 + I_2) + \frac{2K}{t} [xI_1 + (1-x)O_1] = O_1 + O_2 + \frac{2K}{t} [xI_2 + (1-x)O_2]$$

$$\text{or } I_1 + I_2 + \frac{KxI_1}{0.5t} + \frac{K}{0.5t} (1-x)O_1 = O_1 + O_2 + \frac{KxI_2}{0.5t} + \frac{K(1-x)O_2}{0.5t}$$

$$\text{or } \left[I_1 + \frac{KxI_1}{0.5t} \right] + \left[I_2 - \frac{KxI_2}{0.5t} \right] + \left[\frac{K(1-x)}{0.5t} O_1 - O_1 \right] = \left[O_2 + \frac{K(1-x)}{0.5t} O_2 \right]$$

$$\text{or } O_2 \left[\frac{0.5t + K(1-x)}{0.5t} \right] = I_1 \left[\frac{0.5t + Kx}{0.5t} \right] + I_2 \left[\frac{0.5t - Kx}{0.5t} \right] + O_1 \left[\frac{K(1-x) - 0.5t}{0.5t} \right]$$

or $O_2(K - Kx + 0.5t) = I_1(Kx + 0.5t) + I_2(-Kx + 0.5t) + O_1(K - Kx - 0.5t)$

or $O_2 = I_1 \left[\frac{Kx + 0.5t}{K - Kx + 0.5t} \right] + I_2 \left[\frac{-Kx + 0.5t}{K - Kx + 0.5t} \right] + O_1 \left[\frac{K - Kx - 0.5t}{K - Kx + 0.5t} \right]$

or $O_2 = C_0 I_2 + C_1 I_1 + C_2 O_1$

Which is the required Muskingum equation, where

$$C_0 = \frac{-Kx + 0.5t}{K - Kx + 0.5t}, C_1 = \frac{Kx + 0.5t}{K - Kx + 0.5t} \text{ and } C_2 = \frac{K - Kx - 0.5t}{K - Kx + 0.5t}$$

Sum of the coefficients $C_0 + C_1 + C_2$

$$= \frac{1}{K - Kx + 0.5t} [-Kx + 0.5t + Kx + 0.5t + K - Kx - 0.5t] = 1$$

Example 6.8. The storage in a stream reach has been studied; x and K have been identified as 0.28 and 1.6 days. If the inflow hydrograph in the stream reach, as the flood starts coming in and passes, is given by the following table, compute the outflow hydrograph (Plotting is not needed).

Time (hours)	0	6	12	18	24	30
Inflow (m^3/sec)	35	55	92	130	160	140

Solution :

(Engg. Services Exam. 1986)

The Muskingum equation has been derived in the previous Example, in the form

$$O_2 = C_0 I_2 + C_1 I_1 + C_2 O_1$$

Here $x = 0.28$ and $K = 1.6$ days $= 1.6 \times 24 = 38.4$ hours, $t = 6$ hours.

Hence

$$C_0 = \frac{-Kx + 0.5t}{K - Kx + 0.5t} = \frac{-38.4 \times 0.28 + 0.5 \times 6}{38.4 - 38.4 \times 0.28 + 0.5 \times 6} = -0.253$$

$$C_1 = \frac{Kx + 0.5t}{K - Kx + 0.5t} = \frac{38.4 \times 0.28 + 0.5 \times 6}{38.4 - 38.4 \times 0.28 + 0.5 \times 6} = 0.449$$

$$C_2 = \frac{K - Kx - 0.5t}{K - Kx + 0.5t} = \frac{38.4 - 38.4 \times 0.28 - 0.5 \times 6}{38.4 - 38.4 \times 0.28 + 0.5 \times 6} = 0.804$$

$$\text{Sum: } C_0 + C_1 + C_2 = -0.253 + 0.449 + 0.804 = 1 \text{ (OK)}$$

The computations for outflow rate (i.e. O_2) at the end of each interval by the following Muskingum equation are carried out in the Tabular form below.

$$O_2 = C_0 I_2 + C_1 I_1 + C_2 O_1$$

Time (hours)	Inflow (cumecs)	$C_0 I_2 = -0.253 I_2$	$C_1 I_1 = 0.449 I_1$	$C_2 O_1 = 0.804 O_1$	Outflow (cumecs)
(1)	(2)	(3)	(4)	(5)	(6) = (3) + (4) + (5)
00	35	-	-	-	35.00
06	55	$-0.253 \times 55 = -13.915$	$0.449 \times 35 = 15.715$	$0.804 \times 35 = 28.14$	29.94*
12	92	$-0.253 \times 92 = -23.276$	$0.449 \times 55 = 24.695$	$0.804 \times 29.94 = 24.072$	25.49*
18	130	$-0.253 \times 130 = -32.89$	$0.449 \times 92 = 41.308$	$0.804 \times 25.49 = 20.494$	28.91
24	160	$-0.253 \times 160 = -40.48$	$0.449 \times 130 = 58.37$	$0.804 \times 28.91 = 23.244$	41.13
30	140	$-0.253 \times 140 = -35.42$	$0.449 \times 160 = 71.84$	$0.804 \times 41.13 = 33.068$	69.49

Column (6) of the above table gives the values of the outflow hydrograph. The initial computed outflow drops because of sharp increase in inflow.

Example 6.9. The following Table gives the monthly inflow into and contemplated demand from a reservoir

Month	Monthly demand ($M.m^3$)	Evaporation (cm)	Rainfall (cm)	Monthly inflow ($M.m^3$)
Jan.	50	6	1	50
Feb.	75	8	0	40
March	80	13	0	30
April	85	17	0	25
May	130	22	0	20
June	120	22	19	30
July	25	14	43	200
Aug.	25	11	39	225
Sept.	40	13	22	150
Oct.	45	12	6	90
Nov.	50	7	2	70
Dec.	60	5	1	60

Assume the reservoir area as $30 km^2$. The runoff coefficient of area flooded by the reservoir is 0.4. Estimate the minimum storage to meet the demand.

(Civil Services Exam. 1989)

Solution Note that $1 cm \cdot km^2 = \frac{1}{100} \times 10^6 m^3 = 10^4 m^3 = 0.01 M.m^3$.

Table below gives the data about evaporation, rainfall, effective rainfall and net inflow/outflow.

(1)	Month	Jan.	Feb.	March	April	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
(2)	Evap. (E) (cm)	6	8	13	17	22	22	14	11	13	12	7	5
(3)	Rainfall (R) (cm)	1	0	0	0	0	19	43	39	22	6	2	1
(4)	Eff. Rainfall $R_e = 0.4 R$ (cm)	0.4	0	0	0	0	7.6	17.2	15.6	8.8	2.4	0.8	0.4
(5)	Net inflow/outflow $= R_e - E$ (cm)	-5.6	-8.0	-13.0	-17.0	-22.0	-14.4	+8.2	+4.6	-4.2	-9.6	-6.2	-4.6
(6)	Net inflow/outflow volume ($M.m^3$) $= (5) \times 0.01 \times A$	-1.68	-2.4	-3.9	-5.1	-6.6	-4.32	+0.96	+1.38	-1.26	-2.88	-1.95	-1.38

Note that -ve = outflow and +ve = inflow into reservoir

The computations for reservoir capacity are done in a tabular form below.

Month	Monthly demand (Mm ³)	Monthly inflow (Mm ³)	Net inflow/outflow (from above Table) Mm ³	Net demand (M . m ³)	Cumulative demand (M . m ³)	Cumulative inflow (Mm ³)	Excess demand (Mm ³)	Excess supply (Mm ³)
(1)	(2)	(3)	(4)	(5) = (2) + (4)	(6)	(7)	(8) = (6) - (7)	(9)
Jan.	- 50	50	- 1.68	- 51.68	51.68	50	1.68	
Feb.	- 75	40	- 2.40	- 77.40	129.08	90	39.08	
March	- 80	30	- 3.90	- 83.90	212.98	120	92.96	
April	- 85	25	- 5.10	- 90.10	303.08	145	158.08	
May	- 130	20	- 6.60	- 136.60	439.68	165	274.68	
June	- 120	30	- 4.32	- 124.32	564.00	195	369.00	
July	- 25	200	+0.96	- 24.04	588.04	395	193.04	
Aug.	- 25	225	+1.38	- 23.62	611.66	620		8.34
Sept.	- 40	150	- 1.26	- 41.26	652.92	770		117.08
Oct.	- 45	90	- 2.88	- 47.88	700.80	860		159.20
Nov.	- 50	70	- 1.86	- 51.86	752.66	930		177.34
Dec.	- 60	60	- 1.38	- 61.38	814.04	990		175.96

Minimum storage required = Max. of excess demand = 369.00 Mm³

Example 6.10. The amounts of water flowing from a certain catchment area at the proposed dam site are tabulated below. Determine.

(i) The minimum capacity of the reservoir if water is to be used to feed the turbines of hydropower plant at a uniform rate and no water is to be spilled over.

(ii) The initial storage required to maintain the uniform demand as above. (Engg. Services Exam. 1999)

Solution

The computations are done in a tabular form below.

Month	Inflow ($\times 10^6 \text{ m}^3$)	Av. Demand ($\times 10^6 \text{ m}^3$)	Deficit ($\times 10^6 \text{ m}^3$)	Surplus ($\times 10^6 \text{ m}^3$)
(1)	(2)	(3)	(4) = (3) - (2)	(5) = (2) - (3)
January	2.83	10.8533	8.0233	
February	4.25	10.8533	6.6033	
March	5.66	10.8533	5.1933	
April	18.40	10.8533		7.5467
May	22.64	10.8533		11.7867
June	22.64	10.8533		11.7867
July	19.81	10.8533		8.9567
August	8.49	10.8533	2.3633	
September	7.10	10.8533	3.7533	
October	7.10	10.8533	3.7533	
November	5.66	10.8533	5.1933	
December	5.66	10.8533	5.1933	
Sum	130.24×10^6	130.24×10^6		40.0768×10^6

Average demand = $(\Sigma \text{ Inflow})/12 = 130.24 \times 10^6/12 = 10.853 \times 10^6 \text{ m}^3$

This value has been entered in column (3) of the above Table.

Since no water is to be spilled, minimum capacity will be equal to the sum of the surplus water

$\therefore$ Minimum capacity = $\Sigma \text{ Surplus} = 40.0768 \times 10^6 \text{ m}^3$

Initial storage = $\Sigma \text{ Surplus} - \Sigma \text{ Deficit before January}$

$$= \{40.0768 - (2.8633 + 3.7533 + 3.7533 + 5.1933 + 5.1933)\} \times 10^6 \\ = 19.8203 \times 10^6 \text{ m}^3$$

PROBLEMS

1. Explain various types of reservoirs. What do you understand by multipurpose reservoir?
2. Describe in brief various investigations required for reservoir planning.
3. What are the factors on which the selection of the site of a reservoir depend?
4. Define the following : surcharge storage, valley storage, safe yield and secondary yield.
5. What do you understand by mass inflow curve and how is it prepared?
6. What do you understand by demand curve ? Explain the method of calculating reservoir capacity for a specified yield, from the mass inflow curve.
7. Explain how would you determine safe yield from a reservoir of a given capacity.
8. Write a note on reservoir sedimentation. How do you estimate the probable life of a reservoir ?
9. Discuss various methods of reservoir sediment control.
10. What is flood routing ? Explain the basic flood routing equation and outline its method of solution.
11. Explain the graphical method of flood routing.
12. Describe the trial and error method of flood routing.

Dams-I : General

7.1. INTRODUCTION

A dam is a hydraulic structure constructed across a river to store water on its up-stream side. It is an impervious or fairly impervious barrier put across a natural stream so that a reservoir is formed. This water is then utilised as and when it is needed. Due to the construction of the dam, water level in the river at its up-stream side is very much increased, and a large area may be submerged depending upon the water spread of the reservoir so formed.

Classification of Dams. Dams may be classified into different categories, depending upon the purpose or basis of the classification. Table 8.1 gives a summary of various types of dams.

TABLE 7.1. TYPES OF DAMS

Basis of Classification	Types	Common Examples
(a) Classification according to use	(i) Storage dam	Gravity dam, earth dam, rockfill dam, Arch dam etc.,
	(ii) Diversion dam	Weir, barrage
	(iii) Detention dam	Dike, water spreading dam, debris dam
(b) Classification by hydraulic design	(i) Overflow dam	Spillway
	(ii) Non-overflow dam	Gravity dam, earth dams, rockfill dam
(c) Classification by materials	(i) Rigid dams	Gravity dam, arch dam, buttress dam, steel dam, timber dam
	(ii) Non-rigid dams	Earth dam, rockfill dam

7.2. CLASSIFICATION ACCORDING TO USE

Based on use, dams are classified as follows :

- (i) Storage dam,
- (ii) Diversion dam,
- (iii) Detention dam.

Storage dam. This is the most common type of dam normally constructed. Storage dam is constructed to impound water to its upstream side during the periods of excess

(348)

DAMS-I : GENERAL

347

supply in the river (i.e. during rainy season) and is used in periods of deficient supply. Behind such a dam, a reservoir or lake is formed. The storage dams may be constructed for various purposes, such as for irrigation, water power generation or for water supply for public health-purposes, or it may be for a multi-purpose project. A storage dam may be constructed of wide variety of materials, such as stone, concrete, earth and rockfill etc.

Diversion dam. The purpose of a diversion dam is essentially different. While a storage dam stores water at its upstream for future use, a diversion dam simply raises water level slightly in the river and thus provides head for carrying or diverting water into ditches, canals, or other conveyance systems to the place of use. A diversion dam is, therefore, of smaller height and no reservoir is formed to store water. The common examples of diversion dams are weirs and barrages. During the floods, water passes over or through these diversion dams while during periods of normal flow, the river water, partly or wholly, is diverted to irrigation channel etc. A diversion dam may be constructed for irrigation or municipal or industrial uses.

Detention dam. A detention dam is constructed to store water during floods and release it gradually at a safe rate, when the flood recedes. By the provision of artificial storage during the floods, flood damage downstream is reduced. See Fig. 6.1 for the flood hydrograph which is moderated by the flood control reservoir formed by the construction of a detention dam. There are usually two types of detention dams. In the first type discussed above, water is temporarily stored and released through a suitable outlet structure. In the other type of detention dam, water is not released and no outlet structure is provided. Instead, water is held in the reservoir as long as is possible. This held water seeps into pervious banks and foundation strata. Due to this seepage of water, water-level in wells in the adjoining area is increased and lift irrigation may be possible. Also, irrigation may be done in the river bed itself at the downstream side. The seepage water may be sufficient for the growth of the crop and no additional surface watering may be necessary. Such a detention dam is sometimes called *water-spreading dam* or *dike*, and is similar to a weir in constructional features. Some times, detention dams are constructed across tributaries carrying large silt and sediment. In such a case, it is known as a *debris dam* used to trap the sediment and thus to exclude the sediment to flow to the main reservoir formed on the main river to which the tributary meets.

In a multipurpose river valley project, the dam may serve the purposes of storage, flood protection and recreation. The stored water may also be used for irrigation, water power generation, municipal and industrial supply and other purposes.

7.3. CLASSIFICATION ACCORDING TO HYDRAULIC DESIGN

According to hydraulic design dams may be classified as follows (Fig.7.1) :

1. Non-overflow dam.
2. Overflow dam.

Non-overflow dam. A non-overflow dam is the one in which the top of the dam is kept at a higher elevation than the maximum expected high flood level. Water is not permitted to overtop the dam. Hence a non-overflow dam may be constructed of wide variety of materials, such as a earth, rockfill, masonry, concrete etc. Fig. 7.1(a) shows a typical non-overflow type concrete gravity dam.

Overflow dam. An overflow dam is the one which is designed to carry surplus discharge (including floods) over its crest. Its crest level is kept lower than the top of the other portion of the dam (i.e. non-overflow dam). Since water glides over its downstream face it should be made of such a material which is not easily eroded by flowing water. Such dams are generally made of concrete or masonry. An overflow dam is commonly known as a *spillway* [Fig. 7.1(b)].

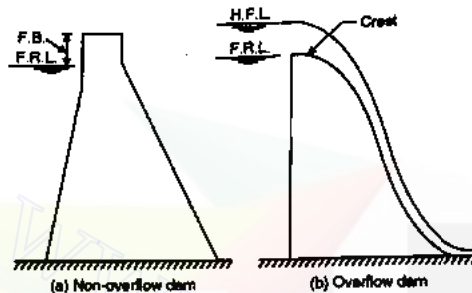


FIG. 7.1

Very often, in a river valley project, the two types of dams are combined. The main dam is kept as non-overflow dam made of either rigid materials such as masonry or concrete or non-rigid material such as earth and rockfill and some portion of dam is kept as overflow dam (spillway) at some suitable location along the main dam.

7.4. CLASSIFICATION ACCORDING TO MATERIAL

According to this most common classification, the dam may be classified as follows:

1. Rigid dams.

2. Non-rigid dams

Rigid dams. Rigid dams are those which are constructed of rigid materials such as masonry, concrete, steel or timber. Rigid dams may be further classified as follows:

1. Solid masonry or concrete gravity dam.

2. Arched masonry or concrete dam.

3. Concrete buttress dam.

4. Steel dam.

5. Timber dam.

Non-rigid dams. Non-rigid dams are those which are constructed of non-rigid materials such as earth and/or rockfill. The most common types of non-rigid dams are :

1. Earth dam.

2. Rockfill dam.

3. Combined earth and rockfill dam.

7.5. GRAVITY DAMS

A *gravity dam* is the one in which the external forces (such as water pressure, wave pressure, silt pressure, uplift pressure etc.) are resisted by the weight of the dam itself. Thus the forces disturbing the stability of the dam are resisted by the gravity forces of the mass of the dam. A gravity dam may be constructed either of masonry or of concrete. *Masonry gravity dams* are now-a-days constructed of only small heights. All major and important gravity dams are now constructed of concrete only [Fig. 7.1(a)]. A gravity dam may be either straight or curved in plan.

Advantages of gravity dams. There is no type of dam more permanent than one of the solid concrete, nor does any other type require less for maintenance. As compared to earth and rockfill dams gravity dams have the following advantages :

1. Gravity dams are relatively more strong and stable than earth dams. They are particularly suited across gorges having very steep side slopes where earth dam, if constructed, might slip.

2. Gravity dams are well adapted for use as an overflow spillway crest. Earth dams cannot be used as overflow dams. Due to this, a gravity overflow dam is often used for the spillway feature of earth and rockfill dams.

3. Gravity dams can be constructed of any height, provided suitable foundations are available to bear the stresses. The height of an earth dam is usually limited by the stability of its slopes requiring a very wide base width. Highest dams in the world are made of gravity dams only.

4. Gravity dam is specially suited to such areas where there is likelihood of very heavy downpour. The slopes of earth dam might get washed away in such a situation.

5. A gravity dam requires the least maintenance.

6. The failure of a gravity dam, if any, is not sudden. It gives enough warning time before the area to downstream side is flooded due to the damage to the gravity dams. On the contrary, an earth dam generally fails suddenly.

7. Deep-set sluices can be used in the gravity dams, to retard the sedimentation or silt deposit in the reservoir. The trap efficiency of a reservoir of an earth dam is more than that of a reservoir of gravity dam.

8. A gravity dam is cheaper in the long run since it is more permanent than any other type. Thus the *benefit-cost* ratio of such a dam is always higher.

Disadvantages. The disadvantages of gravity dam, as compared to an earth dam are as follows :

1. Gravity dams can be constructed only on sound rock foundations. They are unsuitable on weak foundations or on permeable foundations on which earth dams can be constructed with suitable foundation treatment.

2. The initial cost of a gravity dam is always higher than an earth dam. Hence, where funds are limited and where suitable materials are available for the construction of an earth dam, the earth dam may be preferred.

3. If mechanised plants, such as manufacturing and transporting mass concrete, curing of concrete etc. are not available, a gravity dam may take more time to construct.

4. Gravity dams require skilled labour or mechanised plants for its construction.

5. It is very difficult to allow subsequent rise in the height of a gravity dam, unless specific provisions have been made in the initial design.

7.6. ARCH DAMS

An arch dam (Fig. 7.2) is a dam curved in plan and carries at major part of its water load horizontally to the abutments by arch action. This part of water load depends primarily upon the amount of curvature. The balance of the water load is transferred to the foundation by cantilever action. The thrust developed by the water load carried by arch action essentially requires strong side walls of the canyon to resist the arch forces. The weight of arch dams is not counted on to assist materially in

The solid dam made of concrete that is wide upstream in plan is called arch dam

the resistance of external loads. For this reason, uplift on the base is not an important design factor.

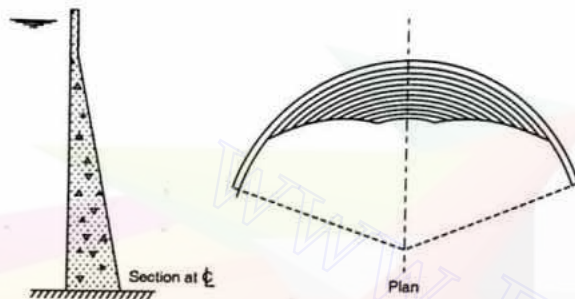


FIG. 7.2. ARCH DAM.

Advantages of Arch Dams

1. Arch dams are particularly adapted to the gorges where the length is small in proportion to the height.
2. For a given height, the section of an arch dam is much lesser than a corresponding gravity dam. Hence, an arch dam requires less material and is, therefore, cheaper.
3. Because of much less base width, the problems of uplift pressure are minor.
4. Since only a small part of water load is transferred to the foundation by cantilever action, an arch dam can be constructed in moderate foundations where gravity dam requiring sound foundation rock may be unsuitable.

Disadvantages

1. It requires skilled labour and sophisticated form work. The design of an arch dam is also quite specialized.
2. The speed of construction is normally slow.
3. It requires very strong abutments of solid rock capable of resisting arch thrust. Hence, it is not suitable in the locations where strong abutments are not available. Unfortunately, only few sites are suitable for this type of dam.

7.7. BUTTRESS DAMS

A buttress dam (Fig. 7.3) consists of a number of buttresses or piers dividing the space to be dammed into a number of spans. To hold up water and retain the water between these buttresses, panels are constructed of horizontal arches or flat slabs. When the panels consist of arches, it is known as *multiple arches type buttress dam*. If the panels consist of flat slab, it is known as *deck type buttress dam*. Other types of buttress dam are described in chapter 9.

Advantages of Buttress Dams

1. A buttress dam is less massive than a gravity dam. Hence, the foundation pressures are less in the case of a buttress dam, and it can be constructed even on weak foundations on which the gravity dam cannot be supported.

These buttresses are the walls of many in depth, which are spaced at regular intervals. They are supported on the foundation.

DAMS-1: GENERAL

2. The water load acts normal to the inclined deck. Hence the vertical component of the water load stabilises the dam against both overturning and sliding and the buttress dam, possesses a factor of safety much greater than that obtained in a gravity dam.

3. The ice pressure is relatively unimportant since the ice tends to slide over the inclined w/s deck.

4. In the case of gravity dam, the height of the dam can be raised only by the provision of crest shutter at overflow section. However in the case of a buttress dam, further raising of the height is possible and convenient by extending buttress and slab as shown in Fig. 7.4. Consequently, buttress dams are used where a future increase in reservoir capacity is contemplated.

5. Power houses and water treatment plants can be housed in between buttresses, thus saving some cost of construction.

6. The amount of concrete used in buttress dam is about 1/2 to 1/3 of the concrete used in gravity dam of the same height. However, the cost of construction of a buttress dam is not low in that ratio because of the increased cost of reinforcement and of form work.

7. Access is possible to the back of upstream face and to foundations between buttresses for periodic inspection and for subsequent grouting and drilling of pressure relief holes if required.

8. Depending on the degree of articulation or structural isolation provided, buttress dams may be designed to accommodate moderate amounts of foundation movement without serious damages.

9. The most efficient use of strength of concrete lead to the economy in quantity required, though the financial economy will not necessarily be directly proportional to the quantity of concrete saved.

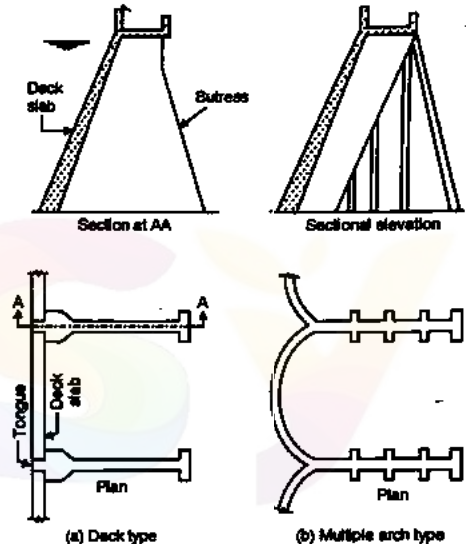


FIG. 7.3. BUTTRESS DAMS.

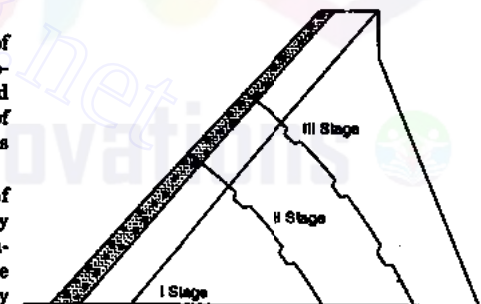


FIG. 7.4. RAISING HEIGHT OF BUTTRESS DAM.

10. The reduction in concrete volume and increase in the surface area to volume ratio provide for better heat dissipation during construction and possibly increased speed of construction because of the larger exposed area and the thinner section do not produce a problem for cooling.

Disadvantages of Buttress Dams

1. Skilled labour requirements and the shuttering concrete ratio are greater than for solid dams. This may lead to higher limit rates and so offset some of the saving due to reduction in the quantity of concrete.

2. Deterioration of upstream concrete surface has serious effects on buttress dams with very thin concrete face.

3. Buttress dam is more susceptible to willful damage. The degree of vulnerability would probably depend upon the thickness of the upstream face and the facility for access from the downstream side.

7.8. STEEL DAMS

Steel dams are constructed with a framework of steel with a thin skin plate as deck slab on the upstream side. In India, no such dams have been constructed. However, in United States three such dams have been constructed : Ash Fork Dam in Arizona (1898), Redridge Dam in Michigan (1905) and Hauser Lake Dam in Montana (1901). Out of these the first two dams gave satisfactory results while the third dam failed only after one year of service. The failure was mainly due to undermining of the foundation by leakage through or under the steel sheet pile.

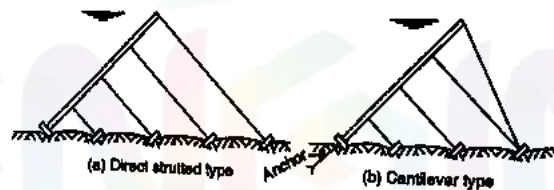


FIG. 7.5. STEEL DAMS.

Steel dams (Fig. 7.5) are generally of two types : *direct strutted type* and (ii) *cantilever type*. In the direct strutted type, the load on the deck plate is carried directly to the foundations through inclined struts. In the cantilever type, the section of the bent supporting the upper part of the deck is formed into a cantilever truss. This arrangement introduces a tensile force in the deck girders which is resisted either by anchoring the deck girder into the foundation at the upstream toe or by framing the entire bent rigidly together so that the moment of the weight of the water on the lower part of the deck may be utilized to offset the moment of the cantilever.

Advantages of Steel Dams

1. With the modern methods of steel fabrication, greater speed can be achieved in construction.
2. They have been found to be cheaper than other rigid dams.

3. Stresses in steel dams are more determinate. Hence, section can be designed economically and with confidence.

4. They have greater flexibility to resist unequal settlement without excessive leakage.

5. They are not affected by frost action.

6. Leaky joints, if any, can be repaired more easily by modern welding processes, than in hollow concrete dams.

Disadvantages

1. Steel dams are lighter, hence are not as adaptable to absorb the shock from vibrations of spilling water.

2. The life of steel dam is known to be shorter than a concrete dam.

3. Steel dam requires greater and more constant maintenance than concrete.

4. They have to be anchored at foundation which is difficult and precarious.

5. There is considerable concentration of bearing stresses.

7.9. TIMBER DAMS

A timber dam is constructed of framework of timber struts and beams, with timber plank facing to resist water pressure. A timber dam is an ideal temporary dam, though a well designed, constructed and maintained timber dam may last 30-40 years. They are suitable to places where timber can be available in plenty.

Timber dams are normally found to be of three types :

1. A-frame type (Fig. 7.6).
2. Rock-filled crib type (Fig. 7.7).
3. Beaver type (Fig. 7.8).

1. A-frame Type Timber Dam

Fig. 7.6 Shows typical A-frame type timber dam. It consists of five component parts : (a) sills, (b) struts, (c) wales, (d) studs and (e) lagging. The sills should be fastened to the ledge rock by wedge bolts or anchor bolts. The lagging should not be less than 5 cm thickness.

2. Rock Filled Crib Type Timber Dam

Fig. 7.7 shows such a type of dam in which cribs of square or round timber are drift bolted together. The timbers are spaced about

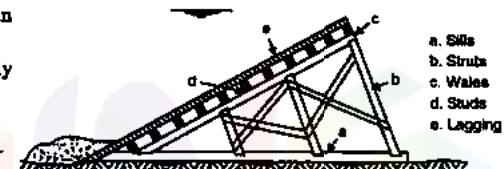


FIG. 7.6 A-FRAME TYPE TIMBER DAM.

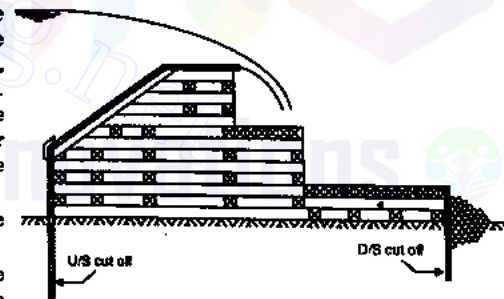


FIG. 7.7 ROCK-FILLED TIMBER CRIB DAM.

2-2.5 m centre to center both ways. The space between them is filled with rock fragments or boulders. In the case of rock foundation the bottom cribs are pinned to the rock foundation. If, however, the dam is constructed on earth foundation, sheet piling is provided both at the u/s as well as d/s side as shown in Fig. 7.7.

3. Beaver Type Timber Dam

Fig. 7.8 shows Beaver type timber dam which is the lowest in cost if plenty of timber is available. This is used only for low height and u/s slope of the dam is not kept steeper than 1 on 2. It consists of a number of timbers the butts of which point downstream, and between the butts are placed spacer logs which are drift pinned. Fill of earth or sand is placed over the bottom of deck for its stability.

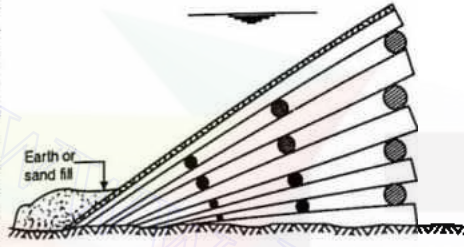


FIG. 7.8. BEAVER TYPE TIMBER DAM.

Advantages

1. Low initial cost.
2. Suitable for any type of foundation.
3. Where only temporary dams are to be constructed timber dams are more suitable.
4. Greater speed in construction can be achieved

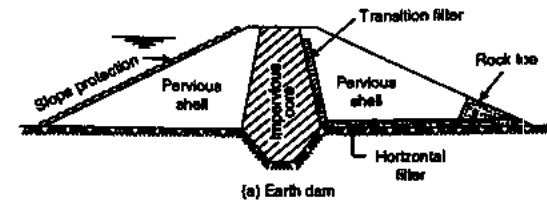
Disadvantages

1. High maintenance cost.
2. Life is short.
3. Suitable only for small heights.
4. Greater seepage loss through the body of the dam.

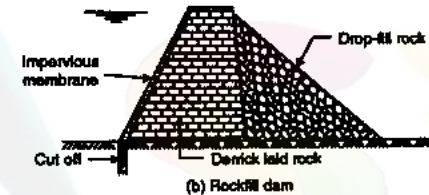
7.10. EARTH DAMS AND ROCKFILL DAMS

Earth dams are made of locally available soils and gravels and therefore, are most common types of dams used upto moderate heights. Their construction involves utilization of materials in the natural state requiring a minimum of processing. With the advancing knowledge of soil mechanics and with the advent of more sophisticated earth moving equipment, earth dams are now becoming more common, even for higher heights. The foundation requirements of earth dams are less stringent than for other types. Fig 7.9(a) shows a typical section of composite earth dam.

A rockfill dam is an embankment which uses variable sizes of rock to provide stability and an impervious membrane to provide water tightness. In modern practice, the rockfill dam has four fundamental parts : (1) dumped rockfill at the downstream, (2) upstream rubble cushion of laid-up stone bounding into the dumped rock, (3) upstream impervious facing resting on rubble cushion, and (4) upstream cutoff to check under seepage. Fig. 7.9(b) shows a typical section. Fig. 7.9(c) shows a combined earth and rockfill dam in which the usual derrick laid rockfill is replaced by the hydraulic earthfill and no upstream impervious membrane is provided.



(a) Earth dam



(b) Rockfill dam



FIG. 7.9 EARTH AND ROCKFILL DAMS.

Advantages of Earth and Rockfill Dams

1. Earth dams can be constructed almost on any type of available foundations. However, rockfill dams impose some restriction on the quality of foundation but they are much less rigid than those required for any other type.
2. They can be constructed rapidly with relatively unskilled labour and with materials available on the spot. Hence, it can be constructed even under famine relief work.
3. They are generally cheaper than other types.
4. They can be subsequently raised in height without much difficulty if such a need arises in future.

Disadvantages

1. They are more vulnerable to damage by floods and fail suddenly without sufficient warning.
2. They cannot be used as overflow dams. Separate spillways are to be provided at some site away from the dam, to pass the surplus discharge.
3. They are not suitable at the locations where heavy downpour is more common.
4. They require heavy maintenance cost and constant supervision.

7.11. PHYSICAL FACTORS GOVERNING SELECTION OF TYPE OF A DAM

The selection of a type of a dam at a given site depends upon many physical factors such as topography, geological and foundation conditions, available materials, suitable site for spillway, data about earthquake etc. It is only in exceptional circumstances that an experienced designer can say that only one type of dam is suitable or most economical. Preliminary designs and estimates are required for several types of dams before one can be shown to be the most economical. Before selecting the best type of dam at a particular site, one must consider the characteristics of each type of dam, as related to the physical features of the site and the adaption to the purposes the dam is supposed to serve, as well as economy, safety and other pertinent limitations. The choice of a dam may also be guided by many local problems such as availability of labour and equipment, accessibility of site, limitations imposed by outlet works and cost of protection needed from spillway discharge as well as time required for its construction. Some of the physical factors governing the selection of type of dam are discussed below.

1. **Topography.** The first choice of dam is usually governed by the topography for the site. A low rolling plains country suggests an earth dam with a separate spillway. A low narrow V-shaped valley suggests an arch dam, provided the top width of valley is less than one-fourth its height and separate site for spillway is available. A narrow stream flowing between high rocky walls (giving rise to a U-shaped valley) would suggest a concrete overflow dam. For intermediate conditions, other factors such as foundation condition, location of suitable site for spillway and availability of materials of construction play an important role in the selection of the type.

2. **Geology and Foundation Conditions.** The next important factor is the geology and foundation conditions. If the foundation consists of sound rock, with no fault or fissures, any type of dam can be constructed on it. Rocks like granite, gneiss and schist make very satisfactory foundation for gravity dam. However, these rocks may have seams or fractures. The removal of disintegrated rock together with the sealing of seams and fractures by grouting will frequently be necessary. Poor rock or gravel foundations are suitable for earth dam, rockfill dam or low concrete gravity dam. Since there will be considerable under-seepage in this case, effective water cutoffs or seals have to be provided. Silt or fine sand foundations have the problems of settlement, seepage and toe-erosion. Hence, such foundations are suitable only for either earth dam or low concrete gravity dam but not rockfill dams. Clay foundations have often the problems of long range consolidation under the weight of the dam, resulting in cracks. Hence, only earth dams are suitable with proper foundation treatment. Gravity dams or rockfill dams are not suitable on clay foundations.

3. **Materials of Construction.** The next important factor is the availability of materials of construction for dam. The cost of construction of a particular type of dam will depend upon the availability of the materials in nearby area so that transportation charges are reduced. If sand, gravel and crushed stone is available, a concrete gravity dam may be more suitable. If, however, coarse and fine grained soils are available an earth dam may be suitable. The preliminary selection of a particular type, based on the first two physical factors, must correspond with the easy availability of the materials required for its construction otherwise that type of dam should be dropped.

4. **Spillway Size and Location.** The safe discharge of flood water through dam is very essential, and for that suitable site for spillway should be available. If the area is such that a large spillway capacity is required, an overflow concrete gravity dam should be preferred. Where small spillway capacity is required and where separate site for spillway location is available, an earth dam may be preferred. Where large discharges are to be derived during the construction of dam, a concrete gravity dam is preferred to an earth dam. If no other site is available for spillway, it has to be accommodated with the main dam across the main river section. In that case, a concrete gravity dam having overflow section in its middle would be a better choice, though an earth dam can also be constructed with central overflow section of concrete.

5. **Roadway.** If a roadway is to be passed over the top of the dam, an earth dam or gravity dam would be preferred.

6. **Length and Height of Dam.** If the length of the dam is very long and its height is low, an earth dam would be a better choice. If the length is small but height is more, gravity dam is preferred.

7. **Life of Dam.** Concrete or masonry gravity dams have very long life. Earth and rockfill dams have intermediate life. However, timber dams are adopted only for temporary storages.

7.12. SELECTION OF SITE FOR A DAM

The following are the requisites of good sites for various types of dams :

1. **Foundations.** Suitable foundations should be available at the site selected for a particular type of dam. For gravity dams, sound rock is essential. For earth dams, any type of foundation is suitable with proper treatment. In general, however, the foundation should be free from seams, open pockets or fault planes. Formation in which hard and soft layers alternate are not generally good, because the penetration of water may weaken the soft layers and lead to movement along them. Alternations of sand, stones and shales may also lead to slipping during excavation of the trench. Beds which dip gently upstream and strike across the valley provide probably the best foundation conditions among sedimentary rocks, as the resultant of the weight of the dam and water pressure will then act nearly at right angles to the bedding planes of the strata. No dam should be built across a fault known to have been active in recent times. The best conditions are when a dam can be built on one uniform formation; if more than one kind of rock is present in the foundation, different bearing strengths may lead to differential settlement of the structure.

2. **Topography.** (i) The river cross-section at the dam site should preferably have a narrow gorge to reduce the length of the dam. However, the gorge should open out upstream to provide large basin for a reservoir.

(ii) A major portion of the dam should preferably be on high ground, as this would reduce the cost and facilitate drainage.

3. **Site for Spillway.** Good site for the location of a separate spillway is essential especially in the case of earth or rockfill dam. However, in the case of gravity dam, spillway may be located at its middle. The best site for a dam may be considered to be one where a deep gorge and a flank at its sides are separated by a hillock

higher than the height of the dam. In that case the main dam may be located in the deep gorge while the spillway can be located in the flank.

4. **Materials.** Materials required for a particular type of dam should be available nearby, without requiring much of transportation. This would very much reduce the cost of construction.

5. **Reservoir and Catchment Area.** (i) The site should ensure adequate storage capacity of reservoir basin at a minimum cost.

(ii) The cost of land and property submerged in the water spread area should be minimum.

(iii) The reservoir site should be such that quantity of leakage through its side and bed is minimum. Reservoir site having the presence of higher permeable rocks reduce the water-tightness of the reservoir.

(iv) The geological conditions of the catchment area should be such that percolation losses are minimum and maximum run-off is obtained.

(v) The reservoir site should be such that it avoids or excludes water from those tributaries which carry a high percentage of silt in water.

(vi) The site should be such that a deep reservoir is formed. A deep reservoir is preferable to shallow one because of (a) lower cost of land submerged per unit of capacity, (b) less evaporation losses because of reduction in the water spread area and (c) less likelihood of weed growth.

6. **Communication.** It would be preferable to select a site which is connected by a road or rail link or can be conveniently connected to the site for transportation of cement, labour, machinery food and other equipment.

7. **Locality.** The surroundings near the site should preferably be healthy and free of mosquitoes etc., as labour and staff colonies have to be constructed near the site.

PROBLEMS

1. Classify various types of dams. Distinguish clearly between rigid and non-rigid dams.
2. Discuss in brief merits and demerits of various types of dam.
3. Discuss, with illustrations the physical factors that govern the selection of type of dam.
4. What are the factors on which selection of site for a dam depends?
5. Write a note on foundation problems for dams and their remedies.

Dams-II : Gravity Dams

8.1. INTRODUCTION

A gravity dam is a structure so proportioned that its own weight resists the forces exerted upon it. This type of dam is the most permanent one, requires little maintenance and is most commonly used. The most ancient gravity dam on record was built in Egypt more than 400 years B.C. of uncemented masonry. Archeological experts believe that this dam was kept in perfect condition for many centuries.

A gravity dam may be constructed either of masonry or of concrete. However, now-a-days with improved methods of construction, quality control and curing, concrete is most commonly used for the construction of gravity dams. A gravity dam is mostly straight in plan and is known as 'straight gravity dam'. However, it may also be slightly curved in plan. A curved gravity dam resists the external forces by its weight and not by arch action. Most of the gravity dams are *solid*, so that no bending stress is introduced at any point and hence, they are sometimes known as 'solid gravity dams'. A gravity dam, however, can be hollow and is known 'hollow gravity dam'.

Gravity dams are particularly suited across gorges with very steep side slopes where earth dams might slip. Where good foundations are available, gravity dams can be built upto any height. The highest dams in the World are of gravity type.

8.2. FORCES ACTING ON A GRAVITY DAM

Following are the forces acting on a gravity dam. Some of these forces have been shown in Fig. 8.1.

1. Water pressure.	2. Weight of dam.
3. Uplift pressure.	4. Pressure due to earthquake.
5. Ice pressure.	6. Wave pressure.
7. Silt pressure.	8. Wind pressure

8.3. WATER PRESSURE

This is the major external force acting on dam. When the upstream face of the dam is vertical, the water pressure acts horizontally. The intensity of pressure varies triangularly, with a zero intensity at the water surface, to a value w/h at any depth h below water surface. When the upstream face is partly vertical and partly inclined, as shown in Fig. 8.1, the resultant water pressure can be resolved into two components:

(i) Horizontal component P_{wh} and (ii) Vertical component P_{wv} due to weight of water supported by the inclined face.

Thus, horizontal force,

$$P_{wh} = \frac{w H^2}{2} \quad \dots(8.1)$$

This acts at a height $H/3$ from the base of the dam.

Vertical component, P_{wv} = Weight of water contained by column $AA'C'B$ and acting at the C.G. of the area.

Similarly, if there is tail water of height H' on the downstream side, it exerts both horizontal pressure ($P_{wh'}$) as well as vertical pressure ($P_{wv'}$).

Thus, horizontal pressure $P_{wh'} = \frac{w H'^2}{2}$

and vertical pressure $P_{wv'}$ = Weight of water contained by column $FF'E'$

8.4. WEIGHT OF THE DAM

The weight of the dam is major resisting force. For analysis purposes, generally, unit length of the dam is considered. The cross-section of the dam may be divided into several triangles and rectangles, and the weights W_1, W_2, W_3 , etc. of each of these may be computed conveniently, along with determination of their lines of action. The total weight W of the dam acts at the C.G. of its section.

8.5. UPLIFT PRESSURE

Water has a tendency to seep through the pores and fissures of the foundation material. It also seeps through the joints between the body of the dam and its foundation at the base, and through the pores of the material in the body of the dam. The seeping water exerts pressure and must be accounted for in the stability calculations.

The *uplift pressure* is defined as the upward pressure of water as it flows or seeps through the body of the dam or its foundation. A portion of the weight of the dam will be supported on the upward pressure of water; hence net foundation reaction due to vertical force will reduce.

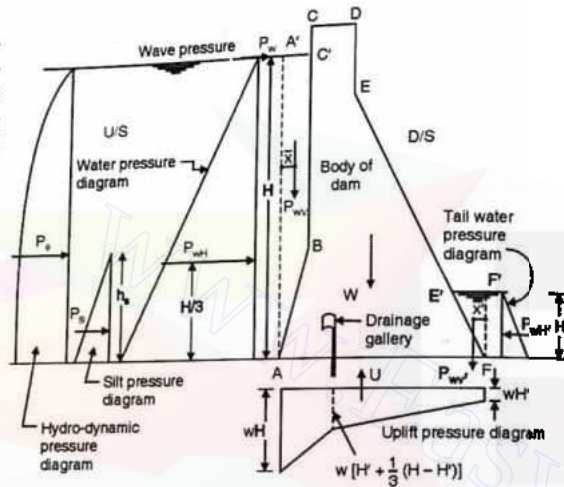


FIG. 8.1 FORCES ACTING ON A GRAVITY DAM.

Area over which uplift acts. The area over which the uplift pressure acts has been a question of investigation from the early part of this century. There are two schools of thought existing in the old literature for this. One school of thought recommends that a value one-third to two-thirds of the area should be considered as effective over which the uplift acts.

Terzaghi and Leliavski, the propagators of the second school of thought, recommend that the effective area may be taken approximately equal to the total area. The present practice in the design is to follow U.S.B.R. recommendation that the total area should be considered as effective to account for uplift.

U.S.B.R. Recommendations. U.S.B.R. suggests the adoption of uplift pressure intensities equal to the hydrostatic pressure of water at the toe and heel joined by a straight line in between. Sometimes drainage galleries are also provided in the body of the dam which releases the uplift pressure built up under it. The magnitude of the uplift pressure recommended at the face of the gallery is equal to the hydrostatic pressure at the toe plus $1/3$ rd the difference of the hydrostatic pressure at the heel and toe. From Fig. 8.1

The uplift pressure at heel $A = wH$

Uplift pressure at toe $F = wH'$

Uplift pressure at gallery $= w \left[H' + \frac{1}{3}(H - H') \right] \quad \dots(8.2)$

Indian Standard Recommendations (IS : 7894-1975) : According to the Indian Standard, there are two constituent elements in uplift pressure : the *area factor* or the percentage of area on which uplift acts and the *intensity factor* or the ratio which the actual intensity of uplift pressure bears to the intensity gradient extending from head water to tail water at various points.

Effective downstream drainage, whether natural or artificial, will generally limit the uplift at the toe of the dam to tail water pressure. Formed drains in the body of the dam and drainage holes drilled subsequent to grouting in the foundation, where maintained in good repair, are effective in giving a partial relief to the uplift pressure intensities under and in the body of the dam. The degree of effectiveness of the system will depend upon the character of the foundation and the dependability of the effective maintenance of the drainage system. In any case, observation of the behaviour of the dam will indicate the uplift pressures actually acting on the structure and when the uplift pressures are seen to approach or exceed design pressures, prompt remedial measures should necessarily be taken to reduce the uplift pressures to values below the design pressures.

Criteria for design. The following criteria are recommended for the calculating uplift forces :

(a) Uplift pressure distribution in the body of the dam shall be assumed, in case of both preliminary and final designs, to have an intensity which at the line at the formed drains exceeds the tailwater pressure by one-third the differential between reservoir level and tailwater level. The pressure gradient shall then be extending linearly to heads corresponding to reservoir level and tailwater level. The uplift shall be assumed to act over 100 percent of the area.

(b) Uplift pressure distribution at the contact plane between the dam and its foundations and within the foundation shall be assumed for preliminary designs to have an intensity which at the line of drains exceeds the tailwater pressure by one-third the differential between the reservoir and tailwater heads. The pressure gradient shall then be extended linearly to heads corresponding to reservoir level and tailwater level. The uplift shall be assumed to act over 100 % area. For final designs, the uplift criteria in case of dams founded on compact and unfissured rock shall be as specified above. In case of highly jointed and broken foundation, however, the pressure distribution may be required to be based on electrical analogy or other methods of analysis taking into consideration the foundation condition after the treatment proposed. The uplift shall be assumed to act over 100 % of the area.

(c) For the extreme loading conditions F and G (see § 8.11), the uplift shall be taken as varying linearly from the appropriate reservoir water pressure at the u/s face to the appropriate tailwater pressure at the d/s face. If the reservoir pressure at the section under consideration exceeds the vertical normal stress (computed without uplift) at the u/s face, a horizontal crack is assumed to exist and to extend from the u/s face towards the d/s face of the dam to the point where the vertical normal stress (computed on the basis of linear pressure distribution without uplift) is equal to the reservoir pressure at the elevation. The uplift is assumed to be the reservoir pressure from the u/s face to the end of the crack and from there to vary linearly to the tailwater pressure at the d/s face. The uplift is assumed to act over 100 % of the area.

(d) No reduction in uplift is assumed at the d/s toe of spillways on account of the reduced water surface elevation (relative to normal tailwater elevation) that may be expected immediately downstream of the structure.

(e) It is assumed that uplift pressure are not affected by earthquakes.

8.6. PRESSURE DUE TO EARTHQUAKE

The earthquake sets up primary, secondary, Raleigh and Love waves in the earth's crust. The waves impart accelerations to the foundations under the dam and causes its movement. In order to avoid rupture, the dam must also move along with it. This acceleration introduces an inertia force in the body of dam and sets up stresses initially in lower layers and gradually in the whole body of the dam. Earthquake wave may travel in any direction. However, it is customary to resolve the general direction of acceleration into horizontal and vertical directions.

Intensity of Earthquake

The intensity of an earthquake at a place is a measure of the strength of shaking during the earthquake, and is indicated by a number according to the Modified Mercalli Scale or M.S.K. Scale of seismic intensities. The comprehensive intensity scale (MSK 64), recommended by IS 1893 (Part 1) 2002, has rating numbers from 1 to 12. The detailed description about MSK 64, along with the damages caused corresponding to each number on the scale is available in IS 1893 (Part 1) : 2002.

Earthquake Zones of India : There are World maps which are divided into different zones according to the severity of earthquake. According to IS 1893-1984, India

was divided in five zones : zone I, zone II, zone III, zone IV and zone V. However, according to its revised version (IS 1893-2002), the seismic zone map is revised with only four zones, instead of five. Erstwhile zone I has been merged to zone II. Hence zone I does not appear in the new zoning map; only zone II, III, IV and V appear. The Killari area has been included in zone III and necessary modifications made, keeping in view the probabilistic hazard evaluation. The Bellary isolated zone has been removed. The part of eastern coast areas have shown similar hazard to that of Killari area; the level of zone II has been enhanced to zone III and connected with zone III of Godavari Graben area.

Spectra of Earthquake : Acceleration Spectra : Spectrum of an earthquake is the representation of the maximum dynamic response of idealised structure during an earth quake. The idealized structure is a 'single degree of freedom system' having a certain period of vibration and damping. The maximum response is plotted against the natural period of vibration (T) and can be expressed in terms of the following :

(i) Maximum absolute acceleration

(ii) Maximum relative velocity

or (iii) Maximum relative displacement.

For the purpose of design, acceleration spectra are very useful as they give the seismic force on the structure directly by multiplying it with the generalized or modal mass of the structure, i.e. Force = mass $\times$ earthquake acceleration. The earthquake acceleration is usually designated as fraction of the acceleration due to gravity and it is expressed as α_g , where α is known as the seismic coefficient.

Design seismic coefficients for different zones (IS : 1893-1984)

According to IS : 1893-1984, the design value of horizontal seismic coefficient (α_h) may be determined by one of the two methods.

(a) Seismic coefficient method (b) Response spectrum method

As per IS : 1893-1984, the seismic coefficient method has been suggested for dams upto 100 m in height and the response spectrum method has to be used for dams above this height. The idea of choosing a value of 100 m is that most of the dams upto this height will have periods (T) shorter than 0.3 seconds and the spectral acceleration may not be influenced by change of period in this range. Further, dams beyond 100 m in height may be thought to be more important requiring more sophisticated analysis.

(a) Seismic coefficient method

As per IS : 1893-1984, the design value of horizontal seismic coefficient (α_h) shall be computed as given by the following expression :

$$\alpha_h = \beta I \alpha_0 \quad \dots (3.8 a)$$

where β = Soil-foundation system factor, the value of which may be taken as 1.0 for dams

I = Importance factor, the value of which is taken as 2.0 for dams

α_0 = Basic seismic coefficient, the value of which for each of the four zones is given in Table 8.1 below.

TABLE 8.1. BASIC SEISMIC COEFFICIENT α_0 (IS : 1893-1984)

Seismic zone	II	III	IV	V
Basic seismic coefficient α_0	0.02	0.04	0.05	0.08

Thus, substituting the values of β and I for dams, Eq. 8.3 (a) reduces as under:

$$\alpha_h = 2 \alpha_0 \quad \dots(8.3)$$

As α_0 varies from 0.02 to 0.08, the value of α_h will vary from 0.04 to 0.16 for zones II to zone V.

(b) **Response spectrum method** : As per IS : 1893-1984, the design value of horizontal seismic coefficient is computed from the following expression :

$$\alpha_h = \beta I F_0 \frac{S_a}{g} \quad \dots(8.4a)$$

where β = Soil foundation system factor, the value of which may be taken as 1.0 for dams

I = Importance factor, the value of which is taken as 2.0 for dams

F_0 = Seismic zone factor for average acceleration spectra, the value of which for each of four zones is given in Table 8.2 below.

$\frac{S_a}{g}$ = average acceleration spectra as read from Fig. 8.2 for appropriate natural period (T) and damping of the structure.

TABLE 8.2. SEISMIC ZONE FACTOR (IS : 1893-1984)

Seismic zone	II	III	IV	V
Seismic zone factor F_0	0.10	0.20	0.25	0.40

Substituting the value of $\beta = 1.0$ and $I = 2.0$, for dams, Eq. 8.4 (a) reduces as under :

$$\alpha_h = 2 F_0 \frac{S_a}{g} \quad \dots(8.4)$$

For determining the value of S_a/g from Fig. 8.2, IS 1893-1984 recommends a damping of 5 percent for dams, and the natural period of vibration (T) is determined from the following expression :

$$T = 5.55 \frac{H^2}{B} \sqrt{\frac{w_m}{g E_s}} \quad \dots(8.5)$$

where

H = height of the dam (m)

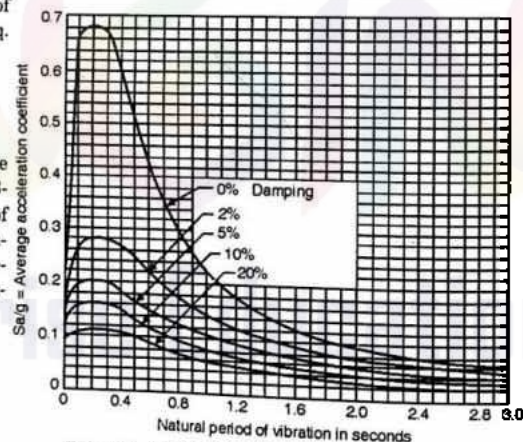


FIG. 8.2. AVERAGE ACCELERATION SPECTRA

B = base width of the dam (m)

w_m = unit weight of the material of dam (kN/m^3)

g = acceleration due to gravity (m/s^2)

and E_s = modulus of elasticity of the dam material (kN/m^2)

Taking $w_m = 24 \text{ kN/m}^3$, $E_s = 2.1 \times 10^7 \text{ kN/m}^2$, $g = 9.81 \text{ m/sec}^2$ and $B = 0.7 H$.

Eq. 8.5 reduces to

$$T = 5.55 \left(\frac{H^2}{0.7 H} \right) \sqrt{\frac{24}{9.81 \times 2.1 \times 10^7}} = 2.71 \times 10^{-3} H \quad \dots(8.5 a)$$

Thus, when $H = 100 \text{ m}$, $T = 0.271$ seconds for which S_a/g for 5% damping comes out to be 0.19 from Fig. 8.2.

Vertical seismic coefficient (α_v) : The vertical seismic coefficient may be taken as half of the horizontal seismic coefficient. Hence

$$\alpha_v = 0.5 \alpha_h \quad \dots(8.6)$$

Recommendations of IS : 1893-2002 : Design spectrum

According to IS 1893-2002, (Part 1) the design horizontal seismic coefficient (re-designated as A_h) shall be determined from the following expression :

$$A_h = \frac{Z}{2} \cdot \frac{I}{R} \cdot \frac{S_a}{g} \quad \dots(8.7)$$

where Z = Zone factor, given in Table 8.3. These zone factors are for the maximum considered Earthquake (MCE) and service life of the structure in a zone. The factor 2 in the denominator of Z is used so as to reduce the maximum considered earthquake (MCE) zone factor to the factor for design basis earthquake (DBE).

I = Importance factor,

R = Response reduction factor, depending on the perceived seismic damage performance of the structure, characterised by ductile or brittle deformations.

$\frac{S_a}{g}$ = average response acceleration coefficient for rock or soil sites, the values of which are given in Fig. 8.3 for 5% damping. For rocky or hard soil sites, the value of S_a/g may be found from the following expression :

$$\frac{S_a}{g} = \begin{cases} 1 + 15 T & \text{for } 0.00 \leq T \leq 0.10 \\ 2.50 & \text{for } 0.10 \leq T \leq 0.40 \\ 1.00/T & \text{for } 0.40 \leq T \leq 4.00 \end{cases} \quad \dots(8.8)$$

Thus, for gravity dam having T between 0.1 and 0.4 seconds, the value of $S_a/g = 2.5$

TABLE 8.3. ZONE FACTOR Z (IS : 1893 : 2002)

Seismic zone	II	III	IV	V
Seismic intensity	Low	Moderate	Severe	Very severe
Zone factor Z	0.10	0.15	0.24	0.36

Design acceleration spectrum for vertical motion : The design vertical acceleration may be taken as two-thirds of the design horizontal acceleration spectrum

Note Though the Code, IS 1893 (Part 1) 2002 has given the values of I and R for buildings, it has yet not given the values of I and R for other structures, such as liquid retaining structures, bridges and retaining walls, stack like structures and dams and embankments. Till parts 2, 3, 4 and 5 of the above Code are finalised, we will use the recommendations of IS 1893-1984 for dams. However, we shall consider zone I to be merged with zone II, as recommended in IS 1893-2002.

(a) EFFECT OF HORIZONTAL EARTHQUAKE ACCELERATION

Horizontal acceleration due to earthquake causes two effects :

1. Inertia force in the body of the dam
- and 2. Hydrodynamic pressure of water

1. Horizontal Inertia force : The inertia force is the product of mass and acceleration and this force acts in the direction opposite to that of the ground motion. The horizontal inertia force acts from upstream to downstream as well as from downstream to upstream. For *reservoir full condition*, the worst case occurs when the earthquake acceleration acts towards the upstream direction and the corresponding inertia force acts in the downstream direction. However, for the *reservoir empty condition*, the worst case occurs when the acceleration due to earthquake acts towards the downstream direction and the corresponding inertia force acts in the upstream direction :

The inertia force dF on any elemental lamina of the dam is equal to the product of the mass of the lamina and the acceleration.

Let dW = weight of the elemental lamina at any height of the dam

g = acceleration due to gravity

$$\alpha_h = \text{horizontal acceleration coefficient} = \frac{\text{Earth quake acceleration}}{\text{Acceleration due to gravity}}$$

Horizontal earthquake acceleration = $\alpha_h g$

Inertia force = mass $\times$ acceleration

$$= \frac{dW}{g} \cdot \alpha_h g = dW \cdot \alpha_h \quad \dots(8.9)$$

The computation of horizontal inertia force on the dam can be done by *seismic coefficient method* for dams upto 100 m height and by *response spectrum method* for dams of height greater than 100 m.

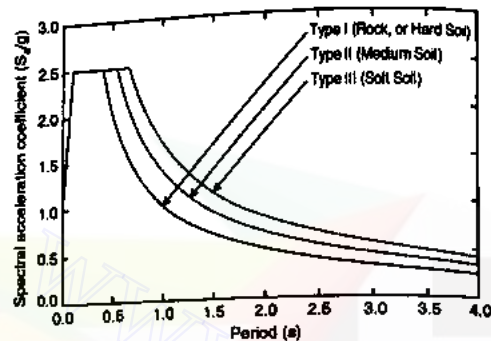


FIG. 8.3. RESPONSE SPECTRA FOR 5% DAMPING (IS : 1893-2002)

(i) Seismic Coefficient Method

In the seismic coefficient method, the horizontal seismic coefficient is assumed to vary triangularly with a maximum value of $1.5 \alpha_h$ at the top of the dam to a value of zero at the base the dam. Thus, the horizontal acceleration is assumed to have the shape of an inverted triangle. If one takes the shape of the dam as triangular (i.e. elementary profile), with zero width at the top and a width B at the bottom, with a height H , and the distribution of horizontal acceleration as an inverted triangle (Fig. 8.4), the value of base shear (F_H) will work out to be equal to $0.5 W \alpha_h$, as shown under.

Consider an elementary strip of thickness dy at a depth y from the top. The width of the strip is

$$B_y = \frac{B}{H} \cdot y$$

The horizontal acceleration coefficient at this level is

$$\alpha_y = 1.5 \alpha_h \left(1 - \frac{y}{H} \right)$$

If w_m is the unit weight of the material,

$$dW = B_y \cdot dy \cdot w_m$$

$\therefore$ Elementary horizontal inertia force on the strip = $dF_y = dW \cdot \alpha_y$, from Eq. 8.9

$$\begin{aligned} \therefore \text{Base shear } F_H &= \int_0^H dF_y = \int_0^H B_y \cdot dy \cdot w_m \cdot \alpha_y \\ &= \int_0^H \left(\frac{B}{H} y \right) \left(1 - \frac{y}{H} \right) 1.5 \alpha_h \cdot w_m dy \\ &= 1.5 \alpha_h \frac{B}{H} w_m \left(\frac{y^2}{2} - \frac{y^3}{3H} \right)_0^H \\ &= 0.25 \alpha_h B H w_m = 0.5 \alpha_h W \end{aligned} \quad \dots(8.10)$$

or

where $W = \frac{1}{2} B H w_m$ = total weight of the dam

However, in general, the dam has some top width from practical considerations.

Hence $F_H \triangleq 0.6 \alpha_h W \quad \dots(8.10 a)$

This compares well with the base shear coefficient given in the response spectrum method by the I.S. Code :

$$F_H = 0.6 W \alpha_h \quad (\text{See Eq. 8.11})$$

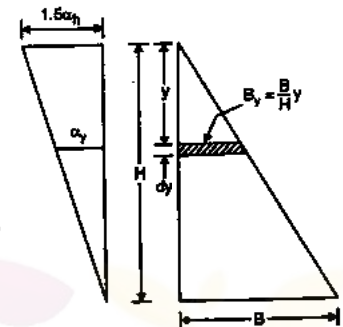


FIG. 8.4 VARIATION OF HORIZONTAL ACCELERATION ALONG THE DEPTH OF DAM

Computation of total horizontal force on the dam and its moment

Let us now take the case of a general profile of a gravity dam with top width b_1 upto depth H_1 with b_2 at depth H_2 and width $b_3 = B$ at total depth H below the top, as shown in Fig. 8.5. The distribution of seismic coefficient is in the form of inverted triangle, with a maximum value of $1.5 \alpha_h$ at the top and zero value at the base of the dam.

For portion of dam between sections 0-0 to 1-1

At any depth y' below the top, consider a strip of thickness dy' . The width of the strip $b'_y = b_1$ and the acceleration coefficient $\alpha'_y = 1.5 \alpha_h (1 - y'/H)$.

Hence elementary horizontal force $dF_1 = (1 \times b_1 dy') w_m \cdot \alpha'_y$

Hence horizontal inertial force for the portion of the dam between 0-0 and 1-1 is

$$F_1 = \int_0^{H_1} (b_1 dy' \cdot w_m) 1.5 \alpha_h (1 - y'/H)$$

$$F_1 = 1.5 \alpha_h w_m b_1 H_1 \left(1 - \frac{H_1}{2H}\right) \quad \dots(1)$$

The moment of the inertia force F_1 about section 1-1 is

$$M_1 = \int_0^{H_1} (b_1 dy' w_m) 1.5 \alpha_h (1 - y'/H) (H_1 - y')$$

$$M_1 = 1.5 \alpha_h w_m b_1 \frac{H_1^2}{2} \left(1 - \frac{H_1}{3H}\right) \quad \dots(2)$$

For portion of dam between sections 1-1 and 2-2

Consider an elementary strip of thickness dy at any depth y below the top of the dam. The width $b_y = b_1 + n(y - H_1)$, where n is the slope of the d/s face (i.e. n horizontal to 1 vertical)

Hence total horizontal inertia force on the portion of the dam above section 2-2 is given by

$$F_2 = F_1 + \int_{H_1}^{H_2} \{b_1 + n(y - H_1)\} dy w_m (1 - y/H) 1.5 \alpha_h$$

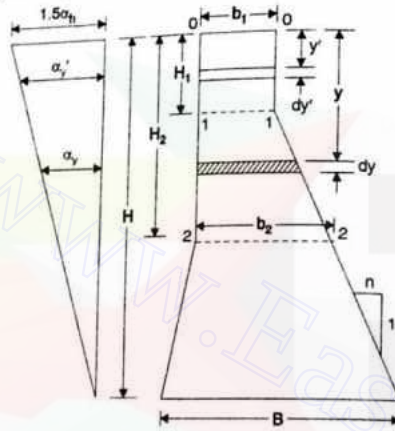


FIG. 8.5. GENERAL PROFILE OF DAM

$$\text{or } F_2 = F_1 + 1.5 \alpha_h w_m (H_2 - H_1) \left[b_1 \left(\frac{2H - H_2 - H_1}{2H} \right) + n \left(\frac{H_2 - H_1}{2} - \frac{2H_2^2 - H_1 H_2 - H_1^2}{6H} \right) \right] \quad \dots(3)$$

The moment of the inertia force F_2 about section 2-2 is given by

$$M_2 = M_1 + F_1 (H_1 - H_1) + \int_{H_1}^{H_2} (b_1 + n(y - H_1)) dy w_m (1 - y/H) 1.5 \alpha_h (H_2 - y) \quad \dots(4)$$

Thus, adopting the procedure outlined above, the total inertia force F and its moment about the base, for the whole dam can be computed.

(ii) Response spectrum method

As per IS 1893-1984, the response spectrum method is adopted for dams of height more than 100 m. According to the response spectrum method, the base shear F_B and the base moment are obtained by the following formulas :

$$F_B = 0.6 W \alpha_h \quad \dots(8.11)$$

and

$$M_B = 0.9 W \bar{h} \alpha_h \quad \dots(8.12)$$

where W = total weight of the masonry or concrete in the dam

$\bar{h}$ = height of the centre of gravity of the dam above the base, and

α_h = design horizontal seismic coefficient.

For any horizontal section at a depth y below the top of the dam, shear force F_y and bending moment M_y may be obtained as follows

$$F_y = C_y' \cdot F_B \quad \dots(8.13)$$

$$M_y = C_m' \cdot M_B \quad \dots(8.14)$$

where coefficients C_y' and C_m' are obtained from Fig. 8.6.

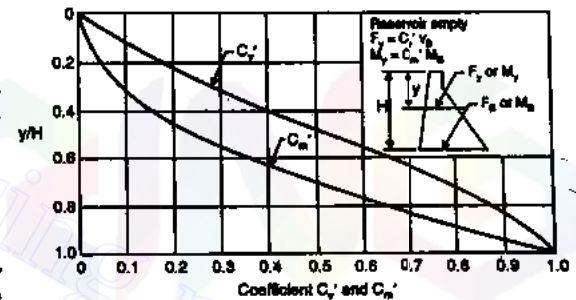


FIG. 8.6. VALUES OF C_y' and C_m'

2. Hydrodynamic pressure of water

The horizontal acceleration of the dam and foundation towards the reservoir causes a momentary increase in water pressure since the water resists the movement owing to its inertia. This *additional water pressure*, over and above the static water pressure, is known as *hydrodynamic pressure*. The following *simplified methods* are used to determine the hydrodynamic pressure which acts in the downstream direction.

(i) *Van Karman's method* : According to Von Karman, the hydrodynamic water pressure is assumed to be parabolic, and its magnitude (P_e) is given by

$$P_e = 0.555 \alpha_h w h^2 \quad \dots(8.15)$$

and it acts at a height of $4h/3\pi$ above the base of the dam, where h is the depth of water in the reservoir.

(iii) **Zanger's method** : In 1952, Zanger presented formulae for computing the hydrodynamic pressure exerted on vertical and sloping face of the dam. The formulae were derived by electrical analogy, based on the assumption that water is incompressible. IS 1893-1984 also recommends the same formulae. The pressure variation is elliptical-cum-parabolic. The hydrodynamic pressure intensity at a depth y below the maximum water level is given by

$$p_{ey} = C_y \alpha_h w h \quad \dots(8.16)$$

where C_y = a dimensionless pressure coefficient, at a depth y below water level.

It is given by

$$C_y = \frac{C_m}{2} \left[\frac{y}{h} \left(2 - \frac{y}{h} \right) + \sqrt{\frac{y}{h} \left(2 - \frac{y}{h} \right)} \right] \quad \dots(8.17)$$

C_m = maximum value of the pressure coefficient for a given constant slope, and its value may be obtained from Fig. 8.7 for different values of θ , where θ is the angle in degrees which the u/s face makes with the vertical. An approximate value of C_m may be obtained from the following relation :

$$C_m = 0.735 \left(1 - \frac{\theta}{90^\circ} \right) \quad \dots(8.18)$$

α_h = design horizontal seismic coefficient to be determined as follows :

In the seismic coefficient method, equivalent α_h at any depth y is given by $\alpha_y = \frac{F_y}{W_y}$

where F_y is the inertia force and W_y is the weight of the part of the dam above level y . In the response spectrum method, α_h is obtained from Eq. 8.4.

θ = angle, in degrees, the u/s face of the dam makes with vertical
 w = unit weight of water
 h = depth of water in reservoir
 y = the vertical distance from the reservoir surface to the elevation in question.

Fig. 8.8 shows the variation of earthquake pressure coefficient C_y with y/h ratio and the angle of inclination.

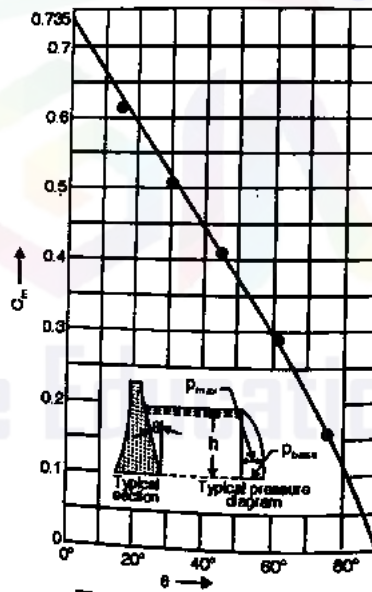


FIG. 8.7. VALUE OF C_m

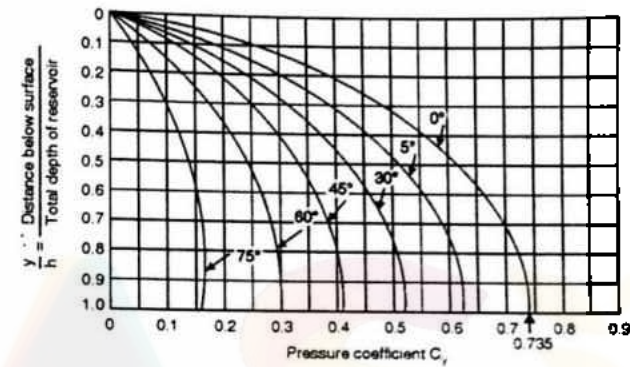


FIG. 8.8. EARTHQUAKE PRESSURE COEFFICIENT

The pressure variation along the depth is shown in Fig. 8.9.

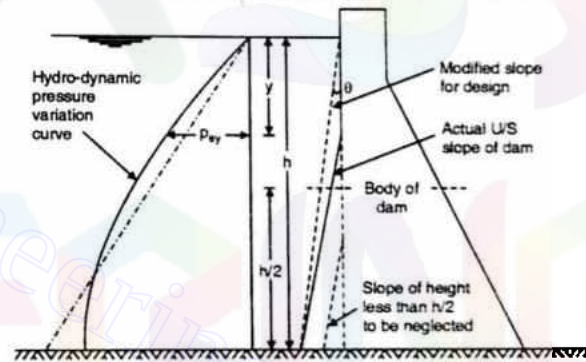


FIG. 8.9 PRESSURE VARIATION DUE TO HYDRODYNAMIC LOAD.

If the u/s face is inclined, and does not extend to more than half the depth of reservoir, it may be considered as vertical for computation of C_m . However, if the slope extends to more than half the depth, overall slope θ upto the whole height may be considered for finding the value of C_m as illustrated in Fig. 8.9.

The total horizontal pressure or hydrodynamic horizontal shear at depth y may be found by integrating the pressure curve above that plane. Taking the pressure variation to be elliptical-cum-parabolic, the total pressure at depth y will be equal to the average of the areas of the quarter ellipse and semiparabola.

$$\text{Hence } P_{ey} = \frac{1}{2} \left(\frac{\pi}{4} p_{ey} y + \frac{2}{3} p_{ey} y \right) = \frac{1}{2} p_{ey} y (0.7854 + 0.6667) = 0.726 p_{ey} y \quad \dots(8.19)$$

Similarly, the moment of pressure about the joint upto which the pressure is taken is given by half the sum of the moments of the quarter ellipse and semi-parabola.

$$\text{Hence } M_{cy} = \frac{1}{2} \left(\frac{4}{3} \pi y \times \frac{\pi}{4} p_{ey} \cdot y + \frac{2}{5} y \times \frac{2}{3} p_{ey} \cdot y \right) = \frac{1}{2} p_{ey} \cdot y^2 \left(\frac{1}{3} + \frac{4}{15} \right) = 0.299 p_{ey} \cdot y^2 \dots (8.20)$$

The corresponding values at the base of the dam are obtained by substituting the value of h for y in the above equation. Thus :

$$p_{eh} = C_m \cdot \alpha_h \cdot wh \dots (8.18 \text{ a})$$

$$P_{eh} = 0.726 p_{eh} \cdot h = 0.726 C_m \alpha_h \cdot wh^2 \dots (8.19 \text{ a})$$

$$M_{eh} = 0.299 p_{eh} \cdot h^2 = 0.299 C_m \alpha_h wh^3 \dots (8.20 \text{ a})$$

and
If there is tail water on the down stream side, hydrodynamic pressure (or suction) will also be exerted on the d/s face of the dam, the value of which may be found from the above formulae, where h is the tail water depth.

Effect of horizontal earthquake acceleration on the vertical component of reservoir and tail water load

Since the hydrodynamic pressure (or suction) acts normal to the face of the dam, there shall, therefore, be a vertical component of this force if the face of the dam against which it is acting is sloping and its magnitude at any section is given by

$$W_{hv} = (P_{e2} - P_{e1}) \tan \phi \dots (8.21)$$

where W_{hv} = increase (or decrease) in vertical component of load due to hydro-dynamic force

P_{e2} = total shear due to horizontal component by hydrodynamic force at the elevation of the section being considered.

P_{e1} = total shear due to horizontal component of hydrodynamic force at the elevation at which the slope of the dam face commence and

ϕ = Actual angle between the face of the dam and the vertical.

The moment due to the vertical component of reservoir and tail water load may be obtained by determining the lever arm from the centroid of the pressure diagram.

(b) EFFECT OF VERTICAL EARTHQUAKE ACCELERATION

Due to vertical earthquake acceleration, the dam as well as reservoir water are accelerated vertically upwards or downwards. This is resisted by the dam and water owing to their inertia. Hence the effect of vertical earthquake acceleration is to change the unit weight of water (w) and the dam material (w_m). An acceleration upwards increases the weights and an acceleration downwards decreases the weights. Thus the altered unit weights of the dam material and water, due to vertical acceleration will be $w_m (1 \pm \alpha_v)$ and $w (1 \pm \alpha_v)$ respectively.

To consider the effect of vertical earthquake acceleration, the vertical seismic coefficient would be as follows :

(a) For seismic coefficient method :

At the top of the dam, it would be equal to 0.75 times α_h value given by Eq. 8.3 and reducing linearly to zero, value at the base.

(b) For response spectrum method

At the top of the dam, it would be equal to 0.75 times α_h value given by Eq. 8.4 and reducing linearly to zero value at the base.

Knowing the value of α_v at any depth y , by each of the above two methods, the increase or decrease in the unit weights (w_m and w) may be obtained by adopting the same procedure as followed for the computation of inertia forces due to horizontal earthquake acceleration.

Note : While considering the inertia forces due to horizontal acceleration, the effect of vertical acceleration on the unit weights are not considered because it is unlikely that both horizontal and vertical accelerations of full magnitude will occur simultaneously.

8.7. ICE PRESSURE

The ice pressure is more important for dams constructed in cold countries, or at higher elevations. The ice formed on the water surface of the reservoir is subjected to expansion and contraction due to temperature variations. The coefficient of thermal expansion of ice being five times more than that of concrete, the dam face has to resist the force due to expansion of ice. This force acts linearly along the length of the dam, at the reservoir level.

IS 6512-1984 recommends that till specific reliable procedures become available for the assessment of ice pressure, it may be provided for at the rate of 250 kN/m² applied to the face of the dam over the anticipated area of contact of ice with the face of the dam.

8.8. WAVE PRESSURE

Waves are generated on the reservoir surface because of the wind blowing over it. Wave pressure depends on the height of the waves developed. Wave height may be calculated from the following formula given by D.A. Molitor (IS : 6512-1972).

$$h_w = 0.0322 \sqrt{V.F.} + 0.763 - 0.271 (F)^{1/4} \text{ for } F < 32 \text{ km} \dots (8.22)$$

$$\text{and } h_w = 0.0322 \sqrt{V.F.} \text{ for } F > 32 \text{ km} \dots (8.22 \text{ a})$$

where

h = height of waves in metres, between trough and crest

V = wind velocity in km per hour

F = fetch or straight length of water expanse in km

Fig. 8.10 shows the configuration of the wave and the approximated wave pressure distribution diagram.

The pressure intensity due to waves is given by

$$p_w = 2.4 w h_w \dots (8.23 \text{ a})$$

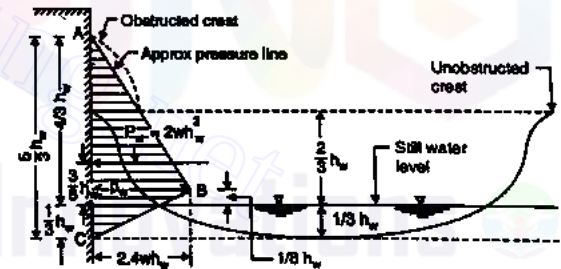


FIG. 8.10 WAVE PRESSURE.

where p_w is the maximum unit pressure which occurs at $\frac{1}{3} h_w$ metres above still water surface. The pressure distribution is curvilinear. However, for design purpose, the pressure distribution may be assumed to be represented by a triangle of height equal to $\frac{5}{3} h_w$. Hence, the total pressure P_w is given by

$$P_w = (2.4 w h_w) \times \frac{1}{2} \left(\frac{5}{3} h_w \right) = 2 w h_w^2 \quad \dots(8.23 b)$$

and this acts at a distance of $\frac{3}{8} h_w (= 0.375 h_w)$ above the reservoir surface, as shown in Fig. 8.10.

8.9. SILT PRESSURE

The river brings silt and debris along with it. The silt load gets deposited to an appreciable extent when dam is constructed. The dam is, therefore, subjected to silt pressure in addition to the water pressure. If γ_s is the submerged unit weight of silt and ϕ is the angle of internal friction and h_s is the height to which the silt is deposited, the silt pressure is given by

$$P_s = \frac{1}{2} \gamma_s h_s^2 \frac{1 - \sin \phi}{1 + \sin \phi} \quad \dots(8.24)$$

If the upstream face is inclined, the vertical weight of silt supported on the slope also acts as vertical force.

The U.S.B.R. recommendation presently followed in the design is to consider silt pressure as a fluid pressure which creates a horizontal pressure with its density equal to 360 kg/m^3 (3.53 kN/m^3), and a vertical pressure with its density equal to 920 kg/m^3 (9.03 kN/m^3). Thus, the total horizontal pressure will be equal to $360 \times \frac{h_s^2}{2} = 180 h_s^2 \text{ kg}$,

and the vertical pressure will be equal to $920 \times \frac{h_s^2}{2} = 460 h_s^2 \text{ kg}$. These pressures are in addition to the usual pressures.

According to Indian Standard (IS : 6512-1984), the silt pressure and water pressure exist together in submerged silt. The following criteria is recommended for calculating forces due to silt :

(a) Horizontal 'silt and water pressure' is assumed to be equivalent to that of a fluid weighing 1360 kg/m^3 (13.34 kN/m^3) and

(b) Vertical 'silt and water pressure' is determined as if silt and water together have a density of 1925 kg/m^3 (18.88 kN/m^3).

Since the horizontal and vertical components due to water have already been taken with $w = 1000 \text{ kg/m}^3$ (9.81 kN/m^3), the additional forces due to silt alone are computed by taking $\gamma_s = 1360 - 1000 = 360 \text{ kg/m}^3$ (3.53 kN/m^3) for horizontal pressure and $\gamma_s = 1925 - 1000 = 925 \text{ kg/m}^3$ (9.07 kN/m^3) for vertical pressure. These values compare favourably with the values recommended by USBR.

8.10. WIND PRESSURE

It is a minor force and need hardly be taken into account for the design of dams. Wind pressure is required to be considered only on that portion of the super structure which is exposed to the action of wind. Normally wind pressure is taken as 1 to 1.5 kN/m^2 for the area exposed to the wind pressure.

8.11. COMBINATION OF LOADING FOR DESIGN

(a) U.S.B.R. Recommendations. All the forces acting on a dam do not act simultaneously. There are various combinations of loads which may act simultaneously. Design of the dam should be based on the most adverse combination of the 'probable load condition'. The U.S.B.R. specifies the 'normal' and 'extreme' load combinations as below:

1. **Normal Load Combination.** (i) Normal water surface elevation, ice pressure, silt pressure and normal uplift. This particular class of loading is to be considered where ice force may develop.

(ii) Normal water surface elevation, earthquake force, silt pressure and normal uplift.

(iii) Maximum water surface elevation, silt pressure and normal uplift.

2. **Extreme Load Combination.** Maximum flood water elevation, silt pressure and extreme uplift with no drain in operation to release the uplift.

Reservoir-Empty Condition. The design is also to be checked for the reservoir empty condition. The condition is primarily important for testing stresses inside the dam and around opening provided in the body of the dam.

(b) Indian Standard Recommendations (IS 6512-1984).

Gravity dam design shall be based on the most adverse load conditions A, B, C, D, E, F or G, given below using the safety factors prescribed. Depending on the scope and details of the various project components, site conditions and construction programme, one or more of the loading combinations A, B, C, D and E may not be applicable ipso-facto and may need suitable modifications.

(i) **Load combination A (construction condition).** Dam completed but no water in reservoir and no tailwater.

(ii) **Load combination B (Normal operating condition).** Full reservoir elevation (or top of gates at crest), normal dry weather tailwater, normal uplift, ice and silt (if applicable).

(iii) **Load combination C (Flood discharge condition).** Reservoir of maximum flood pool elevation, all gates open, tailwater at flood elevation, normal uplift and silt (if applicable).

(iv) **Load combination D.** Combination A with earthquake.

(v) **Load combination E.** Combination B with earthquake but no ice.

(vi) **Load combination F.** Combination C, but with extreme uplift (drainage inoperative).

(vii) **Load combination G.** Combination E, but with extreme uplift (drainage inoperative).

Example 8.1. Seismic coefficient method

A 100 m high concrete gravity dam, shown in Fig. 8.11 is located in seismic zone IV. The unit weight of dam material is 24 kN/m^3 and the modulus of elasticity of the material is $2.05 \times 10^7 \text{ kN/m}^2$. Compute the forces induced due to earthquake, per unit length of the dam, using seismic coefficient method. Use recommendations of IS : 1893-1984.

Solution**(a) Inertia forces and moments**

For zone IV, we get from Table 8.1,

$$\alpha_0 = 0.05$$

Hence from Eq. 8.3

$$\alpha_h = 2 \alpha_0 = 2 \times 0.05 = 0.1$$

The distribution of horizontal seismic coefficient along the depth of the dam will be an inverted triangle, with $\alpha_h^T = 1.5 \alpha_h = 0.15$ at the top and zero value at the bottom.

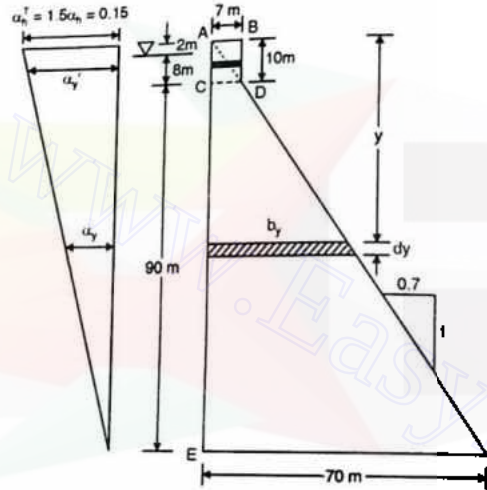


FIG. 8.11

At section CD : 10 m below top

Consider an elementary strip of thickness dy' at y' below the top.

Weight of strip $W_{y'} = (1 \times 7 dy') w_m = 7 \times 24 dy' = 168 dy'$

$$\alpha_{y'} = 1.5 \alpha_h (1 - y'/H) = 1.5 \times 0.1 (1 - y'/100) = 0.15 (1 - 0.01 y')$$

$$\therefore \text{Shear force, } F_{10} = \int_0^{10} \alpha_{y'} W_{y'} = \int_0^{10} 0.15 (1 - 0.01 y') 168 dy' = 25.2 \left[y' - \frac{0.01 y'^2}{2} \right]_0^{10} = 239.4 \text{ kN}$$

$$\begin{aligned} \text{Moment, } M_{10} &= \int_0^{10} \alpha_{y'} W_{y'} (10 - y') = \int_0^{10} 0.15 (1 - 0.01 y') 168 (10 - y') dy' \\ &= 25.2 \left[10 y' - \frac{0.1 y'^2}{2} - \frac{y'^2}{2} + \frac{0.01 y'^3}{3} \right]_0^{10} = 1218 \text{ kN-m} \end{aligned}$$

At the base : 100 m below top

Consider a strip of thickness dy at y below the top.

$$\text{Width of strip } b_y = 7 + 0.7 (y - 10) = 0.7 y$$

DAMS-II : GRAVITY DAMS

Weight of strip $W_y = 1 \times 0.7 y \times dy \times 24 = 16.8 y dy \text{ kN}$

$$\alpha_y = 0.15 (1 - 0.01 y)$$

$$\begin{aligned} \therefore \text{Shear force, } F_{100} &= F_{10} + \int_{10}^{100} \alpha_y \cdot W_y = 239.4 + \int_{10}^{100} 0.15 (1 - 0.01 y) 16.8 y dy \\ &= 239.4 + 2.52 \left[\frac{y^2}{2} - \frac{0.01 y^3}{3} \right]_{10}^{100} = 4321.8 \text{ kN} \end{aligned}$$

$$\begin{aligned} \text{Moment } M_{100} &= M_{10} + F_{10} (100 - 10) + \int_{10}^{100} \alpha_y \cdot W_y (100 - y) dy \\ &= 1218 + 239.4 \times 90 + \int_{10}^{100} 0.15 (1 - 0.01 y) 16.8 y (100 - y) dy \\ &= 22764 + 2.52 \left[\frac{100 y^2}{2} - \frac{y^3}{3} - \frac{y^3}{3} + \frac{0.01 y^4}{4} \right]_{10}^{100} = 21781 \text{ kN-m} \end{aligned}$$

(b) Hydrodynamic pressure and moment :

Depth of water in reservoir = $h = 98 \text{ m}$. The hydrodynamic pressure at any depth y below the reservoir surface is given by

$$p_{ey} = C_y \alpha_h \omega h = 9.81 \times 98 C_y \alpha_h = 961.38 C_y \alpha_h$$

where

$$C_y = \frac{C_m}{2} \left[\frac{y}{h} \left(2 - \frac{y}{h} \right) \right] + \sqrt{\frac{y}{h} \left(2 - \frac{y}{h} \right)}$$

Taking

$$C_m = 0.735 \text{ for vertical face } (\theta = 0)$$

$$C_y = \frac{0.735}{2} \left[\frac{y}{98} \left(2 - \frac{y}{98} \right) \right] + \sqrt{\frac{y}{98} \left(2 - \frac{y}{98} \right)}$$

Hence

$$p_{ey} = \frac{961.38 \times 0.735}{2} \left[\frac{y}{98} \left(2 - \frac{y}{98} \right) + \sqrt{\frac{y}{98} \left(2 - \frac{y}{98} \right)} \right] \alpha_h$$

or

$$p_{ey} = 353.307 \left[\frac{y}{98} \left(2 - \frac{y}{98} \right) + \sqrt{\frac{y}{98} \left(2 - \frac{y}{98} \right)} \right] \alpha_h$$

At 10 m from the top of the dam : $y = 8 \text{ m}$

$$p_{e10} = 353.307 \left[\frac{8}{98} \left(2 - \frac{8}{98} \right) + \sqrt{\frac{8}{98} \left(2 - \frac{8}{98} \right)} \right] \alpha_h = 195.142 \alpha_h$$

where

$$\alpha_h = \frac{F_{10}}{W_{10}} = \frac{239.4}{1 \times 7 \times 10 \times 24} = 0.1425$$

$$p_{e10} = 195.142 \times 0.1425 = 27.808 \text{ kN/m}^2$$

$$\therefore \text{Force (or shear) } P_{e10} = 0.726 p_{ey} \cdot y = 0.726 \times 27.808 \times 8 = 161.51 \text{ kN}$$

$$M_{e10} = 0.299 p_{ey} y^2 = 0.299 \times 27.808 (8)^2 = 532.13 \text{ kN-m}$$

At the base of the dam

$$y = 98 \text{ m.}$$

$$p_{ey} = 353.307 \left[\frac{98}{98} \left(2 - \frac{98}{98} \right) + \sqrt{\frac{98}{98} \left(2 - \frac{98}{98} \right)} \right] \alpha_h = 706.614 \alpha_h$$

where

$$\alpha_h = \frac{F_{100}}{W_{100}} = \frac{4321.8}{1 \left[7 \times 100 + \frac{1}{2} (7 + 70) 90 \right] 24} = 0.05094$$

$$p_{e100} = 706.614 \times 0.05094 = 35.9953 \text{ kN/m}^2$$

$$P_{e100} = 0.726 p_{ey} \cdot y = 0.726 \times 35.9953 \times 98 \triangleq 2581 \text{ kN}$$

$$M_{e100} = 0.299 p_{ey} \cdot y^2 = 0.299 \times 35.9953 (98)^2 = 103364 \text{ kN-m}$$

Example 8.2. Response spectrum method :

Solve example 8.1 using response spectrum method

Solution : Design horizontal seismic coefficient is given by Eq. 8.4 :

$$\alpha_h = \beta I F_0 \frac{S_a}{g} = 2 F_0 \frac{S_a}{g} \quad \dots(8.4)$$

where

$$\beta = 1 \text{ and } I = 2 \text{ for gravity dams.}$$

For zone IV, we get $F_0 = 0.25$ from Table 8.2, as per IS 1893-1984.

$$\text{Hence } \alpha_h = 2 \times 0.25 \frac{S_a}{g} = 0.5 \frac{S_a}{g}$$

The fundamental period of vibration of the dam is given by Eq. 8.5.

$$T = 5.55 \frac{H^2}{B} \sqrt{\frac{w_m}{g E_s}} = 5.55 \frac{(100)^2}{70} \sqrt{\frac{24}{9.81 \times 2.05 \times 10^7}} = 0.274 \text{ seconds}$$

Hence from Fig. 8.4, we get $S_a/g = 0.19$ for $T = 0.274 \text{ s}$ and 5% damping

$$\alpha_h = 0.5 \frac{S_a}{g} = 0.5 \times 0.19 = 0.095$$

(a) **Base shear.** The inertia force or base shear per unit length is given by $F_h = 0.6 W \alpha_h$

where

$$W = \text{weight of the dam} = 1 \left(7 \times 100 + \frac{63 \times 90}{2} \right) 24 = 84840 \text{ kN}$$

$$F_h = 0.6 \times 84840 \times 0.095 = 4835.88 \text{ kN}$$

(b) **Base moment.** $M_h = 0.9 W \bar{h} \alpha_h$

where

$$W = 84840 \text{ kN, as determined above}$$

$$\bar{h} = \frac{7 \times 100 \times \frac{100}{2} + \frac{1}{2} \times 63 \times 90 \times \frac{90}{3}}{7 \times 100 + \frac{1}{2} \times 63 \times 90} = 33.96 \text{ m}$$

$$M_h = 0.9 \times 84840 \times 33.96 \times 0.095 = 246342.6 \text{ kN-m}$$

(c) **Shear at $y (= 10 \text{ m})$ from the top**

$$F_y = C_v' \cdot F_h$$

For $y/h = 10/100 = 0.1$, we get $C_v' \triangleq 0.08$ From Fig. 8.8.

$$\therefore F_{10} = 0.08 \times 4835.88 \triangleq 387 \text{ kN}$$

(d) **Moment at $y (= 10 \text{ m})$ from top**

$$M_y = C_m' M_h$$

For $y/h = 10/100 = 0.1$, we get $C_m' \triangleq 0.02$

$$\therefore M_{10} = 0.02 \times 246342.6 \triangleq 4927 \text{ kN-m}$$

Note : Comparing seismic coefficient method and the response spectrum method, we find that the seismic coefficient method gives some what lower values at the base and considerably lower values near the top.

(e) **Hydrodynamic pressure**

$$p_{ey} = C_y \cdot \alpha_h w h$$

Here $\alpha_h = 0.095$ and is uniform for the entire height

$$h = 98 \text{ m}$$

$$w = 9.81 \text{ kN/m}^3$$

$$C_y = \frac{C_m}{2} \left\{ \frac{y}{h} \left(2 - \frac{y}{h} \right) + \sqrt{\frac{y}{h} \left(2 - \frac{y}{h} \right)} \right\} \text{ where } C_m = 0.735 \text{ for } \theta = 0$$

$$= \frac{0.735}{2} \left\{ \frac{y}{98} \left(2 - \frac{y}{98} \right) + \sqrt{\frac{y}{98} \left(2 - \frac{y}{98} \right)} \right\}$$

$$p_{ey} = 0.095 \times 9.81 \times 98 \times \frac{0.735}{2} \left\{ \frac{y}{98} \left(2 - \frac{y}{98} \right) + \sqrt{\frac{y}{98} \left(2 - \frac{y}{98} \right)} \right\}$$

or

$$p_{ey} = 33.564 \left\{ \frac{y}{98} \left(2 - \frac{y}{98} \right) + \sqrt{\frac{y}{98} \left(2 - \frac{y}{98} \right)} \right\}$$

At a depth of 8 m from reservoir level or 10 m from the top.

$$p_{e10} = 33.564 \left\{ \frac{8}{98} \left(2 - \frac{8}{98} \right) + \sqrt{\frac{8}{98} \left(2 - \frac{8}{98} \right)} \right\} = 18.538 \text{ kN/m}^2$$

$$P_{e10} = 0.726 p_{ey} \cdot y = 0.726 \times 18.538 \times 8 = 107.87 \text{ kN/m}$$

$$M_{e10} = 0.299 p_{ey} y^2 = 0.299 \times 18.538 (8)^2 = 354.74 \text{ kN-m/m}$$

and

At the base ($y = 98 \text{ m}$)

$$p_{e100} = 33.564 \left\{ \frac{98}{98} \left(2 - \frac{98}{98} \right) + \sqrt{\frac{98}{98} \left(2 - \frac{98}{98} \right)} \right\} = 67.128 \text{ kN/m}^2$$

$$P_{e100} = 0.726 p_{ey} \cdot y = 0.726 \times 67.128 \times 98 = 4776 \text{ kN/m}$$

$$M_{e100} = 0.299 p_{ey} y^2 = 0.299 \times 67.128 (98)^2 = 182764 \text{ kN-m/m}$$

and

Example 8.3. Effect of vertical earthquake acceleration

For the profile of the gravity dam of Fig. 8.11 (Example 8.1), determine the effect of vertical earthquake acceleration on the weight of the dam at 10 m below the top

and at the base. Use both the seismic coefficient method as well as response spectrum method.

Solution

(a) Seismic coefficient method

For zone IV, $\alpha_s = 0.05$

$$\alpha_h = 2 \alpha_s = 2 \times 0.05 = 0.1$$

Hence vertical acceleration at top is

$$\alpha_v = 0.75 \alpha_h = 0.75 \times 0.1 = 0.075$$

It varies linearly from a value of 0.75 at the top to zero at the base, as shown in Fig. 8.14.

At 10 m below top

Consider an elementary strip of thickness dy' at y' below the top.

Weight of strip $W_y' = (1 \times 7 dy') 24 = 168 dy'$

$$\alpha_{vy} = 0.075 (1 - y'/100) = 0.075 (1 - 0.01y')$$

∴ Increase or decrease in weight

$$\begin{aligned} &= \int_0^{10} \alpha_{vy} W_y' dy' = \int_0^{10} 0.075 (1 - 0.01y') 168 dy' \\ &= 12.6 \int_0^{10} (1 - 0.01y') dy' = 12.6 \left[y' - \frac{0.01y'^2}{2} \right]_0^{10} = 119.7 \text{ kN} \end{aligned}$$

At the base (100 m below top): Consider an elementary strip of thickness dy at y below the top.

$$\begin{aligned} W_y &= [7 + 0.7(y - 10)] dy \times 24 = 16.8y \cdot dy \\ \alpha_{vy} &= 0.075 (1 - 0.01y) \end{aligned}$$

$$\begin{aligned} \therefore \text{Increase or decrease in weight} &= 119.7 + \int_{10}^{100} (16.8y dy) 0.075 (1 - 0.01y) \\ &= 119.7 + 1.26 \left[\frac{y^2}{2} - \frac{0.01y^3}{3} \right]_{10}^{100} = 2160.9 \text{ kN} \end{aligned}$$

(b) Response spectrum method

For zone IV, $F_0 = 0.25$. Also $S_a/g = 0.19$ (from Example 8.2)

$$\alpha_h = 2 F_0 \frac{S_a}{g} = 2 \times 0.25 \times 0.19 = 0.095$$

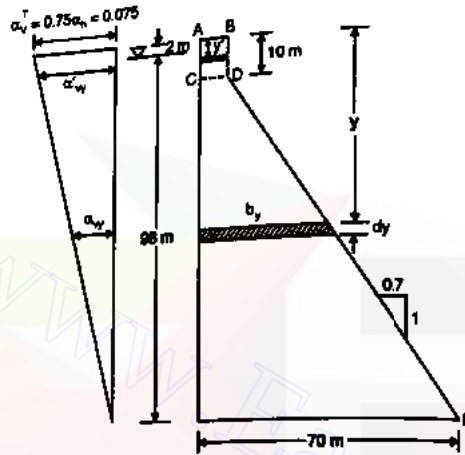


FIG. 8.12

Hence vertical acceleration at top $= 0.75 \alpha_h = 0.75 \times 0.095 = 0.07125$. It varies linearly from a value of 0.07125 at the top to zero at the base.

At 10 m below top

Weight of strip, $W_y' = (1 \times 7 dy') 24 = 168 dy'$

$$\alpha_{vy} = 0.07125 (1 - y'/100) = 0.07125 (1 - 0.01y')$$

∴ Increase or decrease in weight

$$\begin{aligned} &= \int_0^{10} \alpha_{vy} W_y' dy' = \int_0^{10} 0.07125 (1 - 0.01y') 168 dy' \\ &= 11.97 \int_0^{10} (1 - 0.01y') dy' = 11.97 \left[y' - \frac{0.01y'^2}{2} \right]_0^{10} = 113.715 \text{ kN} \end{aligned}$$

At the base (i.e. 100 m from top)

$$W_y = [7 + 0.7(y - 10)] dy \times 24 = 16.8y dy$$

$$\alpha_{vy} = 0.07125 (1 - 0.01y)$$

$$\begin{aligned} \therefore \text{Increase or decrease in weight} &= 113.715 + \int_{10}^{100} (16.8y dy) 0.07125 (1 - 0.01y) \\ &= 113.715 + 1.197 \left[\frac{y^2}{2} - \frac{0.01y^3}{3} \right]_{10}^{100} = 2062.86 \text{ kN} \end{aligned}$$

Example 8.4. Hydrodynamic pressure on inclined u/s face

A 100 m high concrete dam shown in Fig. 8.13 is located in zone IV and has to resist pressure from 98 m depth of water. Find the hydrodynamic pressure at the following locations: (a) 10 m from the top (b) 40 m from the top, and (c) at the base. Take the value of $\alpha_h = 0.1$ determined on the basis of response spectrum method.

Solution

(a) Hydrodynamic Pressure intensities

Hydrodynamic pressure at any depth y below the reservoir surface is given by

$$p_{xy} = C_y \alpha_h w h$$

where $\alpha_h = 0.1$; $w = 9.81 \text{ kN/m}^3$; $h = 98 \text{ m}$.

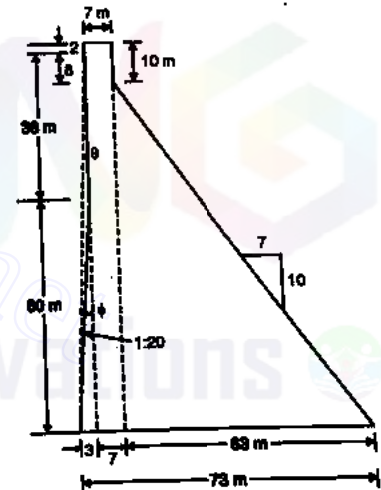


FIG. 8.13

Since the height of sloping face ($= 60$ m) is more than $h/2$ ($= 98/2$), the modified slope for the design will be obtained by joining the point of intersection of the u/s face of the dam and the reservoir surface with the point of intersection of u/s face of the dam and the foundation, as shown in Fig. 8.15. Thus $\theta = \tan^{-1} 3/98 = 1.75^\circ$. Hence from Fig. 8.9, we get $C \approx 0.72$.

Alternatively, from Eq. 8.18, $C_m \approx 0.735(1 - 1.75/90) = 0.72$

$$C_y = \frac{0.72}{2} \left[\frac{y}{98} \left(2 - \frac{y}{98} \right) + \sqrt{\frac{y}{98} \left(2 - \frac{y}{98} \right)} \right]$$

$$\text{and } p_{ey} = \frac{0.72}{2} \left[\frac{y}{98} \left(2 - \frac{y}{98} \right) + \sqrt{\frac{y}{98} \left(2 - \frac{y}{98} \right)} \right] \times 0.1 \times 9.81 \times 98$$

$$\text{or } p_{ey} = 34.61 \left[\frac{y}{98} \left(2 - \frac{y}{98} \right) + \sqrt{\frac{y}{98} \left(2 - \frac{y}{98} \right)} \right]$$

At depth of 8 m below reservoir level or 10 m below top of dam,

$$p_{e10} = 19.116 \text{ kN/m}^2$$

At depth of 38 m below reservoir level or 40 m below top of dam,

$$p_{e40} = 49.002 \text{ kN/m}^2$$

At a depth of 98 m below the reservoir level or 100 m below the top of dam

$$p_{e100} = 69.22 \text{ kN/m}^2$$

(b) Horizontal shear

Horizontal shear at any depth of y from the reservoir level is given by

$$P_{ey} = 0.726 p_{ey} \cdot y \quad \dots(8.19)$$

$$\text{Hence } P_{e10} = 0.726 \times 19.116 \times 8 = 111.026 \text{ kN/m}$$

$$P_{e40} = 0.726 \times 49.002 \times 38 = 1351.867 \text{ kN/m}$$

$$P_{e100} = 0.726 \times 69.22 \times 98 = 4924.864 \text{ kN/m}$$

(c) Moment

$$M_{ey} = 0.299 p_{ey} \cdot y^2 \quad \dots(8.20)$$

$$M_{e10} = 0.299 \times 19.116 (8)^2 = 365.8 \text{ kN-m/m}$$

$$M_{e40} = 0.299 \times 49.002 (38)^2 = 21156.9 \text{ kN-m/m}$$

$$M_{e100} = 0.299 \times 69.22 (98)^2 = 198771.9 \text{ kN-m/m}$$

(d) Vertical component of reservoir water on horizontal section

Since the hydrodynamic pressure acts normal to the face of the dam, there will be a vertical component of this force, if the face of the dam against which it is acting is sloping. The magnitude of this load (W_h) on any horizontal section is given by

$$W_h = (P_{e2} - P_{e1}) \tan \phi$$

where ϕ = the actual slope of the u/s face. Here $\tan \phi = 3/60 = 1/20 = 0.05$

P_{e2} = total shear due to horizontal component of hydrodynamic force at the elevation of the section being considered.

P_{e1} = total shear due to horizontal component of hydrodynamic force at the elevation at which the slope of the dam face commences,

Hence at the base of the dam, $P_{e2} = P_{e100} = 4924.864$ and $P_{e1} = P_{e40} = 1351.867$

$$W_{100} = (4924.864 - 1351.867) 0.05 = 178.65 \text{ kN/m}$$

8.12 MODES OF FAILURE : STABILITY REQUIREMENTS

Following are the modes of failure of a gravity dam :

1. Overturning
2. Sliding
3. Compression or Crushing
4. Tension.

1. **Overturning.** The overturning of the dam section takes place when the resultant force at any section cuts the base of the dam downstream of the toe. In that case the resultant moment at the toe becomes clockwise (or -ve). On the other hand, if the resultant cuts the base within the body of the dam, there will be no overturning.

For stability requirements, the dam must be safe against overturning. The factor of safety against overturning is defined as the ratio of the righting moment (+ve) to the overturning moments, i.e.

$$F.S. = \frac{\Sigma \text{Righting moments}}{\Sigma \text{Overturning moments}} = \frac{\Sigma M_R}{\Sigma M_O} \quad \dots(8.25)$$

Since the magnitude and direction of each and every force considered in the previous articles are known, their moments about the toe can very easily be found and F.S. computed. The factor of safety against overturning should not be less than 1.5.

2. **Sliding.** A dam will fail in sliding at its base, or at any other level, if the horizontal forces causing sliding are more than the resistance available to it at that level. The resistance against sliding may be due to friction alone, or due to friction and shear strength of the joint. Shear strength develops at the base if benched foundations are provided and at other joints if the joints are carefully laid so that a good bond develops. Shear strength also comes into play because of the interlocking of stone in masonry dams.

If the shear strength is not taken into account, the factor of safety is known as *factor of safety against sliding*. The factor of safety against sliding is defined as the ratio of *actual coefficient of static friction* (μ) on the horizontal joint to the *sliding friction*. The sliding factor is the minimum coefficient of friction required to prevent sliding. If ΣH = sum of the horizontal forces causing the sliding and ΣV is the net vertical forces, the *sliding factor* ($\tan \theta$) is given by

$$S.F. = \tan \theta = \frac{\Sigma H}{\Sigma V} \quad \dots(8.26)$$

and factor of safety against sliding (F.S.S) is

$$F.S.S. = \frac{\mu}{\tan \theta} = \frac{\mu \Sigma V}{\Sigma H} \quad \dots(8.27)$$

The coefficient of friction, μ varies from 0.65 to 0.75. The factor of safety against sliding should be greater than the value given in Table 8.4 (IS 6512-1972)

It is considered that a low gravity dam should be safe against sliding considering friction alone. However, in large dams, shear strength of the joint should also be considered for an economical design. The factor of safety in that case is commonly known as the *shear friction factor* S.F.F. and is defined by the equation

$$\text{S.F.F.} = \frac{\mu \Sigma (V) + A \cdot c}{\Sigma H} = \frac{\mu \Sigma (V) + b \cdot c}{\Sigma H} \quad \dots(8.28)$$

where c = average cohesion or shear strength of the joint the value of which varies from 1300 kN/m² to 4500 kN/m², for good rock to 650 to 1300 kN/m² for concrete.

b = width of the joint or section

A = Area of joint = $b \times 1$ for unit length of the dam.

Table 8.4 gives permissible values of factor of the safety against sliding and the S.F.F.

TABLE 8.4. FACTOR OF SAFETY AGAINST SLIDING AND SHEAR FRICTION FACTOR (IS : 6512 - 1972)

S.No.	Loading condition	F.S. against sliding (F.S.S.)	S.F.F.
1.	A, B, C	2.0	4.0
2.	D, E	1.5	3.0
3.	F, G	1.2	1.5

Recommendations of IS 6512-1984 : The above criterion (i.e. Eq. 8.28 and Table 8.4) given in IS 6512-1972 was completely changed in IS 6512-1984. According to the revised version, the factor of safety against sliding is calculated on the basis of partial factor of safety in respect of friction (F_ϕ) and partial factor of safety in respect of cohesion or shear (F_c) by using the following equation :

$$F = \frac{\frac{\mu \Sigma (V)}{F_\phi} + \frac{cA}{F_c}}{\Sigma H} = \frac{\mu \Sigma V}{F_\phi \Sigma H} + \frac{cb}{F_c \Sigma H} \quad \dots(8.29)$$

where F = factor of safety against sliding

and A = Area of the joint = $b \times 1$ for unit length of the dam.

The values of F_ϕ and F_c are given in Table 8.5.

TABLE 8.5. VALUES OF PARTIAL SAFETY FACTORS F_ϕ AND F_c (IS 6512-1984)

S.N.	Load combination	F_ϕ	F_c	
			For dams and the contact plane with foundation	For foundation
				Thoroughly investigated others
(i)	A, B, C	1.5	3.6	4.0 4.5
(ii)	D, E	1.2	2.4	2.7 3.0
(iii)	F, G	1.0	1.2	1.35 1.5

The factor of safety against sliding, computed from Eq. 8.29, should not be less than 1.

3. Compression or Crushing. In order to calculate the normal stress distribution at the base, or at any section, let H be the total horizontal force, V be the total vertical force and R be the resultant forces cutting the base at an eccentricity e from the centre of the base of width b (Fig. 8.14).

The normal stress at any point on the base will be the sum of the direct stress and the bending stress.

$$\text{Thus, direct stress} = \frac{V}{b \times 1}$$

$$\text{Bending stress} = \pm \frac{My}{I} = \pm \frac{V \cdot e}{\frac{1}{6} b^3} = \pm \frac{6V \cdot e}{b^3}$$

Hence the total normal stress p_n is given by

$$p_n = \frac{V}{b} \left(1 \pm \frac{6e}{b} \right) \quad \dots(8.30)$$

The positive sign will be used for calculating normal stress at the toe, since the bending stress will be compressive there, and negative sign will be used for calculating normal stress at the heel.

Thus, the normal stress at the toe is

$$(p_n)_{\text{toe}} = \frac{V}{b} \left(1 + \frac{6e}{b} \right) \quad \dots[8.30(a)]$$

The normal stress at the heel is

$$(p_n)_{\text{heel}} = \frac{V}{b} \left(1 - \frac{6e}{b} \right) \quad \dots[8.30(b)]$$

Fig. 8.14(b) shows the normal stress distribution for a general case when the pressure at both toe and heel are compressive. Evidently, the maximum compressive stress occurs at the toe and for safety, this should not be greater than the allowable compressive stress f for the foundation material. Hence, from strength point of view

$$\frac{V}{b} \left(1 + \frac{6e}{b} \right) \leq f \quad \dots(8.31)$$

When the eccentricity e is equal to $\frac{b}{6}$, we get

$$(p_n)_{\text{toe}} = \frac{V}{b} \left(1 + \frac{6}{b} \times \frac{b}{6} \right) = \frac{2V}{b} \quad \dots(8.32)$$

The corresponding stress at the heel in that circumstance will evidently be zero.

Fig. 8.14(c) shows the pressure distribution for this case.

4. Tension. From equation 8.30(b) the normal stress at the heel is

$$(p_n)_{\text{heel}} = \frac{V}{b} \left(1 - \frac{6e}{b} \right)$$

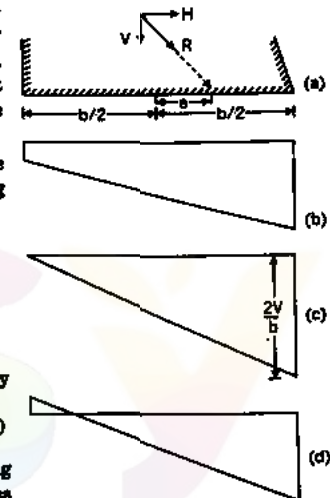


FIG. 8.14. NORMAL STRESS DISTRIBUTION AT BASE.

It is evident that if $e > \frac{b}{6}$, the normal stress at the heel will be -ve or tensile. No tension should be permitted at any point of the dam under any circumstance for moderately high dams. For no tension to develop, the eccentricity should be less than $b/6$. In other words, the resultant should always lie within the middle third.

However, in case of extra high dams, 230 to 260 m, small tension within the permissible limit is generally permitted for comparatively small periods of loading such as heavy flood or earthquake.

Effect of Tension Cracks. Since concrete cannot resist the tension, a crack develops at the heel, which modifies the uplift pressure diagram, as illustrated in Fig. 8.15. Due to tension crack, the uplift pressure increases in magnitude and net downward vertical force or the stabilising force reduces. The resultant force thereby gets further shifted towards the toe and this leads to further lengthening of the crack. The base width thus goes on reducing and the compressive stresses on toe goes on increasing, till the toe fails in compression.

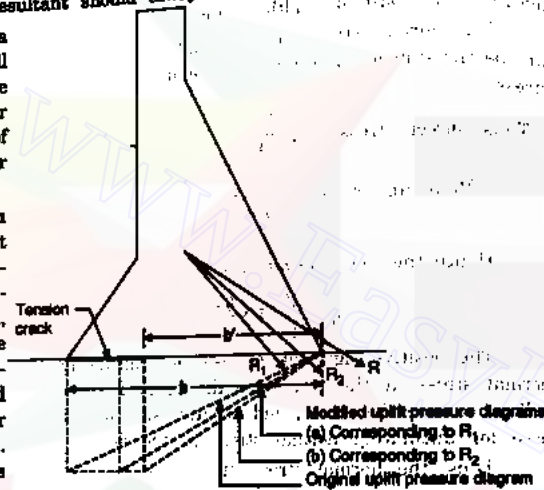


FIG. 8.15. EFFECT OF TENSION CRACKS.

8.13. PRINCIPAL AND SHEAR STRESSES

Principal Stress. Consider an elementary triangular section at either the heel or the toe of the dam section such that stress intensities may be assumed to be uniform on its faces. The face of the dam will be a principal plane as water pressure acts on it in the perpendicular direction, with no accompanying shear stress. Since the principal planes are mutually at right angles, the plane AB, considered at right angles to the face AC, will also have only a normal stress on it, and will be the other principal

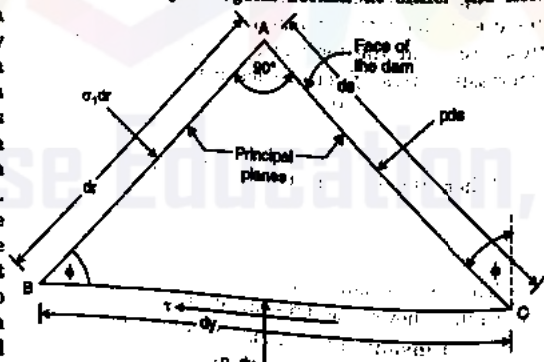


FIG. 8.16. PRINCIPAL STRESSES.

plane. The forces acting on the elementary section are shown in Fig. 8.16.

Let ds , dr and dy be the lengths of AC, AB and BC

p = intensity of water pressure acting normal to the face of the dam

σ_1 = principal stress on plane AB

τ = shear stress on plane BC

p_n = normal stress on plane BC, known from Eq. 8.30

Considering unit length of the dam, the normal forces on the planes AB, BC and CA are respectively $\sigma_1 dr$, $p_n dy$ and $p ds$.

Resolving all the forces in the vertical direction, we get

$$p_n dy = p ds \sin \phi + \sigma_1 dr \cos \phi$$

$$\text{But } dr = dy \cos \phi \text{ and } ds = dy \sin \phi$$

$$p_n dy = p dy \sin^2 \phi + \sigma_1 dy \cos^2 \phi$$

$$\text{or } p_n = p \sin^2 \phi + \sigma_1 \cos^2 \phi$$

$$\text{Hence } \sigma_1 = \frac{(p_n - p \sin^2 \phi)}{\cos^2 \phi}$$

$$\sigma_1 = p_n \sec^2 \phi - p \tan^2 \phi \quad \dots(8.33)$$

Equation 8.33 is known as the principal stress relationship, and is applicable to both upstream and downstream faces. However, for the downstream side, the worst condition will be when there is no tailwater, and hence p will be zero. In that circumstance, the principal stress σ_1 is given by

$$\sigma_1 = p_n \sec^2 \phi \quad \dots(8.34)$$

If p_t is the intensity of hydrodynamic pressure of tailwater due to an earthquake the principal stress at the downstream is given by

$$\sigma_1 = p_n \sec^2 \phi - (p - p_t) \tan^2 \phi \quad \dots(8.35)$$

Similarly, considering the hydrodynamic pressure (p_t) to the upstream side, principal stress is given by

$$\sigma_1 = p_n \sec^2 \phi - (p + p_t) \tan^2 \phi \quad \dots(8.36)$$

It should be noted, however, that for the upstream face σ_1 will always be less than p . Hence σ_1 is the minor principal stress and p is the major principal stress for the upstream face. Also, if u/s face is vertical, $\phi = 0$ and hence $\sigma_1 = p_n$.

Shear Stress : Resolving all the forces in the horizontal direction, we get

$$\tau dy = \sigma_1 dr \sin \phi - p ds \cos \phi$$

$$\tau = \sigma_1 \sin \phi \frac{dr}{dy} - p \frac{ds}{dy} \cos \phi$$

$$\tau = (\sigma_1 - p) \cos \phi \sin \phi$$

$$\text{Substituting the value of } \sigma_1 \text{ from Eq. 8.33, we get}$$

$$\tau = (p_n \sec^2 \phi - p \sec^2 \phi) \cos \phi \sin \phi$$

or

$$\tau = (p_n - p) \tan \phi \quad \dots(8.37)$$

The above equation is applicable for downstream side only. For the upstream side, the magnitude of τ will be the same as that given by Eq. 8.37, but its direction will be reversed. Hence, for upstream

$$\tau = -(p_n - p) \tan \phi \quad \dots(8.38)$$

If tailwater is neglected, the shear stress at the downstream side is given by

$$\tau = p_n \tan \phi \quad \dots(8.37(a))$$

Considering the hydrodynamic pressure due to earthquake, the shear stress at downstream is given by

$$\tau = [p_n - (p - p_e')] \tan \phi \quad \dots(8.39.)$$

Similarly, the shear stress for the upstream side is given by

$$\tau = -[p_n - (p + p_e')] \tan \phi \quad \dots(8.40)$$

However, if u/s face is vertical, $\phi = 0$ and $\tau = 0$ from Eq. 8.40

8.14. QUALITY AND STRENGTH OF CONCRETE / MASONRY (IS : 6512-1984)

According to the Indian Standard, the strength of concrete / masonry shall exceed the stresses anticipated in the structure by a safe margin. The maximum compressive stresses occur at the heel or toe and on planes normal to the faces of the dam. The strength of concrete and masonry varies with age, the kind of cement and other ingredients and their proportions in the work can be determined only by experiment.

(a) **Compressive Strength :** The compressive strength of concrete and masonry shall conform to the following requirements :

(i) **Concrete.** Concrete strength is determined by compressing to failure 15 cm × 30 cm cylinders. The strength of concrete should satisfy early load and construction requirements and at the age of the one year it should be four times the maximum computed stress in the dam or 14000 kN/m² (14 N/mm²), whichever is more. The allowable working stress in any part of the structure shall not also exceed 7000 kN/m² (7 N/mm²).

(ii) **Masonry.** The compressive strength of masonry is determined by compressing to failure 75 cm cubes of masonry fabricated and cured at temperatures approximating to those expected in the structures or 45 × 90 cm cylinders cored out of the structures or blocks made (for the purpose). This strength should satisfy early load and construction requirements and at one year it should be five times the maximum computed stress on the dam or 17500 kN/m² (17.5 N/mm²), whichever is more.

[Note. For the purpose of quality control, correlation between the strength of mortar and that of masonry may be established in suitable small size specimens].
Recommendations of IS 6512-1984 against over stressing : Provision of fillets

If the compressive stress at toe and heel are excessive, they can be brought to the permissible limits by provision of fillets with slopes one horizontal to two vertical (1:2) at the heel and two horizontal to one vertical (i.e. 2:1) at the toe. The heights of these fillets above heel and toe are determined from the following expressions.

$$(i) \text{ For heel : } h_{fu} = 6 H_0 - 0.658 H_0^2 - 1.07 \quad \dots(8.41 a)$$

$$(ii) \text{ For toe : } h_{fd} = 6.5 H_0 - 1.1 H_0^2 - 0.9 \quad \dots(8.41 b)$$

where h_{fu} = height of fillet at heel (i.e. u/s side)

h_{fd} = height of fillet at toe (i.e. d/s side)

H_0 = height of dam in 100 metres

For example, if height of dam = 150 m, $H_0 = 1.5$

$$\therefore h_{fu} = 6 \times 1.5 - 0.658 (1.5)^2 - 1.07 \triangleq 6.45 \text{ m}$$

$$\text{and } h_{fd} = 6.5 \times 1.5 - 1.1 (1.5)^2 - 0.9 \triangleq 6.4 \text{ m}$$

(b) Tensile Strength

(i) Although concrete and masonry are known to possess some tensile strength, quantitative evaluations are uncertain and difficult. Moreover, construction joints can be planes of weakness against tensile strength. Tensile failures in the form of cracking may, in some cases, have significant adverse effects on the structural behaviour of the dam.

(ii) Tensile stresses are not permitted at the u/s face of the dam for load combination B (normal operation condition). However, normal tensile stresses may be permitted in case of other load combinations but their values should not exceed the permissible values given in Table 8.6.

TABLE 8.6. PERMISSIBLE TENSILE STRESS IN CONCRETE AND MASONRY

Load combination	Permissible tensile stress	
	Concrete	Masonry
C	0.01 f_c	0.005 f_c
E	0.02 f_c	0.01 f_c
F	0.02 f_c	0.01 f_c
G	0.04 f_c	0.02 f_c

(iii) Small values of tension on the d/s face may be permitted since it is very improbable that a fully constructed dam is kept empty and downstream cracks which are not extensive and for limited depths from the surface may not be detrimental to the safety of the structure.

Structural Competency of the Foundation

The true conditions at the dam site and reservoir area shall be determined and the design of the dam made to suit the conditions imposed by nature. The ground water level may cause softening of the rocks or the dam may slide on a slippery bedding plane or the impounded water may pass through fissures and caverns around and under the dam. The structural competency of the foundation is not usually amenable to precise and complete evaluation ; data from explorations and tests should be supplemented by engineering and geological judgement. When the quality of the foundation materials approaches or falls below that of the concrete/masonry of the structure on it, the foundation competency becomes a controlling factor and should be appraised in full detail. It is to be noted that field and laboratory tests respectively evaluate relatively small areas and small samples only and the overall characteristics of the foundation including geological structures or other inhomogeneities are interpreted from these limited data.

8.15. STABILITY ANALYSIS

The stability analysis of a given gravity dam section may be carried out by the following methods :

1. Gravity method or two-dimensional method :
 - (a) Graphical method.
 - (b) Analytical method.
2. Trial load twist method.
3. Slab analogy method.
4. Lattice analogy method.
5. Experimental methods :
 - (a) Direct method.
 - (b) Indirect method.

1. Gravity method. This is an approximate method in which the dam is considered to be composed of parallel sided vertical cantilevers, each of which is free to act without supporting or interfering with the adjoining cantilevers. The loads or forces are resisted entirely by the weight of the individual cantilevers of unit length. Other methods of design and analysis are complicated and time consuming, and hence preliminary calculations are always done with the help of this method.

(a) Graphical method.

In the graphical method, the dam section is divided into horizontal section 1-1, 2-2, 3-3 etc. at some suitable interval, or at the places where slope changes (Fig. 8.17). For each section, the sum of all horizontal forces (ΣH) and the sum of all vertical forces (ΣV), acting above that section are calculated and their line of action are graphically located. The resultant force R is then found, and its line of action is located graphically. This is done for each section, and finally, a line is drawn joining the point at which the resultants cut the various sections. The line of resultant force so obtained should evidently lie within the middle third, for no tension to develop. This procedure is adopted both for the reservoir full condition, as well as reservoir empty condition. Both the lines of resultant pressure so obtained should lie in the middle third portion, as shown in Fig. 8.17.

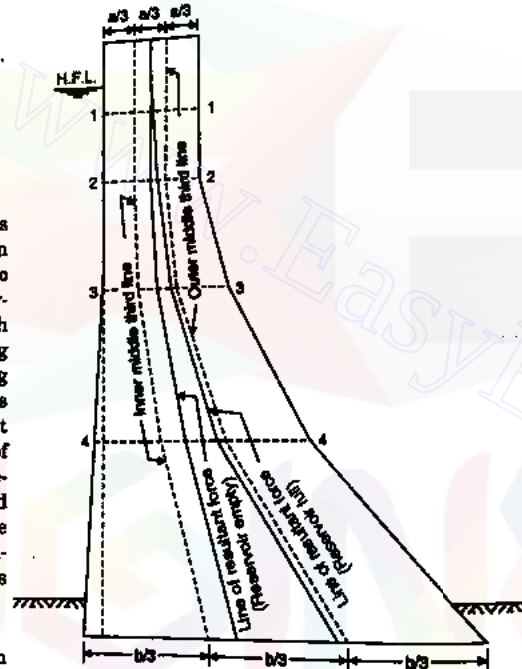


FIG. 8.17. GRAPHICAL METHOD.

(b) Analytical method. The stability analysis by analytical method is done in the following steps :

1. Consider unit length of the dam. Calculate all the vertical loads acting. They include the weight of the dam, weight of water acting on the inclined faces, uplift pressure and inertia forces due to vertical acceleration. Find their algebraic sum (ΣV).

2. Find out the sum of horizontal forces ΣH , and the horizontal pressure due to hydrodynamic pressure.

3. Find out the sum of the overturning moments (ΣM_o) and the sum of righting moments (ΣM_R) at the toe. Also find the algebraic sum of all the moments as under :

$$\Sigma M = \Sigma M_R - \Sigma M_o$$

4. Find out the location (i.e. distance $\bar{x}$) of the resultant force R from toe, by relation

$$\bar{x} = \frac{\Sigma M}{\Sigma V}$$

5. Find out the eccentricity e of the resultant R , from the centre by the relation,

$$e = \frac{b}{2} - \bar{x}, \text{ where } b = \text{base width of the section.}$$

6. Find the normal stress at the toe of dam :

$$P_n = \frac{\Sigma V}{b} \left(1 + \frac{6e}{b} \right)$$

Also, find normal stress at the heel by the expression :

$$P_n = \frac{\Sigma V}{b} \left(1 - \frac{6e}{b} \right)$$

Sometimes, the normal compressive stresses are found by ignoring the uplift.

7. Find out the principal and shear stresses at the toe and the heel by Eqs. 8.35 to 8.40.

8. Find out the factor of safety against overturning by the expression :

$$F.S. = \frac{\Sigma M_R}{\Sigma M_o}$$

9. Find out the factor of safety against sliding by the following expressions :

$$(i) \text{ Sliding factor : } S.F. = \frac{\mu \Sigma V}{\Sigma H}$$

$$(ii) \text{ Shear friction factor : } S.F.F. = \frac{\mu \Sigma V + bc}{\Sigma H}$$

$$(iii) \text{ Factor of safety against sliding : } F = \frac{\mu \Sigma V + \frac{cb}{F_c}}{\Sigma H}$$

where ΣV is the net vertical load, inclusive of the uplift.

The method is illustrated in a tabular form in Examples 8.8 and 8.9.

2. **Trial Load Twist Method.** Gravity method of analysis described above is based on the assumption that the dam behaves as vertical cantilevers independent of each other. The assumption is very nearly true if the dam is constructed in a wide U-shaped valley, and the transverse joints are not keyed or grouted. If the transverse joints are keyed or grouted, the dam will tend to behave as a monolithic structure and the two dimensional analysis or gravity method will not be true. Trial load twist method is based on the assumption that the horizontal elements of the dam will behave as a beam and will transfer the water pressure acting upon it to the abutments by beam action. Further, the height of the dam reduces as it proceeds from the centre towards the side as the valley on both sides rises steeply. The successive cantilever elements, therefore, go on reducing in the height, and their deflection also goes on reducing. Each cantilever element is thus dragged on its one face to greater deflections by deeper adjacent cantilevers on the valley side and is held on by the shorter cantilever on hill side. This develops a twist effect on each cantilever. In the trial load twist method therefore the entire dam is divided into a number of vertical cantilevers and horizontal beam elements. These elements are then supposed to form three structural systems namely :

- (i) *Cantilever structural system* to resist the external load by gravity action through bending and shear along horizontal planes.
- (ii) *Beam structural system* to resist bending.
- (iii) *Twist structural system* to resist twisting moments caused by shearing of horizontal elements and angular rotation of vertical elements.

The portion of the pressures taken by each element is obtained by dividing out the load distribution between them so that the deflection of a common point considered on any element, is the same.

3. Slab Analogy Method. According to this method, the dam is considered as a slab and the analysis carried out is similar to the calculation of stresses in bridge slabs. It is a time consuming and laborious method.

4. **Lattice Analogy Method.** According to this method, the dam is considered to be composed of diagonally braced square frames. The analysis, though simple than the slab analysis method, is quite cumbersome.

5. Experimental Methods. (a) Direct method (Three dimensional model analysis). In this method, geometrically similar three dimensional models, which are exact replica of the prototype, are made of elastic materials. The models are located in similar manner as the prototype. The model is then acted upon by various pressures proportional to those of the prototype. In order to simulate the high water pressure acting on the prototype, a liquid of high specific gravity such as mercury may be used or the effect may be obtained by the jetting action of water. The deflections occurring at various points of the model are thereafter measured. Then from the correlation developed between the model and prototype for the stress and deflection, the stresses for the prototype can be obtained. The model analysis has been found to be quite useful for the analysis of high dams, particularly for the Cupola dam of Italy.

(b) *Indirect method (Photoelastic model analysis)*. Photoelastic method and methods of magnetic and electrical analogy are usually considered under the indirect methods of determining the stresses in the model and hence in the prototype dam. In case of photoelastic method, the stresses in the photoelastic model are determined by an optical instrument known as photoelastic polariscope, wherein a polarised light passes through the model while it is under load. Studies employing photoelastic models are limited. Usually the study is carried out to supplement an analysis by determining regions of stress concentration. The material used for the photoelastic models must be elastic, isotropic and free from initial or residual stresses.

8.16. ELEMENTARY PROFILE OF A GRAVITY DAM

In the absence of any force other than the forces due to water, an elementary profile will be triangular in section, having zero width at the water level at the top, where water pressure is zero, and a maximum base width b , where the maximum water pressure acts. Thus, the section of the elementary profile is of the same shape as the hydrostatic pressure distribution diagram. For reservoir empty condition, a right angled triangular profile as shown in Fig. 8.18, will provide the maximum possible stabilising force against overturning, without causing tension in the base. This is so because the weight of the dam acts at distance $b/3$ from the upstream face and is closer to it. If any triangular profile, other than the right angled one, is provided, its weight will act still closer to the upstream face to provide a higher stabilising force, but tension will be developed at the toe when the dam is empty.

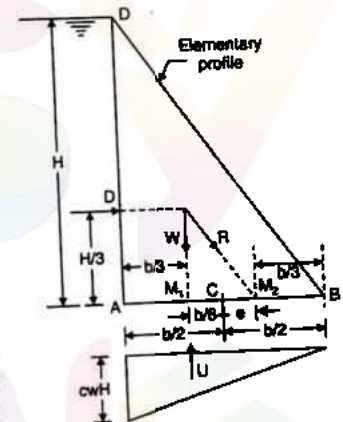


FIG. 8.18. ELEMENTARY PROFILE.

We shall consider the following forces acting on the elementary profile of a gravity dam :

- (1) Weight of the dam (W)

$$W = \frac{1}{2} bH \cdot \rho w$$

where ρ = specific gravity of dam material
 w = unit weight of water.

- (2) Water pressure (P)

$$P = \frac{1}{2} w H^2, \text{ acting at } \frac{1}{3} H \text{ from the base.}$$

- (3) *Uplift pressure* (U)

$$U = \frac{1}{2} c \omega b H$$

where c = uplift pressure intensity coefficient.

Base width of elementary profile. The base width of the elementary profile is to be found under two criteria :

(1) Stress criterion. (2) Stability or sliding criterion

(1) **Stress Criterion.** We have already seen that when reservoir is empty, for no tension to develop, the resultant should act at the inner 'third point' (M_1). For the 'reservoir full' condition, for no tension to develop, the resultant R must pass through the 'outer third' point (M_2) shown in Fig. 8.18.

Taking the moment of all forces about M_1 and equating it to zero (since the moment of R about M_1 is zero), we get,

$$\frac{1}{2} wH^2 \cdot \frac{H}{3} + \frac{1}{2} cw b H \cdot \frac{b}{3} - \frac{1}{2} b Hpw \cdot \frac{b}{3} = 0.$$

Multiplying all the terms by $\frac{6}{wH}$, we get

$$H^2 + cb^2 - b^2 \rho = 0$$

$$\text{or } b^2(\rho - c) = H^2$$

$$b = \frac{H}{\sqrt{\rho - c}} \quad \dots(8.42)$$

The above equation can also be alternatively derived by considering the force triangle in Fig. 8.20. Thus

$$\frac{W-U}{P} = \frac{H/3}{b/3} = \frac{H}{b} \quad \text{or} \quad \frac{\frac{1}{2}bHpw - \frac{1}{2}wcbH}{\frac{1}{2}wH^2} = \frac{H}{b}$$

$$\text{or } b^2(\rho - c) = H^2 \quad \text{or} \quad b = \frac{H}{\sqrt{\rho - c}} \quad \dots(8.42(a))$$

If uplift is not considered, $c = 0$

$$b = \frac{H}{\sqrt{\rho}}$$

(2) **Stability or Sliding criterion.** For no sliding to occur, horizontal force causing sliding should be balanced by the frictional forces opposing the same.

Hence $P = \mu(W - U)$

$$\frac{1}{2} wH^2 = \mu \left(\frac{1}{2} bH \rho w - \frac{1}{2} cbwH \right)$$

$$\text{From which } b = \frac{H}{\mu(\rho - c)} \quad \dots(8.43)$$

If uplift is neglected, $b = \frac{H}{\mu \cdot \rho}$

The width provided for the elementary profile should be greater of the width given by Eqs. 8.42 and 8.43.

Stresses developed in the elementary profile

The normal stress, in general, is given by Eq. 8.30, i.e.

$$p_n = \frac{V}{b} \left(1 + \frac{6e}{b} \right)$$

where $V = (W - U)$ in this case and $e = \frac{b}{6}$.

Hence, for full reservoir case, the normal stresses at the toe is

$$p_n = \frac{(W - U)}{b} (1 + 1) = \frac{2(W - U)}{b}$$

or

$$p_n = \frac{2}{b} \left(\frac{1}{2} b Hpw - \frac{1}{2} cbwH \right)$$

or

$$p_n = wH(\rho - c) \quad \dots(8.44)$$

The corresponding stress at the heel will be

$$p_n = \frac{(W - U)}{b} (1 - 1) = 0.$$

Principal stress at the toe

$$\sigma_1 = p_n \sec^2 \phi = wH(\rho - c) \left[\left(\frac{b}{H} \right)^2 + 1 \right]$$

But

$$\left(\frac{b}{H} \right)^2 = \frac{1}{\rho - c}, \text{ from Eq. 8.42}$$

Hence

$$\sigma_1 = wH(\rho - c) \left[\frac{1}{\rho - c} + 1 \right]$$

or

$$\sigma_1 = wH(\rho - c + 1) \quad \dots(8.45)$$

Shear stress at the toe

$$\tau = p_n \tan \phi$$

or

$$\tau = wH(\rho - c) \times \frac{b}{H}$$

or

$$\tau = wH(\rho - c) \times \frac{1}{\sqrt{\rho - c}}$$

or

$$\tau = wH \sqrt{\rho - c} \quad \dots(8.46)$$

Since the normal stress at the heel is zero, the principal stress and shear stress will be zero there.

Reservoir empty condition. When the reservoir is empty, the only force acting on the elementary profile will be its weight, acting through the first third point M_1 . Hence, the maximum compressive stress at the heel will be $= \frac{W}{b} (1 + 1) = \frac{2W}{b}$, and

the corresponding normal stress at the toe will be $\frac{W}{b} (1 - 1) = \text{zero}$.

Maurice Levy's criterion of elementary profile

Maurice Levy laid down the criterion for fixing the base width of elementary profile of gravity dam, according to which the vertical stress (or the normal stress)

at the heel of the dam under full reservoir condition but without uplift, should be compressive and should be equal to the hydrostatic pressure.

The, according to the criterion, $\frac{\Sigma V}{b} \left(1 - \frac{6e}{b} \right) \geq wH$... (a)

Substituting $\Sigma V = W = \frac{1}{2} b H \rho w$, we get

$$\frac{1}{2} \frac{b H \rho w}{b} \left(1 - \frac{6e}{b} \right) \geq wH$$

$$\left(\frac{p}{2} - \frac{3 \rho e}{b} \right) > 1$$

or

which gives $e = \frac{b}{6} - \frac{b}{3\rho}$, in the limiting case ... (b)

The weight W of the elementary profile dam will pass through M_1 point, which is at a distance of $b/6$ from the centre of the lamina. Let the resultant of weight W and hydrostatic pressure P strike the base at any point A , with an eccentricity e from the centre of the lamina. Then the point of application of weight W will be at a distance of $\left(\frac{b}{6} + e \right)$ from point A . Taking the moments of P and W about point A , we get.

$$\frac{wH^2}{2} \times \frac{H}{3} = \frac{1}{2} b H \rho w \left(e + \frac{b}{6} \right)$$

which gives $e = \frac{H^2}{3b\rho} - \frac{b}{6}$... (c)

Equating (b) and (c), $\frac{b}{6} - \frac{b}{3\rho} = \frac{H^2}{3b\rho} - \frac{b}{6}$ or $\frac{b}{3} \left(1 - \frac{1}{\rho} \right) = \frac{H^2}{3b\rho}$

From which $b = \frac{H}{\sqrt{\rho-1}}$ which is the same as Eq. 8.42 with $c=1$

Hence the base width required according to Maurice-Levy's criterion is the same as that required for no tension basis with uplift intensity coefficient (c) equal to unity.

8.17. PRACTICAL PROFILE OF A GRAVITY DAM

The elementary profile of the gravity dam is only a theoretical profile. However, such a profile is not possible in practice because of the provision of (i) roadway at provisions, the resultant force of the weight of the dam and the water pressure falls outside the middle third of the base of the dam when the reservoir is full. To eliminate tension, some masonry is to be provided to the upstream side. Fig. 8.19 shows the dimensions of the practical profile of a gravity dam.

Freeboard.

Freeboard is the margin provided between the top of dam and H.F.L. in the reservoir to prevent the splashing of the waves over the non-overflow section. It incidentally also takes care of any unforeseen floods in the reservoir. The freeboard usually provided

is $\frac{3}{2} h_w$, where h_w can be found from Eq. 8.22. The top width equal to about 14% of h has been found to be economical

Indian Standard Recommendations for Free board

According to IS 6512-1972, the freeboard adopted shall be one and a half times the corresponding wave height above full reservoir level (FRL) or for the maximum water level condition (MWL) whichever gives the higher crest elevation for the non over flow section of the dam.

The freeboard above maximum water level shall, however, be in no case less than 0.9 m. Wind velocities of 120 km/h over water in the case of full reservoir condition and 80 km/h over water in case of maximum reservoir condition are generally assumed for calculation of wave heights.

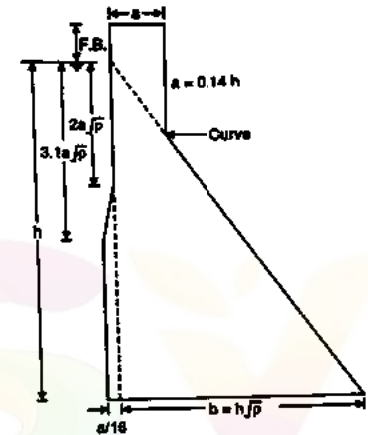


FIG. 8.19. PRACTICAL PROFILE OF A GRAVITY DAM.

However, according to the revised Code recommendations (IS 6512-1984) the free board should be equal to $\frac{4}{3} h_w$ plus wind set-up, where h_w is the wave height. The wave height and wind set-up should be calculated according to the method given by T. Saville

Thus, Free board = $\frac{4}{3} h_w$ + wind set-up

The wind set-up is the piling up of the water at one end of the reservoir and is determined by the following formula by Zuider Zee.

$$S = \frac{V^4 F \cos \beta}{52000 D} \quad \dots (8.47)$$

where S = wind set-up in metres,

V = wind velocity (km/hr)

F = fetch (km)

β = angle between wind direction and fetch

D = average depth of water, in metres, over the fetch.

The wind velocity is taken as 120 km/hr for normal reservoir level condition and 80 km/hr. for maximum water level condition. In no case shall the F.B. be less than 1 m above the maximum water level.

Modern practice is to provide a maximum free board equal to 3 to 4% of the height, though free board equal to 5% or more might prove economical.

8.18. LIMITING HEIGHT OF A GRAVITY DAM: HIGH AND LOW GRAVITY DAMS

From Eq. 8.45, the principal stress at the toe is given by

$$\sigma_1 = wH(\rho - c + 1).$$

In this expression the only variable, changing the value of σ_1 , is H . The maximum value of this principal stress should not exceed the allowable stress (f) for the material. In the limiting case,

$$f = \sigma_1 = wH(\rho - c + 1).$$

From which, the height H is given by

$$H = \frac{f}{w(\rho - c + 1)} \quad \dots(8.48a)$$

For finding the limiting height H , it is usual not to consider the uplift. Hence, putting $c = 0$, we get,

$$H = \frac{f}{w(\rho + 1)} \quad \dots(8.48)$$

If the height of the dam is more than that given by Eq. 8.48, the maximum compressive stress will exceed the permissible stress. This condition is undesirable.

Eq. 8.48 for the limiting height defines the distinction between a low and a high gravity dam. A *low gravity dam* is the one in which the height H is less than that given by Eq. 8.48, so that maximum compressive stress is not greater than the allowable stress. For a general case, taking $w = 9.81 \text{ kN/m}^3$, and $\rho = 2.40$, the limiting height, in metres is given by $H \leq 0.03 f$, where f is the allowable stress in kN/m^2 . If a common value of f is taken equal to be 2940 kN/m^2 (or 30 kg/cm^2) for concrete dam, the limiting height of low gravity dam comes out to be

$$H = \frac{2940}{9.81(2.4 + 1)} \approx 88 \text{ m}.$$

If the height of the dam to be constructed is more than that given by Eq. 8.31, the given extra slopes to the upstream and downstream sides, below the limiting height, to bring the compressive stress within the limits, as illustrated in Fig. 8.20(b).

8.19. PROFILE OF HIGH MASONRY GRAVITY DAM

G. Molesworth gave the following expressions for profile of a high gravity dam of masonry (Fig. 8.21) :

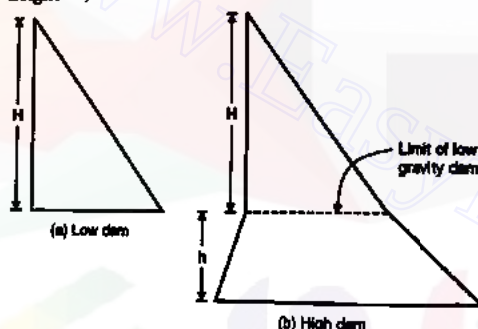


FIG. 8.20. HIGH AND LOW GRAVITY DAM

$$z = \sqrt{\frac{0.05 y^3}{f + (0.03 y)}}$$

subject to a minimum of $\frac{y}{\sqrt{\rho}}$

(F. P. S. units) $\dots(8.49)$

$$\text{and } z = \left(\frac{0.09 y}{f} \right)^4 \quad \dots(8.50)$$

where x = downstream offset from the vertical line (axis of the dam) at a depth y below the maximum reservoir level (feet)

z = upstream offset from the vertical line at a depth y (feet)

b_1 = value of x at $h/4$ (feet)

a = thickness of the dam at the full reservoir level = $0.4b_1$ (feet)

f = allowable pressure (compressive stress) in masonry, in tons per sq. ft. which may vary between 7 to 10 tons/ft².

The above formula is applicable for masonry dams only.

8.20. DESIGN OF GRAVITY DAMS

The preliminary design is always done by the two dimensional method or *gravity method* in which the dam is thought to be composed of a number of cantilevers of unit length, acting independent of adjacent cantilevers. We have already seen in § 8.16 that the elementary profile of a gravity dam is triangular in shape, having zero width at the top. However, a truly triangular section is not practical nor is it necessarily the most economical section.

Top width.

Before discussing the design procedure, let us first study the effect of top width added at the

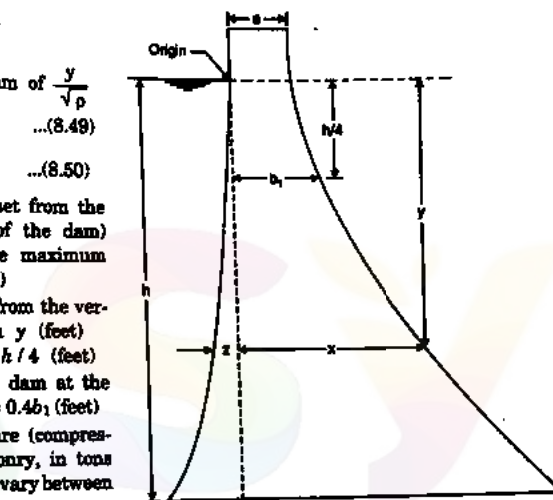


FIG. 8.21

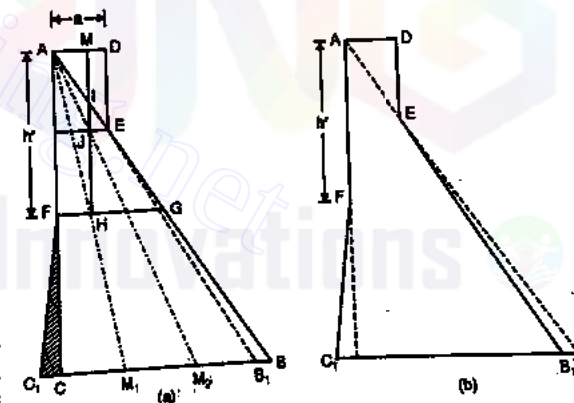


FIG. 8.22

apex of the triangular profile. Fig. 8.22(a) shows the elementary section ABC . If some top width $a = AD$ is provided, the resultant of the dam section will be shifted to the u/s when the reservoir is empty. AM_1 is the inner third point line, and MI is the line passing through the centroid of the added triangle ADE . Both these lines intersect at point H . For all sections below plane FHG , the resultant will, therefore, be shifted to the left of line AM_1 , causing tension at the down stream face when the reservoir is empty. This will, therefore, require the provision of u/s batter FC_1 below the plane FHG . In order to find the depth h' of the plane FHG below which u/s batter is required, we have :

$$FH = AM = \frac{2}{3}a$$

$$FG = 3 FH = 3 \times \frac{2}{3} a = 2a$$

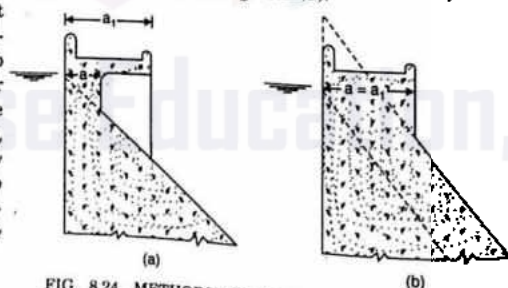
Hence $h' = AF = FG \sqrt{\rho - c}$ (Eq. 8.42)

(where c is the uplift pressure intensity coefficient)

Hence $h' = 2a \sqrt{\rho - c}$... (8.51)

Thus for heights greater than h' , w/s batter will have to be provided. The centroidal line MJ intersects with the outer-third-point line AM_2 at J . Hence, when the reservoir is full, the resultant of all sections below the plane JE is shifted to the w/s side. In order to bring the resultant back to the outer third point line, for the sake of economy, the slope of d/s face may be flattened, bringing it from EB to position EB_1 . Thus, due to the provision of some top width a , the net economical section will be $ADEB_1C_1FA$ as shown in Fig. 8.22(b).

Fig. 8.23 shows the effect of top width of various magnitudes. We see that as the top width is increased, the w/s batter is increased while the d/s slope is decreased. It can be shown, that within limits, the masonry added for the provision of top width decreases, rather than increases, the total masonry volume in the dam.



(b)
FIG. 8.24. METHODS OF PROVIDING TOP WIDTH.

The increase in masonry volume due to provision of top width is counter-balanced by the reduction in the d/s slope at lower levels. However, the most economical top width is the function of height of dam. Creager (1916) has shown that the most economical top width, without considering earthquake effects, is found to be about 14% of the height of dam. However, for dams of low height, the top width provided on the basis of economy (i.e. 14% of height) may have to be increased from other practical considerations, such as provision of roadway on the top etc. Fig. 8.24 shows two alternative methods of providing the top width. Method shown in Fig. 8.24(a) is applicable for low-dams where economical top width a falls short of the top width a_1 required from practical consideration. Method of Fig. 8.24(b) is applicable for high dams.

Free board.

Free board is provided on the requirements of height of waves, as discussed earlier in § 8.17. However, modern practice is to provide a maximum free board equal to 3 to 4 % of the height, though free board equal to 5 % or more might prove economical.

METHODS OF DESIGN OF GRAVITY DAMS :

Gravity dams can be designed by two methods :

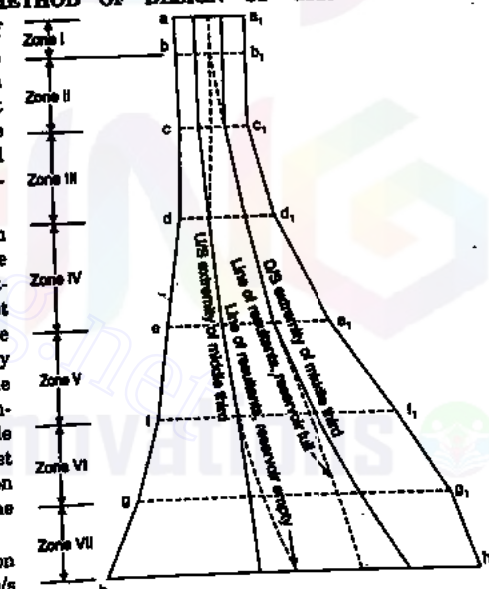
(a) Multiple step method (b) Single step method

(A) MULTIPLE STEP METHOD OF DESIGN OF GRAVITY DAM

For the economic design of gravity dam, the section of the dam is divided into various zones. Each zone is designed in such a way that all the requirements of stability are satisfied. Fig. 8.25 shows typical seven zones in a non-overflow gravity dam.

Zone I. This is the portion a_1b_1b of the dam situated above the maximum reservoir level (or bottom of ice sheet where ice sheet exists). When no ice exists, the height of zone I is controlled by free board requirements and the width is determined by practical considerations or economy for the whole section. In the case of ice sheet the height of block I is fixed on the consideration of sliding of the zone due to ice pressure.

Zone II. This is the portion of the dam in which both the u/s and d/s faces remain vertical. The



h
FIG. B.25. ZONING FOR NON-OVERFLOW GRAVITY DAMS.

position of the bottom plane cc_1 of zone II is such that the resultant forces, when the reservoir is full, passes through the outer third point of the plane cc_1 . However, when reservoir is empty, the line of resultant forces lies well within the middle third, throughout the height of zone II.

Zone III. In this Zone, the u/s face continues to be vertical while the d/s face is inclined. The line of resultant continues to coincide with the d/s extremity of middle third when the reservoir is full. The position of bottom plane dd_1 of zone III is located such that when the reservoir is empty the line of resultant passes through the u/s middle third point.

Zone IV. In this zone, the upstream face also begins to batter so that the lines of resultants lie along the corresponding extremities of middle third. The position of lower limit plane ee , of this zone is governed by the criterion that the maximum inclined pressure at d/s toe, for reservoir full condition is just equal to the allowable limit. The design of zone IV, specially the height as well as both u/s and d/s slopes, are determined by trial, by dividing zone IV into number of convenient blocks, till the bottom of zone IV is reached. It should be noted that *low dams* lie within the limits of zone IV. Zones V, VI and VII are applicable only for high dams.

Zone V. In this zone, the d/s slope is flattened so that the maximum inclined pressure at d/s toe, under reservoir full condition remains within the working stress. Thus the resultant for reservoir full condition remains well within middle third. For the reservoir empty condition, the resultant continues to intersect the u/s extremity of the kern. The lower limit of zone V is marked by plane ff_1 , where the inclined compressive stress on the u/s heel reaches the permissible limit, for the reservoir empty condition.

Zone VI. In this zone, the conditions of the design are determined by the maximum pressures at both u/s and d/s faces under reservoir empty and reservoir full conditions, respectively. The line of resultants under both the conditions lie well within the middle third. The position of the bottom plane gg_1 is reached when the inclined pressure at d/s toe just reaches its maximum value.

Zone VII. This is the zone situated below zone VI, in which maximum compression at the downstream toe exceeds the working limit. This zone is usually eliminated by the revision of the entire design. That is, if the height of dam is so large that it is more than the position of plane gg_1 of zone VI, various changes are made in the upper zones so that the height of the dam lies within the zone VI. If this is not possible, then the height of the dam is reduced or superior material is utilised so that the height is accommodated within VIth zone.

(B) SINGLE STEP METHOD OF DESIGN OF HIGH DAMS

For high dams, going beyond zone IV, it is found that the shape of u/s and d/s slopes are sometimes obtained of unusual shape. Fig. 8.26(a) shows a typical section of high dam obtained by multiple step method. The u/s face has steep slope while smoothy curved or polygonal, may be subject to tensile stresses on surface parallel to the face. Such shape is usually not desirable since the outer layer of such a section

tends to buckle out-ward. Hence, the whole section may have to be revised in such a way that such a reversed curvature is avoided. The simple method would be to start the redesign work from the top, converting the whole dam into a single block controlled by the rules of zone VI.

Fig. 8.26(b) shows the section of the same high dam designed by the single step method. In this method, the u/s slope is kept vertical for some depth, to be determined by trial. It may preliminarily be fixed by Eq. 8.51 such that the plane FG [Fig. 8.24(a)] passes through the intersection of the middle third line AH and the centroidal line MH of the top width triangle. After this height, both the u/s and d/s faces are given such slopes that the maximum inclined pressure at both u/s and d/s face under the conditions of reservoir empty and reservoir full conditions, reach their maximum values simultaneously. This is to be accomplished by trial and error. When this is accomplished, check computations will show that all the stability requirements for the reservoir full or empty conditions are satisfied at all points above the base.

Fig. 8.27 shows the sections of a high dam, designed both by the single step method and multiple step method. It is quite clear from the two sections that the multiple-step design is more economical for the upper portion of the dam. The single-step-method section is understressed at all the points except at the base. Hence if the height of the dam is less, multiple-step-design method would give substantial saving in material.

From the above discussions we draw the following conclusions :

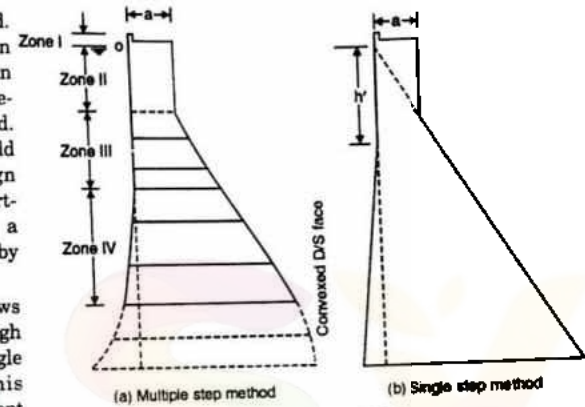


FIG. 8.26. SINGLE STEP METHOD FOR DESIGN OF HIGH

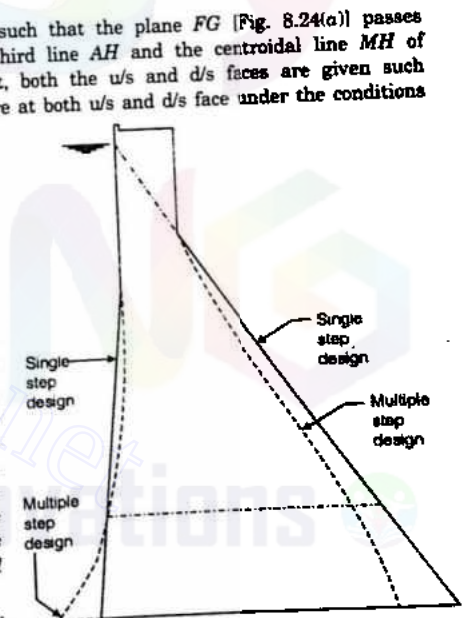


FIG. 8.27. COMPARISON OF MULTIPLE-STEP METHOD AND SINGLE STEP METHOD FOR THE DESIGN OF HIGH DAMS.

(i) Dams of lesser heights can be designed economically only by multiple step-design method.

(ii) It may be economical to increase the masonry strength through the use of more-expensive materials, thus keeping out of zones V and VI.

(iii) High dams beyond zones VI are designed by single step method so that convex-curvature of d/s face is avoided.

Alternative method for the design of high gravity dam.

We have seen earlier that if the height of the dam is greater than that given by equation $h = \frac{f}{w(\rho + 1)}$, the dam is known as a high dam. The design of such a dam, below the limiting height, is done by dividing the height into a number of suitable strips as shown in Fig. 8.28.

Design of first strip

Let b_1 = base width of the first strip.

b = base width of the low dam.

h_1 = height of the dam portion from H.F.L. to the bottom of the first strip.

W_1 = total vertical load of dam and water, above the base of the first strip.

d_1 = height of the first strip.

W = total weight of dam portion and water, on the top of the first strip.

Then the base width b_1 , sufficient to keep the maximum compressive stress within the safe value f given by

$$b_1 = \sqrt{\frac{wh_1^3}{f} \left(1 + \frac{w^2 h_1^4}{4 W_1^2} \right)} \quad \dots(8.52)$$

The projection x_1 on the w/s of the heel is given by the following expression:

$$\frac{\rho w d_1}{24} \{ 3b^2 - b_1^2 + 6x_1(b + b_1) + 2bb_1 \} - \frac{wx_1}{12} (h + h_1)(2b_1 - 3x_1) - W \left(\frac{b_1 - b}{3} - x_1 \right) = 0 \quad \dots(8.53)$$

Design for second strip

Using suffix 2 for the second strip, the corresponding expressions for b_2 and x_2 are as follows :

$$b_2 = \sqrt{\frac{wh_2^3}{f} \left(1 + \frac{w^2 h_2^4}{4 W_2^2} \right)} \quad \dots[8.52(a)]$$

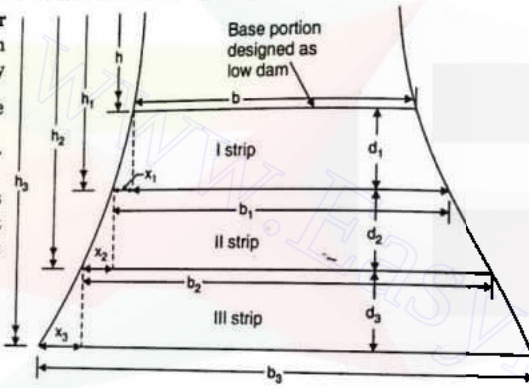


FIG. 8.28. DESIGN OF HIGH GRAVITY DAM.

$$\frac{\rho w d_2}{24} \{ 3b_1^2 - b_2^2 + 6x_2(b_1 + b_2) + 2b_1b_2 \} - \frac{wx_2}{12} (h_1 + h_2)(2b_2 - 3x_2) - W_1 \left(\frac{b_1 - b_2}{3} - x_2 \right) = 0 \quad \dots[8.53(a)]$$

Similarly, the remaining strips are designed, till the base of the dam is reached.

8.21. GALLERIES

A gallery is a formed opening left in a dam. This may run in transverse or longitudinal direction and may run horizontally or on a slope. The shape and size varies from dam to dam and is generally governed by the functions it has to perform. Following are the purposes for which a gallery is formed in the dams.

1. To provide drainage of the dam section. Some amount of water constantly seeps through the upstream face of the dam which is drained off through galleries.
2. To provide facilities for drilling and grouting operations for foundations etc. Drillings for drain is generally resorted to clean them if they are clogged. High pressure grouting and required drilling for it is generally carried out after the completion of dam. This can be best done through galleries.

3. To provide space for header and return pipes for post cooling of concrete and grouting the longitudinal joints after completion of dam.

4. To provide access to observe and measure the behaviour of the structure after its completion by fixing thermo-couples and examining development of cracks etc.

5. To provide an access of mechanical contrivances needed for the operation of outlet gates and spillway gates.

Fig. 8.29 shows two typical shapes of galleries. In the rectangular gallery [Fig. 8.29(a)], all the corners should be rounded so that stress concentration is minimum. Where drainage trough is required, a rectangular trough such as at c is necessary, though shape d is preferred. Fig. 8.29(b) shows another form. To provide walkway, the semi-circular bottom is filled in with an unbounded slab.

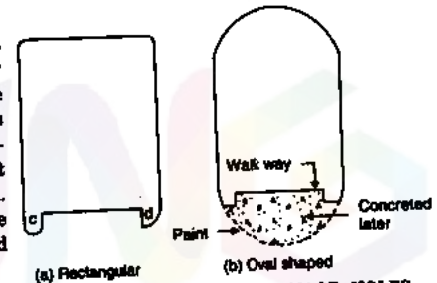


FIG. 8.29. SHAPES OF DRAINAGE HOLES.

Shafts. Vertical openings in the dam are called shafts. Shafts are provided to connect galleries at various levels. Plumber shaft is provided to measure the deflections of dam by suspending a plumb bob in it.

8.22. JOINTS, KEYS AND WATER SEALS

Joints. Various joints provided in dams may be classified under two heads : (a) construction joints, (b) contraction joints.

(a) Construction Joints. Construction joints, usually called horizontal joints are necessary since the entire work of concreting the whole dam cannot be completed in one stretch. The concreting is, therefore, done in various stages. In solid gravity dams, the height between horizontal joints is usually limited to about 1.5 m. This

height between two successive construction joints or horizontal joints is known as *lift*. Evidently, this height is limited to the necessity of providing sufficient cooling between pours. For the first layer immediately on the rock, half this amount, i.e. 0.75 m height is adopted as the lift. Modern treatment of the surface and good concrete create an automatic water-tight horizontal joints, and as such no provisions, such as water stops or keyways etc., are made in the horizontal joints. The contraction joints, though provided for different purpose, also serve as construction joints.

(b) **Contraction Joints.** Contraction joints are mainly provided to avoid cracks caused due to shrinkage of concrete due to temperature changes. The number of cracks

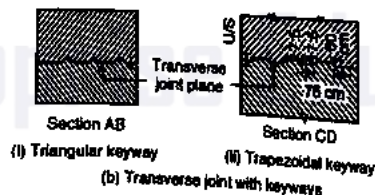
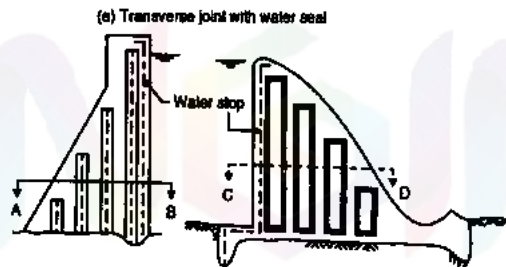
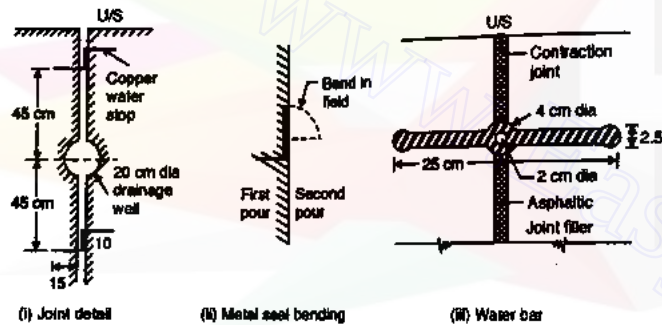


FIG. 8.30. TRANSVERSE JOINTS.

and their size can also be controlled to some extent, by controlled temperature of concrete while setting and curing. Contraction joints are of two types : (i) transverse joints, and (ii) longitudinal joints, depending upon the plane in which they lie.

(i) **Transverse Joints.** Transverse joints are provided normal to the axis of the dam. They are provided to prevent haphazard transverse cracks due to contraction of the concrete. The spacing of such joints depends upon a number of factors, including the position of the structure and the topographic features of dam site. The general practice is to restrict the spacing of the joints to about 15 m or the height of the dam, whichever is less. These joints extend entirely through the structure. In order to ensure intimate contact for transmission of stresses, transverse joints are consolidated either by grouting or by leaving a slot which is filled later when the monoliths have ceased shrinking. However, the transverse joints of solid gravity dams are not generally grouted. In order to make the joints water-tight, keyways or water stops are invariably provided in the transverse joints.

Fig. 8.30(a) shows a transverse joint with two ordinary copper water stops near the u/s side of the joint which confines a 20 cm drainage well which may be filled later with melted asphalt or grouted. Fig. 8.30(a)(ii) shows the method of bending the water stop in the field. When the foundations of the dam are of yielding type, the metal seals are damaged or broken. In that case, water bars of natural or synthetic rubbers and plastics, such as polyvinyl chloride etc. may be used, as shown in Fig. 8.30 (a) (iii). The joint is filled with asphaltic joint-filler or any other joint sealing compound which may provide a water-tight seal by adhesion to the concrete throughout the joint movement.

(ii) **Longitudinal Joint.** These are provided parallel to the axis of the dam to prevent longitudinal cracks. Since longitudinal cracks may coincide closely with the planes of maximum shear, these cracks are objectionable from the stand point of safety. Longitudinal joints run between two adjacent transverse joints and are thus not continuous all along the length of the dam. However, they are continuous in vertical plane, dividing the dam into a number of blocks. Fig. 8.31 shows the positions of transverse and longitudinal joints.

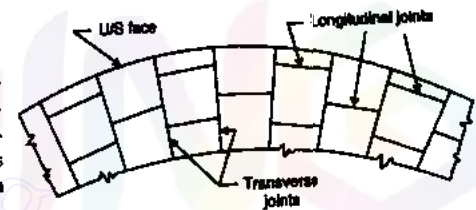


FIG. 8.31. THE SPACING OF LONGITUDINAL JOINTS.

The spacing of longitudinal joints is also limited to 15 m (or 50 ft). Keyways are invariably provided in the vertical longitudinal joints. These keyways are designed to transmit the principal stresses, and are spaced at 1.5 m (or 5 ft), i.e. one key in each lift. Fig. 8.32 shows a typical section through vertical longitudinal joints at Shasta Dam.

Keys. Keys are invariably provided in longitudinal joints to permit the transfer of shearing stresses from one block to the other. However, their provision in the transverse

joints is optional. The adjoining surfaces of each side of the joint are given such a shape as to be interlocked together for the transfer of stresses. Keyways give a measure of shearing strength required at times when the dam is only partly full and the lines of first principal stress are not parallel to the joints. Keyways may be of several shapes, such as triangular [Fig. 8.30(b)], trapezoidal [Fig. 8.30(c)] or trough shaped [Fig. 8.32].

Water stops.

Water stops are provided in both the types of joints, to prevent leakage of water. Water stops are also sometimes known as *water bars*. Water stops may be either of metal, such as annealed copper, steel, monel, metal or sheet lead, or of natural or synthetic rubbers and plastics such as polyvinyl chloride. Metal water stops are provided only in the case of non-yielding foundations. For other cases of yielding foundation, rubber water bars are preferred. Sometimes, along with water bars, drainage well is also used which is filled with asphalt. Water stop, when accompanied by drainage wells are used in pairs [Fig. 8.30(a)].

8.23. CONTROL OF CRACKING IN CONCRETE DAMS

If proper temperature control is not exercised, the large concrete block between the joints may crack due to high temperature gradient between the interior and the surface. These large blocks of concrete are subject to deep as well as surface cracking. The cracks in the interior of the blocks are produced due to heat of hydration liberated by cement thus giving rise to high temperature gradient. The surface cracks may appear due to daily variations of the temperature at surface. The surface cracks are more harmful, since the disintegration starts through them by wedge action. Water enters these surface cracks, accumulates there and then solidifies. The ice so formed begins to expand at 4°C, resulting in widening and deepening of cracks.

Following are some of the methods employed to check or minimise the development of the cracks in mass concrete :

1. **Pre-cooling of concrete.** The concrete is pre-cooled before it is placed in the dam. This is accomplished by cooling the aggregate by refrigerated water, blowing of air through them, cooling of sand and using refrigerated water for the manufacture of concrete. All this involves high cost of preparing the concrete.

2. **Post cooling of concrete.** The post cooling is achieved by circulating refrigerated water through pipes embedded in concrete in each lift. The cooling is begun immediately after a block is laid and is continued till the mass temperature falls to the mean annual temperature of the locality. For the purposes of circulating cool water, thin

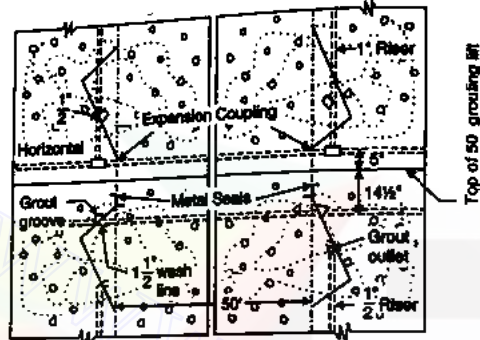


FIG. 8.32. SECTION THROUGH VERTICAL LONGITUDINAL JOINT : SHASTA DAM.

walled tubing 2.5 cm outside diameter and 250 m long joined with expansion coupling are made into coils and are placed in the middle of each lift. The horizontal spacing of these tubings may be between 0.5 to 2 m and the velocity of flow is kept more than 60 cm per second. Resistance thermometers are embedded in the concrete to ascertain the temperature.

3. Using *low heat cement* for the concrete.
4. Using *lower percentage of cement* in the concrete for the interior of the blocks, say about 80 % of that for the exterior.
5. *Restricting the height of lift*, to say 1.5 m.
6. *Allowing considerable time* between laying of two successive vertical lifts. The usual time is about 4 days.
7. *Providing contraction joints* at suitable spacings.

Example 8.5. Considering earthquake forces due to uniform horizontal acceleration $a_h = a$ and uniform vertical acceleration $a_v = a$ in addition to the hydrostatic pressure and uplift pressure, determine the base width of the elementary profile of gravity dam so that resultant passes through the outer third point.

Solution

Let b = base width of elementary profile dam ABC.

The various forces acting on the dam are shown in Fig. 8.33.

(a) Vertical forces

1. Force due to self-weight of dam = $W = \frac{1}{2} b H w \rho$ (↓)
2. Force due to vertical acceleration of earthquake

$$= \alpha W = \frac{\alpha b H w \rho}{2} \uparrow$$

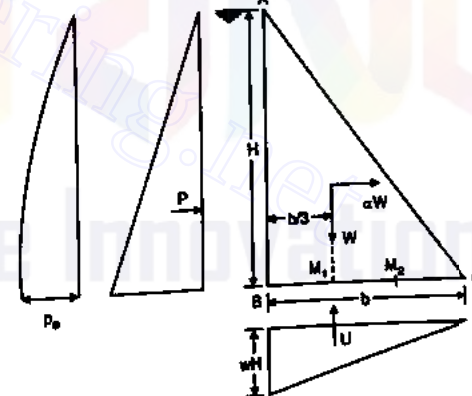


FIG. 8.33.

3. Force due to uplift $= U = \frac{1}{2} b \cdot wH$ (↑)

Hence $\Sigma V = W - \alpha W - U = \frac{1}{2} b w H [(1 - \alpha) \rho - 1]$

where w = unit weight of water, ρ = specific weight of concrete
and α = coefficient of earthquake acceleration (both vertical as well as horizontal).

(b) Horizontal forces

1. Force due to water pressure $P = \frac{1}{2} wH^2$

2. Force due to hydrodynamic pressure of water at base :

$$C_m = 0.735 \left(1 - \frac{\theta}{90^\circ} \right) = 0.735 \text{ (since } \theta = 90^\circ \text{)}$$

$$p_r = C_m \alpha wH = 0.735 \alpha wH$$

$$P_r = 0.726 p_r \cdot H$$

$$M_r = 0.299 p_r H^2 = 0.299 \times 0.735 \alpha wH^3 = 0.2198 \alpha wH^3$$

3. Inertia force (horizontal) $= \alpha W = \frac{1}{2} \alpha b H w \rho$

If the resultant of all forces has to pass through the outer third point M , moment of all these forces at this point must be zero.

$$\Sigma V \times \frac{b}{3} - \left[\frac{wH^2}{2} \times \frac{H}{3} + 0.2198 \alpha wH^3 + \frac{\alpha w b H \rho}{2} \times \frac{H}{3} \right] = 0$$

$$\text{or } \frac{b^2 H w}{6} [(1 - \alpha) \rho - 1] - \frac{bH^3 w \rho \alpha}{6} - \frac{wH^3}{6} [1 + \alpha (1.3186)] = 0$$

$$\text{or } b^2 [(1 - \alpha) \rho - 1] - bH\rho\alpha - H^2 [1 + 1.3186\alpha] = 0$$

$$\text{or } b = H \frac{\rho\alpha \pm \sqrt{\rho^2\alpha^2 + 4(1 + 1.3186\alpha)[(1 - \alpha)\rho - 1]}}{2[(1 - \alpha)\rho - 1]} \quad \dots(8.54)$$

which is the required expression.

Putting $\alpha = 0$ when no earthquake acts, the value of b reduces to

$$b = \frac{H}{\sqrt{\rho - 1}}$$

which is the same as Eq. 8.42 when $c = 1$.

Example 8.6. Analysis of Rectangular profile dam

A rectangular dam has constant width b and height H with reservoir full upto its top. Analyse the dam completely.

Solution

Consider a rectangular dam of width b and height H . (Fig. 8.34)

$$W = bH\rho w; P = \frac{wH^2}{2}; U = \frac{1}{2} cw bH$$

Let the resultant pass through a point M , distant $\bar{x}$ from the toe B . Taking moments about the toe, we have.

$$\Sigma M = W \cdot \frac{b}{2} - \frac{PH}{3} - U \cdot \frac{2b}{3}$$

$$\text{Also } \Sigma W = W - U,$$

$$\bar{x} = \frac{\Sigma M}{\Sigma W} = \frac{W \cdot \frac{b}{2} - \frac{PH}{3} - U \cdot \frac{2b}{3}}{W - U}$$

Substituting the values of W , P and U , we get.

$$\bar{x} = \frac{bH\rho w \cdot \frac{b}{2} - \frac{wH^3}{6} - \frac{2}{3} b \left(\frac{1}{2} cw bH \right)}{bH\rho w - \frac{1}{2} cw bH}$$

$$\text{or } \bar{x} = \frac{\frac{b}{2} (bH\rho w - \frac{1}{2} cw bH) - \frac{wH^3}{6} - (\frac{1}{2} cw bH) \frac{b}{6}}{bH\rho w - \frac{1}{2} cw bH}$$

$$= \frac{\frac{b}{2} \left[\frac{wH^3}{6} + \frac{1}{2} cw bH \cdot \frac{b}{6} \right]}{bH\rho w - \frac{1}{2} cw bH}$$

$$\text{Hence } e = \frac{b}{2} - \bar{x} = \frac{\frac{wH^3}{6} + \frac{1}{2} cw bH \cdot \frac{b}{6}}{bH\rho w - \frac{1}{2} cw bH} = \frac{H^2 + \frac{cb^2}{12}}{b\rho - \frac{1}{2} cb}$$

For no tension to develop

$$e = \frac{b}{6} = \frac{H^2 + \frac{cb^2}{12}}{b\rho - \frac{1}{2} cb}$$

$$\text{or } \frac{b^2}{6} \left(\rho - \frac{c}{2} \right) = \frac{H^2}{6} + \frac{cb^2}{12} \text{ or } b^2 \left(\rho - c \right) = H^2$$

$$\text{From which } b = \frac{H}{\sqrt{\rho - c}} \quad \text{If } c = 0 \text{ (no uplift), } b = \frac{H}{\sqrt{\rho}}$$

Also, the limiting height is given by, $H_{\text{lim}} = b \sqrt{\rho - c}$

From stability considerations, providing a F.S. of 1.5 against sliding, we have

$$\mu (W - U) = 1.5 P$$

$$\text{or } \mu (bH\rho w - \frac{1}{2} cw bH) = \frac{3}{2} \times \frac{wH^2}{2} \text{ or } \mu b \left(\rho - \frac{c}{2} \right) = \frac{3}{4} H$$

$$\text{From which } b = \frac{3}{4} \frac{H}{\mu \left(\rho - \frac{c}{2} \right)}$$

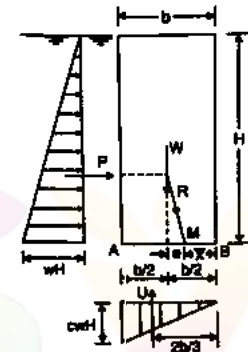


FIG. 8.34. RECTANGULAR PROFILE

If $c = 0$ (i.e. no uplift) $b = \frac{3H}{4\mu\rho}$

Also, limiting height is, $H_{\text{lim}} = \frac{4}{3}b\mu\left(\rho - \frac{c}{2}\right)$

Neglecting uplift, $H_{\text{lim}} = \frac{4}{3}b\mu\rho$

From crushing point of view, when $e = b/6$, we get

$$p_n = \frac{2(W-U)}{b} = \frac{2}{b}\left(bH\rho w - \frac{1}{2}cwbH\right)$$

$$\sigma_c = p_n = 2wH\left(\rho - \frac{1}{2}c\right)$$

$$H_{\text{lim}} = \frac{\sigma_c}{2w\left(\rho - \frac{1}{2}c\right)}$$

If uplift is neglected, $H_{\text{lim}} = \frac{\sigma_c}{2w\rho}$

Example 8.7. Analysis of trapezoidal dam

A trapezoidal dam with u/s face vertical, has top width a and bottom width b . Analyse the dam completely.

Solution :

Consider a trapezoidal profile with top width a and bottom width b , with upstream face vertical, as shown in Fig. 8.35.

W_1 = weight of rectangular portion
 $= aH\rho w$

W_2 = weight of triangular portion
 $= \frac{1}{2}bH \cdot \rho w$

P = Water pressure $= \frac{wH^2}{2}$

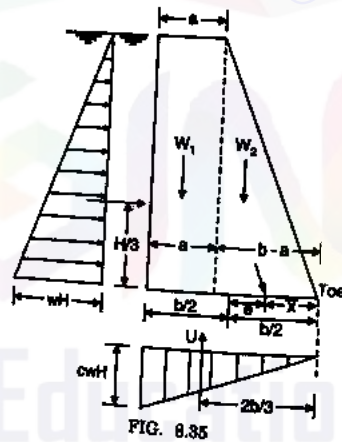
C = uplift pressure $= \frac{1}{2}cwbH$

Let the resultant fall at $\bar{x}$ from the toe.

If ΣM is the sum of moments of all the forces about the toe, we have

$$\Sigma M = \bar{x} \cdot \Sigma W \quad \text{or} \quad \bar{x} = \frac{\Sigma M}{\Sigma W}$$

$$\bar{x} = \frac{aH\rho w\left(b - \frac{a}{2}\right) + \frac{1}{2}bH\rho w \times \frac{2}{3}\left(b - \frac{a}{2}\right) - \frac{wH^3}{6} - \frac{1}{2}cwbH \times \frac{2}{3}b}{aH\rho w + \frac{1}{2}bH\rho w - \frac{1}{2}cwbH}$$



$$\bar{x} = \frac{a\rho\left(b - \frac{a}{2}\right) + \frac{1}{3}b\rho(b-a) - \frac{H^2}{6} - \frac{1}{3}cb^2}{a\rho + \frac{1}{2}b\rho - \frac{1}{2}cb}$$

or

$$\text{Eccentricity } e = \frac{b}{2} - \bar{x} \quad \text{and} \quad p_n = \frac{\Sigma W}{b} \left(1 \pm \frac{6e}{b}\right)$$

Thus, stress at toe and heel can be found.

Example 8.8. Rectangular masonry dam

A rectangular masonry dam is 3 m at the base. Compute the maximum permissible height H (a) when no tension is permissible, and (b) when the factor of safety against sliding is 1.5. Given the following : (i) $\mu = 0.5$, (ii) density of masonry = 24 times the density of water, and (iii) $c = 1$. What will be corresponding values of H if uplift is neglected?

Solution

Given : $\rho = 2.4$; $w = 9.81 \text{ kN/m}^3$

(i) Considering uplift

$$W = bH\rho w = 3 \times 2.4 \times 9.81 H = 70.632 H \text{ kN}$$

$$U = \frac{1}{2}cwbH = \frac{1}{2} \times 1 \times 9.81 \times 3 H = 14.715 H \text{ kN}$$

$$P = \frac{wH^2}{2} = \frac{9.81 H^2}{2} = 4.905 H^2 \text{ kN}$$

$$\Sigma W = 70.632 H - 14.715 H = 55.917 H$$

Taking moments about the toe,

$$\Sigma M = (70.632 H \times 1.5) - (14.715 H \times 2) - 4.905 H^2/3 = 76.518 H - 1.635 H^2$$

$$\bar{x} = \frac{\Sigma M}{\Sigma W} = \frac{76.518 H - 1.635 H^2}{55.917 H} = 1.368 - 0.02924 H^2$$

(a) For no tension to develop, $e = b/6 = 3/6 = 0.5 \text{ m}$

Hence, $e = (1.5 - \bar{x}) = 0.132 + 0.02924 H^2 = 0.5$

From which, $H_{\text{lim}} = 3.55 \text{ m}$

Alternatively, from Eq. 18.45 (b), $H_{\text{lim}} = b\sqrt{\rho - c} = 3\sqrt{2.4 - 1} = 3.55 \text{ m}$

(b) From stability consideration, providing a F.S. of 1.5 against sliding, we have

$$\mu(W - U) = 1.5 P$$

$$0.5(70.632 H - 14.715 H) = 1.5 \times 4.905 H^2 \quad \text{or} \quad 27.9585 H = 7.3575 H^2$$

From which

$$H = H_{\text{lim}} = 3.8 \text{ m}$$

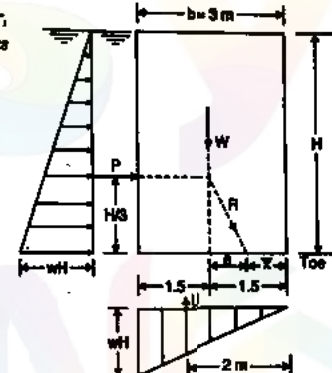


FIG. 8.36

Alternatively, from Eq. 18.47, $H_{lim.} = \frac{4}{3} b \mu \left(\rho - \frac{c}{2} \right) = \frac{4}{3} \times 3 \times 0.5 (2.4 - 0.5) = 3.8 \text{ m}$.

(ii) **Neglecting uplift**

$W = 70.632 H$ and $P = 4.905 H^2$, as before,

$$\Sigma M = 70.632 H \times 1.5 - 4.905 H^3 / 3 = 105.948 H - 1.635 H^3$$

$$\Sigma W = W = 70.632 H$$

$$\bar{x} = \frac{\Sigma M}{\Sigma W} = \frac{105.948 H - 1.635 H^3}{70.632 H} = 1.5 - 0.02315 H^2$$

$$e = \frac{b}{2} - \bar{x} = 1.5 - (1.5 - 0.02315 H^2) = 0.02315 H^2$$

(a) For no tension to develop, $e = b/6 = 3/6 = 0.5 \text{ m}$

$$0.02315 H^2 = 0.5, \text{ from which } H = H_{lim.} \approx 4.65 \text{ m}$$

(b) From stability consideration, providing a F.S. of 1.5 against sliding,

$$\mu W = 1.5 P \text{ or } 0.5 \times 70.632 H = 1.5 \times 4.905 H^2. \text{ From which } H = 4.8 \text{ m}$$

Example 8.9. Trapezoidal masonry dam

A masonry dam 6 m high is 1.5 m wide at top and 4.5 m wide at the bottom, with vertical water face. Determine the normal stresses at the toe and heel for reservoir empty and reservoir full conditions. Take $\rho = 2.4$ and $c = 1$.

Solution (Fig. 8.37)

Consider 1 m length of the dam.

(a) **Reservoir empty**

$$\text{Weight } W_1 = 1.5 \times 6 \times 1 (2.4 \times 9.81) = 211.9 \text{ kN}$$

$$\text{Weight } W_2 = \frac{1}{2} \times 3 \times 6 (2.4 \times 9.81) = 211.9 \text{ kN}$$

$$\Sigma W = 211.9 + 211.9 = 423.8 \text{ kN}$$

Taking moments about the toe (B) we have

$$\bar{x} = \frac{\Sigma M}{\Sigma W} = \frac{211.9 \times 3.75 + 211.9 \times 2}{423.8} = 2.875 \text{ m}$$

$$e = \frac{b}{2} - \bar{x} = 2.25 - 2.875 = -0.625 \text{ m}$$

(i.e. resultant falls to the left of middle point).

$$\therefore (p_n)_{\text{toe}} = \frac{\Sigma W}{b} \left(1 - \frac{6e}{b} \right) = \frac{423.8}{4.5} \left(1 - \frac{6 \times 0.625}{4.5} \right) = 15.7 \text{ kN/m}^2 \text{ (comp.)}$$

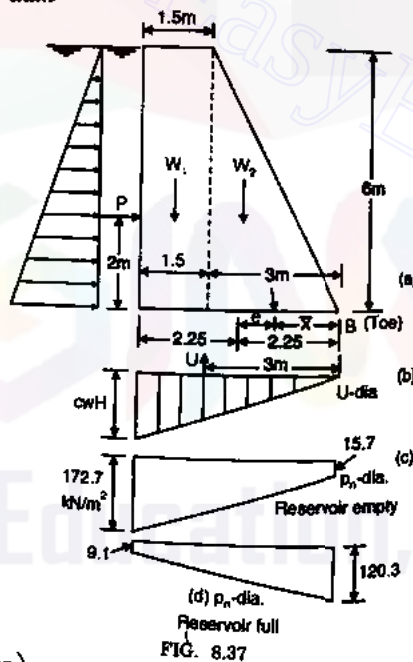


FIG. 8.37

DAMS-II: GRAVITY DAMS

$$\therefore (p_n)_{\text{heel}} = \frac{\Sigma W}{b} \left(1 + \frac{6e}{b} \right) = \frac{423.8}{4.5} \left(1 + \frac{6 \times 0.625}{4.5} \right) = 172.7 \text{ kN/m}^2 \text{ (comp.)}$$

The p_n distribution below the base is shown in Fig. 18.24 (c)

(b) **Reservoir full** : $W_1 = 211.9 \text{ kN}$ and $W_2 = 211.9 \text{ kN}$, as before.

$$P = \frac{wH^2}{2} = \frac{9.81(6)^2}{2} = 176.58 \text{ kN}$$

$$U = \frac{1}{2} cwH \cdot b = \frac{1}{2} \times 1 \times 9.81 \times 6 \times 4.5 = 132.435 \text{ kN}$$

$$\Sigma W = 211.9 + 211.9 - 132.435 = 291.365 \text{ kN}$$

Taking moments about the toe, we have

$$\bar{x} = \frac{\Sigma M}{\Sigma W} = \frac{211.9 \times 3.75 + 211.9 \times 2 - 132.435 \times 3 - 176.58 \times 2}{291.365} = 1.806 \text{ m}$$

$$\therefore e = \frac{b}{2} - \bar{x} = 2.25 - 1.806 = 0.444 \text{ m}$$

$$\therefore (p_n)_{\text{toe}} = \frac{\Sigma W}{b} \left(1 + \frac{6e}{b} \right) = \frac{291.365}{4.5} \left(1 + \frac{6 \times 0.444}{4.5} \right) = 120.3 \text{ kN/m}^2 \text{ (comp.)}$$

$$\therefore (p_n)_{\text{heel}} = \frac{\Sigma W}{b} \left(1 - \frac{6e}{b} \right) = \frac{291.365}{4.5} \left(1 - \frac{6 \times 0.444}{4.5} \right) = 9.1 \text{ kN/m}^2 \text{ (comp.)}$$

The p_n distribution below the base is shown in Fig. 18.37 (d).

Example 8.10. Trapezoidal masonry dam

A dam 6 m high and 1.5 m wide at the top has vertical water face. Find the base width of the dam if no tension is to develop. Take unit weight of masonry as 20 kN/m^3 and $c = 1$.

Investigate the stability of the above dam if the coefficient of friction is 0.6 and maximum allowable compressive stress is 1800 kN/m^2 .

Solution

Let the bottom width be $b \text{ m}$, as shown in Fig. 8.38.

$$W_1 = 1.5 \times 6 \times 20 = 180 \text{ kN}$$

$$W_2 = \frac{1}{2} (b - 1.5) \times 6 \times 20 = 60 (b - 1.5)$$

$$P = \frac{wH^2}{2} = \frac{9.81(6)^2}{2} = 176.58 \text{ kN}$$

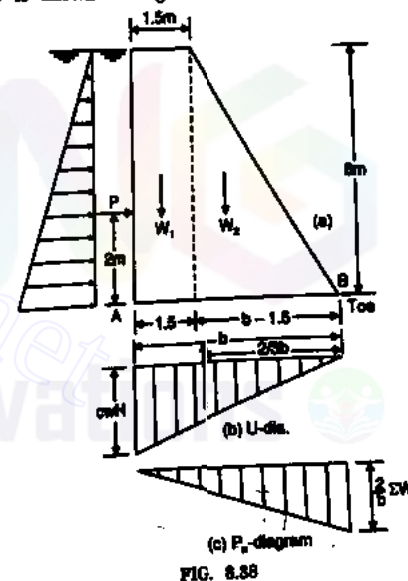


FIG. 8.38

$$U = \frac{1}{2} c w H \cdot b = \frac{1}{2} \times 1 \times 9.81 \times 6 b = 29.43 b$$

$$\Sigma W = 180 + 60(b - 1.5) - 29.43 b = 90 + 30.57 b$$

Taking moments about the toe, we get

$$\bar{x} = \frac{\Sigma M}{\Sigma W} = \frac{180(b - 1.5 + 0.75) + 60(b - 1.5) \frac{2}{3}(b - 1.5) - 176.58 \times 2 - 29.43 b \frac{2}{3} b}{90 + 30.57 b}$$

For no tension to develop, e should be equal to $b/6$.

$$\frac{b}{2} - \bar{x} = \frac{b}{6} \quad \text{or} \quad \bar{x} = \frac{b}{3}$$

$$\text{Hence,} \quad \frac{b}{3} = \frac{180(b - 0.75) + 40(b - 1.5)^2 - 353.16 - 19.62 b^2}{90 + 30.57 b}$$

$$\text{From which, we get} \quad b^2 - 2.944 b - 39.074 = 0$$

$$\text{Hence} \quad b \approx 4.95 \text{ m}$$

Check for crushing

$$\Sigma W = 90 + 30.57 b = 90 + 30.57 \times 4.95 = 241.322 \text{ kN}$$

$$(p_n)_{\max} = \frac{2 \Sigma W}{b} = \frac{2(241.322)}{4.95} = 97.5 \text{ kN/m}^2 < 1800 \text{ OK.}$$

Check for sliding :

$$\text{F.S.} = \frac{\mu \Sigma W}{P} = \frac{0.6 \times 241.322}{176.58} = 0.82$$

Hence the dam is *unsafe* against sliding.

If a minimum factor of safety of 1 is required under the conditions of uplift, we have

$$\mu \Sigma W = P$$

$$0.6(90 + 30.57 b) = 176.58 \text{ from which } b = 8.693 \text{ m}$$

Example 8.11. What will be the base width in the dam of Example 8.10 if uplift is neglected? Investigate the corresponding stability conditions.

Solution

$$W_1 = 180 \text{ kN} ; W_2 = 60(b - 1.5) ; P = 176.58 \text{ kN}$$

$$\Sigma W = 180 + 60(b - 1.5) = 90 + 60 b$$

$$\bar{x} = \frac{\Sigma M}{\Sigma W} = \frac{180(b - 1.5 + 0.75) + 60(b - 1.5) \frac{2}{3}(b - 1.5) - 176.58 \times 2}{90 + 60 b}$$

But for no tension to develop, $\bar{x} = b/3$

$$\frac{b}{3} = \frac{180(b - 0.75) + 40(b - 1.5)^2 - 353.16}{90 + 60 b}$$

$$\text{From which, we get } b^2 + 1.5 b - 19.908 = 0. \text{ Hence } b = 3.774 \text{ m.}$$

$$\text{Check for crushing : } \Sigma W = 90 + 60 b = 90 + 60 \times 3.774 = 318.44 \text{ kN}$$

$$(p_n)_{\max} = \frac{2 \Sigma W}{b} = \frac{2 \times 318.44}{3.774} = 167.7 \text{ kN/m}^2 < 1800. \text{ Hence safe.}$$

Check for sliding

$$\text{F.S.} = \frac{\mu \Sigma W}{P} = \frac{0.6 \times 318.44}{176.58} = 1.076 > 1. \text{ Hence safe.}$$

Example 8.12 : Trapezoidal masonry dam with top shutter

A trapezoidal dam section of 12 m height has vertical upstream face, with 3 m top width and 9 m bottom width. In order to increase the storage, it is proposed to raise the water height by erecting steel shutter at the top of the dam, in line with the vertical face of the dam. Determine the maximum permissible height of shutter so that no tension is developed at the upstream end of the base, under the reservoir full condition. Neglect foundation seepage. Take unit weight of masonry as 21 kN/m³.

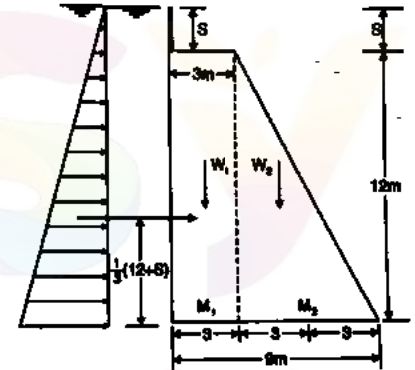


FIG. 8.39

Solution (Fig. 8.39)

Let the height of shutter be s . In order that no tension is developed, the resultant must pass through the outer third point M_2 , distant 3 m from the toe. Hence ΣM at point M_2 must be zero.

Now $W_1 = 3 \times 12 \times 21 = 756 \text{ kN}$ acting at 4.5 m from M_2

$$W_2 = \frac{1}{2} \times 6 \times 12 \times 21 = 756 \text{ kN acting at 1 m from } M_2.$$

$$P = \frac{w H^2}{2} = \frac{9.81 (12 + s)^2}{2} = 4.905 (12 + s)^2 \text{ kN}$$

acting at $\frac{1}{3} (12 + s)$ m above the base.

Taking moments about M_2 ,

$$\Sigma M = 756 \times 4.5 + 756 \times 1 - 4.905 (12 + s)^2 \times \frac{1}{3} (12 + s) = 0$$

$$\text{or} \quad (12 + s)^3 = 2543 \quad \text{or} \quad (12 + s) = 2543^{1/3} = 13.85$$

$$s = 1.85 \text{ m}$$

Hence height of shutter should not exceed 1.85 m.

Example 8.13. A gravity dam has the following dimensions :

$$\text{Height of dam} = 100 \text{ m}$$

$$\text{Free board} = 1 \text{ m}$$

Slope of u/s face = 0.15 : 1.

Taking uniform value of $\alpha_h = 0.1$, determine (i) hydrodynamic earthquake pressure and (ii) its moment at a joint situated 50 m below maximum water surface.

Solution. If θ is the angle that u/s slope makes with vertical, we have

$$\theta = \tan^{-1} \left(\frac{0.15}{1} \right) \approx 8.5^\circ$$

$$C_m = 0.735 \left(1 - \frac{8.5^\circ}{90^\circ} \right) = 0.666$$

Here $h = 100$ m and $y = 50$ m,

$$\therefore C_y = \frac{C_m}{2} \left[\frac{y}{h} \left(2 - \frac{y}{h} \right) + \sqrt{\frac{y}{h} \left(2 - \frac{y}{h} \right)} \right] = \frac{0.666}{2} \left[0.5 (2 - 0.5) + \sqrt{0.5 (2 - 0.5)} \right] = 0.538$$

Now $p_{ey} = C_y \alpha_h w h = 0.538 \times 0.1 \times 9.81 \times 100 = 52.778 \text{ kN/m}^2$.

Hence $P_{ey} = 0.726 p_{ey} \cdot y = 0.726 \times 52.778 \times 50 = 1915.8 \text{ kN/m}$

$$M_{ey} = 0.229 p_{ey} \cdot y^2 = 0.229 \times 52.778 (50)^2 = 30451.4 \text{ kN-m/m}$$

Example 8.14. Trapezoidal dam : A masonry dam 10 m high is trapezoidal in section with a top width of 1 m and bottom width of 8.25 m. The face exposed to water has a batter of 1:10. Test the stability of the dam. Find out the principal stresses at the toe and heel of the dam. Assume unit weight of masonry as 22.4 kN/m^3 , w for water = 9.81 kN/m^3 and permissible shear stress of joint = 1400 kN/m^2 .

Solution. Assuming no free board, stability is tested by considering only three forces. Consider 1 m length of dam.

1. Vertical forces

(a) Self-weight of the dam

$$= \frac{8.25 + 1}{2} \times 10 \times 1 \times 22.4 = 1036 \text{ kN.}$$

(b) Weight of water in column AA'B

$$= \frac{10 \times 1}{2} \times 9.81 = 49.05 \text{ kN.}$$

(c) Uplift force

$$= \frac{1}{2} \times 8.25 \times 9.81 \times 10 = 404.66 \text{ kN}$$

$$\Sigma V = 1036 + 49.05 - 404.66 = 680.39 \text{ kN.}$$

2. Horizontal water pressure

$$\Sigma H = \frac{wh^2}{2} = \frac{9.81 \times 100}{2} = 490.05 \text{ kN.}$$

Moment due to various forces at toe

Consider overturning moment as -ve and restoring moment is +ve.



(a) Due to self-weight of dam :

$$\text{Moment} = \left\{ \left(\frac{1}{2} \times 1 \times 10 \times 22.4 \right) \left(7.25 + \frac{1}{3} \right) \right\} + \left\{ \left(1 \times 10 \times 22.4 \right) \left(6.25 + 0.5 \right) \right\} + \left\{ \left(\frac{1}{2} \times 6.25 \times 10 \times 22.4 \right) \left(\frac{2}{3} \times 6.25 \right) \right\} = 5278 \text{ kN-m (+ve)}$$

(b) Due to column of water in AA'B :

$$\text{Moment} = \frac{1}{2} (10 \times 1 \times 9.81) \left(8.25 - \frac{1}{3} \right) = 388.31 \text{ kN-m (+ve)}$$

(c) Due to uplift force :

$$\text{Moment} = 404.66 \times 2/3 \times 8.25 = 2225.63 \text{ kN-m (-ve)}$$

(d) Due to horizontal water pressure

$$= 490.5 \times 10/3 = 1635 \text{ kN-m (-ve)}$$

$$\therefore \Sigma M = 5278 + 388.31 - 2225.63 - 1635 = +5666.31 - 3860.63 = +1805.68 \text{ kN-m}$$

Calculation of factor of safety

Factor of safety against overturning

$$= \frac{M_R}{M_O} = \frac{(+M)}{(-M)} = \frac{5666.31}{3860.63} = 1.47 < 1.5 \text{ Unsafe.}$$

Factor of safety against sliding

$$= \frac{\mu \Sigma V}{\Sigma H} \quad (\text{Assuming } \mu = 0.75) = \frac{0.75 \times 680.39}{490.5} = 1.04 > 1$$

$$\text{Shear friction factor} = \frac{\mu \Sigma V + b s}{\Sigma H} = \frac{0.75 \times 680.39 + 8.25 \times 1400}{490.5} = 24.8$$

Calculation of stresses

The resultant acts at a distance $\bar{x}$ from toe,

$$\text{where } \bar{x} = \frac{\Sigma M}{\Sigma V} = \frac{1805.68}{680.39} = 2.65 \text{ m.}$$

$$\text{Its distance from centre is } e = \frac{b}{2} - \bar{x} = \frac{8.25}{2} - 2.65 = 1.475 \text{ m}$$

Compressive stress at toe is

$$p_a = \frac{\Sigma V}{b} \left[1 + \frac{6e}{b} \right] = \frac{680.39}{8.25} \left[1 + \frac{6(1.475)}{8.25} \right] = 187.8 \text{ kN/m}^2 \text{ (comp.)}$$

Compressive stress at heel

$$= \frac{\Sigma V}{b} \left[1 - \frac{6e}{b} \right] = \frac{680.39}{8.25} \left[1 - \frac{6(1.475)}{8.25} \right] = -2.9 \text{ kN/m}^2 \text{ (i.e. tension)}$$

$$\text{From Fig. 8.36, } \tan \alpha = \frac{1}{10}, \quad \tan \beta = \frac{6.25}{10} \\ \sec \alpha = \sqrt{1.01}, \quad \sec \beta = \sqrt{1.391}$$

$$\text{Principal stress at toe of dam} = p_a \sec^2 \beta = 167.8 \times 1.391 = 233.4 \text{ kN/m}^2.$$

$$\text{Principal stress at heel} = p_a \sec^2 \alpha - p \tan^2 \alpha = -2.91 \times 1.01 - p (0.01)$$

$$= -2.91 \times 1.01 - 9.81 \times 10 \times 0.01 = -3.9 \text{ kN/m}^2.$$

$$\text{Shear stress at toe, } \tau = p_n \tan \beta = \frac{167.8 \times 6.25}{10} = 104.9 \text{ kN/m}^2$$

$$\text{Shear stress at heel} = -(p_n - p) \tan \alpha = -(-2.91 - 98.1) \times \frac{1}{10} = 10.1 \text{ kN/m}^2.$$

Example 8.15. A concrete dam section is shown in Fig. 8.41. Check the stability of the section and determine the value of sliding factor and shear factor. Find out the magnitude and direction of principal stresses, normal stress and shear stress at toe and heel. Analysis of dam section is to be carried out under the following conditions:

(i) Effect of only horizontal earthquake is to be considered by taking uniform value of $a_h = 0.1$ and by taking $C_m = 0.735$.

(ii) Reservoir full condition.

Take unit weight of concrete = 23.5 kN/m^3 and unit shear for concrete = 1400 kN/m^2 .

Uplift pressure at the base of any horizontal section varies from full reservoir level pressure at u/s to zero or tail water pressure at d/s and is considered to act over 2/3 area of section.

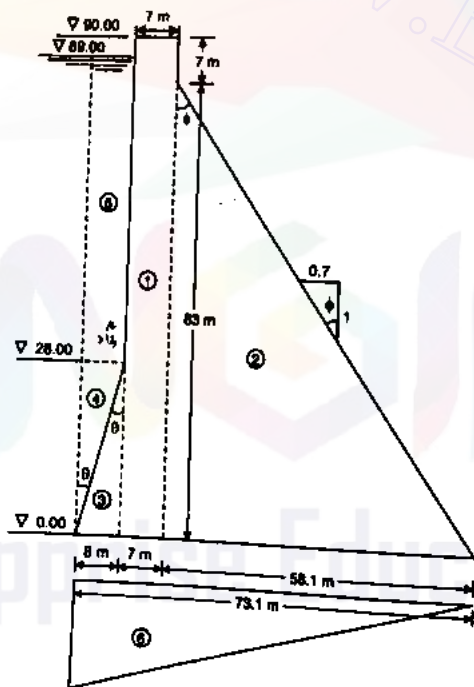


FIG. 8.41.

Solution. The analysis for stability is carried out for 1 m length of dam. The forces considered for the stability analysis are self-weight, uplift water pressure and horizontal earthquake. The analysis can be carried out as detailed in the tabular form (Table 8.7). Note the consideration of uplift since it has been multiplied by 2/3.

TABLE 8.7.

S.N.	Dimensions	Forces (kg.)		Lever Arm (m)	Moment of toe (kN-m)	
		Vertical	Horizontal		+ve	-ve
1.	Self-weight of the dam 7 × 90 × 23.5	14906		61.80	911988	
2.	83 × $\frac{58.1}{2} \times 1 \times 23.5$	5666.2		38.78	2194709	
3.	8 × $\frac{28}{2} \times 1 \times 23.5$	2632		87.77	178362	
4.	Weight of super-imposed column of water 8 × $\frac{28}{2} \times 9.81$	1098.7		70.43	77387	
5.	61 × 8 × 9.81	4787.5		69.10	330891	
6.	Uplift Pressure 89 × $\frac{73.1}{2} \times \frac{2}{3} \times 9.81$	- 21274.3		48.73		1036767
7.	Water pressure $\frac{89^2}{2} \times 9.81$		36882.5	29.67		1152624
8.	Hydrodynamic pressure due to water $0.726 \times 0.735 \times 0.1 \times 9.81 \times 89^2$		4146.4			
9.	Moment due to hydrodynamic pressure of water $0.299 \times 0.735 \times 0.1 \times 9.81 \times 89^3$					151984
10.	Inertial load because of horizontal acceleration Due to 1st rectangle	1490.6		45		66623
11.	Due to 2nd triangle	5666.2		38.78		156760
12.	Due to 3rd triangle	263.2		29.67		2467
	Sum	58710.7	50408.6		3688247	2567220
					$\Sigma M = 1126027$	

Calculation of stresses

From Fig. 8.41, $\tan \phi = 0.7$, $\sec \phi = (\sqrt{1 + 0.49})/1 = \sqrt{1.49} = 1.2207$

and $\tan \theta = 8/28 = 2/7$

Distance where resultant acts from toe is $\bar{x} = \frac{\Sigma M}{\Sigma V} = \frac{1126027}{58710.7} = 19.18 \text{ m.}$

Its distance e from the centre of the base is

$$e = \frac{b}{2} - \bar{x} = 36.55 - 19.18 = 17.37 \text{ m.}$$

Normal stress at toe $p_n = \frac{\Sigma V}{b} \left[1 + \frac{6e}{b} \right] = \frac{58710.7}{73.1} \left[1 + \frac{6 \times 17.37}{73.1} \right]$
 $= 1948 \text{ kN/m}^2 (\pm 1.95 \text{ N/mm}^2)$

Normal stress at heel $p_n = \frac{\Sigma V}{b} \left(1 - \frac{6e}{b} \right) = \frac{58710.7}{73.1} \left(1 - \frac{6 \times 17.37}{73.1} \right)$
 $= -342 \text{ kN/m}^2, \text{ i.e. tensile.}$

Principal stress at toe (there is no tail water) :

$$\sigma_1 = p_n \sec^2 \phi = 1948 \times 1.49 = 2903 \text{ kN/m}^2 (2.9 \text{ N/mm}^2)$$

Its direction is parallel to downstream face.

Principal stress of heel is : $\sigma_1 = p_n \sec^2 \theta - (p + p_e) \tan^2 \theta$

where p = water pressure at base = $89 \times 9.81 = 873.1 \text{ kN/m}^2$

p_e = hydrodynamic pressure at base = $C_m \alpha_h w h$

$C_m = 0.735$

$\alpha_h = 0.1$

$\therefore p_e = 0.735 \times 0.1 \times 9.81 \times 89 = 64.2 \text{ kN/m}^2$

$$\sigma_1 = -342 [1 + 4/49] - [873.1 + 64.2] \times 4/49 = -446 \text{ kN/m}^2.$$

Hence, the dam becomes unsafe as the concrete cannot withstand tensile stress more than $0.02 f_c = 0.02 \times 15 = 0.3 \text{ N/mm}^2 = 300 \text{ kN/m}^2$ for M15 concrete for load combination E (Table 8.6). Its direction is perpendicular to upstream face.

Shear stress at toe $\tau = p_n \tan \phi = 1948 \times 0.7 = 1364 \text{ kN/m}^2$

Shear stress at heel

$$\tau = -[p_n - (p + p_e)] \tan \theta = -[342 - (873.1 + 64.2)] \times 2/7 = 170 \text{ kN/m}^2.$$

Calculation of factor of safety

(a) Factor of safety against sliding as per IS 6512-1972 : Taking $\mu = 0.75$,

$$\text{F.S.S.} = \frac{\mu \Sigma V}{\Sigma H} = \frac{0.75 \times 58710.7}{50408.8} = 0.874 < 1.5 \text{ Unsafe.}$$

(b) Shear friction factor as per IS 6512-1972 :

$$\text{S.F.F.} = \frac{\mu \Sigma V + bc}{\Sigma H} = \frac{0.75 \times 58710.7 + 73.1 \times 1 \times 1400}{50408.8}$$

$= 2.91 < 6$; hence unsafe.

(c) Factor of safety against sliding as per IS 6512-1984 :

For load condition E, taking $F_\phi = 1.2$ and $F_c = 2.4$ (Table 8.5), we get

$$F = \frac{\frac{\mu \Sigma V}{F_\phi} + \frac{cb}{F_c}}{\Sigma H} = \frac{\frac{0.75 \times 58710.7}{1.2} + \frac{1400 \times 73.1}{2.4}}{50408.8} = 1.57 > 1 \text{ (Safe)}$$

(d) Factor of safety against overturning

$$= \frac{\Sigma M_R}{\Sigma M_O} = \frac{(+M)}{(-M)} = \frac{3693247}{2567220} = 1.44 < 1.5$$

The dam section is therefore, unsafe for the present loading conditions.

Example 8.16. (a) Check the stability of the gravity dam section shown in Fig. 8.42 for reservoir empty and reservoir full conditions. Neglect seismic forces, assume reasonable values for uplift and a line of drain holes 6.5 m downstream of the upstream face. For the purpose of this check, assume water level at the top of the dam and no tail water. Also find principal and shear stresses at the toe and heel of the dam. Take unit weight of concrete as 24 kN/m^3 , shear strength of concrete as 1400 kN/m^2 and $\mu = 0.7$.

(b) Check the stability of the dam section by considering seismic forces also, assuming a value of horizontal acceleration coefficient $\alpha_h = 0.1$ and vertical acceleration coefficient $\alpha_v = 0.08$, both uniform over the depth.

Solution

(a) Stability analysis neglecting seismic forces :

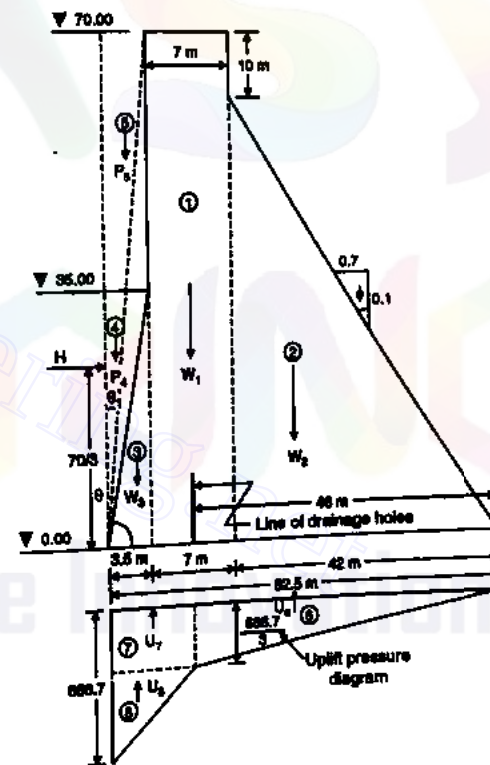


FIG. 8.42

Case I. Reservoir Empty Condition

When reservoir is empty, only self weight of the dam will be acting as force. Other forces namely water pressure and uplift will be zero. The resulting force ΣV_1 and resulting moment ΣM_1 for this case has been worked out in Table 8.8.

Position of resultant from toe :

$$\bar{x} = \frac{\Sigma M_1}{\Sigma V_1} = \frac{1455545}{43470} = 33.48 \text{ m}$$

Its distance from centre is

$$e = b/2 - \bar{x} = 52.5/2 - 33.48 = -7.23 \text{ m}$$

(i.e., the resultant falls to the left of the centre)

Normal compressive stress at toe :

$$p_n = \frac{\Sigma V_1}{b} \left[1 + \frac{6e}{b} \right] = \frac{43470}{52.5} \left[1 + \frac{6(-7.23)}{52.5} \right] = 143.8 \text{ kN/m}^2 \text{ compressive}$$

Normal compressive stress at heel :

$$p_n = \frac{\Sigma V_1}{b} \left[1 - \frac{6e}{b} \right] = \frac{43470}{52.5} \left[1 - \frac{6(-7.23)}{52.5} \right] = 1512.2 \text{ kN/m}^2$$

TABLE 8.8. STABILITY CALCULATIONS NEGLECTING SEISMIC FORCES

S. No.	Item	Description	Forces		Lever arm (m)	Movement at toe		Remarks
			Vertical	Horizontal		+ve	-ve (kN-m)	
1.	W_1	Self weight of dam $70 \times 7 \times 1 \times 24$	11760		45.5	535060		Reservoir Empty Condition
2.	W_2	$60 \times 42 \times \frac{1}{2} \times 24$	30240		28	846720		
3.	W_3	$35 \times 3.5 \times \frac{1}{2} \times 24$	1470		50.17	73745		
		Sum	$\Sigma V_1 = 43470$		Sum	+1455545	0.00	
4.	P_1	Water Pressure $3.5 \times 35 \times \frac{1}{2} \times 9.81$	600.9		51.33	30844		Reservoir full with no uplift
5.	P_2	$3.5 \times 35 \times 9.81$	1201.7		50.75	60888		
	H	$\frac{1}{2} \times 70^2 \times 9.81$		24034.5	70/3		560806	
		Sum	$\Sigma V_2 = 45272.6$	24034.5	Sum	+1547377	-560806	
6.	U_6	Uplift forces $686.7/3 \times 46 \times \frac{1}{2} \times 1$	-5264.7		30.67		161451	Reservoir full with uplift
7.	U_7	$686.7/3 \times 6.5 \times 1$	-1487.9		-48.25		73277	
8.	U_8	$2 \times 686.7/3 \times 6.5 \times \frac{1}{2} \times 1$	-1487.9		50.33		74888	
		Sum	$\Sigma V_3 = 37032.1$	24034.5	Sum	+1547377	-870421	
						$\Sigma M_2 = +876956$		

Principal stress at toe :

$$\sigma_1 = p_n \sec^2 \phi \quad \text{where } \tan \phi = 0.7, \text{ and } \sec^2 \phi = 1.49$$

$$\sigma_1 = 143.8 \times 1.49 = 214.3 \text{ kN/m}^2$$

Principal stress at heel :

$$\sigma = p_n \sec^2 \theta \quad \text{where } \tan \theta = 3.5/35 = 0.1 \text{ and } \sec^2 \theta = 1.01$$

$$\sigma = 1512.2 \times 1.01 = 1527.3 \text{ kN/m}^2$$

Shear stress at toe :

$$\tau = p_n \tan \phi = 143.8 \times 0.7 = 100.7 \text{ kN/m}^2$$

Shear stress at heel :

$$\tau = p_n \tan \theta = 1512.2 \times 0.1 = 151.2 \text{ kN/m}^2$$

Note that there cannot be any sliding or overturning when reservoir is empty.

II. **Reservoir full with no uplift** : Sometimes values of stresses at toe and heel are worked out without considering uplift as the vertical forces are maximum in this case.

Calculation of stresses : $\tan \theta = 0.1$; $\sec^2 \theta = 1.01$; $\tan \phi = 0.7$; $\sec^2 \phi = 1.49$

The values of vertical forces ΣV_2 and moments ΣM_2 have been worked out in Table 8.8 where the ΣV_2 and ΣM_2 represent the sum of vertical forces and sum of moments of all forces when the reservoir is full but when uplift is not acting.

Position of the resultant from the toe is

$$\bar{x} = \frac{\Sigma M_2}{\Sigma V_2} = \frac{986572}{45272.6} = 21.79 \text{ m}$$

Position of the resultant from the centre of the base is

$$e = b/2 - \bar{x} = 26.25 - 21.79 = 4.46 \text{ m}$$

Normal compressive stress at toe

$$p_n = \frac{\Sigma V_2}{b} \left(1 + \frac{6e}{b} \right) = \frac{45272.6}{52.5} \left(1 + \frac{6 \times 4.46}{52.5} \right) = 1301.9 \text{ kN/m}^2 (\pm 1.3 \text{ N/mm}^2)$$

Normal compressive stress at heel

$$p_n = \frac{\Sigma V_2}{b} \left(1 - \frac{6e}{b} \right) = \frac{45272.6}{52.5} \left(1 - \frac{6 \times 4.46}{52.5} \right) = 422.8 \text{ kN/m}^2 (\pm 0.42 \text{ N/mm}^2)$$

Principal stress at toe is

$$\sigma_1 = p_n \sec^2 \phi = 1301.9 \times 1.49 = 1939.8 \text{ kN/m}^2 (\pm 1.94 \text{ N/mm}^2)$$

Principal stress at heel is

$$\sigma = p_n \sec^2 \theta - p \tan^2 \theta = 422.8 \times 1.01 - 9.81 \times 70 \times 0.01 = 420.2 \text{ kN/m}^2 (\pm 0.42 \text{ N/mm}^2)$$

Note. This is the value of minor principal stress at the heel. The value of major principal stress at heel is equal to $9.81 \times 70 = 686.7 \text{ kN/m}^2 (\pm 0.69 \text{ N/mm}^2)$.

Shear stress at toe, $\tau = p_n \tan \phi = 1301.9 \times 0.7 = 911.3 \text{ kN/m}^2 (\pm 0.91 \text{ N/mm}^2)$.

Shear stress at heel is $\tau = -(p_n - p) \tan \theta = -(422.8 - 686.7) \times 0.1 = 26.4 \text{ kN/m}^2$.

The factor of safety against sliding and overturning should be found out only when reservoir is full and full uplift acts, since the condition of overturning and sliding will be more critical in that case.

Case III. Reservoir full with uplift : Values of vertical forces ΣV_3 and moment ΣM_3 have been worked out in Table 8.8.

Position of resultant from the toe is

$$\bar{x} = \frac{\Sigma M_3}{\Sigma V_3} = \frac{676956}{37032.1} = 18.27 \text{ m}$$

Position of resultant from the centre of the base is

$$e = \frac{b}{2} - \bar{x} = 26.25 - 18.27 \text{ m} = 7.98 \text{ m}.$$

Normal compressive stress at toe is

$$p_n = \frac{\Sigma V_3}{b} \left[1 + \frac{6e}{b} \right] = \frac{37032.1}{52.5} \left[1 + \frac{6 \times 7.98}{52.5} \right] = 1348.7 \text{ kN/m}^2 (\pm 1.35 \text{ N/mm}^2).$$

This compressive stress is greater than the one obtained for condition II ; hence this is the worst case.

Normal compressive stress at heel

$$p_n = \frac{37032.1}{52.5} \left[1 - \frac{6 \times 7.98}{52.5} \right] = 62.1 \text{ kN/m}^2 (\text{Comp.}).$$

Principal stress at toe

$$\sigma_1 = p_n \sec^2 \phi = 1348.7 \times 1.49 = 2010 \text{ kN/m}^2 (\pm 2.01 \text{ N/mm}^2).$$

Shear stress at heel

$$\sigma_1 = p_n \sec^2 \theta - p \tan^2 \theta = 62.1 \times 1.01 - 9.81 \times 70 \times 0.01 = 56 \text{ kN/m}^2.$$

Shear stress at toe

$$\tau = p_n \tan \phi = 1348.7 \times 0.7 = 944 \text{ kN/m}^2 (\pm 0.94 \text{ N/mm}^2).$$

Shear stress at heel

$$\tau = -(p_n - p) \tan \theta = -(62.1 - 9.81 \times 70) \times 0.1 = 62 \text{ kN/m}^2.$$

Calculation of factor of safety

$$\text{Factor of safety against overturning} = \frac{\Sigma M(+)}{\Sigma M(-)} = \frac{1547377}{870421} = 1.78 > 1.5$$

$$\text{Factor of safety against sliding} = \frac{\mu \Sigma V_3}{\Sigma H} = \frac{0.70 \times 37032.1}{24034.5} = 1.079 < 2.$$

$$\text{Shear friction factor S.F.F.} = \frac{\mu \Sigma V_3 + b \cdot c}{\Sigma H} = \frac{0.70 \times 37032.1 + 52.5 \times 1400}{24034.5} = 4.14 > 4$$

Safety against sliding according to IS 6512-1984 :

Taking $F_g = 1.5$ and $F_c = 3.6$ for load combination B,

$$F = \frac{\mu \Sigma V + cb}{\Sigma H} = \frac{0.7 \times 37032.1 + 1400 \times 52.5}{24034.5} = 1.588 > 1$$

Hence safe.

(b) **Stability check of dam by considering seismic forces.**

For worst condition consider that :

(a) Horizontal earthquake acceleration acts upstream.

(b) Vertical earthquake acceleration acts downwards.

Hydrodynamic pressure due to water caused by earthquake can be found out from Zanger's formula. Since the slope is upto middle depth, approximate value of θ can be found out by joining heel to the upstream edge,

$$\tan \theta_1 = \frac{3.5}{70} = 0.05 \quad \text{or} \quad \theta_1 = 2.862^\circ$$

$$C_m = 0.735 \times \left(1 - \frac{2.862^\circ}{90^\circ} \right) = 0.712$$

At base $C = C_m$

Therefore, $p_e = C_m \alpha_a w h = 0.712 \times 0.1 \times 9.81 \times 70 = 48.89 \text{ kN/m}^2$

Total pressure $P_e = 0.726 p_e h = 0.726 \times 48.89 \times 70 = 2485 \text{ kN}$

Moment due to this force at base,

$$M_e = 0.299 p_e h^3 = 0.299 \times 48.89 \times 4900 = 71629 \text{ kN-m}.$$

Calculation of forces and moments due to inertial earthquake force is done in the tabular form (Table 8.9). This Table is to be prepared in continuation of the previous Table (Table 8.8) made without considering earthquake forces.

Values of vertical forces ΣV_4 and moment ΣM_4 have been worked out in the Table 8.9.

TABLE 8.9. ADDITIONAL FORCES AND THEIR MOMENTS DUE TO EARTHQUAKE

S. No.	Item	Description	Forces (kN)		Lower arm (m)	Moment of toe (kN-m)		Remarks
			Vertical	Horizontal		+	-	
1.	ΣV_1	$\Sigma V_1 \times \alpha_0$	-3477.6				116444	Inertial force acting at C.G. is considered to be acting upwards
2.	W_1	$W_1 \times \alpha_0$		1178	55		-11160	
3.	W_2	$W_2 \times \alpha_0$		3094	30		-90490	
4.	W_3	$W_3 \times \alpha_0$		147	35/3		1715	
5.	W_4	$W_4 \times \alpha_0$		2485			71629	
6.	P_e	Hydrodynamic pressure					$\Sigma M = -291428$	
		Total	$\Sigma V = -3477.6$	$\Sigma H = 6839$				
		Sum of forces and moments from previous table	$\Sigma V_3 = 37032.1$	$H = 24034.5$		+1547377	-870421	
						+1547577	-1161849	
		Sum	$\Sigma V_4 = 33554.5$	$\Sigma H = 30866.5$			$\Sigma M_4 = 388528$	

Position of resultant from toe is

$$\bar{x} = \frac{\Sigma M_A}{\Sigma V_A} = \frac{385528}{39554.5} = 11.48 \text{ m}$$

Hence the resultant does not lie within the middle third. Its distance from centre is

$$e = \frac{b}{2} - \bar{x} = 26.25 - 11.48 = 14.77 \text{ m}$$

i.e., the resultant falls to the right of centre.

Normal compressive stress at toe

$$p_A = \frac{\Sigma V_A}{b} \left[1 + \frac{6e}{b} \right] = \frac{39554.5}{52.5} \left[1 + \frac{6 \times 14.77}{52.5} \right] \\ = 1718 \text{ kN/m}^2 \text{ (}\triangleq 1.72 \text{ N/mm}^2\text{) compressive.}$$

Normal compressive stress at heel

$$p_n = \frac{\Sigma V_A}{b} \left[1 - \frac{6e}{b} \right] = \frac{39554.5}{52.5} \left[1 - \frac{6 \times 14.77}{52.5} \right] \\ = -440 \text{ kN/m}^2 \text{ (}\triangleq 0.44 \text{ N/mm}^2\text{) tensile.}$$

Since the resultant does not fall within middle third and the value of tensile stress is substantial ($> 0.2 f_c$ for M15 concrete) the dam is *unsafe* under seismic condition.

Principal stress at toe is

$$p_n \sec^2 \phi = 1718 \times 1.49 = 2560 \text{ kN/m}^2 \text{ (}\triangleq 2.56 \text{ N/mm}^2\text{).}$$

Principal stress at heel is

$$\sigma_1 = p_n \sec^2 \theta - (p + p_n) \tan^2 \theta = -440 \times 1.01 - (9.81 \times 70 + 48.89) \times 0.01 \\ = -452 \text{ kN/m}^2 \text{ Hence unsafe.}$$

Shear stress at toe

$$\tau = p_n \tan \phi = 1718 \times 0.7 = 1203 \text{ kN/m}^2.$$

Shear stress at heel

$$\tau = -[p_n - (p + p_n)] \tan \theta = -[-440 - (9.81 \times 70 + 48.89)] \times 0.1 \\ = 1176 \text{ kN/m}^2.$$

Calculations for factor of safety

Factor of safety against overturning

$$= \frac{\Sigma M(+)}{\Sigma M(-)} = \frac{1547377}{1161849} = 1.33 < 1.5. \text{ Hence unsafe.}$$

Factor of safety against sliding

$$= \frac{\mu \Sigma V_A}{\Sigma H} = \frac{0.70 \times 39554.5}{30866.5}$$

$$= 0.761 < 1.5 \text{ Hence unsafe.}$$

Shear friction factor

$$\text{S.F.F.} = \frac{\mu \Sigma V_A + b.c}{\Sigma H} = \frac{0.70 \times 39554.5 + 52.5 \times 1400}{30866.5} = 3.14 > 3.0.$$

Safety against sliding as per IS 6512-1984 :

Taking $F_o = 1.2$ and $F_c = 2.7$ for loading combination B,

$$F = \frac{\frac{\mu \Sigma V}{F_o} + \frac{cb}{F_c}}{\Sigma H} = \frac{\frac{0.7 \times 39554.5}{1.2} + \frac{1400 \times 52.5}{2.7}}{30866.5} = 1.52 > 1 \text{ Hence Safe.}$$

Example 8.17. Design the practical profile of a gravity dam of stone masonry, given the following data :

R.L. of base of dam = 1450 m.

R.L. of F.R.L. = 1480.5 m

Specific gravity of the masonry = 2.4

Safe compressive stress for masonry = 1200 kN/m².

Height of waves = 1 m.

Solution. $h_w = 1 \text{ m}$

Free board = 1.5 $h_w = 1.5 \text{ m}$

$\therefore$ R.L. of top of dam = 1480.5 + 1.5 = 1482 m

Hence height of dam = 1482 - 1450 = 32 m

$$\text{Limiting height of dam} = \frac{f}{w(p+1)} = \frac{1200}{9.81(2.4+1)} = 35.98 \text{ m}$$

Hence the dam is a low gravity dam.

The practical profile of the dam can be designed with respect to recommendations of § 8.17 (Fig. 8.19).

Depth of water $h = 1480.5 - 1450 = 30.5 \text{ m}$.

Top width a can be determined on the following criteria :

(i) $a = 14\%$ of H for economy

$= 0.14 \times 32 \triangleq 4.5 \text{ m}$

(ii) $a =$ width of roadway, if any.

However, keep $a = 4.5 \text{ m}$.

Base width of elementary profile

$$= \frac{h}{\sqrt{\rho}} = \frac{30.5}{\sqrt{2.4}} = 19.7 \text{ m.}$$

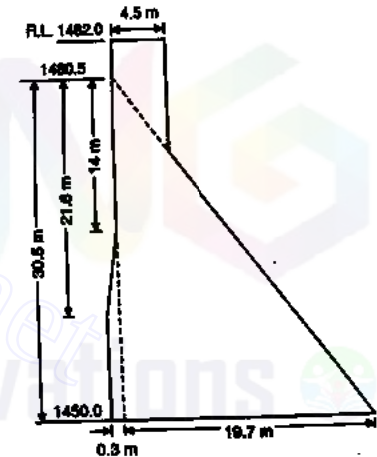


FIG. 8.43

$$\text{Upstream offset} = \frac{a}{16} = \frac{4.5}{16} \triangleq 0.3 \text{ m.}$$

$$\text{Total base width} = 19.7 + 0.3 = 20 \text{ m.}$$

Distance upto which u/s slope is vertical from the u/s water level
 $= 2a\sqrt{p} = 2 \times 4.5 \sqrt{2.4} \approx 14 \text{ m.}$

Fig. 8.43 shows the practical profile of the dam. The stability of this section can now be tested.

8.24. EXAMPLES FROM COMPETITIVE EXAMINATIONS

Example 8.18. The following Fig. (Fig. 8.44 a) gives the profile of a gravity dam with reservoir level as shown. If the coefficient of friction is 0.75, is the dam safe against sliding? Take weight density of concrete = 2.4 tonnes/m³.

(Engg. Services Exam. 1997)

Solution : Let us neglect the uplift pressure. The forces due horizontal water pressure (P) and the self weight of the dam are shown marked in Fig. 8.44 (b)

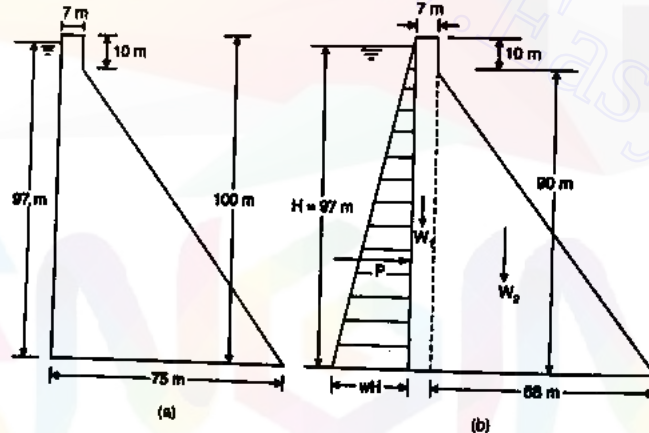


FIG. 8.44

$$P = \frac{wH^2}{2} = \frac{1000}{2} (97)^2 \times 10^{-3} = 4704.5 \text{ tonnes/m}$$

$$\text{Self weight } W_1 = 7 \times 100 \times 2.4 = 1680 \text{ tonnes/m}$$

$$\text{Self weight } W_2 = \frac{1}{2} \times 68 \times 90 \times 2.4 = 7344 \text{ tonnes/m}$$

$$\text{Total self weight} = W = W_1 + W_2 = 1680 + 7344 = 9024 \text{ tonnes/m}$$

$$\therefore \text{F.S. against sliding} = \frac{\mu W}{P} = \frac{0.75 \times 9024}{4704.5} = 1.439 > 1$$

Hence the dam is safe against sliding.

Example 8.19. A masonry dam 10 m high is trapezoidal in section with a top width of 1 m and bottom width of 8.25 m. The face exposed to water has a batter of 1 : 10. Calculate

- Factor of safety against overturning
- Factor of safety against sliding.
- Shear friction factor.

Assume coefficient of friction as 0.75, unit weight of masonry as 2240 kg/m³, and permissible stress of joint = 14 kg/cm². Based on the results, give your remark.

(Engg. Services Exam. 1995)

Solution : This problem was set from Example 8.14 of this book. For solution, see example 8.14.

Example 8.20. Fig. 8.45 (a) shows the section of a gravity dam (non-overflow portion) built of concrete. Calculate the maximum vertical stress at the heel and toe of the dam. Assume weight of concrete as 23.5 kN/m³. Neglect earthquake effects.

(Engg. Services Exam. 2000)

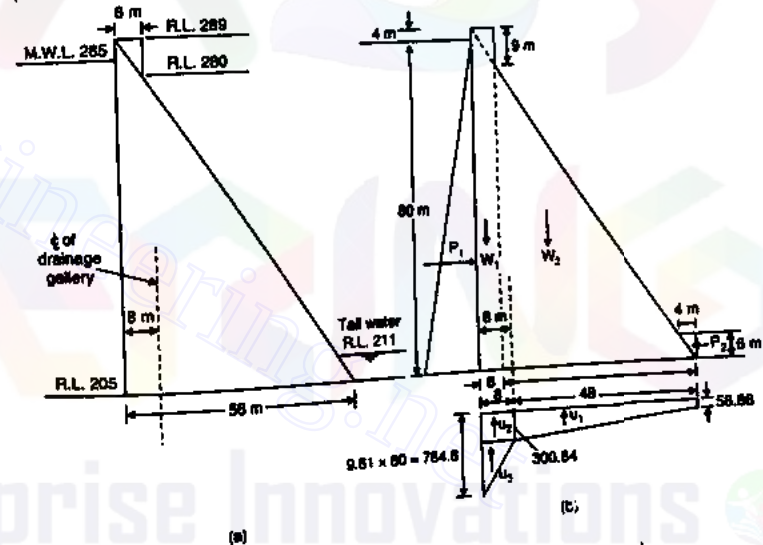


FIG. 8.45

Solution : Various forces acting on the dam are shown marked in Fig. 8.45 (a). The computations are arranged in Tabular form below (Table 8.10).

TABLE 8.10

Designation	Dimensions	Force (kN)		L.A. (m)	Moment about toe (kN-m)	
		Vertical	Horizontal		R.C. +ve	Q/A -ve
W_1	Self weight of dam $8 \times 84 \times 1 \times 23.5$	11844		53.0	627732	
W_2	$\frac{1}{2} \times 50 \times 75 \times 23.5$	44062.5		33.33	1468760	
W_3	Wt. of water on dis face $\frac{1}{2} \times 4 \times 6 \times 9.81$	117.7		1.33	157	
U_1	Uplift pressure $\frac{1}{2} (58.86 + 300.84) \times 48$	-8632.8		29.98		253847
U_2	300.84×8	-2408.7		52		126149
U_3	$\frac{1}{2} (784.8 - 300.84) \times 8$	-1936.8		53.33		103246
P_1	Horizontal water pressure $9.81 \times 80^2/2$		31392 ($\rightarrow$)	26.67		837120
P_2	$9.81 \times 6^2/2$		176.6 ($\leftarrow$)	2	353	
	Sum	43048.9	31213.4 ($\rightarrow$)		2096993	151961
					777831	

$$\bar{x} = \frac{\Sigma M}{\Sigma V} = \frac{777831}{43048.9} = 18.07 \text{ m} ; \quad e = \frac{b}{2} - \bar{x} = \frac{56}{2} - 18.07 = 9.93 \text{ m}$$

$$p_{n, \max} = \frac{\Sigma V}{b} \left[1 \pm \frac{6e}{b} \right] = \frac{43048.9}{56} \left[1 \pm \frac{6 \times 9.93}{56} \right] = 768.73 \left[1 \pm 1.064 \right]$$

$$p_{n, \max} \text{ at toe} = 1586.6 \text{ kN/m}^2$$

$$p_{n, \min} \text{ at heel} = -48.2 \text{ kN/m}^2 \text{ (i.e. tension)}$$

PROBLEMS

1. What do you understand by gravity dam? Explain various forces that act on a gravity dam.
2. Discuss in brief the U.S.B.R. recommendations for determining uplift pressure under the base of a dam, provided with a drainage gallery.
3. Explain how do you account for earthquake effects in the design of a gravity dam.
4. Discuss in brief various modes of failure of a gravity dam.
5. Explain the method of determining principal and shear stresses in a gravity dam.
6. What do you understand by the elementary profile of a gravity dam? Derive expressions for determining base width of such a dam based on (i) stress criterion, (ii) sliding criterion. Also derive expression for normal, principal and shear stresses at the base of such a dam.
7. Give a practical profile of a low gravity dam.

8. Distinguish clearly between a low gravity dam and high gravity dam. Derive the expression used for such a distinction.

Determine the critical height of a low gravity dam of concrete, taking the specific gravity of concrete as 2.40 and allowable compressive stress as 3340 kN/m^2 . [Ans. 100 m]

9. Give a practical profile for a high gravity dam as suggested by G. Molesworth.
10. Explain the step by step method of designing a high gravity dam.
11. What do you understand by galleries and shafts and why are they provided in gravity dams?
12. Explain with the help of diagrams various joints and water seals provided in gravity dams.
13. How do you control the cracking in a concrete gravity dam?
14. Design the practical profile of a gravity dam of stone masonry, given the following data:

R.L. of base of dam = 1250 m

R.L. of H.F.L. = 1280 m

Specific gravity of masonry = 2.4

Safe compressive stress for masonry of dam = 1200 kN/m^2 .

15. Check the stability (at section AB) of the concrete gravity dam section shown in Fig. 8.46. Assume any other data not given. Neglect gallery in calculations.

16. The cross-section of low gravity dam is showing Fig 8.47. Assuming the reservoir to be full, determine: (1) normal stress, principal stress and shear stress at the toe, (2) Factor of safety against sliding and (3) shear friction factor at level A-B. Full uplift pressure is to be accounted according to the U.S.B.R. recommendations. Neglect earthquake pressure, wave pressure and silt pressure.

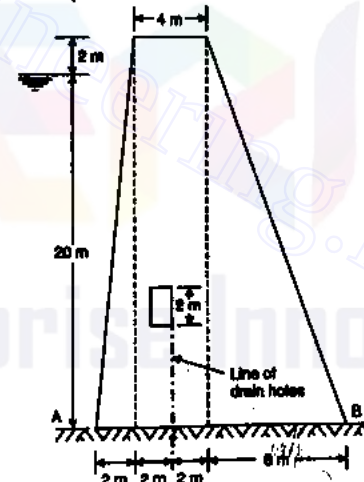


FIG. 8.46

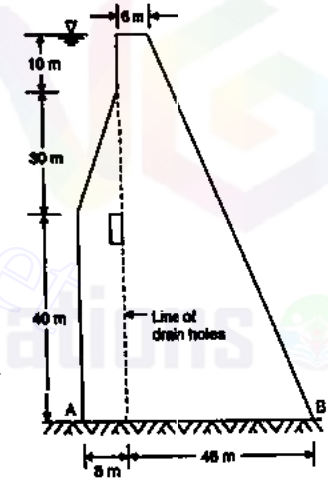


FIG. 8.47

Dams - III : Arch and Buttress Dams

9.1. TYPES OF ARCH DAMS

An arch dam is a dam curved in plan and carries a major part of its water load horizontally to the abutments by arch action. The part of the water load depends primarily upon the amount of curvature. The balance of the water load is transferred to the foundation by cantilever action. The thrust, developed by the water load carried by arch action, essentially requires strong side walls of the canyon to resist the arch forces. Many early arch dams were built of rubble, ashlar etc. However, practically all arch dams constructed during recent years have been built of concrete. The famous Hoover dam of U.S.A., constructed in 1936, is an arch dam of constant radius type having the following dimensions : Height 222 m (726 ft); Length 380 m (1244 ft); Top width 13.7 m (45 ft) and Base thickness 201 m (660 ft).

The arch dams may be divided into the following three classes:

1. Constant radius dam.
2. Variable radius dam, and
3. Constant angle dam.

1. **Constant radius arch dam** (Fig. 9.1). A constant radius arch dam is the one in which the radius to some prominent feature (such as upstream face, or the

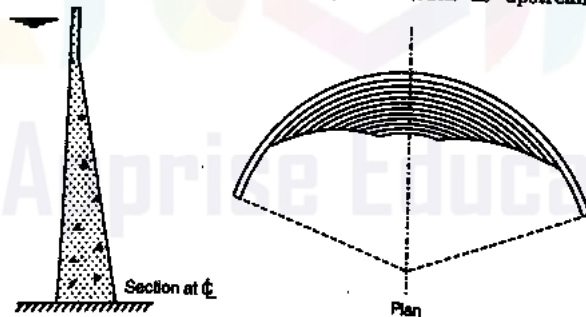


FIG. 9.1. CONSTANT RADIUS ARCH DAM.
(434)

DAMS-III : ARCH AND BUTTRESS DAMS

downstream face, or the arch axis) is constant. In most cases, however, the constant radius refers to the upstream side. In a constant radius arch dam, the arch centres for upstream face, downstream face and the centre line of the horizontal arch rings at various elevations lie on a straight vertical line that passes through the centre of the horizontal arch ring of the upstream face at the crest. Hence this type of dam is also sometimes known as the constant centre arch dam. In most of the simple types, the upstream face is vertical (i.e., its radius is constant at all elevations), and the thickness is proportional to the height. This makes the central angles of the arch rings of the intrados vary at various elevations, the maximum being at the top of the dam and a certain minimum at the bottom of the dam, as shown in Fig. 9.1.

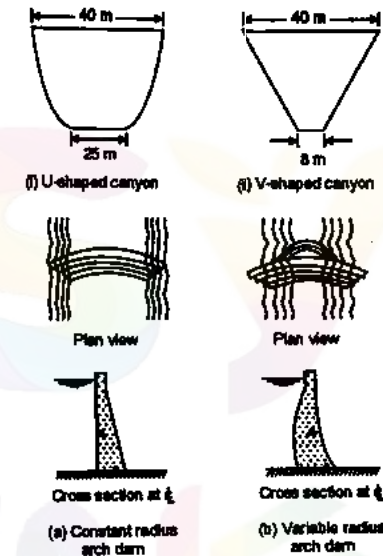


FIG. 9.2 CONSTANT AND VARIABLE RADIUS ARCH DAMS.

A constant radius arch dam is particularly adopted to U-shaped canyons [Fig. 9.2 (a)] as cantilever action will carry a large proportion of the load at lower levels. The constant radius arch dam is less economical than the constant angle arch dam, and the latter is constructed in preference to constant radius arch dam. However, the form work for the constant radius arch dam is much simpler to construct.

2. **Variable radius arch dam** (Fig. 9.3). A variable radius arch dam is the one in which the radii of the extrados curves (i.e., arch rings corresponding to upstream face) and intrados curves (i.e., arch rings corresponding to downstream face) vary at various elevations being maximum at the top and certain minimum at its bottom. This makes the central angle as large as possible so that maximum arch efficiency may be obtained at all elevations. The variable

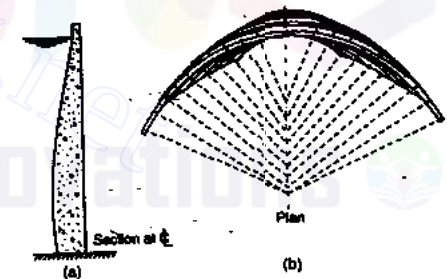


FIG. 9.3 VARIABLE RADIUS ARCH DAM.

radius arch dams have often vertical or even overhanging faces at the upstream side near the abutments and the downstream side near the crown, as shown in Fig. 9.3(b). The centres of the horizontal arch rings at various elevation do not lie on the single vertical line. Hence it is also known as variable centre arch dam.

A variable radius arch dam is often adapted to narrow V-shaped canyons [Fig. 9.2(b)] since arch action can be depended upon at the lower levels. A variable radius arch dam has greater arch efficiency, resulting in saving of concrete. Hence it is preferred in comparison to the constant radius arch dam.

3. Constant angle arch dam (Fig. 9.4). The constant angle arch dam is a special type of variable radius arch dam in which the central angle of the horizontal arch rings is of the same magnitude at all elevations, as shown in Fig. 9.4. It has been shown in § 9.3 that the volume of concrete is minimum when the central angle is $133^\circ 34'$.

A variable radius arch uses about 58 % of the concrete required in constant radius arch dam. A constant angle arch dam requires about 42.6 % of the concrete required in constant radius arch dam. Hence a constant angle arch dam is the most economical.

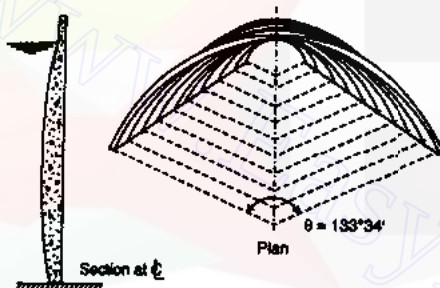


FIG. 9.4 CONSTANT ANGLE ARCH DAM.

9.2. FORCES ON AN ARCH DAM

As in a gravity dam, the various forces acting on an arch dam are :

(i) Water pressure	(ii) Self-weight of the dam
(iii) Silt pressure	(iv) Uplift pressure (negligibly small)
(v) Ice pressure	(vi) Earthquake pressure.

However, the relative importance of the above forces is not the same as in gravity dam. Due to the narrow base width of an arch dam, the uplift pressure is less important than for gravity dam. The internal stresses caused by ice pressure, temperature changes and yielding of supports (abutments) are very important in the arch dam. Ice pressure causes a continuous concentrated load along the arch element at the elevation of the ice. The internal forces caused by temperature changes move the dam upstream during the summer and downstream during the winter. Thus, the weather condition is more important in the stress analysis since it acts with the reservoir loads. The slight yielding of abutments may also cause high internal stresses in the arch.

9.3. DESIGN METHODS FOR ARCH DAMS

The following are some of the theories for the design of arch dams :

1. The thin cylinder theory
2. The elastic theory
3. The trial load analysis.

THE THIN CYLINDER THEORY

In the thin cylinder theory, the stresses in the arch are assumed to be approximately the same as in a thin cylinder of equal outside radius.

Fig. 9.5 shows the section of a thin arch, having average radius r , angle subtended at the centre of the arch equal to θ , and thickness t .

Intensity of hydrostatic pressure, p , at any depth h is given by

$$p = wh \quad \dots(i)$$

where w = unit weight of water.

This horizontal pressure (p) acts in the radial direction. The total downstream component of the horizontal force (P_H) acting along the axis of the river is given by

$$P_H = (\text{Intensity of pressure}) \times (\text{Projected area})$$

$$\text{or} \quad P_H = wh \cdot 2r \sin \theta/2 = 2whr \sin \theta/2 \quad \dots(ii)$$

If R is the reaction at each abutment, its upward component is equal to $2R \sin \theta/2$. Equating this to P_H , for equilibrium, we get

$$2R \sin \theta/2 = 2whr \sin \theta/2 \quad \dots(iii)$$

or $R = whr$
This gives an expression for the abutment reaction and is equal to the maximum compressive stress induced in the arch. If σ is the compressive stress introduced in the arch, at the abutments, we get

$$\sigma = \frac{R}{t} = \frac{whr}{t} \quad \dots(iv)$$

If f is the allowable compressive stress for the material, we get

$$t = \frac{whr}{f} \quad \dots(9.1)$$

This is the expression for the thickness of the arch.

Central angle for minimum concrete. It has been stated earlier that the constant angle arch requires minimum concrete for a fixed combination of span, loading and the permissible cylinder theory stress. Let us now find the numerical value of the central angle θ for minimum concrete.

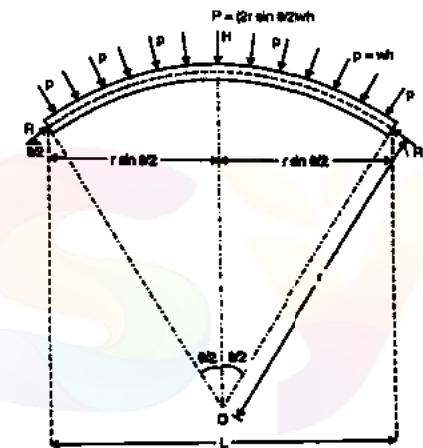


FIG. 9.5 THIN CYLINDER THEORY.

From Eq. 9.1, it is clear that

$$t \propto \text{radius of curvature } r$$

and $t \propto \text{water depth } h$.

Now volume (V) of the concrete, per unit height is given by

$$V = r \cdot \theta \cdot A; \text{ where } A = \text{cross-sectional area of arch}$$

Now $A \propto t \propto r$

or $A = k \cdot r$, where k is a constant

$$V = r \cdot \theta \cdot k \cdot r = k \cdot \theta \cdot r^2 \quad \dots(i)$$

From Fig. 9.5, we observe that

$$\sin \frac{\theta}{2} = \frac{L}{2r} \quad \text{or} \quad r = \frac{L}{2 \sin \theta/2} \quad \dots(ii)$$

Substituting this in (i), we get

$$V = k \cdot \theta \left(\frac{L}{2 \sin \theta/2} \right)^2 = \frac{k L^2}{4} \cdot \frac{\theta}{\sin^2 \theta/2}$$

For minimum volume, $\frac{dV}{d\theta} = 0$

$$\frac{dV}{d\theta} = \frac{k L^2}{4} \left[\frac{\sin^2 \frac{\theta}{2} - \theta \sin \frac{\theta}{2} \cdot \cos \frac{\theta}{2}}{\sin^4 \frac{\theta}{2}} \right] = 0$$

$$\sin^2 \frac{\theta}{2} = \theta \sin \frac{\theta}{2} \cdot \cos \frac{\theta}{2} \quad \text{or} \quad \tan \frac{\theta}{2} = \theta$$

which gives

$$\theta = 133^\circ 34'$$

This is the most economical angle. In actual practice, θ varies between 100° to 140° .

Limitations of cylinder theory

Following are the limitations of the cylinder theory :

1. The arch sections are not thin cylinders. They are also not free at abutments.
2. There is no provision for considering shearing as well as bending stress in the arch.
3. The analysis is based on only the water pressure. Temperature stress and ice thrust have not been considered.
4. Plastic flow of concrete and shrinkage in concrete have also not been accounted for.

5. Stress due to rib shortening and that due to yielding of the abutments have also not been taken into account.

9.4. THE ELASTIC THEORY

The elastic theory of design of an arch dam takes into account the additional stresses developed because of the following effects :

1. Rib shortening
2. Temperature changes
3. Shrinkage
4. Yielding of abutments.

In the case of arch, the ends are fixed. The distance between them, i.e. the span, does not change due to the water pressure acting radially from upstream side. This restriction of the change of span causes additional stresses known as *rib shortening stresses*. Similarly the change in the temperature tends to cause the change in the span and due to fixidity at the end, temperature stresses are introduced. Similarly, additional stresses are introduced if the abutments yield. The concrete on setting shrinks giving rise to further stresses.

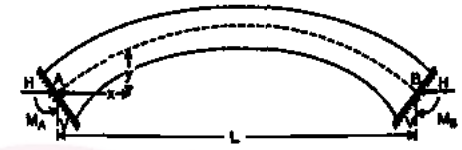


FIG 9.6. THE ELASTIC ARCH.

The cylinder formulae are based on only uniform radial pressure. However, the elastic theory can take into consideration variable loads along the length of arch, including the ice thrust etc.

The following are the formulae of elastic arches (Fig. 9.6):

$$M = \mu + M_A + (M_B - M_A) \frac{x}{L} + Hy \quad \dots(9.2)$$

$$\int \frac{M \cdot y}{EI} ds = 0 = \int \frac{\mu y}{EI} ds + M_A \int \frac{y ds}{EI} + \frac{M_B - M_A}{L} \int \frac{xy \cdot ds}{EI} + H \int \frac{y^2}{EI} ds = 0 \quad \dots(9.3)$$

$$\int \frac{M \cdot x}{EI} ds = 0 = \int \frac{\mu x}{EI} ds + M_A \int \frac{x ds}{EI} + \frac{M_B - M_A}{L} \int \frac{x^2 \cdot ds}{EI} + H \int \frac{xy}{EI} ds = 0 \quad \dots(9.4)$$

$$\text{and} \quad \int \frac{M}{EI} ds = 0 = \int \frac{\mu}{EI} ds + M_A \int \frac{ds}{EI} + \frac{M_B - M_A}{L} \int \frac{x ds}{EI} + H \int \frac{y ds}{EI} \quad \dots(9.5)$$

where M = bending moment at any point on the arch

M_A and M_B = fixed end bending moments at both the ends

μ = bending moment on a straight horizontal freely supported beam

H = horizontal thrust at the abutments.

If temperature effect is taken into account, equations 9.3, 9.4 and 9.5 are modified as below :

$$\alpha TL - \int \frac{M \cdot y}{EI} ds = 0 \quad \dots[9.3(\alpha)]$$

$$\int \frac{M \cdot x}{EI} ds = 0 \quad \dots(9.4)$$

$$\int \frac{M ds}{EI} = 0 \quad \dots(9.5)$$

and

where α is the coefficient of thermal expansion and T is the temperature variation.

9.5. THE TRIAL LOAD ANALYSIS

Both the cylinder theory as well as the elastic theory are based on the assumption that any horizontal arch ring is independent of the arch ring below and above it. This is not true. The arch rings are not independent. Due to this restriction, the actual analysis is extremely complicated. The U.S.B.R. devised the 'trial load analysis', which gives a reasonably satisfactory approximation.

In the trial load method, the dam is assumed to be made up of two system of elements : (a) horizontal arches and (b) vertical cantilevers, as shown in Fig. 9.7. Each system occupies the whole dam. The horizontal component of the water load is resisted jointly by the arch and cantilever action, and the distribution of the load between the arches and the cantilevers is determined by 'trial and error'. The structure and the loading is assumed to be divided between them in such a manner that the computed deflection at any point in the dam, considered as a point in the arch system will be identical with its computed deflections, considered as a point in the cantilever system. If computed deflection are not equal, new loads are assumed until a distribution is found which produces equal arch and cantilever deflections at all points.

The various forces on the arch dam can be supposed to be divided into primary load because of water pressure and secondary load because of other forces. The arches will carry part of primary load and total of secondary load. The cantilever will take the remaining of the primary load and total secondary load. The stresses in the dam can then be computed on the basis of the final load distribution.

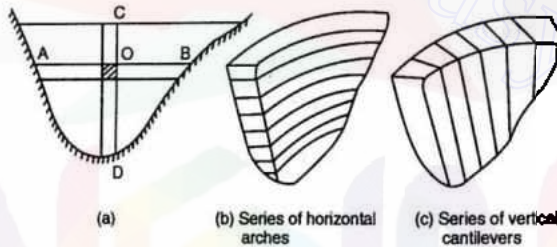


FIG. 9.7 TRIAL LOAD ANALYSIS

9.6. BUTTRESS DAMS

A buttress dam consists of the following component parts:

(1) **Sloping membrane.** The sloping membrane or upstream deck supports the water and transmits the water load to a series of buttresses at right angles to the axis of the dam.

(2) **Buttresses.** The buttresses are constructed at right angles to the axis of the dam, at certain intervals. They support the sloping membrane and transfer the load to the foundation. The function of the buttress is similar to that of a counterfort of a retaining wall.

(3) **Mat foundation footing.** The footings distribute the pressure to the soil.

(4) **Lateral Braces.** They are constructed at right angles to the buttress. They reinforce the buttresses.

(5) **Corbels.** Corbels help in transmitting the load from the upstream deck to the buttresses.

(6) **Concrete Cutoffs.** The cutoff walls are constructed to reduce or prevent seepage.

Fig. 9.8 illustrates a flat-slab type buttress dam showing all the above component parts.

Forces on buttress dam. A buttress dam is also subjected to the same forces as gravity arch dams. However, the ice pressure is not important as the ice tends to slide up the sloping membrane. The total uplift pressure is usually of smaller magnitude since the uplift pressure is relieved by the gaps between the buttresses.

9.7. TYPES OF BUTTRESS DAMS

Buttress dams may be broadly classified under three heads :

- (i) Rigid. (ii) Articulated.
- (iii) Intermediate or semi-rigid.

Rigid type buttress dams have their upstream deck or sloping membrane constructed monolithically and unyielding with their buttresses. No provision is made for unequal foundation settlement or for expansion and contraction. Under this category fall : (a) the multiple arch dam, and (b) multiple dome dams.

Articulated buttress dams are quite flexible, and are not constructed monolithically with the buttresses. The flat-slab buttress dam is an example of this type.

Intermediate or semi-rigid buttress dams fall between rigid type and the articulated type. The design of such dams eliminates the undesirable rigidity of the multiple arch and multiple dome designs, although not securing the high degree of flexibility or articulation incidental to the flat slab design. The round-head buttress dam and the diamond head buttress dam fall under this category.

The various types of buttress dams falling under the above three categories are as follows :

- (1) Flat slab or Ambursen type
- (2) Multiple-arch type
- (3) Multiple dome type
- (4) Cylinder or massive head type
- (5) Columnar buttress type
- (6) Truss buttress type.

(1) **Flat slab or Ambursen type.** The flat slab buttress dam has borne the name of its inventor, Nils F. Ambursen in 1903. The upstream deck or sloping membrane consists of reinforced concrete slab, separated by buttress tongues, and supported by reinforced haunches which are constructed monolithically with the buttress, such as shown in Fig. 9.8.

The most typical feature of the Ambursen type is its articulation or flexibility. The slab is not rigidly attached to the buttresses ; the joint between the slab and the buttresses is filled with some flexible joint compound or asphaltic putty. Such provision permits each slab to act independently.

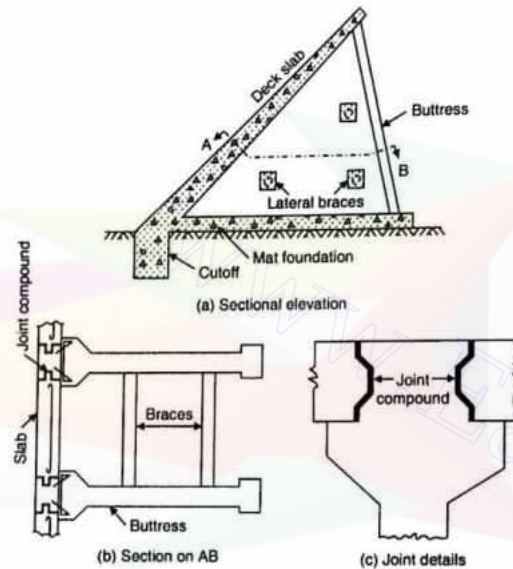


FIG. 9.8. FLAT SLAB BUTTRESS DAM.

The inclination of the deck may be kept between 35 to 45 degrees with horizontal. For proper sliding resistance, too steep slope increases the tendency to sliding, while too flat a slope will cause a very high inclined stress at the heel. For constant buttress spacing, a variation of few degrees in the upstream slope results in an appreciable change in the quantity of concrete. Hence the determination of most economic upstream angle from point of view of saving in the cost is very important. Concrete quantities are worked out for various angles of upstream slope and different buttress spacing and plotted to get a curve for each angle. A master curve is then drawn through the junctions of the curves and vertical line corresponding to each angle. The master curve shows the absolute economic value of buttress spacing and the angle of upstream slope for the given height of the dam. Buttresses spacing varies with the height of the dam from about 5 m for dams under 16 m to 16 m for dams more than 50 m high. The most economical spacing is that which gives the minimum thickness for slab and buttress so that the concrete is fully stressed at all sections.

The other types of flat-slab buttress dams are (i) *fixed slab type* and (ii) *cantilever type*, as shown in Fig. 9.9.

In the *fixed slab type*, the deck slab is cast monolithically with the buttress, with provision of expansion joints at convenient places. Such dams are not very much in use, except for very long heights. The stresses are there both at the upstream as well as downstream face of the slab. The reinforcement placed at the u/s face may

corrode. Also, it is difficult to assess the stresses due to temperature variations, shrinkage of decks and buttresses, and due to settlement of foundations.

In the *cantilever type*, the slab is laid monolithically with the buttresses, with overhangs at both the sides. Thus a joint is left at the centre of each span between two buttresses. Due to cantilever action, steel has to be placed towards the upstream face. It is also subjected to high stresses due to temperature, shrinkage and settlement.

The *Ambursen dam* is called the *free deck type dam*. In this, steel is placed at the d/s face, and hence there is no danger due to rusting. Also the difficulties due to influence of temperature, shrinkage and settlement of foundations are eliminated. This type of dam is particularly adapted to wide valleys where a long dam is required and foundation materials are of interior strength.

(2) **Multiple Arch Type Buttress Dam.** This type of dam consists of a series of arches transmitting its water load to buttresses. Meer Alam dam in India, built in about 1800 A.D. is the earliest recorded example of such a type. This type is used for high heights. The stability of any unit depends upon the adjacent units so that any settlement in one may affect other units. Hence it requires better foundation. For short span (10 to 15 m), circular arches of uniform thickness is economical while for greater spans, arches of variable thickness are provided. If the central angle of each arch is kept between 150° to 180° with semicircular arches (central angle 180°), the horizontal thrust between adjoining arches is eliminated. The arches may be supported on either the single wall stiffened buttresses or double wall hollow buttresses.

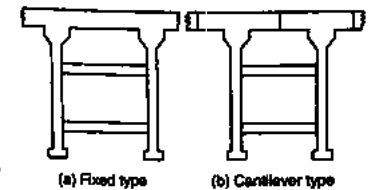


FIG. 9.9. OTHER TYPES OF FLAT-SLAB BUTTRESS DAMS

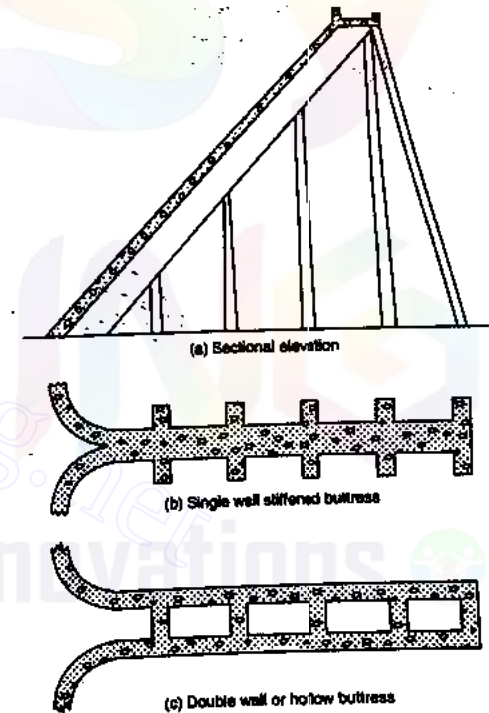


FIG. 9.10. MULTIPLE-ARCH BUTTRESS DAM.

(3) **Multiple Dome Type of Buttress Dam.** This is a modification of multiple arch dam and consists of a dome shaped deck supported on buttresses.

(4) **Cylinder or Massive Head Type.** In this type, the water supporting member is formed by enlarging the upstream end of the buttress until it meets the adjacent surface. When the enlarged head is circular, it is known as *round head deck* [Fig. 9.11 (b)]. When the head converges to a point, it is known as *diamond head deck* [Fig. 9.11(c)]. The pressure acts radially to the surface of the cylindrical type, or perpendicularly to the diamond type and hence all pressures are compressive. Reinforcement is therefore, not required for the deck and there is no danger of corrosion.

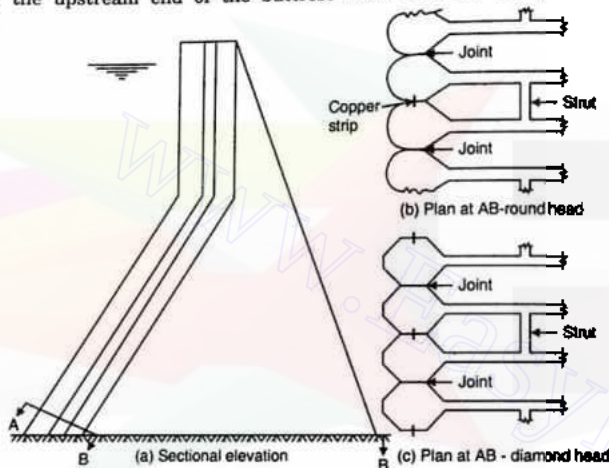


FIG. 9.11. MASSIVE HEAD BUTTERESS DAMS

(5) **Columnar Buttress Type.** This is a modification of flat slab type. In this, the deck-slab is supported on a number of inclined columns, instead of buttresses.

(6) **Truss Buttress Type.** This is another modification of flat slab type. This deck slab is supported on reinforced concrete trusses.

PROBLEMS

1. Discuss in brief various types of arch dams.
2. Prove that the most economical central angle for a constant radius arch dam is $133^\circ 34'$.
3. Discuss various forces that act on an arch dam.
4. What is a buttress dam? Describe the functions of its component parts.
5. Describe with the help of sketches flat-slab buttress dam (Ambursen type.)
6. Sketch various types of buttress dams in common use.
7. Discuss advantages and disadvantages of buttress dams.

Dams-IV : Earth and Rockfill Dams

10.1. INTRODUCTION : TYPES OF EARTH DAMS

Earth dams have been built since the early days of civilisation. Today, as in the past, the earth dam constitutes to be the most common type of dams, since it is generally built of locally available materials in their natural state with a minimum of processing. Earth dams are composed of fragmental materials which maintain their individual identity. These fragments have spaces and voids between them, and derive their strength from their position, friction and cohesion. Some of the earth dams built in antiquity were of considerable size. One earth dam, 17.6 km (11 miles) long, 23.38 m (70 ft) height, and containing about 13 million cubic metres (17 million cubic yards) of embankment was completed in Ceylon in the year 504 B.C.

Depending upon the method of construction, earth dams can be divided into two categories :

- (i) Rolled fill dam
- (ii) Hydraulic fill dam.

In the *rolled fill dam*, the embankment is constructed in successive, mechanically compacted layers. The suitable materials are transported from the borrow pits to the construction site by earth moving machinery. It is then spread by bulldozers, and sprinkled to form layers of limited thickness having proper water content. They are then thoroughly compacted and bonded with the preceding layer by means of power-operated rollers of proper design and weight.

In the case of *hydraulic fill dam*, the materials are excavated, transported and placed by hydraulic methods. Flumes are laid at a suitable falling gradient along the outer edge of the embankment. The material, mixed with water at the borrow pits, is pumped and washed into these flumes. The slush is discharged through the out-lets in the flumes, at suitable interval along their length. The slush thus flows towards the centre of the bank. The coarse materials of the slush settle at the outer edge while the fine materials settle at the centre. No compaction is done.

Rolled-fill earth dams can further be sub-divided into following types :

1. Homogeneous embankment type,
2. Zoned embankment type, and

3. Diaphragm embankment type.

(1) **Homogeneous Embankment Type.** A purely homogeneous type earth dam is composed of single kind of material (exclusive of the slope protection). Homogeneous dams have been built since the earliest times and are used today whenever only one type of material is economically available. However, they are used only for low to moderate heights. A purely homogeneous section has been replaced by a *modified homogeneous* section in which internal drainage system in the form of horizontal filter drain or rock toe, is provided. This controls the action of seepage so as to permit much steeper slopes. The drainage system also keeps the phreatic line well within the body of the dam. For the purpose of further discussions, however, we shall designate the 'homogeneous section' as the one composed of one type of material, with or without internal drain.

Homogeneous dams are usually composed of impervious or semi-impervious soils to provide an adequate water barrier. However, the upstream slope has to be flatter, to make it safe during the 'sudden drawdown condition'. Many successful embankments have been built to relatively pervious sands and sand-gravel mixtures.

(2) **Zoned Embankment Type.** Zoned embankment type earth dam is the one in which the dam is made up of more than one material. The most common type of a rolled earth dam section is that in which a central impervious core is flanked by zones of material considerably more pervious. A suitable drainage system, in the form of a horizontal drain or a rock toe is also provided at the d/s side. If a variety of soils are readily available, the choice of type of earth-fill dam should always be zoned embankment type because its inherent advantages will lead to economy in the cost of construction.

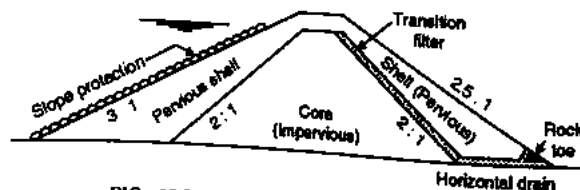


FIG. 10.2. ZONED EMBANKMENT TYPE.

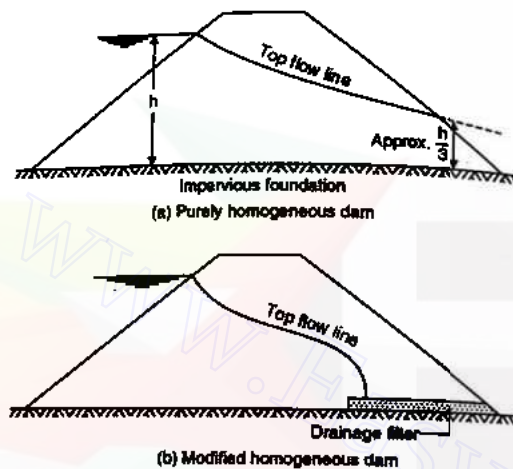


FIG. 10.1. HOMOGENEOUS DAM.

In the zoned embankment type, outer shells are made of pervious, freely draining materials. The shells give stability to the central impervious fill, and at the same time distribute the load over a larger area in the foundation. Sometimes, a semi pervious zone is provided to serve as a transition between the outer shell and the central core. The upstream pervious zone affords stability against rapid drawdown while the downstream pervious zone acts as a drain to control the line of seepage. The central impervious core checks the seepage.

(3) **Diaphragm Embankment Type.** This is a modification over the homogeneous embankment type, in which the bulk of the embankment is constructed of pervious material and a *thin* diaphragm of impermeable material is provided to check the seepage. The diaphragm may be of impervious soil, cement concrete, bituminous concrete, or any other material, and may be placed either at the centre of the section as a central vertical core, or at the upstream face as a blanket.

However, the distinction between a diaphragm type and zoned type must be clearly known. If the horizontal thickness of the diaphragm (made of impervious soil) at any elevation is less than 10 m or less than the height of embankment above any corresponding elevation in the dam, the dam is of the diaphragm type. If the impervious earth diaphragm equals or exceeds these thicknesses, the dam is considered to be 'zoned embankment' type.

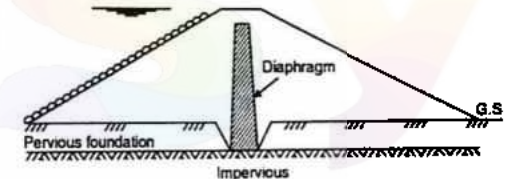


FIG. 10.3. DIAPHRAGM TYPE EMBANKMENT.

10.2. CAUSES OF FAILURES OF EARTH DAMS

In spite of taking great care in construction of earth dams, some failures have occurred in the past. However, a knowledge of the principal lessons learned from failures and damages in the past is an essential part of the training of earth dam designer.

On the basis of investigation reports on most of the past failures, it has now been possible to categorise the types of failures into three main classes :

1. Hydraulic failures : 40%
2. Seepage failures : 30%
3. Structural failures : 30%

HYDRAULIC FAILURES. Hydraulic failures include the following :

- (i) Overtopping
- (ii) Wave erosion
- (iii) Toe erosion
- (iv) Gullyng

(i) **Overtopping.** The earth dam may get overtopped if the design flood is under-estimated, or if the spillway is of insufficient capacity. Faulty operation of spillway gates, etc. may also sometimes lead to overtopping. Insufficient free board or settlement of foundation and embankment may also lead to overtopping.

(iii) **Wave Erosion.** Fig. 10.4 shows failures due to wave erosion. The effects of wave is to notch out earth from the upstream slope in absence of proper slope protection in the form of riprap. Rollers are developed in the waves which try to scoop out the earth. Waves can also cause upstream slips [Fig. 10.4(b)].

(iii) **Toe Erosion.** Toe erosion may occur due to two reasons: (a) erosion due to tail water, and (b) erosion due to cross currents that may come from spillway buckets or from exit areas of outlets. The toe erosion can be avoided by providing thick riprap on the d/s slope, upto a height slightly above the tail water level. Diaphragm walls of sufficient length and height should be provided to check the cross flow towards the earth dam.

(iv) **Gullying.** Downstream slope may fail due to the formation of gullies by heavy downpour. To eliminate failure due to gullying, proper berms, turfing and good drainage system should be provided to the downstream side.

2. SEEPAGE FAILURES.

Seepage failures may be due to (a) piping and (b) sloughing.

(a) **Piping.** The seepage of water through the body and foundation of the earth dam may lead to piping or progressive erosion of concentrated leaks, causing a large number of catastrophic failures. Water seeping through the earth dam may have four bad effects : (i) seeping water generates erosive forces which dislodge particles from the soil structure and causes rearrangement or migration of the fines to voids between larger grains, (ii) the flow with its associated differential pore

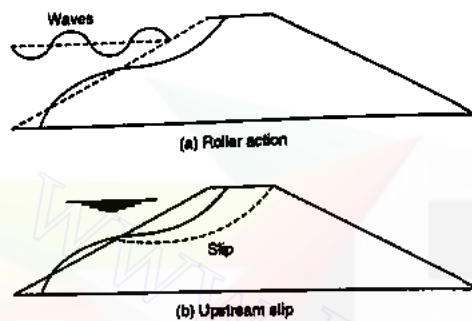


FIG. 10.4. WAVE EROSION.

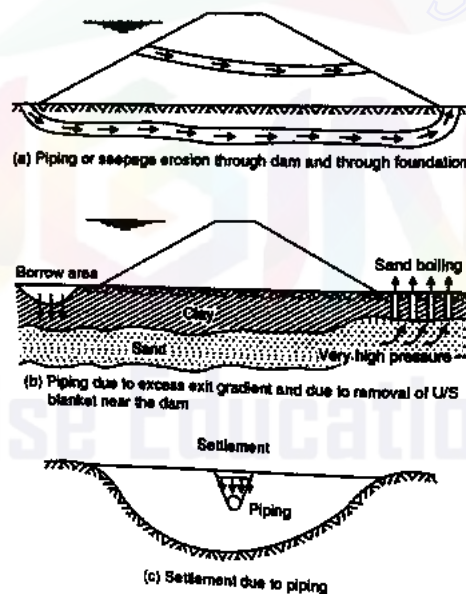


FIG. 10.5. PIPING FAILURES.

pressure can lift portion of soil mass causing sand boiling, (iii) internal erosion of the soil mass, progressive backwards from the point of exit leads to the formation of an open conduit through the soil (i.e. piping), and (iv) the internal pressures in the soil water can reduce that part of the soil strength that is developed by internal friction and thereby lead to weakening of the soil mass and even failure by shear. Most of the serious trouble from piping has resulted from progressive backward erosion of concentrated leaks which develop through or under the dam.

Leaks in the embankment may also lead to piping failure. Probably the most common cause of embankment leaks has been poor construction control which can result in inadequately compacted or pervious layers in the embankment, inferior compaction adjacent to concrete outlet pipes or other structures, or poor compaction and bond between the embankment and the foundation or abutments. Embankment leaks through differential settlement cracks have also been a major source of trouble.

(b) **Sloughing.** Failure due to progressive sloughing or ravelling is closely related to piping. Under the full reservoir condition, the downstream toe remains saturated and may erode, producing a small slump or miniature slide. This miniature slide leaves a relatively steep face, which becomes saturated by seepage from the reservoir and slumps again, forming a slightly higher and more unstable face. This ravelling process can continue till the remaining portion of the dam is too thin to withstand the water pressure and complete failure occurs suddenly as the reservoir breaks through. The failure of the Sinker Creek dam in Idaho in 1943 is good illustration of the mechanics of progressive backward sloughing.

3. STRUCTURAL FAILURES. Structural failures may be due to the following reasons :

- (i) Upstream and downstream slope failures due to construction pore pressures.
- (ii) Upstream slope failure due to sudden drawdown.
- (iii) Downstream slope failure during full reservoir condition.
- (iv) Foundation slide : spontaneous liquefaction.
- (v) Failure by spreading.
- (vi) Failure due to earthquakes.
- (vii) Slope protection failures.
- (viii) Failure due to damage caused by burrowing animals.
- (ix) Damage caused by water soluble materials.

(i) **Upstream and Downstream Slope Failures due to Construction Pore Pressure.** When a dam is built of relatively impervious compressible soil, the drainage is extremely slow and excess pore pressures develop during and immediately after construction. When the permeability is low (less than 10^{-4} cm/sec), there may be no substantial drop in pore pressure in the central zone of the dam by the end of the construction if this lies within the usual range of 2 - 4 years. An initial pore pressure upto almost 100% of total weight of soil above the point considered has been measured in the field. Under these circumstances, the construction stage in many cases, may be more critical from the point of view of stability and the slopes may fail. Experience indicates that there are two types of construction slides, which differ primarily in speed and

magnitude of the movement. The first type of slide occurs slowly and continues at a uniform rate for a period usually of one to two weeks. The failure of North Ridge Dam in Canada (Sept. 1953), is the example of such slow slide. The second type of slide occurs rapidly and suddenly. The major part of the sliding (magnitude of movement) usually equal to one half or more of the total height of the dam is over in few minutes. The failure at Marshall Creek Dam in Kansas (1937) is the typical example of this type, where the major movement covered a period of 15 to 20 minutes and was accompanied by rumbling noise and dense clouds of dust.

(ii) **Upstream Slope Slide During Sudden Drawdown.** For the upstream slope, the critical condition is when the reservoir is suddenly emptied without allowing any appreciable change in water level within the saturated soil mass. This stage is known as sudden draw-down. With complete draw-down, the hydrostatic force acting along the upstream slope at the time of full reservoir is removed without the hydrostatic pressure on the slope to counteract it. Upstream slides have not caused complete failures or loss of water from the reservoir, although they have occasionally blocked the entrances to outlet conduits and made these useless for further lowering of the reservoir. When the upstream slide occurs due to sudden drawdown, the pore pressure along the surface of slide are dissipated to a large extent. Hence there is lesser tendency for the continued sloughing and sliding, and there is small likelihood of catastrophic failure even though a large earth movement has taken place.



FIG. 10.6. US SLOPE FAILURE DURING SUDDEN DRAWDOWN.

(iii) **Downstream Slope Slide during Full Reservoir Condition.** Critical condition for downstream slope occurs when the reservoir is full and percolation is at its maximum rate. The direction of seepage forces tend to decrease stability. Or, in other words, the pore water pressure acting on the soil mass below the saturation line reduces the effective stress responsible for mobilising shearing resistance. There are two types of d/s slides : (a) deep slides, and (b) shallow slides. The deep slides generally pass through the clay foundation and frequently reduce the free board by extending further upstream than the upstream edge of the crest. When such slide occurs, there is no relief of pore pressure and the unstable vertical slide scarp left standing often sloughs or slides again until it breaches the dam and releases the reservoir water in one great flood wave. Shallow slides do not as a rule extend in the embankment in a direction normal to the slope more than 1 to 2 m.



FIG. 10.7. DEEP D/S SLIDES DURING STEADY SEEPAGE.

(iv) **Foundation Slides : Spontaneous Liquefaction.** When the earth dam has foundation of fine silt or soft soil it can slide wholly. Sometimes a soft and weak clayey seam exists under the foundation and the dam can slide over it causing the failure. Excess water pressure on confined sand and silt seams in the foundation may also cause unbalanced conditions causing foundation failure. Expansion of soils on saturation may cause lifting of the slopes and thus may cause failure of foundation. One of the most difficult problems facing the earth dam designer is the analysis of the stability of loose sand foundations against the possibility of liquefaction or flow slides. The failure of the Sheffield Dam in Santa Barbara in 1925 following an earthquake shock probably was caused by liquefaction of lower portion of the dam or the foundation.

(v) **Failure by Spreading.** Failures by spreading have been observed only in connection with fills located above stratified deposits that contain layers of soft clay. Outstanding failures due to spreading are those of Lafayette Dam in California (1928), Marshall Creek Dam in Kansas (1927), and the Hartford flood protection dyke in Connecticut (1941).



FIG. 10.8. FAILURE BY SPREADING

(vi) **Failure due to Earthquake.** Most of the failures due to earthquake have occurred only with respect to those dams which were constructed before 1920 with inferior design details and construction methods. The accumulation of experience concerning the behavior or performance of dams when shaken by earthquakes is small. However, we can speculate with some degree of assurance that most serious damages and failures may be due to the following effects due to earthquake : (1) Cracks in the core of dam leading to leakage and piping failure, (2) Settlement of the crest due to compression of foundation and/or embankment, thereby reducing the free board and increasing the chances of overtopping, (3) Shaking of reservoir bottom causing slow waves (seiches), and hence the failure due to overtopping (4) Sliding of natural hill side causing overtopping and damaging appurtenant structures, (5) Liquefaction of sand below foundation, (6) Acceleration forces on embankment may cause shear slide of appreciable point of the slope of the dam and (7) Fault movement causing reduction in the reservoir capacity and consequent overtopping.

(vii) **Slope Protection Failure.** Slopes are generally protected by riprap (either hand placed or dumped) over a layer of gravel or filter blanket. During a heavy storm, the waves on the surface of the reservoir beat repeatedly against the slope just above the reservoir level. The wave energy is dissipated in turbulent action on and within the rocks of the riprap layer. This action may have two effects : (1) the waves may pass through the voids of riprap and

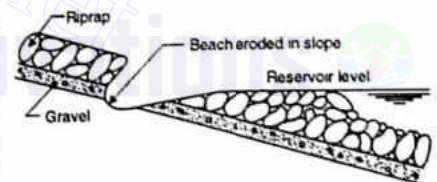


FIG. 10.9. TYPICAL FAILURE OF RIPRAP.

may wash away the filter layer, setting the riprap layer and exposing the embankment to wave erosion, and (2) if the average size of rock comprising riprap is not heavy, it may be washed out of the layer by the hydraulic forces generated by the waves. However, failure of filter layer is more common.

(viii) **Failure due to Damage Caused by Burrowing Animals.** Burrowing animals may cause piping failure of small dams only. Animals like muskrats burrow into embankment either to make homes or to dig passage from one pond to another as shown in Fig. 10.10(a) and (b). If many muskrats are involved, their holes may dangerously honeycomb a small earth dam, making it extremely weak. Ground squirrels normally dig only in dry soil and stop at the point where seepage is encountered. However, if the water level in the dam is very low for a number of years, the squirrels may completely dig dam.

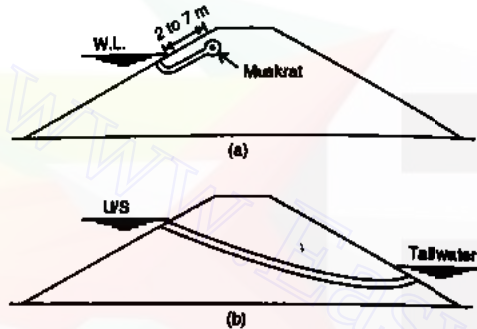


FIG. 10.10. HOLES MADE BY MUSKRATS.

(ix) **Damage caused by Water Soluble Materials.** The leaching of natural deposits of water soluble materials, such as gypsum, has created trouble at many dams. Where considerable foundation leakage occurs, leaching of gypsum of the foundation may cause excessive settlement. The disposition of soluble material previously leached from the natural soil may tend to plug specially designed filters. The failure of the Apishapa Dam in Southern Colorado in 1923 was attributed, at least in part, by some engineers to dissolution of salts from the embankment material.

10.3. CRITERIA FOR SAFE DESIGN OF EARTH DAM

An earth dam must be safe and stable during phases of construction and operation of the reservoir. The practical criteria for the design of earth dams may be stated briefly as follows :

- (1) The embankment must be safe against overtopping during occurrence of the inflow design flood by the provision of sufficient spillway and outlet works capacity.
- (2) The dam must have sufficient free board so that it is not overtopped by wave action.
- (3) The seepage line should be well within the d/s face so that no sloughing of the slope takes place.
- (4) Seepage flow through the embankment, foundation and abutments must be controlled by suitable design provisions so that no internal erosion takes place. The amount of water lost through seepage must be controlled so that it does not interfere with planned project functions.

- (5) There should be no opportunity for the free passage of water from upstream to the downstream either through the dam or through the foundation.
- (6) The portion of the downstream of the impervious core should be properly drained.
- (7) The upstream and downstream slopes should be so designed that they are safe during and immediately after the construction.
- (8) The downstream slope should be so designed that it is safe during steady seepage case under full reservoir condition.
- (9) The upstream slope should be stable during rapid drawdown condition.
- (10) The upstream and downstream slopes of the dam should be flat enough so that shear stress induced in the foundation is enough less than the shear strength of the material in the foundation to ensure a suitable factor of safety.
- (11) The dam as a whole should be earthquake resistant.
- (12) The upstream slope must be protected against erosion by wave action, and the crest and down stream slope must be protected against erosion due to wind and rain.

The above criteria of design have been covered at length in the subsequent articles.

10.4. SECTION OF AN EARTH DAM

The preliminary design of an earth dam is done on the basis of past experience and on the basis of the performance of the dams built in the past. We shall discuss here the preliminary selection of the following terms :

(1) Top width.	(2) Free board.
(3) Casing or outer shells.	(4) Central impervious core.
(5) Cut-off trench.	(6) Downstream drainage system.

(1) Top Width.

The crest width of an earth dam depends on the following considerations :

- (i) Nature of the embankment materials and minimum allowable percolation distance through the embankment at the normal reservoir level.
- (ii) Height of the structure.
- (iii) Importance of the structure.
- (iv) Width of highway on the top of the dam.
- (v) Practicability of construction.
- (vi) Protection against earthquake forces.

Following are some of the empirical expressions for the top width b of the earth dam, in terms of the height Z of the dam :

$$b = \frac{Z}{5} + 3 \quad \dots(10.1)$$

$$b = 0.55 Z^{1/2} + 0.2 Z \quad \dots(10.2)$$

$$b = 1.65 (Z + 1.5)^{1/3} \quad \dots(10.3)$$

Eq. 10.1 is applicable for very low dams. Eq. 10.2 is applicable for dams lower than 30 m. Eq. 10.3 is given by U.S.B.R., and is applicable for dams higher than 30 m.

According to the Indian Standard, no dam should have crest width of less than 6 m.

(2) Free board.

Free board is the vertical distance between the horizontal crest of the embankment and the reservoir level. *Normal free board* is the difference in the level between the crest or top of the embankment and normal reservoir level. *Minimum free board* is the difference in the elevation between the crest of the dam and the maximum reservoir water surface that would result should the inflow design flood occur and should the outlet works and spillway function as planned. Sufficient free board must be provided so that there is no possibility whatsoever of the embankment being overtopped. The U.S.B.R. suggests the following free boards :

TABLE 10.1 U.S.B.R. PRACTICE FOR FREE BOARD

Nature of spillway	Height of Dam	Free Board
Free	Any	Minimum 2 m and maximum 3 m over the maximum flood level
Controlled	Less than 60 m	2.5 m above the top of gates.
Controlled	Over 60 m	3 m above the top of gates.

(3) Casing or outer shells.

The function of *casing or outer shells* is to impart stability and protect the core. The relatively pervious materials, which are not subject to cracking on direct exposure to atmosphere, are suitable for casing. Table 10.2 (a) gives recommendations for suitability of soils used for earth dams as per IS: 8826-1978.

TABLE 10.2 (a). SUITABILITY OF SOILS FOR CONSTRUCTION OF EARTH DAMS

Relative Suitability	Homogeneous Sections	Zoned Earth Dam	
		Pervious Casing	Impervious Core
1. Very Suitable	GC	SW, GW	GC
2. Suitable	CL, CI	GM	CL, CI
3. Fairly suitable	SP, SM, CH	SP, GP	CM, GC, SM, SC, CH
4. Poor	-	-	ML, MI, MH
5. Not suitable	-	-	OL, NI, OH, Pt

The design slopes of the upstream and downstream embankments may vary widely, depending on the character of the materials available, foundation conditions and the height of the dam. The slopes also depend upon the type of the dam (i.e. homogeneous, zoned or diaphragm).

The upstream slope may vary from 2 : 1 to as flat as 4 : 1 for stability. A storage dam subjected to rapid drawdown of the reservoir should have an upstream zone with permeability sufficient to dissipate pore water pressures exerted outwardly in the upstream part of the dam. If only materials of low permeability are available, it is necessary to provide flat slope for the rapid drawdown requirement. However, a steep slope may be provided if free draining sand and gravel are available to provide a superimposed weight for holding down the fine material of low permeability. The

usual downstream slopes are 2 : 1, where a downstream pervious zone is provided, to $2\frac{1}{2}$: 1, where embankment is impervious. Table 10.2(b) gives the side slopes for preliminary design of earth dams according to Terzaghi. Table 10.2(c) gives preliminary dimensions of earth dams according to Strange.

TABLE 10.2(b). SIDE SLOPES FOR EARTH DAMS ACCORDING TO TERZAGHI

	Type of material	Upstream slope	Downstream slope
(i)	Homogeneous well graded material	$2\frac{1}{2}$: 1	2 : 1
(ii)	Homogeneous coarse silt	3 : 1	$2\frac{1}{2}$: 1
(iii)	Homogeneous silty clay, or clay	$2\frac{1}{2}$: 1	2 : 1
	Ht. less than 15 m Ht. more than 15 m	$2\frac{1}{2}$: 1 3 : 1	$2\frac{1}{2}$: 1 $2\frac{1}{2}$: 1
(iv)	Sand or sand and gravel with clay core	3 : 1	$2\frac{1}{2}$: 1
(v)	Sand or sand and gravel with R.C. core wall	$2\frac{1}{2}$: 1	2 : 1

TABLE 10.2(c). PRELIMINARY DIMENSIONS OF EARTH DAMS ACCORDING TO STRANGE

Height of dam above foundation level (m)	Height of dam above H.F.L. (m)	Top width (m)	U/S slope	D/S slope
Upto 4.5	1.2 to 1.5	1.8	2 : 1	$1\frac{1}{2}$: 1
4.5 to 7.5	1.5 to 1.8	1.85	$2\frac{1}{2}$: 1	$1\frac{3}{4}$: 1
7.5 to 15	1.85	2.5	3 : 1	2 : 1
15 to 22.5	2.1	3.0	3 : 1	2 : 1

(4) Central Impervious Core.

The minimum safe thickness of the central impervious core depends the following factors:

- Tolerable seepage loss.
- Maximum width that will permit proper construction.
- Types of the materials available for the core and the shells.
- Design of the proposed filter layers.
- Precedent on similar projects.

It should be noted that the shear strength of core materials is always lesser than the rest of the embankment. Hence a thinner shell is preferable from the point of view of stability. However, a thick core has more resistance to piping which may develop in differential settlement cracks. The governing criterion is that the thickness of the core at any elevation is not less than the height of the embankment at that elevation so that the average hydraulic gradient is less than unity. The width of the core at the crest of the dam should be a minimum of 3 m to permit economical placement and compaction of impervious embankment material by construction equipment, which

may include trucks, dozers and tamping rollers. The top level of the core should be at least 1 metre above the maximum water level to prevent seepage by capillary syphoning.

Fig. 10.11 shows the size range of impervious core in zoned embankments. The minimum core for a dam on pervious foundation is based on consideration of seepage pressures in the foundation.

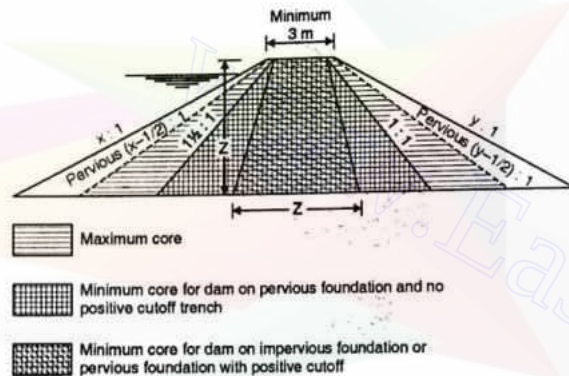


FIG. 10.11. SIZE RANGE OF IMPERVIOUS CORES IN ZONED EMBANKMENT.

Sloping Cores in Earth Dam¹

The impervious core may be placed with axis either vertical (Fig. 10.11), or inclined (Fig. 10.12). The advantages of the vertical core are : (i) the higher pressure exists on the contact between the core and the foundations, reducing the possibility of leakage along the contact, and (ii) thickness of vertical core is greater than the thickness of a sloping core, for a given quantity of impervious soil. However, the advantages of the sloping cores are : (i) the downstream portion of the embankment can be constructed first and the core placed later, (ii) foundation grouting can be carried out while the embankment is being placed and (iii) filter layers between the core and the upstream and downstream pervious zones can be made thinner with less difficulty in construction than the first layers with vertical core dam. The vertical core dam has a steeper upstream

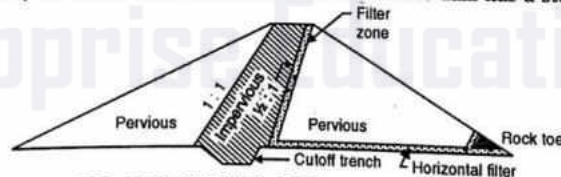


FIG. 10.12. SLOPING CORE IN EARTH DAMS.

¹ Lewis, J.G., "The use of a Sloping Core in Earth and Rockfill Dams". Fifth Congress on Large Dams, Paris, (1955), Vol. I, p. 532.

slope than the sloping core dam, so that the total volumes are about the same. The theoretical minimum volume for a thin core dam is obtained when the core is located in such a way that the upstream and downstream slopes have approximately the same safety factors against shear failure. When the core is sloped upstream, this condition is achieved with an average inclination¹ of about $\frac{1}{2} : 1$.

Suitability of Material for Core

The soils having high compressibility and liquid limit are not suitable as they are prone to swelling and formation of cracks. Soils having organic contents are also not suitable. Table 10.2(a) gives the suitability of soils for impervious core of zoned earth dams. IS : 1498 - 1970 may further be referred for suitability of soils for core. Table 10.3 gives the recommendation based on IS : 8826 - 1978 regarding suitability of soils for construction of core for earth dams in earthquake zones.

TABLE 10.3. SUITABILITY OF SOILS FOR CONSTRUCTION OF CORE OF EARTH DAM IN EARTHQUAKE ZONES

Relative Suitability	Type of Soil
1. Very good	Very well graded coarse mixtures of sand, gravel and fines, D_{60} coarser than 50 mm. D_{30} coarser than 6 mm.
2. Good	(a) Well graded mixture of sand, gravel and clayey fines. D_{60} coarser than 25 mm ; Fines consisting of inorganic clay (CL with plasticity index > 12). (b) Highly plastic tough clay (CH with plasticity index > 20).
3. Fair	(a) Fairly well graded, gravelly, medium to coarse sand with cohesionless fines, D_{60} coarser than 19 mm. D_{30} between 0.5 mm to 3.0 mm. Not more than 25% finer than 75 μ . (b) Clay of medium plasticity (CL with plasticity index > 12).
4. Poor	(a) Clay of low plasticity (CL and CL-ML) with little coarse fraction. Plasticity index between 5 and 8. Liquid limit > 25. (b) Silts of medium to high plasticity (ML or MH) with little coarse fraction. Plasticity index > 10. (c) Medium sand with cohesionless fines.
5. Very poor	(a) Fine, uniform cohesionless silty sand, D_{60} finer than 0.3 mm. (b) Silt from medium plasticity to cohesionless (ML). Plasticity index less than 10.

(5) Cutoff Trench.

Cutoff is required to (a) reduce loss of stored water through foundation and abutments, and (b) prevent sub-surface erosion by piping. Following are the Indian Standard Recommendations (IS : 8826 - 1978).

(i) The alignment of the cutoff trench should be fixed in such a way that its central line should be within the u/s base of the impervious core and it should be keyed into rock or continuous impervious strata.

(ii) The bottom width of cutoff trench may be fixed taking following factors into consideration :

- Provide sufficient working space for compaction equipment,
- Provide sufficient working space to carry out curtain grouting, and

(c) Provide safety against piping.

A minimum width of 4 m is recommended. A bottom width of 10 % of hydraulic head may be provided to safety requirements of piping. This may be suitably increased to satisfy other requirements of mechanical equipments and curtain grouting. The side slopes depend upon sub-strata. Side slopes of at least 1 : 1 or flatter may be provided in case of overburden, while 1/2 and 1/4 : 1 may be provided in soft rock and hard rock respectively.

(iii) The positive cutoff should be taken atleast one metre into continuous impervious substratum.

(iv) The partial cutoff is specially suited for horizontally stratified foundations with relatively more pervious layer near top. The depth of the partial cutoff in deep pervious alluvium will be governed by :

(a) Permeability of substrata, and

(b) Relative economics of depth of excavation governed usually by cost of dewatering versus length of w/s impervious blanket.

(v) The backfill material for cutoff trench shall have same properties as those prescribed for impervious core.

(vi) The cutoff in the flanks on either side should normally extend upto the top of the impervious core, particularly in case of steep abutments.

10.5. DOWNSTREAM DRAINAGE SYSTEM

Filter zones are invariably provided in all earth dams. They are constructed of material appreciably more pervious than the embankment soil. Provision of filter zones at the d/s side serves two purposes :

(1) It reduces the pore water pressure in the downstream portion of the dam, and hence increases its stability.

(2) It checks the piping by checking the migration of the particles.

Generally, three types of drains are provided in the earth dams :

(i) Toe drains [Fig. 10.13(a)]

(ii) Horizontal blanket drains [Fig. 10.13(b)]

(iii) 'Chimney' drains [Fig. 10.13(c)]

Toe drains were installed in some of the oldest homogeneous dams in an effect to prevent softening which was observed at the downstream toe. Horizontal drainage blankets are widely used for dams of moderate height. The greatest disadvantage in the provision of horizontal drain results from the fact of stratification of the embankment material having permeability in the horizontal direction more than the vertical.

In such a circumstance, inspite of horizontal drain, the seepage water may flow horizontally at top of a relatively impervious layer and discharge on the downstream face, causing surface sloughing. In an effort to prevent trouble with stratification and to intercept seepage water before it reaches the downstream slope, many of the higher homogeneous dams have been constructed with inclined or vertical 'chimney' drains. In several recent major dams, the chimney drain has been inclined at a considerable slopes, sometimes upstream and sometimes downstream.

Filter Criteria. The dimensions and permeability of filter drains must be chosen in such a way that the drainage system can carry away the anticipated flow with an ample margin of safety. The rational approach to the design of filter is generally credited to Terzaghi. Considerable experimentation has been performed by the U.S. Corps of Engineers and the U.S.B.R. Generally, a *multilayered filter* is provided, in which each subsequent layer becomes increasingly coarser than the previous one. Such a filter is sometimes known as *inverted filter* or *reverse filter*. The soil to be protected (i.e. the embankment and foundation material surrounding the filter) is known as the *base material*.

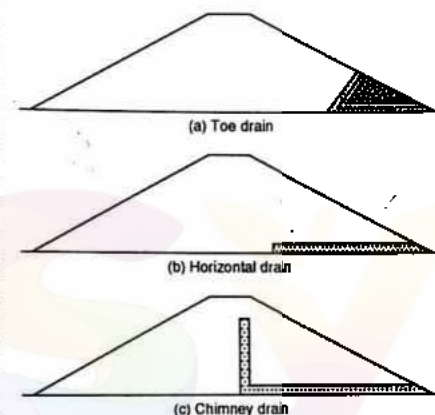


FIG. 10.13. DRAINS IN EARTH DAMS.

The four main requirements to be satisfied by filter material are as follows¹ :

(1) The filter material should be sufficiently fine and properly graded so that the voids of the filter are small enough to prevent base material particles from penetrating and clogging the filter.

(2) The filter material should be sufficiently coarse and pervious compared to the base material so that the incoming water is rapidly removed without any appreciable build up of seepage forces within the filter.

(3) The filter material should be coarse enough not to be carried away through the drainage pipe openings. The drainage pipes should be provided with sufficiently small openings or perforations, or additional coarser layer should be used, if necessary.

(4) The filter layer should be sufficiently thick to provide a good distribution of all particle size throughout the filter and to be able to carry the seepage discharge. The filter thickness should ensure an adequate safety against piping and proper insulation for frost susceptible base material, as the case may be.

According to Terzaghi, the filter material should fulfil the following two criteria:

(1) The D_{15} size of the filter material must not be more than 4 to 5 times D_{85} size of the base material. This prevents the foundation material from passing through the pores of the filter material.

(2) The D_{15} size of the filter material must be at least 4 to 5 times the D_{15} size of base material. This keeps seepage forces within the filter to permissible small magnitude.

¹Earth Manual, U.S. Bureau of Reclamation, Washington, 1960.

The above two criteria may be expressed as

$$\frac{D_{15} \text{ of filter}}{D_{85} \text{ base material}} < 4 \text{ to } 5 < \frac{D_{15} \text{ of filter}}{D_{15} \text{ of base material}}$$

The requirements cited above must be satisfied between any two adjacent layers of the filter. The criteria given by Terzaghi have been further modified as follows¹:

(a) $\frac{D_{15} \text{ of filter}}{D_{15} \text{ of base material}} = 5 \text{ to } 40$, provided that the filter does not contain more than 5 percent of material finer than 0.074 mm (no. 200 sieve).

(b) $\frac{D_{15} \text{ of filter}}{D_{85} \text{ of base material}} = 5 \text{ or less.}$

(c) $\frac{D_{85} \text{ of filter}}{\text{Maximum opening of pipe of drain}} = 2 \text{ or more.}$

(d) The grain size curve of the filter should be roughly parallel to that of the base material.

Drainage of earth dams. The filter or the rock toe discharges the water into a network of drains. The drains may consist of open trenches or filled with rough stones. It may also enclose tile drains in addition and protected by an inverted filter. The number of drains and their interval depends upon (i) nature of soil on which the dam is founded, (ii) nature of the ground, and (iii) type of material used for dams. Fig. 10.14 shows the arrangement of the network of drains.

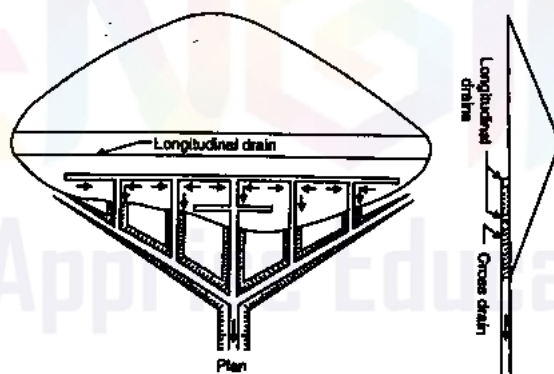


FIG. 10.14. DRAINAGE SYSTEM.

¹Design of small Dams, U.S. Bureau of Reclamation, pp. 174.

10.6. SEEPAGE ANALYSIS

The quantity of water passing through the body and foundation of the earth dam, as well as the distribution of water pressures can be estimated by the theory of flow of fluids through porous media. While computing these quantities with the help of theoretical analysis that follow, the following assumptions must be kept in mind:

- (1) The rolled embankment and the natural soil foundation of the earth dam are *incompressible* porous media. The size of the pore spaces do not change with time, regardless of water pressure.
- (2) The seeping water flows under a hydraulic gradient which is due only to gravity head loss, or Darcy's law for flow through porous medium is valid.
- (3) There is no change in the degree of saturation in the zone of soil through which the water seeps and the quantity flowing into any element of volume is equal to quantity which flows out in the same length of time.
- (4) The hydraulic boundary conditions at entry and exit are known.
- (5) Water is incompressible.

1. LAPLACE EQUATION FOR TWO DIMENSIONAL FLOW.

In earth dams, the flow is essentially two dimensional. Hence we shall consider only two dimensional flows.

Consider an element of soil of size $\Delta x, \Delta y$ and of unit thickness perpendicular to the plane of the paper. Let v_x and v_y be the entry velocity components in x and y directions. Then $\left(v_x + \frac{\partial v_x}{\partial x} \Delta x\right)$ and $\left(v_y + \frac{\partial v_y}{\partial y} \Delta y\right)$ will be the corresponding velocity components at the exit of the element. According to assumption 3 stated above, the quantity of water entering the element is equal to the quantity of water leaving it. Hence, we get

$$v_x (\Delta y \cdot 1) + v_y (\Delta x \cdot 1) = \left(v_x + \frac{\partial v_x}{\partial x} \Delta x\right) (\Delta y \cdot 1) + \left(v_y + \frac{\partial v_y}{\partial y} \Delta y\right) (\Delta x \cdot 1)$$

From which

$$\frac{\partial v_x}{\partial x} + \frac{\partial v_y}{\partial y} = 0 \quad \dots(i)$$

This is the continuity equation.

According to assumption 2:

$$v_x = k_x i_x = k_x \frac{\partial h}{\partial x} \quad \dots(ii)$$

$$\text{and } v_y = k_y i_y = k_y \frac{\partial h}{\partial y} \quad \dots(iii)$$

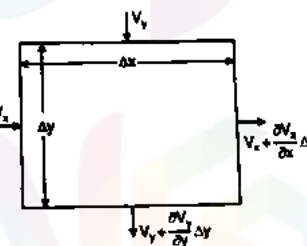


FIG. 10.15.

where

h = hydraulic head under which water flows.

k_x and k_y = coefficients of permeability in x and y directions.

Substituting in (ii) and (iii), we get

$$\frac{\partial^2 (k_x h)}{\partial x^2} + \frac{\partial^2 (k_y h)}{\partial y^2} = 0 \quad \dots(10.4)$$

For an isotropic soil,

$$k_x = k_y = k$$

Hence we get from Eq. 10.4

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} = 0$$

Substituting $\Phi = kh$ = velocity potential, we get

$$\frac{\partial^2 \Phi}{\partial x^2} + \frac{\partial^2 \Phi}{\partial y^2} = 0 \quad \dots(10.5)$$

This is the Laplace equation of flow in two dimensions. The velocity potential Φ may be defined as a scalar function of space and time such that its derivative with respect to any direction gives the fluid velocity in that direction.

This is evident, since we have $\Phi = kh$

$$\frac{\partial \Phi}{\partial x} = k \frac{\partial h}{\partial x} = k \cdot i_x = v_x$$

Similarly,

$$\frac{\partial \Phi}{\partial y} = k \frac{\partial h}{\partial y} = k \cdot i_y = v_y$$

The solution of Eq. 10.5 can be obtained by

(i) Analytical methods, (ii) Graphical method, (iii) Experimental methods.

The solution gives two sets of curves, known as equipotential lines and stream lines (or flow lines), mutually orthogonal to each other, as shown in Fig. 10.16. The equipotential lines represent contours of equal head (potential). The direction of seepage is always perpendicular to the equipotential lines. The path along which the individual particles of water seep through the soil are called stream lines or flow lines.

2. COMPUTATION OF RATE OF SEEPAGE FROM FLOW NET.

A network of equipotential lines and flow lines is known as a flow net. Fig. 10.16 shows a portion of such a flow net. The portion between any two successive flow lines is known as flow channel. The portion enclosed between two successive equipotential lines and successive flow lines is known as field such as that shown hatched in Fig. 10.16.

Let b and l be the width and length of the field.

Δh = head drop through the field.

Δq = discharge passing through the flow channel.

h = total head causing flow

DAMS-IV : EARTH AND ROCKFILL DAMS

= difference between upstream and downstream heads.

Then, from Darcy's law of flow through soils :

$$\Delta q = k \cdot \frac{\Delta h}{l} (b \times 1) \quad \dots(i)$$

(considering unit thickness).

If N_d = total number of potential drops in the complete flow net,

$$\text{Then } \Delta h = \frac{h}{N_d}$$

$$\text{and } \Delta q = k \cdot \frac{h}{N_d} \left(\frac{b}{l} \right) \quad \dots(ii)$$

Hence the total discharge through the complete flow net is given by

$$q = \Sigma \Delta q = k \cdot \frac{h}{N_d} \left(\frac{b}{l} \right) \cdot N_f$$

$$\text{or } q = kh \frac{N_f}{N_d} \cdot \frac{b}{l}$$

where N_f = total number of flow channels in the net.

The field is square, and hence $b = l$.

$$\text{Thus } q = kh \frac{N_f}{N_d} \quad \dots(10.6)$$

This is the required expression for the discharge passing through a flow net, and is valid only for isotropic soils in which

$$k_x = k_y = k.$$

3. SEEPAGE DISCHARGE FOR ANISOTROPIC SOIL.

Let us now consider the case of an anisotropic flow medium in which $k_x \neq k_y$.

For such a case, the flow equation 10.4 becomes

$$k_x \frac{\partial^2 h}{\partial x^2} + k_y \frac{\partial^2 h}{\partial y^2} = 0$$

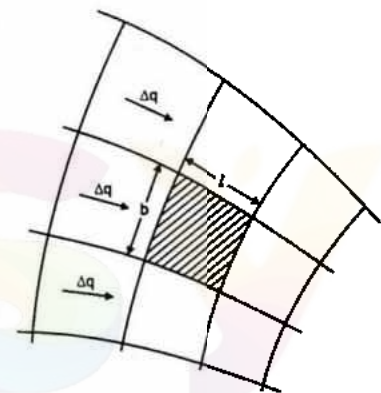


FIG. 10.16. PORTION OF A FLOW NET.

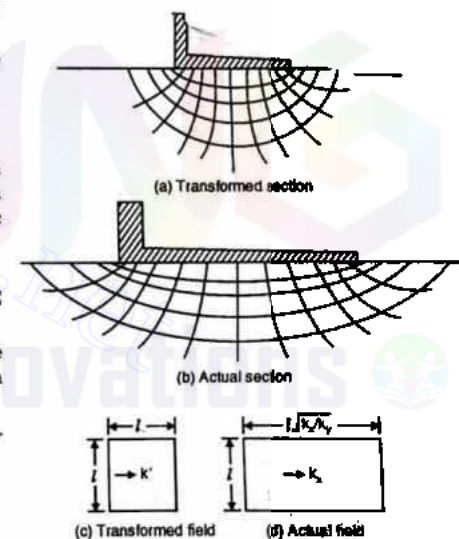


FIG. 10.17. ANISOTROPIC FLOW.

which is not a Laplacian equation. Hence flow net cannot be directly drawn. Rewriting it, we get

$$\frac{k_x}{k_y} \frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} = 0.$$

Let us put $x_n = x \sqrt{k_y/k_x}$

where x_n is the new co-ordinate variable in the x -direction.

Then the above equation becomes,

$$\frac{\partial^2 h}{\partial x_n^2} + \frac{\partial^2 h}{\partial y^2} = 0 \quad \dots(10.7)$$

This is in Laplacian form.

To plot the flow for such a case, the cross-section through anisotropic soil is plotted to a natural scale in the y -direction, but to a transformed scale in the x -direction, all dimensions parallel to x -axis being reduced by multiplying by the factor $\sqrt{k_y/k_x}$. The flow net obtained for this transformed section will now be constructed in the normal manner as if the soil were isotropic. The actual flow net is then obtained by re-transforming the cross-section including the flow net, back to the natural scale by multiplying the x -coordinates by a factor $\sqrt{k_x/k_y}$. The actual flow net, thus will not have orthogonal set of curves.

As shown in Fig. 10.17, field of the transformed section will be a square one, while the field of the actual section (re-transformed) will be rectangular one having its length in x -direction equal to $\sqrt{k_x/k_y}$ times the width in the y -direction.

Let k' = permeability coefficient in x -direction, of the actual anisotropic soil field.
 k' = equivalent permeability of the transformed field.

Then, for the transformed section.

$$\Delta q = k' \frac{\Delta h}{l} \quad (l \times 1) \quad \dots(1)$$

$$\text{For the actual field, } \Delta q = k_x \frac{\Delta h}{(l) \sqrt{k_x/k_y}} \quad (l \times 1) \quad \dots(2)$$

Since the quantity of flow is the same,

$$k' \frac{\Delta h}{l} (l) = k_x \frac{\Delta h}{(l) \sqrt{k_x/k_y}} (l)$$

$$\text{Hence } k' = k_x \sqrt{k_y/k_x} = \sqrt{k_x \cdot k_y} \quad \dots(10.8)$$

Hence the discharge is given by

$$q = k' h \frac{N_f}{N_d} = \sqrt{k_x \cdot k_y} h \frac{N_f}{N_d} \quad \dots(10.9)$$

4. PHREATIC LINE IN EARTH DAM.

Let us now consider the case of a homogeneous earth dam with a horizontal filter, as shown in Fig. 10.18. In order to draw the flow net, it is first essential to

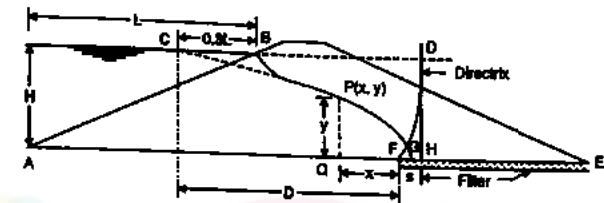


FIG. 10.18. CASAGRANDE'S METHOD OF DETERMINING PHREATIC LINE IN A DAM WITH HORIZONTAL DRAINAGE FILTER

find the location and shape of the phreatic line or the top flow line separating the saturated and unsaturated zones. The phreatic line or seepage line is defined as the line within a dam section below which there are positive hydrostatic pressures in the dam. The hydrostatic pressure on the phreatic line itself is atmospheric or zero. The phreatic line can be located by (i) analytical method, (ii) graphical method, and (iii) experimental method. We shall first discuss the graphical method of the determination of phreatic line by Casagrande method.

Casagrande assumed the phreatic line to be a base parabola with its focus at F, the starting point of the filter FE. The following is the procedure for locating the phreatic line graphically :

1. AB is the upstream face. Let its horizontal projection be L. On the water surface, measure the distance BC = 0.3 L. Then the point C is the starting point of the base parabola.

2. To locate the position of the directrix of the parabola, we utilise the principle that any point on the parabola is equidistant from the focus as well as directrix. Hence with the point C as the centre, and CF as the radius, draw an arc to cut the horizontal line through CB in D. Draw a vertical tangent to the curve FD at D. Evidently, CD = CF. Hence the vertical line DH is the directrix.

3. The last point G on the parabola will evidently lie midway between F and H.

4. In order to locate the intermediate points on the parabola we use the principle that its distances from focus and directrix must be equal. For example, to locate any point P, draw vertical line QP at any distance x from F. Measure the distance QH. With F as the centre and QH as the radius, draw an arc to cut the vertical line through Q in point P.

5. Join all these points to get the base parabola. However, correction is to be made at the entry point. The phreatic line must start from B, and not from C. Also the phreatic line is a flow line, and must start perpendicularly to the u/s face AB which is a 100 % equipotential line. Hence the portion of the phreatic line at B is sketched free hand in such a way that it starts perpendicularly to AB, and meets the rest of the parabola tangentially without any kink. The base parabola should also meet the d/s filter perpendicularly (i.e. vertically) at G.

In order to find the equation of the base parabola, consider any point P on it, with co-ordinate (x, y) with respect to focus F as origin.

From the property of the parabola, we have $PF = QH$

$\sqrt{(x^2 + y^2)} = QF + FH = x + s$ where $s = FH = \text{focal distance}$ (i)

From (i) $x^2 + y^2 = x^2 + s^2 + 2xs$... [10.10(a)]

$$y^2 = s^2 + 2xs$$

$$x = \frac{y^2 - s^2}{2s}$$

or $y^2 = 2xs + s^2$... (ii) ... [10.10(b)]

This is the equation of the base parabola.

In order to get an expression for the discharge q through the body of the dam for the present case of horizontal filter, we observe that, through the vertical section PQ ,

$$q = kiA = k \frac{\partial y}{\partial x} (y \times 1) \quad \dots (iii)$$

But from Eq. (ii), $y = (2xs + s^2)^{1/2}$ and $\frac{\partial y}{\partial x} = \frac{s}{(2xs + s^2)^{1/2}}$... (iv)

Substituting in (iii), we get

$$q = k \frac{s}{(2xs + s^2)^{1/2}} \times (2xs + s^2)^{1/2} \text{ or } q = ks \quad \dots (10.11)$$

This is very simple expression for discharge q in terms of focal distance s . The distance s can either be determined graphically (Fig. 10.18), or can be calculated analytically by co-ordinating coordinates of point C , as follows :

From (i) $s = \sqrt{x^2 + y^2} - x$

At C , $x = D$ and $y = H$

$$s = \sqrt{D^2 + H^2} - D$$

Hence $q = ks = k(\sqrt{D^2 + H^2} - D)$... [10.11(a)]

5. PHREATIC LINE FOR A DAM WITH NO FILTER

(a) General Solution by Casagrande. Fig. 10.19 shows a homogeneous dam with no horizontal drainage filter at the d/s side. The focus in this case will be the lowest point F of the d/s slope, and the base parabola BJG will evidently cut the d/s slope at J and extend beyond the limits of the dam, as shown by dotted line. However, according to exit conditions (Fig. 10.20 a), the phreatic

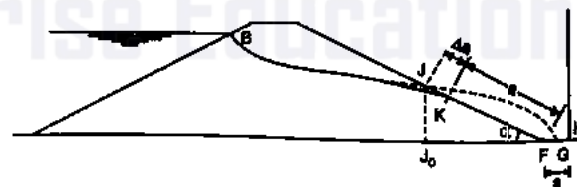


FIG. 10.19. DAM WITH NO DRAINAGE FILTER.

line must emerge out at some point K , meeting the d/s face tangentially there. The portion KF is then known as *discharge face* and always remains wet. The correction Δa , by which the parabola is to be shifted downwards, is found by the value of $\frac{\Delta a}{a + \Delta a}$ given by Casagrande for various values of the slope α of the discharge face. The slope α can even exceed the value of 90° , specially in case of rockfill toe, as shown in Fig. 10.20 (a).

Thus we observe that

$$\frac{\Delta a}{a + \Delta a} = \text{value found from Fig. 10.20(b) for given values of } \alpha \quad \dots (i)$$

$$a + \Delta a = JF = \text{value measured from Fig. 10.19} \quad \dots (ii)$$

Alternatively, from Fig. 10.19, we have

$$JF = (a + \Delta a) = J_0H = J_0F + FH = JF \cos \alpha + s$$

Hence $JF = \frac{s}{1 - \cos \alpha}$

From the above expression, JF (and hence $a + \Delta a$) can be computed.

Solving (i) and (ii), the value a and Δa can be found

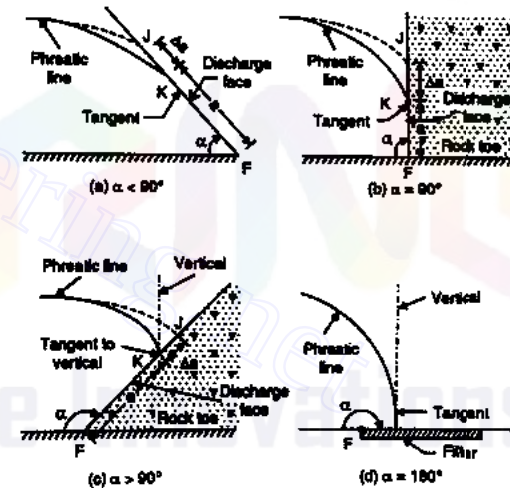
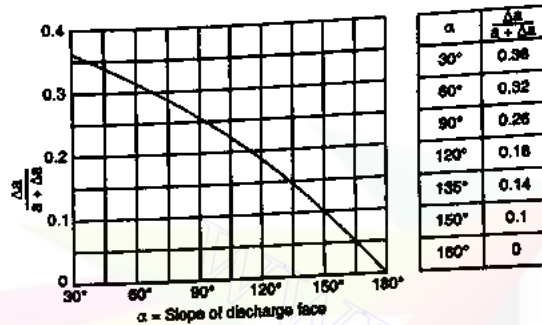


FIG. 10.20(a). EXIT CONDITIONS FOR VARIOUS SLOPES OF THE DISCHARGE FACE.

FIG. 10.20(b). RELATION BETWEEN α AND $\frac{\Delta a}{a + \Delta a}$.

(b) Analytical Solution of Schaffernak and Van Iterson for $\alpha < 30^\circ$. In order to find the value of a analytically, Schaffernak and Van Iterson assumed that the energy gradient $i = \tan \alpha = \frac{dy}{dx}$. This means that the gradient is equal to the slope of the line of seepage, which is approximately true so long as the slope is gentle (i.e. $< 30^\circ$).

For the vertical section KK_1 ,

$$q = k \frac{dy}{dx} y \quad \dots (i)$$

$$\text{But } \frac{dy}{dx} = i = \tan \alpha.$$

$$\text{and } y = KK_1 = a \sin \alpha$$

$$\text{Substituting in (i), we get } q = k a \sin \alpha \tan \alpha \quad \dots (10.12)$$

This is the expression for discharge.

$$\text{Again } q = k \frac{dy}{dx} y = k a \sin \alpha \tan \alpha$$

$$a \sin \alpha \tan \alpha dx = y dy$$

Integrating between the limits:

$$x = a \cos \alpha \text{ to } x = d$$

$$y = a \sin \alpha \text{ to } y = H,$$

and we get

$$a \sin \alpha \tan \alpha \int_{a \cos \alpha}^d dx = \int_{a \sin \alpha}^H y dy$$

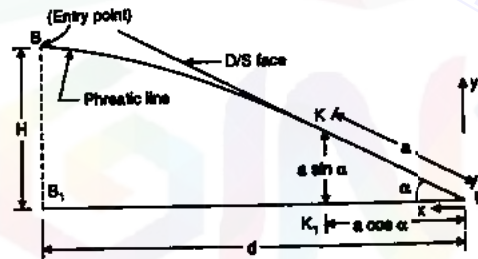


FIG. 10.21. SCHAFFERNAK'S METHOD.

$$\therefore a \sin \alpha \tan \alpha (d - a \cos \alpha) = \frac{H^2 - a^2 \sin^2 \alpha}{2}$$

From which, we obtain, after simplification,

$$a = \frac{d}{\cos \alpha} - \sqrt{\frac{d^2}{\cos^2 \alpha} - \frac{H^2}{\sin^2 \alpha}} \quad \dots (10.13)$$

(c) Analytical Solution by Casagrande for $30^\circ < \alpha < 60^\circ$.

It will be observed that the previous solution gives satisfactory results for slope $< 30^\circ$. For steeper slopes, the deviation from correct values increases rapidly beyond tolerable limits. Casagrande suggested the use of $\sin \alpha$ instead of $\tan \alpha$.



FIG. 10.22. CASAGRANDE'S METHOD.

$$\text{Thus } q = k i A = k \frac{dy}{ds} A \quad \dots (1)$$

$$\text{At } K, s = a, \text{ and } y = a \sin \alpha = A; \text{ hence } \frac{dy}{ds} = \sin \alpha$$

where s = distance measured along the curve.

Substituting in (1), we get

$$q = k \cdot \sin \alpha \cdot a \sin \alpha = k a \sin^2 \alpha \quad \dots (10.14)$$

which is the expression for the discharge.

$$\text{Again, } q = k \frac{dy}{ds} y = k a \sin^2 \alpha.$$

$$\therefore a \sin^2 \alpha ds = y dy$$

Integrating between the following limits

$$s = a \text{ to } s = S \text{ (where } S = \text{total length of the parabola)}$$

$$y = a \sin \alpha \text{ to } y = H, \text{ we get}$$

and

$$a \sin^2 \alpha \int_a^S ds = \int_{a \sin \alpha}^H y dy$$

$$\therefore a \sin^2 \alpha \cdot (S - a) = \frac{H^2 - a^2 \sin^2 \alpha}{2}$$

or

$$a^3 - 2aS + \frac{H^2}{\sin^2 \alpha} = 0$$

from which

$$a = S - \sqrt{S^2 - \frac{H^2}{\sin^2 \alpha}} \quad \dots (10.15)$$

Taking

$$S \triangleq (H^2 + a^3)^{1/2} \text{ for } \alpha \text{ upto } 60^\circ, \text{ we get}$$

$$a = \sqrt{H^2 + d^2} - \sqrt{H^2 + d^2} \frac{H^2}{\sin^2 \alpha}$$

$$a = \sqrt{d^2 + H^2} - \sqrt{d^2 + H^2} \cot^2 \alpha \quad \dots [10.15(a)]$$

or

For $\alpha = 90^\circ$, Eq. 10.15 (a) becomes

$$a = \sqrt{d^2 + H^2} - d \quad \dots (10.15 b)$$

which provides value of a upto 25% deviations from the actual values.

6. CHARACTERISTICS OF PHREATIC LINE (SEEPAGE LINE).

Based on the above discussions, the characteristics of the phreatic line may be summarised below:

1. At the entry point, the phreatic line must be normal to the upstream face since the upstream face is a 100 % equipotential line. For other entry conditions (Fig. 10.23), the phreatic line starts tangentially with the water surface.

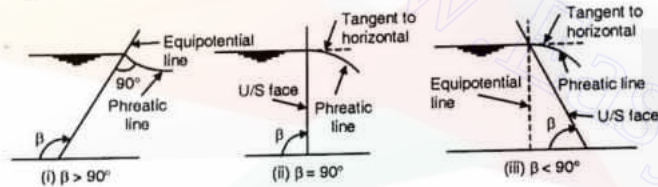


FIG. 10.23. ENTRY CONDITIONS OF PHREATIC LINE.

2. The pressure along the phreatic line is atmospheric. Hence the only change in the head along it is due to drop in the elevation of various points on it. Due to this, the successive equipotential lines will meet it at equal vertical intervals.

3. The focus of the base parabola lies at the break out point of the bottom flow line, where the flow emerges out from relatively impervious medium to a highly pervious medium.

4. When horizontal filter or drainage toe is provided, the phreatic would tend to emerge vertically.

5. In the absence of any filter, the seepage line will cut the downstream slope at some point above the base. The location of this point, and the phreatic line itself, is not dependent on the permeability or any other property, so long as the dam is homogeneous. The geometry of the dam alone decides these.

6. The presence of pervious foundation below the dam does not influence the position of phreatic line.

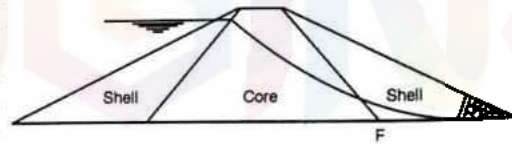


FIG. 10.24. INFLUENCE OF CENTRAL CORE ON PHREATIC LINE.

7. In the case of a zoned dam with central impervious core, the effect of outer shells can be neglected altogether. The focus of the base parabola will be located at the downstream toe of the core (Fig. 10.24).

7. GRAPHICAL DETERMINATION OF FLOW NET.

After having located phreatic line in an earth dam the flow net can be plotted by trial and error by observing the following properties of flow net (Fig. 10.25), and by following the practical suggestions given by A. Casagrande¹.

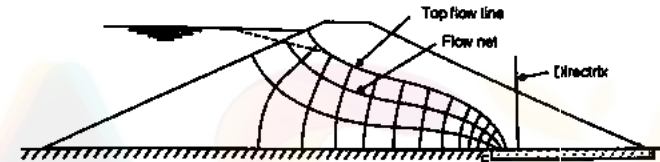


FIG. 10.25. FLOW NET BY GRAPHICAL METHOD.

Properties of flow net. 1. The flow lines and equipotential lines meet at right angles to each other.

2. The fields are approximately squares, so that a circle can be drawn touching all the four sides of square.

3. The quantity flowing through each flow channel is the same. Similarly, the same potential drop occurs between two successive equipotential lines.

4. Smaller the dimensions of the field, greater will be the hydraulic gradient and velocity of flow through it.

5. In a homogeneous soil, every transition in the shape of curves is smooth, being either elliptical or parabolic in shape.

Arthur Casagrande¹ gives the following excellent hints for the beginner in flow net sketching :

1. Use every opportunity to study the appearance of well constructed flow nets. When the picture is sufficiently absorbed in your mind, try to draw the same flow net without looking at the available solution ; repeat this until you are able to sketch this flow net in a satisfactory manner.

2. Four or five flow channels are usually sufficient for the first attempt ; the use of too many flow channels may distract the attention from the essential features.

3. Always watch the appearance of the entire flow net. Do not try to adjust details before the entire flow net is approximately correct.

4. The beginner usually makes the mistake of drawing too sharp transitions between straight and curved sections of flow lines or equipotential lines. Keep in mind that all transitions are smooth; of elliptical or parabolic shape. The size of the squares in each channel will change gradually.

¹A. Casagrande "Seepage Through Dams", J. New Eng. Water Works Association, June, 1937, pp. 136-137.

8. FLOW NET BY ELECTRICAL ANALOGY.

The Darcy's law governing the flow of water through soil is analogous to Ohm's law governing the flow of electric current through conductor. The corresponding analogous quantities are shown in Table 10.4.

TABLE 10.4

Darcy's law for water seepage	Ohm's law for electric flow
$Q = k \frac{H}{l} A$	$I = C \frac{E}{l} A$
Q = quantity of seepage	I = rate of flow of electricity
k = coefficient of permeability	C = electric conductivity coefficient
A = cross-sectional area	A = cross-sectional area
H = pressure head	E = electric potential
l = length of seepage path	l = length of path of electric current

Thus, solutions to seepage problems can be obtained with electric models which have the same geometric shape as the soil through which the water flows. The seepage medium is replaced by an electric conductor consisting of water with some salt or dilute hydrochloric acid. The boundary equipotentials (i.e. 100 % equipotential line at u/s face and 0 % equipotential line at the d/s filter) are made of copper strips. The boundary flow lines (i.e. the phreatic line and the bottom flow line) are simulated by non-conducting strips such as ebonite or perspex etc. An alternating voltage (generally of value 5 to 20 volts) is applied across the boundary equipotential strips. A potential divider is connected in parallel with the alternating current source. Fig. 10.26 shows the electrical analogy tray, the model and the complete circuit diagram.

To determine a line of contour of equal potentials, the potentiometer is adjusted to percentage of the total voltage drop and the probe of a galvanometer is used to find the corresponding balance points (null points) on the model. Changes in the coefficient of permeability of soil zones in the seepage analogue are simulated by changes in the electrical conductivity coefficient in the model. When once the equipotential lines are obtained, orthogonal flow lines conforming to the boundary conditions are then drawn as in the graphical method. Fig. 10.27 shows a typical flow net for steady seepage case for a earth dam having the same foundation material as that of the body

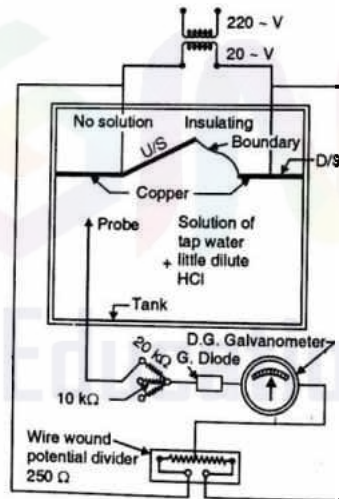


FIG. 10.26. CIRCUIT DIAGRAM FOR ELECTRICAL ANALOGY TRAY (AFTER ALAM SINGH & B.C. PUNMIA).

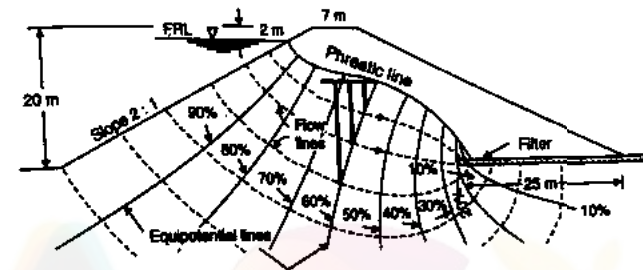


FIG. 10.27 FLOW NET FOR STEADY SEEPAGE CASE

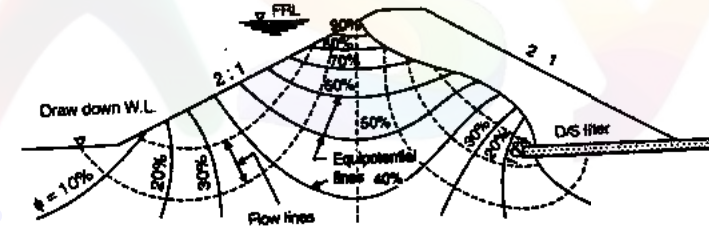


FIG. 10.28 FLOW NET FOR SUDDEN DRAWDOWN CASE (AFTER ALAM SINGH AND B.C. PUNMIA).

of the dam. Fig. 10.28 shows the flow net for the sudden drawdown case for the same dam.

10.7. STABILITY ANALYSIS

We shall be discussing the stability analysis under the following four heads :

1. Stability of downstream slope during steady seepage.
2. Stability of upstream slope during sudden drawdown.
3. Stability of upstream and downstream slopes during and immediately after construction.
4. Stability of foundation against shear.

SWEDISH CIRCLE METHOD OF SLOPE STABILITY.

Of the numerous methods of slope stability which have been devised over the last 40 years, the Swedish Slip Circle Method of stability analysis is the one most generally accepted. In this method the potential failure surface is assumed to be cylindrical, and the factor of safety against sliding is defined as the ratio of the average shearing

¹Alam Singh and B.C. Punmia, 'Critical slope stability of an earthen dam', 2nd Annual No. Institution of Engineers (India), Rajasthan Centre, Jan. 1962.

strength, as determined by the Coulomb's equation $s = c + \sigma \tan \phi$ to the average shearing stress determined by statics on the potential sliding surface.

In order to test the stability of the slope, a trial slip circle is drawn, and the soil material above assumed slip surface is divided into a convenient number of vertical strips or slices. The forces between the slices are neglected and each slice is assumed to act independently as a column of soil of unit thickness and width b . The weight W of each slice is assumed to act at its centre. If this weight of each slice is resolved in normal (N) and tangential (T) components, then the normal component will pass through the centre of rotation (O), and hence does not cause any driving moment on the slice. However, the tangential component T cause a driving moment $= (T \times r)$, where r is the radius of the slip circle. The tangential components of the few slices at the base may cause resisting moment; in that case T is considered negative.

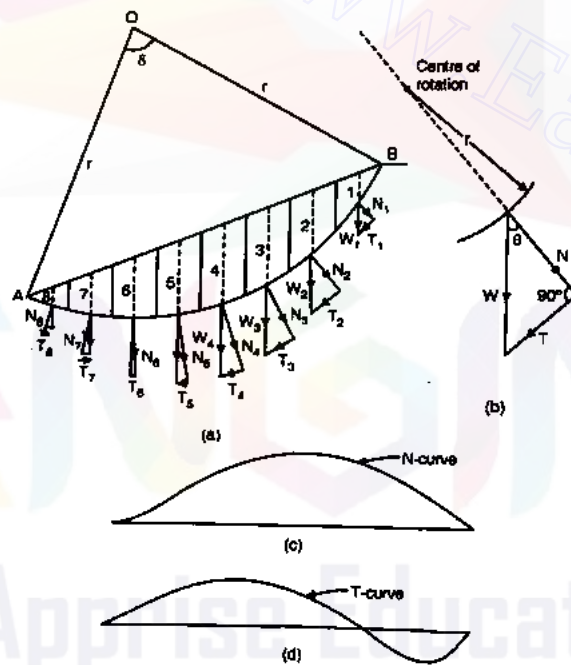


FIG. 10.29 SLIP CIRCLE METHOD

If c is the unit cohesion and ΔL is the curved length of each slice, then the resisting force, from Coulomb's equation is

$$= c \cdot \Delta L + N \tan \phi$$

For the entire slip surface AB , we have

$$\text{Driving moment } M_d = r \Sigma T$$

$$\text{Resisting moment } M_r = r \cdot [c \cdot \Sigma \Delta L + \tan \phi \Sigma N]$$

where ΣT = sum of all tangential components

ΣN = sum of all normal components

$$\Sigma \Delta L = \hat{L} = \frac{2\pi r \delta}{360^\circ} = \text{length } AB \text{ of slip circle.}$$

Hence factor of safety against sliding is

$$F = \frac{M_r}{M_d} = \frac{c \hat{L} + \tan \phi \Sigma N}{\Sigma T} \quad \dots(10.16)$$

In order to find ΣN and ΣT , N -curves and T -curves are drawn (Fig. 10.29(c), (d)) by making the ordinates of these diagrams equal to N and T values for different strips and joining them by smooth curves. The area of these diagrams can then be measured by a planimeter and ΣN and ΣT can be computed.

A simplified *Rectangular Plot Method* has been suggested by Prof. Alam Singh.¹ Referring to Fig. 10.30, let $z_1, z_2, \dots$ be the boundary ordinates of the slices. Let the width of each slice be b , and the width of the last slice be mb . The total weight ΣW of the sliding sector may be written as

$$\Sigma W = b \left\{ z_1 + z_2 + \dots + \left(\frac{1+m}{2} \right) z_n \right\}$$

(in terms of area)

This shows that the weight can be taken corresponding to the end ordinates of each slice. The weight can be resolved into corresponding various N -values (i.e. $N_1, N_2, \dots$ etc.) as abscissa and the constant width b as the ordinate. Similarly, T -rectangle can be constructed. Then ΣN = area of N -rectangle and ΣT = area of T -rectangle.

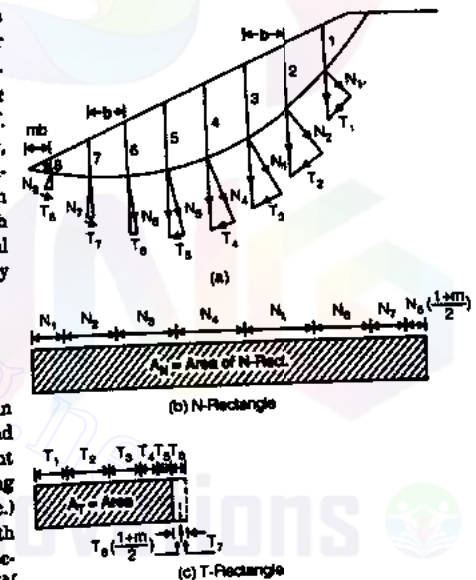


FIG. 10.30. RECTANGULAR PLOT METHOD.

¹Alam Singh : "A Rectangular Plot Method for Slope Stability Analysis", Journal, Indian National Society of Soil, and Mech. and Found. Engrs., Vol 1, No. 2, pp. 18-21, April 1962.

It should be clearly noted that in the rectangular plot method, if there are n -slices, the total number of ordinates will be $(n-1)$, the ordinate at the end of the last strip (9) being zero. If A_N = area of N -rectangle, in sq.cm, γ = unit weight of the soil, and the scale of drawing is 1 cm = x metres, then the area in sq. metres = $A_N \cdot x^2$ and volume $V = A_N \cdot x^2 \times 1 \text{ m}^3$.

Hence $\Sigma N = A_N \cdot x^2 \gamma$

Similarly $\Sigma T = A_T \cdot x^2 \gamma$

The Third method of computing ΣN and ΣT is with the help of Table shown below :

TABLE 10.5

Slice No.	W	$N = W \cos \alpha$	$T = W \sin \alpha$
1	W_1	N_1	T_1
2	W_2	N_2	T_2
3	W_3	N_3	T_3
.	.	.	.
n	W_n	N_n	T_n
sum		$\Sigma N =$	$\Sigma T =$

Method of locating centre of critical slip circle. In order to reduce the number of trials to find the centre of critical slip circle, Fellenious has given a method of locating the locus on which the probable centre may lie. For homogeneous $c-\phi$ soil, centre of critical slip circle lies on a line PQ , in which the point Q has its coordinates H downwards from toe and $4.5 H$ horizontally away, as shown in Fig. 10.31.

The other point P is obtained with the help of directional angles and given in the Table below :

TABLE 10.6

Slope	Slope angle (degrees)	Directional angles (degrees)	
		α	β
0.58 : 1	60	29	40
1 : 1	45	28	37
1.5 : 1	33.8	26	35
2 : 1	26.6	25	35
3 : 1	18.4	25	35
5 : 1	11.3	25	35

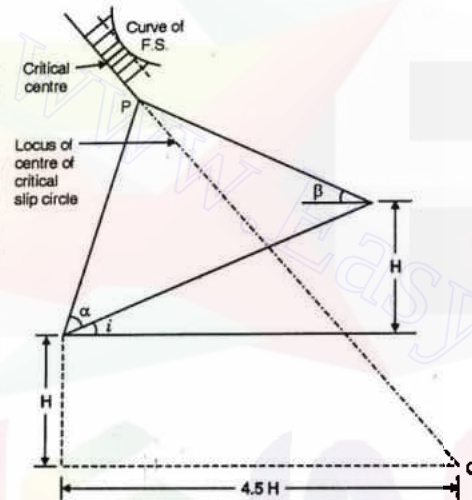


FIG. 10.31. FELLENIUS METHOD.

When the line PQ is obtained, trial centres are selected on it. According to Fellenious, the centre of critical slip surface is located at the point P for purely cohesive soil ($\phi = 0$). However for $c-\phi$ soil, the centre of critical slip surface lies above P , on the line $Q-P$ produced. The factor of safety corresponding to each centre is calculated from Eq. 10.16. These various factors of safety so obtained are plotted as ordinates (Fig. 10.31) on the corresponding centres, and a smooth curve is obtained. The centre corresponding to the lowest factor of safety is then the critical centre.

Centre of critical circle for zoned dams

The above method of locating the centre of critical slip circle, given by Fellenious, is applicable only to simple finite slopes of homogeneous soils. The method is therefore not directly applicable to zoned earth dams consisting of outer shells and inner core, having different properties. However, the number of trials can be considerably reduced if the position of the centre of the first trial slip surface is taken at point P (Fig. 10.31) and the centres of other slip circles are taken in its neighbourhood. In these trials, usually a (3×3) matrix of nine trial centres, including point P in the centre is considered. All the points in this nine-point matrix are taken equidistant in the near vicinity around point P , and the factor of safety for each such trial surface is computed. The point which gives the minimum factor of safety is selected out of these nine points. With this selected point in the centre, a new matrix of 9 points is selected, including this point and the factor of safety for each trial slip circle is found. The point which gives a minimum factor of safety is then selected and the process is repeated. Usually two or three such trials are sufficient for locating the most critical centre.

Critical circle passing below the toe

For most of earth dams in practice, the critical slip circle passes through the toe of the dam. However, in the following mentioned circumstances, the slip surface may pass through the foundations, below the toe of the dam :

(i) For small values of ϕ (usually below 10° to 15°) the critical circle may pass below the toe of the slope if the inclination of the slope is less than 53° .

(ii) If the soils of the dam and the foundation are homogeneous, the critical slip surface may pass below the toe.

(iii) If the foundation soil has the same or lesser shear strength than that of the dam, the critical slip surface may pass below the toe of the slope.

For such cases, the centre P of critical slip surface is located on a vertical line through the mid-point of the slope. Fellenious found that the critical slip surface for such cases subtends an angle of $133\frac{1}{2}^\circ$ at the centre.

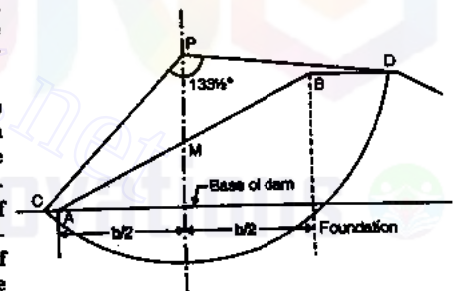


FIG. 10.32. CENTRE OF CRITICAL SLIP CIRCLE PASSING BELOW THE TOE.

Procedure

- (i) Draw vertical line through the mid-point (M) of the side slope.
- (ii) Select a trial centre P on the vertical line such that when a possible slip surface is drawn with P as the centre, the slips surface subtends an angle of $135\frac{1}{2}^\circ$ at P . Compute the factor of safety.
- (iii) Move the next trial centre slightly to the left of P , and draw a new slip surface with reduced radius such that the circle subtends an angle of $135\frac{1}{2}^\circ$ at the centre of the circle. Compute the factor of safety
- (iv) Repeat the procedure with a new centre, till minimum factor of safety is obtained.

10.8. STABILITY OF DOWNSTREAM SLOPE DURING STEADY SEEPAGE

Critical condition for d/s slope occurs when the reservoir is full and percolation is at its maximum rate. The direction of seepage forces tend to decrease stability. In other words, the pore water pressure (U) acting on the soil mass below the saturated line reduces the effective stress responsible for mobilising shearing resistance. The factor of safety in this case is given by

$$F.S. = \frac{c \bar{L} + \tan \phi \Sigma (N - U)}{\Sigma T} \quad \dots(10.17)$$

when ΣU is the total pore pressure on the slip surface.

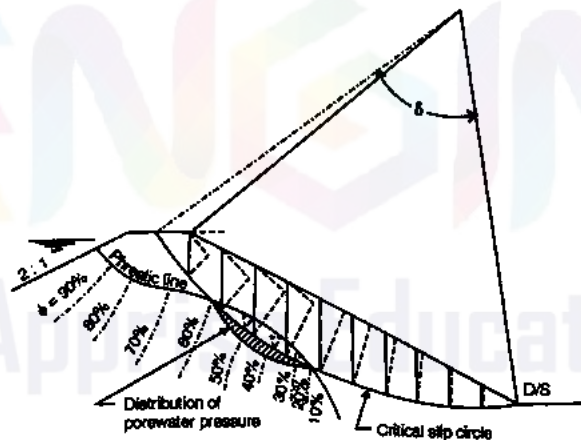


FIG. 10.33. STABILITY OF D/S SLOPE DURING STEADY SEEPAGE.

The pore-water pressure at any point is represented by the piezometric head (h_w) at that point. Thus the variations of pore water pressure along a likely slip surface

is obtained by measuring at each of its intersections with an equipotential line, the vertical height from that intersection to the level at which the equipotential line cuts the phreatic line. The pore pressure represented by vertical height so obtained is plotted to scale in a direction normal to the sliding surface at the respective point of intersection. The distribution of pore-water pressure on the critical slip surface during steady seepage is shown hatched in Fig. 10.33. The area of U-diagram can be measured with the help of a planimeter, or else the rectangular plot method may be utilised. If A_U is the area of U-rectangle, then $\Sigma U = A_U \times x^2 \gamma_w$ where x is the scale of the drawing (i.e. 1 cm = x metres).

In the absence of a flow net, the F.S. of the d/s slope can approximately be calculated from the equation

$$F.S. = \frac{c \bar{L} + \tan \phi \Sigma N'}{\Sigma T} \quad \dots(10.18)$$

where the normal components N' are to be calculated on the buoyant unit weight (γ') of the dam, while T components are to be calculated on the basis of saturated weight.

10.9. STABILITY OF UPSTREAM SLOPE DURING SUDDEN DRAWDOWN**(A) Solution with the Help of Flow Net and Pressure Net.**

For the u/s slope, steady seepage does not represent the critical state, because the seepage pressure then acts inwards from this slope and tends to increase the stability on the upstream side. For the u/s slope, the critical condition is when the reservoir is suddenly emptied without allowing any appreciable change in the water level within the saturated mass of soil. This state is known as sudden drawdown. With complete drawdown, the hydrostatic force acting along the u/s slope at the time of full reservoir is removed. The weight of water within the soil now tends to help a sliding failure, without the hydrostatic pressure on the slope to counteract it. Or, by another interpretation, while the disturbing force is the tangential component of the saturated weight, the shear resistance is considerably reduced on account of development of pore water pressure

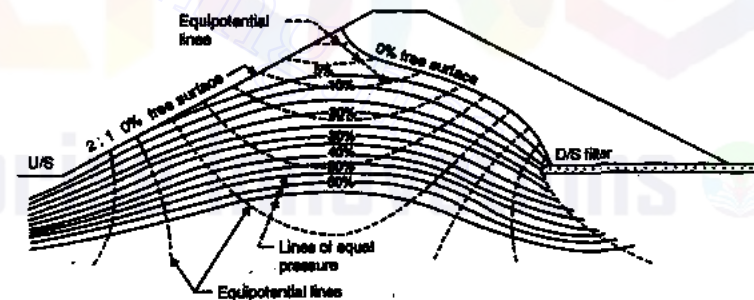


FIG. 10.34. PRESSURE NET FOR SUDDEN DRAWDOWN (AFTER ALAM SINGH & B.C. PUNMIA)

on a likely slip surface. The effect of drawdown on slope stability varies appreciably with opportunity for drainage at the base. If the base is quite impervious, flow lines tend to be horizontal and in outward direction towards the slope, which is quite unfavourable to stability. On the other hand, if the base material is pervious, flow pattern tends to be downwards and this is more favourable with respect to stability.

The magnitude and distribution of pore water pressure on a likely slip surface is estimated from a *pressure-net* which is developed from the flow net. Fig. 10.34 represents the pressure-net for sudden drawdown case. The pressure-net gives lines of equal piezometric heads (h_w) expressed in terms of percentage of total hydraulic heads H when the reservoir is full. At any point, we have

$$\Phi = h_w + y \quad \dots(1)$$

where Φ = hydraulic potential at the point, expressed as percentage of total head H .

h_w = piezometric head, expressed as percentage of total head H .

y = elevation head, expressed as percentage of total head H .

If we want to plot the 'line of equal pressure' corresponding to $h_w = 20\%$ (say), we have

$$h_w = 20 = \Phi - y$$

To find a point corresponding to $h_w = 20\%$ on $\Phi = 30\%$, we get from Eq. (2), $y = 30 - 20 = 10\%$ of H . Thus the point is located. Similarly, to get another point of $h_w = 20\%$ on $\Phi = 40\%$, $y = 40 - 20 = 20\%$ of H and so on. The various points so obtained

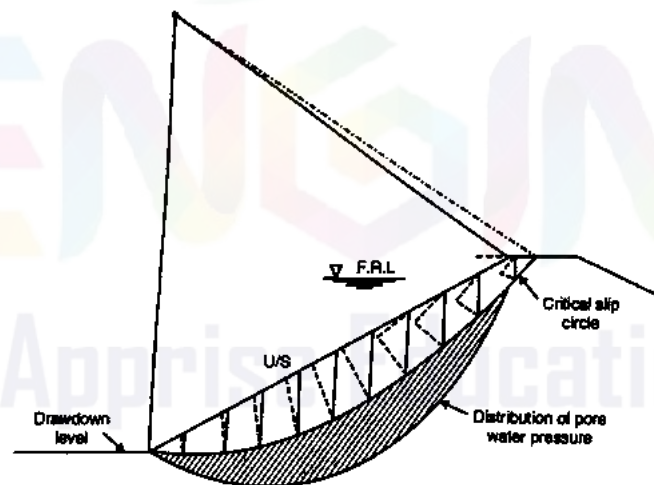


FIG. 10.35. STABILITY ANALYSIS DURING DRAWDOWN.

may be joined by a smooth curve to get a contour for $h_w = 20\%$. In this way, a pressure net may be prepared (Fig. 10.34).

In order to plot the distribution of pore water pressure on the assumed slip surface, the pressure net of Fig. 10.34 is superimposed on the diagram showing the location of slip surface. At the intersection points of the slip surface with the contours of pressure net, radial ordinates of the values of h_w 's of corresponding contours are erected. The curve joining the ends of these ordinates give the distribution of pore water pressure, as illustrated in Fig. 10.35. The F.S. is then calculated by the expression:

$$F.S. = \frac{c \bar{L} + \tan \phi \Sigma(N - U)}{\Sigma T}$$

(B) Approximate Analysis. In the absence of the flow net the factor of safety can be calculated from the expression.

$$F.S. = \frac{c \bar{L} + \tan \phi \Sigma N'}{\Sigma T}$$

in which the N' components are computed with respect to the submerged density (γ') of the u/s slope, while T components are calculated with respect to the saturated density.

(C) Analysis For Sloughing of Upstream Slope During Sudden Drawdown. This is another approximate method for finding the factor of safety against sliding or sloughing of u/s slope during sudden drawdown. Refer Fig. 10.36.

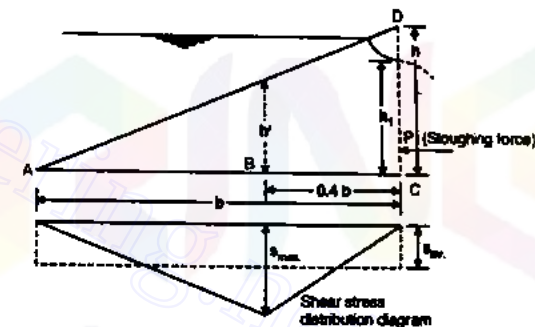


FIG. 10.36. SLOUGHING OF U/S SLOPE DURING SUDDEN DRAWDOWN.

- Let γ' = submerged unit weight of the material in u/s portion of dam.
 γ_s = saturated unit weight of the material in u/s portion of dam
 b = horizontal length of the slope AD
 P = total horizontal shear or sloughing force on the upstream portion of dam
 s_{av} = average unit shear over upstream part of dam
 s_{max} = maximum unit shear over upstream part of dam

h, h_1, h' = heights as marked in Fig. 10.36.

Then the horizontal shear force is given by

$$P = \frac{\gamma_s h^2}{2} \tan^2 \left(45^\circ - \frac{\phi}{2} \right) + \frac{\gamma_w h_1^2}{2} \quad \dots(10.19)$$

$$s_{av} = \frac{P}{b} \quad \dots(10.20(a))$$

and

$$s_{max} = 2 s_{av} = \frac{2P}{b} \quad \dots(10.20(b))$$

The maximum shear occurs at a distance of $0.4 b$ from the w/s point, as shown.

$$\text{Let } R = \text{shearing resistance} = N_c \tan \phi + c \cdot b \quad \dots(10.21)$$

where $N_c = \gamma' A$

and $A = \text{area of w/s slope} = \frac{1}{2} b h$

$$\therefore \text{Factor of safety} = \frac{R}{P} \quad \dots(10.22)$$

This should be greater than 2.

(Note. If the analysis is done by slip circle method, factor of safety may be taken as 1.5.)

The above factor of safety is with respect to average shear. In order to find the factor of safety with respect to maximum shear, we have

$$s_{max} = 2 s_{av} = \frac{2P}{b} \quad (\text{at point B})$$

$$s_r = \text{shearing resistance at } B = h' \gamma' \tan \phi + c = 0.6 h' \gamma' \tan \phi + c \quad \dots(10.23)$$

$$\therefore \text{F.S. at the point of maximum shear} = \frac{s_r}{s_{max}} \quad \dots(10.24)$$

This should be greater than 1.

10.10. STABILITY OF U/S AND D/S SLOPES DURING CONSTRUCTION

When a dam is built of relatively impervious compressible soil, excess pore pressures develop in air and water entrapped in the pore space. This is because the soil mass undergoes a change in the volume due to compaction of the dam during construction and also due to its own weight. In due course of time, this initial excess pore pressure gets gradually dissipated. As the pore-pressure greatly affects the shear strength of soil, it is essential to know its magnitude for stability analysis. The estimation of the construction pore pressure is made with the help of Hilf's equation¹:

$$u = \frac{p_a \Delta}{V_a + h_c V_w - \Delta} \quad \dots(10.25)$$

where u = induced pore pressure

¹Hilf, J.W. "Estimating Construction Pore-pressure in Rolled Earth Dam", Proc. 2nd Int. Conf. Soil Mech., Vol. 3, p.3, p. 234, Rotherdam, July 1948.

p_a = air pressure in the voids of soil mass after initial compaction (absolute; considered to be atmospheric; should be corrected to site elevation)

Δ = embankment compression, in percent of original total embankment volume

V_a = volume of free air voids, in percent of original total embankment volume

V_w = volume of pore water, in percent of original total embankment volume

h_c = Henry's constant of solubility of air in water by volume (= 0.02 at 68° F).

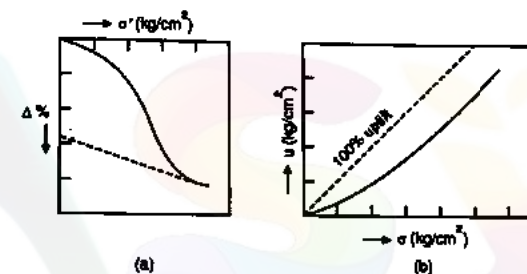


FIG. 10.37. CONSOLIDATION TEST DATA FOR ESTIMATING CONSTRUCTION PORE PRESSURE.

For use in this method a graph is plotted between effective stress σ' and percent consolidation (Δ), as obtained from consolidation test. Fig. 10.37 (a) gives a sample consolidation/load curve. The value of pore pressure u , for various values of Δ are calculated from Eq. 10.25. Knowing effective stress σ' and pore pressure u , the total stress σ is obtained by the relation $\sigma = \sigma' + u$. The results are tabulated as shown in Table 10.7. A plot is then made between pore water pressure (u) and total stresses (σ) as shown in Fig. 10.37 (b).

TABLE 10.7

Consolidation Δ (%)	Effective Stress σ' (kg/cm ²)	Pore-pressure u (kg/cm ²)	Total stress σ (kg/cm ²)
1	0.7	0.065	0.765
2	1.3	0.139	1.439
4	2.3	0.322	2.622
6	3.0	0.576	3.576
8	3.6	0.948	4.548
10	4.1	1.560	5.660
12	4.9	2.680	7.580
14	6.0	6.160	12.160
16	7.9	38.20	46.10

In order to find the construction pore-pressure at the bottom of various slices (Fig. 10.29), the mid-height, $z_1, z_2, \dots, z_n$ of each strip is measured and the total stress

σ (i.e., $\sigma_1, \sigma_2, \dots, \sigma_n$) is calculated. Then, from Fig. 10.37(a), the values of u corresponding to various values of σ are found. The results are tabulated as shown Table 10.8

TABLE 10.8

Slice No.	Height z	Total stress $\sigma = \gamma z$	u
		Sum	Σu

Hence $\Sigma U = (\Sigma u) \times$ (width of strip). Alternatively, using ordinates of u from Table 10.8, pore pressure distribution diagram can be plotted on the slip circle and ΣU can be calculated from its area A_u

Knowing ΣU , the factor of safety of the slope is calculated by the equation :

$$F.S. = \frac{cL + \tan \phi (\Sigma N - \Sigma U)}{\Sigma T}$$

10.11. STABILITY OF FOUNDATION AGAINST SHEAR

Foundations consisting of fine, loose, cohesionless materials or of unconsolidated clays and silts may be very weak in shear and may require thorough investigation. The present method of finding factor of safety of foundation against horizontal shear is an approximate one, and is based on the assumption that earth material has an equivalent liquid unit weight which would produce the same shear stress as the material itself.

If P = total horizontal shear down to rigid boundary, then

$$P = \frac{h_1^2 - h_2^2}{2} \gamma_m \tan^2 \left(45^\circ - \frac{\phi_1}{2} \right) \quad \dots (10.26)$$

where $\gamma_m \tan^2 \left(45^\circ - \frac{\phi_1}{2} \right)$ = equivalent liquid unit weight

ϕ_1 = equivalent angle of friction, given by

$$\tan \phi_1 = \frac{\gamma_m h_1 \tan \phi + c}{\gamma_m h_1} \quad \dots (10.27)$$

where ϕ and c are the shear parameters for the foundation material.

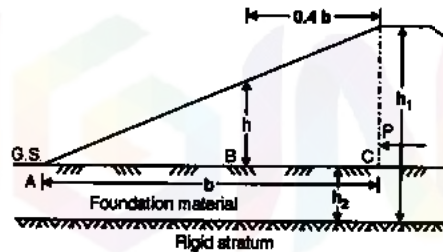


FIG. 10.38. FOUNDATION SHEAR.

γ_m = mean unit weight of the dam and foundation weighted in proportion to the depth of each.

$$\text{i.e. } \gamma_m = \frac{\gamma_D(h_1 - h_2) + \gamma_F h_2}{h_1}$$

where γ_D = unit weight of dam material

γ_F = unit weight of foundation material.

$$\text{Now average unit shear } = s_{av} = \frac{P}{b}$$

Maximum unit shear $= s_{max} = 2 s_{av}$ and occurs at $0.4 b$ from point C.

Let s_1 = unit shear strength below toe (at A) $= c + \gamma_F h_2 \tan \phi$

s_2 = unit shear strength at point C $= c + \gamma_m h_1 \tan \phi$

$$\therefore \text{Average shear strength } s = \frac{s_1 + s_2}{2}$$

Hence overall factor of safety against shear $= \frac{s}{s_{av}}$.

This should be greater than 1.5.

In order to get factor of safety at maximum shear, let

s = unit shear strength at point of max shear (B) $= c + \gamma_{av} h \tan \phi$

where γ_{av} = mean effective unit weight at B $= \frac{\gamma_D h + \gamma_F h_2}{h + h_2}$

$$\therefore F.S. = \frac{s}{s_{max}}. \text{ This should be greater than 1.}$$

10.12. DESIGN TO SUIT AVAILABLE MATERIALS

The section of a zoned earth dam should be so selected that the available materials are utilised to the maximum and limited quantity of other required materials are imported to the site. The available materials may be classified into three heads :

1. Gravel or coarse sand is available along with clayey silt
2. Only fine gravel or coarse sand is available
3. Only silty clay is available.

In addition to these, three general foundation conditions may be there :

- (a) Foundations impervious to a large depth
- (b) Foundations pervious to moderate depth, after which impervious strata is available.
- (c) Foundations pervious to large depth.

We shall treat each one of these conditions separately.

CASE 1. Gravel or Coarse Sand Available along with Clayey Silt.

(a) *Foundations impervious to a large depth.* Figs. 10.39(a) and (b) show the typical sections when the impervious or hard stratum is exposed to the surface, and the foundations are impervious to a large depth. The outer shells are made of the

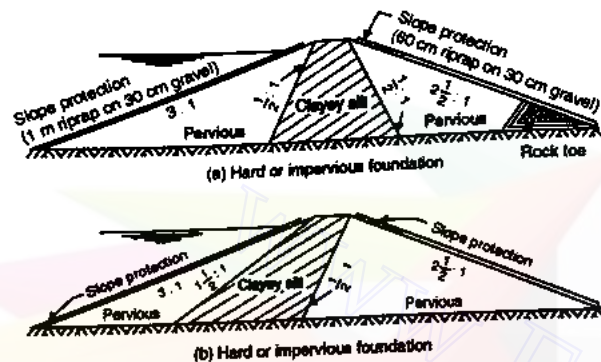


FIG. 10.39

pervious material available at the nearby site. The central core of clayey silt may have its axis either vertical or inclined.

(b) *Foundation pervious to a moderate depth, after which impervious stratum is available.* The construction is similar to the previous case, except that the central core is taken upto impervious foundation level, as shown in Fig. 10.40.

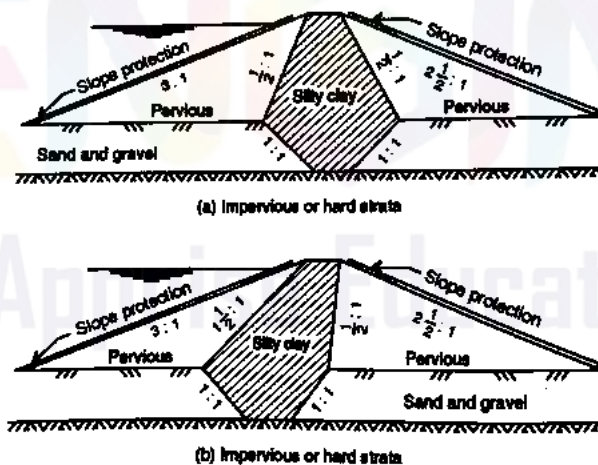


FIG. 10.40.

(c) *Foundation pervious to a great depth.* In this case there is a possibility of large seepage through foundations. Hence some form of cutoff is to be provided. If sufficient impervious material is available, an upstream clay blanket, such as shown in Fig. 10.41(a) is suggested. Otherwise, a sheet pile cutoff may be provided to some reasonable depth. Complete cutoff through foundation is not possible.

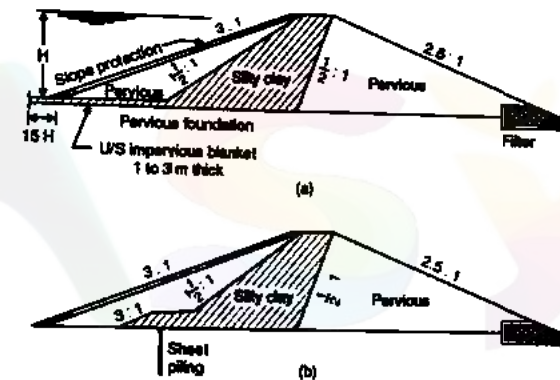


FIG. 10.41

CASE 2. Only Pervious Material Available

(a) *Foundation impervious to a great depth.* Since the impervious material is to be imported, thin central core is provided as shown in Fig. 10.42.



FIG. 10.42

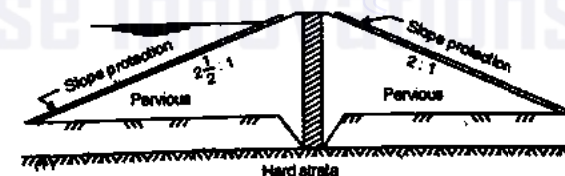


FIG. 10.43

(b) *Foundation pervious to moderate depth, after which impervious stratum is available.* The central thin core, is taken up to the impervious stratum as shown in Fig. 10.43.

(c) *Foundation pervious to a great depth.* In this case, a thin central core is provided along with an upstream blanket of clayey silt (Fig. 10.44).

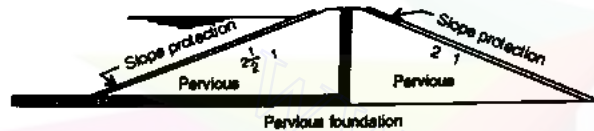


FIG. 10.44.

CASE 3. Only Silty Clay is Available

(a) *Foundation impervious to a great depth.* The whole dam will be homogeneous, made of silty clay.

However, a horizontal drainage filter will be provided upto a great distance inside the body of the dam (Fig. 10.45).



FIG. 10.45.

(b) *Pervious Foundation.* If the foundation is pervious and impervious material is available the whole dam may be made homogeneous, consisting of silty clay with a provision of sheet pile cutoff and a long horizontal filter (Fig. 10.46).



FIG. 10.46.

10.13. SLOPE PROTECTION

The upstream slope should be protected against the wave action and the downstream slope should be protected against ruin. In some instances, provision must be made against burrowing animals. Following are the various materials used for slope protection:

1. Rock riprap (a) Dry dumped (b) Hand packed

2. Concrete pavement
3. Bituminous pavement
4. Precast concrete blocks.

One of the most comprehensive and reliable collection of data on slope protection performance was gathered by the U.S. Corps of Engineers during 1946-47 in a survey of approximately 100 dams in United States. The survey found that :

1. Dumped riprap failed in 5 % of the cases where used, failures being attributed to improper size of stones.
2. Hand placed riprap failed in 30 % of the cases where used, due to the usual method of single course construction.
3. Concrete pavements failed in 30 % of the cases where used, due generally to the inherent deficiencies of this type of construction.

This shows that the layers of dumped rock riprap are more successful than equivalent thickness of carefully hand-packed rock. The efficiency of dumped rock riprap depends upon the following characteristics :

1. Quality of the rock
2. Weight or size of the individual pieces
3. Thickness of riprap
4. Shape of the stones or rock fragments
5. Slope of the embankments
6. Stability and effectiveness of the filter (i.e., gravel layer) on which riprap is placed.

The U.S. Bureau of Reclamation has found a thickness of about 1 m of dumped riprap, placed on 30 to 50 cm of gravel filter to be quite satisfactory for major dams.

A layer or blanket of graded gravel should be provided underneath the riprap when the compacted material of underlying earth-fill is of such gradation that there is danger that fines may be washed out through the voids in the riprap by wave action.

10.14. SEEPAGE CONTROL MEASURES

The seepage control measures are necessitated to prevent adverse effects of water percolating through embankment and its foundation. The following devices are used for seepage control through earth dam :

(A) Embankment seepage control

1. Toe filter
2. Horizontal drainage filter

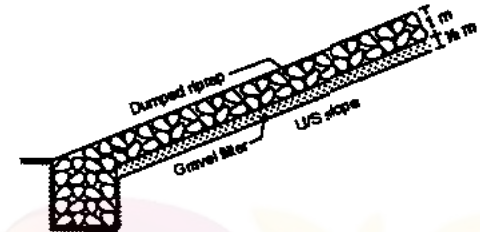


FIG. 10.47. UPSTREAM SLOPE PROTECTION

3. Protective filter d/s of the toe
4. D/s coarse section (embankment zoning)
5. Chimney drain extending upward into embankments.

(B) Foundation seepage control

6. Impervious cutoff
7. Upstream impervious blanket
8. D/s seepage berms
9. Drainage trenches
10. Relief wells.

A brief description of the these measures will now follow.

1. Toe filter. Rock toe keeps the phreatic line well within the section and also facilitates drainage. Its height is generally kept equal to 30 to 40 % of reservoir head. The gradation of various materials and layers should satisfy filter criteria.



FIG. 10.48. DRAINAGE FILTERS.

2. Horizontal drainage filter. The horizontal drainage filter may extend from 25 to 100 % of the distance from downstream toe to the centre line of the dam. The horizontal filter serves the following purposes : (i) It keeps the phreatic line well within the embankment, (ii) It gives greater leakage because of shorter seepage path, (iii) It also provides drainage for foundation, (iv) It accelerates consolidation.

Where a high degree of stratification of the embankment may occur, the rock-fill toe has an advantage over drainage blanket in that it intercepts more layers of the embankment.

3. Filter downstream of the toe. The provision of such filter [Fig. 10.49 (a)] provides additional weight and thus makes the upward flow more safe.



4. Downstream coarse section. This also intercepts the flow through the embankment, and makes the d/s slope safe against piping. It is also an earthquake resistant measure.



FIG. 10.49

5. Chimney drain. When there is high degree of embankment stratification, the horizontal permeability is greater than the vertical, resulting in greater horizontal spread of seepage. A correctly built vertical drain can completely intercept embankment seepage, regardless of the degree of stratification of the embankment. It is also earthquake resistant.

6. Impervious cutoffs.

Whenever economically possible, the seepage should be cutoff. However, no cutoff is absolutely impervious and the reduction of seepage is a relative matter. The various cutoffs may consist of : (1) Cutoff trenches, (2) Grout curtains, (3) Sheet pile cutoffs. It may be impossible or extremely expensive to continue the cutoff to an impervious stratum. In such a case, only *partial cutoff* is provided. For homogeneous foundation material, the effectiveness of partial cutoff is limited, since 90 % depth of cutoff reduces the discharge only by about 64 % and 50 % depth by 25 %.



(a) Horizontal filter drain relatively ineffective



(b) Chimney filter drain intercept embankment seepage

FIG. 10.50. EFFECTIVENESS OF CHIMNEY DRAIN.

7. Downstream Seepage berms (Fig. 10.51). Berms can be used to control seepage efficiently where the d/s top stratum is relatively thin and uniform, or where no d/s top stratum is present.



FIG. 10.51. D/S SEEPAGE BERMS.

They serve two purposes :

- (i) They provide additional weight to resist uplift pressure beneath the top stratum, and
- (ii) They afford some protection against possible sloughing of the d/s slope of the dam as a result of seepage.

8. Drainage trenches (Fig. 10.52). They are provided when top stratum is thin and pervious stratum is also shallow.

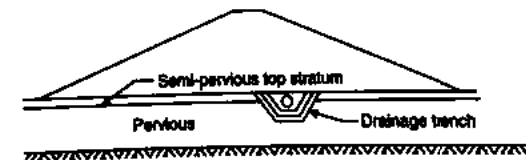


FIG. 10.52. DRAINAGE TRENCH

9. Relief wells (Fig. 10.53). The preliminary purpose of relief well is to reduce the sub-stratum uplift pressure d/s of the dam which otherwise would cause formation of sand boils and possibly sub-surface piping. They intercept the seepage and control the outlet for seepage. They were first used by U.S. Corps of Engineers.

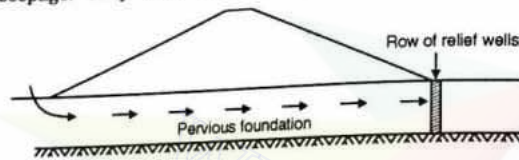


FIG. 10.53. RELIEF WELL.

10. Upstream impervious blanket. Impervious upstream or riverside blankets overlaying a pervious foundation are effective in reducing the quantity of seepage. They also, to some extent, reduce uplift pressure and escape gradients downstream of land side. A part of reservoir head is dissipated through the blanket. The length of the blanket can be determined by the Benett's analysis.

Benett's analysis

Benett gave the mathematical solution for the performance of the u/s impervious blanket, shown in Fig. 10.54.

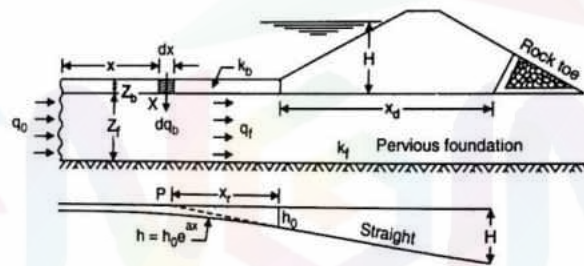


FIG. 10.54. BENETT'S ANALYSIS.

At any point X, under the blanket, the horizontal flow through the foundation (q_f) is equal to the flow through the blanket itself (q_b) upstream from the point plus the inflow q_0 under the upstream end of the blanket.

$$\text{Thus } q_f = q_0 + \int_0^x dq_b \quad \dots(1)$$

where dq_b is the vertical flow through a small element of the blanket of width dx and thickness Z_b . The value of dq_b varies directly with the head loss h at X, and inversely with the blanket thickness Z_b . If k_b is the permeability coefficient of the blanket, we have

$$dq_b = k_b \cdot i \cdot A = k_b \frac{h}{Z_b} dx \quad \dots(2)$$

Substituting in (1), and differentiating with respect to x , we get

$$\frac{dq_f}{dx} = \frac{dq_0}{dx} + \frac{d}{dx} \left[\int_0^x k_b \frac{h}{Z_b} dx \right]$$

But $\frac{dq_0}{dx} = 0$, since q_0 is independent of x .

$$\text{Hence } \frac{dq_f}{dx} = \frac{k_b h}{Z_b} \quad \dots(3)$$

Again $q_f = k_f \frac{dh}{dx} Z_f$ for the foundation

$$\therefore \frac{dq_f}{dx} = k_f Z_f \frac{d^2 h}{dx^2} \quad \dots(4)$$

Equating (3) and (4), we get

$$\frac{d^2 h}{dx^2} = \frac{k_b h}{k_f Z_b Z_f} = \alpha^2 h \quad \dots(10.28)$$

$$\text{where } \alpha = \sqrt{\frac{k_b}{k_f Z_b Z_f}} = \text{constant} \quad \dots(10.29)$$

Eq. 10.28 is the differential equation for the pressure dissipation. Its solution is to be obtained in two cases : (i) Infinite length of the blanket, (ii) Finite length of the blanket.

Also, we will consider two cases of blanket thickness :

(a) Blanket of uniform thickness, and (b) blanket on variable thickness.

(a) **BLANKET OF UNIFORM THICKNESS**

For blanket of uniform thickness, Z_b is constant and hence $\alpha = \sqrt{k_b/(k_f Z_b Z_f)}$ is constant.

(i) **Solution for Infinite length of blanket**

The solution of Eq. 10.28 for infinite length is :

$$h = h_0 e^{\alpha x} \quad \dots(10.30)$$

where h_0 = total head lost through blanket at $x = 0$ (which is taken for convenience in this case at downstream end of the blanket)

Equivalent resistance X_r of the foundation. The resistance X_r is a measure of the efficiency of the blanket of any length x . It may be defined as the length of a prism of the foundation material, of thickness Z_f and permeability k_f , which under head loss h_f would carry flow equivalent to the flow which passes the blanket system.

$$\text{Thus } k_f Z_f \frac{dh}{dx} = k_f Z_f \frac{h}{X_r}$$

$$\text{or } \frac{dh}{dx} = \frac{h}{X_r} \quad \dots(5)$$

But $\frac{dh}{dx} = h_0 a e^{ax}$ (from Eq. 10.30)

or $\frac{dh}{dx} = a h_0 e^{ax} = ah$

Substituting in (5), we get $ah = \frac{h}{X_r}$
 $\therefore X_r = \frac{h}{ah} = \frac{1}{a} = \sqrt{\frac{k_f Z_b Z_f}{h_0}} \quad \dots(10.31)$

Discharge reduction

If there were no blanket,

$$q_f = k_f \frac{H}{X_d} \cdot Z_f$$

Due to the provision of blanket

$$q_f' = k_f \frac{H}{X_r + X_d} Z_f$$

$$\therefore \% \text{ reduction} = \frac{q_f - q_f'}{q_f} \times 100 = \frac{X_r}{X_r + X_d} \times 100 \quad \dots(10.32)$$

(ii) Solution for finite length of blanket

The solution of Eq. 10.29 for finite length is

$$h = h_n (e^{ax} - e^{-ax}) \quad \dots(10.33)$$

where h_n = constant = total head loss through the blanket at the d/s edge of the blanket, and is proportional to the ratio of the blanket resistance to the remainder of the system.

Differentiating Eq. 10.33,

$$\frac{dh}{dx} = ah_n (e^{ax} + e^{-ax})$$

Now, if X_r is the equivalent resistance of the foundation, we have

$$\frac{dh}{dx} = \frac{h}{X_r}$$

$$X_r = \frac{h}{dh/dx} = \frac{h_n (e^{ax} - e^{-ax})}{ah_n (e^{ax} + e^{-ax})}$$

$$\text{or } X_r = \frac{(e^{2ax} - 1)}{a(e^{2ax} + 1)} \quad \dots(10.34)$$

If there were no blanket, $q_f = k_f \frac{H}{X_d} Z_f$

Due to the provision of blanket

$$q_f' = k_f \frac{H}{X_r + X_d} Z_f$$

$$\therefore \% \text{ reduction in discharge} = \frac{q_f - q_f'}{q_f} \times 100 = \frac{X_r}{X_r + X_d} \times 100$$

It can be shown that for any value of a , very little benefit will be gained by extending the blanket beyond a length X_0 , known as the *optimum length of blanket* given by the expression.

$$X_0 = \sqrt{2/a} \quad \dots(10.34 a)$$

(ii) Steps for the design of blanket of uniform thickness

(i) Find a by the expression $a = \sqrt{\frac{k_b}{k_f Z_b Z_f}}$

(ii) Find the optimum length X_0 by $X_0 = \sqrt{2/a}$

(iii) Find X_r by the relation $X_r = \frac{1}{a}$ for infinite length

or $X_r = \frac{(e^{2ax} - 1)}{a(e^{2ax} + 1)}$ for finite length

(iv) Find the hydraulic gradient $i = \frac{H}{X_r + X_d}$

(v) Find the loss of head (or head dissipated) through the blanket by the expression

$$h_0 = \frac{X_r}{X_r + X_d} \cdot H \quad \dots(10.35)$$

(v) Find percentage reduction in discharge.

The actual design procedure will, therefore, be to select various values of x and to find % reduction in discharge, and also to find the head dissipated through the blanket.

The recommended length will be the one beyond which there will be no appreciable reduction in the discharge. However, the *optimum length* of the blanket is given by $X_0 = (\sqrt{2/a})$.

(b) BLANKET OF VARIABLE THICKNESS

It is seen that the blanket is more effective towards the downstream end. Hence for more efficient use of the blanket, we should provide increasing thickness in the down stream direction, thus making Z_b variable. Hence Eq. 10.28 can be expressed as follows :

$$\frac{d^2 h}{dx^2} = \frac{ah}{Z_b} \quad \dots(10.36)$$

where $a = \frac{k_b}{k_f Z_f} = \text{constant}$

while Z_b varies with x .

Considering blanket thickness to vary triangularly, we have $Z_b = S \cdot x$ where S is the slope of blanket thickness. However the solution of Eq. 10.36 for this case is obtained in terms of a series which can not be computed conveniently. On the contrary, if we assume the blanket to be of parabolic profile with its thickness represented by the following equation :

$$Z_b = \frac{\alpha x^2}{n(n-1)} \quad \dots(10.37)$$

Then the solution of Eq. 10.36 is given by :

$$h = h_n x^n \quad \dots(10.38)$$

$$\text{For such a case, } X_r = \frac{h}{(dh/dx)} = \frac{x}{n} \quad \dots(10.39)$$

The head dissipated through the blanket is given by

$$h_0 = \frac{X_r}{X_r + X_d} H \quad \dots(10.35)$$

The percentage reduction in discharge is given by Eq. 10.32.

Design steps

1. Compute the value of α from Eq. 10.36 (α), corresponding to the known values of k_b , k_f and Z_f .

2. Assume a suitable value of length of the blanket, and compute Z_b (from Eq. 10.37) and X_r (From Eq. 10.39) by taking different values of $n = 1.25, 1.5, 1.75, 2.0$ etc.

However, the length of the blanket may be assumed close to the one given by $X = \sqrt{Z}/\alpha$, by assuming average uniform thickness (Z_b) and taking α equal to the one given by Eq. 10.29.

3. Compute head dissipated through the blanket and reduction in discharge.

4. Repeat the above procedure for several other lengths of the blanket.

5. Select the length of the blanket which causes considerable reduction in the discharge and is also economical.

10.15. SLOPE STABILITY ANALYSIS UNDER EARTH QUAKE EFFECTS

It is recognized that an earth dam vibrates when subjected to ground motion during an earthquake requiring thereby a *dynamic analysis* of the structure for its design. Nevertheless, currently accepted design procedure is based on assumption that the portion of the dam above the rupture surface is rigid. Therefore the method given below in IS 1893-1984, which assumes additional horizontal and vertical loads on the soil mass within the rupture surface, is adopted. It is desirable, however, to carry out dynamic analysis for final design of important dams in order to estimate deformations in dams in probable future earthquakes.

Design seismic coefficient (α_h)

As per IS : 1893-1984, the value of design seismic coefficient (α_h) in the horizontal direction is given by Eq. 8.4 reproduced here.

$$\alpha_h = \beta I F_0 \frac{S_a}{g} = 2 F_0 \frac{S_a}{g} \quad \dots(10.40)$$

where β - soil foundation system factor, the value of which is taken as 1.0 for dams.

I - importance factor, the value of which is taken as 2 for dams.

F_0 = seismic zone factor, the value of which for each of the four zones is given in Table 8.2.

$\frac{S_a}{g}$ = average acceleration spectra, as read from Fig. 8.4 for appropriate natural period (T) and 10% damping.

The natural (or fundamental) period (T) of vibration of an earth dam is determined from the following expression.

$$T = 2.9 H \sqrt{\rho/G} \quad \dots(10.41)$$

where T = fundamental period of earth dam, in seconds

H = height of the dam above toe of the slope

ρ = mass density of the shell material

G = modulus of rigidity of the shell material.

Note : The quantity $\sqrt{G/\rho}$ is the *shear wave velocity* through the material of the dam and may be used if known instead of ρ and G .

Equivalent uniform horizontal seismic coefficient and horizontal inertial force

For checking slope failure with the lowest point of the rupture surface at any depth y below the top of the dam, the value of *equivalent uniform horizontal seismic coefficient* is given by

$$\alpha_{hy} = \left(2.5 - 1.5 \frac{y}{H} \right) \alpha_h \quad \dots(10.42)$$

where y = depth of the lowest point of the rupture surface

H = total height of the dam above the toe of the slope

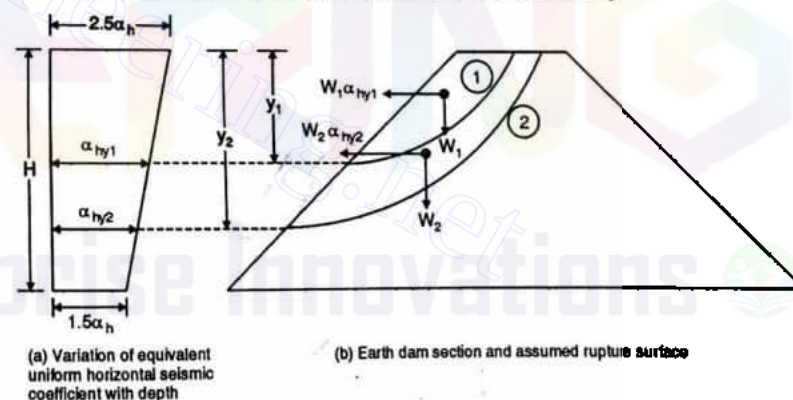


FIG. 10.55. EQUIVALENT UNIFORM HORIZONTAL SEISMIC COEFFICIENT

Fig. 10.55 (a) shows the variation of horizontal seismic coefficient with depth, with value at the top equal to $2.5 \alpha_h$ and value at bottom as $1.5 \alpha_h$. Thus, for assumed

rupture surface 1 (Fig. 10.55 b), if y_1 is the depth of the lowest point of the rupture surface, the value of equivalent uniform horizontal seismic coefficient will be equal to α_{hy1} . Similarly, for rupture surface 2, with y_2 as the depth of the lower point, the value of uniform horizontal seismic coefficient will be α_{hy2} .

The horizontal inertia force acting at the centre of gravity of the rupture wedge will be equal to $W_1 \cdot \alpha_{hy1}$ for rupture surface 1 and $W_2 \cdot \alpha_{hy2}$ for rupture surface 2, as shown in Fig. 10.55 (b), where W_1 and W_2 are the weight of the rupture wedges acting at their centre of gravity.

Equivalent uniform vertical seismic coefficient and vertical inertia force:

The value of equivalent uniform vertical seismic coefficient (α_{vy}) is taken equal to half the corresponding value of equivalent uniform horizontal seismic coefficient. Thus,

$$\alpha_{vy} = 0.5 \alpha_{hy} = \frac{1}{2} \left(2.5 - 1.5 \frac{y}{H} \right) \alpha_h \quad \dots(10.43)$$

where y = depth of the lowest point of the rupture surface under consideration.

The vertical inertia force, acting upwards through the centre of gravity of the rupture wedge will be equal to $W \cdot \alpha_{vy}$ where W is the weight of the rupture wedge.

Choice of rupture surfaces

From the elastic analysis of a typical dam section, it can be found that the crest of the dam is subjected to much larger forces than the toe, the increase in acceleration from the toe to the crest being almost parabolic in nature. This behaviour has been borne out even by actual measurements in different dams in Japan. Therefore, design of upper portions of the dam to resist larger forces become essential. This can be achieved by assuming probable slip surfaces entirely in the upper ranges only, say extending to the upper 7/8 to 3/4 range from the top. Because of larger horizontal forces, the stability will be least for such slip surfaces, specially in the absence of cohesion.

Stability of upstream slope

The stability of the upstream slope of an earth or rockfill dam shall be tested with full reservoir level with horizontal forces due to earthquake acting in the upstream direction and vertical forces due to earthquake (taken as one half of the horizontal) acting upwards. For preliminary design, a factor of safety of unity shall be accepted as being adequate for ensuring stability of upstream slope. The factor of safety need be tested only for failure surface which passes through the lower half of the dam.

Stability of downstream slope

The provision mentioned above for u/s slope shall also apply in determining the stability of the d/s slope except that the horizontal forces due to earthquake should be considered acting in the downstream direction.

Stability during construction stage or for reservoir empty condition

Earthquake forces shall not be normally included in stability analysis for construction stage or for reservoir empty condition. However, where construction or operating schedule requires the reservoir empty condition to exist for prolonged periods, earthquake forces may be included and may be calculated on 50 percent of the value obtained above.

Note : Junction between spillways and abutments shall be constructed with great care in view of the damage that may be caused by differential vibration of the dam and the spillway.

10.16. DESIGN CONSIDERATIONS IN EARTHQUAKE REGIONS

The failures of earth dams due to earthquake have already been discussed in § 10.2. In most of the cases, the designer should anticipate that the dam will crack and settle. Hence he must provide means to control leakage through the cracks and sufficient free board to prevent overtopping in spite of the settlement. The following design details should be provided to protect the dam against failures due to earthquake.

(1) **Cohesionless graded filter downstream.** If a graded filter is provided just to the d/s of the impervious core, it will protect the dam against excessive leakage and progressive piping through the cracks developed by the earthquake. When water flows through the cracks of the core and discharges into d/s graded filter, the core material will either swell and seal the crack, or the particles of the core materials in the crack will be eroded and carried to the d/s filter. The graded d/s filter, in turn, will stop the migrating particles and the crack will thus be sealed (Fig. 10.56).

(2) **Cohesionless graded filter upstream.** When the central core, made of tough clay, develops cracks due to earthquake, an upstream graded filter with appreciable content of fine sand sizes, will prove to be very helpful. The fine sand from the filter will just flow into the cracks of the core and will partially seal them. This was one of the main design provisions for Coyote Dam constructed on the Hayward Fault (Fig. 10.56).

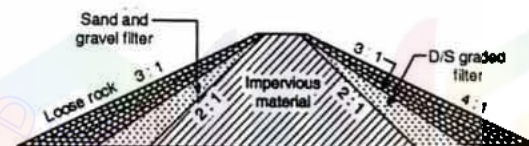


FIG. 10.56. CROSS-SECTION OF COYOTE DAM LOCATED ON THE HAYWARD FAULT.

(3) **Very pervious downstream zone.** Provision should be made so that the discharge through the cracks developed, if any, in the core is discharged to the d/s as quickly as possible so that critically high water pressure may not develop in the d/s portion of the embankment. D/s zones of large quarried rock offers best combination of permeability and stability. Such a zone also provides the greatest security against erosive action of large quantities of water passing over the crest, if overtopped.

(4) **Provision of flatter slopes near the top of the dam.** During an earthquake, the top of the dam vibrates with considerably greater amplitude than the base and may be subjected to worst damage. It is, therefore, desirable to make the top of the dam thicker either by increasing the crest width or by using flatter slopes at the top. The thicker the top is made, the longer is the path of seepage through cracks which may develop, and safer will be the dam against earthquake damage.

(5) **Core treatment.** The core material should be most resistant to piping. A thicker core provides greater piping resistance, and makes the dam safe when the earthquake tend to twist the dam alignment. Inferior concrete or masonry wall should be avoided.

(6) **Foundation treatment.** All the loose or soft soil should be excavated because a dam on soft foundation is more severely shaken.

(7) **Provision of extra free board.** In most of the cases, the worst earthquake damage that may be anticipated is overtopping of the dam, due to (i) the settlement of the crest of the dam, or (ii) rise in the water level due to land slide into reservoir basin, or (iii) upward movement of reservoir bottom due to fault movement. Hence extra free board should be provided.

(8) **Slope stability analysis using horizontal acceleration forces.** While testing the stability of slope, it has become common practice to use horizontal acceleration forces in addition to the other usual forces. Depending upon the location of the area with respect to various seismic zone, the horizontal force is assigned a value of $1/2$ to $1/5$ of the weight of the sliding mass.

10.17. COMPACTION OF ROLLED FILL DAM

The rapid increase in knowledge of soil mechanics since the year 1925 has resulted in substantial progress towards understanding the factors involved in transforming loose earth into a structural material. The development of huge economical earth-moving machines has increased the placing rate of the earth fill many times thereby intensifying the problem of quality control. Construction control is obtained by inspection, testing and reports. Proper control of earthwork requires the use of laboratory facilities.

After having decided the quarry sites for various materials to be used in a dam, the next step is to determine, by laboratory tests, the relationship between the water content and dry density obtained on compaction. The maximum dry density and the optimum water content for each soil can be determined in the laboratory either by Proctor's test or by 'Jodhpur-mini-compactor test'. While compacting these soils in the field, the same dry density should be achieved.

The various earth-moving machinery used in embankment construction are listed below :

1. Excavating equipment, such as power shovels, draglines, scrapers etc.
2. Hauling equipment, such as delivery trucks, scrapers, trucks and belt conveyors.
3. Spreading equipment such as bulldozers, graders etc.
4. Rollers and special compacting equipment.
5. Rock separation equipment.
6. Discs, harrows and ploughs.
7. Watering equipment.

The various compaction equipment used in the field can be grouped under three classes :

- (i) Rollers,

- (ii) Rammers, and
- (iii) Vibrators.

The rolling equipment (rollers) can further be classified into five types :

- (i) Smooth wheel rollers,
- (ii) Sheep's foot rollers,
- (iii) Pneumatic tyred rollers,
- (iv) Lorries, and
- (v) Track laying tractors.

For a detailed description of these, the reader is referred to standard text on 'Soil Mechanics' or 'Earthwork'.

10.18. ROCKFILL DAMS

A rock fill dam is an embankment which uses variable sizes of rock to provide stability, and an impervious membrane to provide water-tightness. Various materials used for the membrane are : earth, concrete, steel, asphalt and wood.

The principal parts of a rockfill dam are as follows (Fig. 10.57):

1. Upstream impervious membrane resting on the upstream rock cushion.
2. Upstream rock-cushion constructed on carefully laid stone bonding with main rockfill on the downstream.
3. Main rockfill on the downstream.
4. Upstream cutoff.

Sometimes, a horizontal pervious zone is provided, instead of rock cushioning, at the u/s to support the impervious membrane (Fig. 10.58).

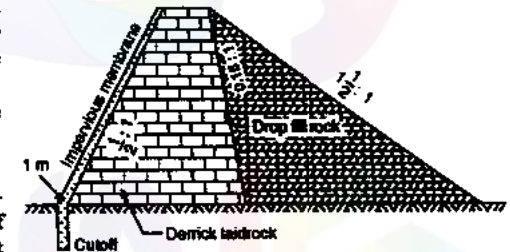


FIG. 10.57. ROCKFILL DAM.

The watertight membrane should be constructed on the u/s slope so that it can be inspected when the reservoir is drawn down, and repairs made as necessary. A cutoff wall should be provided along the contact of the impervious membrane with the foundation and abutments at the u/s toe of the dam to prevent seepage under the dam. The placement of the rockfill zone is one of the most important operations in the construction of a rockfill dam, as it is essential to minimise total settlement and the possibility of damage to the impervious membrane. The rock cushioning provided at the u/s of the main rockfill acts as a cushioning in stabilising the u/s face and equalising the settlement.

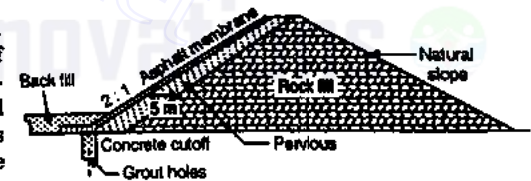


FIG. 10.58. ROCKFILL DAM.

ILLUSTRATIVE EXAMPLES

Example 10.1. For the earth dam of homogeneous section with a horizontal filter as shown in Fig. 10.59, draw the top flow line. If the coefficient of permeability of the soil material used in the dam is 5×10^{-4} cm/sec, find the seepage flow per unit length of the dam.

Solution : The top flow line which is a parabola can either be drawn graphically or by calculation of its ordinates. The procedure has been explained in § 10.6.

Taking the focus F as the origin, the equation of parabola is given by

$$\sqrt{x^2 + y^2} = x + s$$

where s = distance of directrix from the origin and x, y are the co-ordinates of any point on the parabola.

The parabola cuts the reservoir water surfaces at point B , such that

$$FB' = 146 - 30 - 72 + 72 \times 0.3 = 65.6 \text{ m} = x$$

and $BB' = 18 \text{ m} = y$

Then $s = \sqrt{(65.6)^2 + 18^2} - 65.6 = 68.0247 - 65.60 = 2.4247 \triangleq 2.425 \text{ m}$.

Equation of parabola, therefore, is

$$y = \sqrt{2sx + s^2} = \sqrt{4.849x + 5.879}$$

The co-ordinates of parabola are tabulated below :

x	0	10	20	30	40	50	60	65.6
y	2.425	7.374	10.182	12.303	14.137	15.769	17.229	18

The top flow line can, therefore, be drawn by the above table. A correction as explained in § 10.6 at entry and exit should be made.

Seepage flow per unit length of dam = ks .

$$= 5 \times 10^{-4} \times 2.425 \times 100 \text{ cm}^3/\text{sec}/\text{cm} = 12.125 \times 10^{-2} \text{ cm}^3/\text{sec}/\text{cm}$$

$$= 12.125 \times 10^{-6} \text{ cumecs per metre length of the dam.}$$

Example 10.2. For a homogeneous earth dam 52 m high, and 2 m free board, a flow net was constructed and following results were obtained.

Number of potential drops = 25

Number of flow channels = 4

The dam has a horizontal filter of 40 m length at its downstream end. Calculate the discharge per metre length of the dam if the coefficient of permeability of the dam is 3×10^{-3} cm/sec.

Solution. The discharge per unit length is given by

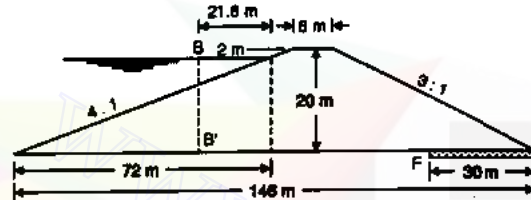


FIG. 10.59.

$$q = kh \frac{N_f}{N_d}$$

Here h = water depth = $52 - 2 = 50 \text{ m}$; $k = 3 \times 10^{-3} \text{ cm/sec} = 3 \times 10^{-6} \text{ m/sec}$
 $N_f = 4$; $N_d = 25$
 $q = 3 \times 10^{-6} \times 50 \times 4/25 = 24 \times 10^{-6} \text{ m}^3/\text{sec}/\text{m}$
 $= 0.00024 \text{ cumec/metre length.}$

Example 10.3. In order to find the factor of safety of d/s slope during steady seepage, the section of the dam was drawn to scale of 1 cm = 4 m, and the following results obtained on a critical slip circle :

Area of N-rectangle = 14.4 sq. cm

Area of T-rectangle = 6.4 sq. cm

Area of U-rectangle = 4.9 sq. cm

Length of arc = 12.6 cm.

Laboratory tests have furnished values of 26° for effective angle and 19.5 kN/m^2 for cohesion. Unit weight of soil = 19 kN/m^3 . Determine the factor of safety of the slope.

Solution. Let the scale be 1 cm = $x \text{ m}$. (Here $x = 4$)

Consider 1 m length of the dam.

$$\Sigma N = A_N(x)^2 \gamma = A_N(16) \times 19 \text{ kN} = 14.4 \times 16 \times 19 = 4377.6 \text{ kN}$$

$$\Sigma T = A_T \times 16 \times \gamma = 6.4 \times 16 \times 19 = 1945.6 \text{ kN}$$

$$\Sigma U = A_U \times 16 \times \gamma_w = A_U \times 16 \times 9.81 = 4.9 \times 16 \times 9.81 = 769.1 \text{ kN}$$

$$\text{Length of arc} = \hat{L} = 12.6 \times 4 = 50.4 \text{ m}$$

$$F = \frac{c' \hat{L} + \Sigma(N - U) \tan \phi'}{\Sigma T}$$

$$\text{or } F = \frac{(19.5 \times 50.4) + (4377.6 - 769.1) \tan 26^\circ}{1945.6} = 1.41$$

Example 10.4. A homogeneous earth dam has the following data :

(a) Dimensions

(i) Height of the dam = 20 m

(ii) Top width of dam = 6 m

(iii) U/S slope = 4 : 1

(iv) D/S slope = 3 : 1

(v) Length of d/s horizontal filter = Height of dam

(vi) Free board = 2 m.

(b) Properties of the material of the dam

(i) Dry density = 17.27 kN/m^3

(ii) Optimum water content = 15 %

(iii) Saturated density = 21.19 kN/m^3

(iv) Average unit weight under steady seepage = 19.62 kN/m^3

- (v) Average angle of internal friction = 26°
 (vi) Average cohesion = 19.13 kN/m^2
 (vii) Coefficient of permeability = $5 \times 10^{-4} \text{ cm/sec}$
 (c) Properties of the foundation material
 (i) Average unit weight = 17.27 kN/m^3
 (ii) Average cohesion = 47.87 kN/m^2
 (iii) Average angle of internal friction = 8°
 (iv) Coefficient of permeability : negligible.

The foundation material consists of 6 m thick layer of clay.

Check the section for

- (i) Stability of d/s slope against steady seepage
 (ii) Sloughing of u/s slope against sudden drawdown
 (iii) Stability of the foundation against shear
 (iv) Seepage through the body of dam.

Permissible factor of safety of slope = 1.5

Permissible seepage = $8 \times 10^{-6} \text{ cumecs}$.

Solution. Fig. 10.60 shows the section of the dam with phreatic line drawn by the method explained earlier.

(a) Stability of d/s Slope During Steady Seepage

In order to check stability of the d/s slope, Fellenius line is first drawn, and a trial centre is taken on it. Fig. 10.60 shows one such centre, with a slip circle passing through the toe. The sliding mass has been divided into strips each of 6 m width. The pore-pressure diagram has been drawn by the method explained in § 10.8. From Fig. 10.60, we obtain the following :

Area of N-diagram = 302.4 m^2

$$\Sigma N = 302.4 \times 21.19 = 6407.86 \text{ kN}$$

Area of T-diagram = 91.2 m^2

$$\Sigma T = 91.2 \times 21.19 = 1932.53 \text{ kN}$$

Area of U-diagram = 98.4 m^2

$$\Sigma U = 98.4 \times 9.81 = 965.3 \text{ kN}$$

Length of the arc = 60 m

$$\begin{aligned} \text{Hence } \text{F.S.} &= \frac{c \hat{L} + \Sigma(N - U) \tan \phi}{\Sigma T} \\ &= \frac{(19.13 \times 60) + (6407.86 - 965.3) \times \tan 26^\circ}{1932.53} = 1.97 > 1.5 \end{aligned}$$

Hence Safe.

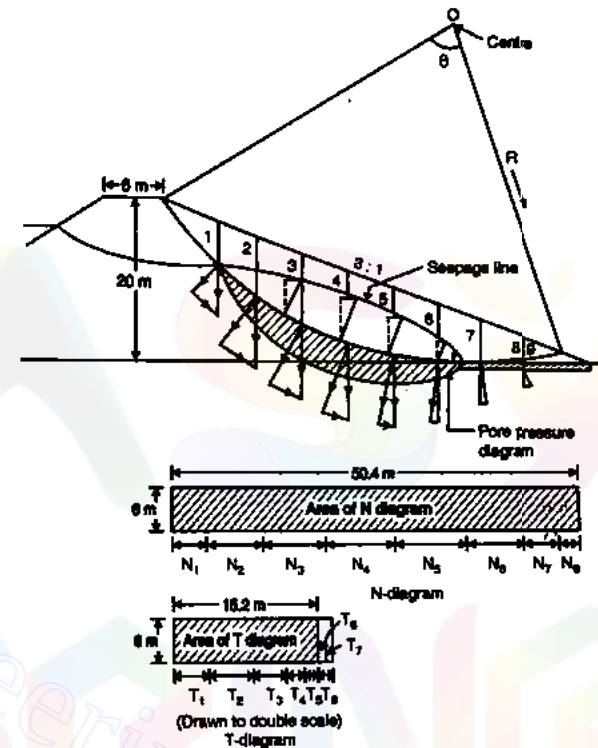


FIG. 10.60

(b) Sloughing of u/s slope against sudden drawdown

The horizontal shear force or sloughing force is given by Eq. 10.19 :

$$P = \frac{\gamma_s h^2}{2} \tan^2 \left(45^\circ - \frac{\phi}{2} \right) + \frac{\gamma_w h_1^2}{2}$$

Here $h = 20 \text{ m}$ and $h_1 = 15 \text{ m}$ (measured from Fig. 10.60).

$$P = \frac{1}{2} \times 21.19 \times (20)^2 \tan^2 \left(45^\circ - \frac{26}{2} \right) + \frac{9.81 \times (15)^2}{2} = 2758.4 \text{ kN}$$

$$s_{av} = \frac{P}{b} = \frac{2758.4}{80} = 34.48 \text{ kN/m}^2$$

$$s_{max} = 2s_{av} = 68.96 \text{ kN/m}^2$$

Shearing resistance R is given by $R = N_t \tan \phi + cb$

Hence $N_c = \gamma' A = (21.19 - 9.81) \times \frac{1}{2} \times 80 \times 20 = 9104 \text{ kN/m}$

$R = 9104 \tan 26^\circ + 19.13 \times 80 = 5970.72 \text{ kN}$

$\therefore \text{F.S.} = \frac{R}{P} = \frac{5970.72}{2758.4} = 2.16 > 2. \text{ Hence safe.}$

The above factor of safety is with respect to average shear. In order to find the F.S. with respect to maximum shear, we have

$s_{\max} = 2 s_{av} = 68.96 \text{ kN/m}^2$

If s_r = shearing resistance at point of maximum shear (i.e. at B in Fig. 10.34)
 $s_r = h' \gamma' \tan \phi + c = 0.6 h \gamma' \tan \phi + c = 0.6 (20) (21.19 - 9.81) \tan 26^\circ + 19.13$
 $= 85.73 \text{ kN/m}^2$

$\therefore \text{F.S.} = \frac{s_r}{s_{\max}} = \frac{85.73}{68.96} = 1.24 > 1. \text{ Hence safe.}$

(c) **Stability Against Foundation Shear :** Refer Fig. 10.38.

The horizontal shear P is given by Eq. 10.26,

$$P = \frac{h_1^2 - h_2^2}{2} \gamma_m \tan^2 \left(45^\circ - \frac{\phi_1}{2} \right)$$

where $\tan \phi_1 = \frac{\gamma_m h_1 \tan \phi + c}{\gamma_m h_1}$ where ϕ and c are the shear parameters for the foundation material.

γ_m = mean unit weight of the dam and foundation weighed in proportion to the depth of each.

Here $\phi = 8^\circ$; $c = 47.87 \text{ kN/m}^2$; $\gamma_m = \frac{\gamma_D (h_1 - h_2) + \gamma_F h_2}{h_1}$

where γ_D = Average unit weight of dam material = 19.62 kN/m^3

γ_F = Unit weight of foundation material = 17.27 kN/m^3 .

$\gamma_m = \frac{19.62 (26 - 6) + 17.27 \times 6}{26} = 19.08 \text{ kN/m}^3$

$\tan \phi_1 = \frac{19.08 (26) \tan 8^\circ + 47.87}{19.08 \times 26} = 0.237 \text{ or } \phi_1 = 13.335^\circ$

$P = \frac{26^2 - 6^2}{2} \times 19.08 \tan^2 \left(45^\circ - \frac{13.335^\circ}{2} \right) = 3817 \text{ kN}$

Average unit shear $s_{av} = \frac{P}{b} = \frac{3817}{80} = 47.71 \text{ kN/m}^2$

Maximum unit shear $= s_{\max} = 2 s_{av} = 95.42 \text{ kN/m}^2$

which occurs at $0.4 \times 80 = 32 \text{ m}$ from point C.

Let s_1 = unit shear strength below toe (at A) $= c + \gamma_F h_2 \tan \phi$
 $= 47.87 + 17.27 \times 6 \tan 8^\circ = 62.43 \text{ kN/m}^2$.

s_2 = unit shear strength at point C $= c + \gamma_m h_1 \tan \phi$
 $= 47.87 + 19.08 \times 26 \times \tan 8^\circ = 117.59 \text{ kN/m}^2$

$\therefore \text{Average shear strength} = \frac{62.43 + 117.59}{2} = 90 \text{ kN/m}^2$

$\therefore \text{Overall factor of safety against shear} = \frac{90}{47.71} = 1.89 > 1.5 \text{ (Safe).}$

Factor of safety at maximum shear :

s = unit shear strength at point of maximum shear (point B)
 $= c + \gamma_{av} h \tan \phi$

where γ_{av} = mean effective unit weight at B $= \frac{\gamma_D h + \gamma_F h_2}{h + h_2}$

$= \frac{19.62 (0.6 \times 20) + (17.27 \times 6)}{(0.6 \times 20) + 6} = 18.84 \text{ kN/m}^3$

$\therefore s = 47.87 + 18.84 (12) \tan 8^\circ = 79.64 \text{ kN/m}^2$

$\therefore \text{F.S.} = \frac{s}{s_{\max}} = \frac{79.64}{95.42} = 0.83 < 1. \text{ Hence unsafe.}$

(d) **Seepage Through the Body of the Dam**

From the construction of parabola (Fig. 10.60).

$s = 2 \text{ m (measured)}$

$q = k s = (5 \times 10^{-4} \times 10^{-3}) \times 2 = 1 \times 10^{-6} \text{ cumecs/m length of the dam.}$

Example 10.5. Design of U/S impervious blanket : For a homogeneous earth dam section shown in Fig. 10.61, design an upstream impervious blanket of 1.2 m uniform thickness. Take the coefficient of permeability of blanket material as 0.06 m/day and that of foundation soil as 72 m/day.

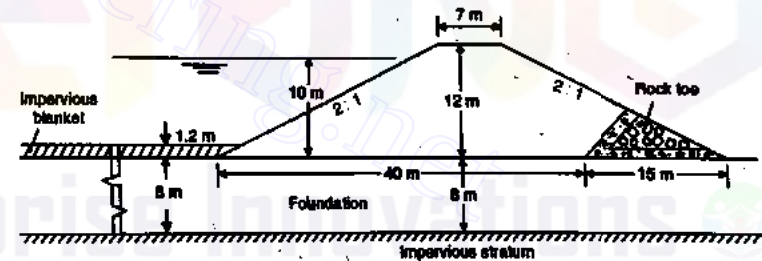


FIG. 10.61.

Solution : Given $Z_b = 1.2 \text{ m}$; $Z_f = 8 \text{ m}$; $k_b = 0.06 \text{ m/day}$; $k_f = 72 \text{ m/day}$

Hence $\alpha = \sqrt{\frac{k_b}{k_f Z_b Z_f}} = \sqrt{\frac{0.06}{72 \times 1.2 \times 8}} = \frac{1}{107.33}$

Optimum length of blanket, $X_0 = \sqrt{2/a} = 107.33 \sqrt{2} \approx 151.8$ m

Let us vary the length of blanket as 25 m, 50 m, 75 m, 100 m, 125 m, 151.8 m and ∞ and calculate the values of X_r , h_0 and % reduction in the discharge. The results are tabulated in Table below. Sample computations are given below for blanket length $x = 50$ m.

$$e^{2ax} = e^{2 \times 50/107.33} = 2.5388$$

Hence

$$X_r = \frac{e^{2ax} - 1}{a(e^{2ax} + 1)} = \frac{(2.5388 - 1) 107.33}{(2.5388 + 1)} = 46.67 \text{ m}$$

$$h_0 = \frac{X_r}{X_r + X_d} H = \frac{46.67}{46.67 + 40} \times 10 = 5.38 \text{ m}$$

$$\% \text{ reduction in } q = \frac{X_r}{X_r + X_d} \times 100 = 53.8\%$$

Length of blanket : x (m)	X_r (m)	h_0 (m)	Reduction in q (%)
25	24.56	3.80	38.04
50	46.67	5.38	53.80
75	64.78	6.18	61.83
100	78.50	6.62	66.24
125	88.28	6.88	68.82
151.8	99.35	7.04	70.45
∞	107.33	7.29	72.85

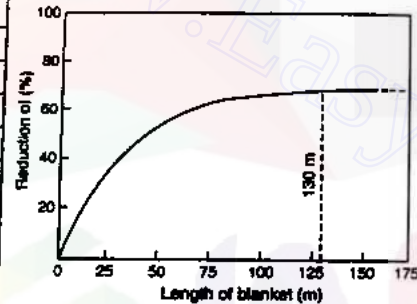


FIG. 10.82

Fig. 10.82 shows the variation of discharge reduction with length of blanket. From the above table as well as graph, we find that by providing the length of blanket beyond 130 m length (say), there would be only slight increase in the head dissipated (h_0) through the blanket as well as in the reduction of seepage discharge. Hence provide 1.3 m thick blanket of 130 m length, though the optimum length of the blanket is 151.8 m.

PROBLEMS

1. Explain, with the help of a sketch, the components of a zoned embankment dam, with their functions.
2. Discuss in brief the causes of failure of earth dams.
3. What are the criteria for safe design of earth dam?
4. Discuss the recommendations for the section of an earth dam.
5. Write a note on 'filter criteria' for earth dam.

6. Derive Laplace equation for seepage through the homogeneous mass of an earth dam. What are its methods of solution?
7. Explain the method of plotting the flownet for seepage through anisotropic soil.
8. Explain the method of plotting phreatic line for an earth dam with horizontal filter at down stream. Also, sketch the phreatic line if there is no filter at the d/s end.
9. Derive the equation for the phreatic surface for an earth dam.
10. Derive an expression for discharge through the body of an earth dam with horizontal filter at the downstream. Also, derive an alternative expression if the flownet for the dam is known.
11. Explain the method of stability analysis of d/s slope during steady seepage.
12. Explain the method of stability analysis of u/s slope during sudden drawdown.
13. Explain the method of checking the stability of earth dam foundation against shear failure.
14. What do you understand by construction pore pressure in earth dams, and how are they determined?
15. Sketch out sections of earth dam to suit various conditions of available material and existing foundation conditions.
16. The cross-section of an earth dam is shown in Fig. 10.63. Check the stability of foundation against shear.

Given : (i) Angle of internal friction for foundation clay : 6°

(ii) Unit cohesion of foundation clay : 45 kg/m^2

(iii) Average unit weight for foundation clay : 1700 kN/m^3

(iv) Average unit weight for dam : 20 kN/m^3

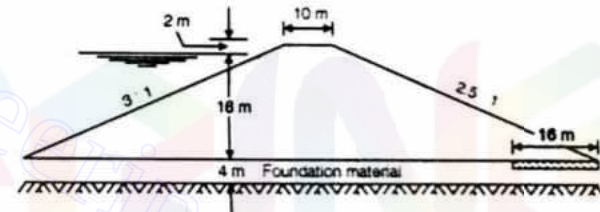


FIG. 10.63.

17. Fig. 10.64 shows a homogeneous earth dam. Calculate the seepage, per metre length, through the body of the dam. Coefficient of permeability of the dam material may be taken as $8 \times 10^{-3} \text{ cm/sec}$.

[Ans $28.8 \times 10^{-5} \text{ cumec/m}$]

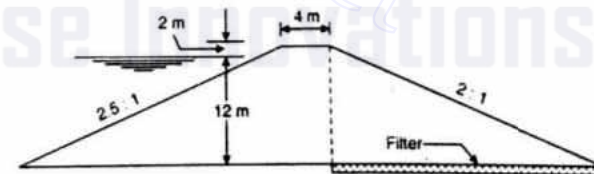


FIG. 10.64

18. In order to find the factor of safety of d/s slope during steady seepage, the section of a homogeneous earth dam was drawn to a scale of 1 cm = 10 m and the following results obtained on a trial circle :

- Area of N-rectangle = 12.15 sq.cm
 Area of T-rectangle = 6.50 sq.cm
 Area of U-rectangle = 4.82 sq.cm
 Length of arc = 11.60 cm. The dam material has the following properties :
 Effective angle of friction = 26°
 Unit cohesion = 20 kN/m^2
 Unit weight of soil = 20 kN/m^3
 Determine the factor of safety of the slope.

[Ans. 1.21]

19. A homogeneous earth dam section shown in Fig. 10.65 has the following data:

- (a) Properties of the material of the dam :
 (i) Dry density = 18 kN/m^3
 (ii) Saturated density = 22 kN/m^3
 (iii) Average angle of friction = 30°
 (iv) Average cohesion = 16 kN/m^2
 (b) Properties of foundation material :
 (i) Average unit weight = 17 kg/m^3
 (ii) Average cohesion = 64 kN/m^2
 (iii) Average angle of internal friction = 7°
 Check the section for the following :
 (i) Sloughing of u/s slope during sudden drawdown.
 (ii) Stability of foundation against shear.

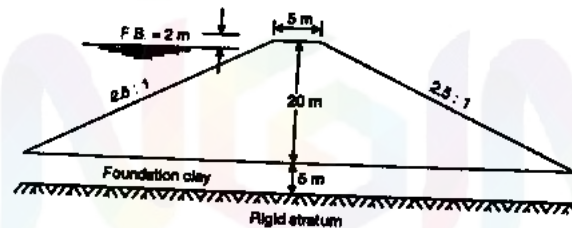


FIG. 10.65.

20. Explain, with a sketch, the component parts and their function, of a rockfill dam.
 21. Explain the method of locating centre of the critical slip circle of stability analysis of the slope of an earth dam.

Dams-V : Spillways

11.1. INTRODUCTION

A spillway is the overflow portion of dam, over which surplus discharge flows from the reservoir to the downstream. A spillway is, therefore, called a *surplusing work*, designed to carry this flood water not required to be stored in the reservoir, safely to the river lower down. Spillways are invariably provided for all storage and detention dams. Ordinarily, this surplus water is drawn from the top of the pool (or reservoir) created by the dam. Spillways are very important structures ; many failures of the dams have been caused by improperly designed spillways or by spillways of insufficient capacity. A spillway is thus the *safety valve* for a dam.

Sufficient capacity of spillway is of paramount importance, specially in earth and rockfill dams where overtopping may be very dangerous. A spillway must have the capacity to discharge major floods without damage to the dam or any appurtenant structures, at the same time keeping the reservoir level below some predetermined maximum level. The design flood discharge required to be passed over the spillway can be determined by flood routing (Chapter 6). In addition to providing sufficient capacity, the spillway must be hydraulically and structurally adequate and must be located so that spillway does not erode or undermine the downstream of the dam.

Types of Spillways

Based on their utility, spillways can be of two types :

- (i) Main spillway (ii) Subsidiary or emergency spillway.

The main spillway is the one which is called upon to work under the design floods. A subsidiary or emergency spillway is provided for additional safety should emergencies not contemplated by normal design assumptions arise. Under normal reservoir operation, emergency spillways are never required to function. Their functioning is required under the situations caused by an enforced shutdown of the outlet works or malfunctioning of spillway gates or when a flood greater than the design flood occurs. The crest of an emergency spillway is therefore, placed at or above the designed maximum reservoir water surface. Emergency spillways are provided for earth and rockfill dams only, to avoid the overtopping of the main dam embankment because of an emergency condition.

Fig. 11.1 shows the main and emergency spillways for an earth dam. Emergency spillways are often formed by lowering the crest of a dyke section below that of

(511)

the main embankment or by utilizing saddles or depressions along the reservoir rim. The emergency spillways are thus constructed of low height earth embankments which, when overtopped during heavy floods, get washed away leaving the main dam having greater free board entirely safe. The embankment of the emergency spillway is therefore, called the *breaching section*, the top width of which is kept less than 9 m.

Based on their most prominent features, pertaining to either its control or to the discharge channel or to some other component, following are the main types of spillways :

1. Free overfall or straight drop spillway.
2. Ogee or overflow spillway.
3. Side channel spillway.
4. Chute or open channel or trough spillway.
5. Conduit or tunnel spillway.
6. Drop inlet or shaft or morning glory spillway.
7. Siphon spillway.

Spillways are often referred to as *controlled* or *uncontrolled* depending upon whether they are gated or ungated.

11.2. STRAIGHT DROP SPILLWAY

This is the simplest type of spillway which is constructed in the form of a low height weir having downstream face either vertical or nearly vertical. Water drops freely from the crest, and the underside of the falling nappe is ventilated sufficiently to prevent a pulsating, fluctuating, jet. Occasionally, the crest is extended in the form of an overhanging lip (similar to that provided in notch falls) to direct the small discharge away from the face of the overfall section [Fig.11.2(b)].

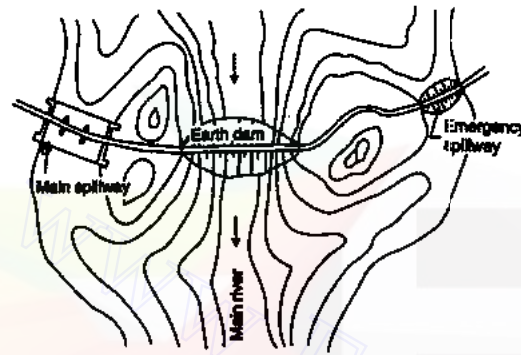


FIG. 11.1. MAIN AND EMERGENCY SPILLWAYS.

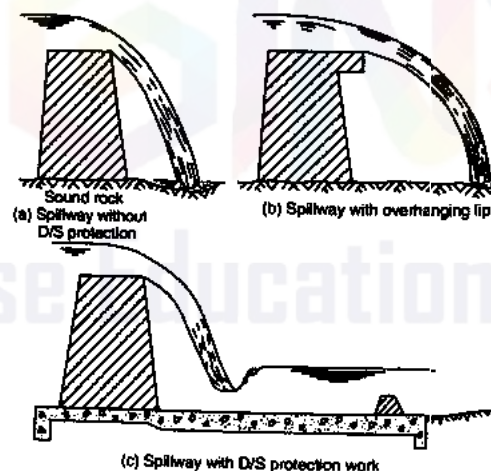


FIG. 11.2. STRAIGHT DROP SPILLWAY.

Fig. 11.2(a) shows a straight drop spillway constructed on sound rock. However, after sometime, the falling jet will form a deep plunge pool. Where erosion is not permissible, a low secondary dam may be constructed to create an artificial pool, or a concrete apron may be provided. If tailwater depths are sufficient, a hydraulic jump will form when the free jet falls on the flat apron. This type of spillway is not recommended for high head since the vibrations caused by the falling jet might crack or displace the structure, with danger from failure by piping or undermining.

Fig. 11.2(c) shows an *impact block basin* useful for low heads. The block gives reasonably good dissipation of energy for a wide range of tailwater depth. The energy is dissipated by means of turbulence caused by the impingement of the flow upon the impact block.

11.3. OGEE OR OVERFLOW SPILLWAY

This is the most common type of spillway provided on gravity dams. The profile of the spillway is *Ogee* or 'S' shaped. The overflowing water is guided smoothly over the crest and profile of the spillway so that the overflow water does not break contact with the spillway surface. If this is not assured, a vacuum may form at the point of separation and cavitation may occur. In addition to cavitation, vibration from the alternate making and breaking of contact between the water and face of the dam may result in serious structural damage. Hence the upper profile of the ogee is made to conform with the lower nappe of a freely falling jet of water over a sharp crested weir, when the flow rate corresponds to the maximum designed capacity of the spillway. Here, the essential difference between the straight drop spillway and the ogee or overflow spillway should be clearly noted. In the former type, the jet falls clearly away from the face of the spillway and the gap between the jet and face is kept ventilated. In the ogee or overflow spillway, the falling water is made to glide over the curved profile of the spillway. A smooth gradual reverse curvature on the downstream face of the spillway is provided. This reverse curve turns the flow on the apron of a stilling basin or into the spillway discharge channel. It is an ideal spillway.

High overflow spillway: (Overflow or ogee spillways are classified as *high* and *low* depending on whether the ratio of the height (P) of spillway crest measured from the river bed to the design head (H_d) is greater than, and equal to or less than 1.33, respectively.) In case of *high overflow spillways*, the velocity of approach head (H_a) which is generally of the order of $0.04 H$ or less, may be considered negligible.

Head: The head (H) is the distance measured vertically from the water surface (upstream of the commencement of drawdown) to the crest axis.

Design head (H_d): The design head is that value of head for which the ogee profile is designed.

Head due to velocity of approach (H_a): It is the velocity head given by

$$H_a = V_a^2 / 2g, \text{ where } V_a \text{ is the velocity of approach.}$$

Total energy head (H_t): It is equal to the actual head plus the head due velocity of approach. Thus,

$$H_t = H + H_a$$

If

$$H = H_d, \quad H_{ad} = H_d + H_a$$

Ogee profile : The ogee profile to be acceptable should provide maximum possible hydraulic efficiency, structural stability, and economy and also avoid the formation of objectionable sub-atmospheric pressures at the surface.

Fig. 11.3(a) shows the flow over a sharp crested weir. The ideal spillway [Fig. 11.3(b)] should have the form of the underside nappe of a sharp crested weir [Fig. 11.3(a)] when the flow rate corresponds to the maximum design capacity of spillway. However, when the flow rate corresponds to the maximum design capacity of spillway, the head during an unanticipated flood may rise higher than the design head [Fig. 11.3(c)] and the lower nappe of the falling jet may leave the ogee profile, thereby causing negative pressure and cavitation. On the contrary, when the head on the spillway is less than the design head, the falling jet would adhere to the profile of ogee, causing a positive hydrostatic pressure and reducing the discharging capacity.

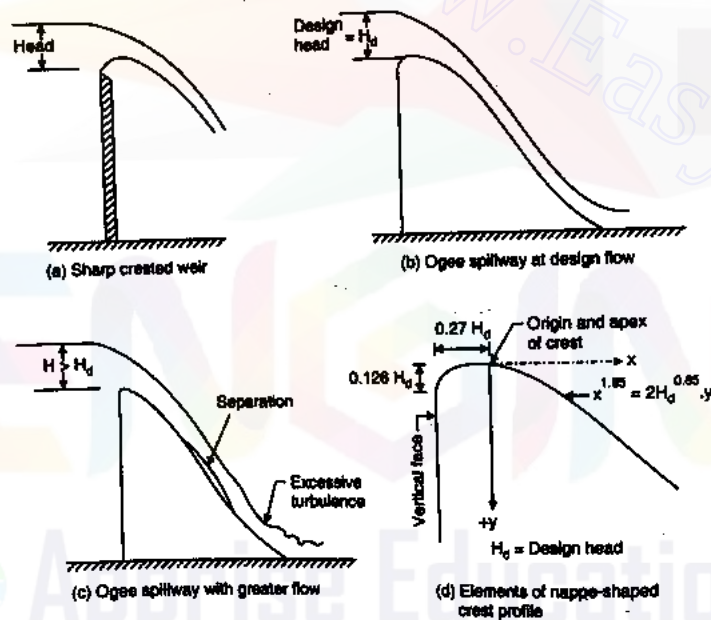


FIG. 11.3. Ogee SPILLWAY.

(a) Crest profile for vertical w/s face

(i) Design criteria of downstream profile

The details of the downstream crest profile for vertical w/s face are shown in [Fig. 11.3(d)]. The downstream curve of the ogee has the following equation:

$$x^{1.85} = 2H_d^{0.85}y \quad \dots(11.1)$$

where x and y are the co-ordinates of the crest profile measured from the apex of the crest, and H_d is the design head excluding head due to velocity of approach.

Table 11.1 gives the values of various parameters used in Eq. 11.1

TABLE 11.1. VALUES OF $x^{1.85}$ AND $H_d^{0.85}$

x	$x^{1.85}$	x	$x^{1.85}$	H_d	$H_d^{0.85}$	H_d	$H_d^{0.85}$	H_d	$H_d^{0.85}$
0.10	0.014	3.00	7.633	0.5	1.110	10.5	14.758	20.5	26.063
0.15	0.030	3.50	10.151	1.0	2.000	11.0	15.354	21.0	26.602
0.20	0.051	4.00	12.996	1.5	2.823	11.5	15.945	21.5	27.140
0.25	0.077	4.50	16.160	2.0	3.605	12.0	16.532	22.0	27.674
0.30	0.106	5.00	19.938	2.5	4.358	12.5	17.116	22.5	28.209
0.35	0.143	6.00	27.515	3.0	5.068	13.0	17.696	23.0	28.741
0.40	0.184	7.00	38.596	3.5	5.801	13.5	18.273	23.5	29.271
0.45	0.228	8.00	46.851	4.0	6.486	14.0	18.847	24.0	29.799
0.50	0.277	9.00	58.257	4.5	7.182	14.5	19.417	24.5	30.327
0.60	0.388	10.00	70.795	5.0	7.855	15.0	19.985	25.0	30.852
0.70	0.520	12.00	99.194	5.5	8.518	15.5	20.580		
0.80	0.662	14.00	131.928	6.0	9.172	16.0	21.112		
0.90	0.823	16.00	168.897	6.5	9.818	16.5	21.671		
1.00	1.00	18.00	210.017	7.0	10.460	17.0	22.229		
1.20	1.401	20.00	255.215	7.5	11.087	17.5	22.783		
1.40	1.864	25.00	386.646	8.0	11.713	18.0	23.335		
1.60	2.386	30.00	540.348	8.5	12.332	18.5	23.885		
1.80	2.967	35.00	718.684	9.0	12.946	19.0	24.433		
2.00	3.605	40.00	920.049	9.5	13.555	19.5	24.978		
2.50	5.447			10.0	14.159	20.0	25.521		

The profile is designed, following Eq. 11.1, for one value of design head (H_d) only which is generally chosen to give the maximum practicable hydraulic efficiency in keeping with the operational requirements, stability and economy. Generally, operational needs dictate that the spillway be operated under other than the design head also. For lower heads of flow, the pressure on the crest will be above atmospheric and for higher heads of flow, the pressure will be less than atmospheric and may, at times, become so low as to develop cavitation. The lower the design head of the crest profile, the greater will be the discharge coefficients for full range of heads. Model tests show that the design head may be exceeded by 25 percent upto which limit no harmful cavitation is seen to develop.

(ii) Design criteria of upstream crest profile

The w/s profile should be tangential to the vertical face and should have zero slope at the crest axis to ensure that there is no discontinuity along the surface of flow. The recommended curve, given below, incorporates the co-ordinates computed in the region close to the sharp-crested weir where accurate experimental data measurements are difficult and fits the experimental data over the remaining portion.

According to the latest analytical studies of U.S. Army, the upstream curve of the ogee shape has the following equation.

$$y = \frac{0.724(x + 0.270 H_d)^{1.85}}{H_d^{0.85}} + 0.126 H_d - 0.4315 H_d^{0.376}(x + 0.27 H_d)^{0.625} \quad \dots(11.2)$$

It should be noted here that the upstream curve at the crest should neither be made too sharp nor too broad. A broader curve at crest supports the lower nappe, produces positive hydrostatic pressure and thus reduces the discharging capacity. If the curve is sharp, the nappe will leave the ogee profile, thereby causing negative pressure and consequent cavitation, and increasing the coefficient of discharge. Table 11.2 gives the dimensionless co-ordinates of the u/s profile.

TABLE 11.2. DIMENSIONLESS COORDINATES OF U/S PROFILE

$\frac{x}{H_d}$	$\frac{y}{H_d}$	$\frac{x}{H_d}$	$\frac{y}{H_d}$
0.0000	0.0000	-0.2000	0.0494
-0.0200	0.0004	-0.2100	0.0556
-0.0400	0.0016	-0.2200	0.0624
-0.0600	0.0036	-0.2300	0.0701
-0.0800	0.0068	-0.2400	0.0787
-0.1000	0.0108	-0.2460	0.0836
-0.1200	0.0158	-0.2500	0.0889
-0.1400	0.0221	-0.2550	0.0948
-0.1600	0.0296	-0.2600	0.1016
-0.1700	0.0339	-0.2650	0.1099
-0.1800	0.0386	-0.2680	0.1165
-0.1900	0.0437	-0.2700	0.1260

(b) Crest profiles for inclined u/s face

In general, the equation for the nappe-shaped profile depends upon (i) head over the crest, (ii) inclination of the upstream face of the spillway and (iii) height of the spillway crest above the stream bed or above the bed of approach/entrance channel, on which the approach velocity depends. Based on extensive experiments conducted by U.S.B.R., U.S. Army Corps of Engineers have developed several standard crests of overflow spillways at its waterways experiment station (W.E.S.) at Vicksburg. These shapes, given in Fig. 11.4 for both vertical as well as inclined face, are known as *WES-standard spillway crests*. IS 6934-1973 has also adopted and recommended these shapes. These shapes can be represented by the following general equation :

$$x' = K H_d^{n-1} y \quad \dots(11.3)$$

where x and y are the coordinates of the profile of the crest of the spillway with origin at the highest point of the crest, H_d is the design head (excluding the head due to velocity of approach) and K and n are constants the values of which, given in Table 11.3, depends on the slope of the upstream face of the spillway. For vertical face, $K = 2.000$ and $n = 1.850$, and hence Eq. 11.3 reduces to Eq. 11.1. For intermediate slopes, approximate values of K and n may be obtained from Fig. 11.5.

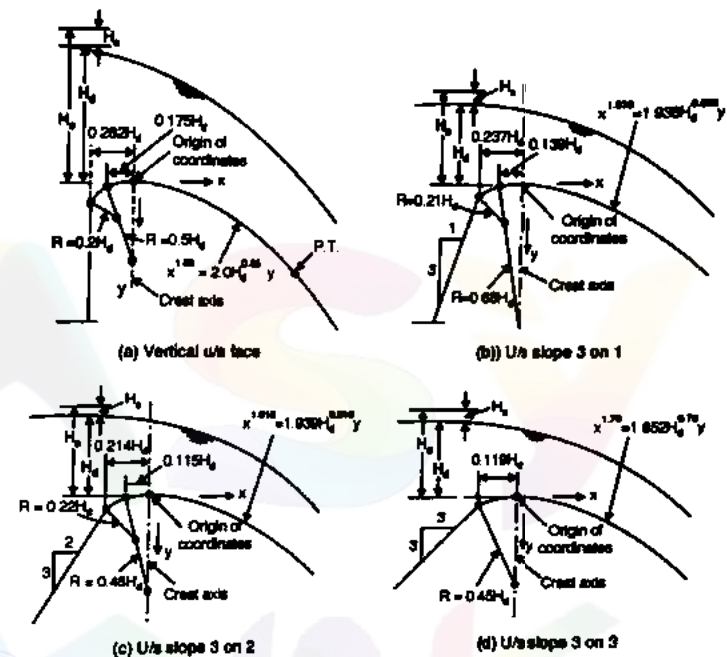


FIG. 11.4. WES-STANDARD SPILLWAY CRESTS

TABLE 11.3. VALUES OF K AND n

S.N.	Slope of u/s face	K	n
1.	Vertical	2.000	1.850
2.	3(V) to 1(H)	1.936	1.836
3.	3(V) to 2(H)	1.939	1.810
4.	3(V) to 3(H)	1.652	1.780

Thus, the equations for spillways shapes for the three standard slopes of u/s face are as under :

(i) For 3(V) to 1(H) : Slope 3 on 1
 $x^{1.836} = 1.936 H_d^{0.836} y \quad \dots(11.3 a)$

(ii) For 3(V) to 2(H) : Slope 3 on 2
 $x^{1.810} = 1.939 H_d^{0.810} y \quad \dots(11.3 b)$

(iii) For 3(V) to 3(H) : Slope 3 on 3
 $x^{1.78} = 1.652 H_d^{0.78} y \quad \dots(11.3 c)$

In all these equations, x is taken positive towards downstream and y is taken positive in downward direction. Eq. 11.3 is applicable only for the positive values of x and y .

Tables 11.4, 11.5, and 11.6 give downstream profile data (as per Eq. 11.3) in cases of upstream slopes of 3 on 1, 3 on 2 and 3 on 3 respectively, given by IS : 6934-1973. Similarly, Table 11.7 gives the coordinates of w/s quadrants for the above three slopes.

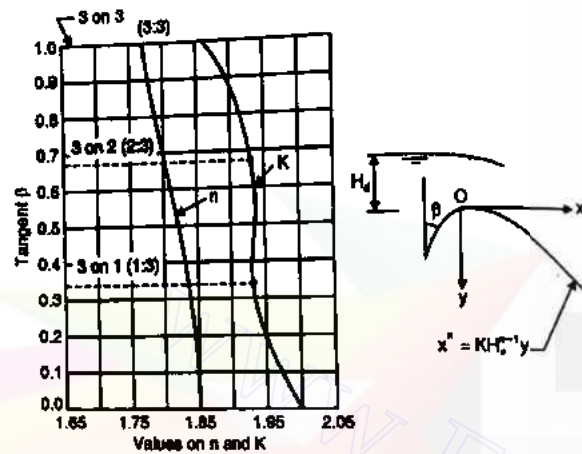


FIG. 11.5. CREST SHAPE, DOWNSTREAM PROFILE, n AND K VALUES (IS : 6934-1973)

TABLE 11.4. DOWNSTREAM PROFILE DATA FOR U/S SLOPE 3 ON 1

x	$x^{1.81}$	x	$x^{1.81}$	x	$x^{1.81}$	x	$x^{1.81}$	H_d	$1.936 H_d^{0.58}$	H_d	$1.936 H_d^{0.58}$
0.05	0.0041	1.90	3.2493	4.70	17.1385	17.50	191.5216	0.50	1.0845	13.50	17.0550
0.10	0.0146	2.00	3.5702	4.80	17.8129	18.00	201.8881	1.00	1.9360	14.00	17.5820
0.15	0.0307	2.10	3.9043	4.90	18.5012	18.50	212.0824	1.50	2.7172	14.50	18.1954
0.20	0.0521	2.20	4.2529	5.00	19.2003	19.00	222.7365	2.00	3.4559	15.00	18.8259
0.25	0.0785	2.30	4.6146	5.50	22.8721	19.50	233.6165	2.50	4.1647	15.50	19.4436
0.30	0.1097	2.40	4.9896	6.00	26.8340	20.00	244.7322	3.00	4.8510	16.00	19.6585
0.35	0.1455	2.50	5.3780	6.50	31.0820	20.50	256.0826	3.50	5.5176	16.50	20.1707
0.40	0.1859	2.60	5.7795	7.00	35.6123	21.00	267.6689	4.00	6.1692	17.00	20.6805
0.45	0.2308	2.70	6.1942	7.50	40.4215	21.50	279.4841	4.50	6.8076	17.50	21.1878
0.50	0.2801	2.80	6.6219	8.00	45.5065	22.00	291.5332	5.00	7.4344	18.00	21.6927
0.55	0.3337	2.90	7.0625	8.50	50.8643	22.50	316.8135	5.50	8.0510	18.50	22.1953
0.60	0.3915	3.00	7.5170	9.00	56.4923	23.00	318.3240	6.00	8.6590	19.00	22.6957
0.65	0.4534	3.10	7.9825	9.50	62.3879	23.50	329.0642	6.50	9.2577	19.50	23.1939
0.70	0.5195	3.20	8.4617	10.00	68.5489	24.00	342.0329	7.00	9.8493	20.00	23.6901
0.75	0.5897	3.30	8.9535	10.50	74.9727	24.50	355.2395	7.50	10.4341	20.50	24.1841
0.80	0.6639	3.40	9.4579	11.00	81.6677	25.00	368.6631	8.00	11.0126	21.00	24.6763
0.85	0.7420	3.50	9.9749	11.50	88.6015	25.50	382.3031	8.50	11.5851	21.50	25.1666
0.90	0.8241	3.60	10.5044	12.00	95.8025	26.00	396.1786	9.00	12.1521	22.00	25.6549
0.95	0.9010	3.70	11.0463	12.50	103.2587	26.50	410.2792	9.50	12.7140	22.50	26.1415
1.00	1.0000	3.80	11.6007	13.00	110.9686	27.00	424.6038	10.00	13.2711	23.00	26.6262
1.10	1.1812	3.90	12.1673	13.50	118.9304	27.50	439.1519	10.50	13.8235	23.50	27.1093
1.20	1.3878	4.00	12.7460	14.00	127.1425	28.00	453.9229	11.00	14.3717	24.00	27.5907
1.30	1.6188	4.10	13.3374	14.50	135.6038	28.50	468.9161	11.50	14.9159	24.50	28.0704
1.40	1.8847	4.20	13.9407	15.00	144.3124	29.00	484.1308	12.00	15.4561	25.00	28.5485
1.50	2.1853	4.30	14.5563	15.50	153.2671	29.50	499.5683	12.50	15.9927		
1.60	2.5201	4.40	15.1837	16.00	162.4667	30.00	515.224	13.00	16.5258		
1.70	2.8891	4.50	15.8233	16.50	171.9097						
1.80	3.2923	4.60	16.4749	17.00	181.6851						

TABLE 11.5. DOWNSTREAM PROFILE DATA FOR U/S SLOPE 3 ON 2

x	$x^{1.81}$	x	$x^{1.81}$	x	$x^{1.81}$	x	$x^{1.81}$	H_d	$1.936 H_d^{0.58}$	H_d	$1.936 H_d^{0.58}$
0.05	0.0044	1.90	3.1955	4.70	16.4626	17.50	177.7865	0.50	1.1060	13.50	15.9644
0.10	0.0155	2.00	3.5064	4.80	17.1020	18.00	187.0868	1.00	1.9390	14.00	16.4415
0.15	0.0323	2.10	3.8302	4.90	17.7524	18.50	196.5988	1.50	2.6929	14.50	16.9154
0.20	0.0543	2.20	4.1666	5.00	18.4135	19.00	206.3213	2.00	3.3965	15.00	17.3855
0.25	0.0813	2.30	4.5157	5.50	21.8805	19.50	216.2633	2.50	4.0720	15.50	17.8545
0.30	0.1131	2.40	4.8773	6.00	25.6126	20.00	226.3937	3.00	4.7211	16.00	18.3196
0.35	0.1495	2.50	5.2514	6.50	29.6056	20.50	236.7417	3.50	5.3490	16.50	18.7819
0.40	0.1904	2.60	5.6377	7.00	33.8554	21.00	247.2959	4.00	5.9600	17.00	19.2418
0.45	0.2357	2.70	6.0363	7.50	38.3584	21.50	258.0058	4.50	6.5566	17.50	19.6987
0.50	0.2852	2.80	6.4470	8.00	43.1155	22.00	269.0203	5.00	7.1407	18.00	20.1539
0.55	0.3389	2.90	6.8697	8.50	48.1114	22.50	280.1885	5.50	7.7139	18.50	20.6057
0.60	0.3967	3.00	7.3049	9.00	53.3555	23.00	291.5596	6.00	8.2771	19.00	21.0556
0.65	0.4585	3.10	7.7511	9.50	58.8401	23.50	303.1328	6.50	8.8316	19.50	21.5033
0.70	0.5244	3.20	8.2096	10.00	64.5655	24.00	314.9069	7.00	9.3779	20.00	21.9489
0.75	0.5941	3.30	8.6798	10.50	70.5265	24.50	326.8817	7.50	9.9169	20.50	22.3922
0.80	0.6677	3.40	9.1617	11.00	76.7222	25.00	339.0560	8.00	10.4491	21.00	22.8337
0.85	0.7452	3.50	9.6552	11.50	83.1501	25.50	351.4291	8.50	10.9751	21.50	23.2730
0.90	0.8264	3.60	10.1603	12.00	89.8086	26.00	364.0003	9.00	11.4951	22.00	23.7106
0.95	0.9113	3.70	10.6769	12.50	96.6957	26.50	376.7689	9.50	12.0097	22.50	24.1460
1.00	1.0000	3.80	11.2040	13.00	103.8096	27.00	389.7342	10.00	12.5192	23.00	24.5797
1.10	1.1883	3.90	11.7443	13.50	111.1486	27.50	402.8953	10.50	13.0239	23.50	25.0117
1.20	1.3910	4.00	12.2950	14.00	118.7112	28.00	416.2517	11.00	13.5240	24.00	25.4418
1.30	1.6078	4.10	12.8570	14.50	126.4958	28.50	429.8029	11.50	14.0197	24.50	25.8705
1.40	1.8386	4.20	13.4302	15.00	134.5009	29.00	443.5477	12.00	14.5116	25.00	26.2972
1.50	2.0832	4.30	14.0145	15.50	142.7251	29.50	457.4860	12.50	14.9993		
1.60	2.3413	4.40	14.6100	16.00	151.1670	30.00	471.6172	13.00	15.4836		
1.70	2.6128	4.50	15.2165	16.50	159.8254						
1.80	2.8980	4.60	15.8340	17.00	168.6990						

TABLE 11.6. DOWNSTREAM PROFILE DATA FOR U/S SLOPE 3 ON 3

x	$x^{1.78}$	x	$x^{1.78}$	x	$x^{1.78}$	x	$x^{1.78}$	H_d	$1.852 H_d^{0.78}$	H_d	$1.852 H_d^{0.78}$
0.05	0.0048	1.90	3.1345	4.70	15.716	16.50	146.934	0.50	1.0785	14.50	14.9110
0.10	0.0166	2.00	3.4342	4.80	16.316	17.00	154.953	1.00	1.8520	15.00	15.3106
0.15	0.0342	2.10	3.7458	4.90	16.926	17.50	163.158	1.50	2.5409	15.50	15.7072
0.20	0.0570	2.20	4.0692	5.00	17.546	18.00	171.548	2.00	3.1810	16.00	16.1011
0.25	0.0848	2.30	4.4043	5.50	20.789	18.50	180.122	2.50	2.7847	16.50	16.4922
0.30	0.1173	2.40	4.7508	6.00	24.272	19.00	188.878	3.00	4.3631	17.00	16.8908
0.35	0.1543	2.50	5.1090	6.50	27.989	19.50	197.816	3.50	4.9205	17.50	17.2667
0.40	0.1957	2.60	5.4782	7.00	31.935	20.00	206.935	4.00	5.4606	18.00	17.6503
0.45	0.2413	2.70	5.8590	7.50	36.109	20.50	216.233	4.50	5.9860	18.50	18.0316
0.50	0.2912	2.80	6.2509	8.00	40.504	21.00	225.710	5.00	6.4988	19.00	18.4105
0.55	0.3450	2.90	6.6538	8.50	42.120	21.50	235.364	5.50	7.0004	19.50	18.7874
0.60	0.2490	3.00	7.0676	9.00	49.952	22.00	245.195	6.00	7.4921	20.00	19.1621
0.65	0.4645	3.10	7.4924	9.50	54.997	22.50	255.202	6.50	7.9747	20.50	19.5349
0.70	0.5300	3.20	7.9279	10.00	60.256	23.00	265.384	7.00	8.4492	21.00	19.9051
0.75	0.5992	3.30	8.3745	10.50	65.723	23.50	275.741	7.50	8.9164	21.50	20.2735
0.80	0.6722	3.40	8.8314	11.00	71.396	24.00	286.270	8.00	9.3766	22.00	20.6411
0.85	0.7488	3.50	9.2991	11.50	77.097	24.50	296.972	8.50	9.8308	22.50	21.0059
0.90	0.8290	3.60	9.7771	12.00	83.358	25.00	307.846	9.00	10.2789	23.00	21.3693
0.95	0.9127	3.70	10.266	12.50	89.640	25.50	318.890	9.50	10.7218	23.50	21.7308
1.00	1.0000	3.80	10.765	13.00	96.120	26.00	330.105	10.00	11.1594	24.00	22.0905
1.10	1.1849	3.90	11.274	13.50	102.800	26.50	341.490	10.50	11.5922	24.50	22.4386
1.20	1.3334	4.00	11.794	14.00	109.675	27.00	353.043	11.00	12.0206	25.00	22.8061
1.30	1.5952	4.10	12.324	14.50	116.744	27.50	364.764	11.50	12.4447		
1.40	1.8202	4.20	12.864	15.00	124.006	28.00	376.653	12.00	12.8647		
1.50	2.0580	4.30	13.415	15.50	131.459	28.50	388.708	12.50	13.2810		
1.60	2.3085	4.40	13.975	16.00	139.102	29.00	400.930	13.00	13.6935		
1.70	2.5716	4.50	14.545			29.50	413.317	13.50	14.1026		
1.80	2.8470	4.60	15.125			30.00	425.869	14.00	14.5084		

TABLE 11.7. COORDINATES OF UPSTREAM QUADRANT FOR U/S SLOPES 3 ON 1, 3 ON 2 AND 3 ON 3.

u/s slope 3 on 1		u/s slope 3 on 2		u/s slope 3 on 3	
$\frac{x}{H_d}$	$\frac{y}{H_d}$	$\frac{x}{H_d}$	$\frac{y}{H_d}$	$\frac{x}{H_d}$	$\frac{y}{H_d}$
0.000	0.0000	0.0000	0.0000	0.000	0.0000
-0.020	0.0004	-0.0200	0.0004	-0.020	0.0004
-0.040	0.0016	-0.0400	0.0016	-0.040	0.0016
-0.060	0.0037	-0.0600	0.0036	-0.060	0.0036
-0.080	0.0067	-0.0800	0.0066	-0.080	0.0065
-0.100	0.0106	-0.1000	0.0104	-0.100	0.0103
-0.120	0.0156	-0.1200	0.0153	-0.120	0.0150
-0.140	0.0291	-0.1400	0.0283	-0.140	0.0275
-0.170	0.0330	-0.1700	0.0365	-0.170	0.0313
-0.180	0.0375	-0.1900	0.0412	-0.180	0.0354
-0.190	0.0425	-0.2000	0.0464	-0.190	0.0399
-0.200	0.0480	-0.2100	0.0554	-0.200	0.0450
-0.210	0.0550				
-0.220	0.0650				
-0.230	0.0800				

Further details of downstream profile

Since Eq. 11.3 is applicable only for the positive values of x and y , it gives the crest shape downstream from the vertex (or the origin of coordinates). The curved profile of the crest is continued till it meets tangentially the straight sloping surface of the downstream face of the overflow dam. The location of the point of tangency depends upon the slope of the d/s face, which usually varies from 0.7:1 to 0.8:1, based on stability requirements. At the end of the straight sloping face of the spillway, a curved circular surface, called bucket is provided to create a smooth transition of flow from the spillway surface to the outlet channel or the river on the downstream side. The bucket also serves the purpose of dissipating the energy as well as prevention of scour at the downstream side, as discussed in §11.10. The radius R (in m) of the bucket is obtained from the following approximate expression :

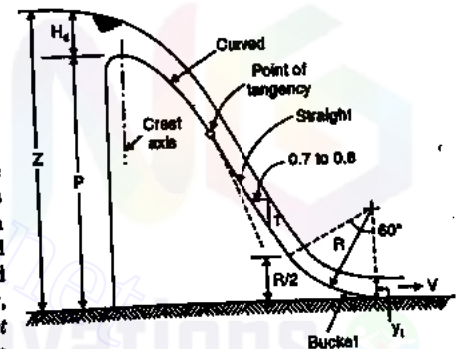


FIG. 11.6. PROVISION OF BUCKET AT D/S TOE

$$R = (10)^a \quad \dots(11.4)$$

$$a = \frac{V + 6.4 H_d + 4.88}{3.6 H_d + 19.52} \quad \dots(11.4 a)$$

where

and V = velocity of flow at the toe of the spillway (m/s), given by

$$V = \sqrt{2g(Z + H_d - y_t)} \quad \dots(11.4 a)$$

Z = fall or vertical distance from u/s reservoir level to the floor level at the d/s toe (Fig. 11.6)

y_t = depth of water at the toe.

H_d = design head, exclusive of velocity of approach.

Generally, $R = P/4$ is found to be satisfactory, where P is the height of spillway crest above approach channel or river bed.

Practical profile of spillway

When the ogee profile, given by Eq. 11.1 or 11.3 is plotted over the triangular profile of the non-overflow section of the gravity dam, it is found (Fig. 11.7 a) that the d/s curved profile of the spillway goes beyond the straight downstream face of the dam, thus requiring the thickening of the d/s face to accommodate the ogee profile. However, this thickening (or extra concrete) can be saved by shifting the curve of the nappe towards the upstream direction till the ogee curve becomes tangential to the down stream face of the dam, as shown in Fig. 11.7 (b).

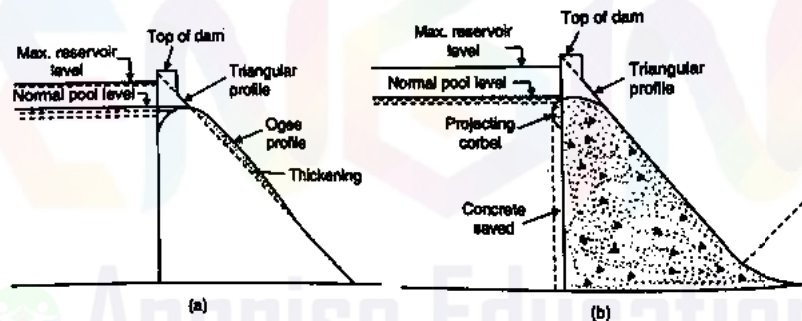


FIG. 11.7. PRACTICAL PROFILE OF SPILLWAY.

Such shifting of the ogee profile towards u/s side will give rise to a projecting portion known as *corbel* at the u/s side of the dam. The portion below the corbel need not be concreted, thus giving rise to saving in concrete, as shown in Fig. 11.7 (b). However, the provision of corbel will interfere with the gate operation.

Overflow dams with crest offsets and risers

Whenever structural requirements permit, removal of some mass from the upstream face, leading to *offsets* and *risers* results in economy (Fig. 11.8). The maximum permitted projection (or offset S) from the crest line is equal to $0.315 H_d$ while the maximum depth of bulging is equal to $0.25 H_d$. Fig. 11.8 (a) shows vertical faced overhang; the vertical depth M (called *riser* of the crest) should be equal to or greater than $0.5 H_d$. Flow conditions would be extremely unstable for M/H_d ratio less than 0.53; hence crests with M/H_d ratio less than 0.5 are not recommended. The crest shape given by Eq. 11.2 is applicable to over hanging crest having M/H_d ratio greater than 0.5. Also, the ratio M/N should not be less than 0.5, though it can have a zero value. For M/N ratio between 0 to 0.5, the flow conditions become extremely unstable.

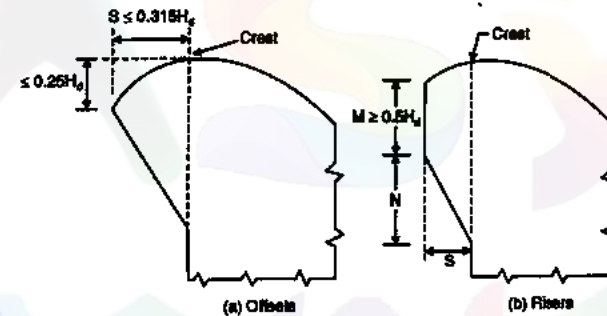


FIG. 11.8. PROVISION OF OFFSETS AND RISERS

Discharge equation. The discharge over an ogee shaped spillway is given by

$$Q = C_d L_e H_e^{3/2} \quad \dots(11.5)$$

where Q = discharge.

C_d = a variable coefficient of discharge which is influenced by several factors such as (i) ratio of actual total head with design total head (ii) depth of approach (iii) relation of actual crest shape to the ideal nappe shape (iv) upstream face slope, (v) downstream apron interference and (vi) downstream submergence. Its value varies from 1.6 to 2.28.

L_e = effective length of crest.

H_e = total head on the crest, including velocity of approach head = $H + H_a$.

Factors affecting coefficient of discharge (C_d)

Various factors affecting the coefficient of discharge (C_d) have already been mentioned above. Let us elaborate some of these factors.

(i) **Ratio of actual total head to the design total head**

Fig. 11.9 gives the coefficient of discharge C_d , as a function of H/H_d for the crest profile given for vertical upstream face, as given by IS : 8034-1973. When $H = H_d$, the value of $C_d = 2.2$ for high overflow spillway. Prototype tests for the low-head ranges and model tests for values of H/H_d above 0.4 form the basis. The coefficient of discharge for inclined upstream face spillways will not differ significantly from those given in Fig. 11.9.

(ii) **Depth of approach channel below spillway crest**

The height (P) of the spillway crest above stream bed or bed of approach channel affects the velocity of approach (V_a), which in turn affects the coefficient of discharge. Fig. 11.10 shows the variation of C_d with P/H_d ratio. When P increases, the velocity of approach (V_a) decreases, but the coefficient of discharge increases. From model tests on spillways, it is known that when $P \geq 1.33 H_d$, the effect of velocity of approach is negligible.

(iii) **Downstream submergence (Fig. 11.11 a)**

The coefficient of discharge reduces due to downstream submergence. Fig. 11.11 (a) shows the variation of C_s/C_d ratio with the degree of downstream submergence in terms of h/H ratio, where C_s is the coefficient of submerged discharge and h is the depth of submergence. The discharge in such case is given by

$$Q = C_s L_e H_c^{3/2} \quad \dots(11.5 a)$$

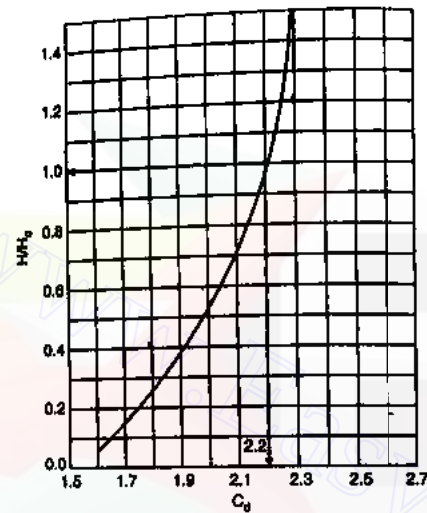


FIG. 11.9. VARIATION OF C_d WITH H/H_d

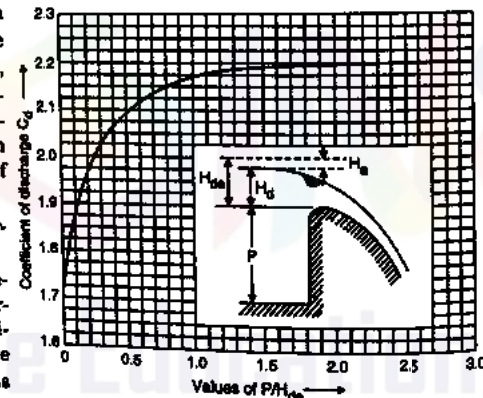


FIG. 11.10. EFFECT OF DEPTH OF APPROACH CHANNEL BELOW SPILLWAY CREST

It would be noticed that even for a degree of submergence of as much as 60%, the reduction of C_d is only 5 percent.

(iv) **D/S apron interference :**

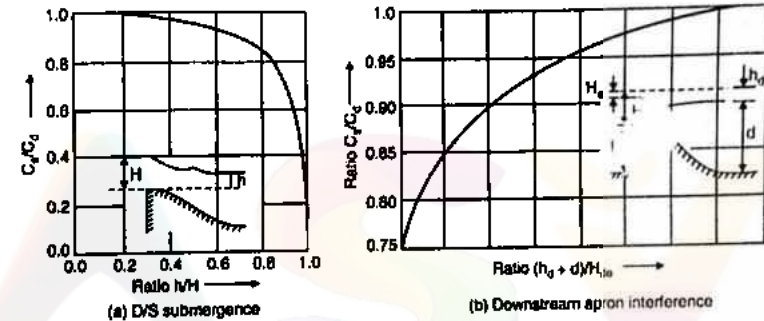


FIG. 11.11. EFFECTS OF D/S SUBMERGENCE AND APRON INTERFERENCE

Fig. 11.11 (b) shows the effect of d/s apron interference on coefficient of discharge (C_d). It is observed that when the value of ratio $(h_d + d)/H_d$ exceeds 1.7, the downstream apron has little effect on coefficient of discharge. However, for lower values of this ratio, the actual coefficient of discharge (C_s) is lower. From the geometry of Fig. 11.11 (b) we have $h_d + d = P + H_d$.

(v) **Slope of upstream face of spillway**

For small values of P/H_d ratio, a spillway with sloping upstream face is slightly higher value of coefficient of discharge than for the spillway with vertical u/s face. However, for large values of P/H_d ratio, the value of C_d for spillways with sloping u/s face tends to decrease.

Effective length (L_e) of the crest

The effective length of the crest is given by

$$L_e = L - 2(NK_p + K_a) H_c \quad \dots(11.6)$$

where L_e = effective length of the crest

L = net length of the crest

N = number of piers

K_p = pier contraction coefficient

K_a = abutment contraction coefficient

The pier contraction coefficient depends upon many factors such as (i) shape and location of pier nose, (ii) thickness of pier, (iii) approach velocity and (iv) actual head

of flow in relation to the design head. For the flow under the design head, the values of K_p can be taken from Table 11.8.

TABLE 11.8. PIER CONTRACTION COEFFICIENT (K_p)

Pier Condition	K_p
1. Square nosed piers with corners rounded on a radius equal to 0.1 of pier thickness.	0.02
2. Round-nosed piers.	0.01
3. Pointed-nose piers.	0.0

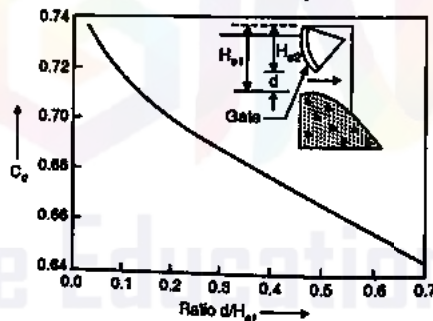
Similarly, the *abutment contraction coefficient* depends upon several factors such as (i) shape of abutment, (ii) angle between upstream approach wall and axis of flow (iii) approach velocity and (iv) actual head of flow in relation to the design head. For the flow under the design head, the abutment contraction coefficient may be taken from Table 11.9.

TABLE 11.9. ABUTMENT CONTRACTION COEFFICIENT (K_a)

Abutment Condition	K_a
1. Square abutment with head wall at 90° to the direction of flow.	0.2
2. Rounded abutment with head wall at 90° to the direction of flow when $0.5 H_d \geq r \geq 0.15 H_d$	0.10
3. Rounded abutment where $r > 0.5 H_d$ and wall is placed not more than 45° to direction of flow (where H_d = design head and r = radius of abutment rounding).	0.0

Orifice flow : Discharge formula at partial gate opening

Orifice flow is the flow that occurs at partial gate opening in a gated spillway. Sub-atmospheric pressures develop on the crest immediately below the gate if the crest profile is steeper than the one conforming to the trajectory. Where a thicker crest is needed for stability and the discharge efficiency is unimportant, the trajectory profile is an obvious choice and avoids sub-atmospheric pressures. Where the crest is to be shaped to the nappe profile, the sub-atmospheric pressure zone can be reduced by placing the gate sill downstream of the crest. In such a case the trajectory is steeper and conforms more nearly to the free overflow lower nappe profile. Model tests have shown that even when the gate is located at the crest axis and the head exceeds the design head (H_d) by 33 $\frac{1}{3}$ %, negative pressure on the crest axis observed will be less than $0.15 H_d$. While deciding on the design and crest profile, a certain extent of sub-atmospheric

FIG. 11.12. COEFFICIENT OF DISCHARGE (C_d) FOR ORIFICE FLOW

pressure may be considered permissible taking into account the rareness and shortness of duration of occurrence of sub-atmospheric pressure.

The discharge for a gated spillway at partial gate opening is given by the following low-head orifice formula :

$$Q = \frac{2}{3} C_{d0} \sqrt{2g} L_e (H_{d1}^3 - H_{d2}^3) \quad \dots(11.7)$$

where C_{d0} = coefficient of discharge of the orifice

L_e = effective length of the spillway crest

H_{d1} = total head above the bottom of the opening

H_{d2} = total head above the top of the opening

Fig. 11.12 shows the variation of coefficient of discharge (C_{d0}) with different values of d/H_{d1} where d is the height of opening. However, C_{d0} is also influenced by approach condition as well as downstream conditions.

11.4. SIDE CHANNEL SPILLWAY

A side channel spillway is the one in which the flow, after passing over a weir or ogee crest, is carried away by channel running essentially parallel to the crest. Discharge characteristics of a side channel spillway are similar to those of an ordinary overflow spillway and are dependent on the selected profile of the weir crest. However, this flow may differ from that of overflow spillway in that the flow in the trough may partly submerge the flow over the crest. Side channel spillway is suitable for earth or rockfill dams in narrow canyons and for other situations where direct over-flow is not permissible. Side channel spillway is also the best choice where a long overflow crest is desired in order to limit the surcharge head and the abutments are steep. This type of spillway is also desirable where the spillway discharge is to be connected to narrow discharge channel or

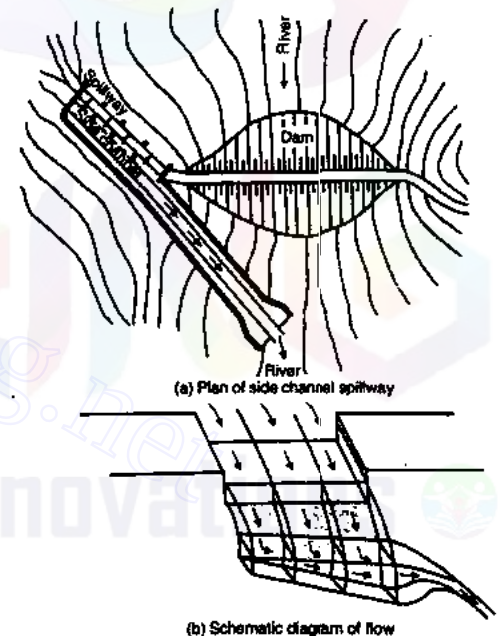


FIG. 11.13. SIDE CHANNEL SPILLWAY.

a tunnel. Fig. 11.13(a) shows the plan of a side channel spillway, while Fig. 11.13(b) shows the schematic diagram of flow through such a spillway. The hydraulics and design of a side channel spillway is beyond the scope of this book and hence has not been included.

11.5. CHUTE OR TROUGH SPILLWAY

A chute spillway is the one which passes the surplus discharge through a steep sloped open channel, called a chute or trough, placed either along a dam abutment or through a saddle. Generally, this type of spillway is provided on earth or rockfill dam, and is isolated from the main dam. Its crest is kept normal to its centre line. It consists of a discharge channel to the river in an excavated trench which is usually paved with concrete in whole or in part. The crest or the spillway proper is usually of insignificant height or actually flat, as shown in Fig. 11.14(c). The chute is sometimes of constant width, but usually narrowed for economy and then widened near the end to reduce discharge velocity. Factors influencing the selection of chute spillways are the simplicity of their design and construction, their adaptability to almost any foundation condition and the overall economy often obtained by the use of large amounts of spillway excavation in the dam embankment.

11.6. CONDUIT OR TUNNEL SPILLWAY

Conduit spillway or tunnel spillway is the one in which a closed channel is used to convey the discharge around or under a dam. The closed channel may be in the form of a vertical or inclined shaft, a horizontal tunnel through earth dam or a conduit constructed with open cut and backfilled with earth materials. These spillways are designed to flow partly full. Full flow is not allowed in the tunnel or conduit, lest the siphonic action may develop and negative pressures may be created in the conduit. To ensure free flow in the tunnel, the ratio

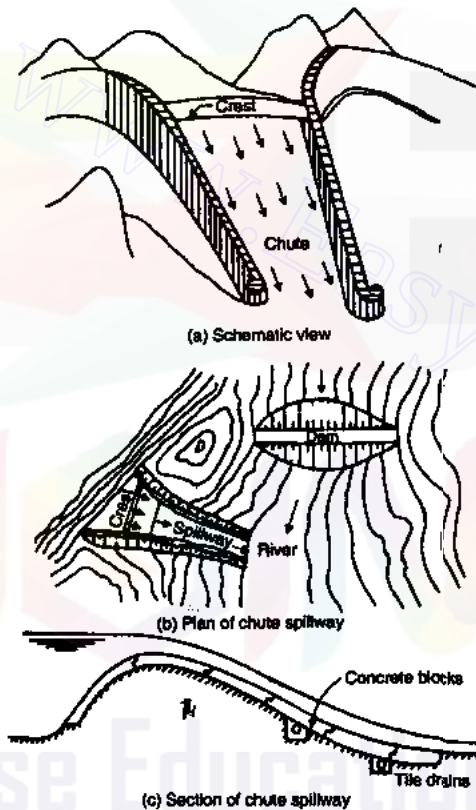


FIG. 11.14 CHUTE OR TROUGH SPILLWAY.

of flow area to the total tunnel area is often limited to 75% and air vents are provided at critical points along the tunnel or conduit to ensure an adequate air supply which will avoid unsteady flow through the spillway.

11.7. SHAFT SPILLWAY

Shaft spillway, drop inlet spillway or morning glory spillway is the one which has horizontally positioned lip through which water enters and then drops through a vertical or sloping shaft, and then to a horizontal conduit which convey the water past the dam. A shaft spillway can often be used where there is inadequate space for other types of spillways. It is generally not desirable to use a spillway over or through an earth dam. Thus, on an earth dam location, if there is no enough space or if the topography prevents the use of a chute or side channel spillway, the best alternative would be to use shaft spillway.

A shaft spillway normally consists of three elements: (i) overflow control weir (called a flared inlet or morning glory), (ii) vertical control, and (iii) closed discharge channel. Small shaft spillway may be constructed entirely of metal or concrete pipe or clay tile. The vertical shaft of large structures is usually made of reinforced concrete and the horizontal conduit is tunnelled in rock. Sometimes the diversion tunnel, used for diverting river water during the construction may be plugged at the upstream end, and then connected to the vertical shaft of shaft spillway as shown in Fig 11.15(a) and (b).

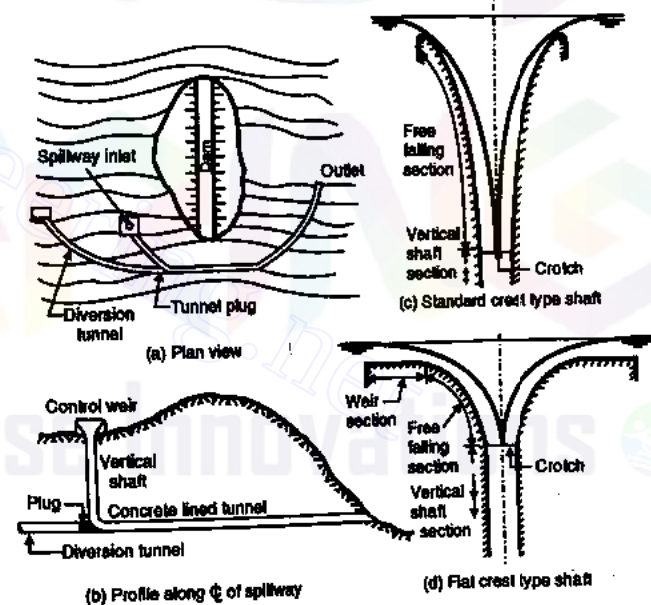


FIG. 11.15 SHAFT SPILLWAY.

There are two general types of shaft spillways : (i) *Standard crest type* and (ii) *Flat crest type*. In the flat crest type [Fig. 11.15(d)], the weir section precedes the free fall section, while in the standard crest type [Fig. 11.15(c)], there is no weir section and the water begins its free fall immediately upon leaving the crest. The standard crest type has the advantage of having smaller crest diameter since its coefficient of discharge is greater than that of flat crest. However, flat crest design has smaller funnel diameter, and is therefore, preferred in cases where the shaft has to be excavated in rock.

11.8. SIPHON SPILLWAYS

Instead of allowing water to spill over the crest of a dam or weir, the surplus water may be discharged downstream through a siphon spillway consisting of one or more siphon units. A siphon spillway is the one which utilizes the siphonic action to discharge the surplus water. Generally, a siphon spillway consists of a closed conduit system formed in the shape of an inverted U, positioned so that the inside of the bend of the upper passageway is at normal reservoir level. When the water level in the reservoir rises above its normal level, water flows over the crest, and then siphonic action takes place. Continuous flow is then maintained by suction effect due to the gravity pull of the water in the lower leg of the siphon.

There are generally two types of siphon spillways :

- (i) Saddle siphon spillway
- (ii) Volute siphon spillway.

SADDLE SIPHON SPILLWAY

Fig. 11.16 shows two types of saddle siphons, the former being more common. It essentially consists of reinforced concrete hood constructed over an overflow section of gravity dam. The inlet or mouth of the main hood is kept submerged in water so that floating debris etc. do not enter the siphon. A small deprimer hood is kept above the main hood, and both these hoods are connected through an air vent. The

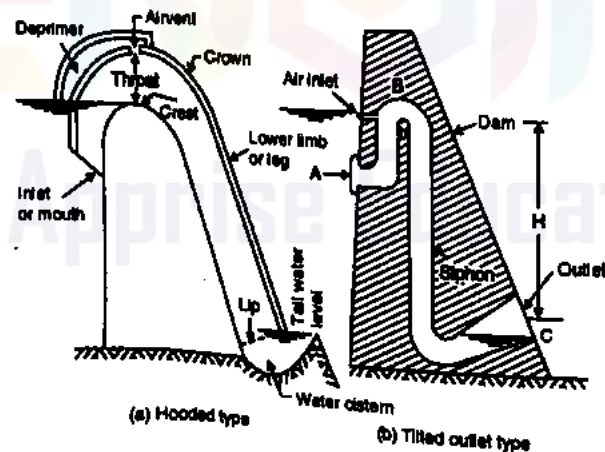


FIG. 11.16. SADDLE SIPHONS.

inlet of the deprimer hood is kept just at the reservoir level. The lower limb or leg of the main hood is generally kept submerged in the tailwater so that there is no air entry from the down stream end.

When the water level in the reservoir increases, it seals the air entry through the mouth of the deprimer dome. Water spills over the crest of the spillway. Since air entry is sealed both from u/s as well as d/s end, the spilling water sucks all the remaining air from the hood within minutes. Siphonic action is established after the air in the bend over the crest has been exhausted. This action is known as *priming*. The siphon runs full and water is discharged downstream under a greater effective (siphonic) head. During receding flood, when water level has gone down just to the reservoir level, air enters through the mouth of deprimer dome and the siphonic action is broken. This action is called *depriming* of the siphon and is achieved through the deprimer dome.

Priming devices. Priming is the action of the siphon spillway from the moment the water first spills over the crest to the instant when the whole siphon is filled with water. Priming is achieved by sealing the down-stream outlets. The following are the common methods employed for early priming in spillways :

1. *Provision of cup type basin.* This is shown in Fig. 11.16(a). The lip of the lower limb is kept immersed in the water stored in the cup. This will prevent the entry of air.

2. *Provision of tilted outlet.* This is another type of saddle siphon in which the vertical shaft has a tilted outlet which maintains a *water seal* [Fig. 11.16(b)]. The air inlet is maintained just at the reservoir level.

3. *Provision of auxiliary or baby siphon* (Fig. 11.17). A small auxiliary or baby siphon is formed just below the crest. When the water level reaches the crest, the baby siphon will run full. The sheet of water issuing from it is arranged to shoot

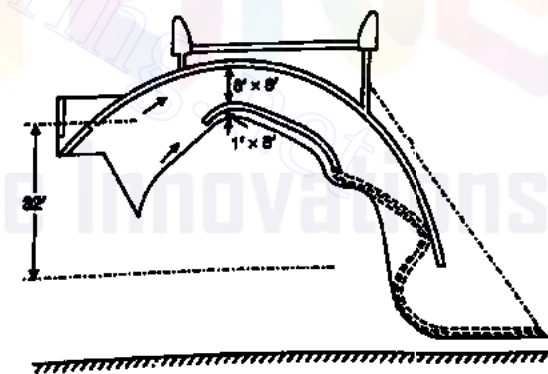


FIG. 11.17 MARAMSILI SIPHON INSTALLATION.

across the lower limb of the siphon so as to seal it and prevent air entering from below. The priming thus takes place. One such installation was made at Maramsili siphon (India) in 1921. The baby siphon is 1 ft. (0.3 m) high and 8 ft. (2.4 m) wide at throat while the main siphon is 8 ft. (2.4 m) high and 8 ft. (2.4 m) wide. The priming is followed just in 15 to 30 seconds. Though priming is good, but the effective top water level for the consideration of storage is not the crest of the main siphon but the crest of the auxiliary siphon. Hence from the point of view of maximum discharge, the auxiliary siphon may be looked upon as an obstruction. As the flow over the main siphon increases, the jet through the baby siphon is depressed down and care must be taken that the combined jet does not strike below the lip of the lower leg. The baby siphon is expensive and difficult to construct.

4. Provision of tudor or step [Fig. 11.9(a)]. A horizontal tudor or step, provided near the lip, deflects the jet to strike the lower limb thus sealing the air entry.

5. Provision of step just after crest [Fig. 11.18(b)]. A step is provided just after the crest to deflect the water which strikes to the opposite side, thus providing a water seal.

6. Provision of hinged adjustable plate. Fig. 11.19(a) shows a system suggested by Heyn in which a flexible steel tongue directs the over-flow across the siphon until priming begins. When weight of water presses down the steel tongue against the inner wall, an unobstructed flow is ensured.

7. Provision of priming weir. A priming weir, shown in Fig. 11.19(d) connected with the main reservoir is made. When the overflow commences, the falling nappe completely seals the air in the crown of the siphon and priming is ensured.

Depriming. Depriming is the reverse process of priming. It is the action of the siphon from the time air starts entering the siphon through the vents until the siphonic action is completely stopped. Depriming is carried out by providing *deprimers*. A deprimer is a siphon breaker air vent and is provided to break the siphonic action when the

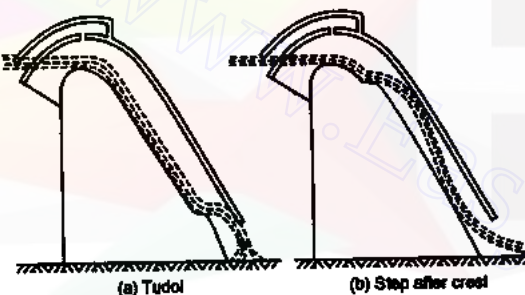


FIG. 11.18

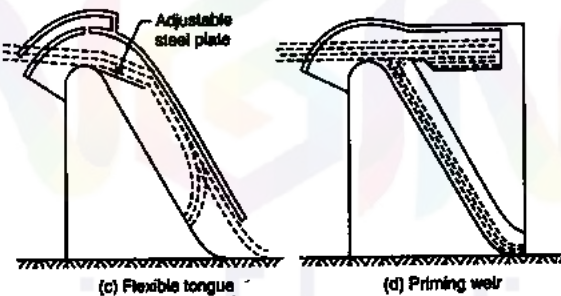


FIG. 11.19

reservoir water surface is drawn below it. If an air vent is not provided, siphonic action once initiated would continue till the reservoir level is brought down to the level at inlet which is provided much below normal reservoir level. The air vent consists of a vent pipe connected to the siphon with its open end in the plane of normal reservoir level. It can be provided either on the side or the top of the crown. The open end should be provided in a horizontal plane to facilitate instantaneous sealing and opening of the pipe. This prevents the development of unsteady flow leading to knocking of the siphon, which is undesirable.

Discharge Through Saddle Siphon

The usual siphon discharge formula is :

$$Q = CA \sqrt{2gH} \quad \dots(11.8)$$

where A = Area of the cross-section at crown = $L \times b$, where L is the length and b is the height of throat.

H = Operating head.

= Reservoir level - Centre of outlet, if outlet is discharging freely.

= Reservoir level - Downstream tail water level if the outlet is submerged.

C = Coefficient of discharge, the average value of which may be taken as 0.65.

The discharge equation can also be expressed in terms of radii of the crest and crown, as shown in Fig. 11.20.

Let R_1 = Radius of crest and R_2 = Radius of crown.

Consider any point P on the vertical section at the throat, and let its radius be r . The L-section of the throat has been shown in Fig. 11.20(b). The section of the throat is rectangular, having length L and height b .

Let V_1 , V_2 , and V be the velocities of flow at crest, crown and point P respectively. The flow through the throat is governed by free vortex flow for which we have :

$$V \times r = \text{Constant.}$$

$$\therefore V_1 R_1 = V_2 R_2 = V \cdot r.$$

Let h_0 be the net operative head at the crest. This head is calculated after making due allowances of all losses occurring in the siphon, that is

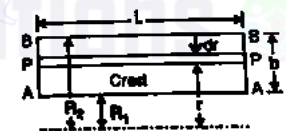
$$h_0 = H - \text{Losses through the siphon.}$$

$$\text{Hence } V_1 = \sqrt{2gh_0}$$

$$\therefore V = V_1 \cdot \frac{R_1}{r} = \frac{R_1}{r} \sqrt{2gh_0}$$



(a) Cross-section



(b) L-section of throat

FIG. 11.20.

Consider a thin rectangular strip of thickness dr at P , and let δq be the discharge through this strip of unit length.

$$\delta q = V \cdot dr \cdot 1 = \frac{R_1}{r} \sqrt{2gh_0} dr.$$

Hence discharge q through the throat per unit length is given by

$$q = \int_{R_1}^{R_2} \delta q = R_1 \sqrt{2gh_0} \int_{R_1}^{R_2} \frac{dr}{r}$$

$$\text{or } q = R_1 \sqrt{2gh_0} \log_e \frac{R_2}{R_1} \quad \dots[11.9(a)]$$

The total discharge through the siphon unit having throat length L is therefore:

$$Q = q \cdot L = R_1 \cdot L \sqrt{2gh_0} \log_e \frac{R_2}{R_1} \quad \dots[11.9(b)]$$

The maximum value of V occurs corresponding to atmospheric pressure head of 10.35 m (34 ft). But to prevent cavitation, the pressure at any point cannot be allowed to approach zero value. Hence safe maximum head is assumed to be 7.92 m (26 ft).

Substituting the values of $h_0 = 7.92$ m, we get

$$Q = R_1 L \sqrt{2g \times 7.92} \log_e \frac{R_2}{R_1} \approx 12.47 R_1 L \log_e \frac{R_2}{R_1}$$

$$\text{or } Q = 28.71 R_1 L \log_{10} \frac{R_2}{R_1} \quad \dots(11.9)$$

Pressure Conditions in the Siphon Spillway

(Refer Fig. 11.20)

Let H = total head at any instant

= difference between the u/s water level and d/s water level if the outlet end is submerged.

or = difference between u/s water level and centre of the outlet if the outlet discharges freely.

h = difference in elevation in the throat and outlet end (i.e. between points A and C).

P_a = atmospheric pressure.

H_l = head loss in the whole siphon.

h_t = head loss between throat and the outlet end.

σ = absolute pressure at which the air dissolved in water starts escaping and cavitation begins (average value of σ may be taken as 2.43 m).

Applying Bernoulli's theorem to the points D and C, we have

$$P_a + H = P_a + \frac{v_c^2}{2g} + H_l \quad \dots(1)$$

$$\text{Hence } \frac{v_c^2}{2g} = H - H_l$$

$$\text{or } v_c = \sqrt{2g(H - H_l)} \quad \dots(2)$$

Hence the discharge through the spillway is

$$Q = A \sqrt{2g(H - H_l)} \quad \dots(3)$$

where A is the area of cross-section of the bore.

Equating this to Eq. 11.7, we get

$$A \sqrt{2g(H - H_l)} = CA \sqrt{2gH}$$

$$\text{or } H - H_l = C^2 H$$

$$\text{Hence } H_l = (1 - C^2) H \quad \dots(11.10 a)$$

Let p_T = pressure head at the throat.

v_T = velocity of flow at the throat.

Applying Bernoulli's theorem between the throat and the exit end, we get

$$p_T + \frac{v_T^2}{2g} + h = p_a + \frac{v_c^2}{2g} + h_t$$

Since the siphon is to run full bore and the area of section is constant between the throat and outlet end, $v_T = v_c$.

$$\text{Hence } p_T + h = p_a + h_t$$

$$\text{or } h = (p_a - p_T) + h_t \quad \dots(4)$$

It should be noted that the pressure (p_T) at the throat cannot be allowed to approach zero value. Since air dissolved in water begins to separate much earlier than the zero value, the value p_T may be assumed to be equal to σ .

$$\text{Hence } h = (p_a - \sigma) + h_t$$

Putting $p_a = 10.35$ m and $\sigma = 2.43$ m, we get

$$h \leq (10.35 - 2.43) + h_t$$

$$h \leq 7.92 + h_t \quad \dots(11.10)$$

Eq. 11.10 limits the maximum difference in elevation between the throat and the outlet end. This limit can be slightly increased by reducing the area of outlet end, in which case v_c will be greater than v_T and Eq. 11.10 will be modified to:

$$h < 7.92 + h_t + \left(\frac{v_c^2}{2g} - \frac{v_T^2}{2g} \right) \quad \dots[11.10 (b)]$$

The quantity $(p_a - p_T)$ or $(p_a - \sigma)$ in Eq. (4) above is known as the *sub-atmospheric pressure head* ($p_{a,T}$) at the crest of the throat. The maximum allowable sub-atmospheric pressure head is equal to probable minimum atmospheric head at the location under consideration minus the minimum tolerable residual pressure head (σ) i.e. $p_{a,T} = p_a - \sigma$.

It should be noted that h is always less than H . The total head loss H_l consists of (i) entry head loss, (ii) velocity head loss, (iii) conversion head loss, (iv) friction head loss, (v) head loss in bends etc. and its maximum value seldom exceeds 20 % of the operating head H .

Again, applying the Bernoulli's theorem between the inlet and the throat, with crest as the datum, we get

$$p_o = p_T + \frac{v_T^2}{2g} + h_{lu}$$

where h_{lu} = head loss from inlet to throat.

$$\text{Hence, we get } \frac{v_T^2}{2g} = (p_o - p_T) - h_{lu} = p_o T - h_{lu} \quad \dots(11.11)$$

Taking $p_T = \sigma = 2.43$ m and $h_o = 10.35$ m, we get

$$\frac{v_T^2}{2g} \leq 7.92 - h_{lu} \quad \dots(11.12)$$

Neglecting h_{lu} , we get $v_T = \sqrt{2g \times 7.92} = 12.46$ m/s. Thus, the maximum possible velocity at the throat is 12.46 m/s.

Classification of Siphon

Siphon spillway may be classified according to head as *high head*, *medium head*, or *low head siphons*.

1. **Medium head siphon.** We have seen in Eq. 11.10 that the difference in level between siphon crest and the exit end is given by

$$h \leq 7.92 + h_L$$

This is true only when the area of the siphon at the throat and at the exit end are equal. Hence a medium head siphon spillway is the one in which the outlet area is the same as the area at the throat.

2. **High head siphon.** In Eq. [11.10 (b)] we have seen that the limit of the value of h can be increased if the velocity at the exit end is greater than the velocity at the throat, the increase being equal to $\left[\frac{v_c^2}{2g} - \frac{v_T^2}{2g} \right]$. This can be achieved by reducing the outlet area by a nozzle or by tapering the lower part of the siphon, or in some other manner increasing the resistance to flow. Such convergent siphons are known as *high head siphons*.

3. **Low head siphon.** A low head siphon is the one in which the outlet tube is made divergent. This gives velocity u , lesser than v_T thus further decreasing the value of h given by Eq. 11.10.

Qualities of Good Siphon

A good siphon should have the following qualities :

1. It should have quick priming.
2. It should have low depth of priming so that the siphon becomes effective even with a small rise in the water level.
3. There should be no knocking at the times of priming and depriming. The flow through the siphon should be steady and continuous.
4. It should have automatic working.
5. It should be self-cleaning so that no silt or debris deposits in the siphon.
6. Air should neither get access to the siphon nor be entrained.

The dissolved air in water should not be separated. The flow should be smooth and free of vibration.

7. It should have high efficiency.
8. It should have low initial and maintenance cost. It should have easy access to all parts for repairs.
9. It should be simple in design and construction.

VOLUTE SIPHON SPILLWAY

Volute siphon spillway is a special type of siphon spillway designed in India by Ganesh Iyer. It consists of a vertical barrel or shaft bent at the discharge end and opened out in the form of a funnel at the top. The top or lip of the funnel is kept at the reservoir level and a number of volutes (like blades of a centrifugal pump or turbine) are fixed in the funnel to induce a spiral motion to the water passing along them. A dome, supported on number of pillars, is placed over the funnel. Over the main dome is attached a deprimer dome. The entrance end of the deprimer dome is kept just at reservoir level.

When water level in the reservoir rises, it seals the air entry and sheet of water enters to the funnel from all sides. The spiral flow of water through the volute causes suction which sucks all the remaining air and priming is thus achieved very quickly. The siphon then runs full. When the reservoir level falls, air enters through the deprimer dome, thus breaking the siphonic action.

The discharge through the volute siphon is given by

$$Q = A \sqrt{2g(H - H_L)} = CA \cdot \sqrt{2gH} \quad \dots(11.13)$$

where A = area of cross-section of pipe.
 C = coefficient of discharge.
 H = maximum operating head.
 H_L = head loss through the siphon.

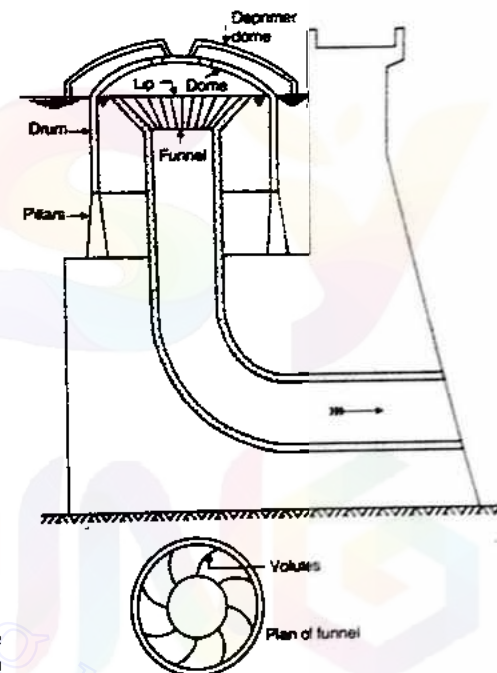


FIG. 11.21. VOLUTE SIPHON SPILLWAY

11.9. DYNAMIC FORCE ON SPILLWAY

When water flows over the curved surface of ogee spillway there is continuous change of velocity, and hence there is change in momentum from section to section. According to Newton's second law of motion, this change in momentum causes a force on the spillways structure. This force is known as the *dynamic force*.

Consider an element of water between two sections A and B on a curved surface [Fig. 11.22(b)]. The resultant of the forces on the element of water is given by

$$\Sigma F = \rho Q \Delta V = \rho Q (V_2 - V_1) \quad \dots(11.14)$$

where ρ = mass density of water = w/g

Q = discharge.

Rewriting Eq. 11.14 in horizontal and vertical directions separately, we get

$$\Sigma F_H = \rho Q (V_{1H} - V_{2H}) \quad \dots[11.14(a)]$$

and

$$\Sigma F_V = \rho Q (V_{1V} - V_{2V}) \quad \dots[11.14(b)]$$

The force F_H and F_V are those acting on a significant free body of fluid and include gravity forces, hydrostatic pressure and the reaction of any object in contact with water. Note that $\rho = w/g = 9.81/9.81 = 1$ in SI units.

See example 11.1 for illustration.

Example 11.1. Fig. 11.13(a) shows an ogee spillway which discharges water with a head of 1.2 m over the crest. Taking the coefficient of discharge as 2.2, compute the dynamic force on the curved section AB which has a constant radius of 3 m.

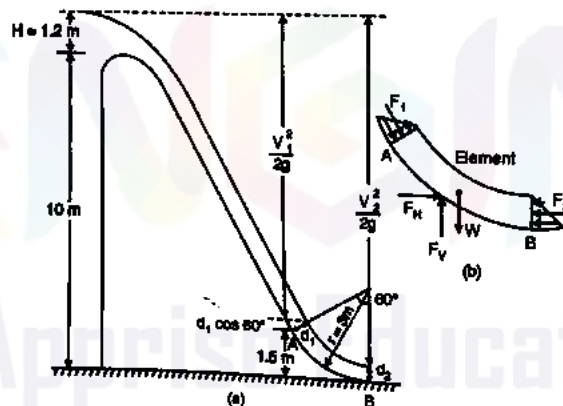


FIG. 11.22.

Solution. The discharge over the spillway is given by

$$Q = CLH^{3/2}$$

or

$$q = \frac{Q}{L} = CH^{3/2} = 2.2 (1.2)^{3/2} = 2.9 \text{ cumecs/m.}$$

Let d_1 and d_2 be the depth of sheet of water at A and B, and V_1 and V_2 be the velocities. Assuming that there is no loss of energy and neglecting approach velocity, we may apply Bernoulli's theorem at w/s water surface, and at sections A and B.

$$\text{Thus we get} \quad 11.2 = 1.5 + d_1 \cos 60^\circ + \frac{V_1^2}{2g} = d_2 + \frac{V_2^2}{2g} \quad \dots(1)$$

$$\text{But} \quad V_1 d_1 = q = 2.9 = V_2 d_2$$

$$\text{Hence} \quad V_1 = \frac{2.9}{d_1} \text{ and } V_2 = \frac{2.9}{d_2}$$

Substituting these in (1) and solving these by trial and error, we get

$$d_1 = 0.212 \text{ m and } d_2 = 0.197 \text{ m}$$

Hence

$$V_1 = 13.7 \text{ m/sec and } V_2 = 14.7 \text{ m/sec.}$$

Fig. 11.22(b) shows the free body diagram of the curved element between sections A and B in which F_x and F_y represent components of force on the water by the curved section AB. If F_1 and F_2 are the resultant hydrostatic forces at section A and B, we have

$$F_1 = \frac{1}{2} w d_1^2 \cos 60^\circ = \frac{1}{2} \times 9.81 (0.212)^2 \times \frac{1}{2} = 0.11 \text{ kN.}$$

$$F_2 = \frac{1}{2} w d_2^2 = \frac{1}{2} \times 9.81 (0.197)^2 = 0.19 \text{ kN.}$$

The weight W of the water body in the curved portion between sections A and B is

$$W = 9.81 \times \frac{60^\circ}{360^\circ} \times 2\pi \left(3 \times \frac{0.212 + 0.197}{2} \right) = 6.3 \text{ kN.}$$

Applying Eq. 11.14(a) for 1 m length of spillway, we get

$$\Sigma F_H = F_1 \cos 60^\circ - F_2 + F_x = \rho Q (V_2 - V_1 \cos 60^\circ)$$

$$\text{or} \quad 0.11 \times \frac{1}{2} - 0.19 + F_x = (1 \times 2.9 \times 1) (14.7 - 13.7 \times \frac{1}{2})$$

$$\text{From which} \quad F_x = 22.9 \text{ kN/m.}$$

Similarly, from Eq. 11.14(b), we have

$$\Sigma F_V = F_y - F_1 \sin 60^\circ - W = \rho Q [0 + 13.7 \sin 60^\circ].$$

$$\text{or} \quad F_y - 0.11 \times 0.866 - 6.3 = (1 \times 2.9 \times 1) (13.7 \times 0.866)$$

$$\text{From which} \quad F_y = 40.80 \text{ kN/m}$$

$$\text{Resultant force} \quad F = \sqrt{F_x^2 + F_y^2} = \sqrt{(22.9)^2 + (40.8)^2} = 46.79 \text{ kN/m}$$

Example 11.3. Compute the discharge over an ogee weir with coefficient of discharge equal to 2.4 at a head of 2 m. The length of the spillway is 100 m. The weir crest is 8 m above the bottom of the approach channel having the same width as that of the spillway.

Solution. Neglecting the approach velocity and end contractions,

$$Q = CLH^{3/2} = 2.4 \times 100(2)^{3/2} = 678.82 \text{ cumecs}$$

The approach velocity is given by

$$v_a = \frac{Q}{\text{Height} \times \text{Width of channel}} = \frac{Q}{(8+2) \times 100} = \frac{678.82}{10 \times 100} = 0.679 \text{ m/sec}$$

$$h_a = \frac{v_a^2}{2g} = \frac{(0.679)^2}{2 \times 9.81} = 0.023 \text{ m}$$

and

$$H_r = H + h_a = 2 + 0.023 = 2.023 \text{ m}$$

$$\text{Hence } Q = 2.4 \times 100 (2.023)^{3/2} = 890.6 \text{ cumecs.}$$

Example 11.3. A dam has siphon spillway whose cross-section is 1 m high and 4 m wide. The tail water elevation at design flow is 6 m below the summit of the siphon and the head water elevation is 1.5 m above the summit. Assuming a coefficient of discharge of 0.6, what is the capacity of the siphon? What head would be required on an ogee spillway ($C = 2.25$) 4 m long to discharge this flow? What length of ogee weir would be required to discharge the same flow as the siphon with a head of 1.5 m on the weir crest?

Solution. (a) For the siphon spillway

Net operative head $= 6 + 1.5 = 7.5 \text{ m}$

$$Q = CA \sqrt{2gH} = 0.6 \times 1 \times 4 \sqrt{2 \times 9.81 \times 7.5} = 29.1 \text{ cumecs.}$$

(b) For the ogee spillway 4 m long.

$$Q = CLK^{3/2}$$

$$h = \left(\frac{29.1}{2.25 \times 4} \right)^{2/3} = 2.19 \text{ m}$$

(c) For the ogee spillway with $h = 1.5 \text{ m}$.

$$L = \frac{Q}{CK^{3/2}} = \frac{29.1}{2.25 \times (1.5)^{3/2}} = 7.04 \text{ m.}$$

It is thus seen that for the same discharge, a siphon spillway works with lesser head over the crest (thereby reducing the afflux), or with lesser length than an ogee spillway.

Example 11.4. A saddle siphon has the following data :

Full reservoir level $= 435.00 \text{ m.}$

Level of centre of siphon outlet $= 429.60 \text{ m.}$

High flood level $= 435.85 \text{ m.}$

High flood discharge $= 600 \text{ cumecs.}$

If the dimensions of the throat of the siphon are : width = 4 m and height = 2 m, determine the number of siphon units required to pass the flood safely. The siphon discharges freely in air.

Solution. Maximum operative head $H = \text{H.F.L.} - \text{R.L. of outlet centre}$

$$= 435.85 - 429.60 = 6.25 \text{ m.}$$

The discharge is given by $Q = CA \sqrt{2gH}$

Assume $C = 0.65$, Also, $A = 4 \times 2 = 8 \text{ m}^2$

$$Q = 0.65 \times 8 \sqrt{2 \times 9.81 \times 6.25} = 57.58 \text{ cumecs.}$$

$$\therefore \text{No. of units required} = \frac{600}{57.58} = 10.42. \text{ Hence provide 11 units.}$$

11.5. Example 11.5. Design an ogee spillway for concrete gravity dam, for the following data :

1. Average river bed level $= 250.00 \text{ m}$
2. R.L. of spillway crest $= 350.00 \text{ m}$
3. Slope of d/s face of gravity dam $= 0.75 : 1$
4. Design discharge $= 6500 \text{ cumecs}$
5. Length of spillway $= 5 \text{ spans with a clear length of } 9 \text{ m each}$
6. Thickness of each pier $= 2 \text{ m}$

Solution

Step 1 : Computation of design head

$$Q = C_d \cdot L_e H_r^{3/2} \text{ where } L_e = L - 2[NK_p + K_a] H_r$$

To start with, let us assume $C_d = 2.2$, applicable for high over flow spillway

Also, let $L_e \triangleq L = \text{clear water way} = 5 \times 9 = 45 \text{ m}$

$$6500 = 2.2 \times 45 H_r^{3/2} \text{ from which } H_r = 16.27 \text{ m}$$

The height (P) of spillway crest above river bed $= 350 - 250 = 100 \text{ m}$

Since $\frac{P}{H_d} \triangleq \frac{P}{H_r} = \frac{100}{16.27} = 6.15$ is greater than 1.33, it is a high overflow spillway,

for which the effect of velocity of approach is negligible.

Also, with reference to Fig. 11.11 (b), we have

$$\frac{h_d + d}{H_{dr}} = \frac{H_{dr} + P}{H_{dr}} = \frac{16.27 + 100}{16.27} = 7.15 > 1.7$$

Hence the discharge coefficient is not affected by down stream apron interference as well as on the tail water conditions.

The above value of H_r has been computed by taking $L_e \triangleq L$. Actual value of L_e is given by $L_e = L - 2[NK_p + K_a] H_r$

Let us assume rounded nose piers and rounded abutments.

Thus we have $K_p = 0.01$ and $K_a = 0.1$. Also $N = 4$. Assuming $H_r \triangleq 17.5 \text{ m}$ (i.e. slightly more than the one computed above), we get

$$L_e = 45 - 2[4 \times 0.01 + 0.1] \times 17.5 = 40.1 \text{ m}$$

Substituting this value of L_e in discharge equation, we get

$$6500 = 2.2 \times 40.1 \times H_r^{3/2} \text{ from which } H_r = 17.58 \text{ m}$$

which is quite near to the assumed value of $H_r = 17.5 \text{ m}$.

Hence the crest profile will be designed for $H_d = 17.58 \text{ m}$, thus neglecting the head due to velocity of approach.

However, velocity of approach, $V_o = \frac{Q}{\text{Area}} = \frac{6500}{(45 + 4 \times 2)(100 + 17.58)} = 1.043 \text{ m/s}$

$$\therefore \text{Velocity head, } H_o = \frac{V_o^2}{2g} = \frac{(1.043)^2}{2 \times 9.81} \approx 0.055 \text{ m}$$

Hence the velocity head is quite small (being only 0.3% of the total head) and it is quite justified in neglecting this.

Step 2. Determination of d/s profile

Let us keep u/s face vertical. The d/s profile suggested by WES, for vertical face is given by Eq. 11.1 :

$$x^{1.85} = 2 H_d^{2.85} \cdot y$$

$$\text{Hence } y = \frac{x^{1.85}}{2 H_d^{2.85}} = \frac{x^{1.85}}{2 (17.58)^{2.85}} = 0.04372 x^{1.85} \quad \dots(1)$$

Let us now determine the tangent point (T.P.), for a downstream slope of 0.75 (H) : 1 (V)

$$\therefore \frac{dy}{dx} = \frac{1}{0.75} \quad \dots(2)$$

However, differentiating Eq. (1), w.r. to x , we get

$$\frac{dy}{dx} = 1.85 \times 0.04372 x^{1.85-1} = 0.0809 x^{0.85} \quad \dots(3)$$

$$\text{Equating (2) and (3), } 0.0809 x^{0.85} = \frac{1}{0.75}$$

$$\text{or } x^{0.85} = 16.484, \text{ from which } x = 27.03 \text{ m}$$

$$y = 19.48 \text{ m}$$

Hence from (1), the coordinates of the tangent points are (27.03 m, 19.48 m).

The co-ordinates of the curve for $x=0$ to $x=27.03$ m are given in the table below.

x (m)	$y = 0.04372 x^{1.85}$	x (m)	y (m)	x (m)	y (m)
1	0.044	7	1.600	18	9.182
2	0.158	8	2.048	20	11.158
3	0.334	10	3.088	22	13.310
4	0.588	12	4.337	24	15.684
5	0.869	14	5.768	26	18.139
6	1.203	16	7.384	27.03	19.480

Step 3. Determination of u/s profile

For vertical u/s face, the u/s profile is given by Eq. 11.2. The u/s curve extends upto $x = -0.27 H_d$. Thus

$$y = \frac{0.724 (x + 0.27 H_d)^{1.85}}{H_d^{2.85}} + 0.126 H_d - 0.4315 H_d^{0.375} (x + 0.27 H_d)^{0.825}$$

Taking

$$H_d = 17.58 \text{ m, we get}$$

$$y = \frac{0.724 (x + 0.27 \times 17.58)^{1.85}}{(17.58)^{2.85}} + 0.126 \times 17.58 - 0.4315 (17.58)^{0.375} (x + 0.27 \times 17.58)^{0.825}$$

$$\text{or } y = 0.0633 (x + 4.7486)^{1.85} + 2.2151 - 1.2643 (x + 4.7486)^{0.825} \quad \dots(4)$$

The curve will extend upto $x = -0.27 \times 17.58 \approx -4.75$ m

Hence the value of y coordinates are calculated for the values of $x = -0.5$ m, -1.0 m, -1.5 m, -2 m, -3 m, -4 m and -4.75 m and tabulated as under.

x (m)	y (m)
-0.5	0.012
-1.0	0.087
-1.5	0.135
-2.0	0.248
-3.0	0.601
-4.0	1.198
-4.75	2.215

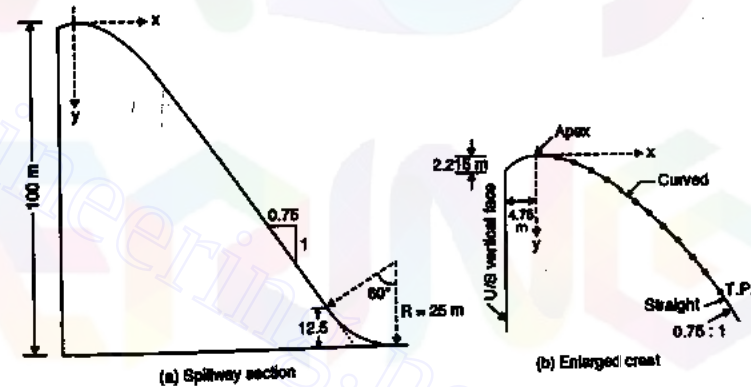


FIG. 11.23

4. Step 4: Design of d/s bucket

The profile of the spillway is shown in Fig. 11.23 (a). A reverse curve at the toe is provided to form a bucket. The radius of the bucket is generally kept at $R = P/4 = 100/4 = 25$ m. The bucket will subtend an angle of 60° at the centre, as shown.

11.10. ENERGY DISSIPATION BELOW SPILLWAYS

When flood discharge passes over the spillway crest, it has high potential energy which gets converted into kinetic energy as it glides along it. This high energy has to be dissipated otherwise it would cause erosion at the downstream toe. There is

a varied practice regarding the extent of protection against erosion below a spillway to be provided. In general, the protection can be rendered by two ways :

- (i) By dissipating the energy by means of hydraulic jump.
- (ii) By directing the jet of water so as to fall away from the structure by a deflector bucket or lip, and dissipating the energy by impact.

Where the energy of flow is to be dissipated before the discharge is returned to the main river channel, the hydraulic jump basin is an effective device for reducing the exit velocity to a tranquil state. However, where the spillway discharge may be safely delivered directly to the river channel, without providing a dissipating or stilling device, the jet is often projected beyond the structure by a *deflector bucket*.

Hydraulic jump computations. The hydraulic jump that occurs at the stilling basin has some distinctive characteristics and assumes a definite form, depending upon the energy of flow which must be dissipated in relation to the depth of flow. The jump form and flow characteristics depends upon the following factors :

- (i) Discharge entering the basin,
- (ii) Critical depth of flow, d_c
- (iii) Froude number parameter $\frac{V}{\sqrt{gd}}$

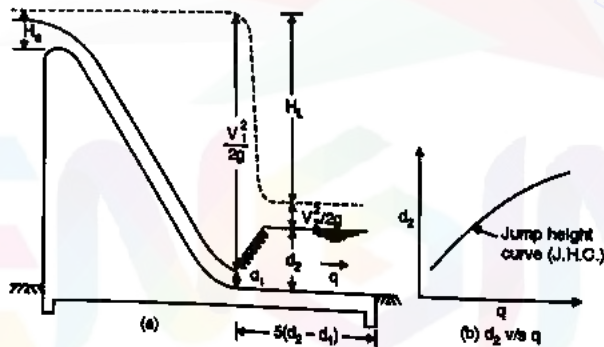


FIG. 11.24.

Fig. 11.24 shows the elements of hydraulic jump. The post jump depth d_2 can be computed for a given discharge as under :

1. For the given discharge q per metre length of the spillway, calculate the head H_t over the crest to the total energy level :

$$H_t = \left(\frac{q}{C} \right)^{2/3}$$

2. Find total energy level at w/s :

$$\text{w/s T.E.L.} = \text{crest level} + H_t$$

3. Assuming no losses, the specific energy E_1 at the toe of the spillway will be equal to the T.E.L. w/s.

$$E_1 = \text{T.E.L. w/s.}$$

4. Knowing E_1 and q , find the prejump depth d_1 by trial and error from the relation:

$$E_1 = d_1 + \frac{V_1^2}{2g} = d_1 + \frac{q^2}{2gd_1^3}$$

5. Calculate Froude Number F_1 :

$$F_1 = \frac{V_1}{\sqrt{gd_1}} = \frac{q}{\sqrt{gd_1^3}} \quad \dots(11.15)$$

6. Compute the post jump depth d_2 from the relations :

$$d_2 = \frac{d_1}{2} \left[\sqrt{1 + 8F_1^2} - 1 \right] \quad \dots(11.16)$$

or by

$$d_2 = \frac{d_1}{2} \left[\sqrt{1 + \frac{8q^2}{gd_1^3}} - 1 \right] \quad \dots(11.17)$$

Plotting jump height curve. Eq. 11.16 or 11.17 can be used for calculating post jump depths d_2 for various values of flow discharge q . Fig. 11.24(b) shows such a curve, known as *jump height curve (J.H.C.)* showing the variation of post jump depth d_2 with the discharge q .

Plotting tails water curve (T.W.C.). The efficiency of the hydraulic jump in dissipating the energy, and the corresponding protection works in the stilling basin will depend upon the position or height of tail water in relation to the post jump depth d_2 for that discharge. Tail water curve [Fig. 11.25(a)] gives the relation between the discharge q and the tail water depth D . This can be prepared by actual gauge readings and measurements may be conveniently made at some control point available downstream. However, the effect of channel retrogression must be taken into account.

Relative position of jump height curve and tail water curve. There may be five conditions that govern the relationship between the jump height curve (J.H.C.) and tail water curve (T.W.C.) :

1. Both the curves coincide [Fig. 11.25(b)].
2. J.H.C. lies lower than T.W.C. at all discharges [Fig. 11.25(c)].
3. J.H.C. lies above T.W.C. at all discharges [Fig. 11.25(d)].
4. J.H.C. lies lower than T.W.C. at small discharges and higher than T.W.C. at large discharges [Fig. 11.25(e)].
5. J.H.C. lies above T.W.C. at small discharges and lower at higher discharges [Fig. 11.25(f)].

1. **Protection works for condition I.** In this condition, both the curves coincide, as shown in Fig. 11.25(b). This is an ideal condition, since the post jump depth required for the formation of hydraulic jump is just available at the channel. The jump formation

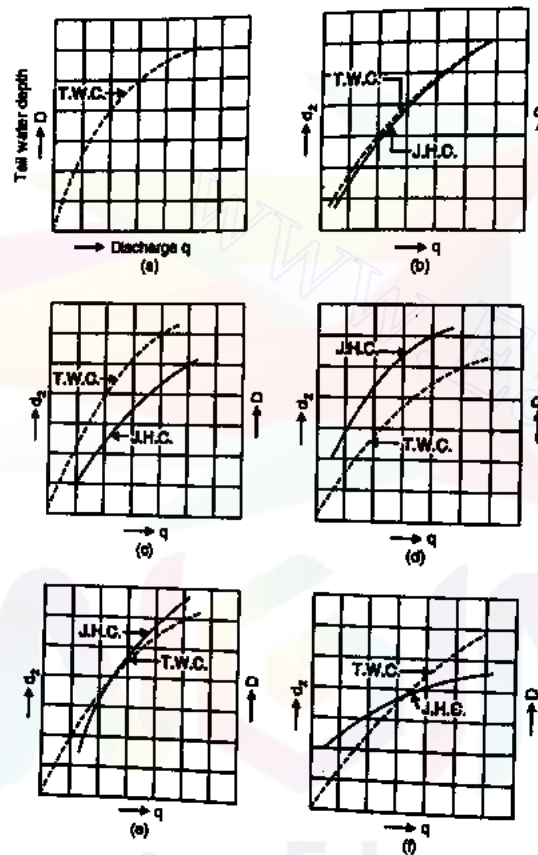


FIG. 11.25

will thus be perfect at all discharges and hydraulic jump will be formed at the toe of the spillway.

Protective measures. In such a case a simple horizontal apron of a length $5(d_2 - d_1)$ will be sufficient, as shown in Fig. 11.24(a).

2. Protection works for condition II. In this condition, shown in Fig. 11.25(c), the jump height curve lies lower than the tail water curve at all discharges. In other

words, the available tail water depth is greater than required for the formation of hydraulic jump. The jump in such a case will be completely submerged and no visible standing wave would occur. Thus, very little energy will be dissipated unless arrangements are made to reduce the tail water depth at the point of formation of hydraulic jump.

Protective measures.

The following protection measures may be provided :

(i) Using a sloping apron above the bed level as shown in Fig. 11.26(a). The jump will be formed on the sloping glacis where depth d_2 is available. This was done at Madden dam on Panama canal.

(ii) Using a low level bucket sharply upturned at the end, as shown in Fig. 11.26(b). This serves the purpose of a deflector.

(iii) Providing end sills with baffles [Fig. 11.26(c)] so that energy is dissipated by impact and friction.

3. Protection works for condition III. In this condition the jump height curve is higher than the tail water curve at all discharges, as shown in Fig. 11.25(d). Thus the available tail water depth is lesser than the depth required for the formation of hydraulic jump.

Protective measures. The protection works depend upon the nature of foundations. The following measures are provided.

- Providing a cistern depressed below bed with sloping glacis upstream [Fig. 11.27(a)].
- Providing a cistern excavated in the rock to a depth required for the formation of jump. Baffles may also be provided [Fig. 11.27(b)].
- Providing a low secondary weir and baffles [Fig. 11.27(c)].

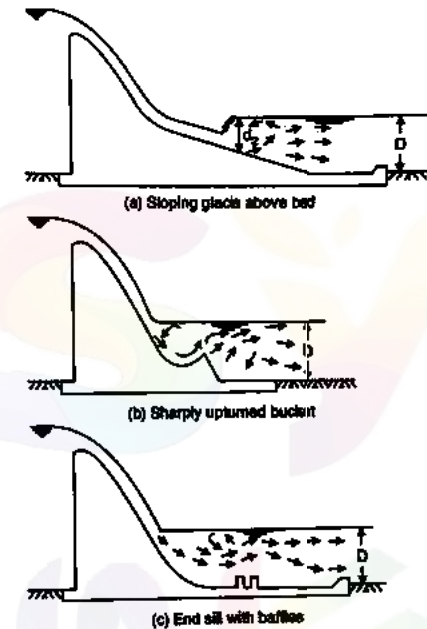


FIG. 11.26. PROTECTION WORKS FOR HIGH TAIL WATER.

- (iv) Providing a sharply upturned bucket or *ski jump bucket* to deflect the jet of water away from the main structure [Fig. 11.27(d)], if sound rock is available at ground surface.

4. Protection works for condition IV. In this condition the jump height curve lies lower than the tail water curve at low discharges, and higher at high discharges, as shown in Fig. 11.25(e). Thus, at the low discharges, the jump will be drowned, while at high discharges, *insufficient* tail water depth will be there.

Protection measures. In such a case, a sloping apron partly above and partly below the bed is provided, with a sufficient length of horizontal apron and end sill. The jump will form higher up on the apron at low discharges and lower down at higher discharges, as shown in Fig. 11.28.

Protection works for condition V. This is just the reverse of condition IV. As illustrated in Fig. 11.25(f), the jump height curve lies above the T.W.C. at low discharge, and lower at high discharges. Thus at low discharges insufficient tail water will be there while at higher discharges the jumps will be drowned. The *protection measure* similar to that shown in Fig. 11.28 is provided. At low discharges, the jump will be formed lower down, while at high discharges, the jump will be formed higher up.

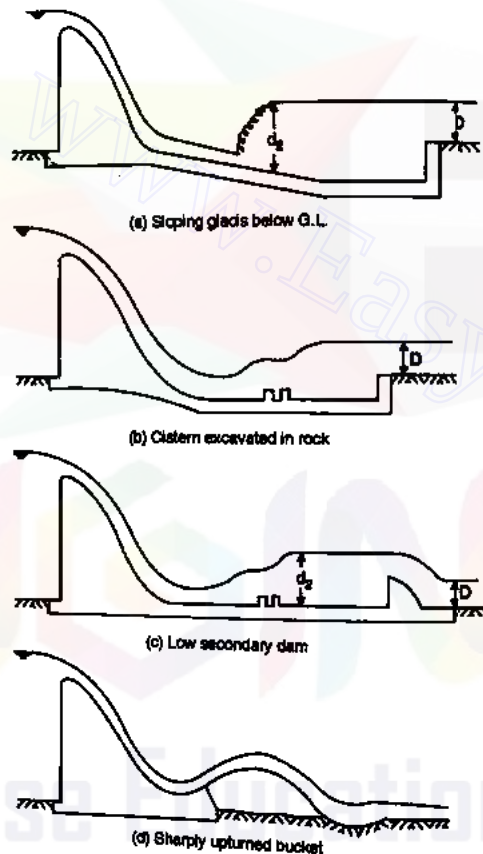


FIG. 11.27. PROTECTION WORKS FOR LOW TAIL WATER DEPTH.

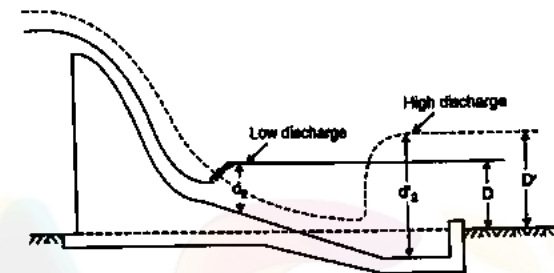


FIG. 11.28. SLOPING APRON PARTLY BELOW AND PARTLY ABOVE THE BED.

11.11. INDIAN STANDARD ON CRITERIA FOR DESIGN OF HYDRAULIC JUMP TYPE STILLING BASINS WITH HORIZONTAL AND SLOPING APRONS

1. Introduction. The Indian Standard IS : 4997 - 1968 lays down the criteria for the design of hydraulic jump type stilling basins of rectangular cross section with horizontal and sloping apron utilising various energy dissipators such as chute blocks, basin or floor blocks and end sill. A *stilling basin* is a structure in which all or part of the energy dissipating action is confined. In a stilling basin the kinetic energy first causes turbulence and is ultimately lost as heat energy. A *hydraulic jump type stilling basin* is basin in which dissipation of energy is accomplished basically by hydraulic jump which may be stabilised using chute blocks, basin blocks, end sill, etc. A hydraulic jump type stilling basin may be either with *horizontal apron* (Fig. 11.29 and 11.31) or with *sloping apron* (Fig. 11.37).

2. Hydraulic Jump Type Stilling Basin with Horizontal Apron.

When the tail water rating curve approximately follows the hydraulic jump curve, or is only slightly above or below it, then hydraulic jump type stilling basin with horizontal apron provides the best solution for energy dissipation. In this case the requisite depth may be obtained on a proper apron near or at the ground level so that it is quite economical. For spillways on weak bed conditions, and weirs, and barrages on sand and loose gravel, hydraulic jump type stilling basins are recommended.

Conjugate depths : Water depths at the beginning and the end of the hydraulic jump are related by the following formula, for horizontal aprons :

$$\frac{D_2}{D_1} = \frac{1}{2} \left[\left(\sqrt{1 + 8F_1^2} - 1 \right) \right] \quad \dots(11.18)$$

where F_1 is the Froude number of the flow at the beginning of the jump. In general, Froude number is given by the following formula :

$$F = \frac{V}{\sqrt{gD}} \quad \dots(11.19)$$

Classification : Hydraulic jump type stilling basin with horizontal apron may be classified into the following two categories :

- (a) **Basin type I** : Stilling basin in which the Froude number of the incoming flow is less than 4.5. This case is generally encountered on weirs and barrages.
- (b) **Stilling basin type II** : Stilling basin in which Froude number of the incoming flow is greater than 4.5. This case is general feature for dams.

Stilling basin type I Fig. 11.29(b) shows the stilling basin type I. The factors involved in the design of stilling basin are : (i) determination of the elevation of basin floor, (ii) basin length, and (iii) basin appurtenances.

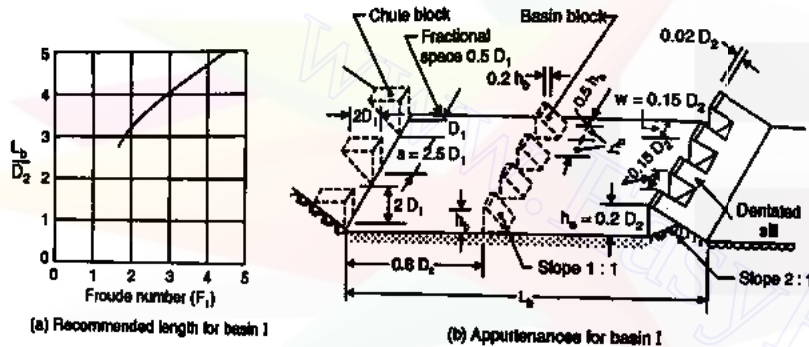


FIG. 11.29. STILLING BASIN TYPE I.

(i) **Elevation of basin floor :**

Let H_L be the head loss in hydraulic jump and q be the discharge intensity.

To calculate H_L from the known upstream and downstream total energy lines, the following procedure may be adopted :

(a) Where the basin is directly downstream from the crest or where the chute is not longer than the hydraulic head, H_L may be assumed equal to the difference in the upstream and downstream total energy levels.

(b) Where the chute length exceeds the hydraulic head, D_1 should be first determined by taking H_L equal to the difference between the upstream and downstream total energy levels. Next calculate friction losses on the chute by Manning's formula and determine the more accurate value of H_L and consequently that of D_1 . The process may be repeated till the desired accuracy is achieved.

Knowing H_L and q , we can calculate D_c , D_1 and D_2 either from the following formulae or from Fig. 11.30.

$$H_L = \frac{(D_2 - D_1)^2}{4 D_1 D_2} \quad \dots(11.20)$$

$$D_2 = \frac{D_1}{2} + \sqrt{\frac{2q^2}{D_1 \cdot q} + \frac{D_1^2}{4}} \quad ; \quad D_c = \left(\frac{q^2}{g} \right)^{1/3}$$

Having obtained D_1 and D_2 the elevation of the basin floor may be calculated by either deducting the specific energy at section 1-1 (prejump) from the energy line at the section or that at section 2-2 (end of jump) from the downstream total energy line.

(ii) **Basin length and depth**

The length of the basin (L_b) will be determined from the curve given in Fig. 11.29(a). The basin will be provided with an end sill, preferably denanted one. In the boulder reach the sloping face of the end sill will be kept on the upstream side. Generally, the basin floor should not be raised above the level required from sequent depth consideration. If the rising of the floor becomes obligatory due to site conditions, the same should not exceed 15 percent of D_2 and the basin in that case should be further supplemented by chute blocks and basin blocks. The basin blocks should not be used if the velocity of flow at the location of basin blocks exceeds 15 m/s and in that case the floor or the basin should be kept at a depth equal to D_2 below the tail-water level. The tail water depth should not generally exceed 10 percent of D_2 .

(iii) **Basin Appurtenances**

The requirements of basin appurtenances, such as chute blocks, basin blocks, end sill are shown in Fig. 11.29(b).

Chute blocks : The chute blocks should be kept at a height equal to $2D_1$ on the glacis slope. Their top length should also be equal to $2D_1$. The width of the chute blocks should be kept equal to D_1 and their spacing as $2.5D_1$. A space equal to $\frac{D_1}{2}$ should be left along each wall.

Basin blocks : The height of the basin blocks in terms of D_1 may be obtained from Fig. 11.31(c). The width and spacing of the basin block should be equal to their height. The upstream face of all the basin blocks should be vertical and in one plane. A half space is recommended adjacent to the wall. The upstream face of the basin blocks should be set at a distance of $0.8D_2$ from the down stream face of the chute blocks.

End sill The height of the dentated end sill is recommended as $0.2D_2$. The maximum width and spacing of dents shall be according to Fig. 11.29(b). A dent is

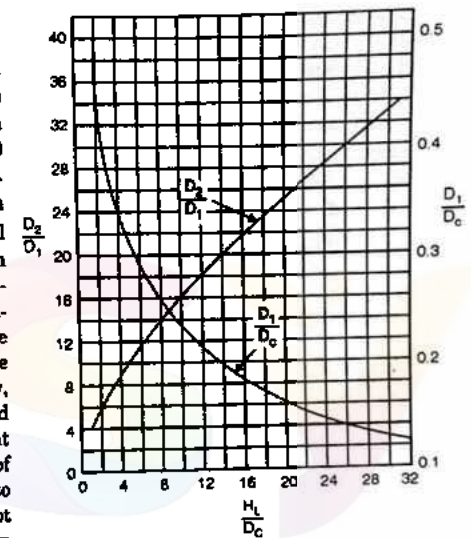


FIG. 11.30. CURVE FOR DETERMINATION OF SEQUENT DEPTH (HORIZONTAL APRON).

recommended adjacent to each side wall. In case of narrow basin, it is advisable to reduce the width and spacing but in the same proportion. It is not necessary to stagger the end sill dents with reference to chute blocks.

Stilling Basin Type II. Fig. 11.31(b) shows the stilling basin type II.

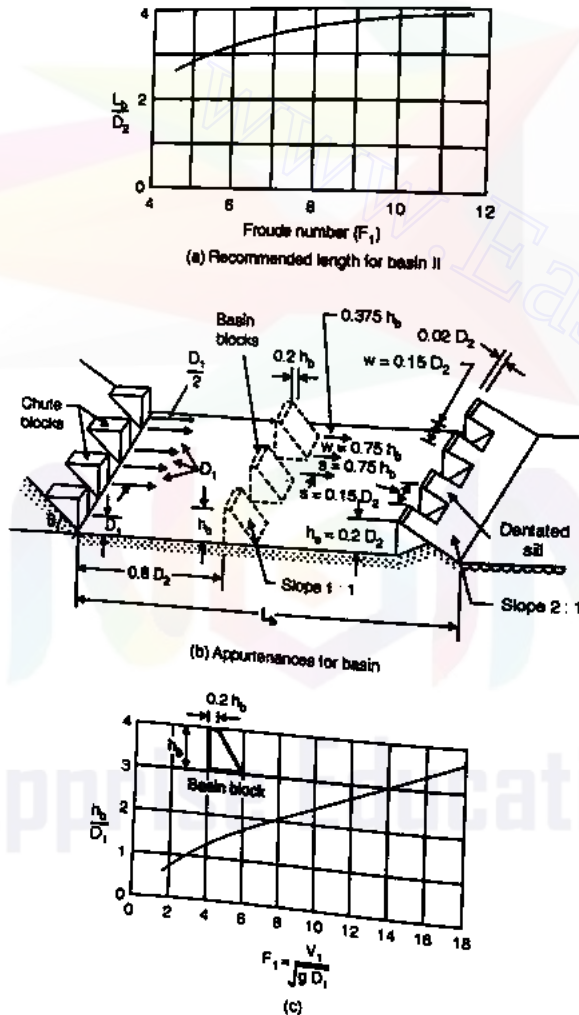


FIG. 11.31. STILLING BASIN TYPE II.

(i) **Elevation of basin floor.** Same procedure is followed as for basin I.

(ii) **Basin length and depth.** The length of the basin is determined from Fig. 11.31(a). The basin should be provided with chute blocks and end sill. The maximum raising of the basin floor shall not exceed 15 percent of D_2 and the basin in that case will be further supplemented by basin blocks. However, when the flow velocity at the location of basin blocks exceeds 15 m/s, no basin blocks are recommended and in that case the floor of the basin should be kept at depth equal to D_2 below the tail-water level. The tail water depth should not generally exceed 10 percent of D_2 .

(iii) **Basin appurtenances.** Requirements of basin appurtenances, such as chute blocks, basin blocks and end sill are shown in Fig. 11.31(b).

Chute blocks. The height, width and spacing of the chute blocks should be kept equal to D_1 . The width and spacing may be varied to eliminate frictional blocks. A space equal to $D_1/2$ is preferable along each wall.

Basin blocks. The height of basin blocks in terms of D_1 can be obtained from Fig. 11.31(c). The width and spacing should be kept three-fourth of the height. These should be placed at distance of $0.8 D_2$ downstream from the chute blocks.

End sill. Same requirements as that for basin I.

3. Hydraulic Jump Type Stilling Basin with Sloping Apron

General. When the tail water is too deep as compared to the sequent depth D_2 , the jet left at the natural ground level would continue to go as a strong current

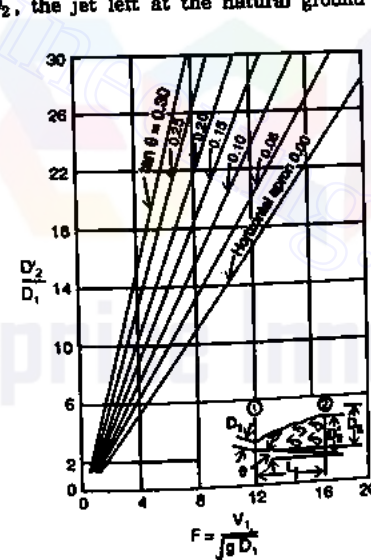


FIG. 11.32 DETERMINATION OF D_2 FOR BASIN III.

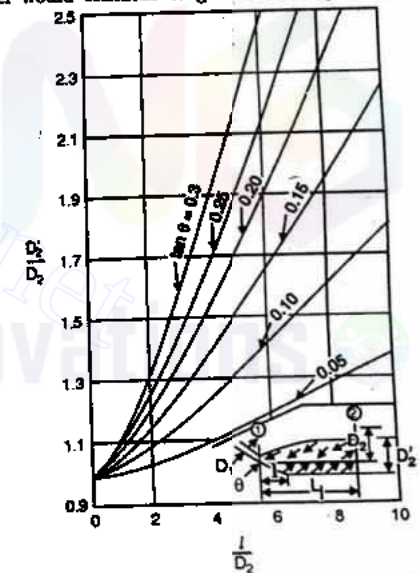


FIG. 11.33. DETERMINATION OF D_2 FOR BASIN IV.

near the bed forming a drowned jump which is harmful to the river bed. In such a case, a hydraulic jump type stilling basin with sloping apron should be preferred as it would allow an efficient jump to be formed at suitable level on the sloping apron.

Conjugate depths. Water depths at the beginning and the end of the hydraulic jump are related by the following formula for a fully sloping apron:

$$\frac{D_2}{D_1} = \frac{1}{2 \cos \theta} \left[\left(\frac{8 F_1^2 \cos^3 \theta}{1 - 2 K \tan \theta} + 1 \right)^{1/2} - 1 \right] \quad \dots(11.21)$$

where D_1 = conjugate depth sequent to D_1 for sloping apron, and may be determined from Fig. 11.32 (for basin III) or Fig. 11.33 (for basin IV).

K = Shape factor. It is defined as a dimensionless parameter which varies with Froude number and the slope of the apron. This has been plotted against slope in Fig. 11.34 on the assumption that it is independent of Froude number.

Classification. The hydraulic jump on a sloping apron may occur in four different forms depending on the tail water conditions, as shown in Fig. 11.35. The action in cases (c) and (d) is same if it is assumed that horizontal floor begins at the end of jump of case (d). Case (b) is virtually case (a) operating with excessive tail-water

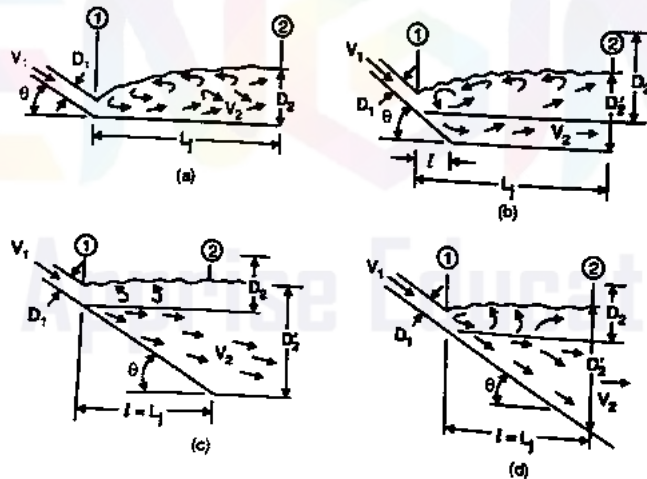


FIG. 11.35 DIFFERENT FORMS OF HYDRAULIC JUMPS ON SLOPING APRON.

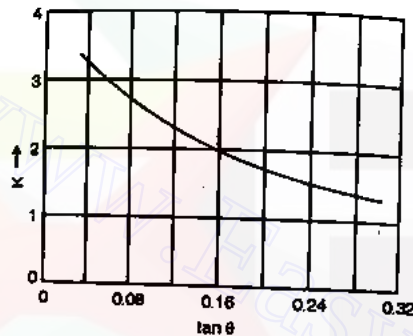


FIG. 11.34 DETERMINATION OF SHAPE FACTOR K .

depth. Case (a) has been dealt with under 'horizontal aprons'. Cases (d) and (b) hereafter will be known as basin III and IV respectively.

Basin III [Fig. 11.37(a)] is recommended for the case where tail water curve is higher than the D_2 curve at all discharges.

Basin IV [Fig. 11.37(b)] is suitable for the case where the tail water depth at maximum discharge exceeds D_2 considerably but is equal to or slightly greater than D_2 at lower discharges.

Design criteria. It is not possible to standardise design criteria for sloping aprons to the same extent as in case of horizontal apron. In this case, greater individual judgement is required. The slope and overall shape of the apron are determined from economic consideration, the length being judged by the type and soundness of the river bed downstream. The following design criteria should serve only as a guide in proportioning the sloping apron designs.

Basin type III. In the design of Basin III, the following procedure may be adopted (Fig. 11.37) :

(1) Assume a certain level at which the front of jump will form for the maximum tail-water depth and discharge.

(2) Determine D_1 from the known upstream total energy line by applying Bernoulli's theorem and calculate F_1 . Then find out the conjugate depth D_2 from Eq. 11.15.

(3) Assume a certain slope and determine the conjugate depth D_2' and the length of the jump for the above Froude number from Fig. 11.32 and 11.36 respectively. The length of the apron should be kept equal to 60 per cent of the jump length.

(4) Test whether the available tail-water depth at the end of the apron matches the conjugate depth D_2' . If not, change the slope or the level of upstream end of the apron or both. Several trials may be required before the slope and the location of the apron are compatible with the hydraulic requirement.

(5) The apron designed for maximum discharge may then be tested at lower discharges, say $\frac{1}{4}$, $\frac{1}{2}$ and $\frac{3}{4}$. If the tail-water depth is sufficient, or in excess of a conjugate depth for the intermediate discharges, the design is acceptable. If not, a flatter slope at lower apron level should be tried or Basin IV may be adopted.

(6) The basin should be supplemented by a solid or dentated end sill of height (h_s) equal to 0.05 to 0.2 D_2 with an upstream slope to 2 : 1 to 3 : 1.

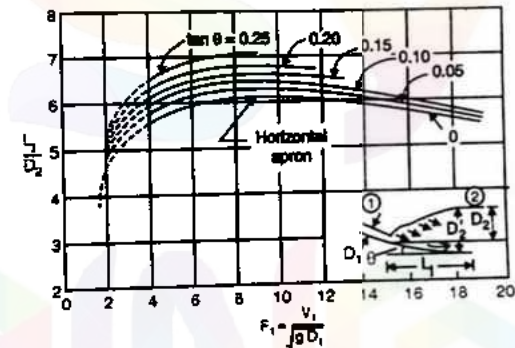


FIG. 11.36 LENGTH OF JUMP IN TERMS OF CONJUGATE DEPTH D_2 FOR BASIN III.

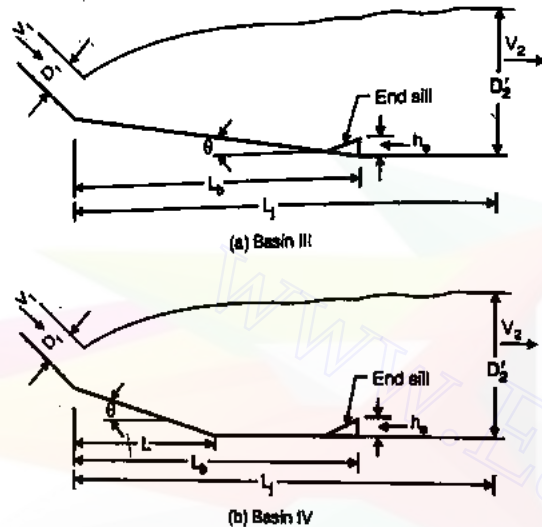


FIG. 11.37. STILLING BASIN TYPES III AND IV.

Basin type IV. In the design of basin IV, the following procedure may be adopted.

1. Determine the discharge at which the tail-water depth is most deficient.
2. For the above discharge, determine the level and length of the apron on the basis of criteria given for horizontal apron.
3. Assume a certain level at which the front of jump will form for the maximum tail-water depth and discharge.
4. Determine D_1 from the known upstream total energy line by applying Bernoulli's theorem and calculate F_1 . Then find out conjugate depth D_2 from Eq. 11.15.
5. Determine a suitable slope (by trial and error) so that the available tail-water depth matches the required conjugate depth D_2' determined from Fig. 11.33.
6. Determine the length of jump for the above slope from Fig. 11.26. If the sum of lengths of inclined portion and horizontal portion is equal to about 60 per cent of jump length, the design is acceptable. If not, fresh trials may be done by changing the level of the upstream end of the jump formation.
7. The basin should be supplemented by a solid or dentated end sill of height 0.05 to 0.2 D_2 and upstream slope of 2 : 1 to 3 : 1.

11.12. SPILLWAY CREST GATES

As stated earlier, spillways are of two types : *controlled* and *uncontrolled*. Controlled spillway is the one the flow over which is controlled by using gates over its crest. By installing gates over spillways, additional storage can be made available, specially

during low water flow. However, during floods, these gates are removed or lifted so that full use of spillway capacity can be made. However, crest gates over spillways for earth dams should be used with caution since faulty operation may cause the rise in the reservoir water level and subsequent overtopping of the earth dam.

The following are some of the common types of gates used for spillways :

1. Flash boards, stop logs and needles.
2. Radial gates.
3. Drum gates.
4. Vertical lift gates.
5. Bear trap gates.
6. Rolling gates.

1. Flash Boards, Stop Logs and Needles

Flash boards : These are the temporary gates used only for small spillway of minor importance. Fig. 11.38(a) shows two types of flash boards. They consist of wooden panels supported by pins on the edges. The pins are supported on pipe sockets along the crest of the dam. Flash board may be either *braced* or *hinged* or *automatic*. Automatic flash boards are designed in such a way that they fall automatically at a certain head.

Stop logs : They consist of horizontal timber planks spanning across piers having grooves. These planks are kept over one another as shown in Fig. 11.38(b), and can be removed either by hand or with the help of any hoisting equipment.

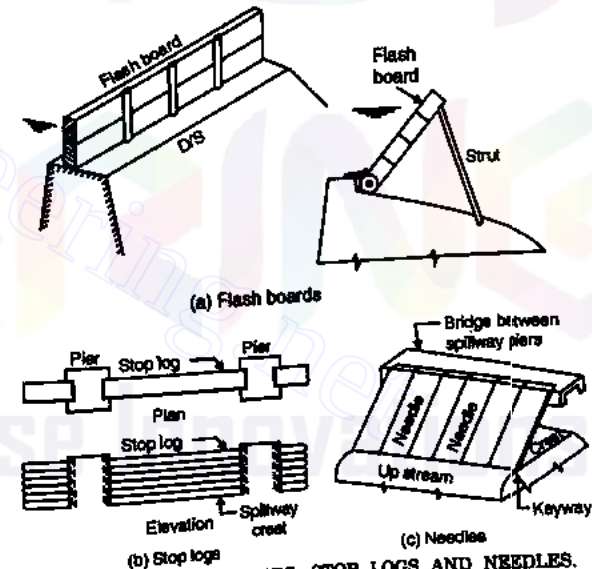


FIG. 11.38. FLASH BOARDS, STOP LOGS AND NEEDLES.

Needles : Needles consists of wooden planks kept in inclined position, with lower ends resting in a keyway on the spillway crest and upper ends at the top of a bridge girder, as shown in Fig. 11.38 (c).

2. Radial Gates

A radial gate, also known as a *tainter gate*, has its water supporting face, made of steel plates, in the shape of sector of a circle, properly braced and hinged at the pivot, as shown in Fig. 11.39. The gate can thus be made to rotate about fixed horizontal axis. The load of the gate and water etc. is carried on bearings, mounted on piers. The gate can be lifted by means of ropes and chains acting simultaneously at both ends or with the help of power driven winches.

3. Drum Gates

Drum gates are normally used for long span. Fig. 11.40 shows the U.S.B.R. type drum gate which consists of a circular sector in cross-section formed by skin plates attached to internal bearings. It is hinged at the centre of curvature in such a way that the entire sector may be raised above the crest or may be lowered so that upper surface becomes coincident with the crest line. The buoyant forces due to head water pressure underneath aid in its lifting. It is enclosed on all the three faces and at ends to form a water-tight vessel.

4. Vertical Lift Gates

A vertical gate consists of a frame work of skin plate at the u/s face along with beams and girder suitably placed. Vertical gates are rectangular in shape, and move vertically in their own plane. Because of the hydrostatic pressure, high frictional forces are developed in the guides. These frictional forces can be reduced by placing cylindrical rollers between the bearing surfaces of the gate and guides. There are several types of vertical lift gates such as (i) Sliding gates, (ii) Fixed wheel gates, (iii) Stoney gates. In the case of *sliding gates*, the frame of the gate bears directly on the downstream guide members, the seal being formed in contact between the two. In the *fixed wheel type* the gates have fixed wheels mounted on each side to carry the water load to a vertical track on down-stream side of the gate groove. This reduces the frictional force and hence fixed wheel lift gates are the one most commonly adopted, for heights upto 15 m and spans of 15 m or more. The gate is hoisted by horizontal lifting beam, the ends of which travel in the gate guides.

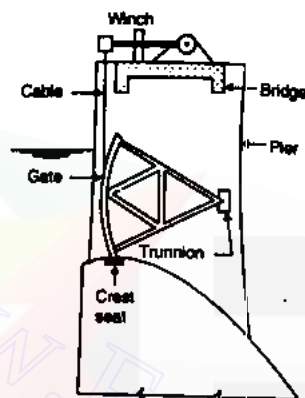


FIG. 11.39. TAINTER GATE.

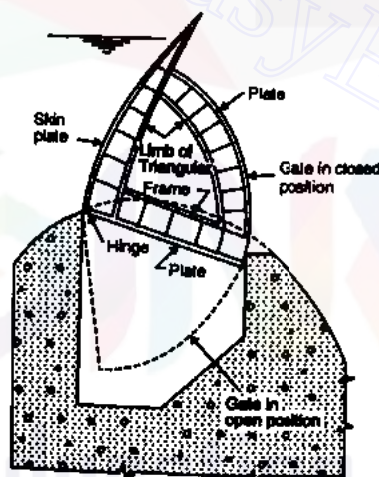


FIG. 11.40. U.S.B.R. DRUM GATE.

Fig. 11.41 shows the plan and elevation of stoney gate. The moving train of rollers is substituted for fixed wheel. These rollers are independent of gates and guides, thus eliminating axle friction. Only rolling friction is developed. The rollers transmit the entire load from a bearing strip on the gate to a roller path on the guide.

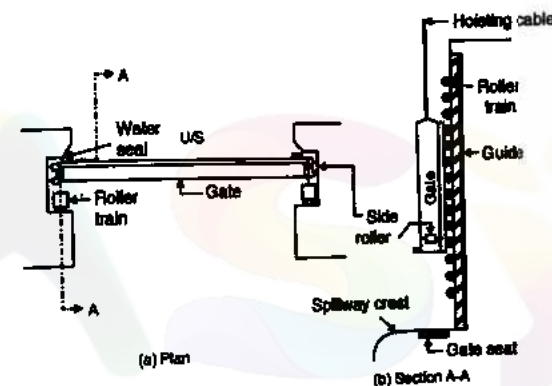


FIG. 11.41. STONEY GATES.

5. Bear Trap Gates

Fig. 11.42 (a) shows a bear trap gate consisting of two leaves of either timber or steel hinged to the dam. These gates are lifted up by admitting water to the space under the leaves. The downward or downstream leaf is often made hollow so that its buoyancy aids in the lifting operation. These gates are often used for low navigation dams.

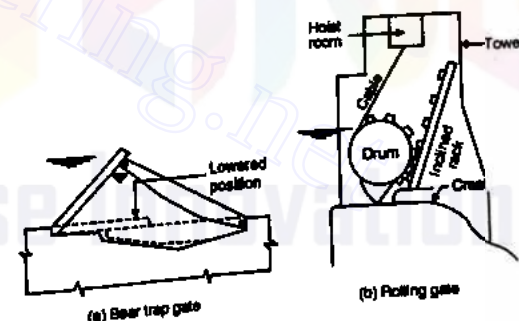


FIG. 11.42.

6. Rolling Gate

Fig. 11.42(b) shows a rolling gate. It essentially consists of steel cylinder as large in diameter as the height of opening and spanning between piers. A heavy annular rim having gear teeth at its periphery encircles each end of the cylinder. Each pier has an inclined rack which engages the gear teeth. The gate is rolled up the inclined rack by means of pull from the hoisting cable operated from the hoist room. A cylindrical segment, attached to the lower portion of the gate, makes contact with spillway.

11.13. OUTLET WORKS

An outlet work serves to regulate to release water impounded by a dam. It provides a controlled release of water from the reservoir in such quantities and at such time as may be required. Outlet may be required for several purposes such as feeding water to power turbines, discharging heavily laden silty water, supplying water to irrigation channels, or for a combination of multipurpose requirements. In certain instances, the outlet works of a dam may be used in lieu of a service spillway in conjunction with an auxiliary or secondary spillway. An outlet works may also act as a flood control regulator, to release water temporarily stored in flood control storage space or to evacuate storage in anticipation of flood inflows.

An outlet works may consist of the following components :

1. Water way.
2. Control device, such as gates, valves etc.
3. Intake structure.

1. **Water way.** The water way for outlet structure may be either in the form of a pipe or tunnel that passes directly through the dam. The water way is also called *sluice way*. In the case of masonry or concrete dam, sluice way may pass through the body of the dam while in the case of earth dams, sluice way is generally placed outside the limits of the embankment. However in case it is essential to place the sluice way through the body of the earth dam, it should have projecting collars at regular intervals. These collars increase the path of water seeping along the boundary of the sluice way. The length of the seepage path should be more so that no damage is done to the earth embankment, since many failures of earth dams have been reported due to the seepage of water around sluice way. If there are n number of collars, each having x projection, then in order that the collars may increase the seepage path atleast by 25%, we have

$$2nx = 0.25L \quad \text{where } L \text{ is the length of the sluice way.}$$

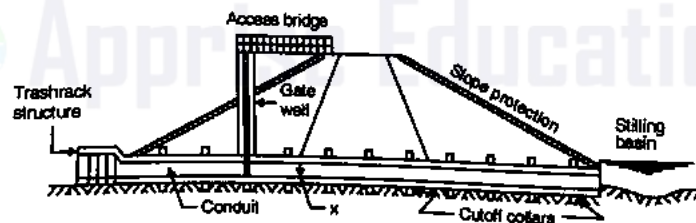


FIG. 11.43. OUTLET STRUCTURE THROUGH DAM.

The inlet of the sluice way must be at minimum reservoir level. However, in big dams outlets may be at different levels. The sluice way may be either circular or rectangular in shape. They must have bellmouth entry.

2. Control devices.

The sluice ways should have simple gates or valves at its entry. Cast iron slide gates, which may be used for control and guard gates are commercially available both for rectangular and circular openings and for design heads upto 15 m. However, simple radial gates may also be used for ordinary surface installation where the sluice way is in the form of an open channel. These gates may be operated either through as gate-well in the case of an earth dam (Fig. 11.43) or through a separate hoisting arrangement kept at top of a gravity dam, or sometimes through the gallery installed in case of gravity dam, as shown in Fig. 11.44.

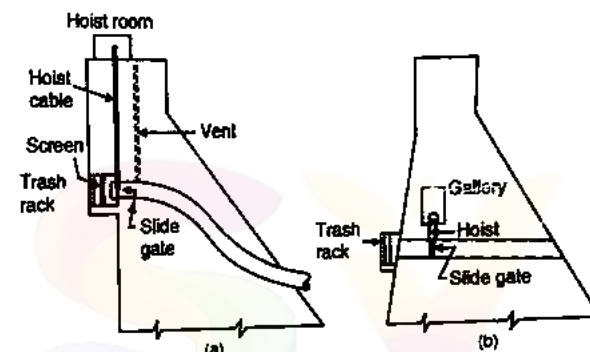


FIG. 11.44. CONTROL OF OUTLET GATES.

3. Intake structure.

An intake structure forms the entrance into the outlet works. It supports trash racks, fish screens and bypass devices, and accommodates the control device (gates and valves). An intake structure may either be submerged or extended as a tower to some height above the maximum reservoir water surface. Submerged intake structure is used when trash cleaning is ordinarily not required. In that case, it only serves as entrance to the outlet conduit. A tower intake is provided if an operating platform is needed for trash racking, maintaining and cleaning of fish screens, or installing stop logs.

Trash racks.

Trash rack is a structure which is provided at the entrance to the intakes and sluice ways, to prevent entrance to debris. These are made in the form of semi-polygonal grid of iron or steel bars, as shown in Fig. 11.45. Steel flats or bars are spaced from 5 to 15 cm centres, depending upon the maximum size of debris which can be permitted in the sluice way. A velocity of 0.5 m/sec is generally allowed when trash racks are inaccessible for cleaning, but it may be increased to 15 m/sec if it is accessible for cleaning purposes. The debris which is collected on the trash rack may be removed either manually or with the help of automatic power driven racks.

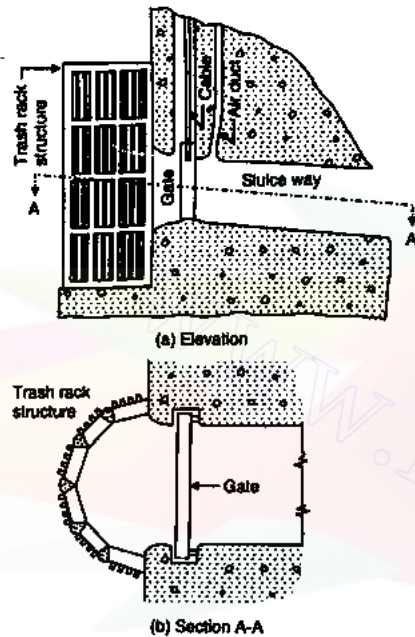


FIG. 11.46. TRASH RACK.

11.14. EXAMPLES FROM COMPETITIVE EXAMINATIONS

Example 11.6. A round crested spillway passes a design discharge of $1 \text{ m}^3/\text{sec}$ per metre length. The coefficient of discharge may be taken as $C_d = 0.7$. If height of the crest above the downstream stilling basin floor level is 10 m , design the (i) depth and (ii) length of the stilling basin. Depth of flow in the stream on the down stream of spillway is 1 m at the design discharge of $1 \text{ m}^3/\text{sec}$. Enquire if the bed of the stilling basin has to be depressed. (Engg. Services Exam. 1987)

Solutions : Given $q = 1 \text{ cumec/m}$ and $C_d = 0.7$

The discharge (q) over the round crested weir, per metre length is given by

$$q = \frac{2}{3} C_d \sqrt{2g} h^{3/2}$$

$$1 = \frac{2}{3} \times 0.7 \times \sqrt{2 \times 9.81} h^{3/2} \text{ from which } h = 0.62 \text{ m.}$$

$$\text{Average fall of water} = 10 + \frac{0.62}{2} = 10.31 \text{ m.}$$

$$\text{Theoretical velocity of flow at the foot of the spillway}$$

$$= \sqrt{2gH} = \sqrt{2 \times 9.81 \times 10.31} = 14.22 \text{ m/sec.}$$

Assuming coefficient of velocity $C_v = 0.9$,

Velocity of approach, $V_1 = 0.9 \times 14.22 = 12.80 \text{ m/sec.}$

Depth of flow at the foot of spillway is given by

$$y_1 = \frac{q}{V_1} = \frac{1}{12.8} = 0.078 \text{ m}$$

$$\text{Froude's Number at u/s, } F_1 = \frac{V_1}{\sqrt{gy_1}} = \frac{12.8}{\sqrt{9.81 \times 0.078}} = 14.62$$

Since $F_1 > 1$, the flow is super-critical. Depth at the downstream side, $y_2 = 1 \text{ m.}$

$$V_2 = \frac{q}{y_2} = \frac{1}{1} = 1 \text{ m/sec.}$$

$$\therefore \text{Froude's number at d/s, } F_2 = \frac{V_2}{\sqrt{gy_2}} = \frac{1}{\sqrt{9.81 \times 1}} = 0.319 < 1$$

Hence the downstream flow is sub-critical and hydraulic jump will be formed

$$\therefore y_2 = \frac{y_1}{2} [\sqrt{1 + 8F_1^2} - 1] = \frac{0.078}{2} [\sqrt{1 + 8(14.62)^2} - 1] = 1.574 \text{ m.}$$

Depth available in the stream at d/s = 1.0 m only.

Hence the stilling basin is to be depressed by value = $1.574 - 1.0 = 0.574 \text{ m.}$

Length of stilling basin = 5 times $(y_2 - y_1)$.

Keep length of stilling basin = $5(y_2 - y_1) = 5(1.574 - 0.078) \approx 7.5 \text{ m.}$

Example 11.7. An overflow spillway shown in Fig. 11.46 passes a discharge of $7.83 \text{ m}^3/\text{sec}/\text{metre width}$ with a fall of 12.5 m . Depth of water available on the down stream is 2.0 m . Calculate the leading dimensions of hydraulic jump stilling basin. (Gate, 1988)

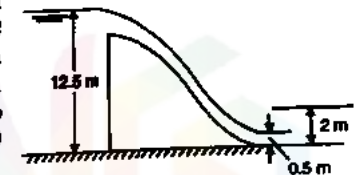


FIG. 11.46

Solution

Depth of flow at the foot of spillway
 $= y_1 = 0.5 \text{ m}$

$$\therefore \text{Velocity of flow, } V_1 = \frac{q}{y_1} = \frac{7.83}{0.5} = 15.66 \text{ m/s}$$

$$\text{Froude number, } F_1 = \frac{V_1}{\sqrt{gy_1}} = \frac{15.66}{\sqrt{9.81 \times 0.5}} = 7.07 \text{ m}$$

Since $F_1 > 1$, the flow is super-critical

$$\text{At the downstream, } d = 2 \text{ m and } V_2 = q/d = \frac{7.83}{2} = 3.915 \text{ m/s}$$

$$\therefore \text{Froude number, } F_2 = \frac{V_2}{\sqrt{gd}} = \frac{3.915}{\sqrt{9.81 \times 2}} = 0.884 \text{ m/s.}$$

Since $F_2 < 1$, the flow at the downstream is sub-critical, and hence hydraulic jump will be formed.

Taking prejump depth $y_1 = 0.5$ m, the post jump depth y_2 is given by

$$y_2 = \frac{y_1}{2} \left[\sqrt{1 + 8 F_1^2} - 1 \right] = \frac{0.5}{2} \left[\sqrt{1 + 8 (7.07)^2} - 1 \right] = 4.76 \text{ m.}$$

Depth available at $d/s = 2$ m

Hence depression of the stilling basin below G.L. = $4.76 - 2 = 2.76$ m

Length of stilling basin = 5 times height of jump.

$$\text{Keep length} = 5 (y_2 - y_1) = 5 (4.76 - 0.5) \triangleq 21.5 \text{ m}$$

Example 11.8. A vertical lift gate $5 \text{ m} \times 2.5 \text{ m}$ size weighing 0.5 tonnes slides along guides (coefficient of friction 0.25) fitted on the side walls of an overflow spillway and its crest. What force will have to be exerted at the hoisting mechanism to lift the gate when the head of water over crest is 2 m. (Engg. Services Exam. 1998)

Solution : Mass of gate = $500 \text{ kg} = 500 \times 9.81 \text{ N}$

Total hydrostatic force exerted by water on the gate is

$$F = \rho g A \bar{h}$$

where A = immersed area of gate = $2 \times 2.5 = 5 \text{ m}^2$

$\bar{h}$ = depth of c.g. of area below the free surface = $2/2 = 1$

$$F = 1000 \times 9.81 \times 5 \times 1 = 49.05 \times 10^3 \text{ N} = 49.05 \text{ kN}$$

Coefficient of friction between gate and guide = 0.25

$\therefore$ Downward frictional force

$$= \mu F = 0.25 \times 49.05 = 12.2625 \text{ kN}$$

$\therefore$ Total downward force

= force due to self wt. of gate + downward frictional force

$$= (500 \times 9.81 \times 10^{-3}) + 12.2625 = 17.1675 \text{ kN}$$

Example 11.9. At an energy dissipater structure below a low spillway, the discharge is $19 \text{ m}^3/\text{s}$ and the energy loss is 1 m at hydraulic jump forming therein. Determine the depths of flow at both ends of the jumps. (Engg. Services Exam. 2002)

Solution : Given $q = 19 \text{ m}^3/\text{s}$ and $\Delta E = 1 \text{ m}$.

$$\text{We have } \Delta E = \frac{(y_2 - y_1)^3}{4 y_1 y_2}$$

$$\text{Hence } \frac{\Delta E}{y_1} = \frac{\left[\frac{y_2}{y_1} - 1 \right]^3}{4 (y_2/y_1)} = \frac{(x - 1)^3}{4x} \text{ where } x = y_2/y_1$$

or

$$\frac{1}{y_1} = \frac{(x - 1)^3}{4x} \quad \dots(1)$$

Also,

$$\frac{y_2}{y_1} = \frac{1}{2} \left[-1 + \sqrt{1 + \frac{8 q^2}{g y_1^3}} \right]$$

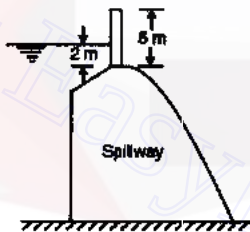


FIG. 11.47

or

$$x = \frac{1}{2} \left[-1 + \sqrt{1 + \frac{8 \times 19^2}{9.81 y_1^3}} \right] = \frac{1}{2} \left[-1 + \sqrt{1 + \frac{294.39}{y_1^3}} \right] \quad \dots(2)$$

Substituting the value of $\frac{1}{y_1}$ from (1) into (2), we get

$$x = \frac{1}{2} \left[-1 + \sqrt{1 + \frac{294.39 (x - 1)^3}{64 x^3}} \right] = \frac{1}{2} \left[-1 + \sqrt{1 + \frac{4.6 (x - 1)^3}{x^3}} \right] \quad \dots(3)$$

Solving Eq. (3) by trial and error we get $x = 2.806$

$$\text{Hence } y_1 = \frac{4x}{(x - 1)^3} = \frac{4 \times 2.806}{(2.806 - 1)^3} = 1.905 \text{ m}$$

and

$$y_2 = x y_1 = 2.806 \times 1.905 = 5.347 \text{ m}$$

PROBLEMS

1. What is a spillway? What are its functions? Enumerate various types of spillways.
2. Write a note on ogee-shaped spillway.
3. What is a siphon spillway? Sketch a saddle siphon spillway and explain the functions of its various component parts.
4. Derive expression for discharge through a saddle siphon spillway. Also, find maximum negative pressure developed at the centre of the throat of saddle siphon spillway.
5. What do you understand by 'priming' and 'depriming'? Describe various devices used for early priming in saddle siphon spillway.
6. Explain the qualities of a good siphon.
7. Describe, with the help of sketch, the working of a volute siphon.
8. Compute the discharge over an ogee spillway with a coefficient of discharge $C = 2.5$ at a head of 4 m . The effective length of the spillway is 100 m . Neglect the velocity of approach. [Ans 200 cumecs]
9. Discuss various methods used for energy dissipation below spillways.
10. A siphon spillway has the following cross-section at its throat: height of throat = 1.5 m and width = 4 m . At the design flow, the tail water is 2 m above the summit. Taking a coefficient of discharge as 0.6 , determine the capacity of the siphon. (a) Determine the head that would be required on an ogee spillway 4 m long to discharge this flow if coefficient of discharge is 2.25 . (b) What length of this ogee weir would be required to discharge the same flow with a head of 2 m on the crest? [Ans. (a) 48 cumecs, (b) $h = 3.05 \text{ m}$, (c) 7.75 m]
11. Explain the design procedure for the standard stilling basin type I.
12. Explain with the help of a sketch, the components and working of a tainter gate.
13. Write a note on U.S.B.R. drum gate.

Diversion Headworks

12.1. INTRODUCTION

Any hydraulic structure which supplies water to the off-taking canal is called a *headwork*. Headworks may be divided into two classes :

1. Storage headwork.
2. Diversion headwork.

A *storage headwork* comprises the construction of a dam across the river. It stores water during the period of excess supplies in the river and releases it when demand overtakes available supplies.

A *diversion headwork* serves to divert the required supply into the canal from the river. A diversion headwork serves the following *purposes*.

1. It raises the water level in the river so that the commanded area can be increased.
2. It regulates the intake of water into the canal.
3. It controls the silt entry into the canal.
4. It reduces fluctuations in the level of supply in the river.
5. It stores water for tiding over small periods of short supplies.

A diversion headwork can further be sub-divided into two principal classes :

1. Temporary spurs or bunds.
2. Permanent weirs and barrages.

Temporary spurs or bunds are those which are temporary and are constructed every year after the floods. However, for important works, weirs or barrages are constructed since they are of permanent nature if designed properly.

Weir. The weir is a solid obstruction put across the river to raise its water level and divert the water into the canal (Fig. 12.1). If a weir also stores water for tiding over small periods of short supplies, it is called a *storage weir*. The main difference between a storage weir and a dam is only in height and the duration for which the supply is stored. A dam stores the supply for a comparatively longer duration.

Barrage. The function of a barrage is similar to that of weir, but the heading up of water is effected by the gates alone (Fig. 12.2). No solid obstruction is put across the river. The crest level in the barrage is kept at a low level. During the floods,

(566)

DIVERSION HEADWORKS

567

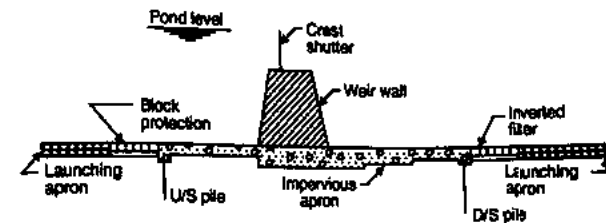


FIG. 12.1. VERTICAL DROP WEIR.

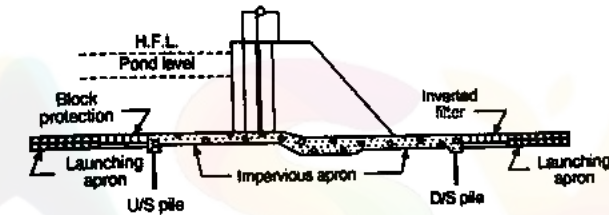


FIG. 12.2. BARRAGE.

the gates are raised to clear off the high flood level, enabling the high flood to pass downstream with minimum afflux. When the flood recedes, the gates are lowered and the flow is obstructed, thus raising the water level to the upstream of the barrage. Due to this, there is less silting and better control over the levels. However, barrages are much more costlier than the weirs.

12.2. COMPONENT PARTS OF A DIVERSION HEADWORK

A diversion headwork consists of the following component parts (Fig. 12.3) :

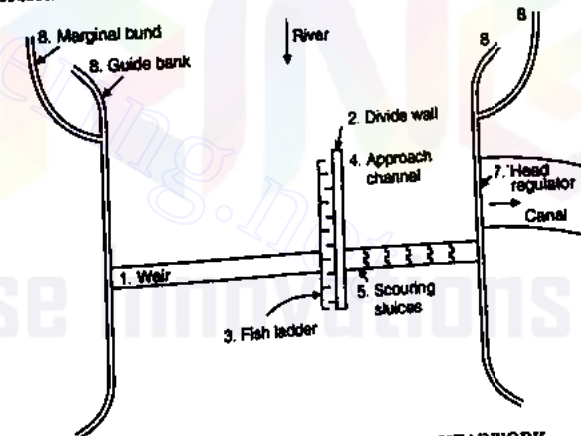


FIG. 12.3. COMPONENT PARTS OF A HEADWORK.

1. Weir or barrage
2. Divide wall or divide groyne
3. Fish ladder
4. Pocket or approach channel
5. Scouring sluices
6. Silt prevention devices
7. Canal head regulator
8. River training works (Marginal bunds and guide banks)

The description and design details of these parts are discussed in the following articles.

12.3. THE WEIR

Weirs are classified into two heads, depending upon the criterion of the design of their floors.

1. Gravity weirs
2. Non-gravity weirs

A gravity weir is the one in which the uplift pressure due to the seepage of water below the floor is resisted entirely by the weight of floor. In the non-gravity type, the floor thickness is kept relatively less, and the uplift pressure is largely resisted by the bending action of the reinforced concrete floor.

Depending upon the material and certain design features, gravity weir (or simply weirs) can further be sub-divided into the following types :

1. Vertical drop weir.
2. Sloping weir :
 - (a) Masonry or concrete slope weir
 - (b) Dry stone slope weir.
3. Parabolic weir.

1. **Vertical drop weir** (Fig. 12.1). A vertical drop weir consists of a vertical drop wall or crest wall, with or without crest gates. At the upstream and downstream ends of the impervious floor, cutoff piles are provided. To safeguard against scouring action, launching aprons are provided both at upstream and downstream end of the floor. A graded inverted filter is provided immediately at the downstream end of the impervious floor to relieve the uplift pressure. Vertical drop weirs are suitable for any type of foundation.

2. **Masonry or Concrete Sloping Weir** (Fig. 12.4(a)). Weirs of this type are of recent origin. They are suitable for soft sandy foundations, and are generally used where the difference in weir crest and downstream river bed is limited to 3 me-

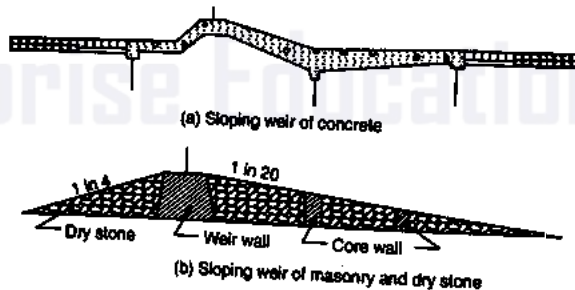


FIG. 12.4. SLOPING WEIRS.

DIVERSION HEADWORKS

tres, When water passes over such a weir, hydraulic jump is formed on the sloping glacis.

3. **Dry stone slope weir** [Fig. 12.4(b)]. A dry stone weir or a rockfill weir consists of a body wall (or weir wall) and upstream and downstream rockfills laid in the form of glacis, with few intervening core walls. Okhla weir on Yamuna river, near Delhi, is the example of such weir.

4. **Parabolic Weir** (Fig. 12.5). A parabolic weir is similar to the spillway section of a dam. The body wall for such a weir is designed as a low dam. A cistern is provided at the downstream side to dissipate the energy. The upstream and down-stream protection works are similar to that of a vertical drop or sloping glacis weir.

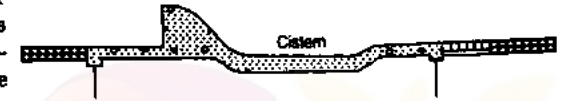


FIG. 12.5. PARABOLIC WEIR.

12.4. LOCATION OF HEADWORKS

A river, in its course, is divided into four distinct regions or stages :

- (i) *The torrential, rocky or mountainous stage* : The river in this stage has very steep bed slope and high velocity.
- (ii) *The sub-mountainous or boulder stage* : The sides and bed of the river are composed of boulder and gravel. There is strong subsoil flow in this region.
- (iii) *Trough stage or alluvial plain* : The cross-section of the river is made of alluvial sand and silt. The bed slopes are small and velocities are gentle.
- (iv) *Delta stage* : The first and last stages are unsuitable for construction of weirs. The choice of site selection lies between the boulder stage and the trough stage. The advantages and disadvantages of the first three stages are given below :

(a) Rocky Stage

(i) *Disadvantages of Rocky Stage*

1. The soil, suitable for irrigation, must have good depth. Such a soil is not available in the rocky stage. This results in a *long idle length of the canal* from the headworks to the plains where good soil is available.
2. A canal taking off from a headwork situated in rocky stage will have to cross more discharge lines due to uneven mountainous ground. Hence, *more cross drainage works* are required.
3. The ground in this region has steep gradients. Hence, either the canal bed will have to be lined to permit high velocities or *more falls* are necessary to dissipate the energy.
4. A shingle-excluding regulator is necessary if the headwork is situated on the rocky stage. This requires a *costly head regulator*.
5. The shingle crossing over the weir may damage it. This requires *frequent repairs* of the weir.

6. The high velocity associated with the rocky stage requires use of better material, increasing the cost of construction.
7. The water in this region does not contain silt and is thus devoid of fertilising materials.
8. The rivers in mountainous regions are more flashy, resulting in floods appearing and disappearing suddenly. This causes frequent failures.

(ii) **Advantages of Rocky Stage :**

1. Good rocky foundations are available at surface or shallow depths. Hence, cost of construction of weir is less.
2. High heads are available for hydroelectric work.
3. There are no chances of the supply channel getting silted, since the steep slopes give rise to high velocities.
4. Comparative silt free water is fed to the turbines.

(d) **Sub-mountainous or Boulder stage**

(i) **Disadvantages**

1. There is strong subsoil flow. This decreases the storage and may cause damage to the floor downstream.
2. As in rocky stage, long idle length of canal is required.
3. There will be more percolation losses from the canal taking off in boulder tracts. This will increase the cost per cumec per net supply.
4. More cross-drainage works are required.
5. In the boulder stage, rivers are more flashy than in trough stage.
6. There is less demand of water at head reaches.

(ii) **Advantages of Boulder Stage :**

1. Cash crops like transplanted rice, tobacco, sugar, tea etc., can be grown in the sub-mountainous regions, where there was no irrigation earlier.
2. Cost of headworks is less due to availability of local material.
3. Less river training works are required.
4. Falls can be utilized for power generation.

(c) **Trough stage**

(i) **Disadvantages of trough reach :**

1. Cost of headworks more due to poor foundation.
2. More river training works are required.
3. There is problem of silt in the canal.

(ii) **Advantages of trough reach :**

1. Subsoil flow is comparatively less.
2. The head works constructed in this region serve large area than a weir in boulder region can. There is no idle length of canal.

DIVERSION HEADWORKS

3. The tracts lying at the foot of hills do not require water with the same urgency as those lower down. Consequently headworks in trough regions are more remunerative as the cultivators will be willing to pay higher assessment.

4. The cross-drainage works across the canals are less.
5. The water contains silt and other fertilising material.

12.5. EFFECTS OF CONSTRUCTION OF A WEIR ON THE REGIME OF RIVER

The weir is an obstruction thrown in the path of water. Due to its construction, therefore, the regime of river is affected in the following ways :

1. The silt supporting power of a river or channel mainly depends upon the hydraulic slope. When the weir is constructed, the heading up of water leads to flattening of the surface slope on the upstream side.

2. Due to decrease in the water surface slope, the silt carrying capacity is decreased, and the bulk of silt charge of the river water deposits in the pond, leading to the formation of irregular shoals at u/s of the weir.

3. Also due to silt excluding devices provided at the head regulator, the canal takes less silt. This results in further deposition of silt in the pond.

4. The water passing over the weir and through the scouring sluices now contains a deficient silt charge because much of it has been deposited u/s. In order to maintain a constant silt charge, the flowing water at the d/s scours the bed. This results in a progressive degradation or retrogression of bed levels downstream. The retrogression may undermine the stability of a work by an increase in exit gradient beyond safe limits. During high floods, the retrogression of bed may be from 0.3 to 0.5 m, while at low water levels, it may be as high as 1.2 to 2 metres.

5. As the silting and consequent shoals formation at u/s increases, the resistance to flow of river is increased due to tortuous route the water has to take about shoals. To overcome this resistance, increased head is required. The river starts regaining its original slope, and the afflux is extended more and more to the u/s. A stage is then reached when the upstream section of river cannot take up any more silt, and the normal silt charge is passed on the downstream side. The silt excluding device will also discharge more silt downstream.

6. Due to this, river below the weir will carry an excessive silt charge with a lower discharge. This will result in progressive silting up downstream, an increase in tortuosity and, therefore, a recovery of bed levels downstream. The process of recovery in tortuosity and, therefore, a recovery of bed levels downstream. The process of recovery of downstream bed levels after the initial retrogression is slow and steady, and it may take 20 to 30 years to regain the original bed slope. The recovery of levels to the d/s may lead loss of control on the silt regulation. Hence, sufficient margin should be provided between canal F.S.L. and the pond level so that the crest level of the head regulator can be increased in the event of necessity.

12.6. CAUSES OF FAILURE OF WEIRS AND THEIR REMEDIES

A weir may fail due to the following reasons :

- (i) **Piping**

- (ii) Rupture of floor due to uplift
 - (iii) Rupture of floor due to suction caused by standing wave
 - and (iv) Scour at the upstream and downstream side of the weir floor.
1. **Piping.** Water seeps under the base of the weirs founded on permeable soils. When the flow lines emerge out at the d/s end of the impervious floor of the weir, the hydraulic gradient or the *exit gradient* may exceed a certain critical value for the soil. In that case, the surface soil starts *boiling* and is washed away by percolating water. With the removal of the surface soil, there is further concentration of flow lines into the resulting depression and still more soil is removed. This process of erosion thus progressively works backwards towards the upstream and results in the formation of a channel or a pipe underneath the floor of the weir, causing its failure.

Remedies. Piping failures can be prevented by :

- (i) Providing sufficient length of the impervious floor so that path of percolation is increased and the exit gradient is decreased.
- (ii) Providing pile at downstream end.

2. **Rupture of Floor Due to Uplift.** If the weight of floor is insufficient to resist the uplift pressure, the floor may burst and effective length of impervious floor is thereby reduced. The final failure, however, is due to the reduction of the effective length with the consequent increase in the exit gradient. Examples of such failures are Khanki weir on Chenab and Narora weir on the Ganga.

Remedies. Failures due to rupture of floor may be prevented by :

- (i) Providing impervious floor of sufficient length.
- (ii) Providing impervious floor of appropriate thickness at various points, and
- (iii) Providing pile at the upstream end so that the uplift pressure to the down stream (d/s) is reduced.

3. Rupture of Floor Due to Suction Caused by Standing Wave.

The standing wave or hydraulic jump formed at the d/s of the weir causes suction which also acts in the direction of uplift pressure. If the floor thickness is insufficient, it may fail by rupture. Examples of such failures are Marala weir on the Chenab and Rasul weir.

Remedies. Failures can be prevented by :

- (i) Providing additional thickness of floor to counterbalance the extra pressure due to the standing wave.
- (ii) Constructing the floor thickness in one concrete mass instead of in masonry layers.

4. **Scour on the Upstream and Downstream of the Weir.** When the natural waterway of a river is contracted, the water may scour the bed both at upstream and downstream of the structure. The scour holes so formed may progress towards the structure, causing its failure. Examples of such failures are Islam weir and Deoha weir.

Remedies. Such failures can be prevented by :

- (i) Taking the piles at upstream and downstream ends of the impervious floor, much below the calculated scour level.
- (ii) Providing suitable length and thickness of launching aprons at u/s and d/s sides, so that stones of the aprons may settle in the scour holes.

12.7. DESIGN OF IMPERVIOUS FLOOR FOR SUB-SURFACE FLOW

We have already seen that the sub-surface flow or the foundation seepage may cause harm in two ways (i) piping, and (ii) uplift. Following the damage to Khanki weir in 1895, a group of Anglo Indian Engineers under the leadership of Colonel J. Clibborn, Dean of the Roorkee College and J.S. Beresford, Inspector General of Irrigation, India, carried out experiments and confirmed the Darcy's law for seepage through granular soils. Later, in 1912, Bligh advanced a theory for the sub-surface flow and published it in his book, '*The practical design of Irrigation Works*'. In 1932, after analysing about 200 dams all over the World, Lane evolved his *weighted creep theory*. Scientific study of the sub-surface flow was, however, made by Pavlovsky (1922), and Khosla and his associates (1936).

BLIGH'S CREEP THEORY.

The design of the impervious floor, or the *apron* is directly dependent on the possibilities of percolation in the porous soil on which the apron is built. Bligh assumes as an approximation that the hydraulic slope or gradient is *constant* throughout the impervious length of the apron. He further assumed the percolating water to *creep* along the contact of the base profile of the apron with the sub-soil, losing head enroute, proportional to the length of its travel. He designated the length of the travel as the '*creep length*', which is the sum of horizontal as well as vertical length of creep. Bligh asserted that no amount of sheet piling or another cut-off could ever stop the percolation unless the cut-off extends upto the impermeable sub-soil strata. Thus, according to Bligh's theory, the total *creep length* L for the case of Fig. 12.6(a) is

$$L = l$$

For the case of Fig. 12.6(b), the total creep length is

$$L = 2d_1 + l + 2d_2$$

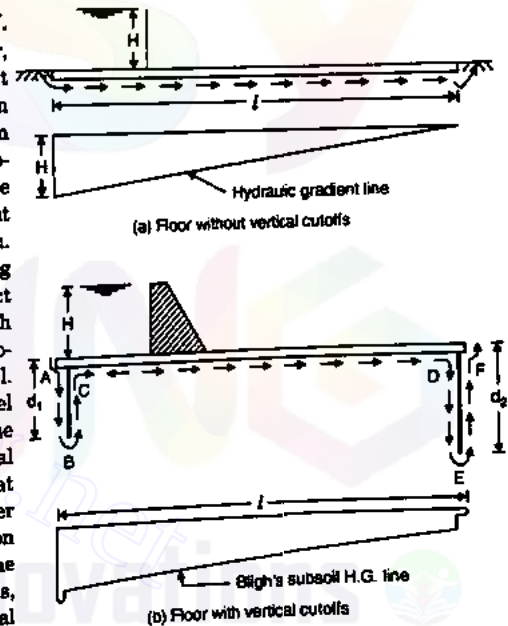


FIG. 12.6. BLIGH'S CREEP THEORY.

This means that in calculating the length of creep, the depth of every cutoff (i.e. vertical creep) is multiplied by the coefficient 2.

If H is the total loss of head, the 'loss of head per unit length of the creep' (c) would be

$$c = \frac{H}{2d_1 + l + 2d_2} = \frac{H}{L} \quad \dots(12.1)$$

He called the 'loss of head per unit length of creep' as *percolation coefficient*. The reciprocal of this (i.e. L/H) is called the *coefficient of creep (C)* and Bligh assigned its safe values for different soils, as given in Table 12.1.

TABLE 12.1. COEFFICIENT OF CREEP.

Type of soil	Value of $C = 1/c$
1. Light sand and mud (as in Nile)	18
2. Fine micaceous sand (as in Northern India rivers)	15
3. Coarse grained sand (as in Central and South India)	12
4. Boulders or shingle, gravel and sand mixed	5 to 9

Design Criteria : Bligh gave two criteria for design :

(i) **Safety against piping** . The length of creep should be sufficient to provide a safe hydraulic gradient according to the type of the soil.

Thus, the safe creep length is given by

$$L = CH \quad \dots(12.2)$$

where C = coefficient of creep = $1/c$.

(ii) **Safety against uplift pressure** . Let h' be uplift pressure head at any point of the apron.

The uplift pressure = wh' ...(i)

If t = thickness of the floor at the point

and ρ = specific gravity of the floor material

Then, downward force (resisting force) per unit area = $t w \rho$...(ii)

Equating the two, we get

$$wh' = tw\rho \quad \dots(iii)$$

or

$$h' = t\rho$$

$$h' - t = t\rho - t = t(\rho - 1)$$

$$\text{From which} \quad t = \frac{h' - t}{\rho - 1} = \frac{h}{\rho - 1} \quad \dots[12.3(a)]$$

where h = ordinate of the hydraulic gradient line measured above the top of the floor.

Providing a factor of safety of 4/3, we have

$$t = \frac{4}{3} \frac{h}{\rho - 1} \quad \dots(12.3)$$

Eq. 12.3 gives the floor thickness t at point of the apron where the uplift pressure ordinate, measured above the top of the floor is h .

For the portion of the floor to the upstream of the barrier, only nominal thickness need be provided since the weight of the water will counterbalance the uplift pressure. It is clear from Fig. 12.7 that out of the total impervious length (l) of the floor, if greater length is provided to the upstream of the barrier, the design will be comparatively economical. However, as we shall see later, a certain minimum length of the impervious floor is always necessary to the d/s of the barrier.

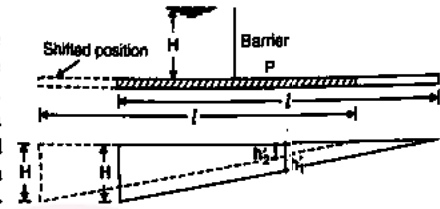


FIG. 12.7 EFFECT OF SHIFTING THE FLOOR RELATIVE TO THE BARRIER.

Limitations of Bligh's Theory

1. Bligh made no distinction between horizontal and vertical creep.
2. Bligh's method holds good so long as the horizontal distance between the pile lines is greater than twice their depth.
3. Bligh did not explain the idea of exit gradient. The safety against undermining cannot simply be obtained by considering a flat average gradient but by keeping this gradient well below critical.
4. Bligh makes no distinction between outer and inner faces of sheet piles or the intermediate sheet pile, whereas according to investigations, the outer faces of the end sheet piles are much more effective than inner ones. Also, intermediate sheet piles of shorter length than the outer ones are ineffective except for local redistribution of pressure.
5. Loss of head does not take place in the same proportion as creep length. Also, the uplift pressure distribution is not linear but follows a sine curve.
6. Bligh does not specify the absolute necessity of providing a sheet pile at d/s end whereas it is absolutely essential to have a deep vertical cut off at d/s end to prevent undermining.

LANE'S WEIGHTED CREEP THEORY

Based on statistical investigations of as many as 278 dams, weirs and barrages all over the World, Lane observed that vertical creep is more effective than the horizontal creep. He therefore modified Bligh's creep theory by evolving 'Lane's Weighted creep theory'. According to this theory, the *weighted creep length (L_w)* is given by

$$L_w = \frac{1}{3} l + V \quad \dots(12.4)$$

where l = the sum of all horizontal contacts and all the sloping contacts having slope less than 45°
 V = sum of all the vertical contacts and all the sloping contacts steeper than 45°

Thus, in Fig. 12.6 (b)

$$L_w = \frac{1}{3}l + 2d_1 + 2d_2$$

To ensure safety against piping, Lane suggested that the weighted creep length must not be less than the following :

$$L_w = C_w \bar{h} \quad \dots(12.4 a)$$

where L_w = weighted creep length given by Eq. 12.4

C_w = Lane's creep coefficient, the value of which depends on the type of soil (Table 12.2).

TABLE 12.2 LANE'S CREEP COEFFICIENT

S.N.	Type of soil	Lane's creep coefficient C_w	Lane's safe Hydraulic gradient ($1/C_w$)
1.	Very fine sand or silt	8.5	1/85
2.	Fine sand	7.0	1/70
3.	Medium sand	6.0	1/60
4.	Coarse sand	5.0	1/50
5.	Fine Gravel	4.0	1/40
6.	Medium Gravel	3.5	1/35
7.	Coarse gravel including cobbles	3.0	1/30
8.	Boulders with some cobbles and gravel	2.5	1/25
9.	Soft clay	3.0	1/30
10.	Medium clay	2.0	1/20
11.	Hard clay	1.8	1/18
12.	Very hard clay or hard pan	1.6	1/16

However, his theory, though an improvement over the Bligh's creep theory, is empirical and lacked the background for a rational basis of design.

Example 12.1. Fig. 12.8 shows the section of a hydraulic structure founded on sand. Calculate the average hydraulic gradient. Also, find the uplift pressures at points 6, 12 and 18 m from the u/s end of the floor and find the thickness of the floor at these points taking $\rho = 2.24$.

Solution

$$\text{Total length of creep} = (2 \times 6) + 22 + (2 \times 8) = 50$$

$$\text{Hydraulic gradient} = \frac{4}{50} = \frac{1}{12.5}$$

(i) **Uplift pressure at a point A, 6 m from u/s**

$$\text{Length of creep upto A} = (6 \times 2) + 6 = 18 \text{ m}$$

$$\therefore \text{Unbalanced head} = h_1 = 4 \left(1 - \frac{18}{50} \right) = 2.56 \text{ m}$$

$$\therefore \text{Uplift pressure} = wh_1 = 9.81 \times 2.56 = 25.11 \text{ kN/m}^2$$

DIVERSION HEADWORKS

$$\text{Thickness } t = \frac{4}{3} \frac{h_1}{\rho - 1}$$

$$= \frac{4}{3} \times \frac{2.56}{2.24 - 1} = 2.76 \text{ m}$$

(ii) **Uplift pressure at point B, 12 m from u/s.**

$$\text{Length of creep upto B} = (6 \times 2) + 12 = 24 \text{ m}$$

Unbalanced head

$$h_2 = 4 \left(1 - \frac{24}{50} \right) = 2.08 \text{ m}$$

$$\therefore \text{Uplift pressure} = wh_2 = 9.81 \times 2.08 = 20.4 \text{ kN/m}^2$$

$$\text{Thickness } t = \frac{4}{3} \frac{h_2}{\rho - 1} = \frac{4}{3} \times \frac{2.08}{2.24 - 1} = 2.29 \text{ m}$$

(iii) **Uplift pressure at point C, 18 m from u/s.**

$$\text{Length of creep upto C} = (6 \times 2) + 18 = 30 \text{ m}$$

$$\therefore \text{Unbalanced head } h_3 = 4 \left(1 - \frac{30}{50} \right) = 1.6 \text{ m}$$

$$\therefore \text{Uplift pressure} = wh_3 = 9.81 \times 1.6 = 15.7 \text{ kN/m}^2$$

$$\text{Thickness } t = \frac{4}{3} \frac{h_3}{\rho - 1} = \frac{4}{3} \times \frac{1.6}{2.24 - 1} = 1.72 \text{ m}$$

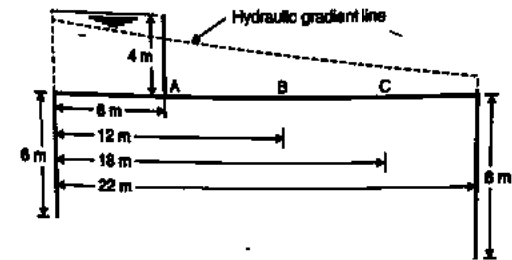


FIG. 12.8. BLIGH'S METHOD.

12.6. KHOSLA'S THEORY

In 1926-27, some siphons on Upper Chenab canal, designed on Bligh's theory, gave trouble. Actual pressure measurements made with the help of pipes inserted in the floors of two of these siphons did not show any relationship with the pressure calculated on the basis of Bligh's theory. This led to the following provisional conclusions by Khosla :

1. The outer faces of the end sheet piles were much more effective than the inner ones and the horizontal length of the floor.
2. The intermediate piles if smaller in length than the outer ones were ineffective except for the local redistribution of pressure.
3. Undermining of the floor started from the tail end. If the hydraulic gradient at exit was more than the critical gradient for the particular soil, the particles would move with the flow of water, thus causing progressive degradation of the sub-soil, resulting in cavities and ultimate failure.
4. It was absolutely essential to have a reasonably deep vertical cutoff at the downstream end to prevent undermining.

Khosla and his associates took into account the flow pattern below the impermeable base of hydraulic structures, to calculate the uplift pressure and exit gradient. Various cases were analysed mathematically. In order to appreciate the essential difference between

Khosla's theory and Bligh's theory. we will start with a simple case of horizontal floor with negligibly small thickness, as shown in Fig. 12.9 analysed by Weaver (1932).

WEAVER'S SOLUTION :

For the case of two dimensional flow occurring under a straight floor, it can be shown that the streamlines are confocal ellipses given by the equation

$$\frac{x^2}{\left(\frac{b}{2} \cosh u\right)^2} + \frac{y^2}{\left(\frac{b}{2} \sinh u\right)^2} = 1 \quad \dots(1)$$

where u = stream line function given by

$$\cosh u = \frac{1}{2}(e^u + e^{-u})$$

$$\text{and } \sinh u = \frac{1}{2}(e^u - e^{-u}) \quad \dots(2)$$

From Eq. 1, any particular streamline can be determined by giving a suitable value to u . To calculate the gradient diagram, we must consider the first streamline AB which 'hugs' to the outline of the floor. Since, the discharge between this line and the solid floor is zero, the stream function $u = 0$.

$$\text{Hence } \cosh u = \cosh 0 = 1 ; \quad \sinh u = \sinh 0 = 0.$$

Substituting in Eq. (1), we get

$$\frac{x^2}{b^2/4} + \frac{y^2}{0} = 1$$

which is true only when $y = 0$. This gives the equation of the straight line AB .

Again, the equipotential lines consists of a set of confocal hyperbolas given by

$$\frac{x^2}{\left(\frac{b}{2} \cos v\right)^2} - \frac{y^2}{\left(\frac{b}{2} \sin v\right)^2} = 1 \quad \dots(3)$$

where v = pressure function, given by

$$v = \pi\phi = \pi \frac{P}{H} \quad \dots(4)$$

where P = pressure head at any point

= height of heading up against the structure.

In order to solve equations (1) and (3) for x and y , let the complex co-ordinate z of any point be represented by

$$z = \frac{b}{2} \cosh w$$

where $z = x + iy$ and $w = u + iv$

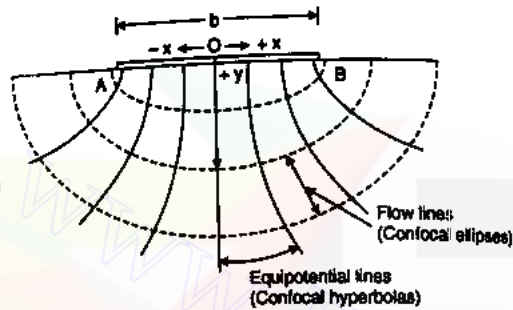


FIG. 12.9. HORIZONTAL FLOOR WITH NEGLIGIBLE THICKNESS.

$$x + iy = \frac{b}{2} \cosh(u + iv)$$

or

$$x + iy = \frac{b}{2} (\cosh u \cos v + i \sinh u \sin v)$$

Separating real and imaginary parts and equating them, we get

$$x = \frac{b}{2} \cosh u \cos v \quad \text{and} \quad y = \frac{b}{2} \sinh u \sin v \quad \dots(5)$$

These values of x and y will be acceptable only if they satisfy both Eqs. 1 and 3.

Thus, substituting these values of x and y in (1), we get

$$\cosh^2 v + \sin^2 v = 1 \quad (\text{satisfied})$$

Similarly, from (3),

$$\cosh^2 u - \sinh^2 u = 1 \quad (\text{satisfied})$$

Thus the values of x and y are given by Eq. 5.

$$\text{Hence } x = \frac{b}{2} \cosh u \cos v$$

In the limiting case, for AB , $u = 0$

$$x = \frac{b}{2} \cosh 0 \cos v = \frac{b}{2} \cos v$$

$$v = \cos^{-1} \frac{2x}{b} \quad \dots(12.5)$$

or

$$\text{But } v = \pi\phi = \pi \frac{P}{H}$$

$$\therefore \phi = \frac{1}{\pi} \cos^{-1} \frac{2x}{b} \quad \dots[12.5(a)]$$

or

$$P = \frac{H}{\pi} \cos^{-1} \frac{2x}{b} \quad \dots[12.5(b)]$$

Thus, for the first flow line AB , the pressures can be obtained by giving x different values in Eq. 12.5. Fig. 12.10 shows the pressure distribution diagrams both by Eq. 12.5 as well as by Bligh's method.

From the study of Fig. 12.10, following two conclusions can be drawn :

1. Slope of pressure diagram at A and B is infinite. Hence, the force at A will be theoretically infinite acting downwards, and that at B will also be infinite acting upwards. This will cause sand boiling, and hence either the floor should be depressed or a cutoff should be provided at the d/s end.
2. D/s half of the floor has more uplift pressure than that calculated by Bligh's method.

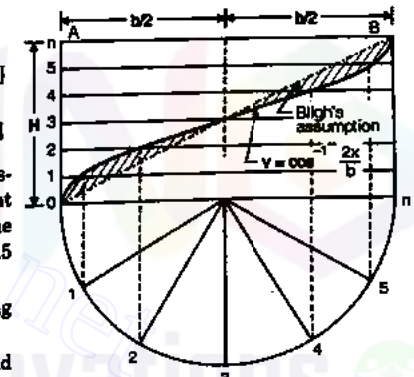


FIG. 12.10. ACTUAL PRESSURE DISTRIBUTION.

COMPOSITE PROFILE OF FLOOR : KHOSLA'S SOLUTION :

Fig. 12.11(a) shows a composite profile of floor or apron, the most general case analysed by A.N. Khosla¹, Dr. N.K. Bose and Dr. E.M. Taylor by the method of Schwarz-Christoffel transformation. Fig. 12.11(b) shows the transformed section. By the mathematical process of transformation, the distorted flow net can be brought back to the normal elliptical and hyperbolic curves.

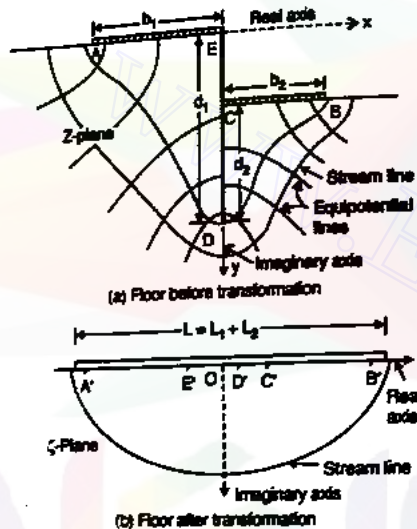


FIG. 12.11. KHOSLA'S CASE OF COMPOSITE PROFILE.

The complete analytical solution, using Schwarz-Christoffel transformation, is given in article 1 of Appendix-1, at the end of the book.

SPECIFIC CASES.

We shall now consider the following specific cases of the general form shown in Fig. 12.11.

- Straight horizontal floor of negligible thickness with pile either at the upstream end or at the downstream end [Fig. 12.12 (a, b)].
- Straight horizontal floor of negligible thickness with pile at some intermediate point [Fig. 12.12(c)].
- Straight horizontal floor depressed below the bed, but with no cutoff [Fig. 12.12(d)].

1. "Design of Weirs on Permeable Foundations", by A.N. Khosla, Dr. N.K. Bose and Dr. E.M. Taylor. Central Board of Irrigation, India, Publication No. 12, 1936.

Case 1 : Pile at some intermediate point (Fig. 12.12 c)

These cases were analysed by Khosla and his associates with the help of Schwarz-Christoffel transformation. An independent analytical solution, for the floor with eccentric or intermediate pile (Fig. 12.12 c) is given in article 3 of Appendix-1 at the end of the book. The uplift pressures, P_E , P_D and P_C at the three key points E, D and C are given by the following equations.

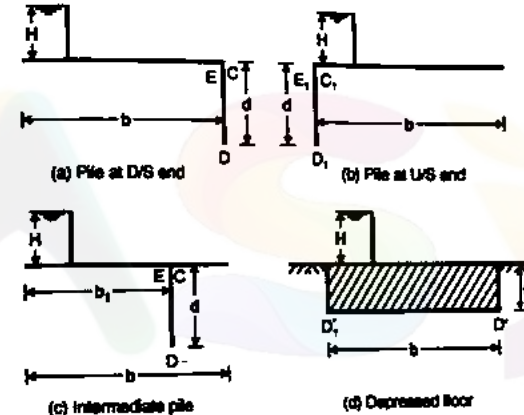


FIG. 12.12. SPECIFIC CASES.

$$P_E = \frac{H}{\pi} \cos^{-1} \left[\frac{\lambda_1 - 1}{\lambda} \right] \text{ or } \Phi_E = \frac{P_E}{H} \times 100 = \frac{100}{\pi} \cos^{-1} \left[\frac{\lambda_1 - 1}{\lambda} \right] \quad \dots(12.6 a)$$

$$P_D = \frac{H}{\pi} \cos^{-1} \left[\frac{\lambda_1}{\lambda} \right] \text{ or } \Phi_D = \frac{P_D}{H} \times 100 = \frac{100}{\pi} \cos^{-1} \left[\frac{\lambda_1}{\lambda} \right] \quad \dots(12.6 b)$$

$$P_C = \frac{H}{\pi} \cos^{-1} \left[\frac{\lambda_1 + 1}{\lambda} \right] \text{ or } \Phi_C = \frac{P_C}{H} \times 100 = \frac{100}{\pi} \cos^{-1} \left[\frac{\lambda_1 + 1}{\lambda} \right] \quad \dots(12.6 c)$$

$$\text{where } \lambda = \frac{\sqrt{1 + \alpha_1^2} + \sqrt{1 + \alpha_2^2}}{2} \quad \dots(12.6 d)$$

$$\lambda_1 = \frac{\sqrt{1 + \alpha_1^2} - \sqrt{1 + \alpha_2^2}}{2} \quad \dots(12.6 e)$$

$$\alpha_1 = b_1/d \text{ and } \alpha_2 = b_2/d \quad \dots(12.6 f)$$

Fig. 12.13 gives the values of uplift pressures Φ_E , Φ_D and Φ_C expressed as percentage of head H , at the three key points E, D and C. The procedure for finding these pressures is explained on the Fig itself, which is self explanatory.

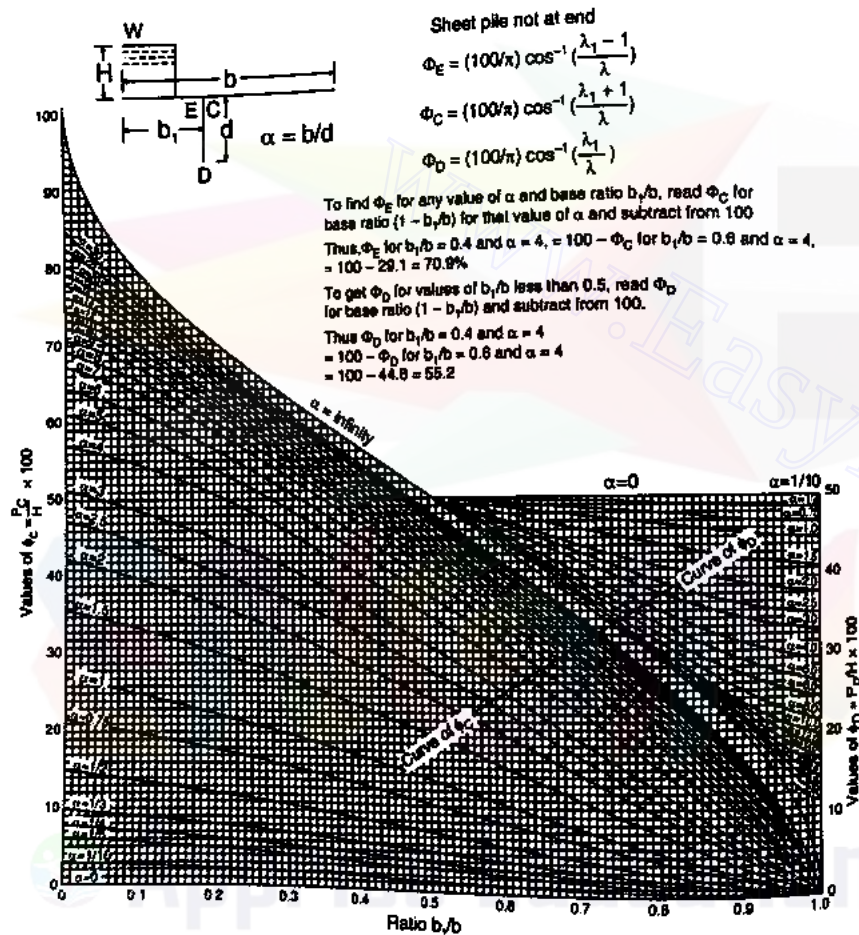


FIG. 12.13. KHOSLA'S CURVES FOR INTERMEDIATE SHEET PILE

Case 2. Pile at downstream end (Fig. 12.12 a)

Article 4 of Appendix -1 gives an independent solution for the case of an impervious floor with pile (or cutoff) at the downstream end (Fig. 12.12 a and Fig. 4 of Appendix-1). The uplift pressure at the key points E, D and C are given by the following equations

$$P_E = \frac{H}{\pi} \cos^{-1} \left[\frac{\lambda-2}{\lambda} \right] \text{ or } \Phi_E = \frac{P_E}{H} \times 100 = \frac{100}{\pi} \cos^{-1} \left[\frac{\lambda-2}{\lambda} \right] \quad \dots(12.7 a)$$

$$P_D = \frac{H}{\pi} \cos^{-1} \left[\frac{\lambda-1}{\lambda} \right] \text{ or } \Phi_D = \frac{P_D}{H} \times 100 = \frac{100}{\pi} \cos^{-1} \left[\frac{\lambda-1}{\lambda} \right] \quad \dots(12.7 b)$$

$$P_C = \frac{H}{\pi} \cos^{-1} \left[\frac{\lambda}{\lambda} \right] = 0 \text{ or } \Phi_C = \frac{P_C}{H} \times 100 = 0 \quad \dots(12.7 c)$$

$$\text{where } \lambda = \frac{1 + \sqrt{1 + \alpha^2}}{2} \text{ and } \alpha = b/d \quad \dots(12.7 d)$$

Fig. 12.14 gives the curves for finding out the values of Φ_E and Φ_D for the case of cutoff at down stream end. This Fig. also gives the curve for finding out the uplift pressure $\Phi_{D'}$ at the point D' situated at the downstream end of a depressed floor, having finite thickness (or depression) d.

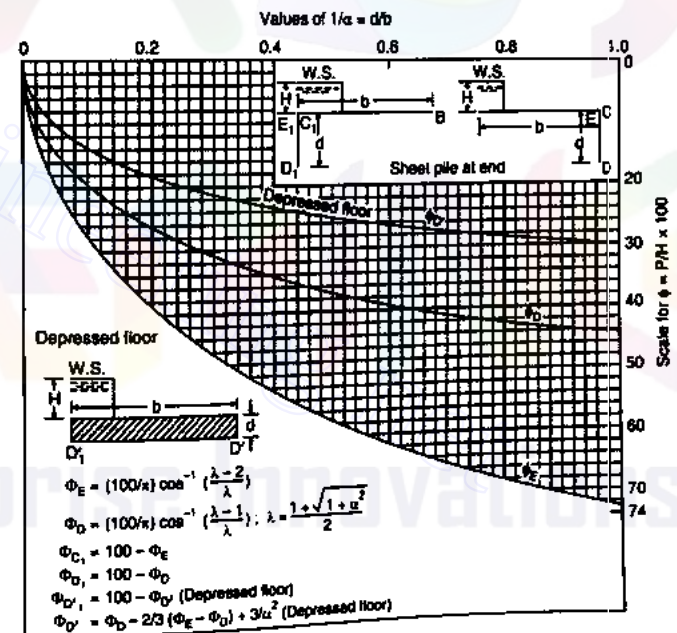


FIG. 12.14. KHOSLA'S CURVES FOR CUTOFF AT D/S END.

For illustration, let us take the case of a horizontal floor of length 20 m with a pile of length 4 m at the downstream (d/s) end.

$$\alpha = \frac{b}{d} = \frac{20}{4} = 5 \text{ and } \frac{1}{\alpha} = \frac{1}{5} = 0.2.$$

Then

Hence from the curves (Fig. 12.14) for this case, when $1/\alpha = 0.2$

we get

$$\Phi_D = 27\% \text{ or } P_D = 0.27 H$$

and

$$\Phi_E = 40\% \text{ or } P_E = 0.40 H$$

Case 3 Pile at the upstream end (Fig. 12.12 b)

If, however, the pile is provided at the upstream end (and not at the downstream end), the pressure at the key points E_1 , D_1 and C_1 are given by the following equation.

$$P_{E_1} = H \quad \dots(12.8 a)$$

$$P_{D_1} = \frac{H}{\pi} \cos^{-1} \left[\frac{1-\lambda}{\lambda} \right] \text{ or } \Phi_{D_1} = \frac{100}{\pi} \cos^{-1} \left[\frac{1-\lambda}{\lambda} \right] \quad \dots(12.8 b)$$

$$P_{C_1} = \frac{H}{\pi} \cos^{-1} \left[\frac{2-\lambda}{\lambda} \right] \text{ or } \Phi_{C_1} = \frac{100}{\pi} \cos^{-1} \left[\frac{2-\lambda}{\lambda} \right] \quad \dots(12.8 c)$$

where

$$\lambda = \frac{1 + \sqrt{1 + \alpha^2}}{2} \text{ and } \alpha = b/d$$

However, Khosla did not provide separate curves for Φ_{D_1} and Φ_{C_1} but recommended to use the principle of reversibility of flow along with curves of Φ_C and Φ_D of Fig. 12.14 (Sheet pile at the downstream end). The principle of reversibility of flow gives the following relations for Φ_{C_1} and Φ_{D_1} :

$$\Phi_{C_1} = 100 - \Phi_E \text{ and } \Phi_{D_1} = 100 - \Phi_D$$

Let us take the example of an impervious floor of 20 m length with upstream cutoff of 4 m depth, so that $\alpha = b/d = 20/4 = 5$. The pressures at key points C_1 and D_1 are calculated by first finding Φ_D and Φ_E (by assuming as if the pile is at the d/s end) and then using the above relations.

Thus for $1/\alpha = 0.2$, we have $\Phi_D = 27\%$ and $\Phi_E = 40\%$

$$\therefore \Phi_{C_1} = 100 - \Phi_E = 100 - 40 = 60\% \text{ of } H$$

and

$$\Phi_{D_1} = 100 - \Phi_D = 100 - 27 = 73\% \text{ of } H$$

Exit gradient

For the case of horizontal impervious floor with cutoff at the downstream end (Fig. 12.12 a), the exit gradient (G_E) is given by the following expression:

$$G_E = \frac{H}{d} \cdot \frac{1}{\pi \sqrt{\lambda}} \quad \dots(12.9)$$

where

$$\lambda = (1 + \sqrt{1 + \alpha^2})/2 \text{ and } \alpha = b/d$$

The proof of the above expression for G_E is given in article 4 to Appendix-1 at the end of the book. Fig. 12.15 gives the curves for variation of λ versus α and variation of the factor $1/\pi \sqrt{\lambda}$ versus α , with the help of which, exit gradient for the given values of parameters b , d and H can be found.

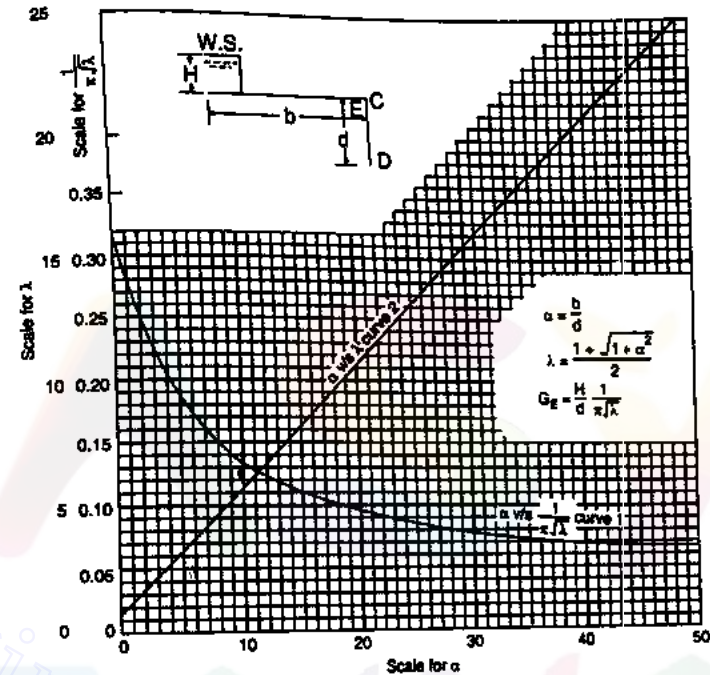


FIG. 12.15. CURVES FOR EXIT GRADIENT

For example, if $b = 20$ m, $d = 4$ m, and $H = 5$ m, we have $\alpha = b/d = 20/4 = 5$. Hence from the exit gradient curve, for $\alpha = 5$, we get $1/\pi \sqrt{\lambda} \approx 0.185$. Substituting these values in Eq. 12.9, we get

$$G_E = \frac{H}{d} \cdot \frac{1}{\pi \sqrt{\lambda}} = \frac{5}{4} \times 0.185 \approx 0.23$$

Alternatively, G_E can be computed analytically by first computing λ . Thus

$$\lambda = (1 + \sqrt{1 + \alpha^2})/2 = (1 + \sqrt{1 + 5^2})/2 = 3.0495$$

$$\therefore G_E = \frac{H}{d} \cdot \frac{1}{\pi \sqrt{\lambda}} = \frac{5}{4} \times \frac{1}{\pi \sqrt{3.0495}} = 0.228$$

Depressed floor A depressed floor is a straight horizontal apron or floor of finite thickness (or depression) d penetrating into the foundation. Khosla also solved this case empirically and gave a curve for uplift pressure Φ_D' at point D' . This curve is shown in Fig. 12.14. This curve is based on the following empirical expression:

$$\Phi_D' = \Phi_D - \frac{2}{3} (\Phi_E - \Phi_D) + 3\alpha^2 \quad \dots(12.10)$$

where α , Φ_E and Φ_D are the corresponding values for an equivalent horizontal apron of negligible thickness of length b with d/s cutoff of depth d .
For example, let us take the case of a depressed floor, having thickness $d = 4$ m and length $b = 10$ m. Then $\alpha = b/d = 10/4 = 2.5$ and $1/\alpha = 1/2.5 = 0.4$. Hence we get from curve for depressed floor, $\Phi_D = 23.5\%$.

From the principle of reversibility of floor,
 $\Phi_{D1} = 100 - \Phi_D = 100 - 23.5 = 76.5\%$

The exit gradient for the case of depressed floor is given by

$$G_E = \frac{0.84}{d} \times P_D \quad \dots(12.11)$$

where $P_D = \Phi_D \times H \quad \dots(12.11 a)$
Permissible Exit Gradient

For alluvial soils, the critical hydraulic gradient may be approximately equal to 1. The permissible hydraulic gradient can be found by adopting a suitable factor of safety of 5 to 7. The permissible exit gradient for three soils are given in Table 12.3.

TABLE 12.3. PERMISSIBLE EXIT GRADIENT

Type of soil	G_E
1. Fine sand	$\frac{1}{6}$ to $\frac{1}{7}$
2. Coarse sand	$\frac{1}{5}$ to $\frac{1}{6}$
3. Shingle	$\frac{1}{4}$ to $\frac{1}{5}$

METHOD OF INDEPENDENT VARIABLES

Fig. 12.12 shows some specific cases only. In actual practice, we may have a number of piles, (i.e. at u/s, d/s and intermediate points), and the floor also has some thickness. Khosla solved the 'actual problem' by an empirical method known as the *method of independent variables*. This consists of breaking up a complex profile into

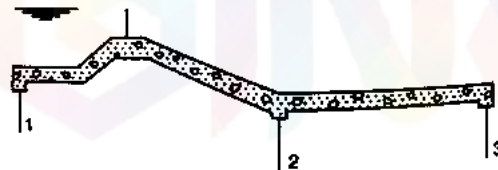


FIG. 12.12.

a number of simple profiles (such as shown in Fig. 12.12), each of which is independently amenable to mathematical treatment, and then applying the corrections due to the mutual interference of pile and due to the thickness and slope of floor.

Thus, to analyse the flow for the floor shown in Fig. 12.16 the complex profile can be broken up into the following simple profiles, and pressures at 'key points' can be obtained :

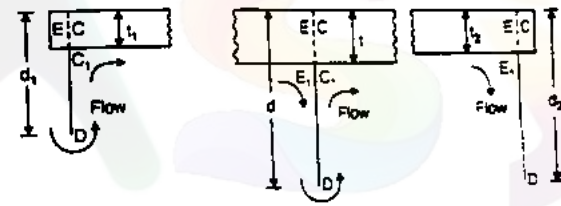
- Straight floor of negligible thickness with pile at u/s end.
- Straight floor of negligible thickness with pile at the intermediate point.

(iii) Straight floor of negligible thickness with pile at the d/s end.
The pressures obtained at the 'key points' by considering the simple profile are then corrected for the following :

- Correction for thickness of the floor
- Correction for the mutual interference of the piles.
- Correction due to the sloping floor.

(1) Correction for the thickness of the floor

Fig. 12.13 and 12.14 give pressure at key points assuming thickness of the floor to be negligibly small. Thus, the pressure at the key points E and C pertain to the level at the top of the floor, while actually the junction of the pile is at the bottom (points E_1 and C_1) of the floor. The pressures at actual points E_1 and C_1 are computed by considering linear variation of pressure between point D and the hypothetical points E and C .



(a) US pile

(b) Intermediate pile

(c) DS pile

FIG. 12.17. CORRECTION FOR THE FLOOR THICKNESS

Thus, when the pile is at the upstream end.

$$\text{Correction for } C_1 = \frac{\Phi_D - \Phi_C}{d_1} \times t_1 \text{ (additive)}$$

$$\therefore \text{Pressure at } C_1 = \Phi_{C1} = \Phi_C + \frac{\Phi_D - \Phi_C}{d_1} \times t_1$$

where t_1 = floor thickness and d_1 = depth of u/s pile

For the intermediate pile :

$$\text{Correction for } E_1 = \frac{\Phi_E - \Phi_D}{d} \times t \text{ (subtractive)}$$

$$\text{Correction for } C_1 = \frac{\Phi_D - \Phi_C}{d} \times t \text{ (additive)}$$

$$\Phi_{E1} = \Phi_E - \frac{\Phi_E - \Phi_D}{d} \times t$$

$$\Phi_{C1} = \Phi_C + \frac{\Phi_D - \Phi_C}{d} \times t$$

Similarly, when pile is at the d/s end,

gradient. Also find whether the section provided is safe against uplift and piping if it is founded on fine sand with permissible exit gradient of 1/6.

Solution

(1) First pile line

$$d = d_1 = 105.00 - 97.00 \text{ m} = 8.00 \text{ m}$$

$$b = 54 \text{ m} ; b_1 = 0.5 \text{ m}$$

$$\alpha = \frac{b}{d} = \frac{54}{8} = 6.75 ; \quad \frac{b_1}{b} = \frac{0.5}{54} \approx 0$$

∴ From the curves, for $\frac{b_1}{b} = 0$ and $\alpha = 6.75$

$$\phi_{C1} = 66.5\% ; \quad \phi_{D1} = 76\% ; \quad \phi_{E1} = 100\%$$

Correction for floor thickness,

$$t = 105.00 - 104.00 = 1.00 \text{ m}$$

$$\therefore \text{Correction} = \frac{\phi_{D1} - \phi_{C1}}{d_1} t = \frac{(76 - 66.5)}{8} \times 1 \approx 1.2 (+)$$

Correction for interference of pile No.2

$$C = 19 \sqrt{\frac{D}{b'}} \left(\frac{d+D}{b} \right)$$

$$\text{Here } d = 104.00 - 97.00 = 7 \text{ m} ; b' = 16 \text{ m} ; b = 54 \text{ m} ; D = 104.00 - 97.00 = 7 \text{ m}$$

$$C = 19 \sqrt{\frac{7}{16}} \left(\frac{7.00 + 7.00}{54} \right) = 3.3\% (+)$$

∴ Corrected pressures are $\phi_{C1} = 66.5 + 1.2 + 3.3 = 71\%$ and $\phi_{D1} = 76\%$.

(2) Intermediate pile line

$$d = d_2 = 105.00 - 97.00 = 8.00 \text{ m} ; b = 54 \text{ m} ; b_1 = 16.5 \text{ m}$$

$$\alpha = \frac{b}{d} = \frac{54}{8} = 6.75 ; \quad \frac{b_1}{b} = \frac{16.5}{54} = 0.306$$

From curves, for $\alpha = 6.75$ and $\frac{b_1}{b} = 0.306$, we get

$$\phi_{C2} = 52\% ; \quad \phi_{E2} = 72.5\% ; \quad \phi_{D2} = 61.5\%$$

Correction for floor thickness

$$\text{Correction for } \phi_{C2} = \frac{61.5 - 52}{8} \times (105.00 - 104.0) \approx 1.2 (+)$$

$$\text{Correction for } \phi_{E2} = \frac{72.5 - 61.5}{8} \times 1 \approx 1.4 (-)$$

Correction for interference of pile No.1

Since pile No. 1 and 2 are of the same length, and are fixed at the same level, the correction for ϕ_{E2} , due to interference of pile No. 1 is $= 3.3\% (-)$

DIVERSION HEADWORKS

Correction for interference of pile No. 3.

$$d = 104.0 - 97.0 = 7.0 \text{ m} ; \quad D = 104.0 - 95.0 = 9.0 \text{ m} ; b' = 37 \text{ m}$$

$$\text{Correction to } \phi_{C2} = 19 \sqrt{\frac{9.0}{37}} \times \left(\frac{7+9}{54} \right) \approx 2.7\% (+)$$

Correction due to slope of 4 : 1

Correction (from Table 12.4) $= 3.3\% (-ve)$

Correction is negative due to upward slope in the direction of flow.

Horizontal length of the slope $= 4 \text{ m}$

$$\therefore \text{Actual correction, to be applied to } \phi_{C2} = 3.3 \times \frac{4}{37} \approx 0.4\% (-ve)$$

Hence the corrected pressures are

$$\phi_{E2} = 72.5 - 1.4 - 3.3 = 67.8\%$$

$$\phi_{C2} = 52 + 1.2 + 2.7 - 0.4 = 55.5\%$$

(3) Pile No. 3 at the d/s end

$$d = d_3 = 103.5 - 95 = 8.5 \text{ m}$$

$$\therefore \frac{1}{\alpha} = \frac{d}{b} = \frac{8.5}{54} \approx 0.158$$

Hence, for curves for pile at d/s end, we get

$$\phi_{E3} = 35\% ; \quad \phi_{D3} = 24.2\%$$

$$\text{Correction for thickness} = \frac{35 - 24.2}{8.5} (103.5 - 102) = 1.9\% (-ve)$$

Correction for interference of pile No. 2

$$d = 102.0 - 95 = 7 \text{ m} ; D = 102.0 - 97 = 5 \text{ m}$$

$$\therefore \text{Correction for } \phi_{E3} = 19 \sqrt{\frac{5}{37}} \left(\frac{5+7}{52} \right) \approx 1.5\% (-)$$

Hence the corrected pressures are :

$$\phi_{E3} = 35 - 1.9 - 1.5 = 31.6\%$$

The percentage pressures and the actual pressure heads at the key points are tabulated below :

TABLE 12.5. MAXIMUM PERCOLATION HEAD $= 104.5 - 103.5 = 1 \text{ m}$

Point	% Pressure (p)	Pressure head (P) m	Point	% Pressure (p)	Pressure head (P) m
C ₁	71	3.55	C ₂	55.5	2.78
E ₂	67.8	3.39	E ₃	31.6	1.58

The pressures at intermediate points may be computed by linear variation.

(4) Check for thickness of floor

Let us check the floor thickness at the three points A, B and C (Fig. 12.19).

$$\text{Pressure head at A} = P_C - \left(\frac{P_C - P_B}{37} \times 6.5 \right) = 2.78 - \frac{2.78 - 1.58}{37} \times 6.5 \approx 2.57 \text{ m}$$

$$\text{Pressure head at B} = 2.78 - \frac{2.78 - 1.58}{37} \times 24.0 = 2 \text{ m}$$

$$\text{Pressure head at C} = 2.78 - \frac{2.78 - 1.58}{37} \times 30 = 1.81 \text{ m}$$

Taking the specific gravity (ρ) of the concrete = 2.24, we have

$$\text{Thickness required at A} = \frac{P_A}{\rho - 1} = \frac{2.57}{1.24} = 2.07 \text{ m.}$$

Actual thickness provided at A = 107.0 - 105 = 2 m

Hence increase the thickness at A to 2.1 m

$$\text{Thickness required at B} = \frac{P_B}{\rho - 1} = \frac{2.0}{1.24} = 1.61 \text{ m}$$

Actual thickness provided at B = 103.5 - 101.5 = 2 m

$$\text{Thickness required at C} = \frac{P_C}{\rho - 1} = \frac{1.81}{1.24} = 1.46 \text{ m}$$

Actual thickness provided = 103.5 - 102.0 = 1.5 m

Hence, the floor thicknesses at B and C are adequate.

(4) Exit gradient

$$\text{Seepage head} = H = 108.5 - 103.5 = 5 \text{ m}$$

$$\text{Depth of cut off} = d = 103.5 - 95 = 8.5 \text{ m}$$

$$\alpha = \frac{b}{d} = \frac{54}{8.5} = 6.35$$

$$\text{Hence, from exit gradient curve, for } \alpha = 6.35, \frac{1}{\pi \sqrt{\lambda}} \approx 0.165$$

$$\text{Alternatively, } \lambda = (1 + \sqrt{1 + \alpha^2})/2 = (1 + \sqrt{1 + (6.35)^2})/2 = 3.716$$

$$\text{and } \frac{1}{\pi \sqrt{\lambda}} = \frac{1}{\pi \sqrt{3.716}} = 0.165$$

$$G_E = \frac{H}{d} \times \frac{1}{\pi \sqrt{\lambda}} = \frac{5}{8.5} \times 0.165 = 0.097 = \frac{1}{10.3}$$

Permissible exit gradient = 1/6. Hence, safe.

12.9. ELEMENTS OF DESIGN FOR SURFACE FLOW

We will consider the following elements of design for surface flow :

1. Design flood discharge
2. Permissible intensity of discharge
3. Afflux
4. Pond level
5. Stable river width : Waterway
6. Protection against scour due to surface flow
7. Hydraulic jump computations and protection against it.

The last item (hydraulic jump computations) has been discussed separately in § 12.11.

1. Design flood discharge

The design discharge adopted for the design of weirs or barrages is usually the observed (or recorded) or computed maximum discharge. Table 12.6 gives the criteria for fixing maximum flood for this purpose. It should be clearly noted that passage of *smaller floods* than the design flood results in shoal formation and meandering of the river channel while passage of *higher floods* than the design flood result in encroachment upon free board of river training works.

TABLE 12.6. CRITERIA FOR FIXING MAXIMUM FLOOD DISCHARGE

Agency	Criteria
1. Central Water Power Commission (CWPC) of India	Standard Project Flood (SPF) or a 100 year flood whichever is higher
2. Indian Standard (IS 6968-1989)	50 to 100 years frequency flood; However, for fixing free board, 500 year frequency flood or Standard Project Flood.
3. Khosla Committee for Engineers	Maximum recorded flood for a period of greater than 50 years

In the above Table, SPF is the flood resulting from most severe combination of hydrological and meteorological factors that are considered reasonably characteristic of the region and is computed by using the Standard Project Storm (SPS). For *pick-up weirs*, a flood of 50-100 years frequency is adopted according to its importance and local conditions.

Experience shows that passage of floods higher than the design discharge *through a barrage* washes off shoals to make available full water way than that provided with a waterway for the maximum flood. Hence, for fixing waterway, a flood of more frequent recurrence interval than the maximum flood is adopted. On the contrary, a flood of long return period is adopted for the structural safety and free board of river training works.

2. Permissible intensity of discharge

Safe permissible intensity of discharge, to determine the length of waterway, depends upon (i) nature of river bed material, and (ii) allowable afflux depending on topography and economy. The average intensity of discharge per metre of water way between abutments is limited to a safe value so as to prevent scouring of river bed material. In boulder stage, the coarse bed material can withstand higher velocities and higher afflux does not create problem due to inundation of land; hence higher discharge intensity may be allowed. In contrast to this, lower discharge intensity is adopted in the alluvial reach where slopes are flat and river bed material is alluvial.

It should be clearly noted that a higher discharge intensity reduces the length of water way, thus resulting in economy, but has the defect of increasing the cost of protection works against greater scouring tendency and deepening to the downstream floor of the barrage, resulting in increased cost due to dewatering and increased thickness of floor. Table 12.7 gives the data about average discharge intensity provided in some prominent barrages and weirs in India.

TABLE 12.7. DISCHARGE INTENSITY THROUGH SOME PROMINENT BARRAGES
IN INDIA

Weir/barrage	Location	Discharge intensity q (cumecs/m)
	Rajasthan	22
Kalabagh	U.P.	28
Sarda	Bihar	32
Gandak	Punjab	34
Nangal	U.P.	40
Kosi barrage and Rishikesh barrage	U.P.	67
Veerbhadra barrage		

\$ Afflux

Afflux is rise of water level on the upstream of the structure, after the construction of the weir/barrage. During floods, the H.F.L. on the upstream of weir/barrage is higher than that at the downstream. Afflux affects the area of submergence and top levels of the marginal banks and guide banks. If the afflux is large, the length of weir will be small because of high discharge intensity over the crest. However, selection of higher value of afflux will increase the cost of river training works and also increase the cost of protection works to the u/s and d/s sides because of increased scour depth. Hence a *safe value* of afflux is adopted. In alluvial sands of Punjab and Haryana, *safe afflux* usually adopted is 1 m while for U.P. it may be upto 2.25 m. IS 6933-1973 has recommended an afflux of 1 m in the case of barrages/weirs on alluvial rivers in upper and middle reaches of the river and 0.3 m in lower river reaches having flat gradients.

4. Pond level

Pond level is the water level in the pond created on the upstream side of the weir, when a part (or full) of the river discharge is being diverted to the canal through the canal head regulator. Pond level is required to be maintained throughout the year, in order to feed the full supply discharge to the offtaking canal. Its value is given by the relation :

Pond level = Full supply level (F.S.L.) of the canal + Head loss thro' regulator.

The head loss through regulator usually ranges from 1.0 to 1.5 m. Pond level is usually fixed considering water level in the winter or that during the low discharge period. Higher the pond level, higher will be the afflux.

5. Stable river width : Recommended waterway
The length of water way (km)

The length of water way (W) provided over the weir, to pass the maximum flood discharge should be equal to the stable river width for that discharge. This is taken equal to Lacey's wetted perimeter (P) given by the generalised equation :

$$W \pm P = C \sqrt{Q}$$

$W \pm P = C \sqrt{Q}$ given by the generalised equation : ...(12.13 a)

where C is the constant in Lacey's equation, the value of which may vary between 3.6 to 6.15, depending upon the discharge and the nature of the bed material. For regime conditions, the best fit value of C may be taken equal to 4.83.

With the restricted water way given by the above equation, the shoaling upstream is mostly eliminated and a nearly straight and stable approach to the weir/barrage is secured.

DIVERSION HEADWORKS

Recommendations of IS : 6966-1989

The recommendations given by IS : 6966-1989 for waterway are as under :

(i) In deep and confined rivers with stable banks, the overall water-way between abutments including the thickness of piers is approximately equal to the actual river width at design flood.

(ii) For shallow and meandering alluvial rivers, the minimum stable width, as under is calculated from Lacey's formula :

$$W = 4.83 \sqrt{Q} \quad \dots(12.13 b)$$

The regime conditions as envisaged by Lacey can occur only at one discharge, frequency of which will vary for different climate and geological region. The overall width of the weir/barrage actually provided may be more or less than that computed theoretically, using Lacey's regime equation for wetted perimeter (Eq. 12.13 b). The ratio of the two is known as the *looseness factor*, the values of which for recently constructed weirs/barrages in India are as under (Table 12.8).

TABLE 12.8. VALUES OF LOOSENESS FACTOR

River stage	Looseness factor
1. Boulder	1.0 to 0.45
2. Alluvial	1.0 to 1.45

It is observed that in weirs constructed with looseness factor greater than 1.5, heavy shoals are invariably formed in the upstream with top upto crest of the weir. Due to these formations, it becomes difficult to feed the canal due to shifting of the channel, caused by shoals. However, restriction of water way (for looseness factor less than 1) removes shoal formation immediately upstream of weir, though at the cost of deep scour formed due to excessive velocities. On the contrary, provision of wider water way (looseness factor greater than 1.0) gives rise to low scour, at the cost of increased cost of construction of weir/barrage and formation of shoals at the u/s side. For weirs and barrages on rivers flowing in alluvial soils, normal practices is to provide a clear water way length of 1.1 to 1.25 times Lacey's regime perimeter obtained for *dominant discharge* (i.e. discharge corresponding to 60 to 75% of observed maximum flood or 1 in 100 years flood in wide and shallow rivers and bankful discharge in incised rivers). More frequent floods higher than the dominant discharge also help in flushing the shoals.

6. Protection against scour due to surface flow

(a) **Scour depth**

(a) **Scour depth**
Due to surface flow, specially during high flood discharge, scouring takes place at upstream and down stream sides of the structure, also around guide banks and other related protection works. The scour depth (R) likely to occur in river bed consisting

The maximum depth of scour depends upon type of reach as indicated in Table 12.10, as recommended by Lacey.

TABLE 12.9. SCOUR DEPTH

S.N.	Type of river bed material	Normal scour depth
		To be based on actual observations
1.	Cohesive soils (clay etc.)	$R = 0.473 \left(\frac{Q}{f} \right)^{1/3}$
2.	Non-cohesive soils, when looseness factor is more than 1	
3.	Non-cohesive soils, when looseness factor is less than 1	$R = 1.35 \left(\frac{Q^2}{f} \right)^{1/3}$ (Lacey's formula)

TABLE 12.10. RECOMMENDED MAXIMUM SCOUR DEPTH

S.N.	Class	Type of Reach	Maximum scour depth
1.	A	Straight reach	1.25 R
2.	B	Moderate reach	1.50 R
3.	C	Severe bend	1.70 R
4.	D	Right angled bend	2.00 R

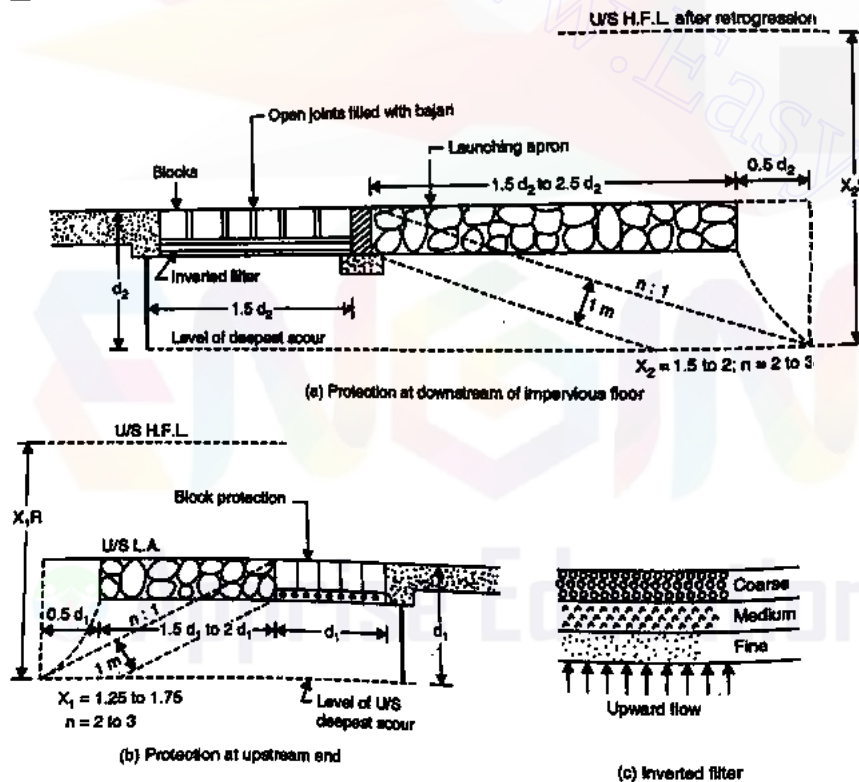


FIG. 12.20. PROVISION AGAINST SCOUR AT D/S AND U/S SIDES

The maximum scour depth indicated in Table 12.10 is measured below the high flood level (H.F.L.)

(b) *Provisions against scour at upstream and down stream sides of pucca or impervious floor of weir/barrage*

As seen earlier under the design of floor for under-seepage (or subsurface floor), impervious floor of certain length is provided, both at the upstream and downstream side of the main structure. The thickness of the impervious floor varies along its length. However, in order that scouring does not damage this impervious floor, resulting in the collapse of the structure, the following provisions are made, as indicated in Fig. 12.20 (a) and (b).

- Cutoffs of certain depths, both at the up stream and downstream ends of pucca floor
- Provision of block protection at the upstream of the pucca floor.
- Provision of inverted filter topped by blocks with open joints at the downstream of pucca floor.
- Provision of pervious aprons, commonly known as *launching aprons*, both at the upstream and downstream of the block protections.

1. Upstream and downstream cutoffs

Cutoffs, either in the form of concrete (or masonry) walls or in the form of sheet-piles, are invariably provided both at the u/s as well as d/s ends of pucca (or impervious) floor. These are taken upto the level of *deepest scour*. The level of deepest scour is taken equal to $X_2 R$ below the d/s H.F.L. (after retrogression) where the value of X_2 is taken between 1.5 to 2. For sloping glacis weir, where energy dissipation is quite efficient due to the formation of hydraulic jump, X_1 is taken equal to 1.5, while X_2 is taken equal to 2 for vertical drop weirs. Similarly, at the upstream side, the level of deepest scour is taken equal to $X_1 R$ below the u/s H.F.L., where the value of X_1 varies between 1.25 to 1.75, the more commonly adopted value being 1.5.

It should be noted that a certain depth of d/s pile is also required to control the exit gradient. Hence the greater of the two depth (i.e. one based on deepest scour level and the other based on exit gradient) should be provided.

2. Block protection at the upstream end of impervious floor

Immediately to the upstream side of u/s cutoff, block protection of length d_1 is provided. The block protection consists of 0.6 to 0.9 m thick stone or concrete blocks laid on 0.3 to 0.6 m thick loosely packed gravel so that the blocks do not settle in the softened soil below. The purpose of the upstream block protections is to protect the u/s cutoff against twisting / bending due to the u/s launching apron, when the launching apron launches into the scour holes.

3. Inverted filter at the downstream of impervious floor

In order to relieve the uplift pressure due to under seepage water, inverted filter is provided *immediately* to the downstream of the pucca floor. It should be noted that the direction of flow of seeping water is upwards. In order that this upward flow of water does not carry fines with it, the filter is inverted, with coarser material at

the top and finer material (slightly coarser than the base material) at the bottom, as shown in Fig. 12.20 (c). In order that thick filter is not washed away during floods, heavy stones or concrete blocks, with open joints, are placed on the top of the inverted filter. The open joints are filled with bajri (clean sand) so that upward flowing water can easily come out, thus dissipating the entire uplift pressure. The length of the filter is taken equal to $1.5 d_2$. The filter unit may be protected at the end by toe wall extending upto 0.5 m below the bottom of the filter.

4. Launching aprons at downstream and upstream ends

At the end of d/s filter or u/s block protection, launching aprons of loose boulder or stones (of size +300 mm and weighting + 40 kg) or precast cement concrete blocks are provided to protect against the scour holes formed during high floods. The blocks are provided to protect against the scour holes formed during high floods. The length of down stream launching apron is kept between $1.5 d_2$ to $2.5 d_2$, the higher value being adopted for flatter launched slopes. The slope of the launched apron depend on the grade and size of river bed material, as indicated in Table below.

TABLE 12.11. SLOPES OF LAUNCHED APRONS

S.N.	River bed material	Slope (n)
1.	River in alluvium sandy reach	Not steeper than 2:1
2.	River in very fine sand and sixty bed	Not flatter than 3:1
3.	River in boulder reach	Upto 1.5:1

Generally, for sloping glacis weirs, where energy dissipation is more efficient due to formation of hydraulic jump, the d/s launching apron length is taken equal to $1.5 d_2$ and its launched slope is taken equal to 2:1. However for vertical drop weirs, the length of d/s launching apron is taken equal to $2.5 d_2$ and its launched slope is taken equal to 3:1.

Similarly, the length of u/s launching apron is kept equal to $1.5 d_1$ to $2 d_1$, the higher value being adopted for flatter launched slope. Generally, the length of u/s launching apron of sloping glacis weir is taken equal to $1.5 d_1$, with a steeper launched slope of 2:1. For vertical drop weirs, this length is taken equal to $2 d_1$, with launched slope of 3:1.

According to Spring, the quantity of stone in launching apron should be sufficient to provide about 1 m thick cover over the launched slope of $n:1$ in the launched position.

Thus, if $n = 3$ and length of apron is $2.5 d_2$, the length of d/s apron in the launched position will be equal to $\sqrt{10} d_2$ and the volume of stones in launched position will be equal to $(\sqrt{10} d_2 \times 1) \text{ m}^3$ per metre length. Hence the thickness of apron in the horizontal position will be equal to $\sqrt{10} d_2 / 2.5 d_2 \approx 1.27 \text{ m}$ in horizontal position. Similarly, if the length of d/s launching apron is $1.5 d_2$ and $n = 2$, the length of d/s apron in launched position will be equal to $\sqrt{5} d_2$ and the volume of stones in launched position will be equal to $(\sqrt{5} d_2 \times 1) \text{ m}^3$ per metre length. Hence the thickness of the apron in the horizontal position will be equal to $(\sqrt{5} d_2) / 1.5 d_2 = 1.49 \text{ m}$ in horizontal position. Similar computations for thickness can be made for upstream launching apron.

12.10. DESIGN OF VERTICAL DROP WEIR

The complete design of a vertical drop weir consists of the design calculations for the following :

1. Hydraulic calculations for fixing various elevations.
2. Design of weir well.
3. Design of impervious aprons.
4. Design of inverted filter and d/s talus.

1. HYDRAULIC CALCULATIONS

Before starting the design calculations, the following data must be known :

- (a) Maximum flood discharge (Q)
- (b) H.F.L. before construction of weir
- (c) D/S bed level
- (d) F.S.L. of canal taking off from the river
- (e) Allowable afflux
- (f) Lacey's silt factor f

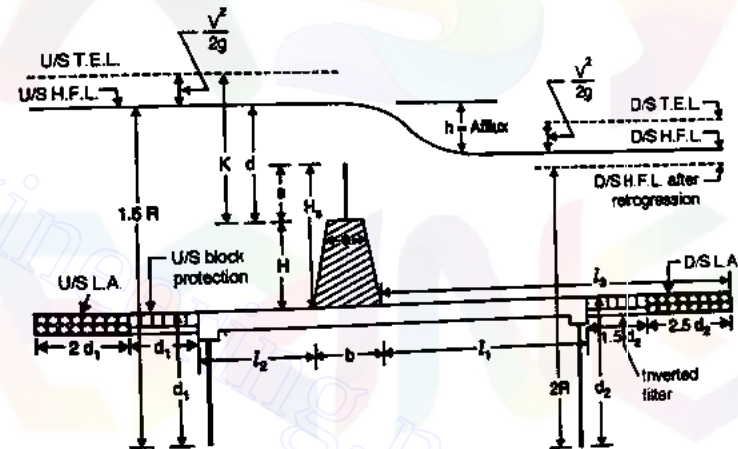


FIG. 12.21. VERTICAL DROP WEIR

Step (i) : The length of water way (L) is calculated from Lacey's regime formula for wetted perimeter (P)

$$P = 4.75 Q^{1/2} \quad \dots(12.13)$$

The length of water way (L) is usually taken between P and $1.2 P$. However, in vertical drop weirs, L is taken equal to P . Hence

$$L = 4.75 Q^{1/2}$$

where L = length of water way in metres
 Q = discharge in cumecs.

Step (ii) : Knowing L , the discharge q per unit width of river is calculated from relation $q = \frac{Q}{L}$... (12.14)

Step (iii) : The regime scour depth is calculated from Lacey's formula,

$$R = 1.35 \frac{q^{2/3}}{f} \quad \dots (12.15)$$

where f = Lacey's silt factor.

Step (iv) : The regime velocity and velocity head are calculated from the expressions:

$$V = \frac{q}{R} \text{ m/sec} \quad \text{and} \quad \text{Velocity head} = \frac{V^2}{2g}$$

Step (v) : Water levels and total energy levels are calculated as follows:

$$\text{Level of d/s T.E.L.} = (\text{H.F.L. before construction}) + \frac{V^2}{2g}$$

$$\text{Elev. of u/s T.E.L.} = \text{d/s T.E.L.} + \text{Afflux}$$

$$\text{Elev. of u/s H.F.L.} = \text{u/s T.E.L.} - \frac{V^2}{2g}$$

Step (vi) : Discharge over the crest of the weir is given by

$$q = 1.70 K^{3/2}$$

$$K = \left(\frac{q}{1.70} \right)^{2/3}$$

where K = total energy head measured above the weir crest.

Crest level = u/s T.E.L. - K

Step (vii) : Pond level = Level of top of gates

= F.S.L. of canal + Head loss through regulator.

Head loss through regulator may be taken as $\frac{1}{2}$ to 1 m.

Height of shutters = s = Level of top of gates - Crest level.

Step (viii) : Protection against scour :

Level of bottom of u/s pile = u/s H.F.L. - 1.5 R

Level of bottom of d/s pile = H.F.L. after retrogradation - 2 R .

2. DESIGN OF WEIR WELL

The design of a vertical drop weir wall is similar to that of the overflow section of gravity dam. In the case of a dam, however, the condition of maximum stress is when the water level above the base is maximum. In the case of weir, greater the discharge the condition of maximum water level above the head and tail water surfaces and the condition of maximum pressure on the work generally connotes that of minimum head on the work. The condition of maximum pressure on the work is that in which the head water is at the level of the crest of the weir or weir shutters and no water is passing over the weir. However, the condition of maximum stress on the weir may be different in certain cases. Hence, it is necessary to check the stability under three states:

State 1. When the upstream water or head water is at crest level or at the level of crest shutter, if any (Fig.12.22(a)) and there is no flow.

State 2. When the water is passing over the weir crest and the weir is submerged (Fig.12.22(b)).

State 3. When the water is passing over the weir crest and the weir is discharging with a clear overfall (Fig.12.22(c)).

Preliminary section of weir wall

The top width a and the bottom width b of the weir wall are selected on the following considerations. The stability of the section is then checked under the above three states of flow.

(i) Top width (a)

$$a = \frac{d}{\sqrt{p}} \quad \dots (12.16)$$

The above formula is on the considerations of the elementary profile (stress criterion) of the weir.

If we consider the width a of a horizontal joint at crest level to ensure safety against sliding, we get

$$a = \frac{d}{\mu p}$$

Assuming $\mu = 2/3$ as the safe value, we get

$$a = \frac{3d}{2p} \quad \dots (12.17)$$

The practical considerations for the fixation of a are :

1. The height of the weir.
2. Nature of the drift carried by the stream.
3. The width required for the crest shutters or for the traffic of any kind.

For crest shutters of height s ,

$$a = s + 1 \text{ meters} \quad \dots (12.18)$$

The top width should be greater of the three values of a given by Eqs. 12.16, 12.17 and 12.18.

(ii) Bottom width (b)

The bottom width b can be calculated by equating the overturning moment to the resisting moments taken at the outer third point. To find the moments, we shall consider all the three states mentioned above.

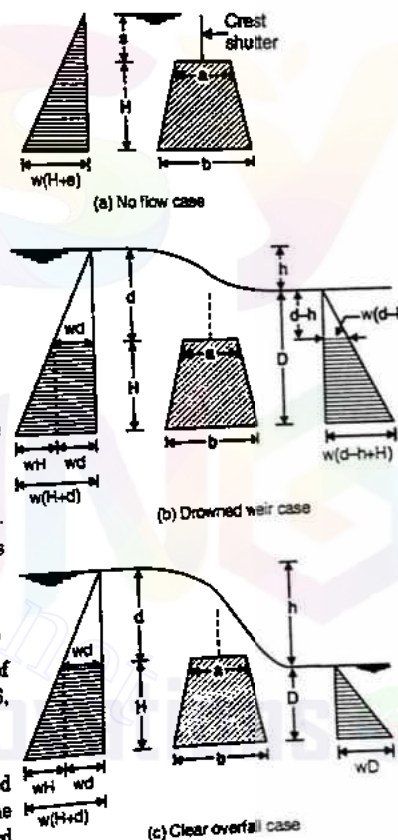


FIG. 12.22. CONDITIONS OF FLOW OVER VERTICAL DROP WEIR.

(a) **State I: Head water at the crest level or at the level of the crest shutters.**

In this state, the head on the weir, i.e., the difference of levels of the head and tail water is a maximum. Fig. 12.22(a) shows the pressure diagram on the w/s face, assuming no water to the d/s side.

The overturning moment is given by

$$M_0 = \frac{w(H+s)^3}{6} \quad \dots(12.19)$$

The resisting moment about the outer third point of the base fulfilling the 'middle third rule' is given by

$$M_r = \frac{w}{12} \left[\{(\rho + 1.5)H + 2.5s\}b^2 + a(\rho H - H - s)b - \frac{1}{2}a^2(H + 3s) \right] \quad \dots(12.20)$$

The above expression is valid when both upstream and downstream slopes are kept equal. If, however, the upstream face is kept vertical, the moment of resistance is increased and is given by

$$M_r = \frac{wHp}{6} (b^2 + ab - a^2) \quad \dots[12.20(a)]$$

Equating the overturning moment to the moment of resistance, the base width b can be determined for State I.

(b) **State II: When the water is passing over the weir crest and the weir is submerged.**

Fig. 12.22(b) shows the pressure distribution diagrams for this case. The overturning moment is given by

$$M_0 = \left[\frac{wH^3}{6} + \frac{wdH^2}{2} \right] - \left[\frac{wH^3}{6} - \frac{w(d-h)H^2}{2} \right] = \frac{whH^2}{2} \quad \dots(12.21)$$

For maximum value of M_0 , h is taken corresponding to the case when the weir is just submerged.

The moment of resistance, with tail water at weir crest level, considering full flotation and fulfilling the 'middle third rule' is given by

$$M_r = \frac{wH(\rho - 1)}{12} (b^2 + ab) \quad \dots(12.22)$$

(for both faces having the same slope)

If the upstream face is vertical

$$M_r = \frac{wH(\rho - 1)}{12} (b^2 + ab - a^2) \quad \dots[12.22(a)]$$

Equating the overturning moment to the moment of resistance, b can be determined.

(c) **State III: When the water is passing over the weir crest and the weir is discharging with free overfall case.**

Fig. 12.22 (c) shows the pressure diagram, from which the overturning moment is given by

$$M_0 = \frac{wH^3}{6} + \frac{wdH^2}{2} - \frac{wD^3}{6} = \frac{w}{6} [H^3 + 3dH^2 - D^3] \quad \dots(12.23)$$

Here there are two variables: d and D . Relation between the two must be known to find the maximum overturning moment. For finding a suitable relation between d and D , river gauging etc. are necessary for the clear overfall case.

When the weir extends the whole width of the river and the latter is of considerable width in comparison to its depth, say width not less than 30 times the depth, the value of d is roughly proportional to that of D , i.e.

$$d = kD$$

where k is constant, and can be known if the river discharge for any one depth D is known. Substituting in Eq. 12.23, we get

$$M_0 = \frac{w}{6} [H^3 + 3kDH^2 - D^3]$$

For maxima $\frac{dM_0}{dD} = \frac{w}{6} (3kH^2 - 3D^2) = 0.$

$$D = H\sqrt{k} \quad \dots(12.24)$$

Hence, the maximum moment is given by

$$M_0 = \frac{w}{6} [H^3 + 3k^{3/2}H^3 - k^{3/2}H^3]$$

$$M_0 = \frac{wH^3}{6} (1 + 2k^{3/2}) \quad \dots(12.25)$$

or

The above expression is after S.V.K. Pillai.

3. DESIGN OF IMPERVIOUS APRON

For the under-seepage, the worst condition will be when the water at the u/s is upto the crest of the shutters, and there is no tail water. Thus the percolation head will be H_s (Fig. 12.21).

If the floor of the weir is designed on Bligh's creep theory, the total creep length is given by Eq. 12.2, i.e.

$$L = CH_s$$

If, however, the floor is to be designed on Khosla's theory, the horizontal length b of the impervious apron is found by the consideration of the permissible exit gradient

$$G_x = \frac{H_s}{d} \frac{1}{\pi \sqrt{\lambda}} \quad \text{From Eq. 12.9.}$$

In this equation G_x , H_s and d are known. Hence, $\frac{1}{\pi \sqrt{\lambda}}$ is calculated. From the exit gradient curve knowing $\frac{1}{\pi \sqrt{\lambda}}$ the corresponding value of α is known. But $\alpha = b/d$. Knowing α and d , b can be determined.

Out of the total horizontal impervious length b (or the total creep length L), the lengths l_1 , l_2 , and l_3 of the floor as indicated in Fig. 12.21 are fixed on the basis of Bligh's recommendations given below:

1. **Downstream Impervious Apron (l_1)** For weir crest with shutters

$$l_1 = 2.21C \sqrt{\frac{H_s}{13}} \text{ in metric units} \quad \dots(12.26)$$

For weir with no crest shutters,

$$l_1 = 2.21C \sqrt{\frac{H}{10}} \text{ in metric units} \quad \dots(12.27)$$

2. Upstream Impervious Apron

$$l_2 = L - l_1 - (b + 2d_1 + 2d_2)$$

3. Total Length of d/s Apron

The total length l_3 of the d/s impervious floor and the d/s pervious aprons is given by the following :

$$\text{For weir with crest shutters} \\ l_3 = 18C \sqrt{\frac{H_s}{13} \times \frac{q}{75}} \text{ in metric units} \quad \dots(12.28)$$

$$\text{For weir with no crest shutters,} \\ l_3 = 18C \sqrt{\frac{H}{10} \times \frac{q}{75}} \text{ in metric units.} \quad \dots(12.29)$$

4. Design of Inverted Filter and Launching Aprons

An inverted filter is provided immediately at the end of d/s impervious apron to relieve the pressure. The inverted filter consists of 90 to 120 cm thick stones placed on graded filter 50 to 75 cm thick. If d_2 is the depth of the pile below the d/s bed, the recommended length of the inverted filter is $1.5 d_2$, as shown in Fig.12.23.

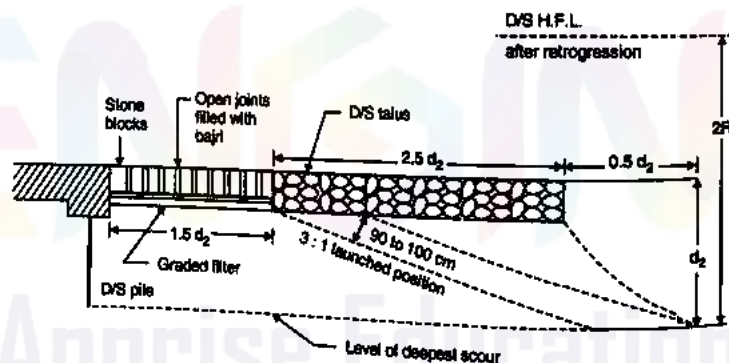


FIG. 12.23. INVERTED FILTER AND LAUNCHING APRON AT D/S END.

After the inverted filter, a length of $2.5 d_2$ of launching apron or talus is provided. The function of the launching apron is to protect the d/s pile from the scour holes progressing in the w/s direction. When the scour holes are formed, the stones of the talus settle in the holes and cover them. Spring recommends that in the launched position over a slope of 3 : 1 the apron must have a thickness 3 ft (90 to 100 cm).

Knowing the inclined length and the thickness, the total volume of stones can be calculated, and hence the thickness in the horizontal position of length $2.5 d_2$ can be easily determined.

A similar launching apron is provided to the w/s side. If d_1 is the depth of the w/s pile, the horizontal length of the launching apron is kept equal to $2d_1$. As in the previous case, the thickness in the launched slope of 3 : 1 is kept equal to protection of a length d_1 is provided. The block protection consists of 0.9 to 1 m thick stones or precast blocks placed on 0.3 to 0.6 m thick loosely packed gravel.

DESIGN EXAMPLE OF A VERTICAL DROP WEIR

Example 12.3. Design a vertical drop weir on Bligh's theory for the following site conditions:

- (a) Maximum flood discharge = 2800 cumecs
- (b) H.F.L. before construction = 285.0 m
- (c) Minimum water level = d/s bed level = 278.0 m
- (d) F.S.L. of canal = 284.0 m
- (e) Allowable afflux = 1 m
- (f) Coefficient of creep = 12
- (g) Permissible exit gradient = 1/6

Assume any other data not given.

Test the floor of the above weir by Khosla's theory and make changes if found necessary.

Solution (Refer Fig.12.21 for symbols used)

(A) Hydraulic Calculations

Step 1. $Q = 2800$ cumecs
 $L = 4.75 Q^{1/2} = 4.75 (2800)^{1/2}$ metres = 250 m
 $q = \frac{Q}{L} = \frac{2800}{250} = 11.2$ cumecs/m.

Step 2. Regime scour depth : Take $f = 1$
 $R = 1.35 \left(\frac{q^2}{f} \right)^{1/3} = 1.35 \left[\frac{(11.2)^2}{1} \right]^{1/3} = 6.75$ m

Regime velocity = $\frac{q}{R} = \frac{11.2}{6.75} = 1.66$ m/second

Velocity head = $\frac{V^2}{2g} = \frac{(1.66)^2}{2 \times 9.81} = 0.14$ m

Step 3. Level of d/s T.E.L. = H.F.L. before cons. + $\frac{V^2}{2g} = 285.00 + 0.14 = 285.14$ m

Afflux = 1 m

Level of w/s T.E.L. = d/s T.E.L. + Afflux = 285.14 + 1 = 286.14 m

IRRIGATION AND WATER POWER ENGINEERING

u/s H.F.L. = u/s T.E.L. - $\frac{V^2}{2g} = 286.14 - 0.14 = 286.00$ m
 Actual d/s H.F.L., allowing 0.5 m for retrogradation = $286.00 - 0.5 = 284.50$ m.

Step 4. $q = 1.70 K^{2/3}$
 $K = \left(\frac{q}{1.70}\right)^{3/2} = \left(\frac{11.2}{1.70}\right)^{3/2} = 3.56$ m.

∴ Crest level = u/s T.E.L. - $K = 286.14 - 3.56 = 282.58$ m.
 Pond level = Level of top of gates
 = F.S.L. in canal + Head loss through regulator

= $284.00 + 0.50$ (say) = 284.50 m
 Height of shutter = s = Level of top of gates - Crest level
 = $284.50 - 282.58 = 1.92$ m

Step 6. R.L. of bottom of u/s pile
 = u/s H.F.L. - $1.5 R = 286 - (6.75 \times 1.5) = 275.88$ m.

Keep the R.L. at 276.0 m
 Depth of u/s cut off = $278.0 - 276.0 = 2$ m.
 Hence, provide concrete cutoff of 2 m depth.
 R.L. of bottom of d/s pile

= d/s H.F.L. after retrogradation - $2R = 284.50 - (6.75 \times 2) = 271.0$ m.

Step 7. Seepage head of water
 H_s = R.L. of gates crest - R.L. of bed = $284.50 - 278.0 = 6.50$ m.
 Height of crest H = R.L. of crest - R.L. of bed = $282.58 - 278.0 = 4.58$ m.
 (Check : $H_s = H + s = 4.58 + 1.92 = 6.50$ m)

(B) Design of Weir Wall

Step 8. Calculation of top width

d = u/s H.F.L. - Crest level = $286.0 - 282.58 = 3.42$ m

∴ Top width $a = \frac{d}{\sqrt{\rho}} = \frac{3.42}{\sqrt{2.24}} = 2.3$ m. ... (i)

From sliding consideration, $\alpha = \frac{3d}{2\rho} = \frac{3 \times 3.42}{2 \times 2.24} = 2.29$ m ... (ii)

From practical considerations, $\alpha = s + 1 = 1.92 + 1 = 2.92$ m. ... (iii)

Hence, provide top width $\alpha = 3$ m.

Step 9. Calculation of bottom width (b)

Considering state I when the water is to the top of the crest gates, with no tail water, the overturning moment is given by :

$$M_0 = \frac{w(H+s)^3}{6} = \frac{wH_s^3}{6} = \frac{9.81 \times (6.5)^3}{6} \approx 449 \text{ kN-m}$$

The moment of resistance is given by

DIVERSION HEADWORKS

$$M_r = \frac{w}{12} \left[((\rho + 1.5)H + 2.5s)b^2 + \alpha(\rho H - H_s)b - \frac{1}{2}\alpha^2(H + 3s) \right]$$

Equating it to the overturning moment and substituting the various values i.e. ($H = 4.58$ m; $s = 1.92$ m; $\alpha = 3$ m; $\rho = 2.24$), we get

$$449 = \frac{9.81}{12} \left[((2.24 + 1.5)4.58 + 2.5 \times 1.92)b^2 + 3(2.24 \times 4.58 - 4.58 - 1.92)b - \frac{1}{2} \times 9(4.58 + 3 \times 1.92) \right]$$

or $549.25 = (21.93b^2 + 11.286b - 46.53)$, from which $b = 4.96 \approx 5$ m

Consider state II, when the water is flowing over the weir and the weir is submerged. Considering the tail water just at the crest of the weir, the overturning moment is given by

$$M_0 = \frac{whH^2}{2}$$

When the tail water is at the crest, d and h will be equal. For this case, the value of d , i.e. head over the crest is given by

$$d = \left[\frac{q^2}{\left(\frac{2}{3}C\right)^2 \times 2g} \right]^{1/3} \quad (\text{neglecting the velocity of approach})$$

where $C = 0.58$ and $q = 11.2$ cumecs/m

$$d = h = \left[\frac{(11.2)^2}{\left(\frac{2}{3} \times 0.58\right)^2 \times 2 \times 9.81} \right]^{1/3} = 3.5 \text{ m}$$

$$M_0 = \frac{whH^2}{2} = \frac{9.81 \times 3.5(4.58)^2}{2} = 360.1 \text{ kN-m}$$

The moment of resistance is given by

$$M_r = \frac{wH(\rho - 1)}{12} (b^2 + ab) = \frac{9.81 \times 4.58(2.24 - 1)}{12} (b^2 + 3b) = 4.643(b^2 + 3b)$$

Equating the two, we get $4.643(b^2 + 3b) = 360.1$

From which $b \approx 8$ m

Adopting the greater of the two, we get $b = 8$ m

Thus the weir wall has top width of 3 m and bottom width equal to 8 m, with equal slopes of u/s and d/s faces.

(c) Design of Impervious and Pervious Aprons

Step 10. Assuming $C = 12$, total creep length is

$$L = CH_s = 12 \times 6.5 = 78 \text{ m.}$$

Step 11. Downstream impervious apron :

$$l_1 = 2.21C \sqrt{\frac{H_s}{13}} = 2.21 \times 12 \sqrt{\frac{6.5}{13}} = 18.8 \text{ m} \approx 19 \text{ m.}$$

IRRIGATION AND WATER POWER ENGINEERING

606

Step 12. Upstream impervious apron (l_2) :

Depth of u/s pile below bed = $d_1 = 278 - 276 = 2$ m

Depth of d/s pile below bed = $d_2 = 278 - 271 = 7$ m.

Depth of d/s pile below bed = $d_2 = 278 - 271 = 7$ m.

$$l_2 = L - l_1 - (b + 2d_1 + 2d_2) = 78 - 19 - (8 + 4 + 14) = 33 \text{ m.}$$

Step 13. Total length of d/s apron

$$l_3 = 18 C \sqrt{\frac{H_1}{13} \times \frac{q}{75}} = 18 \times 12 \sqrt{\frac{6.5}{13} \times \frac{11.2}{75}} = 53.2 \text{ m} \approx 54 \text{ m.}$$

Step 14. Length of filter + launching apron = $l_2 - l_1 = 54 - 19 = 35$ m

Minimum length of inverted filter = $1.5 d_2 = 1.5 \times 7 = 10.5$ m.

Minimum horizontal length of launching apron = $2.5 d_2 = 2.5 \times 7 = 17.5$ m.

Total minimum length of both = $10.5 + 17.5 = 28$ m

But we have to provide 35 m length of both.

Hence, provide the filter of 17.5 m length and launching apron of 17.5 m length.

Make filter of 1 m thick block stones placed on 60 cm thick graded filter material.

Length of launching apron on 3:1 slope = $\sqrt{10} d_2 = 7 \sqrt{10} = 22$ m.

Keeping 1 m thickness in the launched position, volume of stones per metre length of weir = 22 m^3 . Hence thickness in horizontal position = $22/17.5 = 1.3$ m.

Provide 1.5 m thick launching apron.

Step 15. Upstream block protection and launching apron :

$$d_1 = 2 \text{ m}$$

Length of u/s block protection = $d_1 = 2$ m.

Provide 1 m thick block stones over 0.5 m thick graded stones and gravel.

∴ Length of u/s Talus = $2d_1 = 4$ m.

Volume of stones on 3 : 1 launched position, with 1 m thickness

$$= (\sqrt{10} d_1 \times 1) = 6.28 \text{ m}^3$$

∴ Thickness of apron in horizontal position

$$= \frac{6.28}{4} \approx 1.5 \text{ m.}$$

Step 16. Thickness of impervious floor. Provide a nominal thickness of 1 m to the u/s of the weir wall and 1.5 m below the weir wall.

Residual pressure at point A (Fig. 12.24) just at the d/s of the weir wall

$$= H_s - \frac{H_s}{L} x = 6.5 - \frac{6.5}{78} (2 \times 2 + 33 + 8)$$

or

$$H_r = 6.5 - \frac{6.5}{78} \times 45 = 6.5 - 3.9 = 2.6 \text{ m}$$

$$\therefore \text{Thickness} = \frac{4}{3} \frac{H_r}{p-1} = \frac{4}{3} \times \frac{2.6}{2.24-1} = 2.8 \text{ m.}$$

Hence, provide a thickness 2.8 m from the d/s of weir wall (point A) to a point B, 6 m from it.

DIVERSION HEADWORKS

609

Total length of d/s impervious floor = $l_1 = 19$ m.

Pressure at 6 m from the d/s end of the weir wall :

$$H_r = 6.5 - \frac{6.5}{78} (45 + 6) = 2.08 ; \quad t = \frac{4}{3} \frac{2.08}{1.24} = 2.25 \text{ m.}$$

Hence, provide $t = 2.30$ m from 6 to 12 m from d/s end of weir wall.

Pressure at 12 m from the end of the weir wall = $6.5 - \frac{6.5}{78} (45 + 12) = 1.56$ m.

$$t = \frac{4}{3} \frac{1.56}{1.24} = 1.68 \text{ m.}$$

Hence, provide $t = 1.7$ m for the rest of the length of the floor.

(D) Check by Khosla's Theory

(i) Check for the exit gradient :

Total horizontal length of the impervious floor = $33 + 8 + 19 = 60$ m

Depth of downstream pile = 7 m

$$\therefore \alpha = \frac{b}{d} = \frac{60}{7} \approx 8.6.$$

Hence, from Khosla's exit gradient curves,

$$\text{For } \alpha = 8.6, \quad \frac{1}{\pi \sqrt{\alpha}} = 0.14 ; \quad G_z = \frac{H_s}{d} \frac{1}{\pi \sqrt{\alpha}} = \frac{6.5}{7} \times 0.14 \approx \frac{1}{7.8}$$

Permissible exit gradient = $1/6$. Hence, safe.

(ii) Check for thickness of the floor :

Pressure at points C_1 and D_1 of the first pile :

$$\frac{d_1}{b} = \frac{1}{\alpha} = \frac{2}{60} = \frac{1}{30} = 0.033$$

$$\therefore \phi_{D1} \approx 88\% ; \quad \phi_{C1} \approx 83\%.$$

$$\text{Correction for thickness for } \phi_{C1} = \frac{88 - 83}{2} \times 1 = 2.5\% (+)$$

$$\text{Correction for interference of pile 2 on } \phi_{C1} = 19 \sqrt{\frac{D}{b'}} \frac{d+D}{b}$$

Here $b = b' = 60$ m, $D = 7$ m, $d = 2$ m.

$$\therefore \text{Correction} = 19 \sqrt{\frac{7}{60}} \left(\frac{9}{60} \right) = 0.97\% (+)$$

$$\therefore \text{Corrected } \phi_{C1} = 83 + 2.5 + 0.97 \approx 86.5\%$$

$$\therefore \text{Pressure head at } C_1 = H_s \times \phi_{C1} = 6.5 \times 0.865 \approx 5.63 \text{ m} \quad \dots(ii)$$

Pressure at point E_2 and D_2 of the second pile

$$\frac{d_2}{b} = \frac{1}{\alpha} = \frac{7}{60} = 0.117$$

$$\phi_{D2} = 31\% \text{ and } \phi_{E2} = 21\%$$

Correction for thickness of floor to Φ_{m1} ;
 Floor thickness provided near the pile = 1.7 m (Bligh's)

$$\therefore \text{Correction} = \frac{31-21}{7} \times 1.7 = 2.43 \% (-ve)$$

Correction for interference of first pile on $\Phi_{m1} = 19 \sqrt{\frac{D}{b'}} \frac{d+D}{b}$

$$\text{Here } D = 2 \text{ m ; } d = 7 \text{ m ; } b = b' = 60 \text{ m.}$$

$$= 19 \sqrt{\frac{2}{60}} \left(\frac{7+2}{60} \right) = 0.53 \% (-ve)$$

$\therefore$ Correction

$$\therefore \text{Corrected } \Phi_{m1} = 31 - 2.43 - 3.53 = 28.04 \%$$

$$\therefore \text{Pressure head at } E_2 = 6.5 \times 0.2804 = 1.83 \text{ m.}$$

Assuming the linear variation of pressure for intermediate points, the pressures and floor thickness for points A, B and C are as under :

Point A, just to the d/s of weir wall :

$$P_A = 5.63 - \frac{5.63 - 1.83}{60} (33 + 8) = 3.03 \text{ m}$$

$$\therefore t_A = \frac{3.03}{1.24} = 2.45 \text{ m.}$$

Thickness provided by Bligh's theory = 2.8 m. Hence safe.

Thus provide thickness of 2.8 m for a length of 6 m from A to B.

Point B, 6 m from d/s end of the weir wall :

$$P_B = 5.63 - \frac{5.63 - 1.83}{60} (33 + 8 + 6) = 2.66 \text{ m}$$

$$t_B = \frac{2.66}{1.24} = 2.14 \text{ m.}$$

Thickness provided by Bligh's theory = 2.30 m. Hence, safe.

Thus provide a thickness of 2.3 m for a length of 6 m from B to C

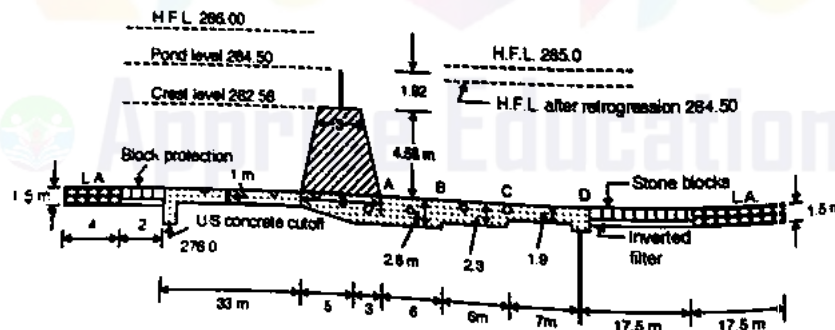


FIG. 12.24. VERTICAL DROP WEIR.

Point C, 12 m from d/s and of the weir wall :

$$P_C = 5.63 - \frac{5.63 - 1.83}{60} (33 + 8 + 12) = 2.27 \text{ m.}$$

$$t_C = \frac{2.27}{1.24} = 1.83 \text{ m.}$$

Thickness provided by Bligh's theory = 1.7 m. Hence, unsafe.

Increase the thickness to 1.9 m from C to D for a length of last 7 m of the impervious floor. The final section of the weir is shown in Fig. 12.24.

12.11. DESIGN OF SLOPING GLACIS WEIR

Sloping glacis weir is a weir of later origin consisting of (i) upstream glacis of slope varying from 1:1 to 2:1 (ii) flat crest of 2 to 3 m width with falling shutters, (iii) down stream glacis of slope varying from 3:1 to 5:1 and (iv) horizontal apron or cistern of adequate length, along with cutoff, block protection, inverted filter and launching aprons, as shown in Figs. 12.4 (a). The main advantage of a sloping glacis weir is that hydraulic jump is formed on the d/s sloping glacis to dissipate the energy of overflowing water. Suction pressure is created in the vicinity of formation of hydraulic jump. Hence we shall first study various elements of hydraulic jump, along with computation of suction pressure.

1. Elements of hydraulic jump :

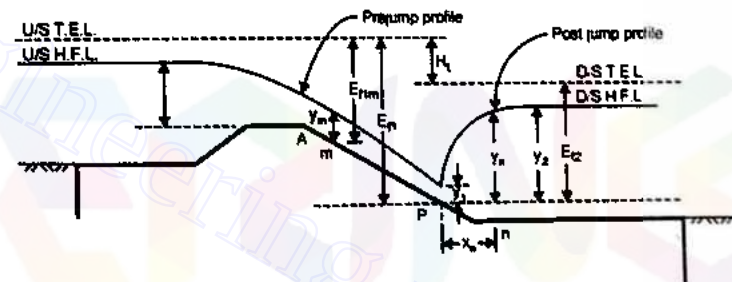


FIG. 12.25. ELEMENTS OF HYDRAULIC JUMP

It can be found from any standard book on hydraulics/fluid mechanics that the conjugate depths y_1 and y_2 before and after the hydraulic jump are related by the equation:

$$y_1 y_2 (y_1 + y_2) = \frac{2q^2}{g} \quad \dots(12.30)$$

where q is the discharge intensity.

The loss of energy (H_L) due to the formation of hydraulic jump is given by

$$H_L = \frac{(y_2 - y_1)^3}{4 y_1 y_2} \quad \dots(12.31)$$

Also, the velocities V_1 and V_2 before and after the jump are given by

$$V_1 = q/y_1 \quad \dots(12.32)$$

$$V_2 = q/y_2 \quad \dots(12.33)$$

and Thus, there are six elements of hydraulic jump, i.e. q, H_1, y_1, V_1, y_2 and V_2 which are related by the above four equations. If out of the above six elements, any two elements (such as q and H_1) are known, the other four elements can be computed with the help of the above four equations. However, the simultaneous solution of the above equations is quite tedious. However, the solution can be easily obtained with the help of Blench curves (Fig. 12.26) which relate q and H_1 to the specific energy E_{f2} after the jump, which in turn is related to y_2 and V_2 as under :

$$E_{f2} = y_2 + V_2^2/2g \quad \dots(12.34)$$

Thus, knowing q and H_1 , E_{f2} can be found from Blench curves (Fig. 12.26) and hence specific energy E_{f1} before the jump is found from the relation :

$$E_{f1} = E_{f2} + H_1 \quad \dots(12.35)$$

Thus, specific energies E_{f1} and E_{f2} before and after the jump are known. The pre-jump and post jump depths y_1 and y_2 are then computed from the following basic relations :

$$E_{f1} = y_1 + \frac{V_1^2}{2g} = y_1 + \left(\frac{q}{y_1}\right)^2 \frac{1}{2g} \quad \dots(12.36)$$

and

$$E_{f2} = y_2 + \frac{V_2^2}{2g} = y_2 + \left(\frac{q}{y_2}\right)^2 \frac{1}{2g} \quad \dots(12.37)$$

The solution of above (cubic) equations is very tedious, since it involves trial and error steps. However, the solution of the above equations can be simplified by using Montague's curves (Fig. 12.27 a) which directly give the values of y_1 and y_2 for different values of E_{f1} and E_{f2} . Fig. 12.27 (b) gives Montague's curves on enlarged scale for y upto 1.2 m. In the Montague's curves the dividing line gives the locus of critical depth (y_c), while the depth y_1 is the depth in the super critical stage ($y < y_c$) and the depth y_2 is in sub-critical stage ($y > y_c$) of the curves. However, the determination from Montague's curves gives only rough values of y_1 and y_2 . Fine values of y_1 and y_2 can finally be obtained from the above two equations (Eqs. 12.36 and 12.37) through trial and error process.

The level of downstream horizontal apron upto which sloping glacis is provided is found by deducting E_{f2} from the downstream T.E.L. If this is provided, the hydraulic jump will be formed just at the end of the sloping glacis. A slightly lower level of the horizontal floor will ensure that the jump will be formed at the point P on the glacis. The length of the jump is usually taken equal to $5(y_2 - y_1)$. Hence in order to obtain turbulence free flow on the downstream side, the length of the horizontal apron should be atleast equal to $5(y_2 - y_1)$, though this length can be reduced by provision of chute blocks, basin blocks and dentated sill, as discussed in Chapter 11.

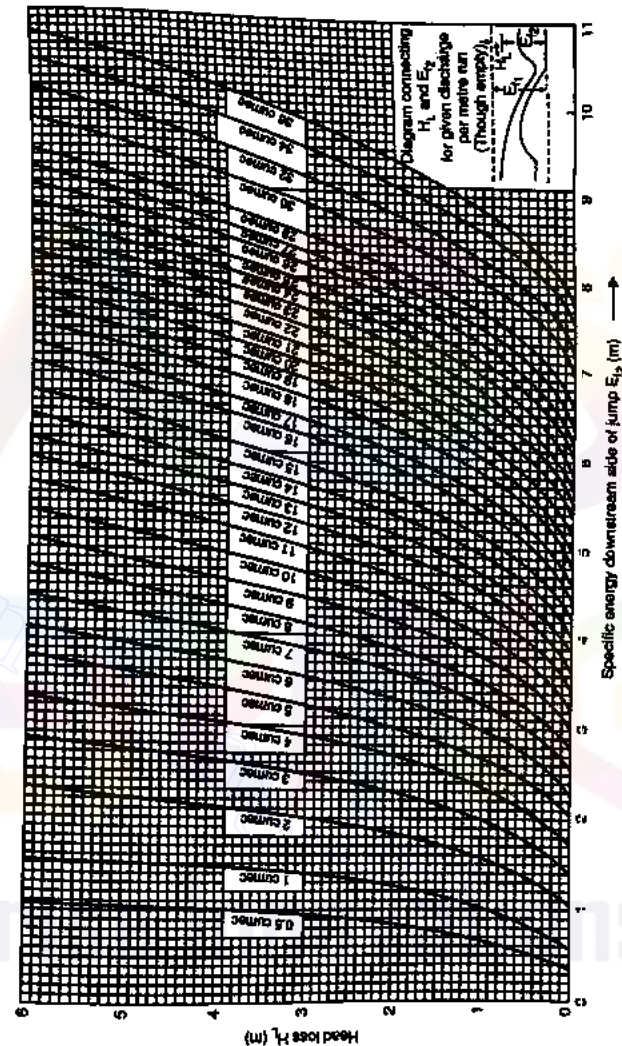


FIG. 12.26 BLENCH CURVES

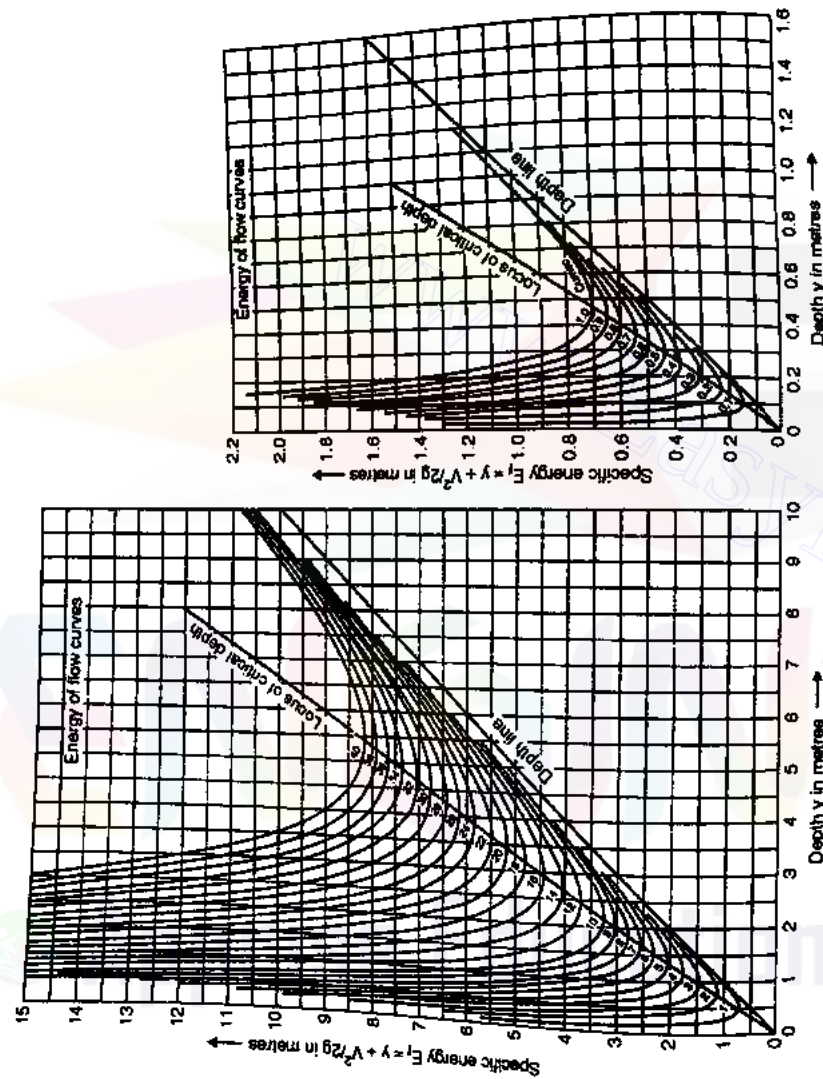


FIG. 12.27. MONTAGUE'S CURVES

2. Determination of water surface profile before the jump:

In order to determine the water surface profile before the hydraulic jump, let us choose any point m on the glacis (Fig. 12.25). It is required to find the depth y_m of the water surface measured above point m .

Procedure

1. Find the elevation of the point m on the d/s sloping glacis, from the geometry.
2. Find specific energy E_{f1m} at point m by subtracting the elevation of m from the u/s T.E.L.
3. Knowing E_{f1m} , find the values of y_m at m either from Montague curve or analytically from Eq. 12.36.
4. Determine the water surface elevation at m by the relation :
Water surface elevation at point $m = \text{R.L. of } m + y_m$
5. Vary the position of point m along the glacis from point A to point P and repeat the above procedure.
6. Join the points corresponding to the water surface elevations so found by a smooth curve, thus getting the prejump profile.

3. Determination of water surface profile after the jump

Procedure :

- (i) Compute the Froude number F_1 of the incoming flow, using the following relation

$$F_1 = \frac{V_1}{\sqrt{g y_1}} = \frac{q}{\sqrt{g y_1^3}} \quad \dots(12.38)$$

- (ii) Select a point n at a horizontal distance x_n from the point P where the jump is located.

- (iii) Find the value of x/y_1 for point n where $x = x_n$ and y_1 is the prejump depth.

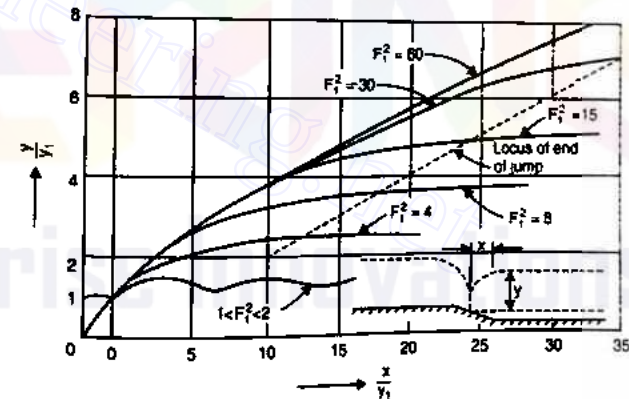


FIG. 12.28. CURVE FOR DETERMINATION OF WATER SURFACE PROFILE AFTER THE JUMP

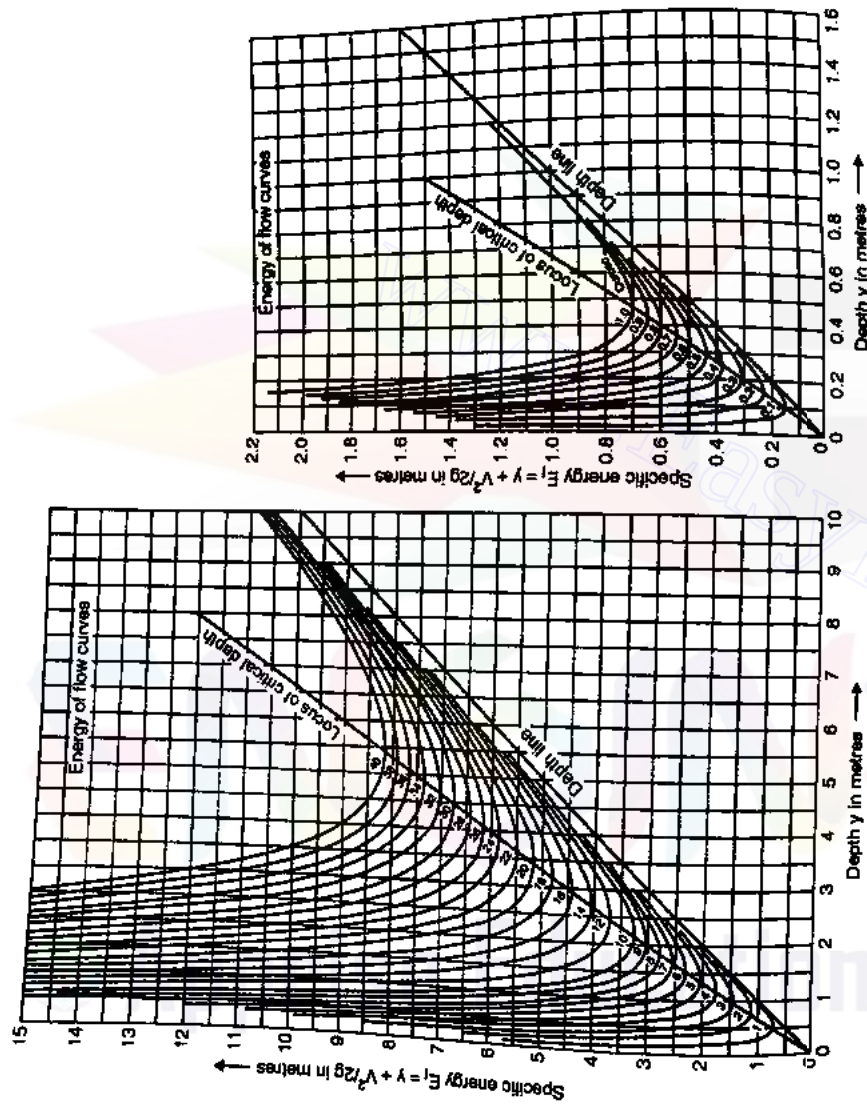


FIG. 12.27. MONTAGUE'S CURVES

2. Determination of water surface profile before the jump:

In order to determine the water surface profile before the hydraulic jump, let us choose any point m on the glacis (Fig. 12.25). It is required to find the depth y_m of the water surface measured above point m .

Procedure

1. Find the elevation of the point m on the d/s sloping glacis, from the geometry.
2. Find specific energy $E_{f/m}$ at point m by subtracting the elevation of m from the w/s T.E.L.
3. Knowing $E_{f/m}$, find the values of y_m at m either from Montague curve or analytically from Eq. 12.36.

4. Determine the water surface elevation at m by the relation :
Water surface elevation at point $m = \text{R.L. of } m + y_m$
5. Vary the position of point m along the glacis from point A to point P and repeat the above procedure.

6. Join the points corresponding to the water surface elevations so found by a smooth curve, thus getting the prejump profile.

3. Determination of water surface profile after the jump

Procedure :

- (i) Compute the Froude number F_1 of the incoming flow, using the following relation

$$F_1 = \frac{V_1}{\sqrt{g y_1}} = \frac{q}{\sqrt{g y_1^3}} \quad \dots (12.38)$$

- (ii) Select a point n at a horizontal distance x_n from the point P where the jump is located.

- (iii) Find the value of x/y_1 for point n where $x = x_n$ and y_1 is the prejump depth.

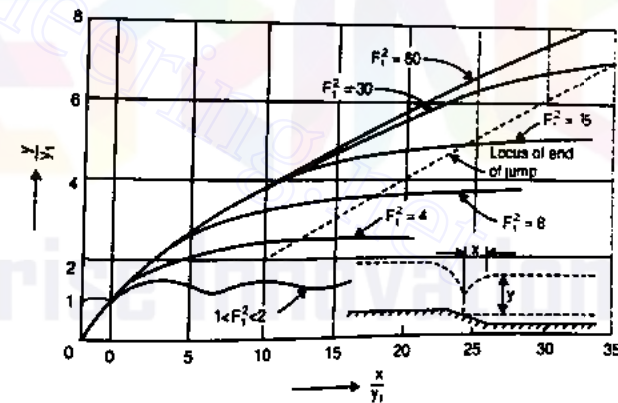


FIG. 12.28. CURVE FOR DETERMINATION OF WATER SURFACE PROFILE AFTER THE JUMP

(iv) Find the value of y/y_1 for this point, corresponding to the Froude number F_1 using curves of Fig. 12.28, for the above value of x/y_1 .

Thus $y_n = y$ ordinate at $n = y_1 \times$ value so obtained.

(v) Water surface elevation = R.L. of $P + y_n = \text{U/S TEL} - E_{f1} + y_n$

(vi) Vary the position of n along the horizontal apron and find the water surface elevation for each position by repeating the above steps. Join these points by a smooth curve to get the water surface profile after the jump.

4. Determination of suction pressure in hydraulic jump trough

The uplift pressure which occurs in the trough of the hydraulic jump is called the *suction pressure* or the *dynamic pressure*. The suction pressure head (h_{us}) at any point in the trough is equal to the vertical ordinate between the subsoil hydraulic gradient line and the water surface profile before or just at the point of formation of hydraulic jump, as shown in Fig. 12.29 (b). Because of the trough formation, this ordinate is quite large at and near the point P .

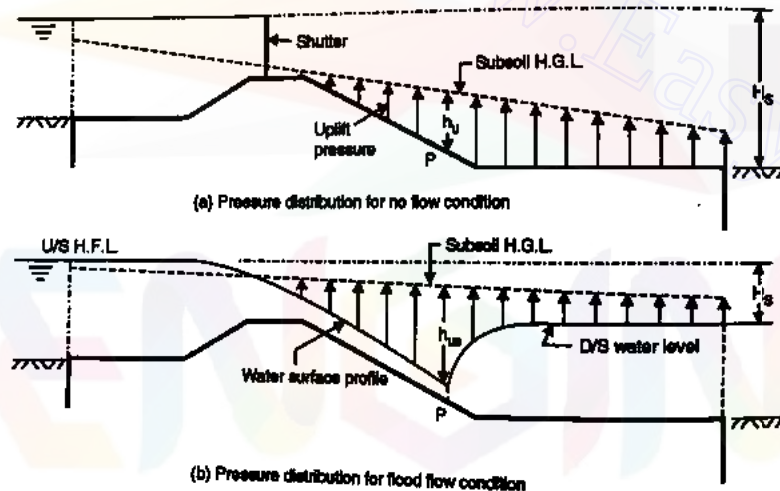


FIG. 12.29. UPLIFT PRESSURE DISTRIBUTION

Fig. 12.29 (a) shows the subsoil hydraulic gradient line (H.G.L.) for no flow condition. The uplift pressure (h_u) at any point P is equal to the vertical ordinate between the floor level and subsoil H.G.L. Fig. 12.29 (b) shows the subsoil H.G.L. along with water surface profile before and after the jump, for flood flow condition. The suction pressure h_{us} at any point is equal to the vertical ordinate between the water surface profile and the subsoil H.G.L. This pressure is maximum at the point P of the formation of hydraulic jump. Comparison of both diagrams shows that uplift pressure is greater for flood flow condition, specially at and near the vicinity of point P of the jump formation. The floor and glacis should be sufficiently thick to resist this suction pressure. However,

for design purposes, the actual suction pressure (or uplift pressure) is usually taken equal to two-thirds of the theoretical value (h_{us}) so obtained. This is because of the following reasons :

- Reduction of actual suction pressure due to back ward rolling of flow.
 - Reduction of suction (or uplift) pressure due to the vertical component of momentum, and
 - Monolithic casting of the concrete floor/glacis leading to beam bending action.
- The floor thickness at any point on the d/s glacis and horizontal apron is found from the relation.

$$t = \frac{h}{\rho - 1}$$

where $h = \frac{2}{3} h_{us}$ for flood flow condition and $h = h_u$ for no flow condition.

5. D/S and U/S Protection against scour

The following provisions are made against scour at the upstream and downstream sides of impervious apron :

- Min. Depth of d/s cutoff : 1.5 R to 2.0 R below d/s H.F.L. after retrogression.
- Min. depth of u/s cutoff : 1.25 R to 1.5 R below u/s H.F.L.
- Length of d/s filter unit : 1.5 d_2 where d_2 is the depth of d/s cutoff below d/s floor.
- Length of d/s launching apron : 1.5 d_2 , with minimum thickness of 1 m on launched slope of 2:1.
- Length of u/s block protection : 1.0 d_1 where d_1 is the depth of U/s cutoff below u/s floor.
- Length of u/s launching apron : 1.5 d_1 with min. thickness of 1 m on launched slope of 2:1.

DESIGN EXAMPLE OF A SLOPING GLACIS WEIR

Example 12.4. Design a sloping glacis weir for the following site conditions :

- Maximum discharge intensity on weir crest = 10 cumecs/m length
- H.F.L. before construction of weir = 255.0 m
- R.L. of river bed = 249.5 m
- Pond level = 254.0 m
- Height of crest shutters = 1 m
- Anticipated downstream water level in the river when the weir is discharging with pond level upstream = 251.5 m.
- Bed retrogression = 0.5 m
- Lacey's silt factor = 0.8
- Permissible exit gradient = 1/7
- Permissible afflux = 1 m.

Solution**Step 1.** Pond level = 254.0 m

Height of shutters = 1 m

Crest level = 254.0 - 1 = 253.0 m

Step 2. $q = 1.70 (K)^{3/2}$

$$K = \left(\frac{q}{1.70} \right)^{2/3} = \left(\frac{10}{1.70} \right)^{2/3} = 3.25 \text{ m}$$

 $\therefore$ Elevation of u/s T.E.L. = R.L. of crest + K = 253.0 + 3.25 = 256.25 m.

Step 3. Regime scour depth $R = 1.35 \left(\frac{Q^2}{f} \right)^{1/3} = 1.35 \left(\frac{10^2}{0.9} \right)^{1/3} \approx 6.5 \text{ m}$
 $\therefore$ Regime velocity = $\frac{q}{R} = \frac{10}{6.5} = 1.54 \text{ m/sec.}$

$$\text{Velocity head} = \frac{V^2}{2g} = \frac{(1.54)^2}{2 \times 9.81} = 0.12 \text{ m.}$$

Step 4. U/S H.F.L. = u/s T.E.L. - 0.12 = 256.25 - 0.12 = 256.13 m.

$$\begin{aligned} \text{Level of d/s T.E.L.} &= \text{H.F.L. before construction} + \frac{V^2}{2g} \\ &= 255.0 + 0.12 = 255.12 \text{ m} \end{aligned}$$

 $\therefore$ Afflux = u/s H.F.L. - d/s H.F.L. = 256.13 - 255.12 = 1.13 m.
 which is very near to the permissible afflux of 1 m.

 The d/s H.F.L. after retrogression = H.F.L. before construction - 0.5
 $= 255.0 - 0.5 = 254.5 \text{ m.}$

D/S T.E.L. after retrogression = 255.12 - 0.5 = 254.62 m.

 $\therefore$ Loss of head at high flood is

$$\begin{aligned} H_L &= \text{u/s T.E.L.} - \text{d/s T.E.L. after retrogression} \\ &= 256.25 - 254.62 = 1.63 \text{ m.} \end{aligned}$$

Step 5. When the weir is discharging with pond level upstream,
Head over the crest = 254.0 - 253.0 = 1 m
 $\therefore$ Discharge $q = 1.70 (K)^{3/2} = 1.70 (1)^{3/2} = 1.70 \text{ cumecs/metre.}$

Loss of head at that time will be given by

$$\begin{aligned} H_L &= \text{pond level} - \text{Anticipated d/s water level at that time} \\ &= 254.0 - 251.5 = 2.5 \text{ m.} \end{aligned}$$

Step 6. Hydraulic jump calculations

Hydraulic jump will be formed on the d/s sloping glacis. The hydraulic jump calculations are done in the tabular form (Table 12.12).

TABLE 12.12. HYDRAULIC JUMP CALCULATIONS

S.No.	Items	High flood condition	Pond level condition
1.	Discharge intensity q	10 cumecs	1.70 cumecs
2.	Head loss H_L	1.63 m	2.50 m
3.	Downstream Specific Energy (E_2) from Blench curves (See Fig. 12.26)	4.30	1.70
4.	Level at which jump will form : (d/s T.E.L. after retrogression - E_2)	254.62 - 4.30 = 250.32 m	251.5 - 1.7 = 249.8 m
5.	Upstream Specific Energy $E_1 = E_2 + H_L$	4.30 + 1.63 = 5.93 m	1.70 + 2.50 = 4.20
6.	Prejump depth D_1 corresponding to E_1 from specific energy curve (See Fig. 12.27) or calculated from the Eq. $E_1 = D_1 + \frac{(q/D_1)^2}{2g}$	1.03 m	0.15 m
7.	Post jump depth D_2 corresponding to E_2 from specific energy curves or calculated from the Eq. $E_2 = D_2 + \frac{(q/D_2)^2}{2g}$	3.96	1.68 m
8.	Height of jump = $D_2 - D_1$	3.96 - 1.03 = 2.96 m	1.68 - 0.15 = 1.53 m
9.	Length of concrete floor required beyond the jump = $5 (D_2 - D_1)$	14.85 m Adopt 15 m length	7.65 m

Step 7. Design of glacis

Let the crest width = 2 m Upstream slope 2 : 1

Downstream glacis slope 3 : 1

Adopt downstream floor R.L. = 249.6 m (i.e. 0.2 m below the lowest level at which jump will form).

Horizontal length of floor beyond the toe of the glacis = 15 m.

Step 8. Depth of sheet piles

$$R = 6.5 \text{ m.}$$

 $\therefore$ R.L. of bottom of scour hole at u/s = 256.13 - 1.5(6.5) = 246.38 m. $\therefore$ Keep R.L. of bottom of u/s pile at 246.30 m.R.L. bottom of scour hole at d/s = 254.5 - 2 $\times$ 6.5 = 241.5 m.

Keep R.L. of bottom d/s pile at 241.5 m.

Provide the intermediate pile at d/s toe of the glacis. as the second line of defence, with its bottom at the same R.L. of 241.5 m.

Step 9. Length of the impervious floor

Maximum percolation head = Pond level - d/s floor level

or

$$H_s = 254.0 - 249.6 = 4.4 \text{ m.}$$

$$\text{Depth of d/s cut off} = d = 249.6 - 241.5 = 8.1 \text{ m.}$$

$$\text{Permissible } G_z = \frac{1}{7} \quad \text{where } G_z = \frac{H_s}{d \pi \sqrt{\lambda}}$$

$$\frac{1}{\pi \sqrt{\lambda}} = G_s \frac{d}{H_s} = \frac{1}{7} \times \frac{8.1}{4.4} = 0.263$$

Hence, from Khoela exit gradient curve

$$\text{For } \frac{1}{\pi \sqrt{\lambda}} = 0.263, \alpha = 1.5 = \frac{b}{d}$$

$$b = \text{length of impervious floor} = \alpha \cdot d = 1.5 \times 8.1 = 12.2 \text{ m}$$

Floor length already provided from other considerations

$$\begin{aligned} &= (\text{horizontal length of w/s glacis}) + (\text{length of crest}) \\ &\quad + (\text{horizontal length of d/s glacis}) \\ &\quad + (\text{length of floor beyond the toe of a glacis}) \\ &= [2(253.0 - 249.5)] + [2] + [3(253.0 - 249.6)] + [15] = 7 + 2 + 10.2 + 15 = 34.2 \text{ m} \end{aligned}$$

Thus the floor length already provided is much more than that required from permissible exit gradient considerations and no upstream floor is required. However, provide a minimum of 1.8 m length of w/s floor so that the total length of impervious floor becomes 36 m.

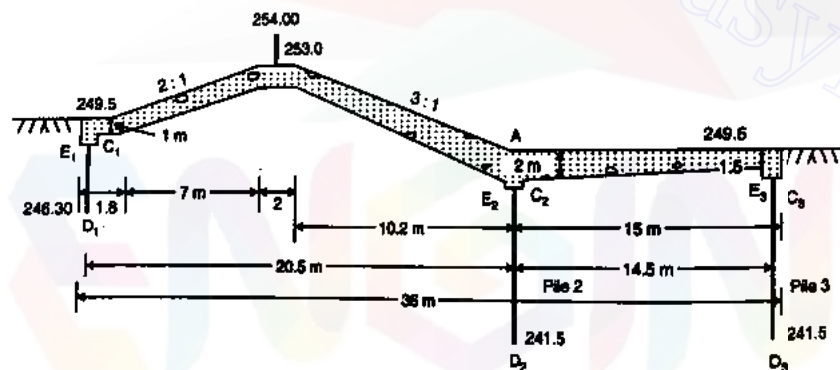


FIG. 12.30.

Step 10. Having known the profile of the glacis and the floor (Fig. 12.30), let us now calculate the pressure at 'key points' by assuming the preliminary thickness of the floor at various points as under.

Floor thickness at w/s of crest = 1.0 m

Floor thickness at d/s end of glacis = 2.0 m (at intermediate pile)

Floor thickness at d/s end of floor = 1.5 m

The calculation of pressures at various key points have been done in Table 12.13. Refer Fig. 12.30.

TABLE 12.13. CALCULATIONS FOR SUBSOIL UPLIFT PRESSURE (LENGTH OF FLOOR $b = 36 \text{ m}$)

Pile line and the pressure point	Depth of pile d	$\frac{1}{\alpha} = \frac{d}{b}$	b_1	$\frac{b_1}{b}$	$\alpha = \frac{b}{d}$	Percentage pressure				
						Before correction	Correction for thickness	Correction for slope	Correction for interference	After correction
Upstream pile	3.2	0.089	...	...	...	...	...	...	...	...
Point C ₁	...	...	...	...	...	72	+ 3.1	...	+ 3.7	78.8
Point D ₁	...	...	...	...	...	82	...	...	...	82
Intermediate pile	8.1	...	21	0.583	4.44	...	...	...	...	...
Point C ₂	...	...	...	...	...	31.5	+ 3.5	...	+ 6.4	41.4
Point D ₂	...	...	...	...	...	45.5	...	...	...	45.5
Point E ₂	...	...	...	...	...	58.5	- 3.2	+ 2.1	- 2.4	55.2
Downstream pile	8.1	0.225	...	...	...	...	...	...	...	...
Point D ₃	...	...	...	...	...	28	...	...	...	28
Point E ₃	...	...	...	...	...	44	- 3.6	...	- 6.4	34

Step 11. Elevation of subsoil H.G. Lines for various conditions. After having known the % pressures at various key points, the elevation of subsoil H.G. line for (i) maximum static head, (ii) high flood condition and (iii) flow at pond level condition can be determined as illustrated in Table 12.14.

TABLE 12.14. ELEVATION OF SUBSOIL H.G. LINES FOR VARIOUS FLOW CONDITIONS

Condition	Datum (d/s floor level or d/s water level) (m)	w/s water level (m)	Ht. / H.G. of subsoil H.G. line above datum (metres)								
			w/s pile line			Intermediate pile line			d/s pile line		
			$\phi_{D_1} = 100\%$	$\phi_{C_1} = 82\%$	$\phi_{C_2} = 78.8\%$	$\phi_{D_2} = 55.2\%$	$\phi_{C_2} = 45.5\%$	$\phi_{C_3} = 41.4\%$	$\phi_{D_3} = 34\%$	$\phi_{C_3} = 28\%$	$\phi_{D_3} = 0\%$
No flow condition $H_s = 254.0 - 249.6 = 4.4 \text{ m}$	249.60	254.00	Ht. 4.40	3.61	3.46	2.43	2.00	1.82	1.50	1.28	0
			Ela. 254.00	253.21	253.06	252.03	251.60	251.42	251.10	250.88	249.60
High flood condition $H_s = 256.13 - 254.5 = 1.63 \text{ m}$	254.50	256.13	Ht. 1.63	1.34	1.28	0.90	0.74	0.67	0.55	0.47	0
			Ela. 256.13	255.84	255.78	255.40	255.24	255.17	255.05	254.97	254.50
Flow at pond level $H_s = 254.0 - 251.5 = 2.5 \text{ m}$	251.50	254.00	Ht. 2.50	2.05	1.97	1.38	1.14	1.04	0.85	0.72	0
			Ela. 254.00	253.55	253.47	252.88	252.64	252.44	252.35	252.22	251.50

Step 12. Plotting the prejump profile for the two flow conditions.

The uplift pressure at any point will be equal to the difference in elevation between the subsoil H.G. line and the elevation of water surface corresponding the jump profiles

for the two flow conditions (or for the d/s floor for no flow conditions). Hence, it is essential to plot the water surface profile of the jump.

Plotting of the water surface on the d/s glacis, before the formation of the jump can be done by finding depths D_1 at various points with the help of Montague curves (Fig. 12.27). The calculations are arranged in Table 12.15.

TABLE 12.15. PRE-JUMP PROFILE (R.L. OF CREST = 253.0 m)

Distance from d/s end of crest (m)	R.L. of glacis (m)	High flood condition $q = 10$ m/s T.F.L. = 254.25		Pond level flow $q = 1.70$ m/s H.F.L. = 254.0	
		$E_p = 254.25 - \text{glacis level}$	D_1	$E_{D_1} = 254.0 - \text{glacis level}$	D_1
3.0	252.0	4.25	1.3	2.00	0.31
6.0	251.0	5.25	1.15	3.00	0.23
8.04	250.32	5.93	1.08	—	—
(jump location for high flood)					
9.0	250.00			4.00	0.16
9.60	249.9			4.25	0.15
(jump location for pond level flow)					

Step 13. Calculations for floor thickness. Fig. 12.31(a) shows the subsoil H.G. line for the two conditions. The floor is to be designed for the maximum unbalanced head found by subtracting elevation of water surface profile (or d/s bed level as the case may be) from the elevation of the corresponding H.G. line.

(i) At the point of formation of hydraulic jump for high flood condition :

Unbalanced head for high flood condition (from Fig. 12.31 a)

$$= 4 \text{ m}$$

Unbalanced static head for no flow condition

$$= 2.56 \text{ m}$$

∴ Design head = 4 m

Since the design head is greater for high flood condition, the floor is designed for 2/3rd of this head

$$= \frac{2}{3} \times 4 = 2.67 \text{ m.}$$

$$\text{Thickness} = \frac{2.67}{2.24 - 1} = 2.15 \text{ m. } (p = 2.24)$$

(ii) At the point of formation of hydraulic jump for flow at pond level condition:

Unbalanced head for pond level condition = 2.9 m

2/3 of this head = 1.93 m

Unbalanced static head for no flow condition = 2.2 m

∴ Design head = 2.2 m

$$\text{Thickness} = \frac{2.2}{1.24} = 1.8 \text{ m}$$

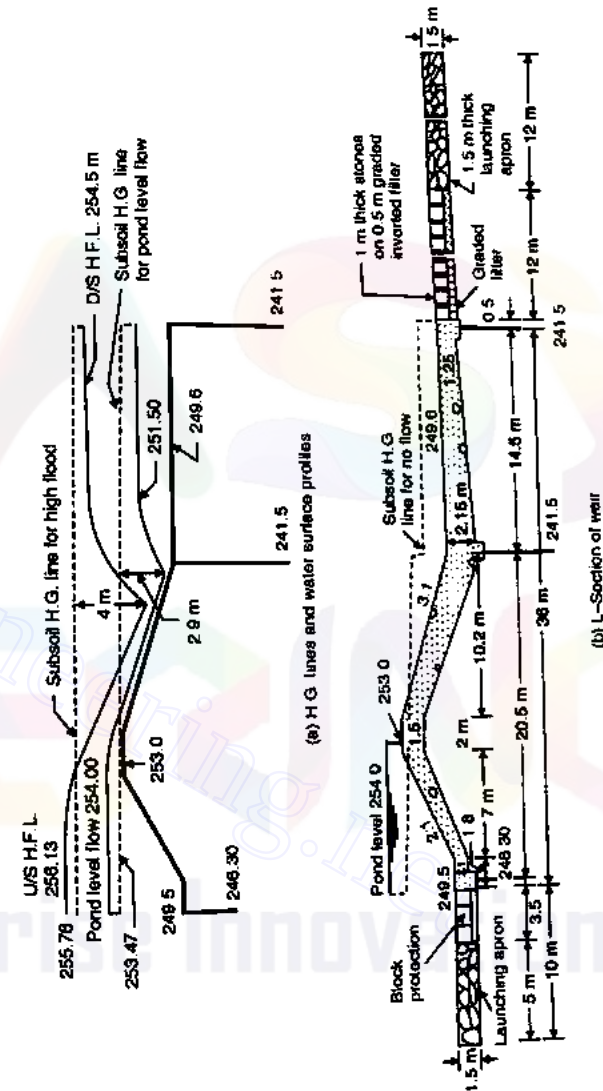


FIG. 12.31 SLOPING GLACIS WEIR

This is less than the thickness of 2.15 m found for high flood condition determined above.

Hence, provide a thickness of 2.15 m at the d/s end of the glacis decreasing it to 1.5 m at the end of the crest. A nominal thickness of 1 m is to be provided for the floor to the u/s of the glacis, as shown in Fig. 12.31(b).

(iii) *Floor thickness at down stream side of sloping glacis*

It will be seen from Fig. 12.31(a), (b) that the design head for the floor to the d/s of the sloping glacis will be governed by the unbalanced static head which is greater than $2/3$ the unbalanced head under the two flow conditions.

Pressure head at point E_2 at d/s end of the floor = 1.5 m.

Thickness required $\frac{1.5}{1.24} = 1.2$ m.

Thus the d/s floor has a thickness of 2.15 m at the d/s end of glacis, and a thickness of 1.25 m at the d/s pile. This reduction in thickness may be done linearly as shown in Fig. 12.31(b).

Step 14. Upstream protection works :

D = depth of u/s scour holes below bed level

$$= 249.5 - 246.38 = 3.12 \text{ m.}$$

Minimum length of u/s block protection = $D = 3.12$ m.

Provide 1 m thick stone blocks laid over 0.5 m thick graded stones for a length of 3.5 m.

Minimum length of u/s launching apron = $1.5 D = 1.5 \times 3.12 \pm 4.7$ m with a thickness of 1 m on a launched slope of 2:1.

However, provide 1.5 m thick launching apron for length of 5 m at the u/s end.

Step 15. Downstream protection works :

D = depth of d/s scour holes below bed level = $249.6 - 241.5 = 8.1$ m,

Length of inverted filter = $1.5 D \pm 12$ m.

Provide 12 m length of inverted filter consisting of 1 m thick stones laid on 0.5 m thick graded inverted filter.

Length of d/s launching apron = $1.5 D = 1.5 \times 8.1 \pm 12$ m. Its minimum thickness on a launched slopes should be 1 m.

Hence, provide 12 m length of launching apron 1.5 m thick.

The longitudinal section of the weir is shown in Fig. 12.31(b).

12.12. DIVIDE WALL

A divide wall or 'groyne' is an embankment, protected on all sides by stones or concrete blocks. It is built at right angles to the axis of the weir separating the u/s and the under sluices. The divide wall extends upstream to a little beyond the beginning of the head regulator and downstream to the 'talus' or 'launching apron' of under sluice portion.

The divide wall is normally constructed of solid masonry. It is necessary to provide well foundation for at least 30 m length from the extreme and taking the well below the deepest possible scour. The divide wall should be designed for the following conditions:

(1) Silt pressure upto the full tank level on the pocket side when the river level is low. At this time there is no water on the weir side.

(2) During floods or high river stage, when the under sluices are discharging, a difference of water level of about 1 m, with higher water level on the weir side be taken for design purposes.

(3) The top width of the wall is kept 1.5 to 2.5 m, while the bottom width is worked out so as to contain the resultant within the middle third of the base.

Functions of the divide wall

(1) The floor level of the under sluices or the pocket portion is generally kept lower than the floor level of the main weir. Hence, a divide wall is essential to separate the two floors. This prevents the turbulent action.

(2) If divide wall is not provided, currents approach the scouring sluices from all directions and their effectiveness is reduced. The divide wall helps in concentrating scouring action of the under sluices for washing out the silt deposited in the pocket by ensuring a straight approach through the pocket.

(3) The divide wall prevents cross-current and flow parallel to the weir. The cross currents and the flow parallel to weir cause formation of vortices and result in deep scour. To prevent this, sometimes, divide walls are also provided at intervals along the main portion of a long weir or at points of change of floor level at both main portion of a long weir or at points of change of floor level both on u/s and d/s side.

(4) The divide wall provides a comparatively 'still pocket' in front of canal head regulator. This helps in more silt deposit in the pocket and entry of comparatively clear water into the canal.

(5) Divide wall incidently serves as one of the side walls of the fish ladders.

12.13. THE FISH LADDER

In big rivers, migrating fishes move from upstream to down stream in search of relatively warm water in the beginning of winter season. Before the monsoons (in May and June) they travel upstream in search of clear water. It is, therefore, essential to make the provision of some space in the construction of the weir for the uninterrupted movements of such migratory fish.

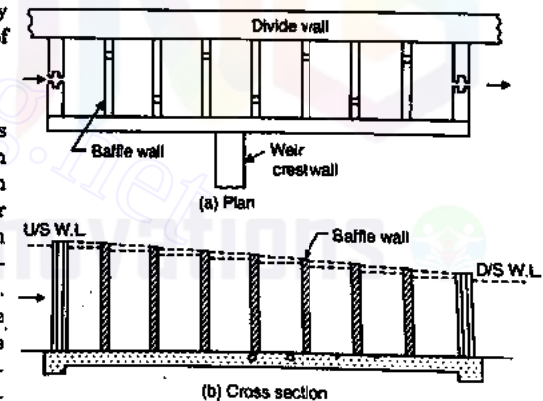


FIG. 12.32. FISH LADDER.

Fig. 12.32 shows the plan and cross-section of a fish ladder. The difference of water levels on the upstream and downstream sides on the weir is split up into water steps by means of the baffle walls constructed across the inclined chute of fish ladder. Most fish can travel upstream in water flowing with a velocity of 3 m/second. The design of fish ladder should be such that velocity of current in the chute is not more than this value. To have effective control, a grooved gate is provided at the extreme u/s and d/s end walls of the fish ladder. The fish ladder is situated near the under sluices where there is always some water in the river section below them.

12.14. THE UNDER SLUICES

The under sluices or the scouring sluices maintain a deep channel in front of the head regulator and dispose off heavy silt and a part of flood discharge on the d/s side of the barrage.

Functions of scouring sluices

- (1) They preserve a clear and defined river channel approaching the regulator.
- (2) They control the silt entry into the canal.
- (3) They scour the silt deposited in the river bed above the approach channel.
- (4) They pass the low floods without dropping the shutter of the main weir, the raising of which entails goods deal of labour and time.
- (5) They provide greater waterway for floods, thus lowering the flood levels.
- (6) They help in impounding fair amount of receding flood so as to secure full storage.

Capacity of under sluices

The discharging capacity of the under sluices should be fixed from the following considerations :

- (1) To ensure proper scouring, its capacity should be at least double the canal discharge.
- (2) The under sluices should have sufficient capacity to discharge winter freshlet (with canal inflow) without the necessity of dropping the weir shutters.
- (3) During high floods, they should be capable of passing about 10 to 15 % of the maximum flood discharge.

Design considerations

- (1) The crest of the under sluices should be lower than the crest of the head regulator by a least 1 to 1.2 metres if special silt exclusion device is not provided. If a silt excluder is provided this difference in level should be at least 1.8 to 2 m.
- (2) The thickness and the length of the impervious floor should be designed on the same lines as the floor of the weir portion.
- (3) Since the discharge intensity, through the under sluices is higher, heavier protection should be provided to the downstream. The length of the talus below the under sluices may be determined from the Bligh's recommendations as under :

$$l_3 = 27C \sqrt{\frac{H_s}{13} \times \frac{q}{75}} \text{ metres.} \quad \dots(12.39)$$

where q = discharge intensity in cumecs/metre.

(4) The length of the impervious apron upstream of the sluice gate may be determined from the Bligh's recommendations as under :

$$l_2 = 3.9C \sqrt{\frac{H_s}{13}} \text{ metres.} \quad \dots(12.40)$$

(5) From the consideration of retrogression, the downstream floor should be depressed upto the likely retrogression bed level downstream.

(6) A bridge downstream of the under sluices should always be provided so that the cranes could move on it to lift the gates if the Stoney's arrangement fail.

12.15. THE CANAL HEAD REGULATOR

A head regulator is a structure constructed at the head of a canal taking off from a reservoir behind a weir or a dam. A head regulator may consist of a number of spans separated by piers and operated by gates similar to that provided in a barrage. The modern tendency is to use bigger spans of 6 to 18 m controlled by Stoney gates or sector gates.

Functions of head regulator

- (1) To make the regulation of supply in the canal easy.
- (2) To control the silt entry in the canal.
- (3) To shut out river floods.

Fig. 12.33 shows the section of canal head regulator.

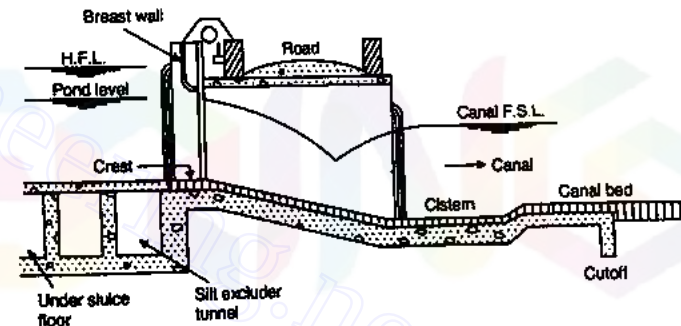


FIG. 12.33. CANAL HEAD REGULATOR.

Design considerations

- (1) The water way of the head regulator should be sufficient to pass the full supply discharge of the canal, with ample factor to safety to allow for any silting up of canal. For a head regulator with broad crest and sloping glacis at the downstream, the following discharge equation is applicable :

$$Q = 1.7(L - knH) H^{3/2} \quad \dots(12.41)$$

where Q = total discharge in cumecs
 L = length of water way in metres
 H = head causing flow
 n = number of end contractions
 k = constant, depending upon the shape of the nose of the pier; varies from 0.01 to 0.03.

(2) The regulators are normally aligned at 90° to the weir, but greater angles, upto 110° are considered preferable for providing smooth entry.

(3) The crest level of the head regulator should be higher than the crest of the under sluices by a minimum of 1 to 1.2 metres if silt excluder is not provided and greater than 1.8 to 2 m if silt excluding device is provided.

(4) The hydraulic jump calculations over the sloping glacis are done in the same way as for sloping glacis weir. The length of cistern below the end of the glacis should at least be equal to $5(D_2 - D_1)$. The level of the cistern bed should be well below the minimum level of the formation of hydraulic jump under various discharge conditions.

(5) The design of the impervious floor should be done on the basis of uplift pressure in the manner similar to that of sloping glacis weir. The worst condition will be during the high floods when the canal is shut and maximum static head acts. In case the floor thickness becomes excessive, a reinforced concrete mat should be provided to resist the uplift pressure by bending action.

(6) A concrete cut-off at the end of the impervious floor should always be provided to keep the exit gradient well within the limits.

(7) The piers separating the various spans of the regulator should be extended beyond the sloping glacis so that they may provide support to the upward bending reinforced concrete mat of the cistern floor.

(8) In order to prevent spilling of water towards the canal during high floods, a reinforced concrete breast wall should be provided from pond level to well above the high flood level. The breast wall is supported between the piers, and is designed for its self weight as well as the water pressure from upstream. They may also be designed for additional weight of gate lifting arrangements, if supported on them.

(9) For the proper operations of the regulator, a bridge is provided, spanning over the piers.

(10) The stability of the piers of the head regulator should be tested to withstand the overturning moment caused by the high pressure head during floods.

(11) There is no empirical formula to determine the length of the *talus* of downstream of head regulator. Generally a *talus* of 4 to 5 times the depth of canal and 0.8 to 1 m thick in concrete blocks or stones are considered to be sufficient.

Types of Regulation

There are two methods of regulation adopted at a head regulator to control the entry of silt into the canal :

- (i) Still pond regulation.
- (ii) Open flow regulation.

Still pond regulation. In this method, the pocket sluices are entirely closed and the canal draws water from the still pond in the pocket. The water in excess of the canal requirements is thus not allowed to escape under the sluice gates. The velocity of water in the pocket is very much reduced on account of excessive water-way since only the supply required for the canal enters the pocket. The silt is thus deposited in the pocket and clear water enters the canal. When the silt deposited has a level about $\frac{1}{2}$ to 1 metre below the crest level of the regulator, the supply in the canal is shut off for about 24 hours and the sluice gates are opened to scour the deposited silt and discharge it downstream. The process is repeated.

Open flow regulation. In this system, the under sluices may be kept open so that the river supply in excess of the canal requirements is escaped. Top water passes into the canal while the bottom water maintains a certain velocity in the pocket to keep the silt to remain in suspension. The advantage of this system is that the canal is not to be closed for scouring the silt. However, the method is very treacherous on account of the uncertain approach channel conditions in the river.

12.16. SILT CONTROL AT HEADWORKS

The entry of silt into the canal can be controlled by :

- (1) Providing a divide wall in the river at the canal side so as
 - (a) to create a trap or pocket.
 - (b) to create the scouring capacity of under sluices by concentrating the current towards them.
- (2) Paving the bottom of the approach channel to reduce disturbance.
- (3) Installing a silt excluder.
- (4) Making entry of clear top water in the canal by
 - (a) providing raised sill in the canal.
 - (b) lowering sill level of scouring sluices.
- (5) Reducing the velocity of water at the intake by providing wider head regulator.
- (6) Avoiding unsteady flow by making the entry smooth.
- (7) Handling carefully the regulation of weir.

There are two types of special works constructed to control the silt entering into the canal :

1. Silt excluder.
2. Silt extractor.

1. Silt excluder. Silt excluder is a device by which silt is excluded from water entering the canal. It is constructed in the bed in front of head regulator.

The fundamental principle on which a silt control device acts lies in the fact that in a flowing stream carrying silt in suspension, the concentration of silt charge in the lower layer is greater than in the upper ones. Hence, the device is so designed that the top and bottom layers are separated without any disturbance. The top water is then led towards the canal while the bottom water containing high silt charge is wasted.

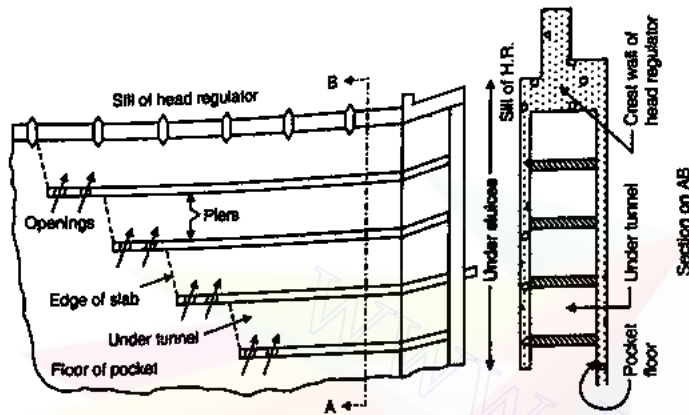


FIG. 12.34. SILT EXCLUDER.

Fig. 12.34 shows a silt excluder — a type used at Khanki weir. The excluder consists of a number of under tunnels resting on the floor of the pocket. The top level of the R.C. roof of the tunnels is kept the same as the sill level of the head regulator. The various tunnels are made of different lengths — the one near the head regulator being of same length as that of the width of the head regulator and the successive tunnels being of decreasing lengths as shown. This arrangement separates the water into two clear layers. The top layer (above the roof of the under tunnels) enters the head regulator, while the bottom layer, containing relatively heavier silt charge goes to the under tunnels and discharges to the d/s of the river through under sluices. The capacity of these tunnels is kept about 10 % of the canal discharge, and the tunnels are so designed that a minimum velocity of 2 to

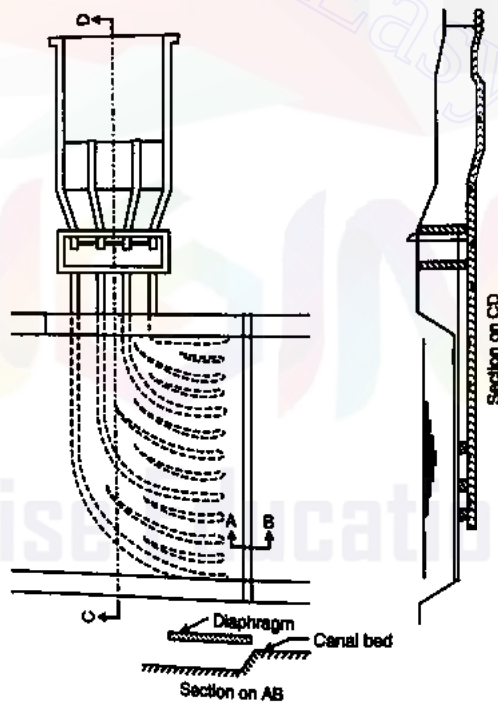


FIG. 12.35. SILT EXTRACTOR.

3 m/sec is maintained. Knowing the discharge and the scouring velocity the total waterway required for the under tunnels can be determined.

2. Silt Extractor or Silt Ejector. A silt extractor or silt ejector is a contrivance by which the silt after it has entered the canal is extracted or thrown out. While a silt excluder is a preventive measure, the silt extractor is a curative measure and is constructed on the canal at some distance away from the head regulator.

Fig. 12.35 shows a silt extractor. It consists of a horizontal diaphragm placed slightly above the canal bed. The canal bed is depressed slightly below the diaphragm so that the height of the silt tunnels below the diaphragm is about 0.5 to 0.8 metres. The diaphragm thus separates the top water level from the bottom layer containing heavier silt charge. The disturbance at the entry of the tunnel is reduced by proper design. However, to prevent the tunnel from being clogged, the velocity is accelerated quickly by steadily reducing sectional area of the tunnels by stream-lined vanes. The discharge through the tunnels are governed by the gate opening of the escape chamber. The escape channel is given a steeper slope so that the silt is discharged back to the river through the shortest route.

12.17. EXAMPLES FROM COMPETITIVE EXAMINATIONS

Example 12.5. An impervious floor of a weir on permeable soil is 16 m long and has sheet piles at both the ends. The upstream pile is 4 m deep and the downstream pile is 5 m deep. The weir creates a net head of 2.5 m. Neglecting the thickness of the weir floor, calculate the uplift pressures at the junction of the inner faces of the pile with the weir floor, by using Khosla's theory.

(Civil Services Exam. 1990)

Solution

Fig. 12.36 shows the floor profile. Here, $b = 16$ m.

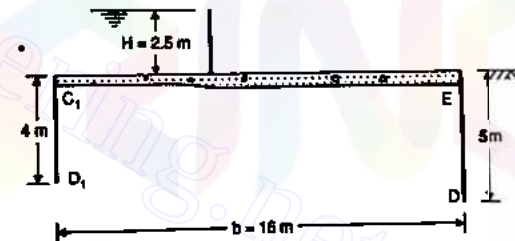


FIG. 12.36

(i) Pressure at point E

$$\Phi_E = \frac{100}{\pi} \cos^{-1} \left(\frac{\lambda - 2}{\lambda} \right)$$

$$\alpha = b/d = 16/5 = 3.2$$

$$\lambda = \frac{1 + \sqrt{1 + \alpha^2}}{2} = \frac{1 + \sqrt{1 + (3.2)^2}}{2} = 2.176$$

$$\Phi_E = \frac{100}{\pi} \cos^{-1} \left(\frac{2.176 - 2}{2.176} \right) = \frac{100}{\pi} (85^\circ.35') \times \frac{\pi}{180} = 47.42\%$$

Let us now apply correction for interference of w/s pile, given by

$$C = -19 \sqrt{\frac{D}{b'}} \left(\frac{d+D}{b} \right)$$

where $d = 5$ m ; $D = 4$ m ; $b' = 16$ m

$$C = -19 \sqrt{\frac{4}{16}} \left(\frac{5+4}{16} \right) = -5.34\%$$

∴ Corrected $\Phi_E = 47.42 - 5.34 = 42.08\%$

$$P_E = 0.4208 \times 2.5 = 1.052 \text{ m}$$

or

(ii) Pressure at point C_1

For finding Φ_{C_1} , imagine the pile to be situated at the downstream end. Then

$$\Phi_{C_1} = 100 - \Phi_E$$

where $\Phi_E = \frac{100}{\pi} \cos^{-1} \left(\frac{\lambda - 2}{\lambda} \right)$ where $\alpha = b/d = 16/4 = 4$

$$\lambda = \frac{1 + \sqrt{1 + \alpha^2}}{2} = \frac{1 + \sqrt{1 + (4)^2}}{2} = 2.562$$

and $\Phi_E = \frac{100}{\pi} \cos^{-1} \left(\frac{2.562 - 2}{2.562} \right) = \frac{100}{\pi} (77.34^\circ) \times \frac{\pi}{180} = 42.96\%$

$$\Phi_{C_1} = 100 - \Phi_E = 100 - 42.96 = 57.04\%$$

Let us now apply correction for interference of d/s pile, given by

$$C = +19 \sqrt{\frac{D}{b'}} \left(\frac{d+D}{b} \right)$$

where $d = 4$ m ; $D = 5$ m ; $b' = b = 16$ m

$$C = 19 \sqrt{\frac{5}{16}} \left(\frac{4+5}{16} \right) = +5.97\%$$

$$\Phi_{C_1} = 57.04 + 5.97 = 63.01\%$$

$$P_C = 0.6301 \times 2.5 = 1.575 \text{ m}$$

Example 12.6. The concrete floor of a head regulator is level with the channel bed (except for the short crest hump) and is 13 m long. The floor is provided with cut off usually at its upstream and downstream ends. The depth of upstream cutoff is 1.5 m (below the floor level) and that the downstream wall is 2 m. Using Khosla's

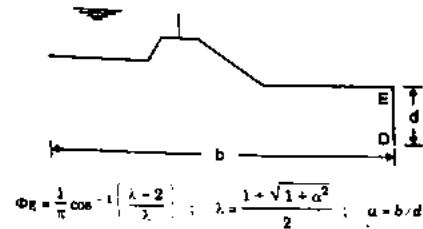


FIG. 12.37

theory (See Fig. for definition sketch and formula) determine the thickness of the floor at its mid length and also at its junction with upstream and downstream cutoff walls. The floor thickness may not be less than 30 cm anywhere. The upstream FSL is 1.5 m above the floor level. If the permissible exit gradient is 0.18, is the floor safe against failure by piping ? (Civil Services Exam. 1982)

Solution: Take $p = 2.24$ for concrete.

(1) Pressure and thickness at the junction E of d/s cutoff with floor

$d = 2$ m ; $b = 13$ m ; $\alpha = b/d = 13/2 = 6.5$; $H_s = 1.5$ m

$$\lambda = \frac{1 + \sqrt{1 + \alpha^2}}{2} = \frac{1 + \sqrt{1 + (6.5)^2}}{2} = 3.7882$$

$$\Phi_E = \frac{1}{\pi} \cos^{-1} \left(\frac{\lambda - 2}{\lambda} \right) = \frac{1}{\pi} \cos^{-1} \left(\frac{3.7882 - 2}{3.7882} \right) = 0.3435 = 34.35\%$$

Correction for interference of w/s sheet pile, given by

$$C = (-) 19 \sqrt{\frac{D}{b'}} \left(\frac{d+D}{b} \right) = -19 \sqrt{\frac{1.5}{13}} \left(\frac{2+1.5}{13} \right) = -1.74\%$$

Neglect the correction due to thickness of the floor.

∴ Corrected $\Phi_E = 34.35 - 1.74 = 32.61\%$

and

$$P_E = 0.3261 \times 1.5 = 0.489 \text{ m}$$

$$\text{Floor thickness, } t_E = \frac{P_E}{p - 1} = \frac{0.489}{2.24 - 1} = 0.394 \approx 0.4 \text{ m}$$

(2) Pressure and thickness at junction C_1 of w/s cutoff with floor

$\Phi_{C_1} = 100 - \Phi_E$, by principle of reversibility of flow.

where

$$\alpha = 13/1.5 = 8.667 ; \lambda = [1 + \sqrt{1 + (8.667)^2}] / 2 = 4.862$$

$$\Phi_E = \frac{1}{\pi} \cos^{-1} \left(\frac{4.862 - 2}{4.862} \right) = 0.3 = 30\%$$

$$\Phi_{C_1} = 100 - \Phi_E = 100 - 30 = 70\%$$

Correction for interference of d/s sheet pile, given by

$$C = (+) 19 \sqrt{\frac{D}{b'}} \left(\frac{d+D}{b} \right) = +19 \sqrt{\frac{2}{13}} \left(\frac{1.5+2}{13} \right) = +2\%$$

Neglect the correction for thickness of floor.
Corrected $\Phi_{C1} = 70 + 2 = 72\%$

$$P_{C1} = 0.72 \times 1.5 = 1.08 \text{ m}$$

However, this uplift pressure will be counter-balanced by the downward weight of impounded water. Hence provide a nominal (minimum) thickness of 0.3 m at the w/s end.

(3) **Pressure and thickness at mid-length**
Assuming linear variation, uplift pressure at mid length is given by

$$P_M = (1.08 + 0.489)/2 \approx 0.785 \text{ m}$$

$$\therefore \text{Floor thickness, } t_M = \frac{0.785}{2.24 - 1} = 0.633 \text{ m} \approx 0.64 \text{ m}$$

(4) **Exit gradient**

$$G_x = \frac{H_s}{d} \cdot \frac{1}{\pi \sqrt{\lambda}} \text{ where } d = 2 \text{ m ; } \lambda = 3.7882 \text{ and } H_s = 1.5 \text{ m}$$

$$G_x = \frac{1.5}{2} \times \frac{1}{\pi \sqrt{3.7882}} = 0.123 < 0.18 \text{ (permissible)}$$

Hence the floor is safe against failure by piping.

Example 12.7. A stream has a width of 30 m, depth of 3 m and a mean velocity of 1.25 m/sec. Find the height of weir to be built on the stream floor to raise the water level by 1 m. Assume value of discharge coefficient as 0.95.
(Engg. Services Exam. 1995)

Solution

The regular depth of water in the stream, equal to the d/s depth (D) of water = 3 m. Since water level is raised by 1 m due to construction of weir, the u/s water depth, $D_u = 3 + 1 = 4 \text{ m}$.

Let x be the height of weir. Then the head h over the weir will be equal to $(4 - x) \text{ m}$.

Now $Q = \text{discharge in stream}$
 $= B \times D \times V = 30 \times 3 \times 1.25 = 112.5 \text{ cumecs}$

But $Q = C \cdot B \cdot h^{3/2}$ (weir formula)

$$\text{where } C = \frac{2}{3} C_d \sqrt{2g} = \frac{2}{3} \times 0.95 \sqrt{2 \times 9.81} = 2.805$$

$$112.5 = 2.805 \times 30 (4 - x)^{3/2}$$

$$\text{From which } (4 - x) = (1.337)^{2/3} = 1.213 \text{ m}$$

$$\text{Hence } x = 4 - 1.213 = 2.787 \text{ m}$$

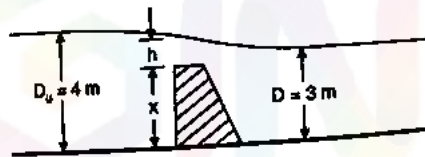


FIG. 12.38

DIVERSION HEADWORKS

Example 12.8. A weir has a solid horizontal floor length of 50 m with two lines of cutoff of 8 m depth, below the river bed at its two ends. The floor thickness is 1 m at the upstream end and 2 m at the downstream end, with its upper level being in flush with the river bed. For an effective head of 5 m over the weir, calculate the uplift pressure at the two inside corner points (junction of bottom of floor with the cutoff) and also exit gradient.

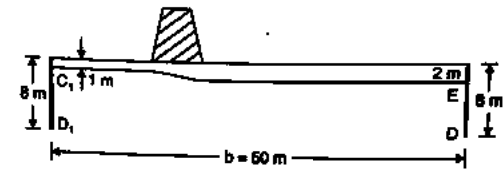


FIG. 12.39

the uplift pressure at the two inside corner points (junction of bottom of floor with the cutoff) and also exit gradient.
(Engg. Services Exam. 1984)

Solution

Give : $H_s = 5 \text{ m ; } b = 50 \text{ m}$

$$d_1 = d_2 = 8 \text{ m}$$

Downstream cutoff

$$\alpha = b/d = 50/8 = 6.25$$

$$\lambda = \frac{1 + \sqrt{1 + \alpha^2}}{2} = \frac{1 + \sqrt{1 + (6.25)^2}}{2} = 3.665$$

$$\Phi_E = \frac{1}{\pi} \cos^{-1} \left(\frac{\lambda - 2}{\lambda} \right) = \frac{1}{\pi} \cos^{-1} \left(\frac{3.665 - 2}{3.665} \right) = 0.3499 \text{ (i.e. 34.99\%)}$$

$$\Phi_D = \frac{1}{\pi} \cos^{-1} \left(\frac{\lambda - 1}{\lambda} \right) = \frac{1}{\pi} \cos^{-1} \left(\frac{3.665 - 1}{3.665} \right) = 0.2409 \text{ (i.e. 24.09\%)}$$

$$\text{Correction due to thickness} = (-) \frac{\Phi_E - \Phi_D}{d} \times t = \frac{34.99 - 24.09}{8} \times 2 \approx 2.73\%$$

Correction due to interference of u/s cutoff, given by

$$C = (-) 19 \sqrt{\frac{D}{b'}} \left(\frac{d + D}{b} \right) = -19 \sqrt{\frac{8}{50}} \left(\frac{8 + 8}{50} \right) = -2.43\%$$

$$\text{Hence correct } \Phi_E = 34.99 - 2.73 - 2.43 = 29.83\%$$

or

$$P_E = 0.2983 \times 5 \approx 1.40 \text{ m}$$

Upstream cutoff

Since $d_1 = d_2 = d$, α and λ are the same as computed above.

$$\text{Hence } \Phi_{C1} = 100 - \Phi_E = 100 - 34.99 = 65.01\%$$

and

$$\Phi_{D1} = 100 - \Phi_D = 100 - 24.09 = 75.91\%$$

$$\text{Correction for thickness} = + \frac{75.91 - 65.01}{8} \times 1 \approx +1.36\%$$

Correction due to interference of d/s cutoff, given by

$$C = +19 \sqrt{\frac{D}{b}} \left(\frac{d+D}{b} \right) = +2.43\%, \text{ computed above.}$$

$$\therefore \text{Corrected } \Phi_{c1} = 65.01 + 1.36 + 2.43 = 68.8\%$$

$$\text{or } P_{c1} = 0.688 \times 5 = 3.44 \text{ m}$$

Exit gradient

$$G_x = \frac{H_s}{d} \cdot \frac{1}{\pi \sqrt{\lambda}} = \frac{5}{8} \times \frac{1}{\pi \sqrt{3.665}} = \frac{1}{9.82}$$

Example 12.9. A river discharges 1000 cum/sec of water at high flood level of RL 103 m. A weir is constructed for flow diversion with a crest length of 255 m and total length of concrete floor as 40 m. The weir has to sustain the under seepage at a maximum static head of 2.4 m. The silt factor and the safe exit gradient for the river bed material are 1.1 and 1/6 respectively. Determine the depth of cutoff required at the downstream end of the concrete floor. Take the level of downstream concrete floor as R.L. 100 m. Check for exit gradient. (Engg. Services Exam. 1998)

Solution : Given $Q = 1000$ cumecs ; $L = 255$ m ; $f = 1.1$

$$q = Q/L = 1000/255 = 3.922 \text{ m}^3/\text{s}/\text{m}$$

$$R = 1.35 \left(\frac{q^2}{f} \right)^{1/3} = 1.35 \left\{ \frac{(3.922)^2}{1.1} \right\}^{1/3} \triangleq 3.25 \text{ m}$$

$$D/s \text{ H.F.L.} = 103 \text{ m}$$

$$\text{R.L. of d/s cutoff} = D/s \text{ H.F.L.} - 1.5 R = 103 - 1.5 \times 3.25 \triangleq 98.13 \text{ m}$$

$$\text{R.L. of d/s concrete floor} = 100.0 \text{ m.}$$

$$\therefore \text{Depth of d/s cutoff} = 100 - 98.13 = 1.87 \text{ m} = d.$$

$$\text{Exit gradient, } G_x = \frac{H_s}{d} \cdot \frac{1}{\pi \sqrt{\lambda}} \text{ (where } H_s = 2.4 \text{ m)}$$

Here,

$$\alpha = b/d = 40/1.87 = 21.39$$

$$\lambda = \frac{1 + \sqrt{1 + \alpha^2}}{2} = \frac{1 + \sqrt{1 + (21.39)^2}}{2} = 11.2$$

$$G_x = \frac{2.4}{1.87} \times \frac{1}{\pi \sqrt{11.2}} = \frac{1}{8.19} < \frac{1}{6}$$

Hence the weir floor is safe against piping.

Example 12.10. A weir across an alluvial river has a horizontal floor of length 60 m and retains 6 m of water under full pond condition. If the downstream sheet pile is driven to a depth of 6 m below the average bed level, calculate the exit gradient. Further, assuming a porosity of 30% and the relative density of soil particles as 2.70, estimate the critical exit gradient and the factor of safety of the system with respect to the exit gradient. (Civil Services Exam. 1989)

Solution :

$$\text{Given : } b = 60 \text{ m ; } H_s = 6 \text{ m ; } d = 6 \text{ m}$$

$$\alpha = b/d = 60/6 = 10$$

$$\lambda = (1 + \sqrt{1 + \alpha^2})/2 = (1 + \sqrt{1 + 10^2})/2 = 5.5249$$

$$G_x = \frac{H_s}{d} \times \frac{1}{\pi \sqrt{\lambda}}$$

$$= \frac{6}{6} \times \frac{1}{\pi \sqrt{5.5249}} = \frac{1}{7.38} = 0.1354$$

$$e = \frac{n}{1-n} = \frac{0.3}{1-0.3} = 0.429$$

$$\text{Critical hydraulic gradient} = \frac{G-1}{1+e} = \frac{2.7-1}{1+0.429} = 1.19$$

$$\therefore \text{Factor of safety with respect to exit gradient} = \frac{1.19}{0.1354} = 8.79$$

Example 12.11. For the following data, make necessary hydraulic calculations for a vertical drop weir using the Bligh's theory

Maximum flood discharge	= 2800 cumecs
H.F.L. before construction	= 285.0 m
Minimum water level	= 278.0 m
Downstream bed level	= 278.0 m
F.S.L. of canal	= 284.0 m
Allowable afflux	= 1 m
Coefficient of creep	= 12
Permissible exit gradient	= 1/6

(Civil Services Exam. 1987)

Solution

This question was set from Author's book (Example 12.3). For solution see example 12.3.

Example 12.12. Following particulars were recorded from a barrage :

(i) Maximum reservoir level	= 212 m
(ii) Pond level	= 211 m
(iii) Downstream high flood level in the river	= 210 m
(iv) Maximum design flood discharge	= 3500 m ³ /s
(v) Crest level of the barrage	= 207 m
(vi) Crest level the head regulator	= 208 m
(vii) Coefficient of discharge	= 2.1 m ^{1/2} /sec for barrage
(viii) Coefficient of discharge	= 1.5 m ^{1/2} /sec for head regulator
(ix) River bed level	= 205 m
(x) design discharge of main canal	= 500 m ³ /sec

IRRIGATION AND WATER POWER ENGINEERING

Determine the number of gates required for the barrage and the head regulator if each gate has 10 m clear span. Neglect (1) end contractions due to piers and abutments and (2) velocity of approach.

If the stilling basin is provided downstream of the barrage for the energy dissipation, find the length and R.L. of the basin floor. Assume that the length of the basin is 5 times the conjugate depth required for hydraulic jump. Neglect losses due to friction. (Engg. Services Exam. 2001)

Solution

1. Design of water way for barrage during flood

High flood discharge = 3500 m³/s

H.F.L. = maximum reservoir level = 212 m

Crest level of barrage = 207 m

Head causing flow, $H = 212 - 207 = 5$ m.

Coefficient of discharge for barrage, $C = 2.1$

Hence the length of waterway, L is given by the weir formula :

$$Q = C L H^{3/2}$$

Neglecting the velocity of approach and ignoring end contractions,

$$L \approx L_w = \frac{Q}{C H^{3/2}} = \frac{3500}{2.1 (5)^{3/2}} = 149.07 \text{ m} \approx 150 \text{ m (say)}$$

Provide 15 bays each of 10 m clear span.

Hence Number of gates for the barrage = 15

2. Design of waterway for canal head regulator

Design discharge of canal = 500 cumecs.

Pond level = 211 m

R.L. of crest of head regulator = 208 m

Head causing flow = 211 - 208 = 3 m.

Coefficient of discharge, $C = 1.5$ for head regulator.

Hence length of water way, L is given by the weir formula :

$$Q = C L H^{3/2}$$

Neglecting the velocity of approach and ignoring end contractions

$$L \approx L_w = \frac{Q}{C H^{3/2}} = \frac{500}{1.5 (3)^{3/2}} = 64.2 \text{ m}$$

However, provide $L = 70$ m, in the form of 7 bays each of 10 m clear span.

Hence number of gates for the head regulator = 7

3. Design of stilling basin

U/L T.E.L. = w/s H.F.L. = 212 m (neglecting the velocity of approach)

D/L T.E.L. = d/s H.F.L. = 210 m (neglecting the velocity of flow

$$H_1 = 212 - 210 = 2 \text{ m.}$$

DIVERSION HEADWORKS

Linear discharge, $q = Q/L = 3500/150 = 23.33$ cumecs/m

Hence we have to compute the value of E_{f_2} for $H_L = 2$ m and $q = 23.33$

Since Blench curves are not given, we have to compute E_{f_2} analytically as under.

$$y_c = (q^2/g)^{1/3} = [(23.33)^2/9.81]^{1/3} \approx 3.81 \text{ m}$$

Now let $Y = y_2/y_c$ and $Z = H_L/y_c = 2/3.81 = 0.5249$

Since $Z < 1$, the value of Y is given by

$$Y = 1 + 0.93556 Z^{0.385} = 1 + 0.93556 (0.5249)^{0.385} = 1.738$$

Hence $y_2 = Y \cdot y_c = 1.738 \times 3.81 = 6.62$ m

$\therefore$ Length of cistern = $5 y_2 = 5 \times 6.62 = 33.1$ m

Also, E_{f_2} is given by the following relation :

$$\frac{E_{f_2}}{y_c} = Y + \frac{1}{2 Y^2} = 1.738 + \frac{1}{2 (1.738)^2} = 1.9035$$

$\therefore E_{f_2} = 1.9035 y_c = 1.9035 \times 3.81 = 7.25$ m

Hence level at which jump will form = d/s T.E.L. - E_{f_2}

$$= 210 - 7.25 = 202.75 \text{ m}$$

However, provide cistern at R.L. 202.6 m

PROBLEMS

1. Explain with help of a diagram, the various component parts, along with their functions, of a diversion headwork.
2. Describe, in brief, various types of weirs. Distinguish clearly between a weir and a barrage.
3. Discuss in brief various causes of failure of weirs and their remedies.
4. Explain Khosla's method of independent variables. How do you apply corrections for (i) thickness of floor, (ii) inclination of floor and (iii) interference of piles?
5. What is meant by scour? What precautions do you take against it in weir design?
6. Explain the procedure for the design of a vertical drop weir.
7. What do you understand by launching apron? How is it designed?
8. Write short notes on the following : (i) Fish ladder, (ii) Divide wall, (iii) Under sluices.
9. What are the methods of controlling entry of silt at the headworks of a canal?
10. Describe, with the help of sketches, the working of a silt excluder.
11. What do you understand by 'silt extractor' and 'silt ejector'?
12. Write a note on 'location of headworks'.
13. The cross-section of a weir is shown in Fig. 12.40. Calculate (a) Uplift pressure at point A and E, (b) Thickness of concrete apron at point B, (c) Exit gradient. Use Khosla's curves.

$$\text{Ans. (a) } \phi_A = 81.5\% ; \phi_E = 25.9\% \quad (b) t_B = 2.2 \text{ m} \quad (c) G_E = \frac{1}{8.9}$$

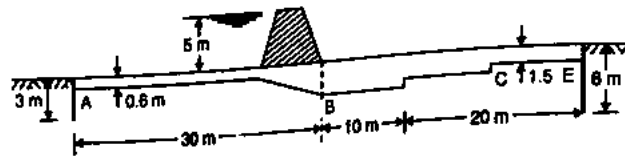


FIG. 12.40.

14. Using Khosla's curves, determine the following for the apron shown in Fig. 12.41.

- Uplift pressure at points E, D, C, E₁, and D₁.
- Exit gradient.

Neglect the effect of thickness of the floor.

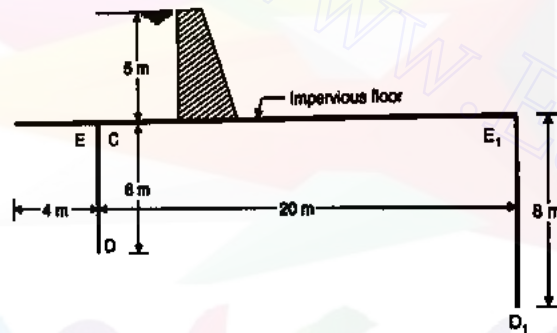


FIG. 12.41.

[Ans. (i) $\phi_E = 87\%$; $\phi_D = 65.6\%$; $\phi_C = 59.1\%$; $\phi_{E1} = 42.9\%$; $\phi_{D1} = 32.5\%$;

(ii) $G_E = \frac{1}{7.1}$

15. Design a vertical drop weir, on Bligh's theory for the following data :

- Maximum flood discharge = 1200 cumecs
- H.F.L. before construction of weir = 172.5 m
- River bed level = 168.0 m
- F.S.L. of canal = 171.5 m
- Allowable afflux = 1 m
- Coefficient of creep = 11.

Assume any other data not given. The weir wall need not be designed, and its dimensions may be taken as follows : (i) top width = 3 m, (ii) bottom width = 6 m.

SECTION-III

DISTRIBUTION SYSTEM

Chapters :

- FLOW IRRIGATION
- IRRIGATION CHANNELS-I : SILT THEORIES
- IRRIGATION CHANNELS-II : DESIGN PROCEDURE
- WATERLOGGING AND CANAL LINING
- CANAL OUTLETS
- CANAL REGULATION WORKS
- CROSS DRAINAGE WORKS

Flow Irrigation

13.1. CANALS : CLASSIFICATION

A canal is an artificial channel, generally trapezoidal in shape constructed on the ground to carry water to the fields either from the river or from a tank or reservoir. Canals can be classified in following ways :

(a) **Classification based on the nature of source of supply**

(1) Permanent canal.	(2) Inundation canal.

A canal is said to be *permanent* when it is fed by a permanent source of supply. The canal is a well made up regular graded channel. It has also permanent masonry works for regulation and distribution of supplies. A permanent canal is also sometimes known as *perennial canal* when the sources from which canal takes is an ice fed perennial river.

Inundation Canals usually draw their supplies from rivers whenever there is a high stage in the river. They are not provided with any headworks for diversion of river water to the canal. They are, however, provided with a canal head regulator. The head of the canal has to be changed sometimes to suit the changing pattern of river course.

(b) **Classification based on financial output**

(1) Productive canal.	(2) Protective canal.

Productive Canals are those which yield a net revenue to the nation after full development of irrigation in the area. *Protective canal* is a sort of relief work constructed with the idea of protecting a particular area from famine.

(c) **Classification based on the function of the canal**

(1) Irrigation canal.	(2) Carrier canal.
(3) Feeder canal.	(4) Navigation canal.
(5) Power canal.	

An *irrigation canal* carries water to the agricultural fields. A *carrier canal*, besides doing irrigation, carries water for another canal. Upper Chenab canal in West Punjab (Pakistan) is the example of one such canal. A *feeder canal* is constructed with the idea of feeding two or more canals. Examples of such canals are : Rajasthan feeder canal and Sirhind feeder.

(d) **Classification based on boundary surface of the canal**

Based on the type of boundary surface, canals may be of the following types :

1. Alluvial canals
2. Non-alluvial canals
- and 3. Rigid boundary canals.

An *alluvial canal* is the one which is excavated in alluvial soils, such as silt. A *non-alluvial canal* is the one which is excavated in non-alluvial soils, such as loam, clay, hard soil (murrum), rock etc. *Rigid boundary canals* are those which have rigid sides and rigid base, such as *lined canals*.

(e) **Classification based on the discharge and its relative importance in a given network of canals**

(1) Main canal.	(2) Branch canal.
(3) Major distributary.	(4) Minor distributary.
(5) Water course.	

Main canal generally carries water directly from the river or reservoir. Such a canal carries heavy supplies and is not used for direct irrigation except in exceptional circumstances. Main canals act as water carriers to feed supplies to branch canals and major distributaries.

Branch canals are the branches of the main canal in either direction taking off at regular intervals. In general, branch canals also do not carry out any direct irrigation, but at times direct outlets may be provided. Branch canals are usually feeder channels for major and minor distributaries. They usually carry a discharge of over 5 cumecs.

Major distributaries usually called *Rajbha*, take off from a branch canal. They may also sometimes take off from the main canal, but their discharge is generally lesser than branch canals. They are real irrigation channels in the sense that they supply water for irrigation to the field through outlets provided along them. Their discharge varies from 1/4 to 5 cumecs.

Minor Distributaries or *minors* take off from branch canals or from distributaries. Their discharge is usually less than 1/4 cumecs. They supply water to the water courses through outlets provided along them.

A *water course* or *field channel* is a small channel which ultimately feeds the water to irrigation fields. Depending upon the size and extent of the irrigation scheme, a field channel may take off from a distributary or minor. Sometimes, it may even take off from the branch canal for the field situated very near to the branch canal.

(f) **Classification based on canal alignment**

According to the alignment, a canal may be classified as under (Fig. 13.1):

- (1) Contour canal.
- (2) Watershed canal.
- (3) Side slope canal.

The characteristic features of these canals are discussed in the next article.

13.2. CANAL ALIGNMENT

A canal has to be aligned in such a way that it covers the entire area proposed to be irrigated, with shortest possible length and at the same time its cost including the cost of cross drainage works is a minimum. A shorter length of canal ensures less loss of head due to friction and smaller loss of discharge due to seepage and evaporation, so that additional areas can be brought under cultivation.

According to alignment, the canals may be of the following types (Fig. 13.1)

- (1) Ridge canal.
- (2) Contour canal.
- (3) Side slope canal.

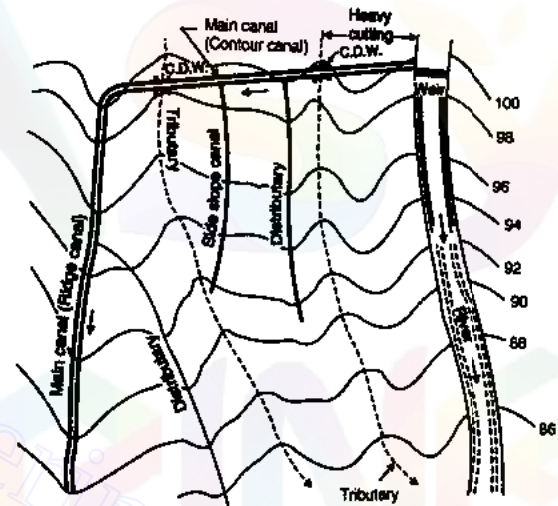


FIG. 13.1. CANAL ALIGNMENT.

1. Ridge Canal

A ridge canal or a watershed canal is aligned along a watershed and runs for most of its length on a watershed. When a channel is on the watershed, it can command areas on both banks and hence a large area can be brought under cultivation. Also, no drainage can intersect a watershed and, hence, the necessity of constructing cross drainage works are obviated.

When the watershed takes a very sharp loop, the canal should be aligned straight to save considerable idle length, as shown in Fig. 13.2. The area of the loop between the watershed and the canal cannot be irrigated by this canal as there will be higher the watershed and some other arrangement has to be made if the irrigation ground on this side and some other arrangement has to be made if the irrigation is important in that area. The canal checks the drainage of this part and a cross-drainage work has, therefore, to be provided. A cost analysis should, therefore, be made before

aligning the canal straight to reduce its length. The canal has also to leave watershed to by-pass towns and villages situated at the watershed.

2. Contour Canal

A channel aligned nearly parallel to the contours of the area is called a contour canal.

When the canal takes off from a river in a hilly area, it is not possible to align the canal on the watershed as the watershed on the top of the hill may be very high and the areas which need irrigation are concentrated in the valley. The canal is then aligned roughly parallel to the contours of the area. The contour chosen for the alignment should be so placed as to include all culturable area of the valley on one side of the canal.

The contour canal can irrigate only on one side. As the ground level on the other side is quite high, there is no necessity of a bank on this side. Hence, a contour canal is sometimes constructed with one bank only, and is known as a *single bank canal*. However, when both the banks are provided, it is known as a *double bank canal*.

The contour canal does not follow the same contour all along. To enable the water to flow by gravity, some surface slope is given. The rate at which the canal alignment leaves one contour and takes up another depends upon its surface slope. It is usual, in highly undulated tracks, to carry the channel in deep cutting across the ridges or spurs and in high embankments across the valleys in order to reduce the unnecessary length of the channel in long detours and sharp curves in the alignment, as shown in Fig. 13.1.

3. Side Slope Canal

It is a channel aligned roughly at right angles to the contours of the country and is neither on the watershed nor in the valley. Such a channel would be roughly parallel to the natural drainage of the country and, hence, it does not intercept any cross-drainage. However, it has very steep bed slope, since the direction of the steepest slope of the ground is at right angles to the contours of the country.

General considerations for alignment

1. The alignment of the canal should be such as to ensure (i) the most economical way of distributing the water to the land, (ii) as high a command as possible, and (iii) minimum number of cross drainage works.
2. The alignment of a canal on a watershed, being the most economical, is preferred. As a general rule, all the watershed lying in a command should be occupied by distributaries.
3. The length of the main canal from the point where it takes off from a river to a point where it mounts on a watershed should be minimum.

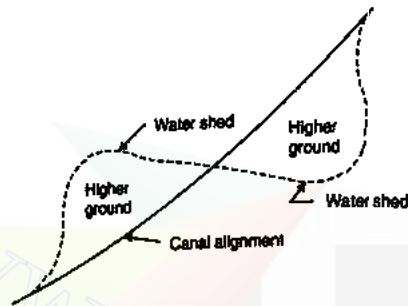


FIG. 13.2 CANAL ALIGNMENT FOR SHARP TURNS IN WATERSHED.

4. The contour alignment should be changed this way or that way in order to reduce the number of cross-drainage works to a minimum.
5. The alignment should avoid villages, roads, cart tracks, cremation places, places of worship and other valuable properties.
6. The alignment should pass through the balanced depth of cutting. If not, it should involve minimum depth of cutting or minimum height of filling.
7. The number of kinks and acute curves should be minimum.
8. Idle length of canal should be minimum and branches etc. should be economically planned.
9. The alignment should not be made in a rocky, brackish or cracked strata.

Alignment of a field channel or water course :

Though the maintenance of a field channel is the responsibility of the farmers, its alignment should have the following features :

1. They should be laid along field boundaries.
2. They should be capable of supplying sufficient water to the tail end.
3. Separate field channels should be provided for high and low lands.
4. The field channels should not pass through rocky, brackish, or cracked strata.

13.3. CURVES

Curves should not be provided along the length of a canal except where necessary. Introduction of a curve in a channel disturbs the regime of channel. The concave side is always under erosion and the convex side has a tendency to silt.

If the provision of curve is a must, minimum radius should be provided as shown in Table 13.1.

TABLE 13.1

Capacity of channel (cumecs)	Minimum radius of the curve (metres)
Less than 0.3	100
0.3 to 3	159
3 to 15	300
15 to 30	600
30 to 85	900
over 85	1500

13.4. INUNDATION CANALS

Introduction of inundation canal in a particular area is resorted under the following situations :

1. When the flood in the river is of such a duration that the water level in the canal remains high and fairly constant for quite a long time.
2. When there is an early flood in the river so that Kharif crops can be irrigated.
3. When the river has a late last flood at the end of rainy season so that Rabi crops can be irrigated.

4. When the area to be irrigated is in close proximity of the canal system so that a large area is not submerged in case there is a worst flood in the river.
5. When the soil has a sufficient self-stabilising power so that bed and banks of the canal remain stable.

In general an inundation canal is similar to a permanent canal. However, the major difference between them is that while in the case of a permanent canal a headwork (such as a weir, barrage or a dam) is constructed, no such works are needed for an inundation canal. But instead, an open cut in the river bank is provided for the inundation canal to take off from the river. The bed level of the cut is kept much above the bed level of the river.

Site selection for the offtake of inundation canals

The selection for the offtake point is very important because the success of inundation canal mostly depends upon it. Following are the main points to be considered for the site selection :

1. At the side of offtake, river course should be straight, and the banks should be stable and high. The river should also flow with an average velocity and should have normal width. The variation of water level should also be low. In case the offtake has to be located on a curve it should be located on the outer side of the curve (i.e. the concave side) so that there is no problem of silting in head reaches.

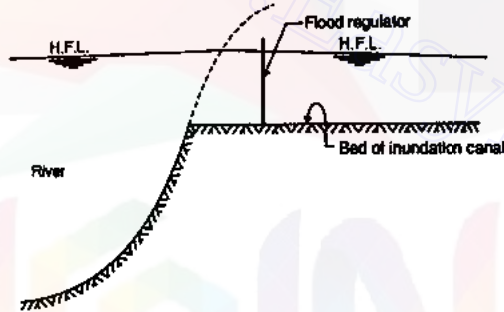


FIG. 13.3. INUNDATION CANAL.

2. The offtake should be located as near the area to be irrigated as possible.
 3. If a bypass of a river is available the offtake should be located at the downstream end of the bypass. There may be less of silt trouble in this case since the velocity in the bypass will be almost equal to that required in the inundation canal.
- Presence of a shoal creates a still pond in the pocket and thus the silt entry is restricted and reduced.

Design considerations for inundation canals

The trouble in the inundation canal arises due to lack of control of water levels, causing deposits of sand and formation of bar at mouth with a sudden fall in the river and sanding the canal. Many a times, the flood water finds its way through the canal and submerges vast stretch of land.

Design considerations for inundation canal are as follows :

1. A head regulator, generally known as *flood regulator*, may be provided a few miles downstream of the offtake point. If the flood regulator is provided at the offtake point, there is a danger of the same being washed away during floods.

2. An escape channel, with an off-take at d/s of the flood regulator and joined to the river may be provided to discharge the flood water back into the river when the flood regulator is closed or to divert the surplus water.
3. Vertical lift gates in tiers may be provided at the flood regulator which may be helpful in excluding silt.
4. Full supply level of inundation canal should be kept equal to the fair irrigating level. It is kept at a level at which water is steady in the river for 40 to 50 days during an inundation season.
5. Canal bed level is kept as low as practicable to take advantages of the depth of water for more period even when the river is in low stage.
6. Full supply discharge of the canal is fixed, based on the requirements of Rabi and Kharif crops.
7. Bed slope may be fixed depending upon the slope of the country, keeping in view that non-scouring velocities are not generated. The slope of the canal bed is generally in the range of 0.10 to 0.25 m/km.
8. Bed width may be calculated thereafter. The cross-section may be kept liberal. B/D ratio is kept small. Inundation canal is made deeper and narrower than other irrigation channels.

Advantages of inundation canal

The advantages are as follows :

1. The cost of construction for inundation canal system is very low because of absence of a headwork.
2. Since the silt and fine sand find its way into the canal, the water carried by inundation canal system has a good manurial qualities.
3. Since the water supply in the canal is intermittent and, hence, irrigation in the particular area is also intermittent. Therefore, there is less trouble of waterlogging and other bad effects of over irrigation.

Disadvantage of inundation canal

The disadvantages are as follows :

1. As there is no headwork the head of the canal is liable to be washed away during floods.
2. The water supply available from canal is not reliable. As water is available only during floods, there is no water for irrigation during other seasons.
3. Duty achieved by inundation canal water is low.
4. The cultivator lacks interest in his work due to unreliable supply.
5. The inundation canal is constantly under silt trouble.
6. The size of the canals will have to be of larger dimensions to admit the full requirement of water within a limited time factor.
7. The cross-section of the canal is not well defined. Thus there is very rough handling of water.

8. The channel alignment is also not very precise. Many a time there are sharp bends at which there may be silting or scouring.

Maintenance of the inundation canal system

There may be a lot of silting trouble in an inundation canal in the head reaches. Following provisions are made in order to maintain the inundation canal system.

1. Construction of *subsidiary heads* so as to supply additional water when the main heads get silted up.
2. Construction of *feeder canal which links up* a number of inundation canals taking off from the same river. Thus the necessity of constructing many subsidiary heads are avoided and the advantages of a good head can be shared with other canals.
3. Building a head regulator with an escape a few km downstream of the take-off. This will permit only required amount of water to get into the canal. The surplus water may enter into escape and is discharged back into river.

13.5. BANDHARA IRRIGATION

Bandhara is a special type of irrigation scheme, somewhere in between inundation type and permanent type of irrigation. It is essentially a minor irrigation scheme wherein small streams which otherwise allow their flow to be wasted are dammed at places by Bandharas (small weirs), and canals are taken off from them for irrigation of small areas.

A series of such dams or bandharas are constructed and the water available in the monsoon is thus made use of. The Bandhara irrigation scheme is very economical. The irrigation capacity of each Bandhara may vary from a few hectares to about 400 hectares.

Location of Bandharas

1. The river should preferably be a perennial one so that water is available from the river all through the year.
2. At the site, good foundation should be available for the construction of Bandhara.

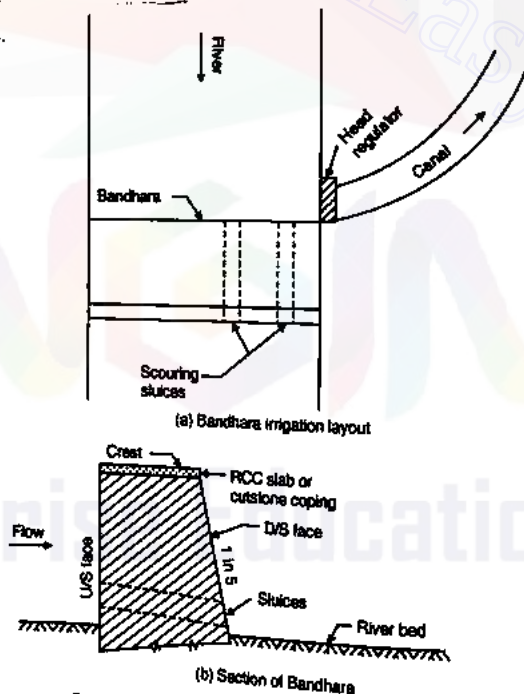


FIG. 13.5. BANDHARA IRRIGATION SYSTEM.

3. The section at the site should be straight, narrow and well defined.
4. The off-taking canal from such a site should have a good command.
5. Culturable commanded area so available should be large and fertile.
6. The natural banks on both sides of the site should be high so that no land is submerged on the upstream side.
7. The canal taking off should not interfere with important drainage of the country.
8. The cost of construction of the works should be low.

Working of Bandhara System

The Bandharas are always constructed in series so that water-spilling over from one Bandhara is checked by the other Bandhara. Each Bandhara and canal system taking off from the Bandhara forms one complete unit. The canal usually takes off from one side and the water in the canal is controlled by a head regulator. The length of the canal system should not be more than 8 km. The canal terminates in a low lying area commanded by Bandhara. This low lying area is termed as *Thal*. Once this type of system is introduced and developed, the farmers can maintain it themselves.

Design of Bandhara

1. Bandhara should be designed as a low dam. It is usually trapezoidal in shape with upstream face vertical and downstream face having a slope of 1 in 2 to 1 in 5.
2. Crest width of Bandhara should be kept equal to $\sqrt{H}$, where H is the height of Bandhara, with a minimum of 1.2 m.
3. Discharge flowing over Bandhara is given by $Q = 1.7 L h^{3/2}$, as the section is considered as a broad crested weir. (L is the length of Bandhara, h is the height of water above the crest of Bandhara in metres and Q is in cumecs.)
4. Scouring sluices of $1 \text{ m} \times 1.2 \text{ m}$ should be provided at the bottom at suitable intervals to clear silt.
5. The section of the Bandhara shown in Fig. 13.5 is checked for the following conditions :

- (a) When there is flow in the river, there will be afflux on the upstream and, hence, the section should be checked for the differential head.
- (b) When there is no flow in the river, water will be upto the crest level of Bandhara on upstream and there will be no water on downstream.

Phad System of Irrigation

The irrigable area under Bandhara is called *Thal*. A *thal* may be divided into *phads* or small blocks. One *phad* may contain a number of fields owned by a number of cultivators but the crops grown upon all the fields of a *phad* will be the same in a particular year. Thus if there are four *phads* in a *thal*, there will be four types of crops grown in a *thal*. The crop grown in a *phad* may be changed from time to time to give rotation to the crops.

Advantages of Bandhara System

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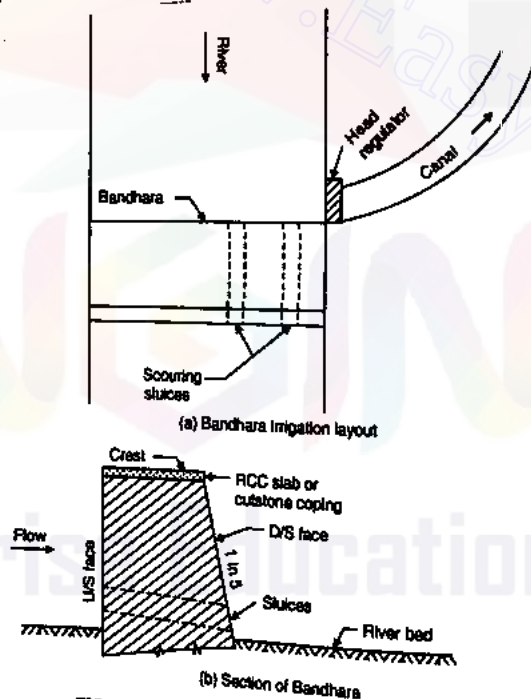


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Advantages of Bandhara System

1. The system of irrigation has a low initial cost.

2. Small quantities of water which would have otherwise gone waste is utilised to a maximum in this system.
3. Length of canal and distribution system being small, seepage and evaporation losses are very less.
4. The area to be irrigated being close to the source, it yields a high duty and intensive irrigation.

Disadvantages of Bandhara System

1. The irrigable area for one Bandhara is more or less fixed and, hence, even if greater quantity of water is available for irrigation it goes waste.
2. If the river is of non-perennial type, the supply of water becomes seasonal and unreliable in summer.
3. Small sources of water are dammed up for irrigation, hence the population living on the downstream of Bandhara which depends upon the river or stream for water supply for their domestic needs goes without water during dry periods.

Suitability of Bandhara System

Bandhara system is suitable for isolated catchments. It is also very suitable in tracts where there are number of small rivers and streams.

PROBLEMS

1. Explain various types of canals, according to various classification systems.
2. Explain various consideration for alignment of a canal.
3. Distinguish between a contour canal and a ridge canal.
4. Write a note on inundation canal.
5. Describe the Bandhara irrigation system.

Irrigation Channels - I : Silt Theories

14.1. TYPE OF CHANNELS

The hydraulic design of a channel or irrigation canal is governed mainly by two factors : (i) quantity of silt in the water carried by the channel and (ii) type of boundary surface of the channel. Based on these, we may have three types of channels/canals :

1. Alluvial channels
2. Non-alluvial channels
- and 3. Rigid boundary channels or lined channels.

Alluvial channels are those which are excavated in alluvial soils, such as silt, and which carry a lot of silt along with water. The boundary or the perimeter of such a channel is therefore made of silt, commonly known as 'alluvium'. The silt content of water flowing in such a channel may vary along its length. The silt content may increase if the velocity of flow is such that it scours the bed and sides of the canal. Similarly the silt content of water may decrease if the velocities are such that 'silting' (or settlement of silt) takes place. The aim of the designer would be to design the channel in such a way that a non-silting non-scouring velocity is obtained for the given design discharge.

Non-alluvial channels are those which are excavated in non-alluvial soils (such as loam, clay, hard soil, rock etc). Such channels generally do not have silting problems as water can flow with *non-silting velocities* (i.e. higher velocities) without fear of scouring of sides and bed of the canal.

Rigid boundary channels are those which have sides and bottom made of rigid material. All lined canals fall under this category.

This Chapter deals with the design of alluvial and non-alluvial channels, while the design of rigid boundary channel (i.e. lined canal) has been given in Chapter 16.

14.2. DESIGN OF ALLUVIAL CHANNELS

The channel or canal which takes off from a river has to draw a fair share of silt flowing in the river. This silt is carried either in suspension or along the bed of the canal. The silt load carried by the channel imposes a difficult problem in a channel design in alluvial soils. The velocity to be allowed in a channel design should

be such that the silt flowing in the channel is not dropped on the bed. In case a channel silts up, its capacity reduces and so it will irrigate less area. Also the velocity should not be large enough to erode away the bed and sides of the channel. If the sides and bed of a channel are eroded away, the cross-section increases and besides other damages because of scour, its full supply depth decreases; it can, therefore, command much less area. A velocity which will just keep the silt in suspension, without scouring the channel is known as *non-silting and non-scouring velocity*.

Many investigators have worked on various existing channels towards the design of non-silting-non-scouring channel section. Notable amongst them are works done by Mr. R.G. Kennedy of Punjab Irrigation Department and Mr. Gerald Lacey, Chief Engineer Irrigation Department U.P. The ideas propagated by them have come to be known as Kennedy's theory and Lacey's theory respectively.

For the design of an irrigation channel, the design discharge Q , and the surface and soil properties such as the rugosity coefficient N and silt factor f are known. The problem consists in determination of the *four unknowns* :

- (i) Area of cross-section (A),
- (ii) Hydraulic mean depth (R),
- (iii) Velocity of flow (V),

and (iv) Bed slope (S). To start with, following two equations are available :

$$Q = A \times V \dots (\text{Continuity equation}) \dots (1)$$

$$\text{and } V = f(N, R, S) \dots (\text{Flow equation}) \dots (2)$$

The flow Eq. (2) may be Manning's equation, or Kutter's equation or any other similar flow equation. However, since there are four unknowns, two more equations must be available for the complete and unique solution. The additional two equations may be obtained from the following criteria :

- (i) Providing channel of best discharging section thus getting another equation between A and R .
- (ii) Limiting equation of velocity from considerations of scouring and silting.
- (iii) Governing the slope by the available ground slope.
- (iv) Fixing a suitable B/D ratio on experience.

Criterion (i) is not good for alluvial soil where a non-silting non-scouring velocity is a must. Criteria (ii) and (iii) are utilised by Kennedy's method of design of channels. Wood's Table gives a table of suitable B/D ratio for various discharges. However, Lacey's theory furnishes four equations for the complete determination of the four unknowns, without depending upon the earlier flow equation by Manning or Kutter.

14.3. KENNEDY'S THEORY

Kennedy selected a number of sites on Upper Bari Doab Canal System, one of the oldest in Punjab (Pakistan) for carrying out investigations about velocity and depth of the channel. The sites selected by him did not require any silt clearance for more than thirty years and were thus supposed to be flowing with non-silting non-scouring velocity.

Kennedy's study revealed the following :

1. The flowing water has to counteract some amount of friction against the bed of the canal. This gives rise to vertical eddies rising up gently to the surface. These eddies are responsible for keeping most of the silt in suspension. Some eddies may start from sides but these are for most of its part horizontal and so do not have any silt supporting power. The *silt supporting power* is, therefore, proportional to the bed width of the stream and not to its wetted perimeter.

2. He also defined critical velocity as non-silting non-scouring velocity and gave a relation between critical velocity to the depth of flowing water. The relation is

$$V_0 = 0.55 D^{0.64} \dots (14.1)$$

$$\text{In general } V_0 = CD^n \dots (14.1 b)$$

where V_0 = Critical velocity,

D = Depth of water over bed portion of a channel.

n = Any index number.

Since the equation has been derived on the basis of observations on one canal system only, it is applicable to only those channels which are flowing in sandy silt of the same quality or grade as that of Upper Bari Doab Canal System.

Kennedy later realised the importance of silt grade on critical velocity and introduced a factor m known as *critical velocity ratio* (C.V.R.) in his equation. The equation is then written as

$$V_0 = 0.55 m D^{0.64} \dots (14.2)$$

$$\text{where } m = \text{C.V.R.} = \frac{\text{critical velocity for the area}}{\text{critical velocity for Upper Bari Doab canal system}} \dots [14.2(a)]$$

Sand coarser than the standard were assigned value of m from 1.1 to 1.2 and those finer than the standard from 0.9 to 0.8. Generally, in a system of canal, higher C.V.R. is assumed in head reaches and lower value of C.V.R. is assumed towards its tail end.

The value of constant C in Eq. 14.1(b) for various grades of material may be assumed as follows :

TABLE 14.1. VALUE OF C

Types of material	Value of C
Light Sandy silt	0.53
Coarser light silt	0.59
Sandy loam	0.65
Coarse silt	0.70

The value of m in Eq. 14.2 for different types of silts are given in Table 14.2.

TABLE 14.2. VALUE OF m

Type of silt	Value of m
1. Light sandy silt in the rivers of Northern India	1.00
2. Somewhat coarser light sandy silt	1.10
3. Sandy, loamy silt	1.20
4. Rather coarser silt or debris of hard soil	1.30
5. Silt of river Indus in Sind	0.70

Kennedy made use of Kutter's equation for finding the mean velocity of flow in the channel :

$$V = \frac{23 + \frac{1}{N} + \frac{0.00155}{S}}{1 + \left(23 + \frac{0.00155}{S}\right) \frac{N}{\sqrt{R}}} \sqrt{RS} = C \sqrt{RS} \quad (\text{in metric units}) \quad \dots(14.3)$$

Thus the limitations of Kutter's equation became incorporated in Kennedy's design procedure.

Fixation of longitudinal slope (S) of channel : Kennedy did not give any equation for the slope of the canal. The slope is decided according to the slope of the ground available. From the plotted longitudinal slope of the alignment, there is the ground available. From the plotted longitudinal slope of the alignment, there is always a certain range of slope, that can be given to a channel at the site. For every such slope, a separate channel can be designed to suit a given discharge. For example, when $Q = 2$ cumecs, $N = 0.0225$ and $m = 1$, we get the following channel dimensions, for various slopes :

Slope	B (m)	D (m)	B/D
1 in 5000	7	0.68	10.8
1 in 4000	3.2	0.65	3.8
1 in 2000	1.5	1.4	1.07

Unfortunately, Kennedy's theory provides no clue as to which of channels will be best suited for a particular discharge and type of soil available. Hence some guidance is necessary to find out as to which channel would suit the need best. To provide this guidance, suitable B/D ratios were fixed in various State Departments on the basis of experience.

Fixation of B/D ratio

(a) **Woods Table :** Wood's normal table is one such criterion which was prepared in Punjab and is very much in use. An extract of Wood's table, alongwith its metric conversion, is given in Table 14.3.

TABLE 14.3. WOOD'S NORMAL DESIGN TABLE : $N = 0.0225$

Discharge		B/D ratio	B		D		Slope 1 in...	V/V ₀	Mean V	
cusecs	cumecs		ft	m	ft	m			ft. / sec.	m. / sec.
10	0.283	2.9	4.75	1.45	1.60	0.49	8333	0.92	1.13	0.344
25	0.708	3.4	7.25	2.21	2.15	0.66	3636	1.01	1.39	0.424
50	1.416	3.7	10.25	3.13	2.75	0.84	4000	1.00	1.56	0.476
100	2.832	4.2	14.50	4.42	3.40	1.04	4444	1.00	1.82	0.555
250	7.079	4.6	22.00	6.70	4.70	1.43	4444	1.01	2.30	0.700
500	14.158	5.7	32.00	9.75	5.65	1.72	5000	1.00	2.54	0.775
1000	28.315	7.6	50.00	15.24	6.50	1.98	5000	1.08	2.89	0.880
2000	56.630	11.3	83.50	25.41	7.40	2.26	5714	1.03	3.10	0.945
5000	141.575	22.5	185.00	56.50	8.20	2.50	6666	0.98	3.20	0.975
10000	283.150	41.0	345.00	106.00	8.50	2.68	6666	1.02	3.38	1.030
20000	566.300	78.0	596.00	212.00	8.90	2.72	8000	0.94	3.20	0.975

Thus, if B/D ratio is selected from the recommended values, one more equation will be available. Hence unique design of the channel can be done, including the bed slope using Kennedy's theory. This has been illustrated in Case 2 of § 14.4 and in Example 14.2.

(b) Empirical Formula and Table for B/D ratio

For channels having $Q < 15$ cumecs :

$$D = 0.5 \sqrt{B} \quad \dots(14.4)$$

However, for channels having $Q \geq 15$ cumecs, the water depth may be selected from the following table (Table 14.4) :

TABLE 14.4. EMPIRICAL RECOMMENDATIONS OF WATER DEPTH (D)

Discharge (cumecs)	Depth D (m)
15	1.7
30	1.8
75	2.3
150	2.6
300	3.0

(c) Recommendations of Central Water Power Commission (C.W.P.C) for B/D ratio

C.W.P.C. has given a graph for B/D ratios for canals carrying discharges upto 300 cumecs, as shows in Fig. 14.1.

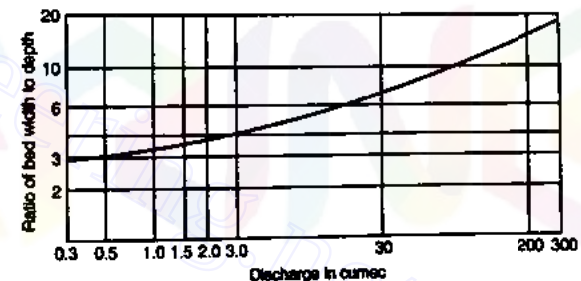


FIG. 14.1. C.W.P.C. RECOMMENDATIONS FOR B/D RATIO.

The above curve can be represented by the following equation :

$$\frac{B}{D} = r = [15 + 6.44 Q]^{0.282} \quad \dots(14.5)$$

For example, when $Q = 14$ cumecs, we get

$$\frac{B}{D} = [15 + 6.44 \times 14]^{0.282} \approx 5.9$$

and when $Q = 28$ cumec, we get

$$\frac{B}{D} = \{15 + 6.44 \times 28\}^{0.382} = 7.5 \text{ m}$$

These values are quite close to the one given in Wood's Table (Table 14.3)

14.4. KENNEDY'S METHOD OF CHANNEL DESIGN

For the design of channel, the following equations are used :

Continuity Equ. : $Q = \text{Area} \times \text{Velocity}$... (1)

$$\text{Kutter's Equ. : } V = \frac{23 + \frac{1}{N} + \frac{0.00155}{S}}{1 + \left(23 + \frac{0.00155}{S}\right) \frac{N}{\sqrt{R}}} \sqrt{RS} = C \sqrt{RS} \quad \dots (2)$$

$$\text{Kennedy's Equ. : } V_0 = 0.55 m D^{0.64} \quad \dots (3)$$

Buckley's recommendation for N

The value of rugosity coefficient N for earthen channels of North India, as suggested by Buckley are given in Table 14.11.

CWPC Recommendations for N

Table 14.5 gives the Central Water and Power Commission (CWPC) practice of selecting Kutter's coefficient N for the canals taking off from reservoirs carrying silt free water.

TABLE 14.5. CWPC PRACTICE FOR N

Type of Soil	Canal Discharge (cumecs)	Value of N
1. Soil other than rock	upto 0.14	0.03
	0.14 to 1.4	0.025
	1.4 to 14	0.0225
	Above 14	0.020
2. Rocky cuts	(i) When rock portion at least 15 cm above the excavated bed level is left out in working out cross-sectional area.	0.035 - 0.05
	(ii) When no portion above bed level is left out	0.05 - 0.080

Recommendations of U.P. Irrigation Department for N

The Irrigation Department of Uttar Pradesh adopts the following value of N :

Discharge (cumecs)	N
upto 14	0.0225
above 14	0.020

Value of N based on nature or condition of channel surface :

It should be noted that the value of N also depends upon the nature or condition of the channel surface, as is clear from Table 14.6.

TABLE 14.6. VALUE OF N

Nature or condition of channel surface	Value of N			
	Perfect	Good	Fair	bad
1. Alluvial Regime	0.017	0.020	0.0225	0.025
2. Loose gravel	0.0275	0.030	0.033	0.035
3. Rock cut, smooth and regular	0.025	0.030	0.033	0.035
4. Rock cut, irregular	0.035	0.040	0.045	0.050
5. With cobble bed and earthen banks	0.025	0.030	0.035	0.040
6. With earthen bed and rubble sides	0.025	0.030	0.033	0.035

Indian Standard Recommendations for rugosity coefficient N

Table 14.7 gives the values of rugosity coefficient N , as recommended by Indian Standard IS : 7112-1973.

TABLE 14.7. VALUES OF N FOR UNLINED CHANNELS (IS : 7112-1973)

Description of channel	Value of N		
	Minimum	Normal	Maximum
(1) Earth straight and uniform			
(a) Clean, recently complete	0.016	0.018	0.020
(b) Clean, after weathering	0.018	0.022	0.025
(c) Gravel, uniform section clean	0.022	0.025	0.030
(d) With short grass few, weeds	0.022	0.027	0.033
(2) Earth winding and sluggish			
(a) No vegetation	0.023	0.025	0.030
(b) Grass, some weeds	0.025	0.030	0.033
(c) Dense weeds or aquatic plants in deep channels	0.030	0.035	0.035
(d) Earth bottom and rubble sides	0.030	0.035	0.040
(e) Stone bottom and weedy banks	0.025	0.035	0.040
(f) Cobble bottom and clean sides	0.030	0.040	0.050
3. Dragline excavated or dredged			
(a) No vegetation	0.025	0.028	0.033
(b) Light brush on banks	0.035	0.050	0.060
4. Channels not maintained (weed and brush growth)			
(a) Dense weeds, high as flow depth	0.050	0.080	0.120
(b) Clean bottom, brush on sides	0.040	0.050	0.080
(c) Same, highest stage of flow	0.045	0.070	0.110
(d) Dense brush, high stage	0.080	0.100	0.140

Note : (1) For normal alluvial soils, it is usual in India to assume a value of $N = 0.02$ for bigger channels ($Q > 15$ cumecs) and $N = 0.0225$ for smaller channels ($Q < 15$ cumecs)

(2) Suitable value of N should be adopted keeping in view the local condition and the above values as a guide.

DESIGN PROCEDURE

Case 1. Given Q , N , m and S (from L-section)

1. Assume a trial value of D in metres.

2. Calculate the velocity V_0 from the equation

$$V_0 = 0.55 m D^{0.64}$$

3. Get area of section A from the continuity Equation :

$$A = \frac{Q}{V_0}$$

4. Knowing D and A , calculate the bed width B from geometry of canal section.

The side slope of the canal in alluvial soil is assumed to be $\frac{1}{2}:1$ when the canal has run for some time.

$$A = BD + \frac{D^2}{2}$$

From which B can be calculated.

5. Calculate the perimeter and the hydraulic mean depth from the following relations:

$$P = B + D\sqrt{5}$$

$$R = \frac{A}{P} = \frac{BD + D^2/2}{B + D\sqrt{5}}$$

6. Calculate the actual mean velocity of flow (V) from Kutter's equation. If this value of velocity (V) is the same as V_0 found in step 2, the assumed depth is correct. If not, repeat the calculations with a changed value of D till the two velocities are the same.

Thus we get the solution for a given value of S . Case 1 is the most common one and its solution is normally done with the help of Garret's diagram. Garret's diagrams (Fig. 15.3) provide solution of Kennedy's regime equation and Kutter's flow equation for various values of slopes as explained in Chapter 15.

Case 2. Given Q , N , m and B/D ratio from Wood's table :

(1) Calculate A in terms of D .

$$\text{Let } \frac{B}{D} = x \quad \text{or} \quad B = Dx$$

$$A = BD + \frac{D^2}{2} = xD^2 + \frac{D^2}{2} = D^2(x + 0.5).$$

(2) The value of velocity V_0 is known in terms of D by Kennedy's equation.

$$V_0 = 0.55 m D^{0.64}$$

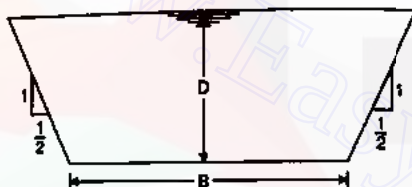


FIG. 14.2. CHANNEL SECTION.

Substitute the values of V_0 and A in the continuity equation and solve for D . Thus

$$Q = A \times V_0 = D^2(x + 0.5) \times 0.55 m D^{0.64}$$

or

$$Q = 0.55 m (x + 0.5) D^{2.64}$$

Hence

$$D = \left[\frac{Q}{0.55 m (x + 0.5)} \right]^{\frac{1}{2.64}}$$

In the above relation Q , m and x are known. Hence D is determined.

3. Knowing D , calculate B and R from the following relations :

$$B = xD$$

and

$$R = \frac{BD + D^2/2}{B + D\sqrt{5}}$$

4. Calculate the velocity V_0 from Kennedy's equation

$$V_0 = 0.55 m D^{0.64}$$

5. Knowing V_0 and R , determine the slope S from Kutter's flow equation. The equation can be solved by trial and error.

14.5. SILT SUPPORTING CAPACITY ACCORDING TO KENNEDY'S THEORY

According to Kennedy's theory, the amount of silt held in suspension is proportional to the upward force of vertical eddies, and varies as the bed width (B) and some power of the velocity of flow in the channel.

Let V_0 = critical velocity in the channel

Q_s = quantity of silt transported by the channel

Q = discharge in the channel.

$$Q_s \propto BV_0^\alpha$$

or

$$Q_s = \alpha B V_0^\alpha$$

...(i)

where α = some constant.

Let p = % of silt in water = Q_s/Q

$$Q_s = pQ$$

Assuming a wide channel with vertical slopes,

$$Q = B.D.V_0$$

(approx.)

$$Q_s = p.B.D.V_0$$

...(ii)

Equating (i) and (ii), we get

$$\alpha BV_0^\alpha = p.B.D.V_0$$

$$V_0^{\alpha-1} = \frac{1}{\alpha} p.D.$$

or

$$V_0 = \left(\frac{p}{\alpha} \right)^{\frac{1}{\alpha-1}} D^{\frac{1}{\alpha-1}}$$

662

or

$$V_0 = CD^{\frac{1}{n-1}} \quad \dots(iii)$$

This equation is similar to Kennedy's equation

$$V_0 = CD^{\frac{1}{n-1}} = CD^{0.64} \quad \dots(iv)$$

Equating the indices of both these equations, we get

$$\frac{1}{n-1} = 0.64$$

$$n = 2.56 \approx 5/2$$

Hence, the quantity of silt transported is given by

$$Q_s = \alpha B V_0^{5/2} \quad \dots(14.6)$$

Kennedy did not determine the value of the constant α .

Example 14.1. Design an irrigation channel on Kennedy's theory, to carry a discharge of 45 cumecs. Take $N = 0.0225$ and $m = 1.05$. The channel has a bed slope of 1 in 5000.

Solution

Step 1: Let us assume a trial depth $D = 2$ m

Step 2: $V_0 = 0.55$ m $D^{0.64} = 0.55 \times 1.05 (2)^{0.64} = 0.9$ m/sec.

Step 3: $A = Q/V_0 = 45/0.9 = 50$ m²

Step 4: For a trapezoidal section having $\frac{1}{2}:1$ side slopes,

$$A = BD + 0.5 D^2$$

$\therefore 50 = B(2) + 0.5(2)^2$, from which $B = 24$ m

Step 5: Perimeter $P = B + D\sqrt{5} = 24 + 2\sqrt{5} = 28.47$ m

$\therefore R = A/P = 50/28.47 = 1.756$ m

Step 6: Compute velocity of flow from Kutter's equation $V = C\sqrt{RS}$

where $S = 1/5000$

$$C = \frac{23 + \frac{1}{0.0225} + \frac{0.00155}{1} \times 5000}{1 + \left[23 + \frac{0.00155}{1} \times 5000 \right] \frac{0.0225}{\sqrt{1.756}}} = 49.4$$

and

$$V = 49.4 \sqrt{\frac{1.756 \times 1}{5000}} = 0.928 \text{ m/sec.}$$

Step 7: Thus $V_0 < V$. In order to increase V_0 , increase the value of D . Let $D = 2.2$ m in the next trial.

$$V_0 = 0.55 \times 1.05 (2.2)^{0.64} = 0.957 \text{ m/sec.}$$

$$A = 45/0.957 = 47.04 \text{ m}^2$$

$$47.04 = B(2.2) + 0.5(2.2)^2 \text{ from which } B = 20.28 \text{ m}$$

$$P = 20.28 + 2.2\sqrt{5} = 25.2 \text{ m}$$

$$R = 47.04/25.2 = 1.867 \text{ m}$$

$$C = \frac{23 + \frac{1}{0.0225} + \frac{0.00155}{1} \times 5000}{1 + \left[23 + \frac{0.00155}{1} \times 5000 \right] \frac{0.0225}{\sqrt{1.867}}} = 49.918$$

$$V = 49.918 \sqrt{\frac{1.867 \times 1}{5000}} = 0.965 \text{ m/sec.}$$

Ratio of the two velocities $= 0.965/0.957 = 1.008 \approx 1$.

Hence no further trial is necessary.

Thus, $D = 2.2$ m and $B = 20.28$ m ≈ 20.3 m

Note: If further trials are made, the final values will be $D = 2.23$ m and $B = 19.8$ m.

Example 14.2. Design an irrigation canal to carry a discharge of 14 cumecs.

Assume $N = 0.0225$, $m = 1$ and $B/D = 5.7$.

Solution:

$$1. \quad \frac{B}{D} = 5.7$$

$$\therefore A = BD + \frac{D^2}{2} = D^2 \left[\frac{B}{D} + 0.5 \right] = D^2(5.7 + 0.5) = 6.2 D^2$$

$$2. \quad V_0 = 0.55 \text{ m } D^{0.64} = 0.55 \times 1 \times D^{0.64}$$

$$Q = A \times V_0 = (6.2 D^2)(0.55 D^{0.64})$$

$$D = \left[\frac{Q}{0.55 \times 6.2} \right]^{\frac{1}{2.64}} = \left[\frac{14}{0.55 \times 6.2} \right]^{\frac{1}{2.64}} = 1.707 \approx 1.71 \text{ m.}$$

$$3. \quad B = 5.7 D = 5.7 \times 1.71 = 9.73 \text{ m}$$

$$\text{and } R = \frac{BD + D^2/2}{B + D\sqrt{5}} = \frac{(9.73 \times 1.71) + \frac{1}{2}(1.71)^2}{9.73 + 1.71\sqrt{5}} = 1.37 \text{ m.}$$

$$4. \quad V_0 = 0.55 \text{ m } D^{0.64} = 0.55 \times 1 \times (1.71)^{0.64} = 0.775 \text{ m/sec.}$$

$$5. \quad \text{But, } V = \frac{23 + \frac{1}{0.0225} + \frac{0.00155}{S} \times 5000}{1 + \left[23 + \frac{0.00155}{S} \times 5000 \right] \frac{0.0225}{\sqrt{R}}} \sqrt{RS} \text{ . Keeping } V = V_0, \text{ we get}$$

or

$$0.775 = \frac{23 + \frac{1}{0.0225} + \frac{0.00155}{S} \times 5000}{1 + \left[23 + \frac{0.00155}{S} \times 5000 \right] \frac{0.0225}{\sqrt{1.37 \times S}}} \sqrt{1.37 \times S}$$

$$\text{or } 0.775 = \frac{23 + 44.5 + \frac{0.00155}{S}}{1 + 0.44 + \frac{2.98}{S} \times 10^{-5}} \times 1.17 \times S^{1/2}$$

$$\text{which reduces to } \frac{0.775}{1.17} \left(1.44 + \frac{2.98}{S} \times 10^{-5} \right) = \left(67.5 + \frac{1.55}{S} \times 10^{-3} \right) S^{1/2}$$

$$\text{or } 67.5 S^{3/2} + 1.55 \times 10^{-3} S^{1/2} - 0.954 S = 1.98 \times 10^{-5}$$

$$\text{or } S^{3/2} + 2.29 \times 10^{-3} S^{1/2} - 1.41 \times 10^{-2} S = 2.93 \times 10^{-7}$$

Solving this by trial and error, we get $S = \frac{1}{5100}$

14.6. DRAWBACKS IN KENNEDY'S THEORY

- (1) Kennedy did not notice the importance of B/D ratio.
- (2) He aimed to find out only the average regime conditions for the design of a channel.
- (3) No account was taken of silt concentration and bed load, and the complex silt carrying phenomenon was incorporated in a single factor m .
- (4) Silt grade and silt charge were not defined.
- (5) Kennedy did not give any slope equation.
- (6) Kennedy used Kutter's equation for the determination of the mean velocity and, therefore, the limitations of Kutter's equation got incorporated in Kennedy's theory of channel design.

14.7. FURTHER WORK ON KENNEDY'S THEORY

1. Empirical formulae for other areas

Kennedy's equation could not be made universally applicable even after the introduction of critical velocity ratio. Regional formulae were thereafter developed to suit the requirements of different canal system by different investigators. In these formulae both the coefficient C as well as power of D were varied. A general feature of these formulae indicated a lower value of C for finer soils. The value of C and n in Eq. 14.1 (b) for different canal systems are as follows :

TABLE 14.8. VALUES OF C AND n

Serial No.	Canal system	C	n
1	Godavari Delta	0.391	0.55
2	Krishna Western Delta	0.530	0.52
3	Lower Chamb Canal	0.567	0.57
4	Egyptian Canals	0.283	0.73

2. Lindley's Formulae

It has already been emphasized that depending upon an available slope, many channels can be designed which have different cross-section. As to which of these channels will suit the requirement can not be found out from Kennedy's theory. In order to

make channel design more definite, Lindley of Punjab irrigation proposed his formulae in 1919. He proposed the following two formulae one relating V and D , similar to Kennedy and the other relating V and B .

$$V = 0.567 D^{0.57} \quad \dots[14.7(a)]$$

and

$$V = 0.274 B^{0.35} \quad \dots[14.7(b)]$$

By eliminating V from Eqs. [14.7 (a),(b)], another relation as originally proposed by Lindley can be obtained. This is given by

$$B = 7.80 D^{1.61} \quad \dots(14.7)$$

(3) Ranga Raju and Misri's method of canal design on Kennedy's theory

Ranga Raju and Misri (1979) gave a simplified method of a trapezoidal channel design on Kennedy's theory, if the slope is given. They used Manning's formula as flow equation. Assuming a trapezoidal channel section with $\frac{1}{2}:1$ side slopes finally attained by channels in alluvial soils, we have.

$$A = BD + 0.5 D^2 = D^2 (B/D + 0.5) = D^2 (r + 0.5) \text{ where } r = B/D \text{ ratio.}$$

and

$$P = B + D \sqrt{5} = D (B/D + \sqrt{5}) = D (r + 2.236)$$

Also,

$$R = \frac{A}{P} = \frac{D (r + 0.5)}{r + 2.236}$$

$$V = \frac{Q}{A} = \frac{Q}{D^2 (r + 0.5)} \quad \dots(i)$$

$$\text{But from Manning's formula, } V = \frac{Q}{A} = \frac{1}{N} R^{2/3} S^{1/2}$$

$$\left(\frac{Q}{A} \right)^2 = \frac{1}{N^2} R^{4/3} S$$

$$\text{From which } S = \frac{Q^2}{A^2} \cdot \frac{N^2}{R^{4/3}} = \frac{Q^2 N^2 (r + 2.236)^{4/3}}{[D^2 (r + 0.5)]^2 [D (r + 0.5)]^{4/3}}$$

or

$$S = \frac{Q^2 N^2 (r + 2.236)^{4/3}}{D^{14/3} (r + 0.5)^{10/3}} \quad \dots(ii)$$

$$\text{From Kennedy's basic equation, } V_0 = 0.55 m D^{0.64}$$

$$Q = V_0 \times A = 0.55 m D^{0.64} [D^2 (r + 0.5)]$$

or

$$D = \left[\frac{Q}{0.55 m (r + 0.5)} \right]^{1/2.64} = \left[\frac{1.818 Q}{m (r + 0.5)} \right]^{0.3750} \quad \dots(14.8)$$

Substituting this value of D in Equ. (ii), we get

$$S = \frac{Q^2 N^2 (r + 2.236)^{4/3}}{\left[\frac{1.818 Q}{m (r + 0.5)} \right]^{2.64} [r + 0.5]^{10/3}}$$

$$\text{or } \frac{SQ^{0.02}}{N^2 m^{2.02}} = \frac{0.299 (r + 2.236)^{4/3}}{(r + 0.5)^{1.513}} \quad \dots(14.9)$$

This is the basic relationship between unknown r and known parameters S , Q , N , m . Knowing the left hand side (for given values of S , Q , N and m), the value of r is found by trial and error.

Alternatively, a plot can be made between $SQ^{0.02}/N^2 m^{2.02}$ and r (Fig. 14.3). The value of r can be read from this plot, for known value of the factor $SQ^{0.02}/N^2 m^{2.02}$. Knowing, r , we get $B = rD$. Hence from Eq (i),

$$V = \frac{Q}{D^2 (r + 0.5)} \quad \text{Thus, } B, D \text{ and } V \text{ are, known.}$$

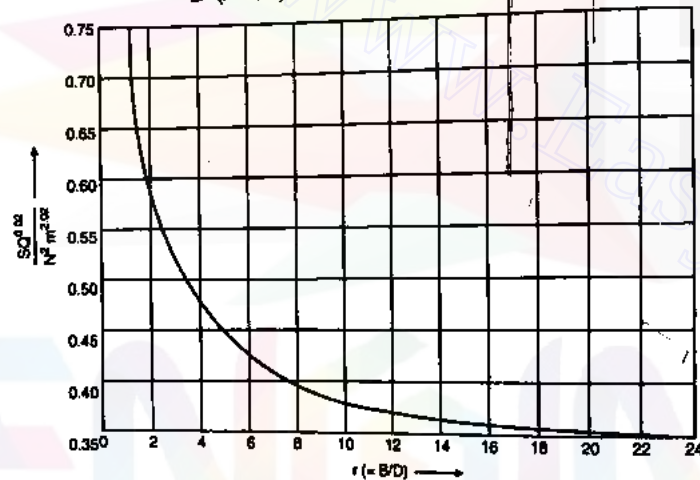


FIG. 14.3.

(4) Semi-empirical curve fitting method of canal design based on Kennedy's theory and CWPC recommendations for B/D ratio

For a trapezoidal section having side slopes $\frac{1}{2}:1$, we have

$$A = BD + 0.5 D^2 = D^2 (r + 0.5) \quad \text{where } r = B/D \text{ ratio.}$$

Also, from Kennedy's basic equation for velocity,

$$V_0 = 0.55 m D^{0.44}$$

Hence

$$Q = A \times V_0 = D^2 (r + 0.5) \times 0.55 m D^{0.44} = 0.55 m (r + 0.5) D^{2.44}$$

Hence

$$D = \left[\frac{Q}{0.55 m (r + 0.5)} \right]^{1/2.44} = \left[\frac{1.818 Q}{(r + 0.5) m} \right]^{0.373} \quad \dots(14.8)$$

From the above equation, one can compute both B and D , if suitable explicit equation in the following form is available :

$$r = f(Q, N, m, S)$$

One such correlation was obtained by Ranga Raju and Misri (in the form of Eq. 14.9), by using Manning's formula as flow equation; however, limitations of Mannings formula got involved in this equation. A large number of canal designs can be obtained by varying the parameters Q , N , m and S in their practical ranges and a suitable correlation can be established between r , Q , N , m and S . This was done by varying the parameters, r , Q , N , m and S in their practical ranges, thus obtaining 1296 number of canal designs which satisfy both Kennedy's as well as Kutter's equations. The resulting solutions so obtained were analysed and the following unique empirical relationship was obtained by a method of curve fitting :

$$r = \left[\frac{1.607 S^{1.83} Q^{0.033}}{N^{3.95} m^{3.298}} - 0.258 \right]^{-0.915} \quad \dots(14.10)$$

Thus, for a given set of values of Q , N , m and S , the value of r (i.e. B/d ratio) can be computed from Eq. 14.10. The value of B/D ratio so obtained can then be compared with the curve of Fig. 14.1 recommended by CWPC. If the matching differs by a wide range, a modification in the selected value of S becomes necessary, and this modified value of S can be found as under :

As stated earlier, the curve of Fig. 14.1 can be represented by the following empirical equation :

$$r = [15 + 6.44 Q^{0.382}] \quad \dots(14.5)$$

Taking the above equation as the optimal condition for the value of r and hence substituting the value of r in Eq. 14.10, we get the following relationship for S :

$$S = \frac{N^2 m^{2.02}}{1.338 Q^{0.02}} \left[0.258 + (15 + 6.44 Q)^{-0.417} \right]^{0.8135} \quad \dots(14.11)$$

From the above equation, the modified slope S is obtained which satisfies both Eqs. 14.11 and 14.5.

Design procedure

(a) Given : Q , N , m and S

Step (i) : Compute B/D ratio (i.e. r) from Eq. 14.10 which satisfies both Kennedy's equation as well as Kutter's equation.

Step (ii) : Compute D from Eq. 14.8.

Step (iii) : Find $B = rD$

See Example 14.4 for illustration.

(b) Given Q , N , m and CWPC curve for r

Step (i) : Find the required value of r either from the curve of Fig. 14.1 or from Eq. 14.5.

Step (ii) : Compute the optimal value of S from Eq. 14.11.

Step (iii) : Compute D from Eq. 14.8.

Step (iv) : Find $B = rD$

Example 14.3. Design a channel on Kennedy's theory, for the following data : $Q = 45$ cumecs, $N = 0.0225$, $m = 1.05$ and $S = 1/5000$. Use the method given by Ranga Raju and Misri.

Solution : Given $Q = 45$ cumecs, $N = 0.0225$, $m = 1.05$ and $S = 1/5000$.

Substituting the values in Eq. 14.9, we have

$$\text{L.H.S.} = \frac{SQ^{0.02}}{N^2 m^{1.49}} = \frac{1}{5000} \times \frac{(45)^{0.02}}{(0.0225)^2 (1.05)^{1.49}} = 0.3863$$

$$\text{Hence } \frac{0.299 (r + 2.236)^{1/3}}{(r + 0.5)^{1.315}} = 0.3863$$

From curve of Fig. 14.3, we get $r \approx 10$

Again solving the above equation by trial and error, starting with $r = 10$, we get $r = 9.7$

$$\text{Now, from Eq. 14.8, } D = \left[\frac{1.818 Q}{m(r + 0.5)} \right]^{0.264} = \left[\frac{1.818 \times 45}{1.05(9.7 + 0.5)} \right]^{0.264} = 2.16 \text{ m}$$

$$\text{and } B = rD = 9.7 \times 2.16 \approx 21 \text{ m}$$

$$V = \frac{Q}{D^2 (r + 0.5)} = \frac{45}{(2.16)^2 (9.7 + 0.5)} = 0.946 \text{ m/s}$$

Check $V_0 = 0.55 m D^{0.64} = 0.55 (1.05) (2.16)^{0.64} = 0.945 \text{ m/s}$. Hence OK.

Example 14.4. Using the method of curve fitting based on Kennedy's theory, design a channel for the following data :

$Q = 45$ cumecs, $N = 0.0225$, $m = 1.05$ and $S = 1/5000$

Solution

The ratio $r (= B/D)$ is given by Eq. 14.10 :

$$r = \left[\frac{1.607 S^{1.63} Q^{0.033}}{N^{2.26} m^{3.293}} - 0.258 \right]^{-0.915} = \left[\frac{1.607 (1/5000)^{1.63} (45)^{0.033}}{(0.0225)^{2.26} (1.05)^{3.293}} - 0.258 \right]^{-0.915} = \left[\frac{1.7031 \times 10^{-6}}{4.9876 \times 10^{-6}} - 0.258 \right]^{-0.915} = 9.7$$

From Eq. 14.8, we get

$$D = \left[\frac{1.818 Q}{m(r + 0.5)} \right]^{0.2786} = \left[\frac{1.818 \times 45}{(9.7 + 0.5) 1.05} \right]^{0.2786} \approx 2.16 \text{ m}$$

$$\text{Hence } B = rD = 9.7 \times 2.16 = 20.95 \approx 21 \text{ m}$$

Example 14.5. Solve example 14.4, using curve of CWPC for B/D ratio.

Solution : From the Eq. 14.5 of curve of Fig. 14.1, we have

$$r = (15 + 6.44 Q)^{0.362} = (15 + 6.44 \times 45)^{0.362} \approx 8.89$$

(Against the value of $r = 9.7$ obtained in the previous example)

The optimal value of S is computed from Eq. 14.11 :

$$S = \frac{N^2 m^{1.49}}{1.338 Q^{0.02}} \left[0.258 + (15 + 6.44 Q)^{-0.417} \right]^{0.6135} \\ = \frac{(0.0225)^2 (1.05)^{1.49}}{1.338 (45)^{0.02}} \left[0.258 + (15 + 6.44 \times 45)^{-0.417} \right]^{0.6135} = 1/4920$$

Also, from Eq. 14.8 :

$$D = \left[\frac{1.818 Q}{m(r + 0.5)} \right]^{0.2786} = \left[\frac{1.818 \times 45}{(8.89 + 0.5) 1.05} \right]^{0.2786} \approx 2.23 \text{ m}$$

$$\text{Hence } B = rD = 8.89 \times 2.23 = 19.82 \approx 20 \text{ m}$$

14.8. LACEY'S REGIME THEORY

"Dimensions, width, depth and slope of a regime channel to carry a given discharge loaded with a given silt charge are all fixed by nature." This idea was first put forward by Lacey. Lindley's theory is also based on the same concept. Lacey succeeded in evolving more generally applicable equations based on his own experiments and the experiments of the past investigators.

Regime channel. Lacey defined regime channel as a stable channel transporting a regime silt charge. A channel will be in regime if it flows in unlimited incoherent alluvium of the same character as that transported and the silt grade and silt charge are all constant.

Incoherent alluvium. It is a soil composed of loose granular graded material which can be scoured with the same ease with which it is deposited.

Regime silt charge. It is the minimum transported load consistent with fully active bed.

Regime silt grade. This indicates the gradation between the small and the big particles. It should not be taken to mean the average mean diameter of a particle.

Regime conditions. A channel is said to be in regime when the following conditions are satisfied.

1. The channel is flowing in unlimited incoherent alluvium of the same character as that transported.
2. Silt grade and silt charge are constant.
3. Discharge is constant.

If the above three conditions are met with fully, then the channel is said to be in *true regime*. However, it is seldom that the above conditions are realized in field. Hence, Lacey gave the idea of initial and final regime for actual channel.

Initial regime. One of the conditions of attaining regime of a channel is that there should be freedom for the channel to form its own section. Initial regime is the state of channel that has formed its section only and yet not secured the longitudinal slope.

Final regime. When a channel is constructed with defective slope, it tries to throw off the incoherent silt on the bed to increase their slopes. To attain the final regime, the channel forms its section first before the final slope. The channel after attaining its section and longitudinal slope will be said to be in final regime.

Permanent regime. When a channel is protected on the bed and side with some kind of protecting material, the channel section cannot be scoured up and so there is no possibility of change of section or longitudinal slope; the channel will then be said to be in permanent regime. Regime theory is not applicable to such channels.

There is only one section and only one longitudinal slope at which the channel will carry a particular discharge with a particular silt grade. Natural silt transporting channels have a tendency to assume a semi-elliptical section. The coarser the silt, greater is the waterway of such a channel and narrower the depth. The finer the silt, greater is the depth and the channel closely approximates a semi-circle.

If a channel is constructed with too small a cross-section for a particular discharge and the slope steeper than required, scour will occur till final regime is attained. Similarly silting will occur in a channel till final regime is attained in a channel with a wider cross-section and flatter slope than required.

Lacey also states that the silt is kept in suspension due to the force of vertical eddies. According to him, the eddies are generated from bed and sides, both normal to surface of generation. Hence, vertical component of eddies generated from sides will also support the silt. Lacey, therefore, assumed hydraulic mean depth (R) as variable, unlike Kennedy who assumed depth D as variable. Since Lacey assumed a semi-ellipse as the cross-section of a regime channel, assumption R as a variable seem to more logical.

Lacey's Regime Equations

On the basis of arguments mentioned above, Lacey plotted a large mass of data to obtain a relationship between (i) V vs. R and (ii) A vs. V . Lacey recognised the importance of silt grade in the problem and introduced a function f known as *silt factor* in the regime relationship. The first two equations, originally suggested by Lacey in F.P.S. units and converted in M.K.S. units, are given below :

$$V = \sqrt{\frac{4}{5} f R} \quad \dots(14.12)$$

$$A f^2 = 140.0 V^5 \quad \dots(14.13)$$

where A = area of the channel section and V = velocity of flow in it.

Regime Flow Equation

By plotting a large mass of data from a number of different sources, Lacey obtained the relationship

$$V = 10.8 R^{2/3} S^{1/3} \quad \dots(14.14)$$

where S = slope of water surface

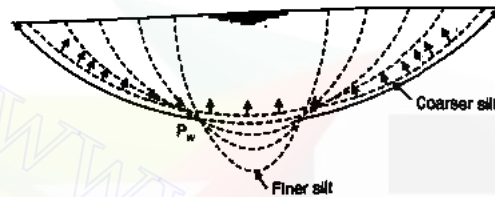


FIG. 14.4. CHANNEL SECTION ACCORDING TO LACEY'S THEORY.

This equation is very useful for finding velocities during high floods. A beginner has not to worry himself to find out the exact values of N in order to apply Manning's and Kutter's formulae. Equation 14.14 contains no silt factor and rugosity coefficient. Such variables were considered to be implicit in the actual depth and slope adopted by the channel in loose alluvium.

Eqs. 14.12, 14.13 and 14.14 are known as *Lacey's fundamental equations* from which other flow equations can be derived as shown below.

1. Perimeter Discharge (P - Q) Relation

By combining Eq. 14.12 and Eq. 14.13, a relationship between P and Q can be obtained.

Raising both sides of Eq. 14.12 to the fourth power, we get

$$V^4 = \frac{4}{25} f^2 R^2$$

Eliminating f^2 between this equation and equation 14.13, we have

$$A \left[\frac{25V^4}{4R^2} \right] = 140.0 V^5 \quad \text{or} \quad A \left[\frac{25}{4R^2} \right] = 140 V$$

Multiplying both sides of this equation by A , we get

$$\frac{25A^2}{4R^2} = 140VA = 140Q \quad \text{where} \quad \frac{A^2}{R^2} = P^2$$

$$\text{Hence} \quad P^2 = \frac{4 \times 140}{25} Q$$

or

$$P = 4.733 Q \approx 4.75 \sqrt{Q} \quad \dots(14.15)$$

Equation 14.15 is a very important equation in as much as it is universally true. Power of Q does not vary, though the coefficient 4.75 varies between certain range.

2. V - Q - f Relation

Multiplying Eq. 14.13 by V on both sides, we get

$$A V f^2 = 140.0 V^6$$

or

$$Q f^2 = 140.0 V^6$$

$$\therefore V = \left[\frac{Q f^2}{140.0} \right]^{1/6} \quad \dots(14.16)$$

This relation is utilized in channel design by Lacey's theory. The value of Q and f in a channel design are known. The value of V can, therefore, be readily found from Eq. 14.16.

3. Regime-Slope Equations: (S - Q - f), (R - S - f) and (S - f - q) relationships

From Eq. 14.14, cubing both sides of the equation

$$V^3 = 1260 R^{2/3} S \quad \dots(14.17)$$

From Eq. 14.12 cubing both sides of the equation

$$V^3 = (2/5)^{3/2} f^{3/2} R^{3/2}$$

Hence, $(2/5)^{3/2} f^{3/2} R^{3/2} = 1260 R^2 S$

or
$$S = \frac{f^{3/2}}{4980 R^{1/2}} = \frac{0.0002 f^{3/2}}{R^{1/2}} \quad \dots(14.18)$$

Equation 14.17 can also be written as

$$S = \left(\frac{V^2}{R} \right)^{5/3} \frac{1}{1260 (RV)^{1/3}} \quad \dots(14.19)$$

From Eq. 14.12, $V^2/R = \frac{2}{5} f$

Also $RV = q$ where q = discharge per unit width.

Hence
$$S = \left(\frac{2}{5} \right)^{5/3} \frac{f^{5/3}}{1260 q^{1/3}} = 0.000178 \frac{f^{5/3}}{q^{1/3}} \quad \dots(14.20)$$

The above equation can also be represented in terms of total discharge.
Rewriting equation 14.19,

$$S = \left(\frac{V^2}{R} \right)^{5/3} \frac{1}{1260 (RV)^{1/3}}$$

Putting

$R = A/P$, we get

$$S = \left(\frac{V^2}{R} \right)^{5/3} \frac{1}{1260 \left[\frac{A}{P} V \right]^{1/3}} = \left(\frac{V^2}{R} \right)^{5/3} \frac{P^{1/3}}{1260 (Q)^{1/3}}$$

But

$$P = 4.75 \sqrt{Q}$$

$$S = \frac{(2/5)^{5/3} f^{5/3} (4.75)^{1/3} Q^{1/6}}{1260 Q^{1/3}}$$

or

$$S = \frac{f^{5/3}}{3340 Q^{1/6}} = \frac{0.0003 f^{5/3}}{Q^{1/6}} \quad \dots(14.21)$$

4. Regime Scour Depth Relation

From Eq. 14.12, as already derived

$$R^2 = \frac{25 V^4}{4 f^2} \quad \text{or} \quad R = \frac{5 V^2}{2 f}$$

From Eq. 14.16, we have

$$V^2 = \left[\frac{Q f^2}{140} \right]^{1/3}$$

Hence

$$R = \frac{5}{2} \frac{Q f^2}{140} \frac{1}{f} = 0.47 (Q/f)^{1/3} \quad \dots(14.22)$$

Now

$$q = RV = \frac{5 V^3}{2 f}$$

Substituting V from Eq. 14.16, we get

$$q = \frac{5}{2} \frac{1}{f} \frac{Q f^2}{140} = 0.21 Q^{1/3} \quad \dots(14.23)$$

Substituting for

$$Q = \frac{q^3}{0.21^3} \quad \text{in equation 14.22, we get}$$

$$R = 0.47 \frac{1}{f^{1/3}} \frac{q^3}{0.21^3} = 1.35 \frac{q^3}{f} \quad \dots(14.24)$$

5. Lacey's Non-Regime Flow Equation

Eq. 14.14 is applicable only for regime flow. The equation to be derived now will be applicable for both regime and non-regime channels.

From Eq. 14.14,

$$V^3 = 1260 R^2 S \quad \text{or} \quad V = 35.5 (R V)^{2/3} \sqrt{RS} \quad \dots(14.25)$$

Equation 14.25 is similar to Chezy's equation, in which Chezy's coefficient C is represented by $35.5 (R/V)^{1/2}$.

Now putting $V = K \sqrt{R}$ in Eq. 14.12

Chezy's coefficient C may be expressed as

$$C = \frac{35.5 R^{1/4}}{K^{1/2} f^{1/4}}$$

Again putting $\frac{35.5}{K^{1/2} f^{1/4}} = \frac{K}{N_a}$...[14.25(a)]

where N_a = Lacey's absolute rugosity coefficient, Eq. 14.25 can be written as

$$V = \frac{K}{N_a} R^{1/4} S^{1/2} \quad \dots(14.26)$$

Lacey called N_a as absolute rugosity coefficient to emphasize that this N_a depends only upon grade and density of boundary material and is independent upon channel conditions. This reason could not be supported by observations.

Comparing it to Manning's equation

$$V = \frac{1}{N} R^{2/3} S^{1/2} \quad \dots(14.27)$$

$$V = \frac{1}{N_a} R^{3/4} (S - s)^{1/2} \quad \dots(14.30)$$

Thus, the value of N_a in the above equation remains independent of channel condition and has a value congruent to the value of N for a channel in 'very good' condition. Buckley recommends following value of N for earthen channels :

TABLE 14.11. VALUES OF N BY BUCKLEY

Channel condition	N
Very good	0.0225
Good	0.025
Indifferent	0.0275
Poor	0.03

The value of slope for these channel conditions can be computed from Eq. 14.28 and 14.30 as indicated below. For channel in *very good condition*, $N = 0.0225$ and $s = 0$. For channel in *good condition*, $N = 0.025$. Assuming $N_a = N$, we get the following values of velocities:

$$\text{From Eq. 14.28, } V = \frac{1}{N_a} R^{3/4} S^{1/2} = \frac{1}{0.025} R^{3/4} S^{1/2}$$

$$\text{From Eq. 14.30, } V = \frac{1}{N_a} R^{3/4} (S - s)^{1/2} = \frac{1}{0.0225} R^{3/4} (S - s)^{1/2}$$

Equating the two velocities, we get

$$\frac{1}{0.025} R^{3/4} S^{1/2} = \frac{1}{0.0225} R^{3/4} (S - s)^{1/2}$$

$$(S - s)^{1/2} = 0.9 S^{1/2}$$

or

$$\text{From which } s = 0.19 S \quad \dots(14.31)$$

This means that for a channel in good conditions, 19% of slope is accounted for by shock losses and 81% is available for frictional resistance. Similarly the shock losses for indifferent and poor conditions of a channel can be calculated. The value of shock losses for these channels respectively account for 33.1% and 43.7% of the gross slope available.

Thus, the values of s due to shock losses for channel with different conditions are as under :

(i) Channel in very good condition: $s = 0$

(ii) Channel in good condition: $s = 0.19 S$

(iii) Channel with indifferent condition: $s = 0.331 S$

and (iv) Channel with poor condition: $s = 0.437 S$

Lacey's idea of 'shock' was unique, as the same brought the idea of form resistance coming into play in flowing mass of water. However, shock losses cannot be actually found out and, hence, equation 14.30 cannot be used in practice. Therefore, different values of N_a ($= N$ value recommended by Buckley) has to be used for different channel conditions.

14.9. LACEY'S THEORY APPLIED TO CHANNEL DESIGN

The channel section can be designed on Lacey's theory by using various equations developed above. The design procedure involves no trial and error steps. For the channel design, the discharge Q and mean diameter of silt particles m_d or silt factor f should be known.

Channel design procedure

$$1. \text{ Calculate the silt factor } f = 1.76 \sqrt{m_d}$$

$$2. \text{ Compute velocity } V = \left(\frac{Q f^2}{140} \right)^{1/3}$$

$$3. \text{ Determine area } A = Q/V$$

$$4. \text{ Compute perimeter } P = 4.75 \sqrt{Q}$$

5. Find out bed width B and depth D of the channel section since A and P are known. The side slope of an irrigation channel is usually $\frac{1}{2} : 1$. Hence :

$$\text{Area } A = BD + \frac{D^2}{2}$$

$$\text{and perimeter } P = B + D \sqrt{5}$$

$$\text{Hence } D = \frac{P - \sqrt{P^2 - 6.944A}}{3.472} ; B = P - 2.236 D$$

$$6. \text{ Calculate } R = \frac{5}{2} \frac{V^2}{f}$$

$$\text{Also calculate } R = \frac{BD + D^2/2}{B + 2.236 D}$$

Both the values of R should be the same ; this will provide a numerical check from steps 1 to 5.

$$7. \text{ Find the slope } S = \frac{f^{5/3}}{3340 Q^{1/6}}$$

Example 14.6. A channel section has to be designed for the following data :

$$\text{Discharge } Q = 30 \text{ cumecs}$$

$$\text{Silt factor } f = 1.00$$

$$\text{Side slope } = \frac{1}{2} : 1$$

Find also the longitudinal slope.

Solution

1. The value of f is given as 1.00

$$2. \text{ Velocity } V = \left[\frac{30 \times 1}{140} \right]^{1/3} = 0.773 \text{ m/sec}$$

$$3. \text{ Area } A = \frac{30}{0.773} = 38.8 \text{ sq.m}$$

$$P = 4.75 \sqrt{Q} = 4.75 \sqrt{30} = 26 \text{ m}$$

$$D = \frac{P - \sqrt{P^2 - 6.944 A}}{3.472} = \frac{26 - \sqrt{676 - 6.944 \times 38.8}}{3.472} = 1.67 \text{ m.}$$

$$B = P - 2.236 D = 26 - 2.236 \times 1.67 = 22.26 \text{ m.}$$

6. Hydraulic mean radius.

$$R = \frac{5}{2} \frac{V^2}{f} = \frac{5}{2} \times \frac{1}{1} (0.773)^2 = 1.49 \text{ m.}$$

$$R = \frac{B D + D^2/2}{B + D \sqrt{5}} = \frac{1.67 (22.26) + 1.67^2/2}{22.26 + 1.67 \sqrt{5}} = 1.49 \text{ m.}$$

Also

Hence, checked.

$$7. \text{ Slope } S = \frac{f^{6/3}}{3340 (Q)^{1/4}} = \frac{1}{3340 (30)^{1/4}} = \frac{1}{3340 (1.764)} = \frac{1}{5880}$$

Hence, the channel has a bed width $B = 22.26 \text{ m}$ and a depth of 1.67 m . The longitudinal slope $S = 1/5880$.

14.10. COMPARISON OF KENNEDY'S AND LACEY'S THEORIES

1. Kennedy introduced the term C.V.R. (m) in his equation to make it applicable for channels of different grades of silt, but he did not give any idea to measure the value of m . Lacey introduced the concept of silt factor f in his equations and suggested a method of determining the value of f by relating it with particle size.

2. Kennedy assumed that silt is kept in suspension because of eddies generated from the bed only, and so he proposed a relation between V and D . Lacey assumed that silt is kept in suspension because of the normal components of eddies generated from the entire perimeter and so he proposed a relation between V and R .

3. Kennedy assumed Kutter's formula for finding the value of mean velocity where in the value of N is to be assumed arbitrarily. Lacey gave his own formula for the velocity and thus a designer has not to choose anything arbitrarily.

4. Kennedy gave no formula for determination of longitudinal slope of the canal. The slope to be given to the canal is based on experience or on Wood's table. Lacey gave a formula for the longitudinal slope of a regime channel.

5. Lacey proposed that the shape of a regime channel should be a semi ellipse. Since the channel section is trapezoidal in shape, it can never attain final regime. Kennedy simply gave the idea that a non-silting and non-scouring channel will be a regime channel.

6. Lacey's theory as applied to channel design does not involve any trial and error procedure whereas Kennedy's theory involves a trial and error procedure for design of channel.

7. Lacey made a distinction between two types of resistance in alluvial channels, one determined by grain size and the other due to irregularities of the channel. Kennedy did not make any such distinction.

8. Basic concept of the theories is the same that the silt remains in suspension due to the force of vertical eddies.

14.11. DEFECTS IN LACEY'S THEORY

1. The theory does not give a clear description of physical aspects of the problem.

2. It does not define what actually governs the characteristics of an alluvial channel.

3. The derivation of various formulae depends upon a single factor f and dependence on single factor f is not adequate. There are different phases of flow on bed and sides and, hence, different values of silt factor for bed and side should have been used.

4. Lacey's equations do not include a concentration of silt as variable.

5. Lacey did not take into account the silt left in channel by water that is lost in absorption which is as much as 12 to 15% of the total discharge of channel.

6. The effect of silt attrition was also ignored. The silt size does actually go on decreasing by the process of attrition among the rolling silt particles dragged along the bed.

7. Lacey did not properly define the silt grade and silt charge.

8. Lacey, however, considers that a regime channel is inherently free from external restraint and shock and has, therefore, a constant N_s for a given size of material. In so far as regime channel is a sediment transporting channel and will normally have a changing pattern of bed ripple formation, this statement is unlikely to be correct.

9. Lacey introduced semi-ellipse as ideal shape of a regime channel which is not correct.

10. Strictly speaking an artificial channel is not a regime channel, and regime theory is not applicable to it.

14.12. TRACTIVE FORCE APPROACH FOR CHANNEL DESIGN

Tractive force approach, which is an improvement over various silt theories discussed above, can be used for the design of alluvial channels. Tractive force or fluid drag is the force exerted by the flowing water on the bed grains to cause its movement. Tractive force is equal to the tangential component of the weight of water flowing in the channel and acts in the direction of flow. For uniform flow, the mean tractive force per unit length of the channel is equal to wAS , where w is the unit weight of water, A is the cross-sectional area of the channel and S is the bed slope of the channel. The tractive force per unit area (or the boundary shear stress τ) is equal to the tractive force per unit length of the channel divided by its perimeter P .

$$\text{Thus, } \tau = \frac{wAS}{P} = wRS \quad \dots(14.32)$$

where R = hydraulic mean radius.

The bed grains start moving when the tractive force exceeds the frictional resistance between them. The tractive force required to initiate the general movement of bed grains is called the critical tractive force and the corresponding stress is known as critical tractive stress (τ_c). The critical tractive stress is a function of sediment concentration

(i.e. its specific gravity) and the average grain size of the bed material. Several investigators have given expressions for τ_c , and some of these are given below.

1. Shield's expression ...(14.33)

$$\tau_c = 0.06 w (G - 1) d$$

where τ_c = critical shear stress (kN/m^2)
 w = specific weight of water (kN/m^3)
 d = grain size diameter (m)

Taking $G = 2.66$ and $w = 9.81 \text{ kN/m}^3$, equation 14.33 reduces to : ...(14.33 a)

$$\tau_c = 0.98 d$$

2. White's equation ...(14.34)

$$\tau_c = 0.786 d (\text{kN/m}^2)$$

where d is the grain diameter in m.

Note that the above equations are for *fully developed turbulent flow*. The value of τ_c for channels may be reduced by 10% for slightly sinuous channels, by 25% for moderately sinuous channels and by 40% for very sinuous channels.

Table 14.12. gives the average values of critical tractive stress (τ_c) for some common types of sediments flowing in the channels.

TABLE 14.12. AVERAGE VALUES OF τ_c

Type of sediment	Average value of τ_c (kN/m^2)
1. Alluvial soils, silty loam and coarse sand	0.0025
2. Medium sand	0.0017
3. Sandy loam	0.0020
4. Fine Gravel	0.0037

14.13. DESIGN OF UNLINED CHANNELS IN ALLUVIAL SOIL AS PER IS : 7112-1973

Indian Standard IS : 7112-1973 has recommended the following three methods for design of unlined channels in alluvial soils.

1. Channel design by Lacey's Equations
2. Channel design by regime type fitted equations
3. Channel design by tractive force approach

1. Method 1. Channel design by Lacey's equations

In this method, the channel may be designed by the following three equations given by Lacey

$$S = \frac{0.0003 f^{1/3}}{Q^{1/4}} \quad \dots(i) \quad \dots(14.21)$$

$$P = 4.75 \sqrt{Q} \quad \dots(ii) \quad \dots(14.15)$$

$$\text{and} \quad R = 0.47 (Q/f)^{1/3} \quad \dots(iii) \quad \dots(14.22)$$

The silt factor f may be determined from the following equation :

$$f = 1.76 \sqrt{m_d}$$

where m_d = mean particle size in mm of the boundary material = D_{50} in mm.
 Alternatively, silt factor f be determined from the following equation.

$$f = \frac{2.39 V^2}{R}$$

where V is the mean velocity of flow (m/s) and R is the hydraulic mean depth (m) of an existing stable channel.

For a design problem, Q and f are known; hence S , P and R can be computed from Eqs (i), (ii) and (iii) respectively. IS : 7112-1973 has also given graphical solution for finding direction B and D corresponding to the above computed values of P and R , for a trapezoidal channel having side slopes $\frac{1}{2}:1$. The corresponding curves for the graphical solution are given in Fig. 14.5.

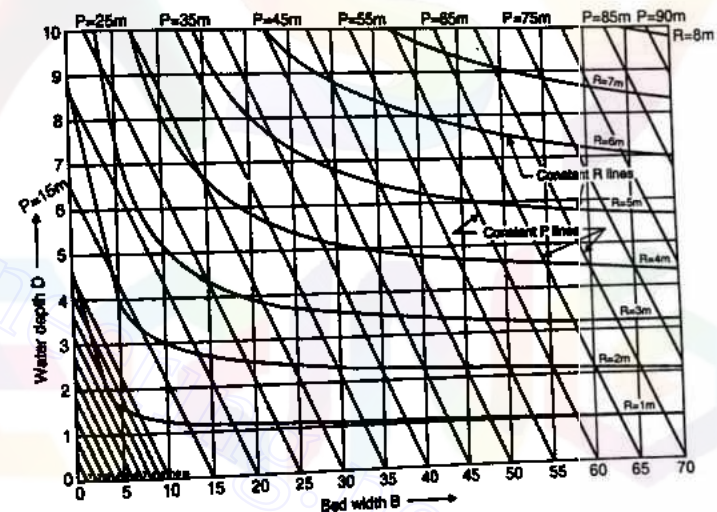


FIG. 14.5. HYDRAULIC CHART OF RELATIONSHIP BETWEEN B , D , P AND R FOR CHANNEL HAVING SIDE SLOPES $\frac{1}{2}:1$.

Alternatively, B , and D can be computed from the simultaneous solution of the following equations :

$$P = B + D \sqrt{5} = B + 2.236 D$$

and

$$R = \frac{BD + 0.5 D^2}{B + 2.236 D}$$

2. Method 2 : Channel design by regime type fitted equations

Alternatively, the channel can be designed using *regime type fitted equations* given in Table 14.13. These equations have been evolved on the basis of data collected from various States in India.

TABLE 14.13. REGIME TYPE FITTED EQUATIONS (IS : 7112 - 1973)

S.N.	Hydraulic parameter	All India canals	Punjab canals	U.P. Canals	Bengal canals
1.	Bed slope S	$\frac{0.000315}{Q^{0.1461}}$	$\frac{0.000251}{Q^{0.1461}}$	$\frac{0.000360}{Q^{0.1460}}$	$\frac{0.0001346}{Q^{0.0376}}$
2.	Wetted Perimeter P	$4.80 Q^{0.0231}$	$7.00 Q^{0.0219}$	$3.98 Q^{0.0200}$	$5.52 Q^{0.4190}$
3.	Hydraulic mean Radius R	$0.515 Q^{0.3400}$	$0.466 Q^{0.3389}$	$0.448 Q^{0.3549}$	$0.438 Q^{1.4454}$

Method 3. Channel design by tractive force approach

IS : 7112-1973 also recommends to design the channel on the tractive force approach, in which the tractive stress (or unit tractive force) exerted on the bed of a channel by the flowing water may be calculated from the following equation :

$$\tau = w \cdot R \cdot S \quad \dots(14.32)$$

The permissible tractive stress is a function of sediment concentration and average particle size (or D_{50} size) of the bed material in case of channels in non-cohesive soils and voids ratio in case of clayey soils.

Design procedure

1. Obtain the permissible tractive stress from observed data on existing channels based on sediment concentration X of flow in the channel and D_{50} size of bed material in the case of non-cohesive soils and voids ratio in the case of cohesive soils. Alternatively, the permissible tractive stress may also be determined by expressions given by various investigators such as Shield, White etc.

2. Select a suitable bed slope of the channel on the basis of average ground slope along the alignment of the channel or on the basis of experience.

3. Taking $\tau = \tau_c$, compute the value of R from the relation :

$$R = \frac{\tau_c}{w \cdot S} \quad \dots(14.32 a)$$

4. Assume a suitable value of rugosity coefficient N from Table 14.7 and obtain the mean velocity (V) of flow from Manning formula :

$$V = \frac{1}{N} R^{2/3} S^{1/2}$$

5. Compute the area of cross-section from the relation : $A = Q/V$

6. Knowing R and A , determine B and D for a trapezoidal channel having side slopes of $\frac{1}{2} : 1$.

Example 14.7. Using tractive force approach, design a channel in alluvial soil for the following data :

IRRIGATION CHANNELS - I: SILT THEORIES

- (i) Discharge : 45 cumecs
- (ii) Bed slope : $1/4800$
- (iii) Manning's $N = 0.0225$
- (iv) Permissible tractive stress : 0.0035 kN/m^2
- (v) Side slopes : $\frac{1}{2} : 1$

Solution : $R = \frac{\tau_c}{w \cdot S} = \frac{0.0035}{9.81 \times 1/4800} = 1.713 \text{ m}$

$$V = \frac{1}{N} R^{2/3} S^{1/2} = \frac{1}{0.0225} (1.713)^{2/3} (1/4800)^{1/2} = 0.918 \text{ m/s}$$

$$\therefore A = Q/V = 45/0.918 = 49 \text{ m}^2$$

$$P = A/R = 49/1.713 = 28.61 \text{ m}$$

$$P = 28.61 = B + 2.236 D$$

$$\therefore A = 49 = BD + 0.5 D^2 \quad \dots(i)$$

Solving these, we get $D \approx 1.94 \text{ m}$ and $B = 24.27 \text{ m}$... (ii)

14.14. SEDIMENT TRANSPORT

The quantity of solids entering the channel is an important factor which controls the cross-section and shape of a true regime channel. Regime theory does not consider this important factor in channel design. However, it has now been realised that the channel design will not be successful unless a detailed study of sediment transport has been made.

The sediment moving in a fluid can be broadly divided in two parts :

1. Bed load
2. Suspended load.

Bed load

Bed load may be defined as the load of bed material in the layer where suspension is impossible for fluid dynamic reasons. Sediment grains in the bed layer are not vertically supported by the flow, but rest on the bed almost continuously, while sliding, rolling and jumping along. Their weight is supported by the static grain particles in the non-moving bed. They move regularly exchanging places with the similar particles in non-moving bed.

Suspended Load

In case of suspended load, the weight of the particles is supported by the surrounding fluid. Since the particles continuously settle with its settling velocity V_s , in relation to the surrounding fluid, it is kept in suspension solely because the flow provides an upward motion due to turbulent exchange due to which a fluid is continuously exchanged over definite distances between horizontal layers. The rising fluid originates from lower layers of higher concentration and descending fluid originates from higher layer of lower concentration so that a surplus of upward moving sediment particles over downward moving sediment particle occurs. This surplus provides an upward motion of particles which counterbalances the general settling of sediment. Suspended load cause an additional hydrosatic pressure on the bed.

The concentration of suspended load C at a height y above the bottom can be determined in terms of known concentration C_a at a reference point at a height a above the bottom by Rouse equation given below :

$$\frac{C}{C_a} = \left[\frac{a}{y} \cdot \frac{D-y}{D-a} \right]^{V_s/V_a} \quad \dots(14.35)$$

where V_s = fall velocity of grain in still water

D = Depth of water

K = Van Karman's constant = 0.4

$$V_s = \text{Shear friction velocity} = \sqrt{\tau_0/\rho} = \sqrt{\frac{w R' S}{\rho}} = \sqrt{\frac{\rho g R' S}{\rho}} = \sqrt{g R' S} \quad \dots(14.36)$$

τ_0 = Intensity of shear stress at bottom.

ρ = mass density of water

R' = modified value of R due to ripple = $R(N'/N)^{1/2}$... (see Eq. 14.42)

Note that the effect of ripples is to reduce the hydraulic mean radius to a value R' from R . For a wide channel, $R \triangleq$ bed width B .

Rate of bed load transport

The initiation of bed load movement is caused by the drag force exerted on it by moving water. Many investigators have worked on the correct estimation for the rate of bed transport but none of the ideas extended so far could be widely accepted.

We shall discuss herein the following two most widely used equations :

(1) Meyer-Peter's Equation, and (2) Einstein's Equation

1. Meyer Peter's Equation

Meyer-Peter's equation, given below, is based on experimental work carried out at Federal Institute of Technology, Zurich.

$$\left(\frac{Q_s}{Q} \right) \left(\frac{N'}{N} \right)^{3/2} wSD = 0.047(w_s - w)d + 0.25(w/g)^{1/3}(q_s)^{2/3} \quad \dots(14.37)$$

where D = depth of flowing water in the channel (m)

S = bed slope of the channel

d = mean grain diameter (m)

q_s = rate of bed load transport per unit width of channel (kN/m/s)

Q = discharge in the channel, if the sides were frictionless (cumecs)

Q_s = actual discharge in the channel (cumecs)

w_s = specific weight of sediment particles (kN/m³)

w = specific weight of water (kN/m³)

N' = Manning's coefficient for plane bed

N = Manning's coefficient for rippled bed

The ratio Q_s/Q takes into account correction for friction of the sides. If the sides are frictionless, $Q_s/Q = 1.0$.

The values of N' and N can be obtained from Strikler's formulae given below:

$$N' = (k_s)^{1/6}/24 \quad \text{and} \quad N = (k)^{1/6}/24 \quad \dots(14.38)$$

where k_s is the effective grain diameter or d_{90} size (in m) and k is the representative size (in m) of roughness of rippled bed.

Note that, N is the rugosity coefficient actually observed by experiments on rippled bed of the channel and its value can be taken as 0.020 for discharges over 11 cumecs and as 0.0225 for discharges less than 11 cumecs.

Now wRS = shear stress $\tau = wDS$ for wide channels where $R \triangleq D$.

Hence bed shear, $\tau_b = (Q_s/Q) \cdot wDS$

Also, critical shear stress, $\tau_c = 0.047 w_s - w) d$.

Substituting these values in Eq. 14.37, we get

$$\tau_b (N'/N)^{3/2} = \tau_c + 0.25 (w/g)^{1/3} (q_s)^{2/3}$$

or

$$q_s = 47450 [\tau_b (N'/N)^{3/2} - \tau_c]^{3/2} \text{ kN/m/hr} \quad \dots(14.39)$$

However, if q_s is expressed in N/m/hr and τ_b and τ_c are in N/m², we have

$$q_s = 1500 [\tau_b (N'/N)^{3/2} - \tau_c]^{3/2} \text{ N/m/hr} \quad \dots(14.39 a)$$

Lastly if q_s is expressed in kg/m/hr and τ_b and τ_c are in kg/m², we have

$$q_s = 4700 [\tau_b (N'/N)^{3/2} - \tau_c]^{3/2} \text{ kg/m/hr} \quad \dots(14.39 b)$$

2. Einstein's Equations

Einstein's equations based on the principles of law of probability, involves a number of assumptions and experimental coefficients. The equations correlate two dimensionless parameters ϕ_* and ψ_* (known as bed load functions) as under :

$$\phi_* = \left(\frac{i_s}{i_b} \right) \phi = \frac{i_s}{i_b} \left[\frac{q_s}{Gw} \left(\frac{1}{G-1} \right)^{1/2} \left(\frac{1}{d^3} \right)^{1/2} \right] \quad \dots(14.40)$$

and

$$\psi_* = \left(\frac{i_s}{i_b} \right) \psi = \frac{i_s}{i_b} \left[\frac{(G-1)d}{R'S} \right] \quad \dots(14.41)$$

where q_s = rate of bed load transport per unit width of channel (kN/m/s)

d = diameter of sediment particles (m)

i_b = fraction of q_s which is of diameter d

i_s = proportion of grains of diameter d in the bed.

G = specific gravity of sediment.

w = specific weight of water (kN/m³)

S = bed slope of the channel

R' = hydraulic mean radius of the channel if the bed were unrippled.

Since R' represents hydraulic mean radius of a channel which has grains roughness only (i.e. unrippled channel), we have from Manning's equation

$$V = \frac{1}{N'} (R')^{2/3} S^{1/2} \quad \dots(a)$$

However, if R is the actual hydraulic mean radius for the rippled bed, we have

$$V = \frac{1}{N} (R')^{2/3} S^{1/2} \quad \dots(b)$$

Equating the above two equations, we have

$$R' = R (N'/N)^{3/2} \quad \dots(14.42)$$

Relationship between ϕ_* and ψ_*

Since the relation between ϕ_* and ψ_* is quite complex, Einstein gave the relationship in the form of graph shown in Fig. 14.6. In order to save space (i.e. the length of the curve), he subdivided the entire graph into three parts : Graph III (for $0.01 > \phi_* > 0.0001$), Graph II (for $1.0 > \phi_* > 0.01$) and Graph I (for $100 > \phi_* > 1.0$), each part having abscissa marked accordingly, as shown.

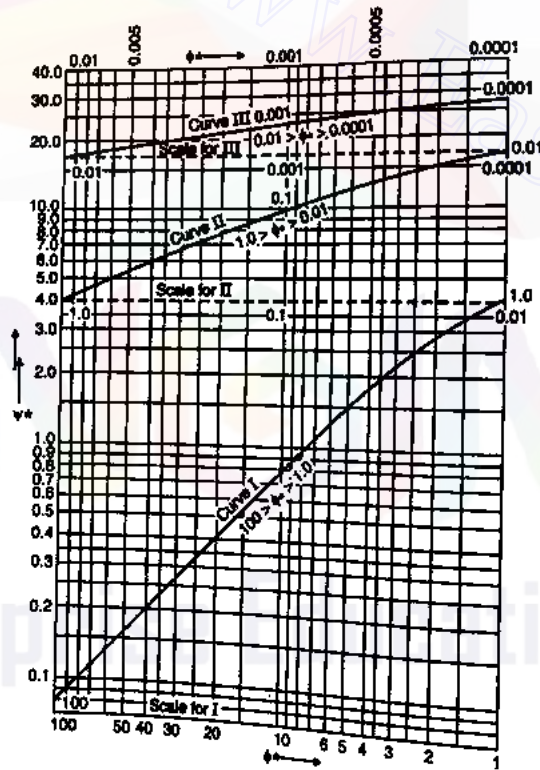


FIG. 14.6. EINSTEIN'S BED LOAD FUNCTIONS

If the value of ψ_* is known (or determined from Eq. 14.41), the corresponding value of ϕ_* is found from the graph and then the value of q_s is computed from Eq. 14.40.

Note the following values for uniform soils :

$$(i) \quad i_s = i_b$$

$$(ii) \quad \phi_* = \phi = \left[\frac{q_s}{Gw} \left(\frac{1}{G-1} \right)^{1/2} \left(\frac{1}{gd^3} \right)^{1/2} \right] \quad \dots(14.40 a)$$

$$\text{and } (iii) \quad \psi_* = \psi = \frac{(G-1)d}{R'S} \quad \dots(14.40 b)$$

Also, for uniform soils, the relation between ϕ and ψ can be expressed by the following equation :

$$\phi = \frac{1}{0.465} e^{-0.201\psi} \quad \dots(14.43)$$

Channel design based on concept of bed load transport

The above equations given by Meyer-Peter and Einstein can be used to design a channel for a given bed load transport. However, solution of these equations yield only the hydraulic mean depth R and the slope S , and not the bed width. Hence bed width B is chosen close to (or slightly less than) Lacey's regime perimeter. Also, the value of Manning's coefficient N for rippled bed has to be estimated on the basis of experience. See examples 14.15 and 14.16 for illustration.

ILLUSTRATIVE EXAMPLES

Example 14.8. Design a channel section by Kennedy's theory given the following data :

$$\text{Discharge} \quad Q = 28 \text{ cumecs}$$

$$\text{Kutter's} \quad N = 0.0225$$

$$\text{Critical velocity ratio} \quad m = 1$$

$$\text{Side slope} \quad = \frac{1}{2} : 1$$

$$B/D = 7.6$$

Find also the bed slope of the channel.

Solution

Let the depth of the channel be D

Hence width B of the channel = $7.6 D$

Velocity in the channel

$$V_0 = 0.55 m D^{0.64} = 0.55 D^{0.64}, \text{ taking } m = 1$$

Area A of the channel section having a side slopes $\frac{1}{2} : 1$ is

$$A = BD + D^2/2 = 7.6 D^2 + D^2/2 = 8.1 D^2$$

$$\text{Continuity Equ.} \quad Q = A V_0$$

$$\therefore 28 = (8.1 D^2) (0.55 D^{0.64}) = 4.46 D^{2.64}$$

$$D^{2.64} = \frac{28}{4.46} = 6.28, \text{ from which } D = 2.01 \text{ m,}$$

$$B = 2.01 \times 7.6 \approx 15.3 \text{ m}$$

Hence

Hydraulic mean radius

$$R = A/P = \frac{A}{B + \sqrt{5} D} = \frac{8.1 D^2}{7.6 D + 2.236 D} = 0.823 D$$

$$\text{Taking } D = 2.01 \text{ m, we get } R = 0.823 \times 2.01 = 1.66 \text{ m}$$

$$V = 0.55 (2.01)^{0.64} = 0.86 \text{ m/s}$$

By applying Kutter's formula

$$V = C \sqrt{RS}$$

$$\text{where } C = \frac{23 + \frac{1}{N} + \frac{0.00155}{S}}{1 + \left(23 + \frac{0.00155}{S}\right) \frac{N}{\sqrt{R}}}, \text{ we get}$$

$$\frac{23 + \frac{1}{N} + \frac{0.00155}{S}}{1 + \left(23 + \frac{0.00155}{S}\right) \frac{N}{\sqrt{R}}} \sqrt{RS} = 0.86$$

$$\text{or } \left(23 + \frac{1}{0.0225} + \frac{0.00155}{S}\right) S^{1/2} = \frac{0.86}{\sqrt{1.66}} \left[1 + \left(23 + \frac{0.00155}{S}\right) \frac{0.0225}{\sqrt{1.66}}\right]$$

$$\text{or } 67.4 S^{3/2} - 0.885 S + 0.00155 S^{1/2} - 1.42 \times 10^{-5} = 0$$

$$\text{Solving by trial and error, we get } S = \frac{1}{6250}$$

$$\text{Check } 67.4 \left(\frac{1}{6250}\right)^{3/2} - 0.885 \left(\frac{1}{6250}\right) + 0.00155 \left(\frac{1}{6250}\right)^{1/2} - 1.42 \times 10^{-5} = 0$$

$$13.65 \times 10^{-5} - 14.18 \times 10^{-6} + 1.96 \times 10^{-5} - 1.42 \times 10^{-5} = 0$$

$$15.61 - 15.60 \approx 0$$

$$\text{Hence, } S = \frac{1}{6250}; B \approx 15.3 \text{ m}; D = 2.01 \text{ m.}$$

Example 14.9. Find the channel section and discharge Q that can be allowed to flow in it, if $B/D = 5.7$, bed slope $= 1/5000$ and $N = 0.0225$. Use Kennedy's theory.

Solution

$$\text{Given } B/D = 5.7. \text{ Hence, } B = 5.7 D$$

For a channel with side slope $\frac{1}{2}:1$

$$R = \frac{A}{P} = \frac{BD + D^2/2}{B + \sqrt{5} D} = \frac{5.7 D^2 + 0.5 D^2}{5.7 D + 2.236 D} = \frac{6.2 D^2}{7.936 D} = 0.781 D$$

Assume critical velocity ratio $m = 1$.Then $V = V_c = 0.55 D^{0.64}$ by Kennedy's formula and $V = C \sqrt{RS}$ by Kutter's formula.

$$\text{Hence, } 0.55 D^{0.64} = \frac{23 + \frac{1}{N} + \frac{0.00155}{S}}{1 + \left(23 + \frac{0.00155}{S}\right) \frac{N}{\sqrt{R}}} \sqrt{RS}$$

$$\text{Putting } N = 0.0225$$

$$\begin{aligned} \text{R.H.S.} &= \frac{23 + \frac{1}{0.0225} + 0.00155 \times 5000}{1 + \left[23 + \frac{0.00155 \times 5000}{1}\right] \frac{0.0225}{\sqrt{0.781 D}}} \sqrt{0.781 D S} \\ &= \frac{75.15}{1 + \frac{30.75 \times 0.0225}{0.885 \sqrt{D}}} \sqrt{\frac{0.781 D}{5000}} = \frac{66.5 D^{1/2}}{(1 + 0.781 D^{-1/2}) 70.7} \end{aligned}$$

$$\text{Equating L.H.S. and R.H.S. we get } 70.7 \times 0.55 D^{0.64} + 0.55 \times 0.781 \times 70.7 D^{0.14} = 66.5 D^{1/2}$$

$$38.88 D^{0.64} - 66.5 D^{0.5} + 30.37 D^{0.14} = 0$$

Solving the above equation by trial and error, we get $D = 1.7 \text{ m}$

$$\text{Hence, } B = 5.7 \times 1.7 = 9.69 \text{ m}$$

$$V = 0.55 \times 1 \times (1.7)^{0.64} = 0.773 \text{ m/sec}$$

$$A = BD + D^2/2 = 6.2 D^2 = 6.2 (1.7)^2 = 17.91 \text{ m}^2$$

$$\text{Discharge } Q = AV = 17.91 \times 0.773 = 13.85 \text{ cumecs.}$$

Example 14.10. Design an irrigation channel in alluvial soil according to Lacey's silt theory, given the following data :

$$\text{Full supply discharge} = 15 \text{ m}^3/\text{sec}$$

$$\text{Lacey's silt factor} = 1.0$$

$$\text{Channel side slopes} = \frac{1}{2} : 1$$

Solution

$$1. \text{ Velocity } V \text{ in the channel} = \left[\frac{Q f^2}{140}\right]^{1/6} = \left[\frac{15 \times 1}{140}\right]^{1/6} = 0.589 \text{ m/sec.}$$

$$2. \text{ Area } A \text{ of the channel section} = \frac{Q}{V} = \frac{15}{0.589} = 21.765 \text{ m}^2$$

$$3. \text{ Hydraulic mean radius } R = \frac{5 V^2}{2 f} = \frac{5 (0.589)^2}{2} = 1.187 \text{ m.}$$

$$4. R = 1.187 = \frac{A}{B + D \sqrt{5}} = \frac{21.765}{B + D \sqrt{5}}$$

$$\text{or } B + D\sqrt{5} = \frac{21.765}{1.187} = 18.336 \quad \dots(i)$$

$$\text{and } A = BD + D^2 / 2 = 21.765 \quad \dots(ii)$$

Solving equations (i) and (ii), We get

$$B = 15.3 \text{ m and } D = 1.37 \text{ m}$$

$$5. \text{ Check : } P = 4.75 \sqrt{Q} = 4.75 \sqrt{15} = 18.4 \text{ m}$$

$$\text{Also, } P = B + D\sqrt{5} = 15.3 + 1.37\sqrt{5} = 18.4 \text{ m}$$

Note. In Example 14.26, first A and P were computed from which B and D were determined. A check was then made by finding out the value of R by Lacey's formula as well as from the values of B and D . In this example, first A and R were computed in the order to determine B and D . A numerical check has been made in step (5) by finding values of P from the calculated values of B and D and from that of Lacey's formula. Both the methods are correct and students can follow any procedure.

$$4. \text{ Slope } S = \frac{1}{3340(Q)^{1/6}} = \frac{1}{3340(15)^{1/6}} = \frac{1}{5245}$$

Example 14.11. The slope of channel in alluvium is $S = 1/5000$; Lacey's silt factor = 0.9 and channel side slope = $\frac{1}{2} : 1$. Find the channel section and maximum discharge which can be allowed to flow in it

Solution

$$\text{We have } S = \frac{f^{5/3}}{3340 Q^{1/6}}$$

$$\text{or } Q = \left[\frac{f^{5/3}}{3340 S} \right]^6 = \frac{f^{10}}{(3340 S)^6} = \left[\frac{5000}{3340} \right]^6 \times (0.9)^{10} = 3.924 \text{ m}^3/\text{sec}$$

$$\text{Also } S = \frac{f^{3/2}}{4980 R^{1/2}}$$

$$\text{or } R = \frac{f^3}{(4980 S)^2} = \frac{(0.9)^3 \times 5000 \times 5000}{4980 \times 4980} = 0.735$$

$$\text{Now, } P = 4.75 \sqrt{Q} = 4.75 \sqrt{3.924} = 9.41 \text{ m}$$

$$\text{Hence } A = PR = 9.41 \times 0.735 = 6.916 \text{ sq.m}$$

$$\text{But } A = BD + D^2/2 = 6.916 \text{ m}^2 \quad \dots(i)$$

$$P = B + \sqrt{5} D = 9.41 \text{ m}$$

$$\text{Solving equations (i) and (ii), we get } B = 7.45 \text{ m and } D = 0.88 \text{ m} \quad \dots(ii)$$

Example 14.12. Bed load transport by Meyer-Peter's Equation

A wide channel 3 m deep has a bed slope of $1/4400$. The bed material consists of uniform grain size of 0.3 mm. Determine the quantity of bed load moved by the channel by Meyer-Peter's equation. Take representative size of roughness of channel bed, $k = 1.5$ mm, and specific gravity of sediment particles as 2.65.

Solution

$$\text{From Eq. 14.39, } q_s = 47450 (\tau_b (N^*/N)^{3/2} - \tau_c)^{1/2} \text{ kN/m/hr.}$$

$$\text{Here, } \tau_b = wDS = 9.81 \times 3 \times \frac{1}{4400} = 6.689 \times 10^{-3} \text{ kN/m}^2$$

$$N^* = \frac{(k_s)^{1/4}}{24} \text{ where } k_s = d = 0.3 \text{ mm}$$

$$\text{and } N = \frac{k^{1/4}}{24} \text{ where } k = 1.5 \text{ mm}$$

$$\therefore \left(\frac{N^*}{N} \right)^{3/2} = \left(\frac{d}{k} \right)^{1/4} = \left(\frac{0.3}{1.5} \right)^{1/4} = 0.6687$$

$$\tau_c = 0.047 (w_s - w) d$$

$$\text{where } w = 9.81 \text{ kN/m}^3, w_s = wG = 9.81 \times 2.65 = 26 \text{ kN/m}^3 \text{ and } d = 3 \times 10^{-4} \text{ m}$$

$$\therefore \tau_c = 0.047 (26 - 9.81) \times 3 \times 10^{-4} = 0.228 \times 10^{-3} \text{ kN/m}^2$$

Substituting the values in Eq. 14.39, we get

$$q_s = 47450 [6.689 \times 10^{-3} \times 0.6687 - 0.228 \times 10^{-3}]^{1/2} = 13.12 \text{ kN/m/hour}$$

Example 14.13. Bed load-transport by Einstein's equation

Determine the bed load transported by the channel of Example 14.12 by Einstein's equation. Neglect side friction.

Solution : For uniform soils

$$\psi_s = \psi = \frac{(G-1)d}{R S} \text{ and } R' = R \left(\frac{N^*}{N} \right)^{3/2}$$

For wide channel, $R = D = 3 \text{ m}$

As found in example 14.12, $(N^*/N)^{3/2} = 0.6687$

$$\therefore R' = 3 \times 0.6687 = 2.006 \text{ m}$$

$$\text{Hence } \psi = \frac{(2.65-1)3 \times 10^{-4}}{2.006 \times 1/4400} = 1.086$$

From Einstein's curve (Fig. 14.6), for $\psi_s = \psi = 1.086$, we get $\phi_s = \phi = 7.0$

$$\text{From Eq. 14.40 (a), } \phi_s = \phi = \frac{q_s}{Gw} \left(\frac{1}{G-1} \right)^{1/2} \left(\frac{1}{gd^3} \right)^{1/2}$$

$$\begin{aligned} \text{or } q_s &= \phi Gw (G-1)^{1/2} (gd^3)^{1/2} \\ &= 7 \times 2.65 \times 9.81 (2.65-1)^{1/2} (9.81)^{1/2} (3 \times 10^{-4})^{1/2} \text{ kN/m/s} \\ &= 4.348 \times 10^{-3} \text{ kN/m/s} = 3.804 \times 10^{-3} \times 3600 \text{ kN/m/h} = 13.7 \text{ kN/m/h} \end{aligned}$$

IRRIGATION AND WATER POWER ENGINEERING

692

Example 14.14. Determination of concentration of suspended load

Determine the concentration of suspended load at 1.0 m above the channel bed of Example 14.12 if the concentration at 0.3 m above the bed is 400 ppm. The particles have a fall velocity of 0.03 m/sec.

Solution : Given : $C_a = 400$ ppm at 0.3 above bed and $V_s = 0.03$ m/sec.

Here, $a = 0.3$ m. $D = 3$ m and $y = 1$ m

Now $V_s = \sqrt{g R' S}$, where $R' = 2.006$ m found in the previous example.

$$V_s = \sqrt{\frac{9.81 \times 2.006}{4400}} = 0.0669 \text{ m/sec.}$$

$k =$ Von Karman's constant $= 0.4$

$$\frac{V_s}{V_* k} = \frac{0.03}{0.0669 \times 0.4} = 1.121$$

$$\frac{C}{C_a} = \left[\frac{a}{y} \times \frac{D-y}{D-a} \right]^{V_s/V_* k}$$

From Eq. 14.35,

$$C = 400 \left[\frac{0.3}{1} \times \frac{3-1}{3-0.3} \right]^{1.121} = 74.1 \text{ ppm}$$

Example 14.15. Channel design on Meyer-Peter's equations

Using Meyer-Peter's equations, design an irrigation channel carrying a full supply discharge of 45 cumecs with a bed load concentration of 55 ppm by weight. The average grain diameter of bed material = 0.3 mm and $G = 2.67$.

Solution

Quantity of bed load to be transported $= (55 \times 10^{-6}) \times 45 \times 9.81 \times 60 \times 60 = 87.41 \text{ kN/m/hr}$

Lacey's regime perimeter $P = 4.75 \sqrt{Q} = 4.75 \sqrt{45} = 31.86 \text{ m}$

Let us fix the channel width $B = 28$ m (i.e. slightly less than P).

$\therefore$ Rate of bed load transported, $q_s = 87.41/28 = 3.12 \text{ kN/m/hr}$

Let the effective grain diameter, k , be equal to 0.4 mm (i.e. slightly more than the average grain diameter of 0.3 mm).

$$\text{Hence } N^* = \frac{(k_s)^{1/6}}{24} = \frac{(0.4 \times 10^{-3})^{1/6}}{24} = 0.0113$$

Lacey's silt factor $= 1.76 \sqrt{m_d} = 1.76 \sqrt{0.3} = 0.964$

Hence from the Eq. $N_s = 0.0225 f^{1/4}$, we get $N = N_s = 0.0225(0.964)^{1/4} = 0.0223$

$$\therefore \frac{N^*}{N} = \frac{0.0113}{0.0223} = 0.507$$

Now, $\tau_b = (Q_s/Q) w R S = w R S$ by assuming $Q_s/Q = 1$

$$\tau_b = 9.81 R S \text{ kN/m}^2$$

$$\text{Also, } \tau_c = 0.047 (w_s - w) d = 0.047 \times 9.81 (2.67 - 1) \times 0.3 \times 10^{-3} = 0.231 \times 10^{-3} \text{ kN/m}^2$$

IRRIGATION CHANNELS - I: SILT THEORIES

693

Now

$$q_s = 47450 [\tau_b (N^*/N)^{3/2} - \tau_{cl}]^{3/2}$$

Substituting the values of various terms, we get.

$$3.12 = 47450 [9.81 R S (0.507)^{3/2} - 0.231 \times 10^{-3}]^{3/2} \quad \dots(a)$$

From which, we get $R S = 0.525 \times 10^{-3}$

Also, $A = P R = 31.86 R$

From Manning's formula $Q = \frac{1}{N} A R^{2/3} S^{1/2}$

$$\therefore 45 = \frac{1}{0.0223} \times 31.86 R \times R^{2/3} S^{1/2}$$

or

$$R^{5/3} S^{1/2} = 0.031$$

or

$$R^{10/3} S = 0.000992 \quad \dots(b)$$

Hence from (a) and (b), $R^{7/3} = 0.000992 / 0.525 \times 10^{-3} = 1.8897$

From which $R = 1.314 \text{ m}$

Hence from (a), $S = 0.525 \times 10^{-3} / 1.314 = 0.3997 \times 10^{-3} \triangleq 1/2500$

For a trapezoidal channel with $\frac{1}{2} : 1$ side slopes,

$$R = \frac{BD + 0.5 D^2}{B + 2.236 D} \text{ where } B = 28 \text{ m and } R = 1.314$$

$$\therefore 28 D + 0.5 D^2 = 1.314 (28 + 2.236 D)$$

or

$$0.5 D^2 + 25.06 D - 36.792 = 0 \text{ from which } D \triangleq 1.43 \text{ m}$$

Hence the channel dimension are : $B = 28 \text{ m}$; $D = 1.43 \text{ m}$ and $S = 1/2500$

Example 14.16. Channel design on Einstein's Equations

Redesign the channel of example 14.15, using Einstein's equations.

From Eq. 14.40 (a), $\phi_* = \phi = \frac{q_s}{G w} \left(\frac{1}{G-1} \right)^{1/2} \left(\frac{1}{g d^3} \right)^{1/2}$

Substituting the given values, we get

$$\phi = \left(\frac{3.12}{3600} \right) \left(\frac{1}{2.67 \times 9.81} \right) \left(\frac{1}{2.67 - 1} \right)^{1/2} \left[\frac{1}{9.81 (0.0003)^3} \right]^{1/2} = 1.573$$

Hence from Fig. 14.6, we get $\psi_* = \psi \triangleq 3.4$ for $\phi = \phi_* = 1.573$

Now $\psi_* = \psi = \frac{(G-1) d}{R' S}$

$$\therefore R' S = \frac{(2.67 - 1) 0.0003}{3.4} = 0.1474 \times 10^{-3}$$

But

$$R' = R (N^*/N)^{3/2} \text{ where } N^*/N = 0.507 \text{ from previous example.}$$

$$R' = R (0.507)^{3/2} = 0.361 R$$

$$\therefore 0.361 R S = 0.1474 \times 10^{-3}$$

or

$$R S = 0.4083 \times 10^{-3} \quad \dots(1)$$

Also, from Example 14.15, we have

$$R^{10/3} S = 0.000992$$

Hence from (1) and (2), we get $R^{1/2} = 2.4298$ or $R = 1.463$ m

and from (1), $S = 0.4083 \times 10^{-3} / 1.463 = 1/3580$

For a trapezoidal channel with $\frac{1}{2}:1$ side slopes,

$$R = \frac{BD + 0.5 D^2}{B + 2.236 D} \text{ where } B = 28 \text{ m selected in Example 14.15}$$

$$1.463 = \frac{28 D + 0.5 D^2}{28 + 2.236 D}$$

or $0.5 D^2 + 24.78 D - 40.96 = 0$, from which $D \approx 1.60$ m

Hence the channel dimensions are :

$$B = 28 \text{ m; } D = 1.6 \text{ m and } S = 1/3580$$

14.15. DESIGN OF NON-ALLUVIAL CHANNELS

As defined earlier, non-alluvial channels are those which flow through non-alluvial soils such as loam, clay, moorum and other hard soils, including boulders and rocks. In the non-alluvial soils, water is relatively clear, due to which there is no problem of silting. These channels are, therefore, considered as stable channels. Non-alluvial channels are designed on the basis of *maximum permissible velocity* that could be permitted in the channel without causing any scour either in the bed or in the sides. Table 14.14 gives the values of permissible velocities in various types of bed material.

TABLE 14.14. VALUES OF PERMISSIBLE VELOCITIES

Type of bed material	Permissible velocity (m/s)	Type of bed material	Permissible velocity (m/s)
1. Ordinary soils	0.8 to 0.9	6. Hard soil & Moorum	1.0 to 1.2
2. Loam; lean clay	0.4 to 1.0	7. Gravel	1.2 to 1.5
3. Ordinary clay	0.5 to 1.2	8. Boulders	1.5 to 1.8
4. Heavy clay	0.6 to 1.8	9. Soft rock	1.8 to 2.4
5. Light loose sand	0.5 to 0.8	10. Hard rock	2.5 to 4

Side slopes

The side slopes (H:V) of channels in ordinary soils, including clay, are generally kept as 1:1 in cutting and 1.5:1 in filling. However, in grit (or gravel), soft rock though the sides may be kept even vertical in very hard rocks.

Flow equations

The design of non-alluvial channels is done on the basis of the following two flow equations.

- (i) Chezy's formula, and (ii) Manning's formula

1. Chezy's formula

$$V = C \sqrt{RS} \quad \dots(14.44)$$

where C is Chezy's coefficient usually determined from the following equation by Bazin.

$$C = \frac{87}{1 + K/\sqrt{R}} \quad \dots(14.45)$$

where the K is Bazin's coefficient the value of which may be taken 1.2 to 1.4 for earth channels in good condition and between 1.7 to 1.8 for earth channels in poor condition.

(i) Manning's formula

$$V = \frac{1}{N} R^{2/3} S^{1/2} \quad \dots(14.46)$$

where N is Manning's coefficient (Table 14.15) the values of which depends upon the type of surface of channel.

TABLE 14.15. VALUES OF MANNING'S COEFFICIENT N

Type of surface	Value of N
1. Earth channels; clean straight and uniform	0.016 to 0.020
2. Earth channels; clean but weathered	0.018 to 0.025
3. Earth channels; with grass and weeds	0.022 to 0.033
4. Channels in gravel with stones	0.03 to 0.035
5. Channels in rock; smooth and uniform	0.025 to 0.04
6. Channels in rock; rough	0.035 to 0.050

Design procedure

For a channel design, the discharge Q and bed slope S are generally known. The value of maximum permissible velocity and side slopes ($r:1$) are chosen on the basis of type of soil through which the channel has to flow. Generally, Manning's formula is selected as the flow equation.

Step 1: Determine the area of cross-section from the continuity equation :

$$A = Q/V \quad \dots(i)$$

Step 2 : Determine hydraulic mean radius from Manning's formula (Eq. 14.46).

$$R = \left(\frac{VN}{S^{1/2}} \right)^{3/2} \quad \dots(ii)$$

Step 3. Determine wetted perimeter (P) from the relation

$$P = A/R \quad \dots(iii)$$

Step 4 For a trapezoidal channel with side slopes $r:1$ (Fig. 14.7), the area of cross-section (A) and wetted perimeter (P) are given by the following expressions.

$$A = (B + rD) D \quad \dots(14.47)$$

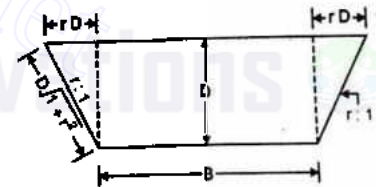


FIG. 14.7

IRRIGATION AND WATER POWER ENGINEERING

and

$$P = B + 2(D\sqrt{1+r^2}) \quad \dots(14.48)$$

and

In the above two equation, the L.H.S. (i.e. A and P) are known from (i) and (ii) respectively. Hence B and D can be computed by solution of the above two simultaneous equations.

Most efficient channel of trapezoidal section

A most efficient channel is the one which has minimum wetted perimeter for a given area. Let the side slopes of the trapezoidal section be $r(H):1(V)$. Then if B is the bed width and D is the water depth,

$$A = (B + rD)D \quad \dots(i)$$

and

$$P = (2\sqrt{1+r^2})D + B \quad \dots(ii)$$

Substituting the value of B from (i) into (ii), we get

$$P = 2D\sqrt{1+r^2} + \frac{A}{D} - rD \quad \dots(iii)$$

or

$$\frac{dP}{dD} = 0 = 2\sqrt{1+r^2} - \frac{A}{D^2} - r$$

Hence from (i),

$$B = \frac{A}{D} - rD = (2\sqrt{1+r^2} - r)D - rD \quad \dots(iv) \quad \dots(14.49)$$

or

$$B = 2(\sqrt{1+r^2} - r)D$$

Substituting the value of B in (i) and (ii) we get

$$P = (2\sqrt{1+r^2})D + 2(\sqrt{1+r^2} - r)D \quad \dots(v)$$

or

$$P = 2D(2\sqrt{1+r^2} - r)$$

and

$$A = (2D(\sqrt{1+r^2} - r) + rD)D = D^2(2\sqrt{1+r^2} - r) \quad \dots(14.50)$$

Hence

$$R = \frac{A}{P} = \frac{D^2(2\sqrt{1+r^2} - r)}{2D(2\sqrt{1+r^2} - r)} = \frac{D}{2}$$

Hence for the most efficient channel, $R = D/2$. This is one condition obtained for having minimum value of P . However, P is dependent on two variables : D and r . A second condition can therefore be obtained by putting $dP/dr = 0$, keeping D as constant.

Now

$$P = 2D(2\sqrt{1+r^2} - r) = 4D\sqrt{1+r^2} - 2rD$$

$$\frac{dP}{dr} = 0 = 4D \cdot \frac{1}{2}(1+r^2)^{-1/2} \times 2r - 2D = \frac{4rD}{\sqrt{1+r^2}} - 2D$$

or

$$\frac{2r}{\sqrt{1+r^2}} = 1 \quad \text{or} \quad 4r^2 = 1 + r^2$$

From which

$$r = \frac{1}{\sqrt{3}} \quad \dots(14.51)$$

IRRIGATION CHANNELS - I : SILT THEORIES

697

Hence the most efficient trapezoidal channel is the one which has side slopes of $1/\sqrt{3}:1$ ($H:V$) and $R = D/2$

Example 14.17. Design of channel in non-alluvial soil

Design a channel for $Q = 45$ cumecs in non-alluvial soil having maximum permissible velocity of 0.9 m/sec. The available bed slope is 1 in 4000 . Assume Manning's $N = 0.025$.

Solution

Step 1 :

$$A = Q/V = 45/0.9 = 50 \text{ m}^2$$

Step 2 :

$$R = \left[\frac{VN}{S^{1/2}} \right]^{3/2} = \left[\frac{0.9 \times 0.025}{(1/4000)^{1/2}} \right]^{3/2} \approx 1.7 \text{ m}$$

Step 3

$$P = A/R = 50/1.7 = 29.45 \text{ m}$$

Step 4 Let us provide a trapezoidal section of the channel with sides slopes of $1:1$ (i.e. $r = 1$)

$$A = (B + D)D = 50 \quad \dots(a)$$

and

$$P = B + 2D\sqrt{2} = 29.45 \quad \dots(b)$$

Substituting the value of B from Eq. (b) into Eq. (a), we get

$$D^2 + (29.45 - 2.828D)D = 50$$

or

$$1.828D^2 - 29.45D + 50 = 0 \text{ from which } D = 1.929 \text{ m} \approx 1.93 \text{ m}$$

and

$$B = 24 \text{ m}$$

Example 14.18. Design of non-alluvial channel

Using Bazin's formula, design a non-alluvial channel carrying discharge of 15 cumecs with a mean velocity of 0.75 m/s. The channel has bottom width as five times the depth, and has side slopes of $1:1$. Assume Bazin's coefficient $K = 1.3$.

Solution

Step 1 :

$$A = Q/V = 15/0.75 = 20 \text{ m}^2$$

Step 2 :

$$A = (B + D)D \text{ where } B = 5D \text{ and } A = 20 \text{ m}^2$$

$$20 = (5D + D)D = 6D^2, \text{ from which } D = 1.826 \text{ m} \approx 1.83 \text{ m}$$

$$B = 5D = 5 \times 1.826 = 9.13 \text{ m}$$

and

Step 3 :

$$P = B + 2D\sqrt{2} = 9.13 + 2 \times 1.83\sqrt{2} = 14.31 \text{ m}$$

$$R = A/P = 20/14.31 \approx 1.4 \text{ m}$$

Step 4 :

$$C = \frac{87}{1 + K/\sqrt{R}} = \frac{87}{1 + 1.3/\sqrt{1.4}} = 41.44$$

Step 5 :

$$V = C\sqrt{RS}, \text{ from which}$$

$$S = \left(\frac{V}{C} \right)^2 \frac{1}{R} = \left(\frac{0.75}{41.44} \right)^2 \frac{1}{1.4} \approx \frac{1}{4270}$$

Hence the channel dimension are :

$$B = 9.1 \text{ m}; D = 1.83 \text{ and } S = 1/4270$$

14.16. EXAMPLES FROM COMPETITIVE EXAMINATIONS

Example 14.19. A most efficient trapezoidal section is required to give a maximum discharge of $21.5 \text{ m}^3/\text{s}$ of water. The slope of the channel bottom is 1 in 2500. Taking $C = 70 \text{ m}^{1/2}/\text{sec}$. in Chezy's equation, determine the dimensions of the channel. Also determine the value of Manning's n taking the value of velocity of flow as obtained for the channel by Chezy's equation. (Engg. Services Exam. 1997)

Solution : We have seen in §14.15 that a most efficient trapezoidal section is the one which has $R = D/2$ and $r = 1/\sqrt{3}$. Corresponding to these, the values of B , A , P and R are as under :

$$\begin{aligned} B &= 2D(\sqrt{1+r^2} - r) \\ A &= D^2(2\sqrt{1+r^2} - r) \\ \text{and } P &= 2D(2\sqrt{1+r^2} - r) \end{aligned} \quad \text{where } r = 1/\sqrt{3} = 0.5774$$

Note: The students are required to prove the above expressions.

Now, $Q = V \times A = C \sqrt{RS} \times A$

Substituting $Q = 21.5 (\text{m}^3/\text{s})$, $C = 70 (\text{m}^{1/2}/\text{s})$ and $S = 1/2500$, we have

$$21.5 = 70 \sqrt{R} \left(\frac{1}{\sqrt{2500}} \right) A = \frac{70}{50} A \sqrt{R} \quad \dots(i)$$

Now $A = D^2 \left(2\sqrt{1+\frac{1}{3}} - \frac{1}{\sqrt{3}} \right) = 1.732 D^2$ and $R = 0.5 D$

$$21.5 = \frac{70}{50} (1.732 D^2) \sqrt{0.5 D} = 1.7146 D^{2.5}$$

From which $D = 2.75 \text{ m}$

$$B = 2D \left(\sqrt{1+\frac{1}{3}} - \frac{1}{\sqrt{3}} \right) = 1.1547 D = 1.1547 \times 2.75 \approx 3.18 \text{ m}$$

Hence the channel has the following dimensions :

$$B = 3.18 \text{ m}; D = 2.75 \text{ m; side slopes } 1/\sqrt{3} (H) : 1$$

Now from Chezy's formula, $V = C \sqrt{RS} = 70 \left(\frac{2.75}{2} \right)^{1/2} \left(\frac{1}{2500} \right)^{1/2} = 1.642 \text{ m/s}$

From Manning's equation, $V = \frac{1}{n} R^{2/3} S^{1/2}$

$$1.642 = \frac{1}{n} \left(\frac{2.75}{2} \right)^{2/3} \left(\frac{1}{2500} \right)^{1/2} = \frac{0.02473}{n}$$

From which,

$$n = 0.0161$$

Example 14.20. Design a regime channel to carry a discharge of 100 cumecs in soil having a silt factor of 1.1, using Lacey's theory. Assume side slopes of 0.5 horizontal to 1.0 vertical. (Engg. Services Exam. 1984)

Solution : Given : $Q = 100$ cumecs; $f = 1.1$; side slopes $0.5 : 1$

$$V = \left(\frac{Q f^2}{140} \right)^{1/4} = \left(\frac{100 (1.1)^2}{140} \right)^{1/4} = 0.976 \text{ m/sec.}$$

$$A = Q/V = 100/0.976 = 102.46 \text{ m}^2$$

$$P = 4.75 \sqrt{Q} = 4.75 (100)^{1/2} = 47.5 \text{ m}$$

$$D = \frac{P - \sqrt{P^2 - 8.944 A}}{3.472} = \frac{47.5 - \sqrt{(47.5)^2 - 8.944 \times 102.46}}{3.472} \approx 2.36 \text{ m}$$

$$B = P - 2.236 D = 47.5 - 2.236 \times 2.36 \approx 42.2 \text{ m}$$

Check : $R = \frac{5 V^2}{2 f} = \frac{5 (0.976)^2}{2 \times 1.1} \approx 2.16 \text{ m}$

Also, $R = \frac{BD + D^2/2}{B + 2.236 D} = \frac{42.2 \times 2.36 + 0.5 (2.36)^2}{42.2 + 2.236 \times 2.36} \approx 2.16 \text{ m}$

Hence OK.

Lastly, $S = \frac{f^{6/3}}{3340 Q^{1/4}} = \frac{(1.1)^{6/3}}{3340 (100)^{1/4}} \approx \frac{1}{6140}$

Example 14.21. Design a channel by Lacey's theory for 40 cumecs capacity. The side slopes may be assumed to be 1:1. The average size of the bed material may be taken as 0.8 mm. (Civil Services Exam. 1986)

Solution

Silt factor, $f = 1.76 \sqrt{m_d} = 1.76 \sqrt{0.8} \approx 1.57$

$$V = \left(\frac{Q f^2}{140} \right)^{1/4} = \left(\frac{40 (1.57)^2}{140} \right)^{1/4} = 0.943 \text{ m/sec}$$

$$A = Q/V = 40/0.943 = 42.41 \text{ m}^2$$

$$P = 4.75 \sqrt{Q} = 4.75 \sqrt{40} = 30.04 \text{ m}$$

For a trapezoidal canal with 1:1 side slopes, we have

$$A = D \times \frac{B + (B + 2D)}{2} = D(B + D)$$

$$P = B + 2(D\sqrt{2}) = B + 2.828 D$$

Hence $D(B + D) = 42.41 \quad \dots(i)$

and $B + 2.828 D = 30.04 \quad \dots(ii)$

Substituting the value of B from (ii)

into (i), we get

$$D^2 + D(30.04 - 2.828 D) = 42.41$$

$$\text{or } 1.828 D^2 - 30.04 D + 42.41 = 0$$

from which $D = 1.56 \text{ m}$

Hence from (ii),

$$B = 30.04 - 2.828 \times 1.56 = 25.63 \text{ m} \approx 25.7 \text{ m}$$

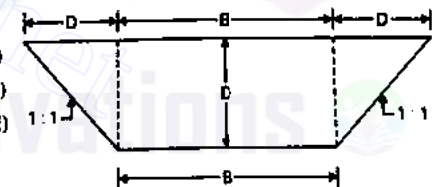


FIG. 14.8.

Check :

$$R = \frac{5}{2} \frac{V^2}{f} = \frac{5}{2} \frac{(0.943)^2}{1.57} = 1.416 \text{ m}$$

$$A = 1.56 (25.63 + 1.56) = 42.416 \text{ m}^2$$

$$P = 25.63 + 2.828 \times 1.56 = 30.04$$

$$R = 42.416 / 30.04 \approx 1.412 \text{ m. Hence OK.}$$

Finally,

$$S = \frac{f^{1/3}}{3340 Q^{1/6}} = \frac{(1.57)^{1/3}}{3340 (40)^{1/6}} = \frac{1}{2912}$$

Example 14.22. A certain stretch of a lined trapezoidal channel has one side vertical wall and the other 45° sloping wall. It is to deliver water at 30 m³/sec with a velocity of 1 m/sec. Compute bed width and flow depth for minimum lining area. (Engg. Services Exam. 1988)

Solution :

Given : $Q = 30$ cumecs; $V = 1$ m/sec.

$$A = Q/V = 30/1 = 30 \text{ m}^2$$

The canal section is shown in Fig. 14.9.

$$A = D [B + (B + D)]/2 = 0.5 D (2B + D)$$

$$\text{Hence } 0.5 D (2B + D) = 30 \text{ m}^2 \quad \dots(1)$$

For minimum lining area, wetted perimeter will be minimum.

$$P = D + B + D\sqrt{2} = B + 2.414 D \quad \dots(2)$$

FIG. 14.9.

$$\text{But from (1), } B = \frac{1}{2} \left(\frac{30}{0.5 D} - D \right) = \frac{30}{D} - \frac{D}{2} \quad \dots(3)$$

$$\text{Hence from (2), } P = \frac{30}{D} - \frac{D}{2} + 2.414 D = \frac{30}{D} + 1.914 D \quad \dots(4)$$

$$\therefore \frac{dP}{dD} = 0 = -30 D^{-2} + 1.914, \text{ from which } D \approx 3.96 \text{ m}$$

$$\text{Hence from (3), } B = \frac{30}{3.96} - \frac{3.96}{2} \approx 5.6 \text{ m}$$

$$\text{Check : } A = 0.5 D (2B + D) = 0.5 \times 3.96 (2 \times 5.6 + 3.96) \approx 30 \text{ m}^2$$

Hence OK.

Example 14.23. A trapezoidal channel with a base of 6 m and side slopes of 2 horizontal to 1 vertical conveys water at 17 m³/sec with a depth of 1.5 m. Is the flow situation sub or super critical ? (Engg. Services Exam. 1988)

Solution : Given : $B = 6$ m; $D = 1.5$ m; $Q = 17$ m³/sec.

The canal section is shown in Fig 14.10, from which

$$A = 1.5 \left(\frac{6 + 12}{2} \right) = 13.5 \text{ m}^2.$$

$$V = Q/A = 17/13.5 = 1.259 \text{ m/sec.}$$

$$\text{Wetted perimeter } P = 6 + 2\sqrt{(3)^2 + (1.5)^2} = 12.71 \text{ m}$$

$$\therefore R = A/P = 13.5/12.71 = 1.062$$

$$\text{Froude's number} = \frac{V}{\sqrt{gR}} = \frac{1.259}{\sqrt{9.81 \times 1.062}} = 0.39 < 1$$

Hence the flow is sub-critical.

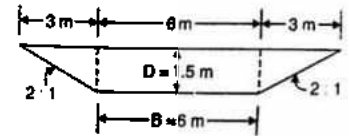


FIG. 14.10.

Example 14.24. A trapezoidal channel with bottom width 3.5 m and side slopes 1 H : 1 V on the left and 1.5 H : 1 V on the right with $n = 0.016$ and a bed slope of 2.6 in 10000, carries a discharge of 8 m³/sec. Determine the normal depth and the average shear stress on the channel bed. (Engg. services Exam, 2003)

Solution :

Given $B = 3.5$ m ; $Q = 8$ m³/sec.

Left side slope = 1 : 1

Right side slope = 1.5 : 1

The channel section is shown in Fig.

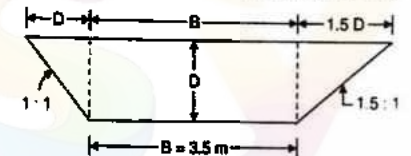


FIG. 14.11.

14.11.

$$A = D \left[\frac{B + (B + 2.5 D)}{2} \right] = D (B + 1.25 D) = D (3.5 + 1.25 D) \quad \dots(1)$$

$$P = B + D\sqrt{2} + \sqrt{D^2 + (1.5 D)^2} = B + 3.217 D = 3.5 + 3.217 D \quad \dots(2)$$

$$\therefore R = \frac{A}{P} = \frac{D (3.5 + 1.25 D)}{3.5 + 3.217 D} \quad \dots(3)$$

$$\text{Also, } V = Q/A = 8 / D (3.5 + 1.25 D) \quad \dots(4)$$

From Mannings formula, we have

$$V = \frac{1}{n} R^{2/3} S^{1/2}$$

$$\therefore \frac{8}{D (3.5 + 1.25 D)} = \frac{1}{0.016} R^{2/3} \left(\frac{2.6}{10000} \right)^{1/2}$$

$$R = \left[\frac{7.9382}{D (3.5 + 1.25 D)} \right]^3 \quad \dots(5)$$

Equating (3) and (5), we get

$$\frac{D (3.5 + 1.25 D)}{3.5 + 3.217 D} = \frac{22.966}{D^{1.5} (3.5 + 1.25 D)^{1.5}}$$

or

$$[D (3.5 + 1.25 D)]^{2.5} = 78.281 + 71.951 D \quad \dots(6)$$

Solving Eq. (6) by trial and error, we get $D \approx 1.5$ m

IRRIGATION AND WATER POWER ENGINEERING

702

Hence
$$R = \frac{1.5(3.5 + 1.25 \times 1.5)}{3.5 + 3.217 \times 1.5} \approx 0.97 \text{ m}$$

Check : From (5),
$$R = \left[\frac{7.9382}{1.5(3.5 + 1.25 \times 1.5)} \right]^{1.48} = 0.98 \text{ m}$$

Both the values are nearly equal. Hence OK.

Average shear stress on channel bed $= wRS$

$$= 9.81 \times 0.97 \times \frac{2.6}{10000} = 2.47 \times 10^{-3} \text{ kN/m}^2$$

$$= 2.47 \text{ N/m}^2$$

Example 14.25. A channel is 40 m wide 2.6 m deep, with a slope 1 in 5000. The representative bed material size D_{65} is 0.64 mm, and mean diameter of the bed material is 0.38 mm. Manning's $n = 0.021$. Neglecting the side friction, find the bed load transported by the channel in tonnes per day by Meyer-Peter equation, considering B/D as large and $\tau_c = 0.075 d$. (Engg. Services Exam 2003)

Solution : Given $B = 40 \text{ m}$; $D = 2.6 \text{ m}$; $S = 1/5000$

$$D_{65} = 0.64 \text{ mm}; m_d = 0.38 \text{ mm}; n = 0.021; \tau_c = 0.075 d$$

From Eq. 14.39 (b), bed load transported is given by

$$q_s = 4700 \left[\tau_b (N^*/N)^{3/2} - \tau_c \right]^{1/2} \text{ kg/m/hr.}$$

where $\tau_b = wDS = 1000 \times 2.6 \times \frac{1}{5000} = 0.52 \text{ kg/m}^2$

$$N^* = \frac{(k_s)^{1/6}}{24} = \frac{(D_{95})^{1/6}}{24} = \frac{(0.64 \times 10^{-3})^{1/6}}{24} = 0.0122$$

$$N = \text{actual Manning's coefficient} = 0.021 \text{ (given)}$$

$$\tau_c = 0.075 d \text{ where } d = \text{mean diameter} = 0.38 \text{ mm}$$

$$= 0.075 \times 0.38 = 0.0285 \text{ kg/m}^2$$

$$q_s = 4700 \left[0.52 \left(\frac{0.0122}{0.021} \right)^{3/2} - 0.0285 \right]^{1/2} \approx 426 \text{ kg/m/hr}$$

$$= 426 \times 10^{-3} \times 24 \text{ t/m/day} = 10.22 \text{ t/m/day}$$

$$\therefore \text{Bed load transported by 40 m wide channel}$$

$$= 10.22 \times 40 \text{ t/day} \approx 409 \text{ t/day}$$

Example 14.26. Calculate the bed width for an irrigation channel to carry a discharge of 5 cumecs. Side slopes of the channel are 1/2 horizontal to 1 vertical. Assume a bed slope of 0.2 m per km. Using Kutter's equation $V = C \sqrt{RS}$, check the depth. The value of C is 30. V is mean velocity of flow in m/sec and R is hydraulic mean radius in m.

Solution : Given $Q = 5 \text{ m}^3/\text{sec}$; $S = 0.2/1000$; $m = 0.8$
Assuming $D = 1 \text{ m}$ is the first trial,

IRRIGATION CHANNELS - I: SILT THEORIES

703

$$V_0 = 0.55 m D^{0.44} = 0.55 \times 0.8 (1)^{0.44} = 0.44 \text{ m/s}$$

$$A = Q/V_0 = 5/0.44 = 11.36 \text{ m}^2$$

For a trapezoidal section having 1/2 : 1 side slopes,

$$A = BD + 0.5 D^2 = D(B + 0.5 D)$$

$$11.36 = 1(B + 0.5 \times 1) \text{ from which } B = 10.86 \text{ m}$$

$$P = B + D\sqrt{5} = 10.86 + 1 \times 2.236 \approx 13.1 \text{ m}$$

and

$$R = A/P = 11.36/13.1 \approx 0.867 \text{ m}$$

$$\text{Actual } V = C \sqrt{RS} = 30 \left(0.867 \times \frac{0.2}{1000} \right)^{1/2} = 0.395 \text{ m/s.}$$

Thus,

$$V_0 > V$$

In order to reduce V_0 given by Kennedy's equation, reduce D in the second trial.

Second trial : Let $D = 0.8 \text{ m}$.

$$V_0 = 0.55 \times 0.8 (0.8)^{0.44} = 0.381 \text{ m/s}$$

$$A = 5/0.381 = 13.1 \text{ m}^2$$

$$B = \frac{A}{D} - 0.5 D = \frac{13.1}{0.8} - 0.5 \times 0.8 = 15.99 \text{ m}$$

$$P = B + 2.236 D = 15.99 + 2.236 \times 0.8 = 17.77 \text{ m}$$

$$R = 13.1/17.77 = 0.737 \text{ m}$$

$$\text{Actual } V = 30 \left(0.737 \times \frac{0.2}{1000} \right)^{1/2} = 0.384 \text{ m/s}$$

This,

$$V_0 > V. \text{ Hence reduce } D \text{ further in the next trial.}$$

Third trial : Let $D = 0.7 \text{ m}$

$$V_0 = 0.55 \times 0.8 (0.7)^{0.44} = 0.35 \text{ m/s.}$$

$$A = 5/0.35 = 14.28 \text{ m}^2$$

$$B = \frac{14.28}{0.7} + 0.5 \times 0.7 = 20.75 \text{ m}$$

$$P = 20.75 + 2.236 \times 0.7 = 22.31 \text{ m}$$

$$R = 14.28/22.31 = 0.64 \text{ m}$$

$$\text{Actual } V = 30 \left(0.64 \times \frac{0.2}{1000} \right)^{1/2} \approx 0.34 \text{ m/s}$$

Since

$$V \approx V_0, \text{ no further trials are required.}$$

Hence

$$D = 0.7 \text{ m and } B = 20.75 \text{ m}$$

Example 14.27. Design an irrigation channel by Kennedy method to carry 50 cubic meters per second of discharge with base width to depth ratio as 2.5, the critical velocity ratio is 1.1. Assume Kutter's rugosity coefficient as 0.025 and side slope of the channel as 0.5 horizontal to 1.0 vertical. (Engg. Services Exam. 1994)

Solution : Given $B/D = 2.5$; $Q = 50 \text{ cumecs}$; $m = 1.1$; $N = 0.025$

IRRIGATION AND WATER POWER ENGINEERING

704

For a trapezoidal channel with $\frac{1}{2}:1$ side slopes,

$$A = BD + 0.5 D^2 = D^2 \left(\frac{B}{D} + 0.5 \right) = D^2 (2.5 + 0.5) = 3 D^2$$

$$V_0 = 0.55 m D^{0.44} = 0.55 \times 1.1 D^{0.44} = 0.605 D^{0.44}$$

$$Q = A \times V_0 = (3 D^2) (0.605 D^{0.44}) = 1.815 D^{2.44}$$

$$D = \left(\frac{Q}{1.815} \right)^{1/2.44} = \left(\frac{50}{1.815} \right)^{1/2.44} = 3.51 \text{ m}$$

$$B = 2.5 D = 2.5 \times 3.51 = 8.78 \text{ m}$$

$$R = \frac{BD + 0.5 D^2}{B + 2.236 D} = \frac{8.78 \times 3.51 + 0.5 (3.51)^2}{8.78 + 2.236 \times 3.51} = 2.224 \text{ m}$$

$$V_0 = 0.605 (3.51)^{0.44} = 1.351 \text{ m/sec.}$$

But

$$V = \frac{23 + \frac{1}{N} + \frac{0.00155}{S}}{1 + \left(23 + \frac{0.00155}{S} \right) \frac{N}{\sqrt{R}}} \sqrt{RS}$$

$$1.351 = \frac{23 + \frac{1}{0.0225} + \frac{0.00155}{S}}{1 + \left(23 + \frac{0.00155}{S} \right) \frac{0.025}{\sqrt{2.224}}} \sqrt{2.224 \times S}$$

$$\text{or } 1.351 = \frac{23 + 40 + 0.00155/S}{1.3856 + \frac{2.598}{S} \times 10^{-3}} \times 1.491 S^{1/2}$$

$$\text{or } \frac{1.351}{1.491} \left(1.3856 + \frac{2.598}{S} \times 10^{-3} \right) = \left[63 + \frac{1.55}{S} \times 10^{-3} \right] S^{1/2}$$

$$\text{or } 63 S^{3/2} + 1.55 \times 10^{-3} S^{1/2} - 1.255 S = 2.354 \times 10^{-5}$$

$$\text{or } S^{3/2} + 2.46 \times 10^{-5} S^{1/2} - 1.99 \times 10^{-2} S = 3.737 \times 10^{-7}$$

Solving this by trial and error, we get $S \approx \frac{1}{2600}$

Example 14.28. Design a regime channel for a discharge of $35 \text{ m}^3/\text{s}$ with silt factor of 0.9 by Lacey's theory, taking side slopes as $1 H : 2 V$.

(Engg. Services Exam. 2008)

Solution : The above Example is practically the same as Example 14.6 except that Q has been increased from 30 to 35 cumecs.

Given : $Q = 35 \text{ cumecs}$; $f = 0.9$

Side slopes = $1 (H) : 2 (V) = \frac{1}{2} (H) : 1 (V)$

IRRIGATION CHANNELS - I: SILT THEORIES

705

1. $V = \left(\frac{Q f^2}{140} \right)^{1/3} = \left(\frac{35 \times 0.9^2}{140} \right)^{1/3} = 0.766 \text{ m/sec.}$
2. $A = Q/V = 35/0.766 \approx 45.67 \text{ m}^2$
3. $P = 4.75 \sqrt{Q} = 4.75 \sqrt{35} = 28.1 \text{ m}$
4. $D = \frac{P - \sqrt{P^2 - 6.944 A}}{3.472} = \frac{28.1 - \sqrt{28.1^2 - 6.944 \times 45.67}}{3.472} = 1.833 \text{ m} \approx 1.84$
5. $B = P - 2.236 D = 28.1 - 2.236 \times 1.833 \approx 24 \text{ m}$
6. $S = \frac{f^{3/2}}{3340 Q^{1/2}} = \frac{0.9^{3/2}}{3340 (35)^{1/2}} \approx \frac{1}{7200}$

Example 14.29. Design an irrigation canal for a discharge of 15 cumecs, $N = 0.0225$, $m = 1$ and $B/D = 5.7$. Use Kennedy's theory.

(Civil Services Exams 1987)

Solution : This question is exactly the same as example 14.2 except that discharge Q has been increased from 14 cumecs to 15 cumecs. For solution, refer Example 14.2.

Example 14.30. Design the section of an unlined canal in a loamy soil to carry a discharge of 50 cum/sec with permissible velocity of 1 m/sec. Assume side slopes of 2:1 and B/D ratio as 6.0. Using Manning's formula, calculate the bed slope of the canal, $N = 0.0225$. (Civil Services Exam. 1983).

Solution : Given : $Q = 50 \text{ cumecs}$;
 $V = 1 \text{ m/sec}$; $N = 0.0225$; $B/D = 6.0$

Side slopes = $2 (H) : 1 (V)$

$$A = Q/V = 50/1 = 50 \text{ m}^2$$

For a canal of side slopes 2:1

$$A = D \left[\frac{B + (B + 4D)}{2} \right] = D (B + 2D)$$

$$\text{or } A = D^2 \left(\frac{B}{D} + 2 \right) \text{ where } A = 50 \text{ m}^2 \text{ and } B/D = 6$$

Hence $50 = D^2 (6 + 2)$ from which $D = 2.5 \text{ m}$

$$\therefore B = 6D = 2.5 \times 6 = 15 \text{ m}$$

$$P = B + 2 \sqrt{D^2 + 4D^2} = B + 4.472 D = 15 + 4.472 \times 2.5 = 26.18 \text{ m}$$

$$R = A/P = 50/26.18 \approx 1.91 \text{ m}$$

$$\text{From Manning's formula, } S = \left[\frac{VN}{R^{2/3}} \right]^2 = \left[\frac{1 \times 0.0225}{(1.91)^{2/3}} \right]^2 \approx \frac{1}{4980}$$

Example 14.31. In a wide stream, a suspended load sample taken at a height of 0.3 m from the bed indicated a concentration of 1000 ppm of sediment by weight. The stream is 5.0 m deep and has a bed slope of $1/4000$. The bed material can be assumed to be uniform size with a fall velocity of 2.0 cm/s. Estimate the concentration of the suspended load at mid-depth.

(Civil Services Exam. 1991)

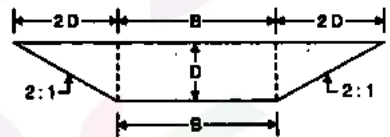


FIG. 14.12.

IRRIGATION AND WATER POWER ENGINEERING

706

Solution : Given $C_a = 1000$ ppm at 0.3 m above bed ; $V_s = 2.0$ cm/sec = 0.02 m/sec
 $a = 0.3$ m ; $D = 5.0$ m and $y = 2.5$ m
 $R = D = 5$ m for wide channel.

$$\text{Shear friction velocity } V_* = \sqrt{gRS} = \sqrt{9.81 \times 5 \times \frac{1}{4000}} = 0.1108$$

$$\frac{V_s}{V_* k} = \frac{0.02}{0.1108 \times 0.4} = 0.452$$

From Eq. 14.35,

$$\frac{C}{C_a} = \left[\frac{a}{y} \times \frac{D-y}{D-a} \right]^{V_s/V_* k}$$

$$C = 1000 \left[\frac{0.3}{2.5} \times \frac{5-2.5}{5-0.3} \right]^{0.452} = 288 \text{ ppm}$$

Example 14.32. A stable channel is to be designed for a discharge of 40 m³/sec and silt factor of unity. Calculate the dimensions of the channel using Lacey's regime equation. Also calculate the dimensions of the channel if it were to be designed on the basis of Kennedy's method with critical velocity ratio equal to unity, and the ratio of bed width to depth of flow the same as obtained from Lacey's method.
 (Civil Services Exam. 1993)

Solution : Given : $Q = 40$ cumecs ; $f = 1$

(a) Canal design by Lacey's theory

$$1. \quad V = \left(\frac{Qf^2}{140} \right)^{1/4} = \left(\frac{40 \times 1^2}{140} \right)^{1/4} = 0.812 \text{ m/sec.}$$

$$2. \quad A = Q/V = 40/0.812 = 49.29 \text{ m}^2$$

$$3. \quad P = 4.75 \sqrt{Q} = 4.75 \sqrt{40} = 30.04 \text{ m}$$

4. For a channel with $\frac{1}{2} : 1$ side slope,

$$D = \frac{P - \sqrt{P^2 - 6.944A}}{3.472} = \frac{30.04 - \sqrt{(30.04)^2 - 6.944 \times 49.29}}{3.472}$$

$$= 1.833 \text{ m} \pm 1.84 \text{ m}$$

$$B = P - 2.236D = 30.04 - 2.236 \times 1.833 = 25.94 \pm 26 \text{ m}$$

$$5. \quad S = \frac{f^{1/3}}{3340 Q^{1/4}} = \frac{1^{1/3}}{3340 (40)^{1/4}} = \frac{1}{6177}$$

(b) Canal design by Kennedy's theory

$$\frac{B}{D} = \frac{25.94}{1.833} = 14.15 ; m = 1$$

Let

$$1. \quad A = BD + 0.5 D^2 = D^2 \left(\frac{B}{D} + 0.5 \right) = D^2 (14.15 + 0.5) = 14.65 D^2$$

$$2. \quad V_* = 0.55 m D^{0.64} = 0.55 \times 1 \times D^{0.64}$$

IRRIGATION CHANNELS - I: SILT THEORIES

707

$$Q = A V_* = (14.65 D^2) (0.55 \times D^{0.64}) = 8.058 D^{2.64}$$

$$D = \left(\frac{Q}{8.058} \right)^{1/2.64} = \left(\frac{40}{8.058} \right)^{1/2.64} = 1.835 \pm 1.84 \text{ m}$$

$$B = 14.15 D = 14.15 \times 1.835 = 25.96 \pm 26 \text{ m}$$

Thus, we get the same values of B and D , as obtained by Lacey's theory.

Example 14.33. Design a Lacey's regime channel flowing through material of average diameter 0.328 mm, for the following data :

Culturable commanded area = 100000 hectares

Intensity of irrigation

Rabi = 40%

Kharif = 30%

Outlet discharge factor

Rabi = 1800 hac/cumec

Kharif = 800 hac/cumec

Assume conveyance losses to be 10%

(Civil Services Exam. 1994)

Solution

$$\text{Area under Rabi} = 100000 \times 0.4 = 40000 \text{ hac}$$

$$\text{Area under Kharif} = 100000 \times 0.3 = 30000 \text{ hac.}$$

$$\therefore \text{Outlet discharge for Rabi} = \frac{40000}{1800} = 22.22 \text{ m}^3/\text{s}$$

$$\text{and outlet discharge for Kharif} = \frac{30000}{800} = 37.5 \text{ m}^3$$

$$\therefore \text{Design discharge for canal} = 1.1 \times 37.5 = 41.25 \text{ cumecs}$$

$$\text{Silt factor } f = 1.76 \sqrt{m_d} = 1.76 \sqrt{0.328} \pm 1$$

$$V = \left(\frac{Qf^2}{144} \right)^{1/4} = \left(\frac{41.25 \times 1^2}{144} \right)^{1/4} = 0.812 \text{ m/sec}$$

$$A = \frac{Q}{V} = \frac{41.25}{0.812} = 50.81 \text{ m}^2$$

$$P = 4.75 \sqrt{Q} = 4.75 \sqrt{41.25} = 30.51$$

$$D = \frac{P - \sqrt{P^2 - 6.944A}}{3.472} = \frac{30.51 - \sqrt{(30.51)^2 - 6.944(50.81)}}{3.472}$$

$$= 1.863 \pm 1.87 \text{ m}$$

$$B = P - 2.236D = 30.51 - 2.236 \times 1.863 = 26.34 \text{ m} \pm 26.4 \text{ m}$$

$$S = \frac{f^{1/3}}{3340 (Q)^{1/4}} = \frac{1^{1/3}}{3340 (41.25)^{1/4}} = \frac{1}{6210}$$

Example 14.34. Write the Rouse's equation for distribution of suspended sediment in the vertical of a channel and label all the terms.

IRRIGATION AND WATER POWER ENGINEERING

708

In a wide alluvial channel having suspended sediment load, the depth of flow is 2.8 m. If the suspended load sample indicated a concentration of 700 ppm at a point 10 cm above the bed and 30 cm below water surface, estimate the concentration at a point 10 cm above the bed in the same vertical. The exponent in Rouse's equation can be assumed as 0.4. (Civil Services Exam. 2002)

Solution

Rouse equation for distribution of suspended load is given by

$$\frac{C}{C_a} = \left[\frac{a(D-y)}{y(D-a)} \right]^{1/V_s} \quad \dots(14.35)$$

Here, the exponent $V_s/k V_s$ has been given as equal to 0.4.

Also, $D = 2.8$ m; $a = 2.8 - 0.3 = 2.5$ m; $C_a = 700$ ppm; $y = 0.1$ m

$$C = 700 \left[\frac{2.5(2.8 - 0.1)}{0.1(2.8 - 2.5)} \right]^{0.4} = 6109 \text{ ppm}$$

Example 14.35. A distributory is to be designed to irrigate 3600 ha in Rabi and 1400 ha in Kharif. Kor depth (critical stage watering depth) and Kor period (critical stage water period) for Rabi and Kharif are 13.5 cm and 4 week, 19.00 cm and 2½ weeks respectively. Design the distributory by Lacey's theory and silt factor 0.85. (Civil Services Exam. 2005)

Solution :

Area for Rabi = 3600 ha

Kor depth for Rabi = 13.5 cm

Kor period for Rabi = 4 weeks

$$\therefore \text{Duty for Rabi crop} = \frac{8.64 b}{\delta} = \frac{8.64 \times 4 \times 7}{0.135} = 1792 \text{ ha/cumes}$$

$$\therefore \text{Discharge for Rabi} = 3600/1792 \approx 2.01 \text{ cumeecs}$$

Area for Kharif = 1400 ha

Kor depth for Kharif = 19 cm

Kor period for Kharif = 2½ weeks

$$\therefore \text{Duty for Kharif} = \frac{8.64 \times 2.5 \times 7}{0.19} = 795.8 \text{ ha/cumes.}$$

$$\therefore \text{Discharge for Kharif} = 1400/795.8 = 1.76 \text{ cumeecs}$$

Hence design discharge for distributory = 2.01 cumeecs.

Channel design by Lacey's theory

$$1. \quad V = \left(\frac{Q}{140} \right)^{1/4} = \left[\frac{2.01(0.85)^2}{140} \right]^{1/4} = 0.487 \text{ m/sec.}$$

$$2. \quad A = 2.01/0.487 = 4.304 \text{ m}^2$$

$$3. \quad P = 4.75 \sqrt{Q} = 4.75 \sqrt{2.01} = 6.73 \text{ m}$$

IRRIGATION CHANNELS - I: SILT THEORIES

709

$$4. \quad D = \frac{P - \sqrt{P^2 - 6.944 A}}{3.472} = \frac{6.73 - \sqrt{(6.73)^2 - 6.944 \times 4.304}}{3.472} = 0.806 \text{ m} \approx 0.81 \text{ m}$$

$$5. \quad S = \frac{f^{1/3}}{3340 (Q)^{1/4}} = \frac{(0.85)^{1/3}}{3340 (2.01)^{1/4}} = \frac{1}{4919}$$

PROBLEMS

1. Describe Kennedy's silt theory. A channel is silting badly in the head reach. How would you proceed to determine its cause and what remedies would you suggest.
2. Explain the procedure of designing a channel with Kennedy's theory.
3. For a channel, the discharge (Q), rugosity (N), critical velocity ratio (m) and the bed width-depth ratio (B/D) are given. Explain how would you design the channel using Kennedy's theory?
4. Derive an expression for the silt supporting capacity of a channel according to Kennedy's theory.
5. Explain Lacey's silt theory.
6. Describe the method of designing a canal based on Lacey's theory.
7. Compare Kennedy's and Lacey's silt theories. Why is Lacey's conception superior to that of Kennedy's?
8. What do you understand by (a) regime channel, (b) initial and permanent regime of channels?
9. Using Lacey's theory, design an irrigation channel for the following data :
Discharge $Q = 50$ cumeecs
Silt factor $f = 1$
Side slopes = $\frac{1}{2} : 1$ [Ans. $B = 29.6$ m ; $D = 1.98$ m, $S = 1/6400$]
10. Using Lacey's basic regime equations, derive an expression for Lacey's scour depth.
11. With the help of basic regime equations given by Lacey, derive the perimeter-discharge relationship.
12. The slope of a channel in alluvial soil is 1/5000. Find the channel section and the maximum discharge which can be allowed to flow in it. Take Lacey's silt factor $f = 1$. The channel is of trapezoidal section, having side slopes $\frac{1}{2} : 1$. [Ans. $b = 22.3$ m, $d = 1.97$ m, $Q = 30$ cumeecs]

13. Using Kennedy's theory, design a channel section for the following data :
Discharge $Q = 14$ cumeecs
Kutter's $N = 0.0225$
Critical velocity ratio $m = 1$
Side slopes $\frac{1}{2} : 1$
Bed slope = $\frac{1}{5000}$ [Ans. $b = 9.7$ m ; $d = 1.71$ m]

15

Irrigation Channels - II : Design Procedure

15.1. LONGITUDINAL SECTION OF A CANAL

After having fixed the alignment of canal on shajra sheet as indicated in Chapter 13, the longitudinal section of the alignment is taken on the field. The longitudinal section should thereafter be plotted to a horizontal scale of 1 cm = 160 m and to vertical scale of 2 cm = 1 m. Vertical scale should be changed if desired according to the magnitude of fall available.

For drawing longitudinal section, following steps are generally followed :

- (1) Plot the ground level along the alignment of the channel with the reference to convenient datum.
- (2) Mark the full supply level (F.S.L.) and the bed level of the parent channel just on the upstream of head regulator of the off-taking channel for reference purposes.
- (3) Draw the F.S.L. of the off-taking channel keeping following points in view. This is generally done (i) to provide for the loss of head at the head regulator, (ii) to meet the demand for extra supplies in the canal at a future date, (iii) to maintain the flow even if the channels get silted upto some extent in its head reaches. For main canals taking off from river, F.S.L. is kept about 1 m lower than the pond level of the reservoir.
- (b) F.S.L. should be above the ground level for most of its length ; F.S.L. may not be above isolated high patches of ground. Such areas can be irrigated by lift irrigation. The F.S.L. should, however, be only some centimetres above the ground level. Since the canal is running on watershed, the ground level on both sides of the canal will be lower and cross slopes available will be ample to provide irrigation to the entire command.
4. Choose bed slope close to Lacey's slope or that given by Wood's Table. If the slope available on the ground is lesser than that suggested by Lacey or Wood's should, however, be made to reduce silt factor as far as possible by preventing silt entry at head in such a case.

(710)

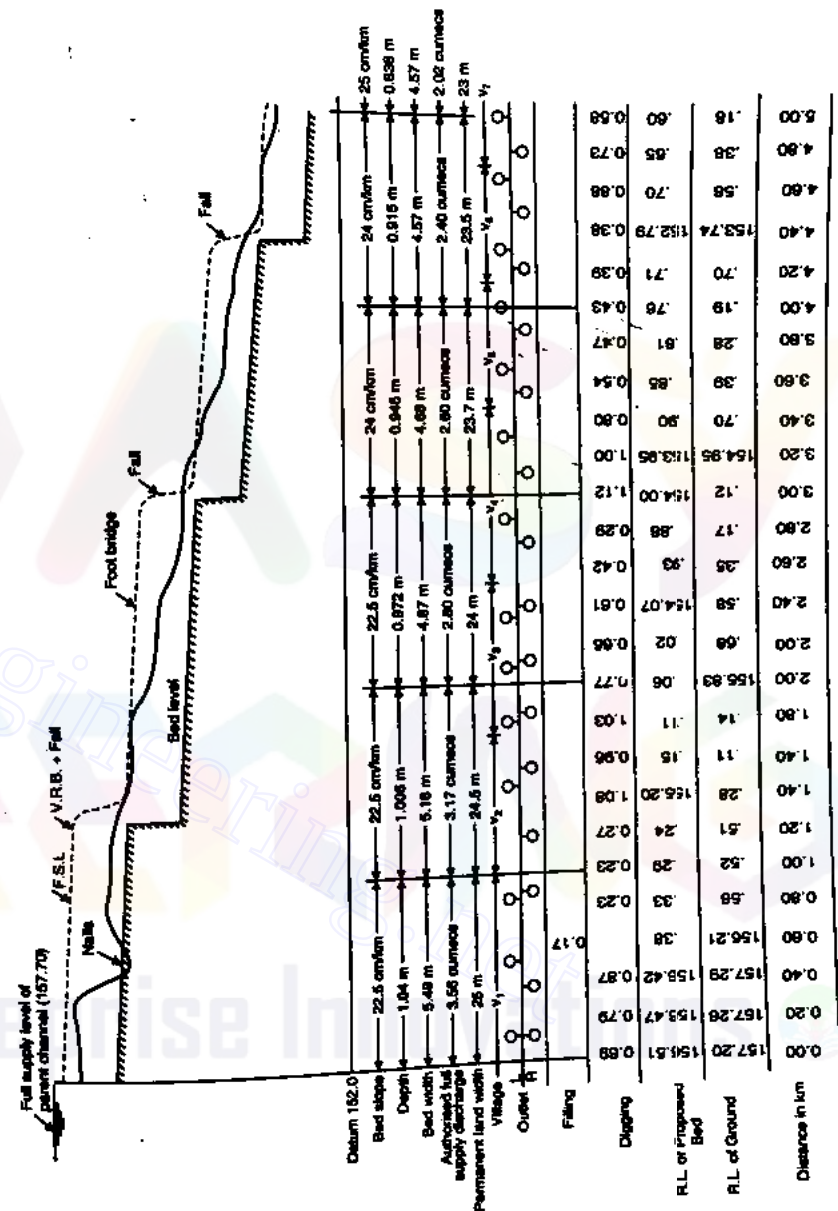


FIG. 15.1. LONGITUDINAL SECTION OF A CANAL.

5. Provide falls if the slope available in the ground is steeper than the one to be given to the canal bed. The magnitude of the drop should be such that F.S.L. of the canal downstream of fall should remain below the ground level for about half a km before emerging out of the ground level. Unbalanced earth work on both sides of falls should be kept minimum. The location of falls has been discussed in detail in Chapter 18.

(6) Keep the channel in balanced depth of cutting or filling as far as possible.

Fig 15.1 shows the longitudinal section of a canal.

15.2. BALANCING DEPTH

A canal section will be economical when the earth work involved at a particular section has an equal amount of cut and fill. Usually a canal section has a part in cutting and part in filling as shown in Fig. 15.2. If the amount of cut is equal to the amount of fill, it has to be paid for once only. More so, the necessity of a borrow pit or soil bank is entirely avoided. For a given cross-section there is always only one depth of cutting for which the cutting and filling will be equal. The depth is known as *balancing depth*. The balancing depth is worked out as under :

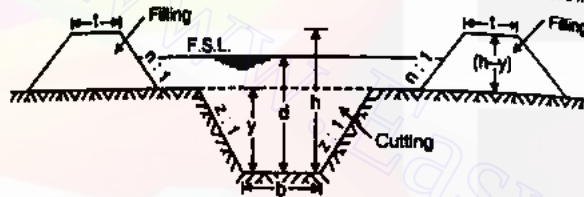


FIG. 15.2. BALANCING DEPTH.

If h = vertical height of top of bank from bed of canal.

b = bed width of the channel.

t = top width of the canal bank.

$n : 1$ = side slope of bank in filling.

$z : 1$ = side slope of canal in cutting.

d = full supply depth of the channel.

y = depth of cutting

$$\text{Area of the cut} = by + zy^2 = y(b + zy)$$

$$\text{Area of fill} = 2\{(h - y)t + n(h - y)^2\}.$$

Equating the area of cut and fill

$$y(b + zy) = 2\{(h - y)t + n(h - y)^2\}$$

$$\text{or } y^2(2n - z) - (b + 4nh + 2t)y + h(2t + 2nh) = 0 \quad \dots(15.1)$$

A canal is usually constructed with a side slope of 1:1 in cutting and a slope 1.5:1 in filling. Therefore, putting $n = 1.5$ and $z = 1$ in Eq. 15.1 we get.

$$y^2 - (b/2 + 3h + t)y + h\left(t + \frac{3}{2}h\right) = 0 \quad \dots(15.2)$$

From Eq. 15.1 the depth y from the ground line and the bed level is calculated. A line roughly parallel to the hydraulic gradient line is drawn on the L-section which envelops maximum number of ground points. Bed line is then drawn parallel to it and y metre below this line.

15.3. LOSSES IN CANALS

When water continuously flows through a canal, losses take place due to seepage, deep percolation and evaporation. These losses are some times known as *transmission losses*. These losses should be properly accounted for, otherwise lesser quantity of water will be available for cultivation at the tail end. Water losses in canals can be broadly classified under three heads :

(i) Evaporation losses, (ii) Transpiration losses., (iii) Seepage losses.

(i) *Evaporation losses*. The loss due to evaporation is generally a small percentage of the total loss in unlined canal. It hardly exceeds 1 to 2 percent of the total water entering into the canal. The evaporation losses depend upon (i) *Climatic factors* such as temperature, humidity and wind velocity, and (ii) *Canal factors* such as water surface area, water depth and velocity of flow. Maximum loss is there in summer months when temperatures are high and wind velocities are also high. Similarly, losses are maximum in unlined canals due to wider water surface area, shallower water depth and low velocity. The average evaporation loss per day may vary between 4 mm to 10 mm.

(ii) *Transpiration losses*. The transpiration loss takes place through lot of vegetation and weeds growth along the bank of canal. However, this forms a extremely small part of total loss.

(iii) *Seepage losses*. Seepage losses constitute major portion of loss in an unlined canal. The seepage losses are due to (i) absorption of water in the upper layers of soil below the canal bed, and due to (ii) *percolation* of water into the water table, thus raising the water table. If, however water table is much lower, seepage losses are only due to absorption. Percolation losses are always much more than the absorption losses.

Rate of water loss

Canal losses may be expressed in any one of the following methods :

- as cumecs per million square metre of the wetted perimeter.
- as depth of water lost per day over the wetted perimeter.
- as percentage of the canal discharge.
- as percentage per kilometer length of the canal.

Out of these four methods, the first method is the simplest, and is quite popular. In absence of any other data, the transmission losses may be taken as 2.5 cumecs per million square meter of wetted perimeter for unlined channels and as 0.60 cumecs per million sq. metre of wetted perimeter for lined canals.

In U.P., the loss Q_L in cumecs per km length of unlined canal is given by

$$Q_L = \frac{1}{200} (B + D)^{2/3} \quad \dots(15.3)$$

where B and D are in metres.

In Punjab and Haryana, the losses in unlined canals in cumecs per million square metre of wetted perimeter is given by the expression

$$Q_L = 1.9 Q^{0.0025} \quad \dots(15.4)$$

The Central Water Power Commission (CWPC) recommends the following values of losses in unlined canals (Table 15.1).

TABLE 15.1. CWPC RECOMMENDATION FOR LOSSES IN UNLINED CANALS

Soil Type	Transmission loss (cumecs/million sq meter of wetted perimeter)
1. Rock	0.81
2. Black cotton soil	1.83
3. Alluvial soil	2.74
4. Decayed rock or gravel	3.0

Losses are sometimes expressed as percentage of total discharge, as under.

Main canal and branches : 15% to 20 %

Distributaries and minors : 6% to 7%

Water courses : 17% to 22%

Losses in lined canals: In Punjab, the following formula is used for the determination of total losses in cumecs per Mm^2 (million square meters) of wetted perimeter

$$K = 0.349 Q^{0.666} \quad \dots(15.5)$$

where Q is the discharge in cumecs.

However as per recommendations of C.W.P.C. and IS: 1745-1968, the transmission losses in lined canals can be taken as 0.8 cumecs/ Mm^2 of the wetted perimeter.

15.4. SCHEDULE OF AREA STATISTICS AND CHANNEL DIMENSIONS

The design of channel cross-section from km to km is carried out in a Tabular form called the *Schedule of Area Statistics and Channel Dimensions*.

The schedule of area statistics and channel dimensions is shown in Tables 15.2 and 15.7. Working of the Table is explained below and has later been illustrated by Example 15.3.

TABLE 15.2. SCHEDULE OF AREA STATISTICS AND CHANNEL DIMENSIONS

	Reaches km	Area to be irrigated					Channel Dimensions											V/V ₀ Ratio or silt factor
		Gross commanded area (G.C.A.)	Culturable commanded area (C.C.A.)	Rabi (%)	Perennial (%)	Kharif (%)	Outlet discharge factor	Outlet discharge	Losses in reach	Total loss	Total discharge	Bed slope	Bed width	Water depth	Design discharge	Height of Bank	Width of Bank	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)
0																		
1																		
2																		
...																		
n																		

Col. 1. The actual design of channel is carried from km to km. Sometimes an off-taking channel may take off in between the kilometre interval. The channel dimensions are then also found out at the downstream of the cross regulator.

Col. 2. Column 2 indicates the gross commanded area, i.e. the entire area under the command of the channel below the particular kilometre at which it is being designed.

Col. 3. In this column, the area actually under cultivation below the particular km under the command of the channel is indicated.

Col. 4 to 6 indicate the percentage of area under Rabi, perennial and Kharif crops. Mostly Rabi crops are controlling crops for finding the discharge of the channel. Channels either designed for Rabi or Kharif crops suffice the purpose for sugar cane irrigation. However, under some local conditions the discharge needed for sugar-cane irrigation is found out.

Col. 7. Indicates the outlet discharge factor for the controlling crop, calculated by the methods indicated in Chapter 3.

Col. 8. Indicates the outlet discharge required and can be found out by multiplying area to be irrigated to the outlet discharge factor.

Col. 9. Indicates the losses in the reach from kilometre to kilometre. Generally losses per million square metre of the wetted surface area is known and so the channel losses in different reaches have to be calculated on the basis of tentative design for channels.

Col. 10. Indicates the total losses in the channel below any particular kilometre of the channel.

Col. 11. Indicates the total discharge for which the channel has to be designed. This discharge includes outlet discharge and losses.

Col. 12 to 18. These columns pertain to channel dimensions and are filled up after designing the channel in each reach based on Kennedy theory using Garret's diagrams or on Lacey's theory using Lacey's diagrams.

15.5. USE OF GARRET'S DIAGRAMS FOR CHANNEL DESIGN ON KENNEDY'S THEORY

Garret's diagrams give the graphical method of designing the channel dimensions based on Kennedy's theory. The original diagrams were prepared in F.P.S. units. Fig. 15.3 (a) and (b) show two such diagrams converted into metric units. The diagram has the discharge plotted on the abscissa. The ordinates on left indicate the slope and that on right the water depth in the channel and critical velocity V_0 . The discharge lines are curved and bed width lines are shown dotted.

The procedure in designing the channel consists of the following steps.

1. Find out the discharge for which the channel is to be designed. This can be found from Schedule of Area Statistics.
2. Find out the slope of the channel from its longitudinal section.
3. In Fig. 15.3, follow the discharge line and find out its intersection with the horizontal lines from slope. Interpolation may be done if needed to locate the discharge line. Mark the intersection point P .

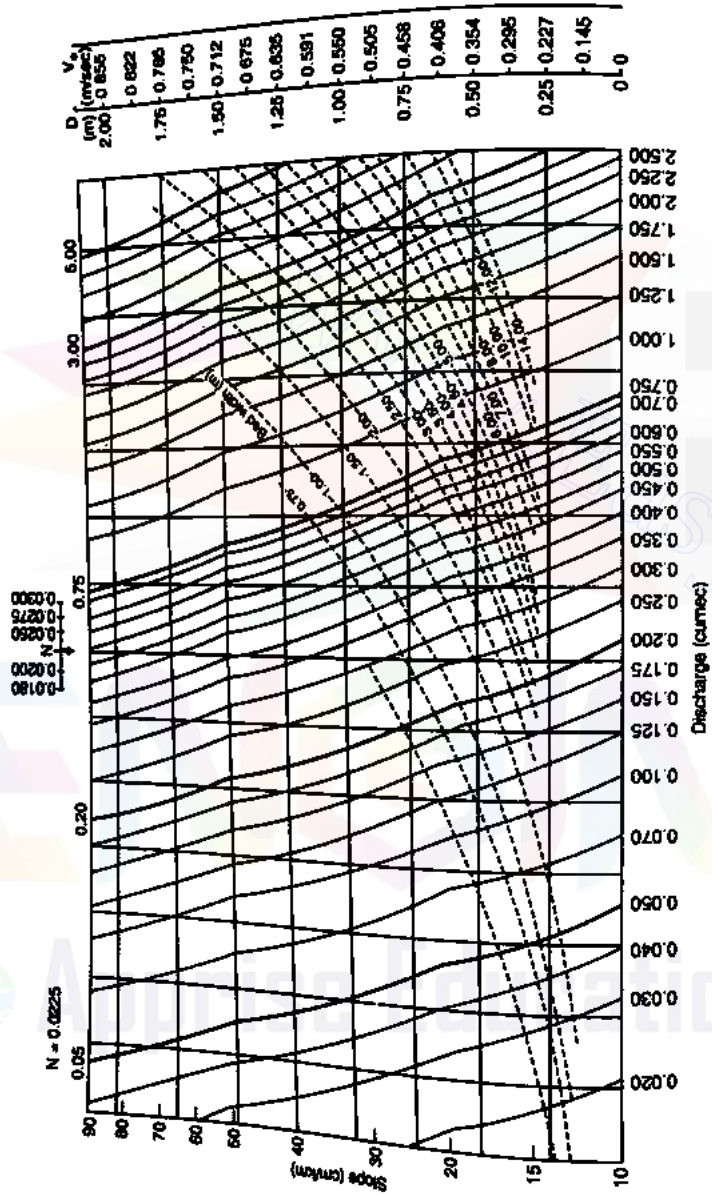


FIG. 15.3 (a) GARRETS DIAGRAMS FOR Q BETWEEN 0.02 CUMEC TO 5.0 CUMECs

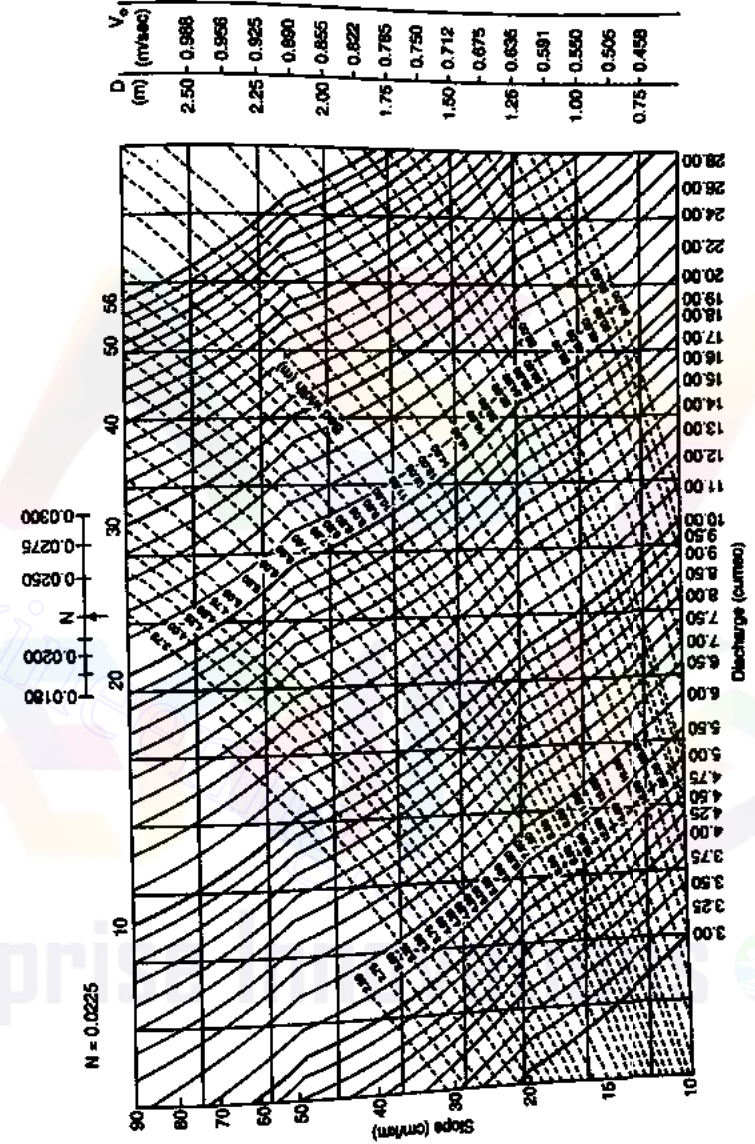


FIG. 15.3. (b) GARRETS DIAGRAMS FOR Q BETWEEN 3 CUMECs TO 56 CUMECs

4. Draw a vertical line through the point of intersection. This will intersect several bed width curves. Each point of intersection of vertical line and bed width curve gives a depth and critical velocity V_c corresponding to this depth on right hand side ordinate.
5. Choose a pair of bed width, depth and V_c corresponding to point of intersection obtained in step 4.
6. Calculate the area (A) of channel section corresponding to bed width-depth obtained in step 5.
7. Calculate the velocity in the channel corresponding to this area A .
8. Calculate the ratio V/V_c . This should be equal to unity or equal to the value of m given for channel design.
9. Repeat the procedure with other values of bed width and depth till the value of V/V_c obtained is the same as the value of m given for channel design.

To simplify the procedure, the result should be arranged in a tabular form as illustrated in Example 15.1. In practice, the design of channel is carried out by Garret's diagram. The diagram is applicable only for channel sections having a side slope of $\frac{1}{2}:1$.

The diagrams have been drawn for $N=0.0225$. However, a nomogram at the top of each diagram helps in making the use of these diagrams for any other value of N . For any other value of N , the vertical line drawn at the intersection of the discharge and slope lines has to be shifted either to the left or the right side to the extent given on the nomogram.

15.6. USE OF LACEY'S REGIME DIAGRAMS

These diagrams have been prepared by Lacey to provide a graphical solution for channel design. There are two separate diagrams—one is used for determining bed width (Fig. 15.4 a and b) and depth and the other to determine the regime slope (Fig. 15.5). The diagrams have been plotted for channel section having a side slopes of $\frac{1}{2}:1$.

Procedure of channel design with the help of these diagrams is as under :

1. To find the *bed width and depth*, use Fig. 15.4 (a) and (b). Find the intersection of discharge Q with silt factor f .
2. Read the value of bed width from abscissa and the value of depth from the ordinate.
3. To determine *regime slope* S , use Fig. 15.5. Find again the intersection and discharge Q with silt factor f .
4. Read the value of ordinate corresponding to this point of intersection and determine slope S .

15.7. CROSS-SECTION OF AN IRRIGATION CHANNEL

A canal is generally taken in such a way that its section is partly in cutting and partly in filling in order to approach close to the balancing depth. Many times,

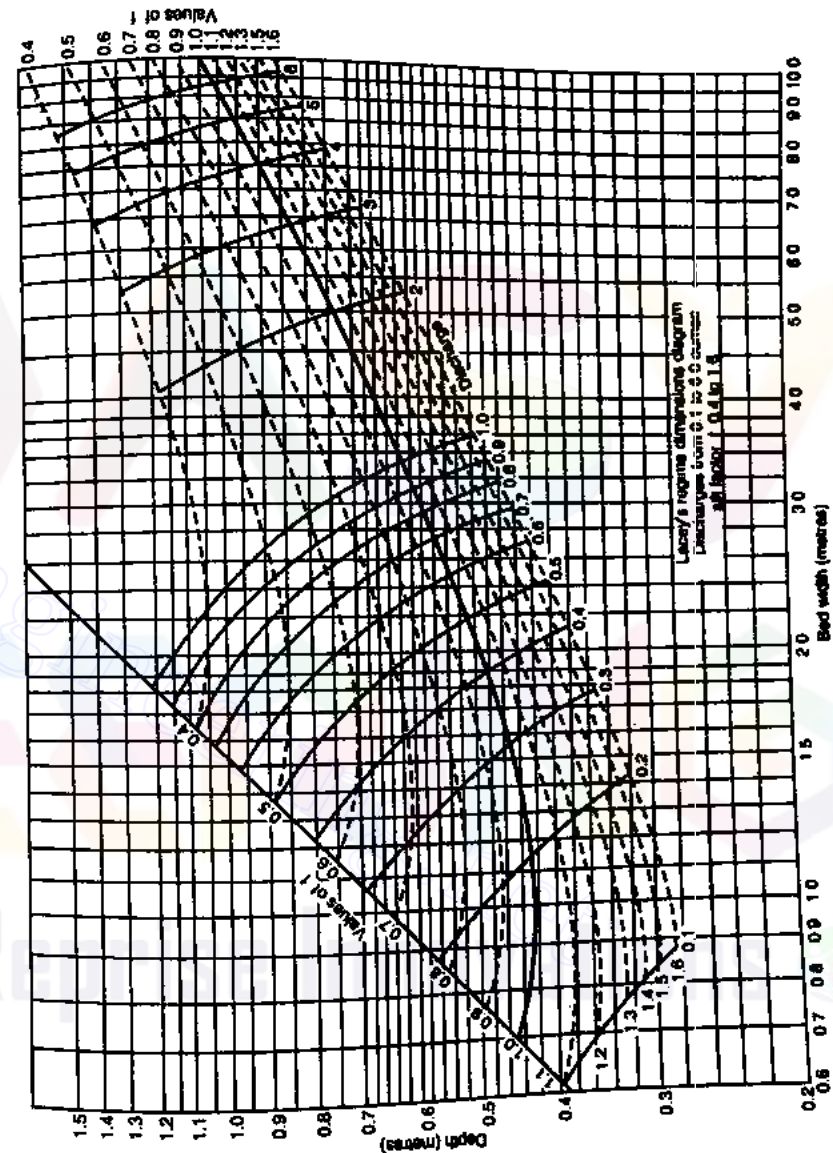


FIG. 15.4. (a) LACEY'S REGIME DIAGRAMS FOR B AND D (Q BETWEEN 0.1 TO 6 CUMECs)

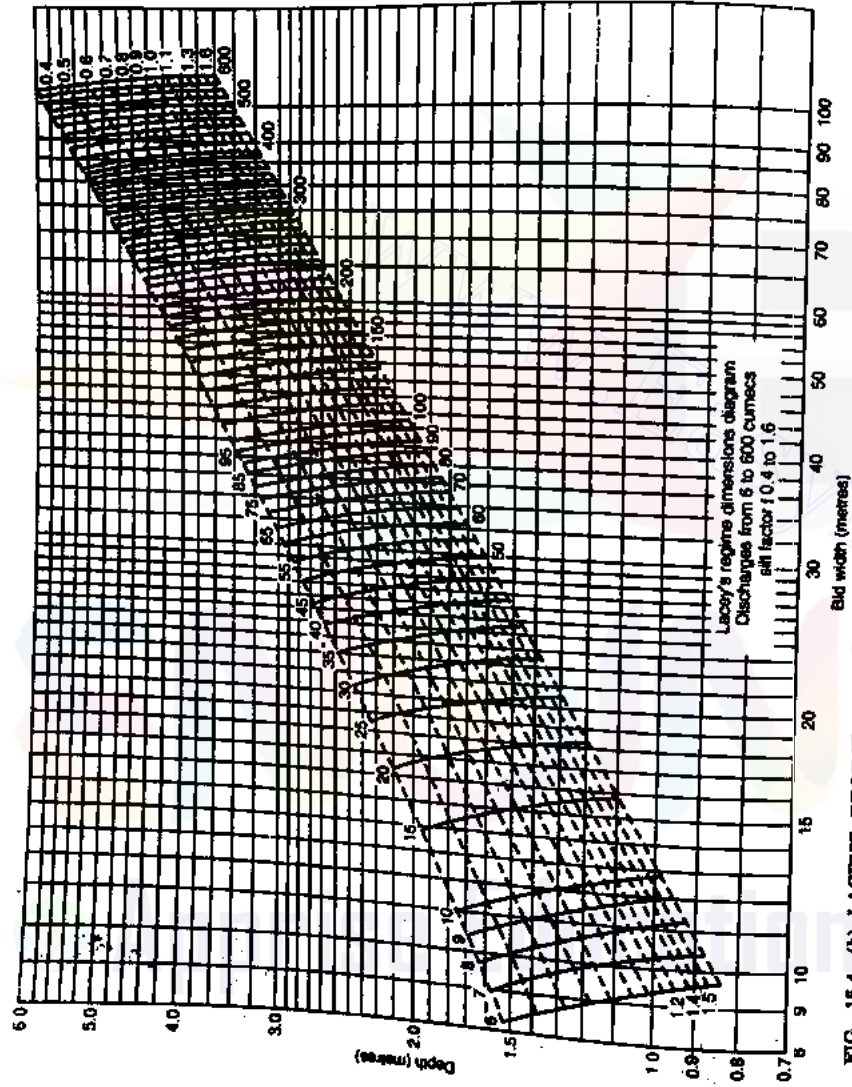


FIG. 15.4. (b) LACEY'S REGIME DIAGRAMS FOR B AND D (Q BETWEEN 6 TO 600 CUMEC/S)

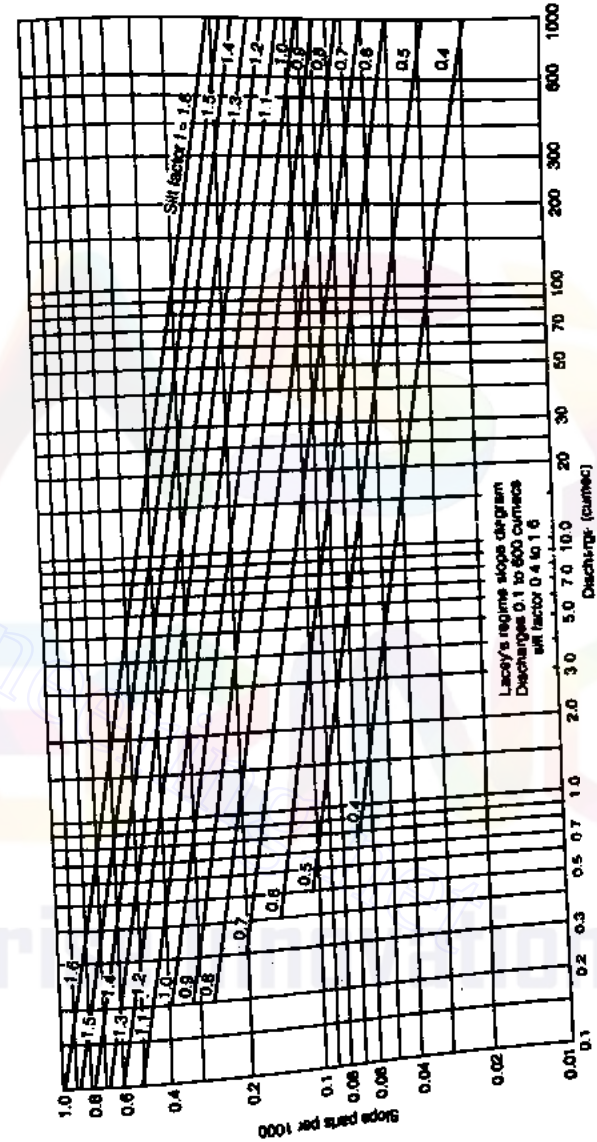


FIG. 15.5. LACEY'S REGIME DIAGRAMS FOR SLOPE

however, the canal has to be carried through deep cutting or filling. A channel section may, therefore, be either :

1. In cutting 2. In filling, or 3. In partial cutting and filling.

The channel section in these three conditions are shown in Fig. 15.6. When the ground level is above the top of the bank, the canal is said to be in cutting. Similarly, when the ground level is below the bed level of canal, it is said to be in filling. A canal is in partial cutting or filling when the ground level is in between bed level and top of bank.

A canal can have a dowla section on one side or a bank section on both sides. The section may also be designed to have a bank section on one side and dowla section on the other side. Usually the left bank of canal has a dowla section and service road and right bank of the canal has a bank section.

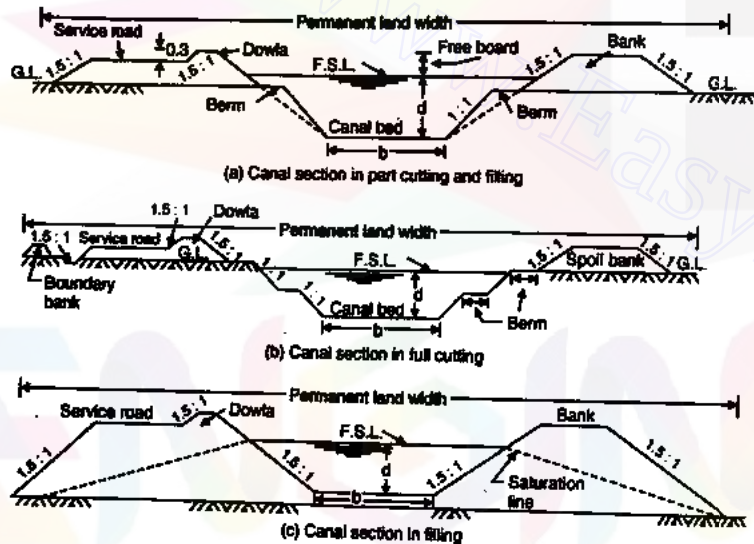


FIG. 15.6. TYPICAL CANAL SECTIONS.

Side Slopes

For the computation of the values of hydraulic mean depth R , the area and wetted perimeter are worked out with $\frac{1}{2}:1$ side slopes for design purpose even though in execution actually flatter slopes depending upon stability and type of soils are adopted. $\frac{1}{2}:1$ side slope is assumed with the presumption that the sides of the channel get silted up to $\frac{1}{2}:1$ slope and channel capacities would be reduced accordingly in due course. However, in case a channel takes off from such storage reservoirs where silt

free water is available, the actual slope to be provided during construction is assumed for design purposes. The side slopes depend upon the nature of the soil and the depth of cutting or filling. The minimum slope usually allowed is given in Table 15.3.

TABLE 15.3. SIDE SLOPES OF CHANNEL.

S.No.	Type of soil	Cuttings	Fillings
1	Hard Rock	$\frac{1}{8}:1$ to $\frac{1}{4}:1$	
2	Soft Rock	$\frac{1}{4}:1$ to $\frac{1}{2}:1$	
3	Hard clay or gravel	$\frac{3}{4}:1$ to $1:1$	$\frac{1}{2}:1$
4	Soft clay & alluvial soils	$1:1$	$\frac{1}{2}:1$ to $2:1$
5	Sandy loam	$\frac{1}{2}:1$	$2:1$
6	Light sand	$2:1$	$2:1$ to $3:1$

Berm : Berms are narrow strip of land left at the ground level between the inner toe of the bank and top edge of cutting. The width of the berm is variable but it is kept such that the bed line and bank line remain parallel.

Let $r_1:1$ be the slope in cutting and $r_2:1$ be the slope of the bank in filling. Then from Fig. 15.7, we have

$$AB = r_1 y + \text{initial berm width} + r_2 (d - y)$$

But $AB = r_2 d$, for bed line and bank line to be parallel.

Hence from the above two relations, we get

$$\text{Initial berm width} = (r_2 - r_1) y$$

This shows that the initial berm width depends only on the depth of cutting. Since the depth cutting varies with the variation of ground level, the initial berm width also varies.

After the channel runs for some time, the silt is deposited on the sides of the section and gradually the sides get silted to a slope of $\frac{1}{2}:1$. The position of the berm

therefore, shifts from ground level to the F.S.L. and its width becomes equal to $(r_2 - \frac{1}{2}) D$

where D is the full supply depth. Taking $r_2 = 1\frac{1}{2}$, the final berm width becomes equal to the depth D of the canal when the filling of bank has been done in $1\frac{1}{2}:1$ slope. The berms when fully formed serve the following purposes :

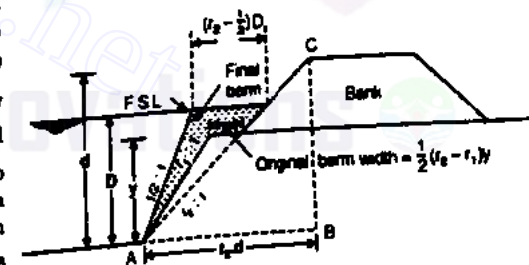


FIG. 15.7. INITIAL AND FINAL POSITION OF BERM

IRRIGATION AND WATER POWER ENGINEERING

1. The silt deposited on the sides is very compact and impervious. It serves therefore, as fairly impervious lining which reduce the absorption losses and prevents leaks and consequent breaches.
2. The possibility of leaks and breaches are reduced as they bring the saturation line more inside the body of the banks.
3. The berms provide a bigger water way ; its capacity, therefore, increases and it can thereby absorb positive fluctuations.
4. They protect the bank from erosion because of wave action.
5. Open borrow pits can be taken on the berms.
6. They sometimes provide an additional inspection path.
7. They provide additional strength to banks and thus make it safe against branches.

Minimum berm width may be adopted as under (Table 15.4) :

TABLE 15.4

Discharge	Berm width (metres)
Upto 4.25 cumecs	$0.6 + \frac{1}{2} \text{ F.S. depth}$
4.25 cumecs to 28 cumecs	$1.25 + \frac{1}{2} \text{ F.S. depth}$
28 cumecs and above	$1.25 + \frac{\text{F.S. discharge}}{28} + \frac{1}{2} \text{ F.S. depth}$

Minimum berm width = $0.5 + \frac{1}{4}$ (width of combined side slopes of cutting and filling)

Maximum berm width = $0.5 + \frac{1}{2}$ (width of combined side slopes of cutting and filling)

Recommendations of IS : 7112-1973 for berm width

Fig. 15.8 shows three typical sections of canals : (a) partly in cutting and partly in filling, (b) wholly in filling and (c) wholly in cutting as per IS : 7112-1973. Accordingly, the recommendations for berm widths are as under :

(a) When the canal is partly in cutting and partly in filling (Fig. 15.8 a), the berm may be kept at the ground level, with its width = $2D$ (i.e. twice the fully supply depth).

(b) When the canal is wholly in filling (Fig. 15.8 a) the berm may be kept at the full supply level with its width equal to $3D$.

(c) When the canal is wholly in cutting (Fig. 15.8 c), the berm may be kept at the full supply level with its width = $2D$.

Additional berm : Additional berm is provided when the 'channel' is in 'deep cutting'. Such a berm may be provided at the ground level, as shown in Fig. 15.8 (c). In such a circumstance, the width of the original berm, provided at full supply level may be reduced.

Free Board :

Free board is the gap or the margin of height between F.S.L. and top of the bank. It is possible to ensure that water does not overtop the bank. Free board in

IRRIGATION CHANNELS - II : DESIGN PROCEDURE

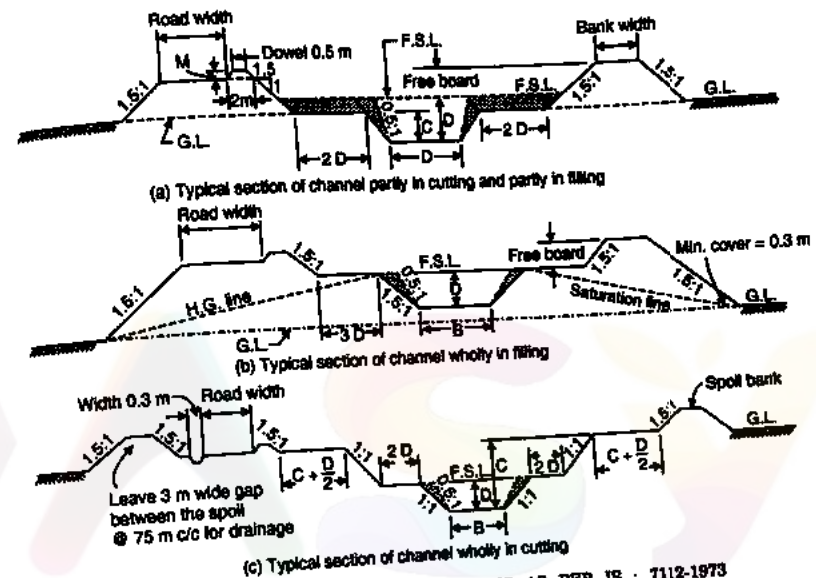


FIG. 15.8. TYPICAL CANAL SECTIONS AS PER IS : 7112-1973

a channel is governed by the consideration of size of canal, its location, water surface fluctuations, rain water inflow etc. The usual value of free board provided is given in Table 15.6.

U.S.B.R. recommendations for free board

$$(a) \text{ For } Q \leq 0.6 \text{ cumecs, F.B.} = \sqrt{0.46 D} \pm 0.68 \sqrt{D} \quad \dots(15.6 a)$$

$$(b) \text{ For } Q \geq 85 \text{ cumecs, F.B.} = \sqrt{0.76 D} \pm 0.87 \sqrt{D} \quad \dots(15.6 b)$$

(c) For $85 > Q > 0.6$ cumecs, the free board can be interpolated

Lacey's recommendations for free board

$$\text{F.B.} = 0.20 + 0.15 Q^{1/4} \quad \dots(15.7)$$

where Q is the discharge in cumecs.

Top width of Bank :

The purpose of the bank is to retain water so long as the berm is not formed. They thus have to withstand full pressure of water above the ground level. They should, therefore, be sufficiently strong to withstand this initial strain. No special design is, however, done and it is customary to provide width of bank as per Table 15.5. If the canal is completely in filling the adequacy of the bank section and stability of its slope should be tested as per the methods suggested in Chapter 10.

Recommendations of IS : 7112-1973 for minimum top width of banks

Table 15.5 gives the Indian Standard recommendations for minimum top width of banks of canals.

IRRIGATION AND WATER POWER ENGINEERING

TABLE 15.5. MINIMUM TOP WIDTH OF BANKS (IS : 7112 - 1973)

S.No.	Discharge (cumec)	Minimum top width (m)	
		Inspection Bank	Non-Inspection Bank
1	0.15 to 7.5	5.0	1.5
2	7.5 to 10.0	5.0	2.5
3	10.0 to 15.0	6.0	2.5
4	15.0 to 30.0	7.0	3.5
5	30.0 and above	8.0	5.0

Sometimes, the following empirical formula is used for top width :

$$\text{Top width} = D + 0.3 r^{1/2} \quad \text{where } r = B/D \text{ ratio.} \quad \dots(15.8)$$

It is customary to provide a minimum top width of 1.5 m.

Saturation line or hydraulic gradient line

When water flows through an unlined earth channel, seepage takes place through the banks. Due to this seepage, the banks get saturated in the lower portion while the upper portion remains either dry or moist. The line demarcating these two portions is called the *saturation line* or the *hydraulic gradient line* or *phreatic line*. The saturation line is actually parabolic in shape, but for banks of lesser height (less than 5 m), it is usually taken as a *straight line* (Fig. 15.8 b) starting from the point where the full supply line meets the inner sloping face of the bank. The slope of the saturation line is known as *saturation gradient* or *hydraulic gradient*, and its value depends upon the type of the soil of the banks. Indian Standard (IS : 7112-1973) recommends the following empirical values of the hydraulic gradient for banks less than 5 m in height.

Type of soil	Hydraulic gradient (H/V)
1. Silty soils	4:1
2. Silty sand	5:1
3. Sandy soils	6:1

However, for banks more than 5 m in height, the true position of the saturation line (or phreatic line) shall be determined and the stability of its slope should be checked. In all the cases, however, the saturation line shall have a minimum cover of 0.3 m.

If, however, the saturation line cuts the outer face of the bank, or if sufficient cover is not available, back berm or counter berm should be provided (as indicated in § 15.11) or the top width of the banks should be increased.

Dowel or Dowla :

A dowla is provided by the side of inspection road as shown in Fig. 15.9. Top of Dowla is kept above the F.S.L. by a margin of free board. They are provided as

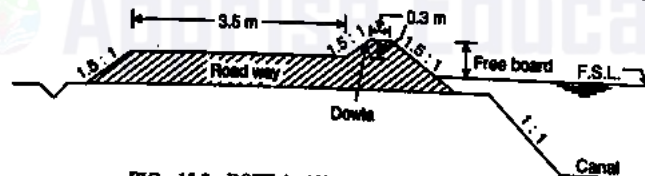


FIG. 15.9. DOWLA AND INSPECTION ROAD.

a measure of safety for automobiles driven on the service road. They act as kerbs on the side of roadway towards the canal. Top width of dowla is kept from 0.3 m to 0.6 m and is about 0.3 m above the service road.

Inspection Roadway :

It is very necessary to have access to all parts of canal system so that proper inspection may be done. It will also increase the efficiency of the maintenance. Main canals and branch canals have roadway on both sides. In case the road can be provided only on one side, it is customary to provide it on the left bank of canal. The banks of the canal are classified as left and right by facing the canal along the direction of flow. Usually the road on right serves the purpose of transport of the material. On smaller branches and distributaries the roadway is provided on left side only and it is meant for inspection. Road is not provided along minors. The canal road is usually unmetalled since they are used for limited purpose only. Since the canal divides the area in two parts, sometimes a cart road is provided on both sides. The cart road may be kept 3.0 to 5.0 m wide. Many a times the canal road provides the only access to the interiors. It is also sometimes required to let the inspection road be a thorough fare. The state P.W.D.(B. and R.) may also use it. In such a case the road surface must be metalled before leaving it open to public.

In case of dowla section, the service road is adjoining to the dowla. The road way is provided 0.3 to 0.8 m above F.S.L. depending upon the size of the channel. In case of bank section, the road way is provided just a few cm above the ground level. It becomes very difficult to inspect a canal when the roadways is in this position because the bank section is on the right and one has to get down at many a places in order to inspect the canal.

Standards of canal cross-section

Table 15.6 gives the commonly followed standards of canal cross-section.

TABLE 15.6. STANDARDS OF CANAL CROSS-SECTION

S.No.	Item	Main and Branch canals		Major distributaries		Minor distributaries	
		150 to 30 cumecs	30 to 10 cumecs	10 to 5 cumecs	5 to 1 cumecs	1 to 0.3 cumecs	Below 0.3 cumecs
1.	Maximum crest width of bank	3 m	2.5 m	2.25 m	2 m	1.5 m	1.0 m
2.	Free board	0.9 m	0.75 m	0.6 m	0.5 m	0.4 m	0.3 m
3.	Width of berms	Minimum berm width = $0.6m + 1/4th$ of the width of combined side slopes of cutting and embankment Maximum berm width = $0.6m + 1/2th$ width of combined slopes					
4.	Width of road way	6 m	5 m	5 m	3.5 m	3.5 m	Nil
5.	Depth of earth cover over saturation gradient	1 m	0.8 m	0.5 m	0.5 m	0.5 m	0.5 m
6.	Width of land to be acquired : (Clear of banks) when canal cutting is deeper than balancing depth	5 m	Half the height of the bank above ground subject to a minimum of 1.5 m				
7.	Width of land to be acquired : Clear of banks when canal cutting depth is less than balancing depth	Full height of bank plus 5 m	Full height of bank plus 5 m	Full height of bank above ground level plus 1.5 m			

15.8. BORROW PIT

When the earth work in filling exceeds excavated quantity at a particular section, borrow pits are required to make good the requirements of filling. *Outside borrow pits* are generally not preferred due to mosquito nuisance. *Inside borrow pits* are usually advocated as they get silted up during the course of time. The outside borrow pit if used should not be dug deeper than 0.3 m, so that the land may be reclaimed after the completion of the job. The outside borrow pit should start from a distance not less than 5 m from the toe of embankment in case of small channels and 10 m from the toe in case of large channels. Inside borrow pit should have a width not more than half the width of the canal and should be dug in the centre. Their depth should not be more than 1 m. Inside borrow pit should not be continuous and a portion not less than half the length of the borrow pit should be left between successive borrow pits. The arrangement of internal borrow pit is shown in Fig. 15.10.

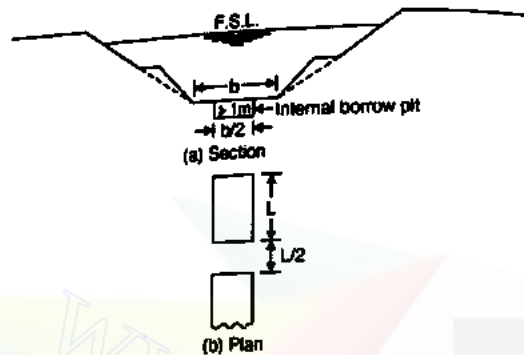


FIG. 15.10. ARRANGEMENT OF INTERNAL BORROW PITS.

15.9. SPOIL BANK

When the quantity of excavated earth is much in excess of the quantity required for filling it has to be deposited in the form of spoil banks. If the quantity of the extra earth available is small, it is used to widen the service road, but the service road can only be widened to a certain limit beyond which its maintenance may be difficult. The height of the spoil bank is not kept large, but on costly land the height of spoil bank is increased to involve minimum width of land. A longitudinal drain is always made between the spoil bank and service road. Cross drains should also be provided at about 100 m interval and the spoil banks should be discontinuous at these points.

15.10. PERMANENT LAND WIDTH

Width of land required for construction of canal cross-section is known as land width. *Permanent land width* is the distance between the outer toes of the canal bank plus a few metres on both sides for the construction of a drain or line of trees etc. It is the land width which is permanently required before starting the construction of canal.

During construction stage of the canal, some extra land width is required for staking of materials and machinery and for borrow pits. This is known as *temporary land width* and is acquired for temporary use. The land is returned to its owner after completion of the job. A nominal compensation has to be paid to the owner for temporary acquisition.

15.11. BACK BERM, COUNTER BERM

Even after providing the usual embankment section of a bank, the saturation gradient may cut the downstream side of the bank. In such a case the saturation line should always be covered by 0.3 to 0.6 m of earth. This is best done by the help of counter berm as shown in Fig. 15.11.

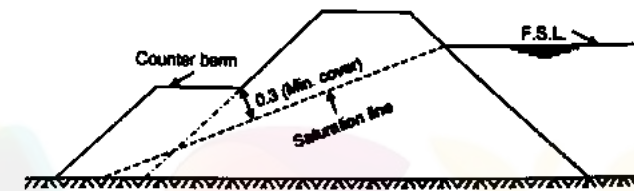


FIG. 15.11. COUNTER BERM.

15.12. MAINTENANCE OF IRRIGATION CHANNELS

After the construction of irrigation system in an area is completed, it becomes essential to maintain it for its proper and efficient functioning. There are various reasons due to which a canal may cease to function efficiently. These are :

(i) Silting of canal	(ii) Breaching of canal due to weak banks
(iii) Weed growth	(iv) Overflow of canal banks

1. Silt Removal :

When the silt is deposited on the bed and sides, the capacity of the canal reduces. It is better to exclude silt by providing silt excluder and ejector as explained in Chapter 12. Curved wing-cum silt vanes and silt tunnels-cum-curved wings can be fitted near the head of an off-taking canal for exclusion of silt. However, none of these methods can put a complete check over the entry of silt in the channel and hence silt is deposited in the channel bed even after the best care.

The following measures are adopted to remove the silt :

(a) *Flushing*. Flushing of the canal with clear water will lift up deposited silt. Absolutely clear water should be used for flushing but if this is not available then the water which contains minimum quantity of silt should be used for flushing. Flushing should generally be over done to cause some scour. This will create room for further silting and will thus reduce the frequency of flushing.

(b) *Silt scouring fleet*. The method consists of having three lower barges connected to the upper barge by a cable operated by winch. The lower barges have movable shutters. The silt is kept agitated by manoeuvring the barge up and down. The method was used in Punjab but was unsuccessful.

(c) *Bundle of thorny bushes* tied together and pressed down by weight of stones are pulled inside the channel by animals. They are quite useful in dislodging the fine muddy silt.

(d) *Iron rakes* are also dragged in the channel to dislodge silt.

(e) *Reduction of area of flow.* Loaded boats are put across the section to reduce the area of flow and increase the velocity of flow.

(f) *Stirring of silt by water jet.* A pump fitted with a pipe and nozzle is placed on a barge. The high velocity jet is directed to the bed to stir the silt and prevent silting.

(g) *Dredging.* A dredger is very rarely employed for removing silt from canal as it is very costly method.

(h) *Excavation.* The silt deposited in a channel is cleared off by manual labour. The method is quite costly as it requires recurring expenditure. This method is generally adopted for silt clearance in distributaries and minors. The silt must be deposited clear off the channel so that it does not find its way back to the channel.

2. Strengthening of Canal Banks

To prevent breaching of the canal bank, it should be strengthened properly so that valuable loss of irrigation and property is prevented due to breaching a canal section. There are four methods of strengthening a canal bank :

- (a) External silting system.
- (b) Internal silting system.
- (c) Formation of berm by internal silting.
- (d) Formation of back berm.

(a) *External silting system.* In this method subsidiary banks are constructed which run parallel to the main banks. Cross bunds are constructed within the subsidiary bank and main bank at a distance of 150 to 1500 m to form compartment of silting or *silting tanks*. Water is allowed to get into the compartment from upstream side and is held there for some time before discharging it back to the canal from outlet end. When the cross bunds are separated at a distance of 150 to 300 m, the capacity of the bank is small and only a portion of full supply discharge is taken into the compartment. It is then known as *into out system*. But when the length of the compartment is large say 1200 to 1500 m, full discharge of the canal can be taken inside the compartment and it is then known as *long reach system*. The method is practised only when no water is required for irrigation downstream of the reach.

(b) *Internal silting system.* In this system the canal banks are set back away from their original positions. The section of the canal provided is large than required and, therefore, its velocity is low. The section, therefore, gets silted up very quickly. To induce silting and accelerate the process, low submersible spurs are constructed. The silted berm is shown in Fig. 15.12.

Hanging groyne or suspended groyne is also very useful for inducing silting.

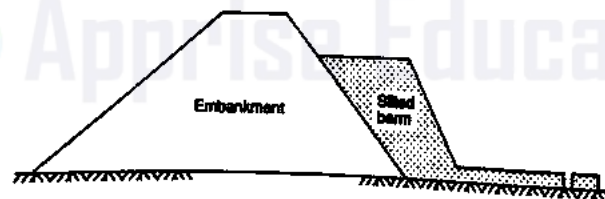


FIG. 15.12. SILTED BERM.

(c) *Formation of berms by internal silting.* Banks can also be strengthened by formation of berm by internal silting. The silting of the berm can be accelerated by constructing permeable spurs from the side of the channel section.

(d) *Formation of back berm.* A back berm may be formed if the saturation line crosses the downstream slope of the bank as explained in § 15.11.

3. Weed control

Water weeds are unwanted plants that grow profusely in water under certain favourable conditions. They tend to reduce the discharging capacity of channel by reducing the area of the channel section and velocity of flow. The problem of weed growth is more marked in Deccan where the heavy weed growth may reduce the channel discharge to even less than 15%. The nuisance has, therefore, to be checked to permit the channel to function efficiently.

There are a variety of weeds growing on canal bed, water surface and water marks. They tend to thrive better in a range of 20° to 30° C. Weed growth is not possible in channels having high velocity of flow but when the channel has velocity less than 0.6 m per second weed growth is generally possible. The deposition of silt has no direct effect on weed growth, yet profuse weed growth is known to take place where silt is deposited. Light has a considerable effect on weed growth. Weed growth is accelerated in presence of light.

The *weed growth* can be checked by passing higher velocity than regime velocity in the channel. This will keep the silt in suspension and will make water turbid. Thus the light rays are cut off and silt is not deposited on channel bed.

Yet another way of weed control is *rush rotation*. In the process of rush rotation, the channel is run with full supply discharge for some time and then it is left completely dry for some time. This helps in excluding more light when higher depth is flowing, thus reducing the weed growth. During closure weed is unable to resist scorching rays of sun. Long duration closure has killing effect on the weed growth.

Weed removal may done by plucking them by hand and burning them when canal is dry.

15.13. MAINTENANCE OF SERVICE ROADS

The canal service road is unmetalled and, therefore, in monsoon season grass and small bushes grow on the road surface. The surface of road also wears and tears off due to some traffic over it. Sometimes unauthorised traffic also pass over canal roads which make the conditions still bad. The maintenance of service roads, therefore, consists of :

- (i) Removal of grass and small bushes.
- (ii) Levelling of road surface.
- (iii) Ramming and watering of top wearing surface.

Maintenance is usually carried out after the monsoon season. To check unauthorised traffic of cart, sometimes a check barrier is constructed at every crossing. The check barrier consists of a small earth mound with a slope of 1:4 on upstream and downstream side with some top width. Jeeps can easily be crossed over these mounds but for trucks and bullock-carts it becomes a real barrier.

15.14. CANAL BREACHES

Canal breach or the tearing of the canal bank takes place when canal is in filling. During a canal breach, a big gap forms on the side and the large canal discharge rushes out doing unlimited damage.

Reasons for canal breaching.

1. *Breaching due to defective design and construction of the bank.* The canal bank is not strong enough to resist water pressure and a breach occurs.
2. *Breaching due to exposure of saturation gradient.* The soil particles on the outer slope are dislodged and flown away along with the seeping water. A cavity is thus formed in the bank which ultimately develops into a breach.
3. *Breaching due to rush of water through rat holes or "shories".*
4. *Piping* near downstream toe may ultimately cause subsidence of canal banks.
5. *A willful cut* in the canal bank either for unauthorised irrigation or for allowing the flood water to get into canal to save an area from submergence.

On Lower Ganga Canal near Nadrai Aqueduct some *owlposts* are provided. It is believed that owls eat away rats and hence prevent the development of rat holes.

Repair of Breaches

A breach can be repaired by the procedure described below :

1. Immediately after a breach occurs, the canal discharge should be diverted to a nearby escape channel on the upstream side or the canal should be closed from the head to stop the outflow completely. Many a times it is not possible to close the canal from head as the irrigation of the area in between may suffer.
2. Provide steps in the sides of the gap so that the earth to be deposited for closing the gap forms a good bond with old surface.
3. Derive two lines of ballpiles in the gap in the inner side of the bank and fill the space in between by planks and bushes and with sand bages on top.
4. Step 3 should cut down the outflow considerably and velocities will reduce. If a second line of defence is needed, the same may be provided by two rows of sand bage with earth in between behind the pile line.
5. Collect a sufficient amount of earth on both sides of the gap. The earth collected should either be from spoil bank nearby or from borrow pits.
6. The opening in between the pile line and the gap is then filled with earth in layers, which should be properly compacted at optimum moisture content and the new bank is prepared.

The bank so formed is inspected carefully for few days to watch its proper functioning.

15.15. REGULATION OF CANAL SYSTEM

The process of distribution of water is called *regulation*. The principle underlying the distribution of water is apportioning available canal supplies on the basis of irrigable

area. If the water is needed only occasionally then it may be supplied according to the general agreement between cultivators and canal administration.

A canal has to be run with full supplies if there is a keen demand in the commanded area, with part supplies if a part of commanded area only needs water and with no supplies if the demand falls below a certain minimum. The closure of a main canal is limited, as the area covered is large and there is always some demand in some parts of its commanded area though the channel has to be run with low supplies frequently. When the demand is only on a part of the system the distributaries which need water are kept open and the rest are closed. The difficulty in the distribution arises when there is a keen demand in the entire system but the available supplies from the river is unable to meet this demand. A straight forward but less effective solution should be to run all the distributaries with reduced supplies. Another alternative, a more effective one is, to run the distributaries with full supplies in rotation. This system is known as *Osrabandi* in U.P. and *Warabandi* in Punjab. Amongst the various disadvantages of the former method, the important ones are (i) silting of the channels, (ii) increased weed growth, (iii) increased seepage and water logging, (iv) low head at outlets. The advantage of the latter system is that the full closure of smaller channels may facilitate silt clearance, inspection and repair of masonry works.

A roaster is generally prepared which details the supplies allotted to different channels and names of different channels which will be closed down from week to week. The roaster is usually adhered to during season. Sometimes the demand in the area is very fluctuating, when a flexible regulation from week to week is adopted in accordance with the anticipated demand in the ensuing week. The anticipated demand is furnished to the engineer incharge through telegram sent by the revenue staff for their respective areas.

In order to strictly adhere to the regulation, it is imperative to know the discharge flowing in channel by plotting stage discharge curves as explained in 15.22. The procedure of measurement of discharge is therefore, described in subsequent articles.

15.16. MANAGEMENT OF IRRIGATION WATER

With increased need of water for irrigation it has become absolutely necessary to avoid wastage of irrigation water and use it as efficiently as possible. Government of India has now created 32 Command Area Development Authorities (CADA) in 12 different states so that irrigation potential created can be properly utilized. Various steps to check the wastage of water are discussed below :

- (i) *Land development*. This includes land consideration, proper layout, shaping and grading of land and proper cultivation of land at the appropriate times.
- (ii) *Field application of water*. An appropriate method of application of water has to be selected. CADA is now helping farmers to manage the field ditches with regulating structure beyond the outlets for proper supply of water to farm land.
- (iii) *Irrigation by rotation*. This method should be practiced when available supplies are lower than the demand.
- (iv) *Method of charging*. Volumetric basis of charging is extremely helpful in preventing wastage. However, if its implementation pose a problem, charging on

a crop season-area basis is a proper alternative. It requires careful planning and formation of committees for different outlets so that farmers at head and tail can get together and decide about allocation.

(v) *Improved agronomic practices.* Scientific principles in finding NIR and FIR, proper choice of high yielding varieties of crops and crop rotation help in more efficient use of water.

(vi) *Educating farmers* to develop a scientific and technical temperament and seeking their active participation in construction, maintenance and operation of field channels helps in better use of irrigation water.

15.17. MEASUREMENT OF DISCHARGE OF A CANAL

It is necessary to know the discharge flowing in a channel section. The efficient use of irrigation water largely depends upon the measurement of water. The measurement of discharge may give an idea of the amount of silting and canal losses.

Principal methods commonly used for measurement of discharge are mentioned below :

1. Area velocity method.
2. Chemical method.
3. Weir method.
4. Meter flume method.
5. Stage discharge curve method.

15.18. AREA VELOCITY METHOD.

Area-velocity method is a more general method and is applicable for rivers and canals both. This method consists of measuring area of the channel section and the mean velocity of flow through it. The product of the area with mean velocity yields the discharge.

Measurement of velocity : The main problem in measuring the discharge is the accurate determination of velocity. The velocity distribution in an open channel flow is shown in Fig. 15.13. The velocity is minimum at centre and decreases towards sides. For measuring the velocity in the channel section, a straight and well defined reach of the channel section should be selected. The site should also be free from the disturbing influence of hydraulic structures. A structure will be affected by back water curve whereas the discharge site located on the down-stream of the structure will be affected by the falling curve. The cross-sectional area of the channel should be fairly constant.

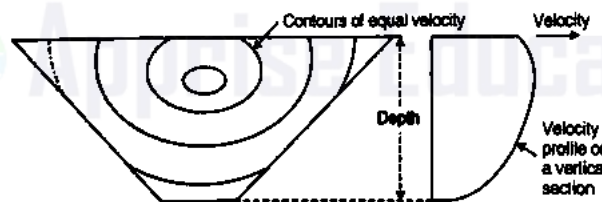


FIG. 15.13. VELOCITY DISTRIBUTION IN CHANNEL SECTION.

The velocity is measured by the following methods :

1. Surface floats.
2. Subsurface floats.
3. Twin floats.
4. Velocity rod or floats.
5. Current meter.
6. Pitot tube.

Surface float method : Surface floats are made of light material such as cork or hollow wooden blocks so that they may float on the surface. They are painted on top and sometimes attached with a flag to easily distinguish it from any other floating object.

Two wire ropes are stretched across the channel section one at the beginning and the other at the end of selected reach for conducting the test. This distance is called *test reach*. A third rope is stretched upstream of the test reach by 15 to 20 m. All the three ropes are marked at various points so as to divide the channel width in various compartments (Fig. 15.16) the centre line of which defines the part or track for floats. The floats are dropped in water along various paths near the third rope. They acquire uniform velocity by the time they enter the test reach. The time of travel of the test reach of only those floats which follow their correct track is recorded.

$$\text{Then the velocity of float} = \frac{\text{Distance travelled by float}}{\text{Time taken}}$$

For large channels the length of test reach should be 50 m to 75 m. For lined and small channels this length should be from 15 to 20 m. Wind affects the velocity of float. Observation, therefore, should not be taken where there is strong breeze.

The method of observing the velocity is the same with other types of floats also.

Sub-Surface floats or double floats: The surface float only gives the surface velocity. It is usually required to find the mean velocity of channel section. Sub-surface float finds the mean velocity across a vertical section.

The sub-surface float (Fig. 15.14) consists of hollow metal sphere or cylinder attached to the surface float by means of a chord. The length of chord is adjustable so as to reduce or increase the depth of the sub-surface float. Mean velocity is obtained by adjusting the length of the chord such that the centre of sphere is about 0.2 d above the bed level.

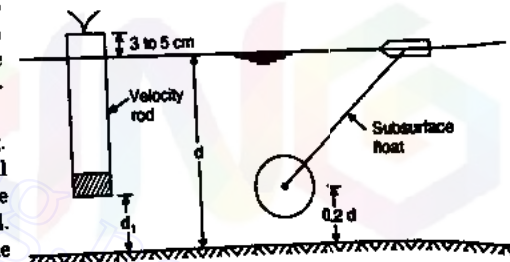


FIG. 15.14. SUB-SURFACE FLOAT OR ROD.

The centre of sphere is about 0.2 d above the bed level.

Twin float

Twin float consists of two hollow metal spheres attached with a chord. Upper sphere floats on the surface and the lower sphere is so weighed as to keep the float vertical. Mean velocity is obtained if the lower float is just clear off the bottom.

Velocity rod or rod floats: They are made of hollow tube or wooden rod. They are usually made telescopic with one tube sliding into the other. They are weighed with lead shots in such a way that when the rods are let afloat, they stand vertical. Their length is adjusted in such a way that the bottom of float may be clear off the bottom by a few centimetres. The mean velocity is calculated by the formula

$$V_m = V \left[1.02 - 0.116 \sqrt{\frac{d_1}{d}} \right] \quad \dots(15.9)$$

where V = the observed velocity

d = depth of flow in the channel

d_1 = clearance between lower-end of rod and bed.

Current meter: A current meter is a device which measures the velocity in the channel directly. The instrument consists of a wheel which is caused to revolve by the force of current when immersed in water and is provided with a gear to record the number of revolutions of the wheel. According to the revolving parts used the current meter may be classified into :

(1) Conical cup type current meters such as Anemometer and Price current meter.

(2) Helicoidal vanes type current-meter such as Haskell current meter and Amsler current-meter.

Current meters with conical cups are more popular.

Rating of current. The current meter is placed in an artificial channel of known velocity. The instrument is suspended in the channel to a suitable depth. The number of revolutions recorded by the meter corresponding to the velocities is determined. A graph is plotted with velocity per second as ordinate and number of revolutions per second as abscissa. This is known as *rating curve*. Velocity in the channel is directly proportional to the number of revolutions and hence rating curve is a straight line.

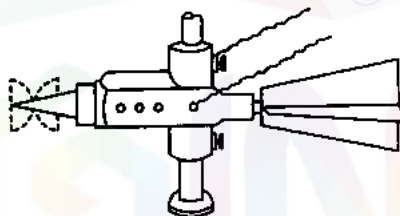


FIG. 15.15. CURRENT METER (AMSLER TYPE)

Measurement of Velocity: On channels of not too large a depth, the instrument is usually mounted on a rod on which it can be fixed in any position. The rod is made to rest on the bed of the channel, where the velocity is to be measured. Whereas on large channels, the current meter is suspended from a boat by a cable and is weighed down by a heavy stream-lined anchor.

Current meter is used in the following ways :

(i) **Single point method.** The current meter is immersed to 0.6 of the depth of the stream. At this depth velocity recorded is the mean velocity. The channel section is divided into various compartments and the velocity and the centre of each compartment is noted. Single point method is suitable for streams having a larger depth where

the meter cannot be suspended at $0.8 d$. The method gives error of 5% to the mean value.

(ii) **Double point method.** In this method the meter is introduced at $0.2 d$ and $0.6 d$ of the stream. Average of the two velocities gives the mean velocity in the section. Maximum error in this method is about 2.5%. This is suitable for ordinary streams.

(iii) **Integration method.** The meter is introduced in the channel at various depths such as $0.1 d$, $0.2 d$, $0.3 d$, $0.4 d$ and so on. The velocities recorded at these depths are then averaged to find the mean velocity. Integration method gives accurate results.

Sometimes when the velocity in the channel section is high, it may not be possible to introduce the meter at $0.8 d$ or $0.6 d$. In that case it is introduced at $0.2 d$ and the surface velocity is found out. The surface velocity is multiplied by an appropriate coefficient to find the mean velocity.

Pitot tube. The method is very suitable for laboratory though not so suitable for the field as the entrance of the tube may get clogged by fine silt. The pitot tube is suspended in the stream with its nozzle pointing upstream. Due to dynamic pressure exerted on the mouth of the tube by flowing water, the level of water in the vertical limb of the tube rises above the water surface of the channel by an amount h . The velocity in the channel is then equal to $\sqrt{2gh}$.

Measurement of area of flow. The cross-sectional area is generally divided into a number of small vertical strips. For measuring the width of the strip, a steel cable is suspended from two towers erected on either side of the bank. From the cables are hung ropes or chains to touch surface of water at regular intervals and divide the surface of water into suitable strips of known width. Depth of water at each point is measured by a sounding rod. The width and depth of each strip being known, cross-sectional area of various strips can be calculated.

Thus if A_1, A_2, A_3 , etc. are the areas of the strips and V_1, V_2, V_3 , etc. the corresponding mean velocities, then the channel discharge is given by

$$Q = A_1 V_1 + A_2 V_2 + A_3 V_3 \dots = \Sigma AV \quad \dots(15.10)$$

15.10. CHEMICAL METHOD

A solution of known strength and weight of some chemical, preferably salt, is injected uniformly over the cross-section. It is then allowed to mix thoroughly as it travels downstream with water such that stream has uniform strength. From some

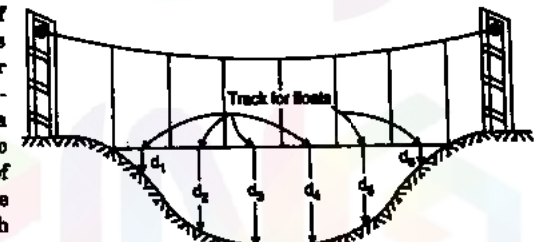


FIG. 15.16. MEASUREMENT OF AREA.

where on the downstream side a sample of water is collected. The concentration of salt of the sample is determined by titrating the sample with silver nitrate and potassium chromate. Since the water may contain salt originally, the concentration of salt in undosed chromate. Since the water may contain salt originally, the concentration of salt in the sample, the volume of water Q flowing per second can be calculated by:

$$Q = \frac{w_1 - w_0}{w_2 - w_0} q \quad \dots(15.11)$$

where w_0 is the weight of salt/ m^3 in undosed water

w_1 is the weight of salt/ m^3 of injected salt solution.

w_2 is the weight of salt/ m^3 of dosed water obtained from the sample

q = quantity of solution injected in m^3/sec .

15.20. WEIR METHOD

The water is allowed to flow over a broad crested or a sharp crested weir and the head over the weir is measured to find the discharge. A rectangular or triangular notch can also be fitted and the head over the notch can be related to the discharge flowing over it.

General equation for the flow over a broad or sharp crested weir is :

$$Q = \frac{2}{3} C_d \sqrt{2g} (L - 0.2 H_1) (H_1^{3/2} - h_a^{3/2}) \quad \dots(15.12)$$

where $H_1 = H + h_a$

h_a = head due to velocity of approach = $V^2/2g$

H = Head of water over the crest.

Value of C_d varies in accordance with the shape of the crest.

For Broad Crested Weirs $C_d = 0.557$.

Narrow Crested Weirs $C_d = 0.623$.

For clear overfalls $C_d = 0.868$.

When the discharge is measured over a V-notch

$$Q = \frac{8}{15} C_d \sqrt{2g} H^{5/2} \tan \theta/2 \quad \dots(15.13)$$

where θ is the angle of Notch.

15.21. METER FLUME

Discharge measurement by a meter flume in open channel is similar to the discharge measurement by a Venturimeter in a pipe flow. The channel width is restricted to form a throat with smooth entry and exit transitions on both sides.

The depth of water at the throat and at the upstream normal section is recorded by means of two gauge walls provided as shown in Fig.15.17. The throat has a higher discharge intensity and a higher velocity compared to normal section. The depth at the throat, therefore, is lesser than that at normal section. The difference in water level indicates the pressure loss. Knowing the depth of water, the area, a_1 at the

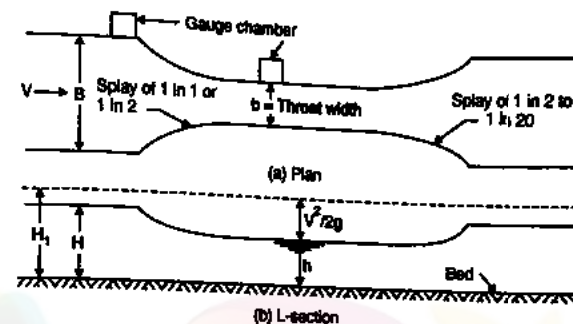


FIG. 15.17. METER FLUME.

normal section and a_2 at the throat section can be calculated. If H is the depth at the normal section and h is depth at throat the discharge Q is given by the formula:

$$Q = \frac{C a_1 a_2 \sqrt{2g} (H - h)}{\sqrt{a_1^2 - a_2^2}} \quad \dots(15.14)$$

where C is the coefficient of flume to be determined experimentally.

15.22. STAGE DISCHARGE CURVE

Stage discharge curve is a curve between discharge and depth of water in the channel. Stage or depth is plotted as ordinate and discharge is plotted as abscissa. The curve may also be plotted on logarithmic scale ; then the curve may be in the form of a straight line. In order to plot stage discharge curve statistical data is collected initially for discharge at various stages of flow. This range of discharge data collected should cover the usual range between minimum and maximum flow in a river. A stage discharge curve is shown in Fig. 15.18.

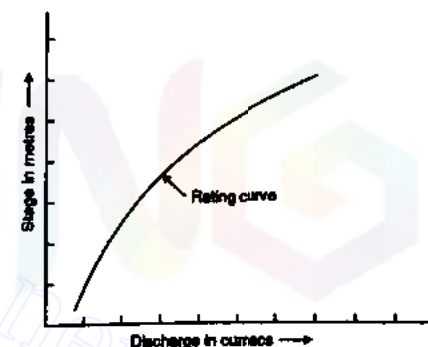


FIG. 15.18. STAGE DISCHARGE CURVE.

Once a stage discharge curve is plotted, it can be used for finding out the discharge at any subsequent period by measuring the stage. The stage discharge curve must be checked periodically as the regime conditions of the channel are always changing.

Example 15.1. Design a channel by Kennedy's theory, using Garret's diagram for the following data :

Full supply discharge = 7 cumecs
 Rugosity coefficient = 0.0225
 Bed slope = $1/4444$
 Critical velocity ratio = 1.
 Side slope of channel = $\frac{1}{2} : 1$.

Solution

$$S = \frac{1}{4444} = 22.5 \text{ cm/km.}$$

From the Garret's diagram [Fig. 15.3 (b)] find the intersection of the slope line ($S = 22.5 \text{ cm/km}$) with discharge line ($Q = 7 \text{ cumecs}$) and draw a vertical line. This line cuts many bed width lines; various values of B and D so obtained are tabulated. The values of B and D which give critical velocity ratio equal to 1 will be adopted.

$$Q = 7 \text{ cumecs; } N = 0.0225; S = 22.5 \text{ cm/km.}$$

Bed width (B) from diagram	Depth (D) from diagram	Area A = $BD + \frac{D^2}{3}$	V_0 from diagram	$V = \frac{Q}{A}$	$\frac{V}{V_0}$	Remarks
6	1.5	10.13	0.72	0.693	0.966	small
7	1.35	10.26	0.673	0.682	1.015	nure
6.75	1.38	10.15	0.685	0.685	1.005	satisfactory

Hence adopt $B = 6.75 \text{ m}$ and $D = 1.38 \text{ m}$.

Example 15.2. Design an irrigation channel in alluvial soil according to Lacey's theory, using Lacey's regime diagrams, given the following data :

Full supply discharge = 15 cumecs
 Lacey's silt factor = 1.0
 Channel side slopes = $\frac{1}{2} : 1$.

Solution.

From Lacey's regime diagram [Fig. 15.4 (b)] read the values of B and D corresponding to the intersection of silt factor line f with discharge line.

Hence the value are : $B = 15.1 \text{ m}$; $D = 1.38 \text{ m}$.

Similarly from Fig. 15.5 the value of slope

$$S = 0.19 \text{ part per } 1000 = \frac{0.19}{1000} = \frac{1}{5250}$$

Example 15.3. Design and prepare the longitudinal section, schedule of area statistics and channel dimensions of an irrigation channel from the following data.

Longitudinal levelling of channel alignment gives the following results :

Below km	Ground level (m)	Below km	Ground level (m)
0.0	157.20	2.6	155.35
0.2	157.25	2.8	155.17
0.4	157.29	3.0	155.12
0.6	156.21	3.2	154.95
0.8	156.56	3.4	154.70
1.0	156.52	3.6	154.39
1.2	156.51	3.8	154.28
1.4	156.23	4.0	154.19
1.6	156.11	4.2	154.10
1.8	156.14	4.4	153.74
2.0	155.88	4.6	153.58
2.2	155.68	4.8	153.38
2.4	155.53	5.0	153.18

Assume datum at level 150.00 m.

Full supply level of parent channel 157.70 m

Bed level of parent channel 156.00 m.

Commanded area are given as follows :

Below km	Gross commanded area (Hectares)	Culturable commanded area (Hectares)
0	24,000	18,000
1	22,000	14,000
2	18,000	12,000
3	16,000	11,000
4	14,000	10,000
5	12,000	8,000

Total loss below km 5 is 0.5 cumec. Losses are to be calculated at a rate 2 cumecs per million square meter of wetted perimeter.

Intensity of irrigation	Rabi (Wheat) 30%	Kharif (Rice) $12 \frac{1}{2} \%$
Kor Period	4 weeks	$2 \frac{1}{2}$ weeks
Kor-depth	13.4 cm	19 cm
Side slope of the channel is	$\frac{1}{2} : 1$	
	$m = 1$; Kutter's $N = 0.0225$.	

Design Procedure

1. The longitudinal section of the ground is plotted in Fig. 15.1.
2. The slope of the channel is assumed with the help of Lacey's diagrams and is tabulated in schedule of area statistics.
3. A trial slope line for full supply level is marked keeping in view the guidelines of § 15.1. The position and depth of falls are tentatively marked.
4. The actual channel design is thereafter carried out by filling schedule of area statistics.

IRRIGATION AND WATER POWER ENGINEERING

742

5. The outlet discharge factor for controlling crops is determined as follows :

$$(a) \text{ Outlet discharge factor for Rabi} = \frac{8.64b}{\Delta}$$

where $b = 4 \times 7 \text{ days}$; $\Delta = 0.184 \text{ m}$.
Hence outlet discharge factor = $\frac{8.64 \times 4 \times 7}{0.184} = 1800 \text{ hectares/cumec}$

$$(b) \text{ Outlet discharge factor for Kharif} = \frac{8.64b}{\Delta}$$

where $b = 2.5 \times 7 \text{ days}$; $\Delta = 0.19 \text{ m}$.
Hence, outlet discharge factor = $\frac{8.64 \times 2.5 \times 7}{0.19} = 780 \text{ hectares/cumec}$.

The outlet discharge factor for the controlling crops is one which gives maximum discharge.

Let us assume the culturable commanded area as 16,000 hectares.

Area under Rabi crop with intensity of irrigation as 30%

$$= 16,000 \times \frac{30}{100} = 4800 \text{ hectares}$$

Area under Kharif crops with intensity of irrigation as $12\frac{1}{2}\%$

$$= 16,000 \times \frac{12.5}{100} = 2000 \text{ hectares}$$

$$\therefore \text{ Discharge needed for Rabi crops} = \frac{4800}{1800} = 2.67 \text{ cumecs}$$

$$\text{and Discharge needed for Kharif crops} = \frac{2000}{780} = 2.63 \text{ cumecs}$$

Hence outlet discharge factor to be adopted = 1800 hectares per cumec.

Design of channel at various sections

The design is started from the tail end where the losses are known.

1. At km 5.00

Culturable commanded area below 5 km = 8000 hectares

Intensity of Rabi irrigation = 30 %

Area to be irrigated for Rabi crops

$$= 0.30 \times 8000 = 2400 \text{ hectares}$$

Outlet discharge factor = 1800 hectare/cumec

$$\therefore \text{ Discharge needed for Rabi crop} = \frac{2400}{1800} = 1.34 \text{ cumecs}$$

Total losses after 5 km = 0.5 cumec (given)

$$\text{Total discharge at km 5.00} = 1.34 + 0.5 = 1.84 \text{ cumecs}$$

Design discharge (Q) will be 10% more than total discharge.

$$\text{Therefore, design discharge} = 1.1 \times 1.84 = 2.02 \text{ cumecs}$$

IRRIGATION CHANNELS - II : DESIGN PROCEDURE

743

$$V/V_0 = 1.0 \text{ and } N = 0.0225$$

Assume slope

$$S = 0.25 \text{ m/km} = 1/4000$$

From Garret's Diagram Fig. 15.3 (a) :

Discharge Q (cumecs)	Slope S	Bed width B (m)	Water Depth D (m)	Area (A) = BD + D ² /3 (m ²)	Velocity V = Q/A (m/sec)	Critical Velocity (V _c) m/sec	C.V.R.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
2.02	$\frac{1}{4000}$	5.50	0.73	4.27	0.472	0.448	1.05
		4.90	0.79	4.16	0.485	0.472	1.03
		4.55	0.84	4.17	0.484	0.488	0.992

Hence adopt values : $B = 4.55 \text{ m}$; $D = 0.84 \text{ m}$.

2. At km 4.00

Gross commanded area below km 4.00 = 1400 hectares

Culturable commanded area = 10,000 hectares

Intensity of Rabi irrigation = 30 %

$$\text{Area to be irrigated for Rabi crops} = \frac{30}{100} \times 10,000 = 3000 \text{ hectares}$$

$$\text{Outlet discharge required} = \frac{3000}{1800} = 1.67 \text{ cumecs.}$$

$$\text{Losses below km 5.00} = 0.5 \text{ cumec.}$$

Losses in channel reach between km 5.00 to km 4.00.

For calculation of losses, the perimeter of the section at km 5.00 shall be taken into consideration as the section at km 4.00 at this stage cannot be calculated.

Wetted perimeter of the channel section at km 5.00

$$= B + \sqrt{5}D = 4.55 + \sqrt{5} \times (0.840) = 6.445 \text{ m.}$$

Thus loss in reach of km 5.00 to km 4.00

$$= \frac{6.445 \times 1000 \times 2}{10^8} = 0.013 \text{ cumec.}$$

$$\therefore \text{ Total losses below km 4.00} = \text{Losses below km 5.00} + \text{Losses in the reach of km 5.00 to km 4.00} = 0.50 + 0.013 = 0.513 \text{ cumec.}$$

$$\therefore \text{ Total discharge required at km 4.00} = 1.67 + 0.513 = 2.183 \text{ cumecs}$$

$$\text{Adopt design discharge (Q) 10\% more than total discharge} = 1.1 \times 2.183 = 2.40 \text{ cumecs.}$$

Similar method is followed for the determination of discharges at km 3.00, km 2.00 etc.

The channel design and other details are tabulated in Schedule of Area Statistics (Table 15.7).

TABLE 15.7. SCHEDULE OF AREA STATISTICS AND CHANNEL DIMENSIONS

Below km	Gross commanded area		Area to be irrigated			Outlet discharge factor	Outlet discharge	Losses in reach	Total losses	Total discharge	Channel Dimensions						Velocity (V) of flow	V/V ₀
	Ac	Culturable commanded area	Rabi (intend) 20%	Sugar	Rice (12.5%)						Bed slope	Bed width	Water depth	Design discharge	Height of Bank	Width of Bank		
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)
0	94000	18000	4800	...	2000	1800	2.88	0.0160	0.570	3.23	22.5	5.50	1.04	3.56	...	2.0	0.570	1.016
1	22000	14000	4200	...	1700	1800	2.33	0.0140	0.555	2.88	22.5	5.20	1.007	3.17	...	2.0	0.555	1.00
2	18000	12000	3600	...	1600	1800	2.00	0.0126	0.541	2.541	22.5	4.86	0.976	2.80	...	2.0	0.528	1.00
3	18000	11000	3300	...	1575	1800	1.833	0.0126	0.527	2.369	24.0	4.70	0.946	2.80	...	2.0	0.530	1.00
4	14000	10000	3000	...	1200	1800	1.67	0.0100	0.513	2.183	24.0	4.66	0.915	2.40	...	2.0	0.521	1.00
5	18000	8000	2400	...	1000	1800	1.34	...	0.580	1.34	25.0	4.58	0.840	2.02	...	2.0	0.484	0.992

Example 15.4. Calculate the economical depth of cutting for the cross-section of the canal shown in Fig. 15.19. The bed width of the channel is 5 m and top width of banks are 2 m. Side slope of excavation is 1 : 1 and of bank 1 $\frac{1}{2}$: 1. Height of banks from bed is 2.92 m throughout.

Solution

For calculating the economical depth of digging we use the relation :

$$\text{Quantity of digging} = \text{Quantity of banking}$$

That is, sectional area of digging = sectional area of two banks

$$By + zy^2 = 2(h-y)t + 2n(h-y)^2$$

$$B = 5 \text{ m}, z = 1, h = 2.92, t = 2 \text{ m}, n = 1.5$$

Substituting the value in the above equation, we get

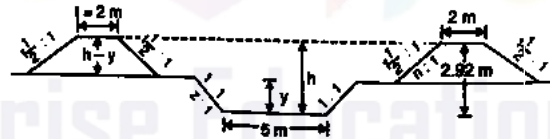


FIG. 15.19. CROSS-SECTION.

$$5y + y^2 = 2(2.92 - y)2 + 2 \times 1.5(2.92 - y)^2$$

$$\text{or } 5y + y^2 = 11.68 - 4y + 25.5 - 17.52y = 3y^2$$

$$\text{or } 2y^2 - 26.52y + 37.18 = 0 \quad \text{or } y^2 - 13.26y + 18.59 = 0$$

From which $y = 1.6 \text{ m}$.

Therefore, the economical depth of cutting = 1.60 m.

PROBLEMS

1. What do you understand by the balancing depth? Derive an expression for the same.
2. What do you understand by schedule of area statistics? How is it prepared?
3. Explain, step by step, the method of designing an irrigation channel if the culturable commanded area and the details of crops along the canal are given.
4. Draw sketches to show the section of canal, (a) partly in cutting and partly in filling, (b) wholly in cutting, (c) wholly in filling.
5. What is berm? Why is it provided in the canal?
6. Write short notes on the following :
(i) Free board in canals, (ii) Permanent land width, (iii) inspection road, (iv) berm, (v) counterberm, (vi) dowl.
7. Explain the measures adopted to remove silt from the canal.
8. How do you control weed growth in a canal?
9. Explain various methods of discharge measurement in a canal.
10. Using Lacey's diagrams, design an irrigation channel on Lacey's theory for the following data :

$$\text{Discharge } Q = 50 \text{ cumecs}$$

$$\text{Silt factor } f = 1$$

$$\text{Side slopes } = \frac{1}{2} : 1 \quad \left(\text{Ans} = 29.6 \text{ m}; D = 1.95; S = \frac{1}{6400} \right)$$

11. Using Garret's diagrams, design channel section on Kennedy's theory for the following data :

$$\text{Discharge } Q = 14 \text{ cumecs}$$

$$\text{Kutters } N = 0.0225$$

$$\text{Critical velocity ratio } m = 1$$

$$\text{Side slopes } = \frac{1}{2} : 1$$

$$\text{Bed slope } = \frac{1}{5000}$$

12. What do you understand by balancing depth? Derive an expression for the same. Hence, find the value of the balancing depth for the canal shown in Fig. 15.20. [Ans. 1.54 m]

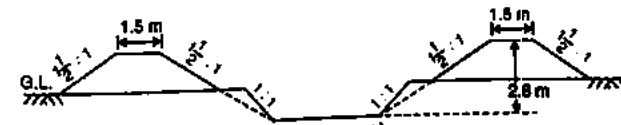


FIG. 15.20.

When the agricultural land productivity is affected by high water table due to complete saturation of soil, it is called waterlogging.

16

Waterlogging and Canal Lining

16.1. WATERLOGGING

An agricultural land is said to be waterlogged when its productivity or fertility is affected by high water-table. The depth of water-table at which it tends to make the soil water-logged and harmful to the growth and subsistence of plant life depends upon the height of capillary fringe, which is the height to which water will rise due to capillary action. The height of capillary fringe is more for fine-grained soil and less for coarse-grained ones. The normal height of the capillary fringe met within agricultural soils varies from 0.50 to 1.60 m. The crop yield is adversely affected when the capillary meniscus surface rises to within 0.6 m of ground surface. The land will, therefore, be waterlogged when the water-table is within 1.50 m [i.e. $0.6 + 0.9$] to 2.1 m [i.e. $0.6 + 1.5$] below the ground surface.

The adverse effects of high water-table upon the yield of crops also depend upon the nature of crop grown. Plants with the longer roots going deeper are more susceptible to high water-table.

The depth of water table which adversely affects the growth of different crops is given below :

Crops	Depth of water table
1. Wheat	0.9 to 1.2 metres
2. Cotton	1.5 to 1.8 m
3. Rice	0.6 m
4. Sugarcane	0.3 m
5. Fodder crop	1.2 m
6. Lucerne	2.1 to 2.4 m

The problem of waterlogging in our country is a matter of deep concern. The total water logged land is estimated at 26.3 lakh hectares in area. West Bengal and Punjab have the water logged area as 10.33 lakh and 10.12 lakh hectares. Uttar Pradesh comes third in the list with an area of 2.03 lakh hectares. The waterlogging is spreading. The magnitude of the problem will be evident from the fact that during 40 years period from 1918 to 1958, the water-table in Kalinadi-Ganga and Hindan-Kalinadi Doab (U.P.) has risen by 2 to 2.3 m and in Arid Pandu, Arind Sengar and Sengar Yamuna Doab by about 3 m. The percolation and adsorption losses from canals is a dual loss

(746)

WATERLOGGING AND CANAL LINING

747

in as much as water that could beneficially be utilized for irrigation is wasted and in addition the same causes damage to agricultural land due to waterlogging.

16.2. EFFECTS OF WATERLOGGING

The infertility of the soil when an area becomes waterlogged is usually due to the following reasons:

1. Inhibiting activity of soil bacteria

The liberation of plant food is dependent upon the activity of soil bacteria, which requires adequate amount of oxygen in the air for proper functioning. When the soil pores within the root zones of the crops normally grown are so saturated as to effectively cut off the normal circulation of air, the land is said to be waterlogged.

2. Decrease in available capillary water

Plant life draws its substance from the soil-solution round the soil particles which is drawn into the plants by capillary action and osmosis. If the water-table is high, the roots of the plants are confined to the top layers of the soil above the water table while if the water table is lower, the roots of plants have more room for growth.

3. Fall in soil temperature

A waterlogged soil warms up slowly and due to lower temperature, action of soil bacteria is sluggish and plant food available is less.

4. Defective air circulation

When the water-table is high, the drainage becomes impossible and the carbon dioxide liberated by the plant roots cannot be dissolved and taken away. Consequently fresh air containing oxygen is not drawn and activity of soil bacteria and plant growth suffers.

5. Rise of salt

The rise of water-table also causes accumulation of alkali salts in the surface soil by the upward flow of water which is established in waterlogged lands. If the underlying layers contain alkali salts in solution they are brought up with water which evaporates leaving the salt on the surface.

The alkaline deposit changes the pH value of soil. Soils with pH value 7.0 to 8.5 gives normal yields, with pH value 8.0 to 9.0 the yield decreases ; when pH value rises to 11.0, the soil becomes infertile.

6. Delay in cultivation operations

In waterlogged areas, cultivation operations such as ploughing and mulching are either impossible or difficult and in any case they are delayed. Sowing of crops and their growth are also delayed. Crop yield is poor and it arrives late in market causing loss to cultivators.

7. Growth of wild flora

In waterlogged soils, natural flora such as *water hyacinth* grows profusely. This reduces the crop yield. A cultivator has to waste money and time both for clearing it out.

8. Adverse effect on community health.

The climate of a waterlogged area becomes damp. Formation of stagnant pools may become breeding places for mosquitoes. The climate thus becomes extremely detrimental to the health of community.

16.3. CAUSES OF WATERLOGGING

Waterlogging in any particular area is normally the result of several contributory factors. The main factors causing waterlogging are given below :

1. Inadequate surface drainage

When the surface drainage is not adequate, the heavy precipitation in the area is not drained off quickly and the rain water remains stagnant over the area for considerable time. This gives rise to heavy percolation and water-table rises in the area.

2. Seepage from canal system

In nature, the water-table is in state of equilibrium. The amount of inflow is practically equal to the amount to outflow. Thus equilibrium is upset by the construction of a new canal system as a new constant source of inflow due to seepage is introduced. The water-table in the area, therefore, rises. The rise takes place up to a level where the increased inflow is balanced by the increased outflow. The increased outflow is mainly because of increase in soil evaporation due to nearness of water-table, increased transpiration from plants and increased discharge in seepage drains. Seepage losses from canal have been discussed in §16.5.

3. Over-irrigation of fields

When the irrigation water applied to the field is in excess of the requirement of the crop, deep percolation takes place which is retained in the intermediate zone augmenting the ground water storage.

4. Obstruction of natural drainage

If a natural drainage is obstructed by irrigation channel, rail or road embankments, it will not be able to pass the rain water of catchment. There will thus be flooding of land and consequent water logging.

5. Obliteration of natural drainage

Sometimes the cultivators plough up and obliterate an existing natural drainage. This results in stoppage of storm water flow, consequent flooding and waterlogging.

6. Inadequate capacity for arterial drainage

The arterial drainage or Nadi may not have adequate capacity to pass the heaviest floods in the entire catchment. As such the functions of all the drains connected to the arterial drain is seriously hampered. The flood water of local drains thus spreads over the countryside for days and heavy percolation into the subsoil causes alarming rise in water table.

7. Construction of a water reservoir

Similar to the seepage from a canal, the seepage from the reservoir augments the water-table and may cause water-logging.

8. Natural obstruction to the flow of ground water

Sometimes subsoil does not permit free flow of subsoil water due to some natural obstruction. This may accentuate the process of raising the water-table.

The creation of a high false water-table or perched water-table also leads to waterlogging.

16.4. REMEDIAL MEASURES

In devising anti-waterlogging measures, the nature and magnitude of various factors, enumerated in previous article, should be correctly assessed and allowed for. Various remedial measures adopted for prevention of waterlogging are discussed below :

1. *Efficient Surface drainage.* An efficient drainage system which permits a quick flow of rain water in short period helps to reduce the waterlogging. They have a low initial cost of construction.

2. *Under-drainage by tile drains.* The drainage of agricultural land is done more satisfactorily by the drains (Refer § 16.6). A suitable tile drain can hold the water table at a predetermined level which will be most beneficial to the crops. It has a large initial cost.

3. *Reducing percolation from canals.* Measures for substantial reduction of percolation losses from canal have to be undertaken. This can be achieved in the following way :

(a) *Lining of irrigation channels* so as to make bed and sides of canal impervious.

(b) *Lowering of full supply level of irrigation channels.* If the full supply level of irrigation channel is reduced, there will be lesser seepage loss from embankment. The effective head between full supply level and field will also reduce and, therefore, chances of a wasteful use of water is avoided.

(c) *Construction of intercepting drains.* These drains are constructed along the canals particularly at high embankment to arrest seepage flow. The distance of seepage drains from the canal should be such that while it intercepts seepage and percolation, it is not so near that it may draw water directly from the canal. This distance will depend upon the difference of level between the water surface in the canal and the natural ground water-table.

4. Restriction of Irrigation

(a) The cultivators should be educated for economic use of water and induced to divide his field into "Kiarie" to avoid wastage. He should also be encouraged to supplement his water requirement from open wells and tube wells.

(b) Area with high water table may be allowed only for Kharif irrigation and during Rabi the cultivators may irrigate from open wells and tube wells.

5. *Lining of water courses.* The losses by percolation from cultivator's watercourses are of the order of 20% and above. Their lining, therefore, further checks the inflow of canal water to subsoil through water courses.

6. *Removing obstruction in natural drainage.* Drainage crossing with road, railways and canals should be remodelled to make it more efficient.

7. **Prevention of seepage from water reservoir.** Adequate and suitably designed toe filters are provided so that seepage ultimately finds its way into the natural stream.

8. **Depletion of ground water storage by pumping.** The surplus groundwater which causes undesirable rise in the water-table can be pumped out by :

(a) **Shallow well pumping.** Water is pumped from top aquifers to depress the water-table. This water may be utilized for irrigation in some other area.

(b) **Deep well pumping.** The water is pumped out from several water bearing strata by a series of wells scattered over large areas and discharge is used for further irrigation.

In area where the danger of waterlogging has become imminent, further canal irrigation should not be introduced. Instead, tube wells should be sunk and the area should be irrigated by tube wells.

Irrigation from masonry wells also reduces waterlogging.

9. **Changes in crop pattern.** A change in crop pattern may minimise the damage to plant line.

10. **Adoption of sprinkler method for irrigation.** This reduces the percolation losses from watercourses as only predetermined amount of water is applied to the land.

16.5. LOSSES IN CANAL

The losses in canal comprise evaporation from the surface and seepage through the bed and sides of the drains.

Loss due to evaporation from a canal system depends upon the climatic conditions of the region and hence it can never be prevented. However, losses by evaporation forms a minor part, hardly 1 to 5% of seepage loss and hence, in most of the cases evaporation loss is not significant.

Loss due to seepage is the most significant as this forms the major portion of the loss of the canal water. The seepage loss depends mainly upon the following factors :

1. Position of subsoil water-table.

2. Porosity of soil and subsoil.

3. Extent of absorbing medium.

4. Design of canal cross-section.

(a) Depth of water in canal : greater the depth, greater is the loss of water.

(b) Velocity of water in the canal : the loss decreases with increase in velocity.

5. Physical properties of canal water :

(a) Temperature of water. The loss increases with increase in temperature of water.

(b) Amount of silt carried in suspension. The loss decreases with an increase in amount of silt carried on suspension.

6. Conditions of canal system. The loss decreases with the age of canal and increases with the extent of absorbing medium.

The losses in canal are usually measured by a simple method known as *inflow and outflow method*. In the method a long reach of the canal is selected. Discharge

observations are taken at the beginning and end of this reach for several days continuously. A fairly high level with constant gauge should be maintained in the reach. The outlets or any off-taking channel should be completely closed during observation period. The difference between the discharge entering the reach and that leaving the reach is the loss occurring in the reach. The losses are expressed in the following different ways :

(a) **As cumecs per million square metres of wetted perimeter.**

(i) **In north India**

Ordinary clay loam	1.77 to 2.65
Loose sandy soil	5.39 to 6.19
Gravelly soil	10.61 to 21.21

(ii) **In South India**

Decayed rock or gravel	3.05
Alluvial or red soil	2.44
Black cotton soil	1.53
Rock	0.91

In U.P. the following formulas are in use to determine the canal losses.

$$dq = 4.8603 \times 10^{-3} C(B+D)^{2/3}$$

where, dq = Loss in cumecs per km

B = Bed width in m

D = Depth in m

C = Constant, the value of which is 0.7 for main and branch canals and 1.0 for other canals.

(b) **As a depth of water lost in 24 hours over the area of wetted perimeter.**

(c) **As a percentage of the discharge of the channel.**

(d) **As a percentage per km length of the canal.**

The magnitude of seepage losses from canal will be evident from fact that on Upper Ganga Canal the losses for main canal and branches are 15%, for distributaries and minors 20% and for water courses 22%. The total evaporation losses are less than 1%. On Upper Bari Doab Canal system, the losses estimated for main canal and branches are 12.2%, for distributaries and minors 9.2% and for water courses 12.5%.

Losses in canal irrigation U.P. are assessed as below :

Sandy soil	25 to 50%
Sandy loams	15 to 25%
Fine sandy loams	10 to 20%
Heavy clay loams	5 to 15%

Depending upon the position of water-table the seepage loss from the canal occurs in two ways, namely :

(i) **Absorption**

(ii) **Percolation.**

1. **Absorption.** When the water-table is considerably below the ground level, the water seeping through pores is unable to join the water-table and wets the subsoil locally forming a saturated bulb as shown in Fig.16.1. The zone in between this saturated zone and the zone of capillary moisture remains unsaturated.

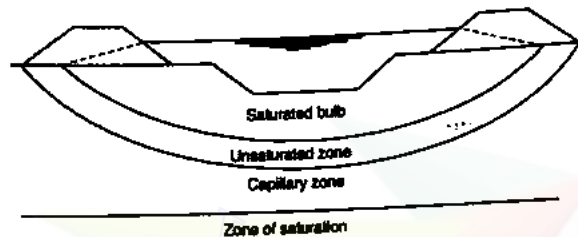


FIG. 16.1. SEEPAGE LOSS BY ABSORPTION.

2. Percolation. When the water-table is close to the ground level, the seepage water may establish a direct and continuous flow in between the canal section and water table. Thus the space between the water-table and canal bed enclosed by the two flow lines, marked in Fig. 16.2 remains completely saturated with water. When once a direct flow is established, the seepage loss depends upon the head difference between the full supply level of canal and water-table.



FIG. 16.2. LOSS OF CANAL WATER DUE TO PERCOLATION.

16.6. LAND DRAINAGE

Benefits of Drainage

The introduction of drainage system in an area has the following advantages:

1. Adequate drainage improves soil structure and increases and perpetuates the productivity of soil.
2. Waterlogged saline and alkaline lands can be reclaimed.
3. Adequate drainage leads to early ploughing and planting.
4. Drainage of the area lengthens crop growing seasons.
5. Drainage reduces water-table in an area and thereby :
 - (a) provides more available plant food by increasing depth of root zone soil.
 - (b) increases soil ventilation.
 - (c) favours growth of soil bacteria.
 - (d) assures high soil temperature.
6. Drainage decreases soil erosion and gully by increasing water infiltration into soil.
7. Excess salt from soil can be leached out.

Types of Drains

The drains can be divided into two main types, namely :

- (i) Open drains,
- (ii) Closed drains.

Open drains can be further sub-divided into :

1. **Shallow surface drains.** It reduces percolation to ground water but is of no use if water has already permeated to ground water reservoir. This drains excess irrigation water applied to the field and accelerates removal of storm water.

2. **Deep open drains.** These can be used to reduce water logging without the provision of the tile drains. These will then have to be provided at a distance of 0.75 km or even less. They are also commonly used as outlet drains for closed drain system.

Under-drains or Tile-drains : Tile drains are located at suitable depths below the ground surface above the impervious clay stratum impeding the natural percolation of water. They are preferably placed in a medium or high permeability.

16.7. DESIGN AND MAINTENANCE OF OPEN DRAINS

Open drains are very useful in removing the local accumulation of water expeditiously. These drains are not needed if the country has a good slope with sufficient number of well defined drainage lines nor they are needed in arid areas with very little rainfall. They are mostly needed in deltaic tracts where there are some under-ground obstructions of impervious material across natural drainage lines.

The drainage system in an area is generally introduced many years after the introduction of a canal system especially when the rise of water-table is very slow.

Layout. The alignment of drains should follow natural drainage lines, keeping in view the amount of falls required for the surface slope generally called *drainage head*.

Slope. A slope of 1/300 to 2/300 is generally satisfactory in deltaic tract.

Capacity of drain. The drain is designed for a capacity to carry subsoil water in addition to the natural drainage of the country and the quantity of water expected from rains. Capacity may be kept such as to remove surface water in 3 to 5 days, since no harm to crop occurs during their submergence upto 3 to 5 days.

It is considered uneconomical to design the earthen section of surface drain to cope with maximum anticipated discharge of the catchment. In Uttar Pradesh, the design discharge for the earthen section is kept only 50% of the maximum anticipated discharge but the masonry works on the drains are designed for maximum discharge. For the plains of Ganga Yamuna Doab, the maximum anticipated discharge is calculated at 0.054 cumecs per hectare of the catchment area while the design discharge for masonry is calculated at 0.108 cumecs per hectare of catchment area. Drainage system in Punjab is designed at 0.0343 cumecs per hectare.

Section. The design of a drain is done as per the design of irrigation canal, but critical velocity is not given any consideration. However, if the drain has to carry the rain water, then $V/V_0 \geq 1$. It is desirable to provide a cunnette (a small drain in the bed of larger drain) in the centre of the drain with bed gently slopping towards it to carry the small seepage discharge in case a drain is receiving both storm and supply level is kept about 0.6 m below the ground level.

Maintenance. The main problem of maintenance is the removal of aquatic weeds. A reference may be made to § 15.12 for the same.

16.8. UNDER-DRAINS OR TILE DRAINS

The drainage of waterlogged agricultural land by surface drains is not very satisfactory for the following reasons :

1. Valuable agricultural land is wasted in the construction of surface drains.
2. Bridges across the drains are essential. Even after the bridge construction, the natural communication for agricultural operation is restricted.
3. Valuable plant foods are washed down in the open drains.

Thus the drainage of agricultural land is done more satisfactorily by covered tile drains. A tile drain is located at suitable depth below the ground surface and is usually covered with filter of coarse sand and bajri. The gravitational water at the level of tile drain is under pressure due to the head of water above it. It enters the tile drains through the open joints generally up to 10 cm wide at the end of tiles. The drains have their outlets in natural or artificial channels which are so situated that they are not usually submerged in normal rains.

16.9. LAYOUT OF A TILE DRAIN SYSTEM

The layout of tile drain is governed by the topography of the area. Various possible alternative plans of layout for tile drainage system is shown in Fig. 16.3 and are discussed below :

1. **Natural System.** The natural system is preferred in rolling topography, where drainage of isolated area is required. In this system the mains and connected laterals are provided in natural course as shown in Fig. 16.3(a).

2. **Grid iron layout.** The drainage system consists of laterals and submains. Laterals are provided only on one side of submain. The system is used when the land is practically level or where the land slopes away from the submain on one side and when the entire area has to be drained.

3. **Herring bone pattern.** In this system laterals join the submains from each side alternately [Fig. 16.3(c)]. It is adopted when the submain is laid in depression. The land along the submain is double drained but since it is in a depression, it probably requires more drainage than the land on the adjacent slopes.

4. **Double main system.** The system has two mains with separate laterals for each as shown in Fig. 16.3(d). This system is adopted when the bottom of depression is wide. This arrangement reduces the length of the laterals and eliminates the break in slope of the lateral at the edge of depression.

5. **Intercepting tile drains.** In this system there is no lateral drain. A submain is provided at the toe of slope as shown in Fig. 16.3(e). This arrangement is adopted when the main source of drainage is from a hilly land.

Mains and submains should be kept away from trees as the roots of trees can easily enter the open joints of the under drains.

16.10. FLOW OF GROUND WATER TO DRAINS

The tile drains should be fixed in such a fashion that the water table can be maintained at a pre-determined level. This can be achieved by keeping it about 0.30 m below the level at which water table is desired to be maintained.

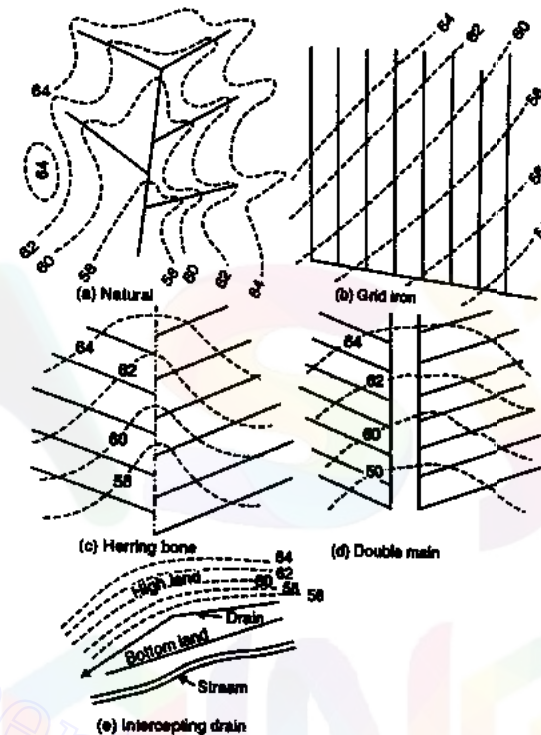


FIG. 16.3. LAYOUT OF TILE DRAINS.

With reference to Fig. 16.4, let L be the spacing of the drains, and a its height above the impervious stratum. Maximum height of drained water-table above the stratum

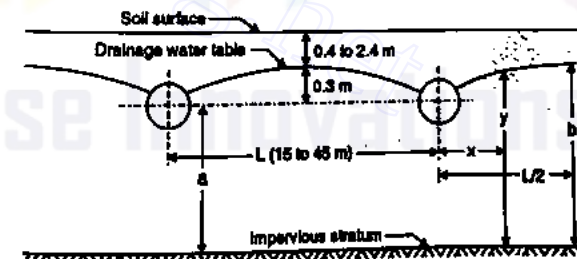


FIG. 16.4. GROUND WATER IN A DRAINED FIELD.

is b . Let y be the height of the drained water-table above the stratum at distance x from the centre of the drain.

Assumptions

1. The hydraulic gradient at a distance x from the drain is dy/dx .
2. Flow lines are parallel and cross-sectional area of flow at a distance x is $y \times 1 (-y)$.
3. Flow q towards the drain is inversely proportional to the distance from the drains.

If Q_D is the design flow per meter length of drain,

then $q = \frac{1}{2} Q_D$ when $x = 0$
and $q = 0$ when $x = L/2$.

$$\text{Therefore } \frac{q}{Q_D/2} = \frac{L/2 - x}{L/2} \quad \dots(16.1)$$

As per Darcy's Law

$$q = k i A = k y \frac{dy}{dx} \quad \dots(16.2)$$

From Eq. 16.1 and Eq. 16.2

$$\frac{Q_D}{L} (L/2 - x) = k y \frac{dy}{dx} \quad \dots(16.3)$$

or

$$\frac{Q_D}{2kL} (L - 2x) = y \, dy$$

$$\text{Integrating } \frac{Q_D}{2kL} (Lx - x^2) = \frac{y^2}{2} + C$$

When $x = 0$, $y = a$: So $C = -a^2/2$

$$k = \frac{Q_D (Lx - x^2)}{L (y^2 - a^2)} \quad \dots(16.4)$$

When $x = L/2$, $y = b$, hence

$$L = \frac{4k (b^2 - a^2)}{Q_D} \quad \dots(16.5)$$

Eq. 16.5 shows that spacing of the drains is independent of drain size. Actually an increase in drain diameter produces slight increase in yield. Q_D will depend upon infiltration discharge into the ground which should be removed by the drain. Its value is usually assumed at 1% of average annual rainfall at the place, to be removed by the drain in 24 hours.

Let P_{av} (metres) be the average annual rainfall at the place. Then

$$Q_D = \frac{0.01 P_{av} (L \times 1)}{24 \times 3600} = \frac{P_{av} \cdot L}{8.64 \times 10^6} \text{ cumecs/m length of drain.}$$

Substituting the value of Q_D in Eq. 16.5, we get

$$L = \frac{4k (b^2 - a^2)}{P_{av} L / 8.64 \times 10^6}$$

or

$$L = \left[\frac{(8.64 \times 10^6) 4k (b^2 - a^2)}{P_{av}} \right]^{1/2}$$

See Example 16.4 for illustration.

Drainage coefficient

Drainage coefficient is the rate at which water is removed by the drain. Its value depends upon the rainfall, but also varies with type of soil, type of crop, degree of surface drainage etc. It is expressed as the depth of water in cm (or m) to be removed in 24 hours from the drainage area. The common values of the drainage coefficient, suggested for humid regions by the U.S. Soil Conservation Service are: 1 to 2.5 cm/day for mineral soils and 1.25 to 10 cm per day for organic soils for different crops. Its value is so fixed that 1% of the average annual rainfall in that area is removed per day.

16.11. LINING OF IRRIGATION CHANNELS

1. Necessity

Lining of canal is necessary:

- (i) to minimise the seepage losses in canal.
- (ii) to increase the discharge in canal section by increasing the velocity.
- (iii) to prevent erosion of bed and side due to high velocities.
- (iv) to retard the growth of weeds, and
- (v) to reduce maintenance of canal.

2. Advantages of Lining.

The main advantages derived by canal lining are mentioned below:

1. The lining of canals prevents seepage loss and thus more area can be irrigated by the water so saved. The cost of irrigation is, therefore, reduced.
2. The lining of canal is an important anti-water logging measure as it reduces seepage to the adjoining land.
3. The lining provides a smooth surface. The rugosity coefficient, therefore, decreases. The resistance to flow also decreases and hence the velocity of flow in the lined canal increases.
4. The increased velocity minimises the losses due to evaporation.
5. The increased velocity helps to provide a narrow cross-section for lined channels.
6. Higher velocity helps in providing a flatter hydraulic gradient or bed slope. Thus better command can be obtained.
7. Higher velocity prevents silting of channel.
8. Lining makes the banks more stable in light textured soil.
9. Lining reduces maintenance costs and possibility of breaching due to increased stability of section.
10. Lining of a canal prevents or reduces weed growth.

11. Lining of a canal increases available head for power generation as a flatter gradient can be provided.
12. Canal lining assures economical water distribution.
13. Canal lining prevents water to come in contact with harmful salts during transit.
3. **Disadvantages of Canal Lining**

The canal lining has certain disadvantages although the advantages far out-weigh the meagre disadvantages. The disadvantages of canal lining are mentioned below:

1. Canal lining requires a heavy initial investment.
2. Lining being permanent, it is difficult to shift the outlets very often.
3. It is very difficult to repair the damaged lining.
4. A lined channel section is without a berm. The additional safety provided by the berm for vehicular and pedestrian traffic is, therefore, absent in a lined channel.
4. **Suitability of Canal Lining Material**

A canal lining material, to be suitable, should have the following properties :

1. The material used for lining should provide complete *water tightness*.
2. The material used should have low coefficient of rugosity so as to make the section *hydraulically more efficient*.
3. The material chosen for canal lining should be *strong and durable*.
4. The lining should not have a very *high initial cost*. Subsequent maintenance cost of canal lining should be *very low*.
5. The material used should be able to *resist growth of weeds and attack of burrowing animals*.
6. The material used should be unaffected by *trampling of cattles*.
7. The material should withstand high velocity.
8. The material should permit construction of required slope easily.

16.12. TYPES OF LINING

The following are the important types of concrete lining used in India:

- (a) **Hard surface type lining :**
 1. Cement concrete lining.
 2. Shotcrete lining.
 3. Precast concrete lining.
 4. Cement mortar lining.
 5. Brick lining.
 6. Stone blocks, or undressed stone lining.
 7. Asphaltic lining.
- (b) **Earth type lining :**
 8. Soil cement lining.
 9. Clay puddle lining.
 10. Sodium carbonate lining.
- (c) **Buried and protected membrane type lining :**
 11. Prefabricated light membrane lining.
 12. Bentonite soil and clay membrane lining.
 13. Road oil lining.

1. **Concrete lining :** Concrete lining has found only a limited use in India because of the high initial cost. Concrete as a lining material has excellent hydraulic properties.

Thickness of concrete lining : The thickness of lining is governed by the requirement of imperviousness and structural strength. Usual thickness provided is from 5 to 10 cm for M 15 concrete and is 7.5 cm to 15 cm for M 10 concrete.

Bank slope : Earthen banks of canals upon which lining has to rest are made at a self-supporting slope so that no earth pressure is exerted upon lining.

Reinforcement : Reinforcement is not provided except where it may be particularly required. Reinforcement interferes with the working of mechanical equipment but if provided it prevents formation of wide gap and possible faulting in case of failure of such grade. It also reduces width of temperature crack.

Joints : Construction joints localise volume changes due to shrinkage of concrete and avoid cracking. Lining is usually provided in panels not more than 3 m. Different types of joints provided are shown in Fig. 16.5.

Where lining operations are continuous, transverse grooves or longitudinal and transverse grooves both, at a spacing of 3 to 5 m are formed.

Construction of concrete lining : Prepare a subgrade and compact it fully. Saturate subgrade to a depth of 30 cm in sandy soil and 15 cm in other soils and then treated in any of the following manner before laying concrete :

- (a) Lay a base coat of 1 : 4 cement sand slurry on the subgrade. This is cheapest and best treatment.
- or (b) Spread oil paper on the subgrade.
- or (c) Spread crude oil on subgrade.

The concrete used for lining generally has a mix ratio 1 : 4 : 8 or 1 : 3 : 6 or 1 : 4 : 6. The concrete is usually laid in alternate blocks. The form work is removed after seven days and the remaining blocks are laid then. After the concreting is over, the entire lined surface is properly cured.

2. **Shotcrete lining :** In this type of lining, a mixture of cement and sand (1 : 4) is shot at the subgrade through a nozzle. The thickness of this type of lining (1 : 4) is shot at the subgrade through a nozzle. The thickness of this type of lining varies from 2.5 to 6.5 cm. Shotcrete consumes large amount of cement. Shotcrete can be placed on irregular subgrade and hence fine dressing of subgrade is not required. Shotcrete is also used for repair of old but sound concrete lining.

3. **Precast concrete lining :** This type of concrete lining consists of precast slabs usually 90 cm × 30 cm in size. The thickness of each slab is from 5 to 6.5 cm. The blocks are manufactured with an interlocking arrangement. The slabs are laid

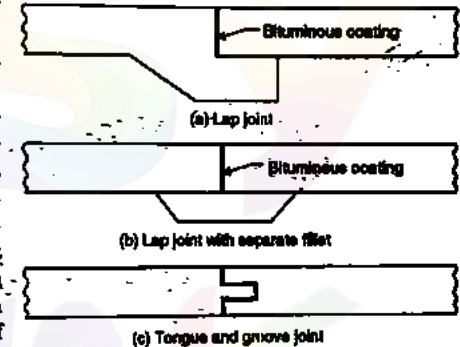


FIG. 16.5. DIFFERENT TYPES OF JOINTS.

on well prepared and compacted subgrade. The joint is grouted with cement or is sealed with asphalt to prevent any seepage through joint.

4. **Cement mortar lining** : Cement mortar lining has never been used as such. It is usually used as sandwich material between brick layers. Thickness for this type of lining is kept from 1 to 4 cm. The method of construction is similar to that of concrete lining. A large amount of cement is consumed in this type of lining and hence it is quite costly.

5. **Brick lining** : This type of lining consists of a single or double layer of brick masonry or a layer of brick masonry followed by a layer of tiles laid in mortar. The first layer is laid on a 12 mm layer of 1 : 6 cement mortar. The mortar is first spread on a properly compacted and wetted subgrade. A 12 mm thick layer of plaster in 1 : 3 cement mortar is laid over the first layer. Then the second layer is laid over it in 1 : 3 cement mortar. The size of brick especially made is restricted to $80 \times 15 \times 6$ cm for convenience of handling.

This type of lining has been used in Punjab on Bhakra and Haveli canals. On Bhakra canal its performance has been found to be extremely satisfactory. This lining is hydraulically as efficient as that of concrete lining. In case of any settlement of subgrade, the mortar joints between bricks provide for numerous cracks which are so fine that they do not permit any seepage through them. In case of any abnormal settlement, the failure in this types of lining will be local and lining can be repaired easily.

6. **Asphaltic lining** : Asphaltic lining is carefully controlled mixture of asphalt and grade aggregate mixed and placed at a high temperature of 200°C and covered with a 30 cm layer of earth material for a protection. The mix is placed either by hand or by an equipment similar to that of concrete.

7. **Soil cement lining** : Soil cement is a mixture of cement and soil. Soil used in soil cement should not contain more than 35% of fine fraction passing 200 sieve. The mixture, in proportion of 90 to 95 % soil and 10 to 5% cement is thoroughly mixed dry and is then added with water to raise the moisture content to optimum. It is, therefore, placed on the subgrade and properly compacted. After laying in a reach is complete, it is covered with wet sand or wet straw for seven days for curing.

In U.S.A. plastic soil cement has been used with a higher consistency to get a faster rate of construction.

8. **Clay puddle lining** : Clay puddle is produced from clay by first exposing clay to weathering. It is then mixed with water to bring it to the saturation and is pugged thoroughly by trampling under man's or cattle's feet. The thickness of lining is 30 cm. It is then protected by a layer of earth material.

9. **Sodium carbonate lining** : The mixture consists of clayey soil and sodium carbonate in a proportion of at least 10% clay and 6% sodium carbonate. Thickness of lining is kept as 10 cm. This type of lining is used on small canal and water coarse and not found to be durable.

10. **Stone block lining** : This consists of undressed stone block set in mortar and laid over a prepared subgrade. The lining is able to check seepage effectively but

has a considerable resistance to flow of water. Dressed stone block lining is effective but costly.

11. **Pre-fabricated light weight membrane** : They are matted fibres of asbestos or jute and is coated with asphalt. It is laid on a smooth and prepared subgrade, and is covered with a layer of earth material.

12. **Bentonite and clay membrane** : This consists of bentonite or clay blanket 4 cm thick laid over a prepared subgrade, and covered with earth.

13. **Road oil lining** : The road oil sprinkled on subgrade in a thickness of about 1.5 mm is sufficient enough to saturate subgrade to a depth of 8 cm. The subgrade is then rolled so that oil enters the soil pores.

16.13. DESIGN OF LINED CANAL

Fig. 16.6 shows two types of lined canal sections : (i) triangular shaped or curved channel and (ii) trapezoidal shaped or flat bottom shaped channel. The side slopes are so selected that they are nearly equal to the angle of repose of the soil so that no earth pressure is imposed on the lining. The corners are rounded off to improve the hydraulic efficiency. Triangular shaped section is adopted for small discharges, as it is the best discharging section. For higher discharges, trapezoidal section may be adopted. If the limiting velocity is taken as 1.8 m/sec, then circular section can be adopted only if the discharge is less than 85 cumecs. However, Indian Standard recommends only trapezoidal section for all discharges.

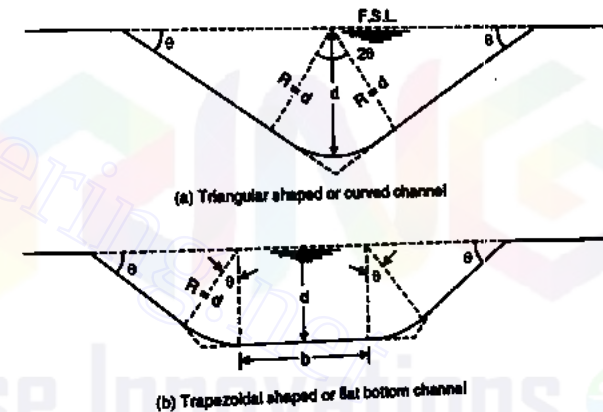


FIG. 16.6. SECTION OF LINED CHANNEL.

In order to design a lined canal section following data is required: (i) discharge Q , (ii) rugosity coefficient N , (iii) longitudinal slope S , (iv) side slopes, and (v) limiting velocity V .

The procedure used for design of trapezoidal shaped channel is given below

1. Knowing limiting velocity V , the rugosity coefficient N and longitudinal slope, S , work out the value of hydraulic mean depth R by the use of Manning's equation:

$$V = \frac{1}{N} R^{2/3} S^{1/2} \quad \dots(16.6)$$

2. Find the area of cross-section from the continuity equation :

$$A = \frac{Q}{V}$$

3. Find out the wetted perimeter of the section

$$P = \frac{A}{R}$$

4. Also, for trapezoidal shaped channel (Fig. 16.6 b), the area of cross-section

$$A = bd + nd^2 \frac{\theta}{\pi} + \frac{2d^2 \cot \theta}{2} = bd + d^2 \theta + d^2 \cot \theta \quad \dots(16.7)$$

$$\text{or} \quad A = bd + d^2 (\theta + \cot \theta) \quad \dots(16.8)$$

$$\text{Perimeter} \quad P = b + 2d\theta + 2d \cot \theta = b + 2d(\theta + \cot \theta)$$

Since A and P are known from steps (3) and (4), the value of bed width and depth d can be worked out from equations 16.7 and 16.8. The values of N may be taken from Table 16.1. See Example 16.1 for illustration.

TABLE 16.1. VALUES OF RUGOSITY COEFFICIENT N FOR LINED CHANNELS WITH STRAIGHT ALIGNMENT (IS : 4745-1968)

S. No.	Surface characteristics	Value of N
1.	Concrete with surface as indicated below :	
	(a) Formed, no finish	0.013-0.017
	(b) Trowel finish	0.012-0.014
	(c) Float finish	0.013-0.015
	(d) Float finish, some gravel on bottom	0.015-0.017
	(e) Gunite, good section	0.016-0.019
	(f) Gunite, wavy section	0.018-0.022
2.	Concrete bottom float finished sides as indicated below :	
	(a) Dressed stone in mortar	0.015-0.017
	(b) Random stone in mortar	0.017-0.020
	(c) Cement rubble masonry	0.020-0.025
	(d) Cement rubble masonry plastered	0.016-0.020
	(e) Dry rubble (riprap)	0.020-0.030
3.	Gravel bottom sides as indicated below :	
	(a) Formed concrete	0.017-0.020
	(b) Random stone in mortar	0.020-0.023
	(c) Dry rubble (riprap)	0.023-0.033
4.	Brick	0.014-0.017
5.	Asphalt	
	(a) Smooth	0.013
	(b) Rough	0.016
6.	Wood planed clean	0.011-0.013
7.	Concrete lined excavated rock with :	
	(a) Good section	0.017-0.020
	(b) Irregular section	0.020-0.027

16.14 INDIAN STANDARD ON DESIGN OF LINED CANALS (IS : 4745-1968)

Fig. 16.7 shows the details of cross-section of lined canal according to IS : 4745-1968.

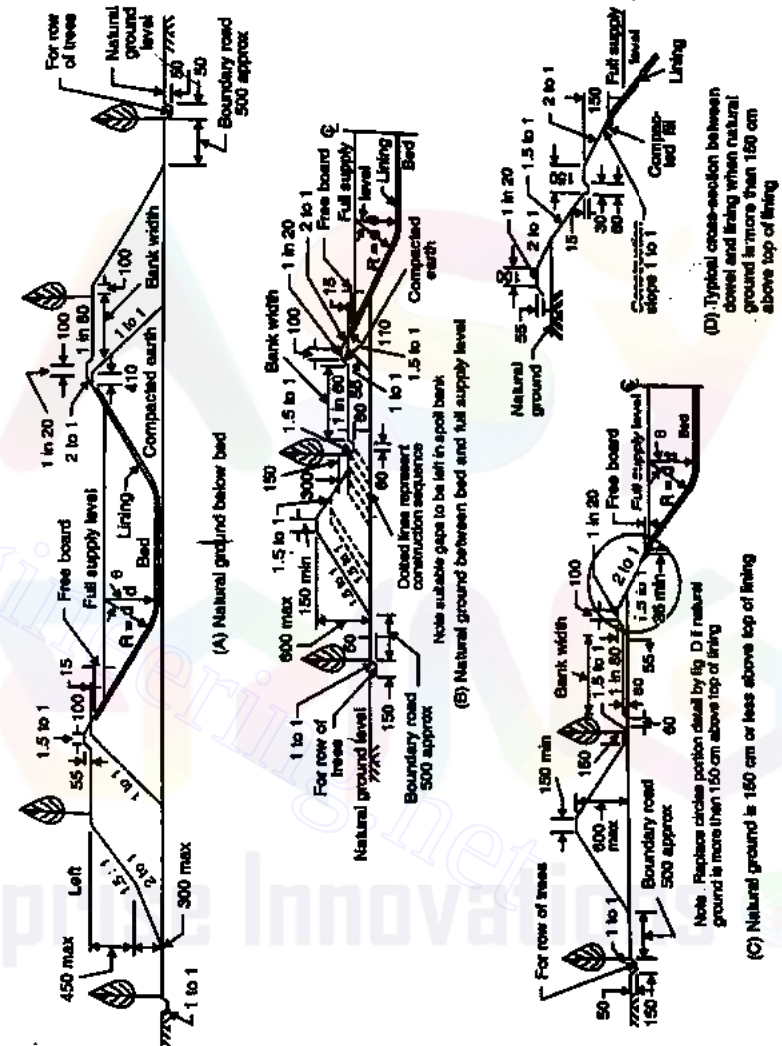


FIG. 16.7. TYPICAL CROSS-SECTION OF LINED CANAL (IS : 4745-1968) (ALL DIMENSIONS ARE IN CENTIMETRES)

1. **Side slopes.** Lining is usually made to rest on slopes corresponding to the angle of repose of the natural soil so the slopes shall be such that no earth pressure is exerted on the back of the lining.

2. **Free board.** A free board of 0.75 m shall be provided. Free board shall be measured from the full supply level to the top of lining.

3. **Bank widths.** The following bank widths shall be provided:

- (a) 8 m for cutting and filling reaches for main canals,
- (b) 6.5 m for cutting reaches for branches canals, and
- (c) 6.5 m on left side and 5 m on the right side for filling reaches for branch canals

4. **Dowel.** This shall be provided in accordance with Fig. 16.7.

5. **Roadways and banks.** These shall be provided in accordance with Fig. 16.7.

6. **Velocity.** Allowable velocity of about 1.5 to 2.0 m/sec is recommended for lined canal.

Table 16.2 gives the values of limiting velocities in different kinds of linings.

TABLE 16.2. LIMITING VELOCITIES IN DIFFERENT TYPES OF LININGS

Type of Lining	Permissible Velocity (m/sec)
1. Cement concrete lining (un-reinforced)	2.0 to 2.6
2. Burnt clay tile and brick lining	1.6 to 1.8
3. Asphaltic concrete	1.4 to 1.6
4. Flag stone lining	1.5 to 1.7

Critical velocity ratios are not applicable to lined canals but the possibility of silting cannot be neglected and, hence, the critical velocity ratio should be aimed at higher than unity.

The velocity may be computed from the following formula :

$$V = \frac{R^{2/3} S^{1/2}}{N}$$

where V = mean velocity

R = hydraulic mean depth

S = slope of the canal, and

N = Rugosity coefficient as given in Table 16.1

Example 16.1. Design a trapezoidal shaped concrete lined channel to carry a discharge of 100 cumecs at a slope of 25 cm/km. The side slopes of the channel are 1.5 : 1. The value of N may be taken as 0.016. Assume the limiting velocity as 1.5 m/sec.

Solution. Use Manning's equation

$$V = \frac{1}{N} R^{2/3} S^{1/2}$$

$$\text{where } S = \frac{25}{1000 \times 100} = \frac{1}{4000}$$

$$V = 1.5 \text{ m/sec}$$

$$N = 0.016$$

$$1.5 = \frac{1}{0.016} R^{2/3} \frac{1}{\sqrt{4000}}$$

or

$$R = 1.87 \text{ m.}$$

The channel section is assumed to be trapezoidal as shown in Fig. 16.6(b). For side slope $1\frac{1}{2}:1$, $\cot\theta = 3/2 = 1.5$ and $\theta = 0.59$ radian.

Area of cross section of the channel

$$A = \frac{Q}{V} = \frac{100}{1.5} = 66.67 \text{ m}^2$$

Perimeter of the section

$$P = \frac{A}{R} = \frac{66.67}{1.87} = 35.65 \text{ m.}$$

$$\text{From Eq. 16.7 } 66.67 = bd + d^2(\theta + \cot\theta)$$

$$35.65 = b + 2d(\theta + \cot\theta)$$

Substituting for θ and $\cot\theta$

$$66.67 = bd + d^2(2.09)$$

$$35.65 = b + d(4.18)$$

...(a)

...(b)

From Eq.(b)

$$b = 35.65 - 4.18d$$

Substituting it in equation (a)

$$d(35.65 - 4.18d) + 2.09d^2 = 66.67$$

$$d^2 - 17.1d + 31.9 = 0$$

$\therefore$

$$d = 2.145 \text{ m}$$

and

$$b = 35.65 - 4.18(2.145) = 26.68 \text{ m.}$$

Thus required bed width = 26.68 m and depth is 2.15 m.

Example 16.2. Solve example 16.1 when the limiting value of velocity is not given, but $b/d = 8$.

Solution. From Eq. 16.7, substituting the values of θ and $\cot\theta$,

$$A = bd + d^2(\theta + \cot\theta)$$

or

$$A = bd + 2.09d^2$$

and

$$P = b + 4.18d.$$

Therefore, velocity in the channel

$$V = Q/A = \frac{100}{bd + 2.09d^2} \quad \dots(1)$$

Hydraulic mean depth

$$R = \frac{A}{P} = \frac{bd + 2.09d^2}{b + 4.18d}$$

Using Manning's equation

$$V = \frac{1}{N} R^{2/3} S^{1/2} = \frac{1}{0.016} R^{2/3} \frac{1}{\sqrt{4000}} = \frac{R^{2/3}}{1.01} \quad \dots(2)$$

Hence

$$R^{2/3} = 1.01 V$$

Substituting the values of R and V , we have

$$\left[\frac{bd + 2.09 d^2}{b + 4.18 d} \right]^{2/3} = 1.01 \left[\frac{100}{bd + 2.09 d^2} \right]$$

This is a equation between b and d . But $b = 8d$

$$\left[\frac{8d^2 + 2.09 d^2}{8d + 4.18 d} \right]^{2/3} = 1.01 \left[\frac{100}{8d^2 + 2.09 d^2} \right]$$

or

$$\left[\frac{10.09}{12.18} d \right]^{2/3} = 1.01 \left[\frac{100}{10.09 d^2} \right]$$

or

$$d^{8/3} = \frac{101}{10.09} \left[\frac{12.18}{10.09} \right]^{2/3} = \frac{101 \times 1.134}{10.09} = 11.348$$

$$d = (11.348)^{3/8} = 2.487 \approx 2.49 \text{ m}$$

$$b = 8d = 8 \times 2.487 = 19.89 \text{ m}$$

Therefore, required bed width = 19.89 m $\approx$ 20 mand depth = 2.49 m $\approx$ 2.5 m

Example 16.3. Design a concrete lined channel to carry a discharge of 45 cumecs at a slope of 1 in 10,000. The side of the channel are $1\frac{1}{2}:1$ and Manning's N may be taken as 0.018.

Solution. The channel is assumed to be of triangular section as shown in Fig. 16.6 (a)

$$\text{Area } A = \pi d^2 \frac{\theta}{\pi} + 2 \frac{d^2}{2} \cot \theta = d^2 (\theta + \cot \theta)$$

where d = depth of central portion

$$\text{Perimeter } P = 2\pi d \frac{\theta}{\pi} + 2d \cot \theta = 2d (\theta + \cot \theta)$$

Hydraulic mean depth,

$$R = \frac{d^2 (\theta + \cot \theta)}{2d (\theta + \cot \theta)} = d/2.$$

From Manning's equation

$$V = \frac{1}{N} R^{2/3} S^{1/2} = \frac{1}{0.018} \left(\frac{d}{2} \right)^{2/3} \frac{1}{\sqrt{10000}} = \frac{d^{2/3}}{2.86} \quad \dots (a)$$

Also

$$V = \frac{Q}{A} = \frac{45}{d^2 (\theta + \cot \theta)}$$

where $\cot \theta = 1.25$, since side slope is $1.25:1$

$$\therefore \theta = 0.675 \text{ radian}$$

Hence,

$$V = \frac{45}{d^2 (0.675 + 1.25)} = \frac{45}{d^2 (1.925)} \quad \dots(b)$$

Equating (a) and (b) we get

$$\frac{d^{2/3}}{2.86} = \frac{45}{d^2 (1.925)}$$

Hence

$$d^{2/3} = \frac{45 \times 2.86}{1.925} = 66.8$$

From which

$$d = 4.83 \text{ m}$$

Example 16.4. Design a concrete lined channel of trapezoidal section to carry a discharge of 250 cumecs at a slope of 1 in 6000. The side slopes of the channel are to be made as $1\frac{1}{2}:1$ and a limiting depth of 3 m is to be maintained. Take N for the lining material as 0.015.

Solution : For a channel of trapezoidal section, we have

$$A = bd + d^2 (\theta + \cot \theta)$$

and

$$P = b + 2d (\theta + \cot \theta)$$

For $1\frac{1}{2}:1$ side slope, $\cot \theta = 1.5$ and $\theta = 0.59$ radian.Taking the above values along with $d = 3$ m, we get

$$A = bd + d^2 (0.59 + 1.5) = 3b + 9(2.09) = 3b + 18.81 \quad \dots(1)$$

and

$$P = b + 2d (0.59 + 1.5) = b + 6(2.09) = b + 12.54 \quad \dots(2)$$

Hence

$$R = \frac{A}{P} = \frac{3b + 18.81}{b + 12.54} \quad \dots(3)$$

Now,

$$Q = V \times A = \frac{1}{N} A R^{2/3} S^{1/2}$$

or

$$250 = \frac{1}{0.015} (3b + 18.81) \times \left(\frac{3b + 18.81}{b + 12.54} \right)^{2/3} \frac{1}{\sqrt{6000}}$$

or

$$\frac{(3b + 18.81)^{5/3}}{(b + 12.54)^{2/3}} = 290.47 \quad \dots(4)$$

Solving the above equation by trial and error, we get

$$b \approx 44.8 \text{ m}$$

Hence

$$b = 44.8 \text{ m and } d = 3 \text{ m.}$$

Example 16.5. The annual rainfall in a certain area is 80 cm. Find the spacing of drains if one percent of average annual rainfall is to be drained in 24 hours.

Given : depth of impervious stratum from top of soil surface = 10 metres. Position of drains is 1.8 m below top soil surface and the depth of highest position of water table below top soil surface = 1.5 m. Take permeability $k = 1 \times 10^{-4}$ m/sec.

Solution: Let the spacing of the drains be L metres

One percent of average rainfall in an area equal to $L \times 1$ is

$$= \frac{0.80 \times L \times 1}{100} \text{ cubic metres per metre length}$$

This is to be removed in 24 hours.

Hence $Q_D = \frac{0.80 L}{100 \times 24 \times 3600}$ cumecs
 $b = 10 - 1.5 = 8.5 \text{ m}$ and $a = 10 - 1.8 \text{ m} = 8.2 \text{ m}$

Now $L = \frac{4k(b^3 - a^3)}{Q_D} = \frac{4 \times 1 \times 10^{-4} [(8.5)^3 - (8.2)^3]}{Q_D}$
 or $L = \frac{4 \times 10^{-4} [(8.5)^3 - (8.2)^3] \times 100 \times 24 \times 3600}{0.80 L} = \frac{21643.2}{L}$

From which $L = 147.1 \text{ m}$.

Hence, drains should be spaced at 147 m c/c.

Example 16.6. Given $L = 30 \text{ m}$; $Q_D = 4 \times 10^{-6}$ cumecs
 $a = 8 \text{ m}$ and $b = 8.3 \text{ m}$. Find k .

Solution

Substituting these values in Eq. 16.5 :

$$L = 30 = \frac{4k[(8.3)^3 - (8)^3]}{4 \times 10^{-6}}$$

Hence $k = \frac{30 \times 4 \times 10^{-6}}{4[(8.3)^3 - (8)^3]} = 6.13 \times 10^{-5} \text{ m/sec}$

Example 16.7. Given $L = 50 \text{ m}$, $a = 10 \text{ m}$, $b = 10.3 \text{ m}$ and $k = 1 \times 10^{-5} \text{ m/sec}$.

If the drains carry 1% of average annual rainfall in 24 hours, find the average annual rainfall.

Solution

$$Q_D = \frac{4 \times 1 \times 10^{-5} [(10.3)^3 - (10)^3]}{50} = 4.872 \times 10^{-6} \text{ cumecs.} \quad \dots(1)$$

If P_{av} is the average annual rainfall then one percent of average rainfall = $\frac{P_{av}}{100}$

Since discharge Q_D is for 1 m length assumed, area being served by one drain = $L \times 1$

$$Q_D = \frac{P_{av} \times L \times 1}{24 \times 3600 \times 100} = \frac{50 P_{av}}{24 \times 3600 \times 100} \quad \dots(2)$$

Hence $4.872 \times 10^{-6} = \frac{50 P_{av}}{24 \times 3600 \times 100}$

From which $P_{av} = 0.84 \text{ m} = 84 \text{ cm}$

Example 16.8. There are two different areas A and B to be drained by means of closed tile drainage system.

If $\frac{k_A}{k_B} = \frac{2}{1}$; $\frac{L_A}{L_B} = \frac{1}{1.5}$ and $\frac{(b^3 - a^3)_A}{(b^3 - a^3)_B} = \frac{5}{6}$

find the ratio $\frac{(Q_D)_A}{(Q_D)_B}$. If the drains carry 1% of average annual rainfall in 24 hours in both the cases, find the ratio $\frac{P_A}{P_B}$.

Solution

As per equation 16.5, $Q_D = \frac{4k(b^3 - a^3)}{L}$
 $\frac{(Q_D)_A}{(Q_D)_B} = \frac{4k_A(b^3 - a^3)_A L_B}{4k_B(b^3 - a^3)_B L_A} = \frac{2}{1} \times \frac{5}{6} \times 1.5 = 2.5$

If 1% of average annual rainfall is carried by the drain in 24 hours then

$$Q_D = \frac{L \times 1 \times P}{24 \times 60 \times 60 \times 100}$$

$$\frac{(Q_D)_A}{(Q_D)_B} = \frac{L_A \times P_A}{L_B \times P_B}$$

$$\frac{P_A}{P_B} = \left(\frac{L_B}{L_A} \right) \left\{ \frac{(Q_D)_A}{(Q_D)_B} \right\} = 1.5 \times 2.5 = 3.75$$

16.15. ECONOMICS OF CANAL LINING

At times, the choice of canal lining is to be done on financial considerations besides the technical feasibility. For determining its economic viability, an analysis of benefits accrued from canal lining has to be worked out in terms of money. The benefits accrued from lining should be greater than extra cost to be incurred on lining.

Calculation of benefit

The major benefit then can be readily assessed in terms of money is from the saving of seepage water which would have been lost from unlined channel. This water when supplied to farmers will yield revenue.

Let q cumecs be the water saved in the lined reach and R_1 rupees be the cost of water per cumec. Then the total saving = $q \cdot R_1$ rupees.

The other benefit is through a considerable reduction in the maintenance cost of a lined channel, compared to that of an unlined channel. Let p be the percentage saving in annual maintenance cost which is rupees R_2 for unlined channel. Then the saving in annual maintenance cost = $p R_2$ rupees.

Thus total benefit (B) per year = $(q R_1 + p R_2)$ rupees. Saving in maintenance cost is usually taken to be around 40% so that $p = 0.4$

Besides the above benefits, there will be a considerable indirect benefits accrued from the prevention of water logging caused due to excess seepage, but as these are difficult to assess in terms of money, the minimum annual benefit (B) is considered as $B = (q R_1 + p R_2)$

Annual cost of extra expenditure on lining

Let the extra expenditure on canal lining be C rupees. If P_t is the total perimeter of lining, L is length of lining and c is the cost of lining in Rupees per m^2 , $C = c P_t \cdot L$. The extra expenditure is to be calculated by subtracting the saving in land cost, cost of drainage works, cost of bridges and earth work etc. from the total cost of lining.

Let the life of lining be N years and i be the percentage rate of interest expressed as fraction per year. Then the capital recovery factor or annual cost of extra expenditure on lining is equal to :

$$= C \times i (1+i)^N \{ (1+i)^N - 1 \}^{-1} \quad \dots(16.9)$$

Therefore benefit cost ratio (B.C.R.) is given by

$$\text{B.C.R.} = \frac{[q R_1 + p R_2] \{ (1+i)^N - 1 \}}{C \times i (1+i)^N} = \frac{B \{ (1+i)^N - 1 \}}{C i (1+i)^N} \quad \dots(16.10)$$

Note : If rate of interest is 8% per annum, $i = 0.08$.

For the project justification, the benefit cost ratio should be greater than 1. See Example 16.4 for illustration.

16.16. DRAINAGE AND PRESSURE RELEASE ARRANGEMENTS BEHIND CANAL LINING

The thickness of canal lining is usually very small, and hence the lining can not withstand any water pressure from the back of the lining. The water pressure or hydrostatic pressure may be caused by seeping rainwater in the backfill, even when the water table in the area remains below the canal bed. Continuous rain water infiltration behind the lining will make the backfill saturated. In the case of drawdown in the canal, the water in the backfill gives rise to build up of hydrostatic pressure by the provision of drainage and pressure release arrangements. This can be done by the following methods:

- Provision of pressure release valves embedded in drainage relief pockets
- Provision of weep holes in the bed and sides of the lined canal.
- Provision of longitudinal and transverse drains below lining.

1. Provision of pressure release valves

A pressure relief/release valve is a device like a check valve which allows the water from the subgrade to flow into the canal but prevents entry of canal water into the subgrade. Thus, it is a one way valve or a flap valve opening upwards into the canal. These valves open out as soon as the differential pressure becomes about 10 cm. The diameter of the valve varies from 50 mm (used on side slopes) to 150 mm (used generally in the bed). These valves are made of abrasion resistant material, generally of cast iron or heavy duty plastic. Non-metallic valves give better performance than metallic valves.

Pressure release valves are provided in small drainage relief pockets of size 60 cm $\times$ 60 cm filled with graded filter, as shown in Fig. 16.8 (a). The graded filter should be designed in such a way that there is no flow of bed soil through the valve. The drainage relief pockets are cubical pockets provided at isolated

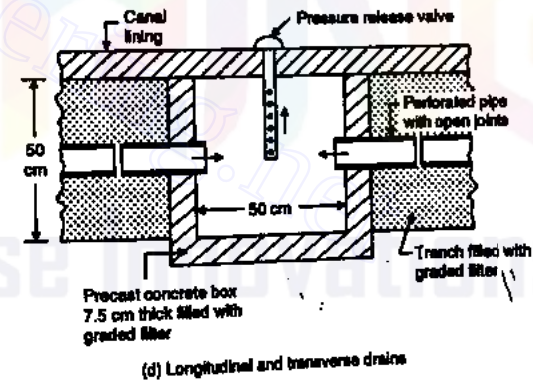
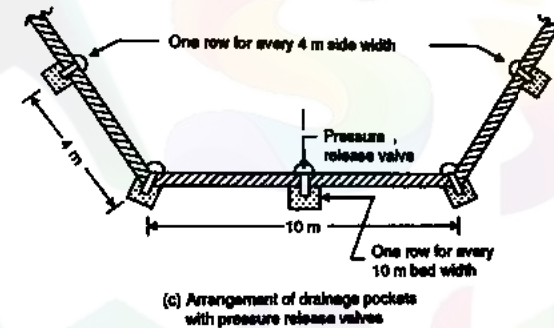
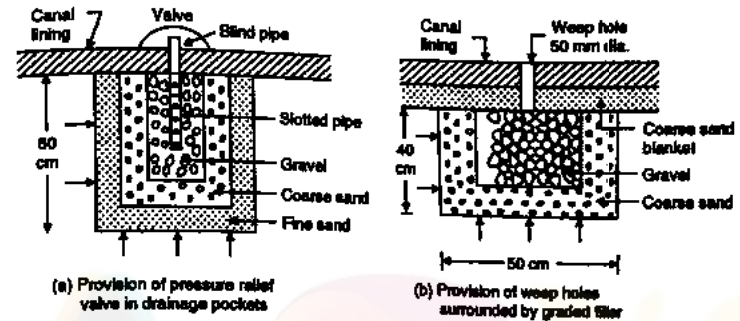


FIG. 16.8. PRESSURE RELIEF ARRANGEMENTS IN LINED CANAL.

locations in the bed as well as sides of the lined canal, below the canal lining, at suitable spacing (Fig. 16.8 c). These are provided at 15 to 20 m intervals, along rows in such a way that at least one row is provided in the bed for every 10 m bed width, and one row on the sides for every 4 m side width.

2. Provision of weep holes

Weep holes are small holes (about 50 mm dia.) made in the lining in the bed as well as sides. The bottom of the weep hole open out in small pocket etc. containing inverted graded filter consisting of two or three materials (Fig. 16.8 b). These holes are suitably spaced in the bed and sides of lined canal.

3. Provision of longitudinal and transverse drains

A longitudinal drain consists of a continuous perforated open jointed pipe encased in a graded filter, having outlets in a masonry or precast concrete box of 50 cm × 50 cm size filled with graded filter, as shown in Fig. 16.8 (d). The box has an outlet into the lined canal through a pressure relief valve. Transverse drains, having cross-section similar to that of a longitudinal drain, are provided at a suitable interval in the bed as well as sides.

16.17. SALT PROBLEMS IN IRRIGATED SOILS

The gradual accumulation of salts on the ground or in the root zone of crop leads to saline and alkaline conditions. This results in loss of permeability, toxicity and anaerobic conditions. Plants fail to derive their nutrients at the required rate owing to the high soil moisture tension. This causes unhealthy growth of plants resulting in loss of yield. The irrigated soil may become saline or alkaline because of the following reasons :

1. Rise in water table

A rise in water table with a high evaporation results in build up of dissolved salts. High evaporation and transpiration rates with poor drainage of soil is the major cause of salinity of soil.

2. Climatic conditions

In the region of heavy rainfall, leaching of soil takes place due to heavy infiltration. In arid and semi-arid climates, rainfall is small hence the leaching is limited and the soil is likely to become saline and alkaline.

3. Quality of irrigation water used

If the quantum of dissolved salts in irrigation water exceeds a certain limit, the excess salts accumulate in the root zone and create salt problem. Sodium ions and chlorides are very harmful compared to sulphates and bicarbonates.

4. Nature of soil and under-irrigation

Heavy soils like clays have poor permeability due to which leaching is extremely slow and hence, are more prone to salinity and alkalinity compared to sandy soils. Brackish water can be used for sandy soils (light soils) if enough water can be provided to satisfy irrigation and leaching requirements. Under-irrigation by sprinkler may result in salinity due to non availability of leaching water.

5. Sea water intrusion

In coastal areas, sea water intrudes below the ground water. If ground water is extracted from below the interface, saline water comes out which can make the land saline or alkaline if used as a source of irrigation.

Leaching Requirement

When soil or the irrigation water is saline, the accumulated salt should be leached out by irrigating it with more water. The fraction of irrigation water that must be leached through the root zone to keep the salinity of soil below a specific limit is termed as *Leaching Requirement* L_R . It is usually expressed as a ratio :

$$L_R = \frac{\text{Depth of drainage water } D_d \text{ per unit area}}{\text{Depth of irrigation water } D_i \text{ per unit area}}$$

From the consideration of salt balance

$$\frac{D_d}{D_i} = \frac{C_i}{C_d} = \frac{E_{ci}}{E_{cd}} \text{ and hence } L_R = \frac{D_d}{D_i} = \frac{C_i}{C_d} = \frac{E_{ci}}{E_{cd}} \quad \dots(16.11)$$

where C_i and C_d are the salt concentration in irrigation and drainage water respectively and E_{ci} , E_{cd} are corresponding electrical conductivity.

The depth of irrigation water D_i to be applied consists of (i) consumptive requirement $D_c (= C_u)$ and (ii) Leaching requirement D_d .

$$D_i = D_c + D_d = D_c + L_R D_i$$

$$\text{Hence } D_i = \frac{D_c}{1 - L_R} = \frac{D_c \cdot E_{cd}}{E_{cd} - E_{ci}} \quad \dots(16.12)$$

The electrical conductivity of drainage water (E_{cd}) may be assumed on the basis of permissible salt tolerance limit of the crop to be grown. Generally, E_{cd} is assumed to be equal to $2E_{cs}$, where E_{cs} is the electrical conductivity of saturated soil extract (i.e. water solution extracted from the soil at its saturation percentage). Hence Eq. 16.11 reduces to

$$L_R = \frac{E_{ci}}{2E_{cs}} \quad \dots(16.11a)$$

Leaching water can be added with irrigation water or can be applied separately. When the intake of soil is low, i.e. for heavy soil, a better practice is to apply small quantities more frequently. Leaching before irrigation season is often more beneficial. After leaching, the deeper layers are at field capacity and, hence, seeding can be done without additional water needed for germination.

Reclamation of salt affected land

The reclamation of salt affected land is costly and should be resorted to only if it is very necessary. Various measures adopted for reclamation depends upon the reasons causing the salinity or alkalinity. Some of these measures are discussed below.

(i) Anti water logging measures

Surface drainage, sub-surface drainage, pumping and recycling, controlling of seepage loss etc. are some of the methods that has already been discussed earlier. As water logging conditions reduce, the salt affected land also improves and becomes culturable.

(ii) *Use of proper quality of water for irrigation*

The quality of water fit for irrigation has been discussed in Chapter 3. If the salinity or alkalinity of land has been caused due to the continued use of poor quality water, its replacement with proper quality of water improves the land condition.

(iii) *Leaching of soil salts*

In order to regain the salt affected land, the salt has to be leached out by supplying additional quantity of water. Leaching water can be added along with irrigation water where there is no limit on supply.

If the intake rate of soil is not sufficient to absorb extra dose, it is advisable to irrigate more frequently with smaller depths in each application rate.

If limited supply is available during the crop seasons, leaching should be practiced during off season periods usually by flooding several times. The drying of beds between consecutive leaching applications stimulates root growth, aids aggregation and permits formation of surface cracks which helps in better infiltration during subsequent flooding.

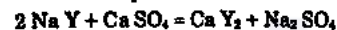
For reclamation of land covered with encrusted salt layers, alternate flooding and surface drainage prior to leaching can remove appreciable quantities of soluble salts and increase efficiency of leaching carried out subsequently. Leaching is good for saline soils. On alkaline soils, the use of certain chemicals with subsequent leaching is more useful.

(iv) *Choice of crops*

Proper choice of salt tolerant crops, crop rotation etc. increases fertility and reclamation of salt affected land. Various salt tolerant crops are : barley, sugar beet tobacco, turnips, mustard, cotton, sugarcane and some varieties of grass. Research scientists have also evolved salt tolerant varieties of rice, wheat, bajra etc. The acidic nature of cell saps in the roots of these crops neutralize the alkalinity of the soil. The movement of roots through the soil increases its permeability. This results in aerobic conditions and nitrification of soil owing to the enhanced bacterial activity.

(v) *Chemical Methods*

The method is based on the principle that the addition of some salts or acids can neutralize harmful salts. Sulphur, sulphuric acid, gypsum, iron sulphate and lime stone are some of the common chemicals used for leaching operations. Use of Gypsum ($\text{CaSO}_4 \cdot 2\text{H}_2\text{O}$) is popular, because of its low cost and cheap availability. Chemical reaction which takes place in this process is



where Y represents the exchangeable constituent. Owing to the replacements of Na ion by Ca ion, soil regains its normal character making it permeable, fibrous, aerated and fit for agriculture. The quantity depends upon the quantity of exchangeable sodium. It is generally assessed that 4.1 tonnes of gypsum/hectare of land for 33 cm depth of soil is needed for reclamation. Powdered gypsum is thoroughly mixed with soil and the field levelled and flooded with water so that exchange between Ca & Na takes place and the dissolved Na ion can be leached down by percolating water.

If soil condition is good and irrigation water is saline, gypsum is not added to the soil; instead gypsum solution is added to irrigation water before it is applied to the field. Addition of chemical fertilizers, usually nitrogen and organic matter along with chemical treatment is necessary for early reclamation and better fertility.

Example 16.9. An unlined channel in alluvial soil has a seepage loss of 2.5 cumecs/million sq.m of wetted perimeter. The channel has a wetted perimeter of 25 m and has maintenance cost of Rs. 1 per sq.m of wetted perimeter. There is a huge scarcity of water in the area and as such the canal is to be lined with cement concrete lining 12 mm thick so as to reduce seepage loss to 0.02 cumecs/million sq.m of wetted perimeter. The lined channel will have a wetted perimeter of 20 m. The extra cost of lining works out to be Rs. 100/- per sq.m. If the average annual revenue per cumec of water is Rs. 25 lakhs and percentage reduction in annual maintenance cost is 40%, decide whether it is economically feasible to provide canal lining. Assume the life of canal lining as 50 years and the interest rate is 6 per cent per annum.

Solution

For the purpose of calculations assume 1 km length of channel.

(a) Annual benefit

(i) **Seepage.** Area of wetted perimeter for unlined channel in 1 km
 $= 25 \times 1000 = 25000 \text{ sq. m}$

$$\text{Seepage loss} = 25000 \times 2.5 \times 10^{-6} = 0.0625 \text{ cumecs}$$

Area of wetted perimeter for lined channel in 1 km $= 20 \times 1000 = 20,000 \text{ sq. m}$

$$\text{Seepage loss} = 20000 \times 0.02 \times 10^{-6} = 0.0004 \text{ cumecs}$$

So, saving in water loss $= 0.0625 - 0.0004 = 0.0621 \text{ cumec}$.

$$\begin{aligned} \text{Annual revenue saved} &= \text{Rs. } 0.0621 \times 25 \times 10^5 \\ &= \text{Rs. } 155250. \end{aligned}$$

(ii) **Maintenance.** Annual maintenance cost of unlined channel.
 $= 1 \times 25000 = \text{Rs. } 25000$

$$\text{Saving in maintenance cost} = 0.4 \times 25000 = \text{Rs. } 10000$$

$$\text{Total annual benefit} = \text{Rs. } 155250 + 10000 = \text{Rs. } 165250.$$

(b) Annual cost

Area of lining per km $= 20.0 \times 1000 = 20000 \text{ sq.m}$

$$\text{Cost of lining } C = 100 \times 20000 = 2000000$$

Interest rate = 6% per annum. Hence $i = 0.06$. Also, $N = 50$.

Annual cost of lining or capital recovery factor

$$= \left[\frac{C \times i (1+i)^N}{(1+i)^N - 1} \right] = \frac{2000000 \times (0.06 \times (1.06)^{50})}{(1.06)^{50} - 1}$$

$$= \text{Rs. } 126888$$

$$\text{Benefit cost ratio} = \frac{165250}{126888} = 1.302$$

As this is more than 1, the provision of canal lining is economically justifiable.

Example 16.10. Determine the leaching requirement when the electrical conductivity of drainage water from soil was found as 20 m mho/cm at 20% reduction in the yield of a crop. Take $E_{cl} = 1.5$ m mho/cm. What will be the required depth of water to be applied if the consumptive use is 55.5 mm?

Solution

$$L_x = \frac{E_{cd}}{E_{cl}} \times 100\% = \frac{1.5}{20} \times 100 = 7.5\%$$

$$D_i = \frac{D_c}{(1 - L_x)} = \frac{55.5}{1 - 0.075} = 60 \text{ mm}$$

Example 16.11. The saturated extract of a soil is 11 m mho/cm at 25% reduction in the yield of a crop. If the electrical conductivity of irrigation water is 1.4 m mho/cm, estimate the leaching requirement. What will be the required depth of water to be applied to the field if the consumptive use requirements of the crop is 85 mm? Assume suitable value of electrical conductivity of leaching water.

Solution : Given : $E_{cl} = 1.4$ m mho/cm and $E_{cs} = 11$ m mho/cm

Let us assume electrical conductivity of leaching water (E_{cd}) as twice the electrical conductivity of saturated extract of soil, i.e. $E_{cd} = 2 E_{cs}$. Hence from Eq. 16.11 (a)

$$L_x = \frac{E_{cd}}{E_{cl}} = \frac{E_{cd}}{2 E_{cs}} = \frac{1.4}{2 \times 11} \times 100 = 6.4\%$$

$$\text{Now } L_x = \frac{D_d}{D_i} = \frac{D_i - D_c}{D_i} = \frac{D_i - 85}{D_i} \times 100\%$$

$$\text{Hence } \frac{D_i - 85}{D_i} \times 100 = 6.4$$

$$\therefore 6.4 D_i = 100 D_i - 8500$$

$$\text{From which } D_i = 90.8 \text{ mm}$$

16.18. EXAMPLES FROM COMPETITIVE EXAMINATIONS

Example 16.12. A trapezoidal channel with side slopes 1.5 horizontal : 1 vertical is required to carry 15 m³/s of flow with a bed slope of 1 in 4000. If the channel is lined, the Manning's coefficient n will be 0.014 and it will be 0.028 if the channel is unlined. Calculate the average boundary shear stress if a hydraulically efficient lined channel is adopted. What percentage of earth work is saved in a lined section relative to an unlined section when hydraulically efficient section is used in both cases? The free board can be assumed to be 0.75 m in both cases and the lining can be assumed to be upto the top of the section. (Civil Services Exam. 1991)

Solution

A hydraulically efficient channel is the one which has minimum perimeter for a given area of cross-section (or for a given depth D). Let B be the bed width.

For side slope of 1.5(H):1(V),

$$\text{Length of sloping side} = (1.5^2 + 1^2)^{1/2} D \triangleq 1.8 D$$

WATERLOGGING AND CANAL LINING

$$\therefore P = B + 2(1.8 D) = B + 3.6 D \quad \dots(i)$$

$$\text{Also, } A = (B + 1.5 D) D \quad \dots(ii)$$

$$\text{or } B = \frac{A}{D} - 1.5 D$$

$$\begin{aligned} \text{Hence } P &= \left(\frac{A}{D} - 1.5 D \right) + 3.6 D \\ &= \frac{A}{D} + 2.1 D \quad \dots(iii) \end{aligned}$$

For minimum perimeter for a given area, $\frac{dP}{dD} = 0$.

$$\therefore \frac{dP}{dD} = A(-1 D^{-2}) + 2.1 = 0$$

$$\text{or } \frac{A}{D^2} = 2.1 \text{ which given } A = 2.1 D^2 \quad \dots(iv)$$

$$\text{Hence from (iii)} \quad P = \frac{2.1 D^2}{D} + 2.1 D = 4.2 D$$

$$R = \frac{A}{P} = \frac{2.1 D^2}{4.2 D} = \frac{D}{2} \quad \dots(v)$$

$$\text{and } B = \frac{A}{D} - 1.5 D = \frac{2.1 D^2}{D} - 1.5 D = 0.6 D \quad \dots(vi)$$

Hence for a most efficient section with 1.5(H):1(V) side slopes, we have

$$A = 2.1 D^2, R = D/2 \text{ and } P = 4.2 D$$

For lined channel : $n = 0.014$

$$\text{Now } Q = \frac{1}{n} A R^{2/3} S^{1/2}$$

$$\therefore 15 = \frac{1}{0.014} (2.1 D^2) \left(\frac{D}{2} \right)^{2/3} \left(\frac{1}{4000} \right)^{1/2}$$

$$\text{or } D^{4/3} = 10.0396 \text{ from which } D = 2.375 \text{ m}$$

$$B = 0.6 D = 0.6 \times 2.375 = 1.425 \text{ m}$$

$$\text{and } R = D/2 = 2.375/2 = 1.188 \text{ m}$$

Average boundary shear stress is given by Eq. 14.32

$$\tau = \omega R S = 9.81 \times 1.188 \times \frac{1}{4000} = 2.912 \times 10^{-3} \text{ kN/m}^2 = 2.912 \text{ N/m}^2$$

Taking a F.B. of 0.75 m, the total depth of cutting $D = 2.375 + 0.75 = 3.125 \text{ m}$

$$\therefore A_1 = (B + 1.5 D) D = (1.425 + 1.5 \times 3.125) 3.125 = 19.1 \text{ m}^2$$

For unlined channel : $n = 0.028$

$$\therefore 15 = \frac{1}{0.028} (2.1 D^2) \left(\frac{D}{2} \right)^{2/3} \left(\frac{1}{4000} \right)^{1/2} \text{ from which } D \triangleq 3.08 \text{ m}$$

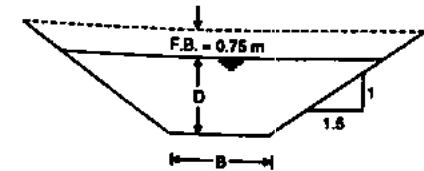


FIG. 16.9

IRRIGATION AND WATER POWER ENGINEERING

$$B = 0.6 \times 3.08 = 1.848 \text{ m}$$

$$R = 3.08/2 = 1.54 \text{ m}$$

$$\tau = 9.81 \times 1.54 \times 1/4000 = 3.777 \times 10^{-3} \text{ kN/m}^2 = 3.777 \text{ N/m}^2$$

$$y = D + 0.75 = 3.08 + 0.75 = 3.83 \text{ m}$$

$$A_2 = (1.848 + 1.5 \times 3.83) 3.83 = 29.08 \text{ m}^2$$

Hence percentage saving in earthwork done due to lining

$$= \frac{A_2 - A_1}{A_2} = \frac{29.08 - 19.1}{29.08} \times 100 = 34.32\%$$

Example 16.13. Design a lined canal to carry $100 \text{ m}^3/\text{s}$ on a slope of 1 in 2500. The maximum permissible velocity is 2 m/s , $n = 0.013$ in Manning's formula and the side is $1.25 H : 1.0 V$. (Engg. Services Exam. 1996)

Solution : Given $V = 2 \text{ m/s}$, $n = 0.013$.

The canal section is shown in Fig. 16.10 from which

$$A = D \times \frac{1}{2} [B + (B + 2.5 D)] = D (B + 1.25 D) \quad \dots(1)$$

$$P = B + 2 \sqrt{(1.25 D)^2 + D^2} = B + 3.2 D \quad \dots(2)$$

But required $A = Q/V = 100/2 = 50 \text{ m}^2$. Hence from (1), we get

$$D (B + 1.25 D) = 50 \quad \dots(3)$$

From Manning's formula

$$V = \frac{1}{n} R^{2/3} S^{1/2}$$

$$\therefore 2 = \frac{1}{0.013} R^{2/3} \left(\frac{1}{2500} \right)^{1/2}$$

$$R^{2/3} = 1.3, \text{ from which}$$

$$R = (1.3)^{3/2} = 1.482$$

$$\therefore P = A/R = \frac{50}{1.482} = 33.73 \text{ m}$$

Hence from (2), $B + 3.2 D = P = 33.73 \text{ m}$

Substituting the value of B from (4) into (3), we get

$$D [33.73 - 3.2 D + 1.25 D] = 50$$

$$\text{or } 1.95 D^2 - 33.73 D + 50 = 0, \text{ from which } D = 1.637 \text{ m}$$

$$\text{Hence from (4)} \quad B = 33.73 - 3.2 \times 1.637 = 28.49 \text{ m}$$

$$\text{Check : } A = 1.637 (28.49 + 1.25 \times 1.637) = 50 \text{ m}^2 \quad \text{Hence OK.}$$

$$\text{Hence } B = 28.49 \pm 28.5 \text{ m and } D = 1.637 \pm 1.64$$

Example 16.14. A lined rectangular channel with $n = 0.015$ is 5 m wide and has a flow depth of 2 m with the bed slope of 1 in 1600. Retaining the rectangular shape of channel section and the same total area of lining, to what maximum extent

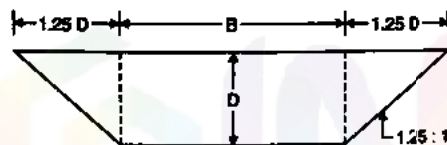


FIG. 16.10.

...

WATERLOGGING AND CANAL LINING

can be discharge be increased without changing the bed slope? Does the flow change from sub-critical to super-critical by this increase in discharge? (Gate 1988)

Solution : Given $B = 5 \text{ m}$, $D = 2 \text{ m}$; $S = 1/1600$

$$\text{Area of lining per m run} = B + 2D = 5 + 2 \times 2 = 9 \text{ m}^2/\text{m}$$

Let B_1 be the new width and D_1 be the new depth.

$$\text{Area of lining per m run} = B_1 + 2D_1$$

Since area of lining is constant, $B_1 + 2D_1 = B + 2D = 9$

Hence

$$B_1 = 9 - 2D_1 \quad \dots(1)$$

Now

$$Q = \frac{1}{n} (B_1 D_1) R_1^{2/3} S^{1/2}$$

For a rectangular channel, $R_1 = D_1$

$$\therefore Q = \frac{1}{n} S^{1/2} B_1 D_1^{4/3} = \frac{1}{n} S^{1/2} (9 - 2D_1) D_1^{4/3}$$

or

$$Q = \frac{S^{1/2}}{n} [9 D_1^{4/3} - 2 D_1^{7/3}]$$

In order to obtain maximum discharge, differentiate Q w.r. to D_1 .

$$\therefore \frac{dQ}{dD_1} = 0 = \frac{S^{1/2}}{n} \left[\frac{9 \times 4}{3} D_1^{1/3} - \frac{2 \times 7}{3} D_1^{4/3} \right]$$

or

$$15 D_1^{1/3} = \frac{16}{3} D_1^{4/3} \text{ or } D_1 = \frac{45}{16} = 2.8125 \text{ m}$$

Hence

$$B_1 = 9 - 2 \times 2.8125 = 3.375 \text{ m}$$

Now, earlier,

$$V = \frac{1}{n} R^{2/3} S^{1/2} = \frac{1}{0.015} (2)^{2/3} \left(\frac{1}{1600} \right)^{1/2} = 2.646 \text{ m/s.}$$

$$\text{Froude No.} = \frac{V}{\sqrt{gD}} = \frac{2.646}{\sqrt{9.81 \times 2}} = 0.597 < 1.$$

Later,

$$V_1 = \frac{1}{0.015} (2.8125)^{2/3} \left(\frac{1}{1600} \right)^{1/2} = 3.32 \text{ m/s}$$

...

$$\text{Froude No.} = \frac{3.32}{\sqrt{9.81 \times 2.8125}} = 0.632 < 1$$

Hence the flow does not change from sub-critical to super-critical.

Example 16.15. A lined irrigation canal with trapezoidal cross-section has 5 m bed width, 2.5 m depth and $1.5 (H) : 1 (V)$. Longitudinal bed slope of the canal is 1 in 1000 and Manning's $n = 0.16$. What is the maximum carrying capacity of the canal? What area of land in hectares the canal can irrigate if the crop has 150 mm field irrigation requirement in a Kor period of 10 days.

(Engg. Services Exam. 2002)

Solution : For $1.5 (H) : 1 (V)$ side slopes, $\cot \theta = 1.5$ and $\theta = 0.588$ radian

Also, $D = 2.5 \text{ m}$ and $B = 5 \text{ m}$

$$A = BD + D^2 (\theta + \cot \theta) = 5 \times 2.5 + 6.25 (0.588 + 1.5) = 25.55 \text{ m}^2$$

$$P = B + 2D (\theta + \cot \theta) = 5 + 2 \times 2.5 (0.588 + 1.5) = 15.44 \text{ m}$$

$$R = A/P = 25.55/15.44 = 1.655 \text{ m}$$

Now,

$$Q = V \times A = \frac{1}{n} AR^{2/3} \sqrt{S}$$

$$= \frac{1}{0.016} \times 25.55 (1.655)^{2/3} \sqrt{1/1000} = 70.85 \text{ m}^3/\text{s}$$

Taking Kor period of 10 days, volume of water supply by the channel

$$= 70.85 \times 10 \times 24 \times 3600 = 61.046 \times 10^6 \text{ m}^3$$

$$\therefore \text{Area to be irrigated} = \frac{\text{Volume}}{\text{depth}} = \frac{61.046 \times 10^6}{0.15} = 407 \times 10^6 \text{ m}^2$$

$$= 40700 \text{ hectares}$$

PROBLEMS

1. What is waterlogging ? What are its ill-effects ?
2. What are causes of waterlogging ?
3. Describe various methods adopted as anti-waterlogging measures.
4. What do you understand by 'land drainage' ?
5. Explain the methods of designing of various drains.
6. What is canal lining ? what are its advantages ?
7. Write the requirements of good lining material.
8. Enumerate various types of linings used for canal.
9. Design the methods of design of a lined canal.
10. Design a trapezoidal shaped concrete lined channel to carry a discharge of 40 cumecs at a slope of 10 cm/km. The side slopes of the channel are $1\frac{1}{2} : 1$. The value of N may be taken as 0.015. Assume a limiting velocity as 1.5 m/sec.
11. Redesign the channel of problem 10 if instead of the limiting velocity, the limiting B/D ratio is to be kept as 8.
12. Redesign the channel of problem 10 if channel is to be triangular shaped.

Canal Outlets

17.1. INTRODUCTION

An outlet is a small structure which admits water from the distributing channel to a water course or field channel. Thus, an outlet is a sort of head regulator for the field channel delivering water to the irrigation fields. The responsibility of maintenance of the distributing channel and the whole canal network lies with Government, while that of the field channel lies with the farmer. The outlet is the connecting medium for the two.

Since the efficiency of a canal network mostly depends upon functioning of the outlets and other regulation works, a canal outlet should fulfill the following requirements:

1. The outlet should be strong, with no moving parts liable to be damaged or requiring periodic attention and maintenance.
2. It should be so designed that the farmer cannot tamper with its functioning and any interference from him should be easily detectable.
3. Since a large number of outlets are fixed on a distributing channel, the most essential requirement is that it should be cheap.
4. The design should be simple so that it can be constructed or fabricated by local masons or technicians.
5. It should be possible for the outlet to work efficiently with a small working head.
6. The outlet should draw its fair share of silt carried by the distributing channel.
7. From the farmer's point of view, the outlet should give a fairly constant discharge. However, from the canal regulation point of view, the outlet should draw proportionately more or less discharge with the varying supply in the distribution channel.

17.2. TYPES OF OUTLETS

Outlets may be classified under the following three heads :

1. Non-modular Outlet
2. Semi-module or Flexible Module
3. Rigid Module.

Non-modular Outlet : A non-modular outlet is the one in which the discharge depends upon the difference in level between the water levels in the distributing channel and the water course. The discharge through such an outlet varies in wide limits with the fluctuations of the water levels in the distributing and the field channels. The common examples under this category are : submerged pipe outlet, masonry sluice and orifices, and wooden shoots.

Semi-module or Flexible outlet. A flexible outlet or semi-module is the one in which the discharge is affected by the fluctuations in the water level of the distributing channel while the fluctuations in the water levels of the field channel do not have any effect on its discharge. The various outlets in common use that fall under this category are pipe outlet, Kennedy's gauge outlet, Crump's open flume outlet and pipe-cum open flume outlet.

Rigid Module. A rigid module is the one which maintains constant discharge, within limits, irrespective of the fluctuations in water levels in the distributing channel and/or field channel. The most common outlet that falls under this category is the Gibb's rigid module.

17.3. IMPORTANT DEFINITIONS

To understand the criteria for judging the behaviour and function of outlets, the following definitions are useful :

1. **Flexibility :** It is ratio of rate of change of discharge of an outlet to the rate of change of the discharge of the distributing channel.

$$\text{Thus, } F = \frac{dq}{q} \bigg/ \frac{dQ}{Q} \quad \dots(17.1)$$

where F = Flexibility

q = Discharge through the outlet

Q = Discharge of the distributing channel.

Now, for the field channel.

$$q = kH^m$$

where k = constant, m = outlet index

and H = head acting on the outlet.

$$\begin{aligned} \therefore dq &= m k H^{m-1} dH \\ \therefore \frac{dq}{q} &= \frac{m k H^{m-1} dH}{k H^m} = m \frac{dH}{H} \end{aligned} \quad \dots(1)$$

Similarly, for the parent channel.

$$Q = CD^n$$

where C = constant and n = canal index

D = depth of water in the canal.

$$\begin{aligned} \therefore dQ &= n CD^{n-1} dD \\ \therefore \frac{dQ}{Q} &= \frac{n CD^{n-1} dD}{CD^n} = n \frac{dD}{D} \end{aligned} \quad \dots(2)$$

CANAL OUTLETS

Dividing (1) and (2), we get

$$F = \frac{\frac{dq}{q}}{\frac{dQ}{Q}} = \frac{m \frac{dH}{H}}{n \frac{dD}{D}} = \frac{m}{n} \frac{D}{H} \frac{dH}{dD}$$

Since any change in the water depth results in an equal change in the head causing flow, we have $dH = dD$. Thus the expression for flexibility becomes :

$$F = \frac{m}{n} \frac{D}{H} \quad \dots(17.2)$$

2. Proportionality

A *Proportional outlet* is the one on which the flexibility (F) is equal to unity. Thus, in a proportional outlet, the rate of change of its discharge is equal to the rate of change of the discharge of the distributing channel. For proportionality, putting $F = 1$ in Equation 17.2, we get the relation.

$$\frac{H}{D} = \frac{m}{n} \quad \dots(17.3)$$

The ratio H/D is known as the *setting*. In a proportional outlet therefore, setting is equal to the ratio of the outlet and the canal indices.

From the point of view of proportionality, an outlet is classified into three types:

- Proportional outlet
- Hyper-proportional outlet
- Sub-proportional outlet

These will be explained both with respect to their *flexibility* as well as *setting*.

3. Setting

Setting is the ratio of the depth of the sill or the crest level of the module below the full supply level of the distributing channel to the full supply depth of the distributing channel.

$$\text{Thus, setting} = \frac{H}{D}$$

In the case of a proportionate outlet we have the setting equal to the ratio of outlet index to the channel index, i.e.

$$\text{Setting} = \frac{H}{D} = \frac{m}{n}$$

For a channel of trapezoidal shape, $Q \propto D^{5/3}$, i.e. $n = 5/3$

For an orifice type outlet, $q \propto H^{1/2}$, i.e. $m = 1/2$

Hence, for such a combination, we get

$$\text{Setting} = \frac{H}{D} = \frac{m}{n} = \frac{1/2}{5/3} = 0.3$$

Hence, for the pipe outlet to be proportional, the outlet is set at 0.3 times the depth below the water surface.

4. Hyper proportional outlet

A hyper proportional outlet is the one in which the flexibility is greater than one, i.e. the discharge in the outlet changes by a larger percentage than the percentage change in the discharge of the distributing channel. Thus, for a hyper proportional outlet

$$F > 1$$

$$\frac{m}{n} \frac{D}{H} > 1$$

$$\text{or } \frac{H}{D} < \frac{m}{n} \quad \dots(17.4)$$

Eq. 17.4 shows that the numerical value of setting (H/D) is less than the ratio m/n or in other words, an outlet is hyper proportional if the setting is higher than required for proportionality.

5. Sub-proportional outlet

A sub proportional outlet is the one in which the flexibility is lesser than one, i.e. the discharge in the outlet changes by a smaller percentage than the percentage change in the discharge of the distributing channel. Thus, for a sub-proportional outlet:

$$F < 1$$

$$\frac{m}{n} \frac{D}{H} < 1$$

$$\text{or } \frac{H}{D} > \frac{m}{n} \quad \dots(17.5)$$

Eq. 17.5 shows that the numerical value of setting H/D is greater than ratio m/n , or in other words, an outlet is sub-proportional if the setting is lower than required for proportionality.

6. Sensitivity

It is defined as the ratio of the rate of change of discharge of an outlet to the rate of change in the level of the distributing surface, referred to the normal depth of the channel. Thus,

$$S = \frac{dq/q}{dG/D} \quad \dots(17.6)$$

where S = Sensitivity of the outlet
 q = Discharge through the outlet
 dq = Change in the discharge of the outlet
 G = Gauge reading, so set that $G=0$ when $q=0$
 D = Depth of water in the distributing channel.

$$dG = dD.$$

$$S = \frac{dq/q}{dD/D} \quad \dots(1)$$

$$\text{But } F = \frac{dq/q}{dQ/Q}, \text{ where } \frac{dQ}{Q} = n \frac{dD}{D}$$

CANAL OUTLETS

$$F = \frac{dq}{q} / n \frac{dD}{D} \quad \dots(2)$$

Comparing (1) and (2), we get

$$S = nF \quad \dots(17.7)$$

It is evident from Eq. 17.7 that the sensitivity of a rigid module is zero.

7. Efficiency

Efficiency is the measure of a conservation of head by an outlet. It is defined as the ratio of the head recovered to the head put in. Less is the working head required for the functioning of the outlet, more will be its efficiency.

8. Drowning ratio

It is ratio between the depths of water level over crest on the downstream and upstream of the module.

9. Minimum modular head

The minimum modular head or the *minimum modular loss* is the minimum loss of head or the difference between the upstream and downstream water levels which is absolutely necessary to be maintained to enable the module to pass its design discharge.

10. Modular limits and modular range

The *modular limits* of an outlet are the upper and the lower limits of any one or more factors beyond which an outlet is incapable of acting as a module or semi-module. *Modular range* is the range between the modular limits. It is the range of various factors which a module or semi-module works as designed.

17.4. NON-MODULAR OUTLETS : PIPE OUTLET

The most common types of non-modular outlets are : submerged pipe outlet, masonry sluice and orifices, and wooden shoots. Fig. 17.1 shows a pipe outlet. The pipes vary from 10 to 30 cm in diameter, and are frequently laid on a light concrete foundation to prevent uneven settlement and consequent leakage. They are generally fixed horizontally at right angles to the direction of flow.

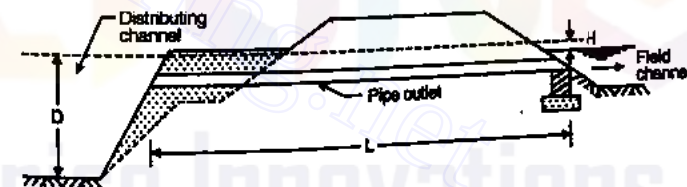


FIG. 17.1. PIPE OUTLET

The loss of head, H through the outlet is given by
 $H = (\text{Entry loss}) + (\text{Frictional loss}) + (\text{Velocity head at the exit})$

$$H = 0.5 \frac{v^2}{2g} + \frac{4fLv^2}{2gd} + \frac{v^2}{2g} \quad \dots(17.8a)$$

IRRIGATION AND WATER POWER ENGINEERING

788

or

$$H = \frac{V^2}{2g} \left[1.5 + \frac{4fL}{d} \right] \quad \dots(17.8)$$

From which

$$V = \sqrt{2gH} \left(\frac{d}{1.5d + 4fL} \right)^{1/2}$$

The discharge q through the pipe outlet is given by

$$q = A \times V = A \sqrt{2gH} \left(\frac{d}{1.5d + 4fL} \right)^{1/2} \quad \dots(17.9)$$

or

$$q = CA \sqrt{2gH}$$

$$\text{where } C = \text{Coefficient of discharge} = \left(\frac{d}{1.5d + 4fL} \right)^{1/2} \quad \dots(17.9 a)$$

 L = Length of the pipe (in m) d = Diameter of the pipe (in m) A = Area of cross-section of the pipe (in m^2)

If, however, the diameter of the pipe is expressed in cm, the coefficient of discharge is given by

$$C = \frac{1}{20} \left[\frac{d}{\left(\frac{1.5d}{400f} + L \right) f} \right]^{1/2} \quad \dots(17.9 b)$$

Indian Standard (IS : 7986-1976) has given the above expression for the coefficient of discharge, which depends on friction factor, length of the pipe and size of the pipe outlet.

In all the above expressions, f is the coefficient of fluid friction for pipes. Its value would be 0.005 and 0.01 for clean and slightly encrusted iron pipes respectively. For earthen ware pipes its value can be taken as 0.0075.For example, if $d = 20$ cm, $L = 10$ m and $f = 0.005$ we get

$$C = \frac{1}{20} \left[\frac{20}{\left(\frac{1.5 \times 20}{400 \times 0.005} + 10 \right) 0.005} \right]^{1/2} = 0.632$$

Similarly, if $d = 25$ cm, $L = 5$ m and $f = 0.005$, we get

$$C = \frac{1}{20} \left[\frac{25}{\left(\frac{1.5 \times 25}{400 \times 0.005} + 5 \right) 0.005} \right]^{1/2} = 0.725$$

The value of the coefficient of discharge varies from 0.61 to 0.76. Its average value, determined at CWPRS, Poona is 0.73.

It is a common practice to place the pipe at the bed of the distributing channel to enable the outlet to draw fair share of silt charge. In U.P., pipes are generally laid horizontally at about 22 cm below the water surface of the distributing channel.

CANAL OUTLETS

787

The inlet and exit ends of the pipe should be fixed in masonry to prevent tampering. Fig. 17.2 shows the typical layout of a pipe outlet as recommended by Indian Standard, IS : 7986-1976.

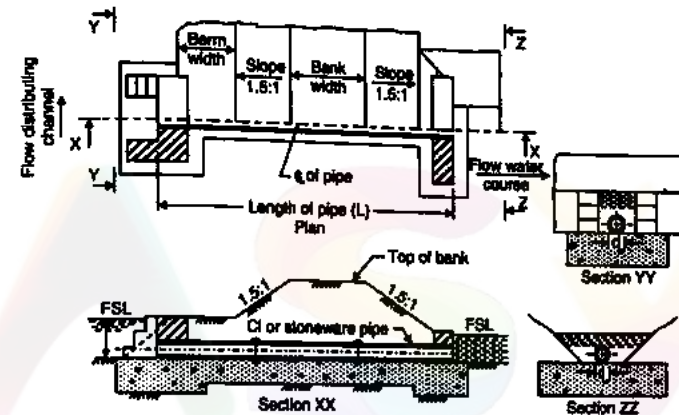


FIG. 17.2. TYPICAL LAYOUT OF PIPE OUTLET

It is apparent from Fig. 17.1 that the discharge of the outlet can be easily increased by digging the head of the water course and thereby lowering the water level in it. This will increase the head H and therefore, the discharge will be increased. On the other hand, the farmers having higher land will get lesser discharge, since the head H is decreased due to heading up of the water. As the discharging capacity of the outlet is quite uncertain, proper and equitable distribution of water is very difficult. Non-modular outlets, however, can work with very small available heads. Due to considerable disturbance at the entrance to the pipe outlet, they usually take their due share of silt. In order to increase the silt conductivity further, the upstream end of the pipe is depressed, making the pipe to slope upwards as shown in Fig. 17.3. So long as

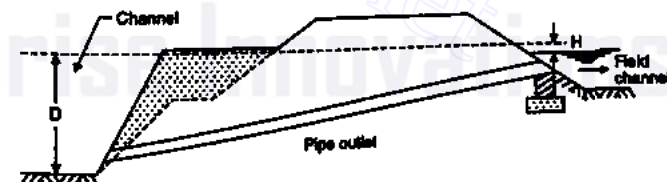


FIG. 17.3. TILTED PIPE OUTLET

the head H remains the same, this arrangement will not change the discharge, while the silt conductivity is increased.

Example 17.1. A submerged pipe outlet has the following data :

F.S.L. of distributory	= 100.00 m
F.S.L. of water course	= 99.90 m
Length of the pipe	= 9 m
Diameter of the pipe	= 20 cm
Coefficient of friction	= 0.005

Find the discharge through the outlet.

Solution

Given : $d = 20$ cm, $L = 9$ m; $f = 0.005$

Working head $H = 100.00 - 99.90 = 0.1$ m

$$C = \frac{1}{20} \left[\frac{d}{\left(\frac{1.5d}{400f} + L \right) f} \right]^{1/2} = \frac{1}{20} \left[\frac{20}{\left(\frac{1.5 \times 20}{400 \times 0.005} + 9 \right) 0.005} \right]^{1/2} = 0.645$$

Area of cross-section of pipe $A = \frac{\pi}{4} d^2 = \frac{\pi}{4} (0.2)^2 = 0.0314 \text{ m}^2$

Hence $q = CA \sqrt{2gH} = 0.645 \times 0.0314 \sqrt{2 \times 9.81 \times 0.1} = 0.0284 \text{ cumec}$

Example 17.2. Design a submerged pipe outlet for the following data :

Discharge through outlet	= 0.04 cumecs
F.S.L. of distributing canal	= 100.00 m
F.S.L. of water course	= 99.90 m
Full supply depth of distributing canal	= 1.1 m

Assume an average value of coefficient of discharge as 0.7.

Solution : Given $q = 0.04$ cumec and $C = 0.7$

Available head, $H = 100.00 - 99.90 = 0.10$ m

$$q = CA \sqrt{2gH}$$

$$0.04 = 0.7 A \sqrt{2 \times 9.81 \times 0.1}$$

From which $A = 0.0408 \text{ m}^2 = \frac{\pi}{4} d^2$

$$\therefore \text{Pipe dia. } d = \sqrt{\frac{4 \times 0.0408}{\pi}} = 0.228 = 22.8 \text{ cm}$$

Hence use a pipe of 25 cm diameter.

Example 17.3. Redesign the submerged pipe outlet for the data of Example 17.2 if old C.I. pipe is used for which $f = 0.01$

Solution : Given : $q = 0.04$ cumecs and $f = 0.01$

CANAL OUTLETS

The coefficient of discharge depends upon f , L and d . Since d is not given let us assume $C = 0.7$, to start with. Then, as found in Example 17.2, we get $d = 0.228 \text{ m} = 22.8 \text{ cm}$. Using this diameter for further trial, we get

$$C = \frac{1}{20} \left[\frac{d}{\left(\frac{1.5d}{400f} + L \right) f} \right]^{1/2} = \frac{1}{20} \left[\frac{22.8}{\left(\frac{1.5 \times 22.8}{400 \times 0.01} + 9 \right) 0.01} \right]^{1/2} \approx 0.57$$

In the next trial, taking $C = 0.57$, we get

$$0.04 = 0.57 A \sqrt{2 \times 9.81 \times 0.1}, \text{ from which } A = 0.05 \text{ m}^2$$

$$\therefore d = \sqrt{\frac{4 \times 0.05}{\pi}} \approx 0.25 \text{ m}$$

Using this diameter and substituting in Eq. 17.9 (b), we get $C = 0.583$

$$\therefore 0.04 = 0.583 A \sqrt{2 \times 9.81 \times 0.1} \text{ from which } A = 0.049 \text{ m}^2$$

$$\therefore d = \sqrt{\frac{4 \times 0.049}{\pi}} \approx 0.25 \text{ m} = 25 \text{ cm}$$

Hence provide a pipe of 25 cm dia.

17.5. SEMI-MODULE OR FLEXIBLE OUTLETS

In the case of a flexible outlet, the discharge is affected by the change in the water level of the distributing channel but not with the change in the water level of the field channel. The following are the common types of flexible outlets :

- Pipe outlet discharging freely in the atmosphere,
- Kennedy's gauge outlet.
- Open flume outlet
- Orifice semi-modules.

(1) Pipe outlet discharging freely

If the pipe outlet is so set that it discharges freely in the atmosphere, the discharge through it becomes independent of the water level in the field channel, and hence it acts as semi-module. The discharge cannot be increased by the cultivator by digging the water course. Such outlets, therefore, worked well. Later, however, the farmers invented a method of increasing the discharge by constructing a ramp in the water-course thereby heading up the water to the top level of the pipe and drowning it.

For the outlet to be proportional, $F = 1$. Hence $\frac{H}{D} = \frac{m}{n}$. Taking $n = \frac{5}{3}$ and $m = \frac{1}{2}$, the setting is equal to 0.3. Usually a pipe outlet is set lower than this and is sub-proportional. The efficiency of the pipe outlet is high and its silt conduction is good.

(2) Kennedy's gauge outlet :

This was the earliest type of semi module, invented by R.G. Kennedy, Chief Engineer, Irrigation, Punjab, sometime in 1906. This outlet is made of cast iron and consists of three main parts (Fig. 17.4):

- An orifice with bellmouth entry.
- A long expanding delivery pipe.
- An intervening vertical air column above the throat.

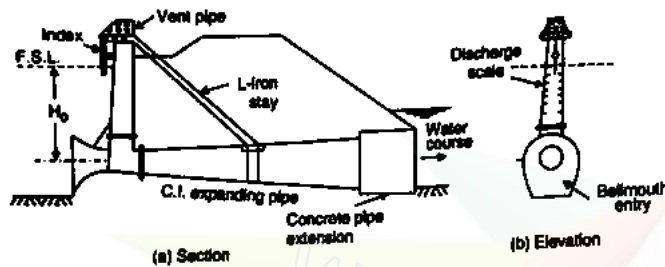


FIG. 17.4. KENNEDY'S GAUGE OUTLET

Water from the distributing channel enters the outlet through bell mouthed approach. From the narrowest diameter at the throat, the jet shoots across the air in the base of the vertical air vent pipe. The base of the air vent pipe is conical and allows free circulation of air around the jet. Thus the jet is exactly in the same condition as if it were discharging into open air, making the discharge independent of the water-level in the water course. The jet is then received by a cast iron expanding pipe, about 3 m long. At the end of the C.I. pipe, cement concrete pipe extension is usually added and the water is then discharged in the field channel.

The discharge through this semi-module is given by

$$q = Ca \sqrt{2g H_0} \quad \dots(17.10)$$

where a = Area of cross-sections of the pipe at the throat

C = Coefficient of discharge which may be as high as 0.97.

It has been found by experiments that the losses of head by friction, eddies, etc., are about $0.22 H_0$ and that the available working head or the minimum modular head should not be less than this value.

This module has however, been superseded because its discharge could very easily be increased by closing air vents of the vent pipe. After the air vents are closed, the jet will suck the air of the chamber and its pressure drops below the atmosphere. This increases the discharge. Some safeguards like enclosing the air holes in a chamber and fixing the pipe to angle irons embedded in ground to prevent tampering were provided. This worked for sometime, but the farmer, later, found a method of plugging the vent pipe through the bell mouth entry. Due to this difficulty, and also due to high cost of its manufacture, this outlet is no longer in use.

(3) Open flume outlets :

An open flume outlet is a sort of weir with constricted throat at expanding flume on the downstream. Due to this, hyper-critical velocity is ensured and hydraulic jump is formed. The formation the hydraulic jump makes the discharge through the outlet independent of the water-level in the water course, making it a semi-module.

There are two types of open flume outlets :

(i) Crump's open flume outlet.

(ii) Punjab open flume outlet.

Crump's open flume outlet

Fig. 17.5 shows the plan and longitudinal section of Crump's open flume outlet. The upstream face is set back by a distance x equal to the width of the opening of the module. This enables the outlet to take a fair share of silt. This throat is constricted to ensure hyper-critical velocity. The length of the throat is kept equal to $2.5 H$.

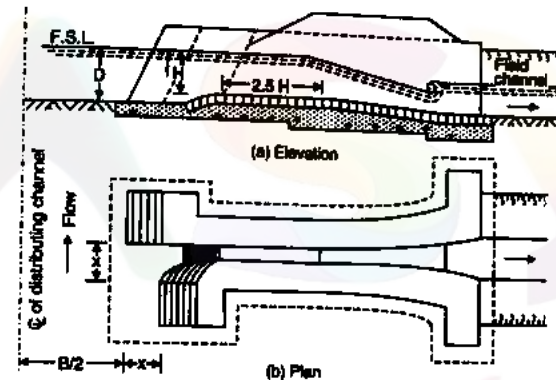


FIG. 17.5. CRUMP'S OPEN FLUME OUTLET

The discharge is given by the expression.

$$q = CB_t H^{3/2} \quad \dots(17.11)$$

where B_t = Throat width (metres)

C = Coefficient of discharge, the theoretical value of which is equal to 1.71.

q = Discharge through the outlet, in cumecs

H = Head over the crest, in metres.

Due to various losses, the actual value of C is less than 1.71 and depends upon the width of the throat, as indicated below :

Throat width	Value of C
0.06 to 0.09 m	1.60
0.09 to 0.12 m	1.64
over 0.12 m	1.68

Now $q = C B_1 H^{3/2}$
 $0.06 = 1.8 \times 0.97 H^{3/2}$
 or $H = 0.66 \text{ m}$

From which
 Now minimum modular head required for open flume outlets
 $= 0.2 H = 0.2 \times 0.66 = 0.132 \text{ m} = 13.2 \text{ cm}$

This is less than the available working head of 15 cm. Hence the design is satisfactory.

$$\text{Setting of outlet} = \frac{H}{D} = \frac{0.66}{0.85} = 0.776 < 0.9$$

Hence the out let will work as hyper-proportional outlet.

17.4. RIGID MODULE

The three common types of rigid modules, having no moving parts are :

1. Gibb's rigid module.
2. Khanna's rigid module.
3. Foote module.

Gibb's module is described below.

Gibb's module

The outlet was designed by A.S. Gibb's, formerly Executive Engineer, Punjab Irrigation. Fig. 17.9 shows the plan and the section of the module. The essential feature of the outlet is an eddy chamber, semi-circular in plan, round which water flows (giving rise to a free vortex flow. Water enters through an inlet pipe having bell mouth entry and is directed to the eddy chamber through a 180° rising pipe in which free vortex flow is developed. The characteristic feature of the free vortex flow is that the product of the velocity and radius is constant for all filaments (i.e. $V \times r = \text{constant}$). Thus the water at the outer circumference of the chamber has greater radius and hence lesser velocity, resulting in rise of water level there. Thus the water surface in the eddy chamber slopes down towards the inner circumference. A series of baffles are suspended from the roof of the eddy chamber, with their lower edges sloping at the required height above the sill of the module. If the head causing flow increases, water banks up at the outer circumference of the eddy chamber and impinges against the baffles imparting an upward, rotational, direction of flow to the water, which spins round in the compartment between two successive baffles and finally drops on the oncoming stream of water, thus dissipating excess energy. This keeps discharge constant for a wide range of variation in the head. The number of baffles coming into action depends upon the variations in the head causing flow. The angle of eddy chamber varies from a semi-circle to $1\frac{1}{2}$ complete turns depending upon the discharge and the range of working required for the module.

Gibb give the following formula for the discharge through the outlet :

$$q = r_s \sqrt{2g} (d_i + h_o)^{1.5} \left[\frac{m^3 - 1}{m^3} \log_e m + \frac{1}{m} \log_e m - \frac{m^2 - 1}{2m^2} \right] \quad \dots(17.14)$$

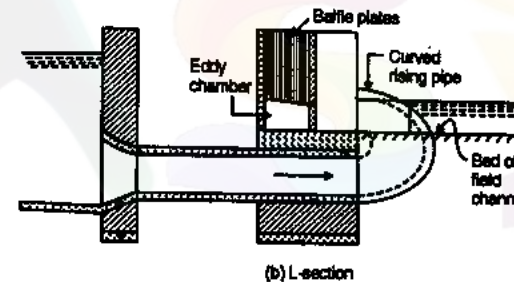
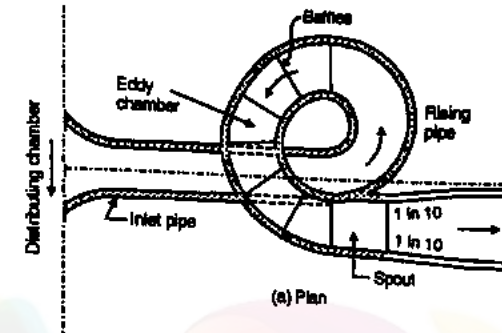


FIG. 17.9. GIBB'S MODULE

where r_o = radius of outer semi-circle
 r_i = radius of inner semi-circle.

$$m = \frac{r_o}{r_i}$$

d_i = depth of water at inner circumference.

h_o = head at outer circumference.

Gibb's formula is valid only for his standard design in which $m = 2$ and $\frac{h_o}{D} = \frac{1}{7}$.

PROBLEMS

1. What is an outlet ? Write down the requirements that an outlet should fulfill.
2. What do you understand by flexibility of an outlet ? Derive an expression for the same.
3. Define proportionality of an outlet. Distinguish between a hyper-proportional outlet and a sub-proportional outlet. Find out expressions for the setting of both the types.

IRRIGATION AND WATER POWER ENGINEERING

706

4. Define sensitivity of an outlet. Find the relation between sensitivity and flexibility of an outlet.
5. Distinguish clearly between non-modular and semi-modular outlets. Give Examples.
6. What do you know about Kennedy's gauge outlet and explain the use of it.
7. Give the formulae used for calculating discharge of any three types of outlets used in irrigation canals. What is the normal setting for each type ?
8. Describe with sketch the working of Crump's open flume outlet.
9. What do you understand by a rigid module ? Describe the working of Gibb's module.

18

Canal Regulation Works

18.1. INTRODUCTION

Any structure constructed to regulate the discharge, full supply level or velocity in a canal is known as a *regulation work*. Such a structure is necessary for the efficient working and safety of an irrigation channel. The various regulation works may be categorised as under :

1. Canal fall.	2. Head regulator or head sluice.
3. Cross regulator.	4. Canal escape.
5. Canal outlet.	

(A) CANAL FALLS

18.2. NECESSITY AND LOCATION OF FALLS

A *fall* is an irrigation structure constructed across a canal to lower down its water level and destroy the surplus energy liberated from the falling water which may otherwise scour the bed and banks of the canal.

We have seen in Chapter 14 that the canal requires a certain slope, depending upon the discharge, to overcome the frictional losses. This slope may vary from 1 in 4000 for a discharge of about 1.5 cumecs to about 1 in 8000 for a discharge of 3000 cumecs. This slope is, therefore, quite flat in comparison to the available ground slope of an average value of 5 to 20 cm per kilometre length (*i.e.*, 1 in 200 to 1 in 50). Thus the ground slope in nature is always very much steeper than the design bed slope of irrigation canal, based on the silt theories. If an irrigation canal, taking off from its head, is in cutting, it will soon meet with condition when it will be entirely in embankment.

It has been stated earlier in Chapter 15 that if the canal is in embankment, the cost of construction and maintenance is *very high* and at the same time the percolation and seepage losses are excessive. Also, there is always a danger of the adjacent area being flooded if some cut or breach takes place in the canal banks. Hence, the canal should never be in high embankment. However, the divergence between the gentle bed slope of canal and the steep ground slope throws the canal in embankment after a certain distance though it started in cutting at its head. To overcome this difficulty, *falls* are introduced at appropriate places, and the water surface of the canal is lowered. Arrangements are made to dissipate the excess energy liberated from the falling water.

(789)

The location of a fall is decided from the following considerations :

1. For the canal which does not irrigate the area directly, the fall should be located from the considerations of economy in cost of excavation of the channel with regard to balancing depth and the cost of the falls itself.

2. For a canal irrigating the area directly, a fall may be provided at a location where the F.S.L. outstrips the ground level, but before the bed of the canal comes into filling. After the drop, the F.S.L. of the canal may be below the ground level for $\frac{1}{2}$ to $\frac{1}{4}$ kilometer.

3. The location of the fall may also be decided from the consideration of the possibility of combining it with a regulator or a bridge or any other masonry work.

4. A relative economy of providing large number of small falls v/s small number of big falls should be worked out. The provision of small number of big falls results in unbalanced earth-work, but there is always some saving in the cost of the fall structure.

18.3. DEVELOPMENT OF FALLS

The ancient people always tried to avoid falls by aligning canals along zig-zag route in order to increase the length of the canal and thus dissipate the excess energy head in friction. The Eastern Yamuna Canal constructed by Mughal Emperors had no falls, and the canal, followed a sinuous path. The falls were first constructed by the British in India in the nineteenth century. The development of falls, since then, took place gradually. Among the earlier type of falls are : Ogee falls, rapids and stepped falls. Later, notch falls, vertical falls and glacis type falls were developed.

1. Ogee fall

The Ogee fall was first constructed by Sir Proby Cautley on the Ganga Canal (Fig. 18.1). This type of fall has gradual convex and concave curves, with an aim to provide a smooth transition and to reduce disturbance and impact. This preserved the energy (without dissipating it). Due to this, the Ogee fall had the following defects :

- (i) There was considerable draw down effect on the u/s resulting in bed erosion.
- (ii) Due to smooth transition, the kinetic energy was preserved till sufficient depth was scoured out below the fall to ensure the formation of the hydraulic jump.

2. Rapid fall

Fig. 18.2 shows a *rapid fall*. Such falls were provided on Western Yamuna Canal and were designed by Lieut. R.F. Croften. Such a fall consists of a glacis sloping at 1 vertical to 10 to 20 horizontal. The long glacis assured the formation of hydraulic jump. The gentle slope admitted timber traffic. Hence, the fall worked admirably. However, there was very high cost of construction.

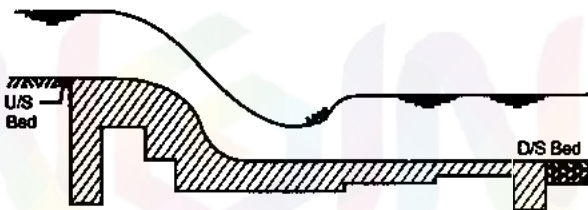


FIG. 18.1. OEGEE FALL.

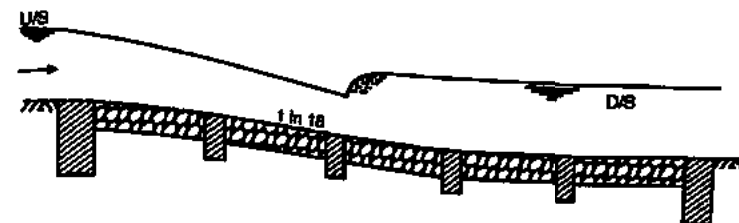


FIG. 18.2. RAPID FALL.

3. **Stepped fall** : Stepped fall was a next development of the rapid fall. One such type was provided at the tail of main canal escape of Sarda Canal. The cost of this fall was also too high.

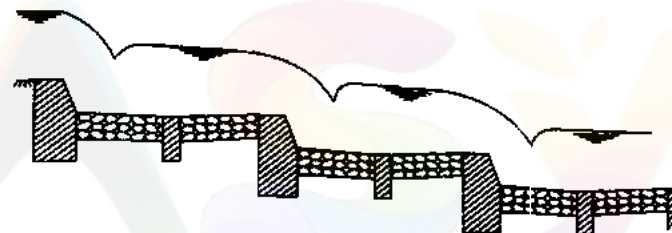


FIG. 18.3. STEPPED FALL.

4. Notch fall

Soon after the development of stepped fall, the efficiency of vertical impact on the floor for energy dissipation came to be recognised. The vertical fall came in the field along with the cistern. However, with greater discharges, vertical fall gave trouble. Hence, these were superseded for a time by the notch fall. The trapezoidal notch fall was first designed by Ried in 1864.

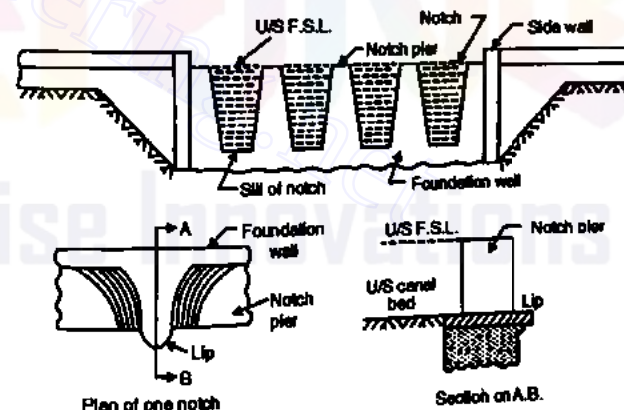


FIG. 18.4. TRAPEZOIDAL NOTCH FALL.

The fall consists of one or more trapezoidal notches in a high crested wall. A flat circular lip projects downstream of each notch to disperse water. The notches were designed to maintain the normal water depth in the u/s channel at any two discharge values. The depth discharge relation was thus maintained with close approximation. As the channel approached the fall, there were neither drawdown nor heading up of water. The trapezoidal fall was very successful and was adopted in India for many years. It was also copied all over the World where it is still in use. There was one serious defect in these falls that they could not be used as regulators in addition.

5. Vertical drop fall

In the vertical drop fall, the nappe impinges clear into the water cushion below. In the earlier types of vertical falls, the dimensions of cistern were put in arbitrarily in light of experience of the designers. Another device in the form of grid was usually used in the cistern intercepting the dropping jet of water. The grid consisted of baulks of timber horizontal or inclined and spaced some centimeters apart. These were later abandoned because the timber grid got clogged and rotted and had to be replaced frequently.

The Sarda type fall developed on the Sarda Canal Project in U.P and C.D.O. type fall developed in Punjab are some of the recent types of vertical drop falls. In these falls, the high velocity jet enters the deep pool of water in the cistern and the dissipation of energy is affected by the turbulent diffusion.

6. Glacis type fall

The efficiency of the hydraulic jump as a very potent means of destroying the energy of canal falls was brought out clearly by the research work of the Miami Conservancy Board of Engineers in a publication by Sharman M. Woodward on "Theory of Hydraulic Jump and Back Water Curves" (1918). The glacis type of fall utilised the standing wave phenomenon for dissipation of energy. The glacis fall may be (i) straight glacis type, or (ii) parabolic glacis type, commonly known as the *Montague type*. The straight glacis fall may be with baffle platform and baffle wall. In such a case, the formation of jump takes place on the baffle platform. This type was first developed by Inglis and is called *Inglis fall*.

Meter and non-meter falls

Meter falls are those which also measure the discharge of the canal. The non-meter falls do not measure the discharge. For a fall to act as a meter, it must have broad weir type crest so that the discharge co-efficient is constant under variable head. Generally glacis type fall is suitable as a meter. The vertical drop fall is not suitable as a meter due to the formation of partial vacuum under the nappe.

Contracted and full width falls

A fall may either be constructed of the full channel width or it may be contracted. The contracted falls, the later type, are also known as the flumed falls.

18.4. CLASSIFICATION OF FALLS

Falls may be divided into four principal classes with reference to the approach condition :

Class I. Falls designed to maintain the depth discharge relation

In this class, the normal depth discharge curve of the channel is maintained, and there is neither drawdown nor heading up of water as the channel approaches the fall. The fall under this category are : (i) trapezoidal notch falls, and (ii) low-crested rectangular notch falls. In the former the depth discharge relationship is not so well maintained as the latter. The second type of fall is liable to cause silting if the channel is run at a lower level than designed, for long periods. However, if the periods of low water is small, the silt usually gets washed off.

Class II. Falls designed to maintain a fixed supply level in the channel above the work

Such type of falls maintain a nearly fixed water surface level upstream of the work, and necessary under the following circumstances.

- (i) When the hydraulic power station is combined with the fall.
- (ii) When a subsidiary canal takes off some distance above the work.

The principal types of falls under this class are : (i) the Siphon falls (Siphon Spillways) and (ii) high crested weir falls. The falls under this category suffer from the defect of silting caused in the channel above the work. The crested falls are generally constructed without fluming, and hence the discharge intensity per unit length of the crest is low, making the energy dissipation simpler. It can also be used as a meter.

Class III. Falls designed to admit the variations of the surface level above the work at the will of the operator

The falls under the category are designed to admit the variations of water level on the upstream of the work according to the requirements. They also serve as regulators. The regulation is done with the help of (i) sluice gates, (ii) horizontal stop logs, (iii) vertical needles. In practice, falls of this class are confined exclusively to rectangular notches as the trapezoidal notch cannot be used for this purpose.

Class IV. Miscellaneous Types

Falls under this class are designed from special considerations without any reference to the conditions of approach. The principal types under this class are as follows :

- (i) *Cylinder fall or Well fall* (Fig. 18.5)

In this type of fall, water is thrown into a well over a crest from where it escapes near its bottom. The energy is dissipated in the well in turbulences. They

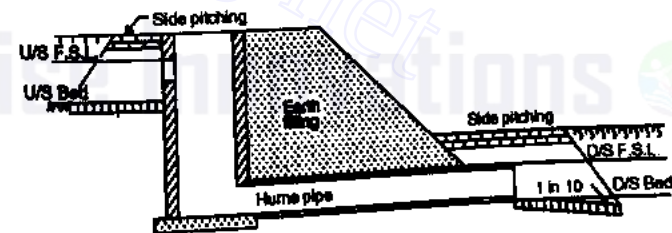


FIG. 18.5. CYLINDER FALL.

are quite suitable and economical for low discharges and high drops, and are used at tail escapes of small channels.

(ii) *Chute or rapids*

They conduct the stream on open inclined trough.

(iii) *Pipe falls*

In this a pipe replaces chute.

18.5. CISTERN DESIGN

The *cistern* or *cistern element* is that portion of the fall down stream of the crest wall where the surplus energy of water leaving the crest is destroyed. The complete cistern element consist of (i) sloping glacis (if any), (ii) the cistern, (iii) roughening devices, and (iv) device for deflecting the high velocity jet.

The object of cistern is three fold : (i) to reduce the intensity of impact of the dropping jet against the downstream floor, (ii) to provide cushion to destroy the energy of the drop, and (iii) to produce reverse flow by providing a suitable end-wall to ensure an impact in the cistern.

The cistern element can be divided into the following four classes :

Class I : Cistern element in which there is impact from a stream of water falling freely under gravity

In this case, the energy is dissipated by means of impact and deflection of velocity, suddenly, from the vertical to the horizontal direction. To protect the floor from the impact of falling water, water cushion is provided by depressing the floor below the impact of falling water, water cushion is provided by depressing the floor below the downstream bed of the channel (Fig. 18.6). Theoretical treatment to workout the required length and depth of cistern has escaped notice so far but a lot of empirical formulae have been developed by various engineers, based on their experience on such works.

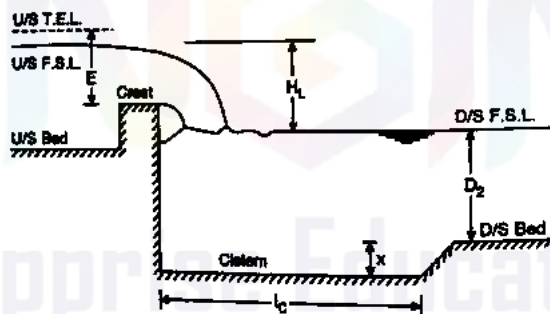


FIG. 18.6. CISTERN.

Following are some of the formulae for cistern design, in metric units :

(i) *Sir Proby Cautley's recommendations*

He put the depth of cistern at one half the height of the fall without any reference to the discharge intensity.

(ii) *Captain Dyas formula*

$$x = 0.82 H_L^{1/2} \cdot D_2^{1/3}$$

...(18.1)

where x = depth of cistern below d/s bed, in metres

H_L = height of drop in metres

D_2 = depth of water downstream in the channel.

(iii) *E.L. Glass formulae*

Glass developed the following formulae for works of Bihar and Orissa :

$$x = 1.85 E^{1/2} H_L^{1/3} - D_2$$

...(18.2)

$$L_c = x + D_2$$

...(18.3)

where $E = u/s$ total energy, above the crest

L_c = length of the cistern

(iv) *Etchevery formulae*

These were developed in America.

$$L_c = 3 \sqrt{H_L E}$$

...(18.4)

$$x = \frac{1}{6} L_c$$

...(18.5)

(v) *Montagu's formulae*

These were developed in Punjab.

$$x = \frac{1}{2} E_2$$

...(18.6)

$$L_c = 4E_2$$

...(18.7)

where E_2 = energy of flow downstream, for the discharge intensity q and the fall H_L .

(vi) *Blench formula*

$$x = 2 D_c + \frac{1}{4} \left(H_L - \frac{3}{8} D_c \right) - D_2$$

...(18.8)

According to him, the fall should not be designed in such a way that H_L is less than $\frac{3}{8} D_c$, where D_c is the critical depth.

(vii) *U.P. Irrigation Research Institute formulae*

$$L_c = 5 \sqrt{E H_L}$$

...(18.9)

$$x = \frac{1}{4} (E H_L)^{2/3}$$

...(18.10)

Class II. Cistern element for impact by a horizontal stream

In this type, the energy is dissipated by the formation of hydraulic jump. The discharge, after passing over the crest, is carried over a sloping glacis. The sloping glacis is given a reverse curvature at its lower end to turn the hypercritical jet to a horizontal direction before it impinges against the subcritical flow of lower channel in the cistern. Hydraulic jump is thus formed.

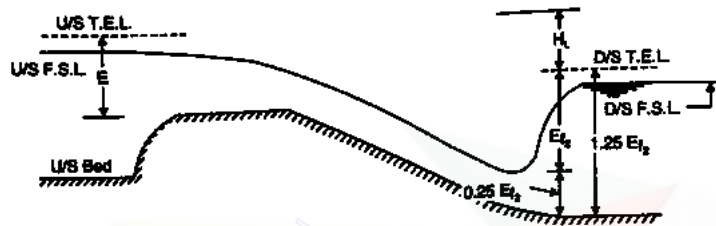


FIG. 18.7. CISTERN.

For a given drop (H_L) in the energy line and the discharge intensity (q), there will be a definite value of downstream specific energy (E_2) and the downstream depth (D_2) required for the jump formation. Theoretically, the bed of the cistern should be provided at the lowest level of the jump formation. However, as an additional safety, the depth of the cistern is increased by 25% of E_2 . Thus the R.L. of cistern is kept to the R.L. of d/s total energy line minus $1.25 E_2$. In case, however, the downstream bed level is lower than the cistern level determined from the above consideration, the cistern should be provided at the d/s bed level. The length of cistern is kept equal to $5 E_2$ for normal soil and $6 E_2$ for sandy soils.

Class III. Cistern for impact by a stream flowing on an inclined glacis

In this case, the glacis is carried straight into cistern without giving any reverse curvature. The hydraulic jump is thus formed on the glacis itself, for the dissipation of energy. However, the falling jet of water has a vertical component while on glacis, and the formation of hydraulic jump on the glacis does not help in dissipating it. Thus the energy dissipation is not perfect, and additional roughening devices have to be provided for the complete destruction of excess energy. However, Montagu and Dr. Bose have derived equations of ideal glacis profile (a curved profile) that imparts maximum horizontal acceleration to the stream at all states of discharge. By imparting maximum horizontal acceleration, the maximum horizontal velocity is attained in any given length of work. Such a design is, therefore, conducive to economy in construction.

Class IV. Cistern without impact

In this type, hydraulic jump is not formed and, therefore energy is dissipated without any impact. The energy dissipation takes place by the provision of *roughening devices*. This case arises in case of falls with large drowning ratio or in low falls where impact is not possible. In such circumstances, the roughening devices are the only means available for energy dissipation.

Roughening Devices

The design of various roughening devices depends upon the experience, and no theoretical treatment is available. Following are some of the roughening devices used on falls :

- (i) Baffle wall, (ii) Friction blocks or arrows,

CANAL REGULATION WORKS

- (iii) Dentated sill,
- (v) Biff wall,

(i) **Baffle wall** : A baffle wall is a sort of low weir constructed at the end of the cistern to serve two purposes :

(a) to head up water to its upstream to such a height that hydraulic jump is formed, and

(b) to withstand the actual impact of the high velocity jet to dissipate the energy.

(ii) **Staggered friction blocks or arrows** : Staggered friction blocks are one of the most useful and simple devices to dissipate the energy. They consist of rectangular blocks of concrete. Their height may be upto $\frac{1}{4}$ th water depth and are 1.5 to 2.0 times the height of the block. The distance between successive lines is equal to twice the height. Arrows are specially shaped friction blocks [Fig. 18.8 (1) and (2)]. Both these are built on d/s floor of the falls below the glacis or cistern with the object to divide the bottom high velocity water laterally. They just serve to reduce the bottom velocity of water leaving the pucca floor of the fall.

Montagu gave the following empirical formula for the determination of the distance upto which roughening of bed is required by the arrows or friction blocks :

$$L = C \frac{D_2^{1/2} H_L^{1/2}}{x}$$

where x = Depth of the cistern below downstream bed, in metres
 D_2 = Depth downstream, in metres
 H_L = Drop at the fall, in metres

- (iv) Deflectors,
- (vi) Cellular or ribbed pitching.

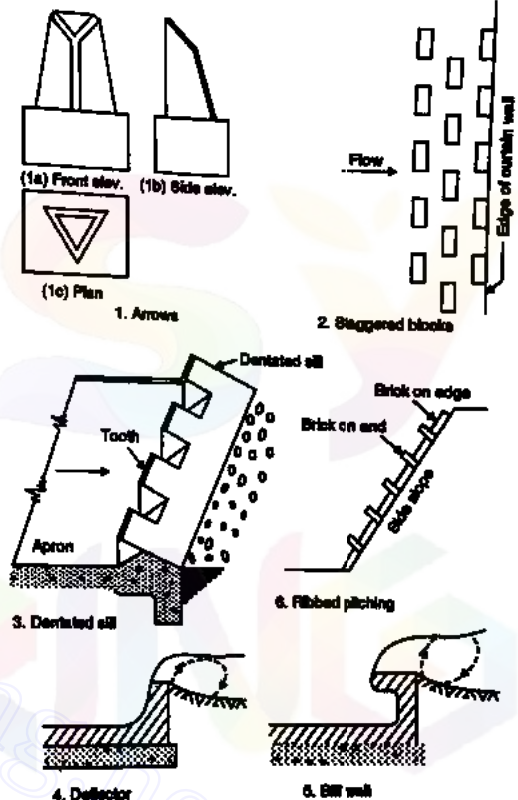


FIG. 18.8. ROUGHENING DEVICES.

- $C =$ a coefficient
 $= 1$ for vertical impact
 $= 2$ for horizontal impact
 $= 4$ to 6 for inclined impact
 $= 8$ for no impact.

(iii) **Dentated Sill** : A dentated sill, shown in Fig. 11.8(3) is provided at the end of cistern if high velocity jet persists to the end of the cistern. The object of the sill is to deflect up the high velocity jet from near the bed and to break it.

(iv) **Deflector** : A deflector [Fig. 18.8(4)] is of uniform height, unlike the dentated sill. Its object is to deflect up the high velocity jet near the bed causing a reverse roller.

(v) **Biff wall** : Fig. 18.8(5) shows a biff wall designed by Colyer in Punjab. It is provided at the end of cistern, causing a deep pool of water behind it in the cistern. Its object is to deflect back the water from the cistern to create super turbulence in it.

(vi) **Cellular or ribbed pitching** : Ribbed pitching is constructed on the sides by putting bricks flat and on edge alternatively, as shown in Fig. 18.8(6). This provides the roughening of the perimeter to destroy surplus energy down-stream of the fall.

18.6. DESIGN OF SARDA TYPE FALL

This type of fall was designed and developed for Sarda Canal System of U.P. In that area, thin veneer of sandy-clay overlies a stratum of pure sand. Hence, the main requirement was to provide a number of falls with small drops, so that the depth of cutting is kept a minimum. This fall has, therefore, been constructed for drops varying from 0.9 to 1.8 metres (3 to 6 ft.) In the earlier designs, the cistern was not depressed below the d/s floor and the d/s wings were not flared. This resulted in the erosion of banks to the d/s of the work. Extensive model experiments were then conducted at the Bahadradab Research Station and some recommendations were made. The design criteria given below are based on these recommendations.

Fig. 18.10 shows the drawings of Sarda fall. The complete design consists of the design of the following component parts :

(1) Crest,	(2) Cistern
(3) Impervious floor,	(4) D/s protection,
(5) U/s approach.	

1. Design of crest

(i) **Length of crest**. The length of the crest is kept equal to the bed width of the canal, and no fluming is done in this type of fall. Sometimes, however, the length of the crest is kept equal to the bed width of canal plus the water depth, to take into account the anticipated increase in discharge at a future date.

(ii) Shape of the crest and discharge formula

Two types of crests are used, as shown in Fig. 18.9. The rectangular crest is used for discharges upto 14 cumecs (500 cusecs) and trapezoidal crest is used for discharges over 14 cumecs.

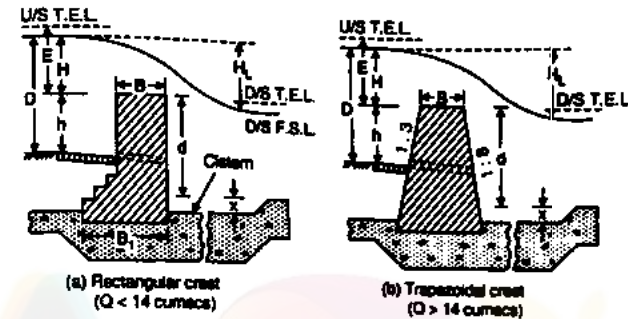


FIG. 18.9. CREST DESIGN.

For the rectangular crest :

Top width of crest is given by $B = 0.55 \sqrt{d}$ metres ... (18.12)

Base width is given by $B_1 = \frac{H+d}{p}$... (18.13)

For masonry crest, p may be taken equal to 2.

Discharge is given by $Q = 1.835 L H^{3/2} \left(\frac{H}{B} \right)^{1/4}$... (18.14)

where

$Q =$ discharge in cumecs

and

$L =$ length of the crest in metres.

For a trapezoidal crest

Top width of crest is given by : $B = 0.55 \sqrt{H+d}$

U/s batter = 1 : 3

D/s batter = 1 : 8

Thus the base width is determined by the batter.

Discharge is given by $Q = 1.99 L H^{3/2} \left(\frac{H}{B} \right)^{1/4}$... (18.16)

(iii) Crest level

From Eq. 18.14 or 18.16, the value of H is known.

$\therefore$ R. L. of crest = u/s F. S. L. - H

Height of crest above bed = $h = D - H$.

For falls over 1.5 m, the stability of the crest wall should be tested by actual analysis.

Brick pitching is laid on a slope of 10 : 1 of for 2 to 4 metres length u/s of the crest, and drain holes are provided in the crest at this level to drain out the u/s bed during the closure of the canal.

2. Design of cistern

The length and the depression of the cistern are given by the following equations:

$$\dots(18.17)$$

$$L_c = 5 (E H_L)^{1/2} \dots(18.18)$$

and

$$x = \frac{1}{4} (E H_L)^{2/3}$$

3. Design of impervious floor

The total length of impervious floor is determined either by Bligh's theory (for small works) or by Khosla's theory. The maximum seepage head occurs when there is water on the u/s side upto the top of the crest and there is no flow to the d/s side. As indicated in Fig. 18.9, the maximum seepage head is equal to d . Out of the total impervious floor length, a minimum length (L_d), to be provided to the d/s of the crest, is given by the following expression :

$$L_d = 2 (D + 1.2) + H_L \text{ metres} \dots(18.19)$$

The balance of the impervious floor length may be provided under and u/s of the crest.

The thickness of the impervious floor is determined on the principle discussed in chapter 12. However, a minimum thickness of 0.3 m to 0.4 m is provided for the floor to the u/s of the crest. For the floor to the d/s of the crest, the actual thickness depends upon the uplift pressures subject to a minimum of 0.3 to 0.4 m for small falls and 0.4 to 0.6 for large falls. The cistern and the d/s impervious floor should have a top lining of brick on edge, in lime or cement mortar, so that floor can be repaired as and when needed. A vertical cutoff of 1 to 1.5 m depth is always provided to the d/s of the impervious floor.

4. D/S protection : The d/s protection consists of (i) bed protection, (ii) side protection, and (iii) d/s wings.

(i) **Bed protection :** The bed protection consists of dry brick pitching about 20 cm thick resting on 10 cm ballast. Table 18.1 gives the length of the pitching and the number of curtain walls (cutoffs) to be provided

TABLE 18.1. DETAILS OF BED PITCHING.

Head over crest (metres)	Total length of pitching on the d/s (metres)	Remarks	Curtain wall Number	Depth (m)
upto 0.3	3.0	Sloping at 1 in 10	1	0.30
0.30 to 0.45	$3.0 + 2 H_L$	Horizontal up to end of masonry wings and then sloping at 1 in 10	1	0.30
0.45 to 0.60	$4.5 + 2 H_L$		1	0.45
0.60 to 0.75	$6.0 + 2 H_L$		1	0.60
0.75 to 0.90	$9.0 + 2 H_L$		1	0.75
0.90 to 1.05	$13.5 + 2 H_L$		2	0.94
1.05 to 1.20	$18.0 + 2 H_L$		2	1.06
1.20 to 1.50	$22.5 + 2 H_L$		3	1.35

(ii) **Side protection :** Side pitching, consisting of one brick on edge, is provided after the warped wings. The side pitching is curtailed at any angle of 45° from the end pitching in plan. Generally, warping of masonry wings is done from vertical to slope of 1 : 1. Hence, the side pitching is warped from a slope of 1 : 1 to $1 \frac{1}{2} : 1$. The pitching supported on a toe wall $1 \frac{1}{2}$ brick thick and of depth equal to half the d/s water depth.

(iii) **D/s wings :** The d/s wings are kept vertical for a length of 5 to 8 times $\sqrt{E H_L}$ from the crest, and are then warped or flared to a slope of 1 : 1 or $1 \frac{1}{2} : 1$. An average splay of 1 in 2.5 to 1 in 4 for attaining the required slope is given to the top of the wings. The wings follow a circular arc, tangential at the starting point of warp, in plan.

The wing walls are designed as earth retaining structures. In the absence of elaborate stability calculations, the width of the wings at any level may be kept equal to $\frac{1}{3}$ rd of the height above that level. For heavy works, actual design calculations may be made.

5. Design of u/s approach : For discharge upto 14 cumecs, the u/s wings may be splayed, straight at an angle of 45° . For greater discharges, the wings are kept segmental with radius equal to 5 to 6 times H , subtending an angle of 60° at the centre, and then are carried straight into the berm. The embedment in the berms or earth banks should be a minimum of 1 m. The foundations of the u/s wings are kept on the u/s impervious floor itself.

DESIGN EXAMPLE OF SARDA TYPE FALL

Example 18.1. Design a Sarda type fall for the following data :

- (i) Full supply discharge $\frac{u/s}{d/s} = 40 \text{ cumecs}$
- (ii) Full supply level $\frac{u/s}{d/s} = \frac{218.30 \text{ m}}{216.80 \text{ m}}$
- (iii) Full supply depth $\frac{u/s}{d/s} = \frac{1.8 \text{ m}}{1.8 \text{ m}}$
- (iv) Bed width $\frac{u/s}{d/s} = \frac{26 \text{ m}}{26 \text{ m}}$
- (v) Bed level $\frac{u/s}{d/s} = \frac{216.50 \text{ m}}{215.00 \text{ m}}$
- (vi) Drop 1.5 m

Design the floor on Bligh's theory taking coefficient of creep = 8. Check the design by Khosla's theory and make changes if necessary. Safe exit gradient may be taken equal as 1/5.

Solution

Step 1. Calculation of H and d

$$L_c = 5 (E H_L)^{1/2} \quad \dots(18.17)$$

$$x = \frac{1}{4} (E H_L)^{2/3} \quad \dots(18.18)$$

and

3. Design of impervious floor

The total length of impervious floor is determined either by Bligh's theory (for small works) or by Khosla's theory. The maximum seepage head occurs when there is water on the u/s side upto the top of the crest and there is no flow to the d/s side. As indicated in Fig. 18.9, the maximum seepage head is equal to d . Out of the total impervious floor length, a minimum length (l_d), to be provided to the d/s of the crest, is given by the following expression :

$$l_d = 2 (D + 1.2) + H_L \text{ metres} \quad \dots(18.19)$$

The balance of the impervious floor length may be provided under and w/s of the crest.

The thickness of the impervious floor is determined on the principle discussed in chapter 12. However, a minimum thickness of 0.3 m to 0.4 m is provided for the floor to the u/s of the crest. For the floor to the d/s of the crest, the actual thickness depends upon the uplift pressures subject to a minimum of 0.3 to 0.4 m for small falls and 0.4 to 0.6 for large falls. The cistern and the d/s impervious floor should have a top lining of brick on edge, in lime or cement mortar, so that floor can be repaired as and when needed. A vertical cutoff of 1 to 1.5 m depth is always provided to the d/s of the impervious floor.

4. D/S protection : The d/s protection consists of (i) bed protection, (ii) side protection, and (iii) d/s wings.

(i) *Bed protection* : The bed protection consists of dry brick pitching about 20 cm thick resting on 10 cm ballast. Table 18.1 gives the length of the pitching and the number of curtain walls (cutoffs) to be provided

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0.30 to 0.45	$3.0 + 2 H_L$		1	0.30
0.45 to 0.60	$4.5 + 2 H_L$		1	0.45
0.60 to 0.75	$6.0 + 2 H_L$		1	0.60
0.75 to 0.90	$9.0 + 2 H_L$		1	0.75
0.90 to 1.05	$13.5 + 2 H_L$		2	0.94
1.05 to 1.20	$18.0 + 2 H_L$		2	1.06
1.20 to 1.50	$22.5 + 2 H_L$		3	1.35

(ii) *Side protection* : Side pitching, consisting of one brick on edge, is provided after the warped wings. The side pitching is curtailed at any angle of 45° from the end pitching in plan. Generally, warping of masonry wings is done from vertical to slope of 1 : 1. Hence, the side pitching is warped from a slope of 1 : 1 to $1\frac{1}{2}$: 1. The pitching supported on a toe wall $1\frac{1}{2}$ brick thick and of depth equal to half the d/s water depth.

(iii) *D/s wings* : The d/s wings are kept vertical for a length of 5 to 8 times $\sqrt{E H_L}$ from the crest, and are then warped or flared to a slope of 1 : 1 or $1\frac{1}{2}$: 1. An average splay of 1 in 2.5 to 1 in 4 for attaining the required slope is given to the top of the wings. The wings follow a circular arc, tangential at the starting point of warp, in plan.

The wing walls are designed as earth retaining structures. In the absence of elaborate stability calculations, the width of the wings at any level may be kept equal to $\frac{1}{3}$ rd of the height above that level. For heavy works, actual design calculations may be made.

5. Design of u/s approach : For discharge upto 14 cumecs, the u/s wings may be splayed, straight at an angle of 45° . For greater discharges, the wings are kept segmental with radius equal to 5 to 6 times H , subtending an angle of 60° at the centre, and then are carried straight into the berm. The embedment in the berms or earth banks should be a minimum of 1 m. The foundations of the u/s wings are kept on the u/s impervious floor itself.

DESIGN EXAMPLE OF SARDA TYPE FALL

Example 18.1. Design a Sarda type fall for the following data :

(i) Full supply discharge : $\frac{u/s}{d/s} = 40 \text{ cumecs}$

(ii) Full supply level : $\frac{u/s}{d/s} = \frac{218.30 \text{ m}}{218.80 \text{ m}}$

(iii) Full supply depth : $\frac{u/s}{d/s} = \frac{1.8 \text{ m}}{1.8 \text{ m}}$

(iv) Bed width : $\frac{u/s}{d/s} = \frac{26 \text{ m}}{26 \text{ m}}$

(v) Bed level : $\frac{u/s}{d/s} = \frac{216.50 \text{ m}}{215.00 \text{ m}}$

(vi) Drop : 1.5 m

Design the floor on Bligh's theory taking coefficient of creep = 8. Check the design by Khosla's theory and make changes if necessary. Safe exit gradient may be taken equal as 1/5.

Solution

Step 1. Calculation of H and d

IRRIGATION AND WATER POWER ENGINEERING

812

Since the discharge is more than 14 cumecs, trapezoidal crest will be adopted.
The discharge is given by

$$Q = 1.98 LH^{3/2} \left(\frac{H}{B} \right)^{1/5} \quad \dots(1)$$

Here, L = width of canal = 26 m
 Q = 40 cumecs.

$$B = 0.55 \sqrt{H+d}$$

But

$$H+d = D + \text{drop in the level}$$

$$H+d = 1.8 + 1.5 = 3.3 \text{ m}$$

$$B = 0.55 \sqrt{H+d} = 0.55 \sqrt{3.3} = 1.0 \text{ m} \quad \dots(2)$$

Substituting the value of B in (1), we get

$$40 = \frac{1.98 \times 26 H^{3/2} \times 1/5}{(1.0)^{1/5}}$$

$$H^{3/2} = \frac{40 \times 1}{1.98 \times 26} = 0.774$$

From which

$$H = (0.774)^{2/3} = 0.86 \text{ m} \quad \dots(4)$$

Substituting this in (2) we get

$$d = 3.3 - H = 3.3 - 0.86 = 2.44 \text{ m}$$

Height of crest above bed = $D - H = 1.8 - 0.86 = 0.94 \text{ m}$

Step 2. Design of crest

A trapezoidal crest will be adopted.

Top width B (found in step 1) = 1 m

d/s batter = 1:3 ; u/s batter = 1:8

Assuming canal side slope 1:1, velocity of approach is given by

$$V_a = \frac{40}{(27 + 1.8) 1.8} = 0.8 \text{ m/sec}$$

$$\text{Velocity head} = \frac{V_a^2}{2g} = \frac{(0.8)^2}{2 \times 9.81} = 0.032 \text{ m}$$

∴ u/s T. E. L. = u/s F. S. L. + velocity head = 218.30 + 0.032 = 218.332 m

R.L. of crest = u/s F. S. L. - H = 218.30 - 0.86 = 217.44 m

Value of $E = \text{u/s T. E. L.} - \text{R.L. of crest} = 218.332 - 217.44 = 0.892 \text{ m}$

Step 3. Design of cistern

Depth of cistern = $x = \frac{1}{4} (E H_L)^{2/3} = \frac{1}{4} (0.892 \times 1.5)^{2/3} = 0.304 \text{ m}$

Length of cistern = $l_c = 5 (E H_L)^{1/2} = 5 (0.892 \times 1.5)^{1/2} = 5.8 \text{ m}$

Keep the length of cistern = 6 m

R. L. of bed of cistern = R. L. of d/s bed - x = 215.00 - 0.304 = 214.696 m.

CANAL REGULATION WORKS

813

Keep the cistern at R.L. 214.69 m.

∴ Final value of x = 215.00 - 214.69 = 0.31 m.

Step 4. Design of impervious floor

Seepage head = $H_s = d = 2.44 \text{ m}$

Bligh's coefficient = 8

∴ Length of impervious floor or Creep length = $C \times H_s = 8 \times 2.44 = 19.5 \text{ m}$

Provide u/s cut-off $d_1 = 1.0 \text{ m}$ and d/s cutoff $d_2 = 1.6 \text{ m}$.

The vertical length of creep = $2(1 + 1.6) = 5.2 \text{ m}$.

∴ Length of horizontal impervious floor = $19.5 - 5.2 = 14.3 \text{ m}$

Provide 15 m length of impervious floor.

Minimum length of impervious floor to the d/s of toe of the crest wall
= $l_d = 2(D + 1.2) + H_L = 2(1.8 + 1.2) + 1.5 = 7.5 \text{ m}$

Provide $l_d = 8 \text{ m}$. The balance of the length = $15 - 8 = 7 \text{ m}$ is to be provided under and u/s of the crest.

Calculation for uplift pressure and thickness

Total creep length = $15 + 2(1 + 1.6) = 20.2 \text{ m}$

(i) The uplift pressure under the u/s floor will be counter-balanced by the weight of water, and hence no thickness is required. However, provide a minimum thickness of 0.4 m.

(ii) For other points, the maximum vertical ordinate between the Bligh's H.G. line and the floor level gives the uplift pressure.

Maximum unbalanced head under the toe of the crest = 1.66 m

This can also be found by calculations as under :

$$\text{Static head} = 2.44 \left(1 - \frac{7 + 2 \times 1.0}{20.2} \right) + x = 2.44(1 - 0.446) + 0.31 = 1.66 \text{ m}$$

$$\therefore \text{Thickness required} = \frac{1.66}{\rho - 1} = \frac{1.66}{2.24 - 1} = 1.34 \text{ m}.$$

Provide 1.40 m thick cement concrete floor overlaid by 0.2 m brick pitching.

(Note : While working out the thickness, the expression has not been multiplied by the safety factor 4/3, since the additional 0.2 m brick pitching will provide the additional safety.)

(iii) Provide a minimum thickness of 0.6 m overlaid by 0.2 m thick brick pitching at the d/s end of the floor.

(iv) The thickness of the floor at intermediate points may be varied as per requirements of uplift pressure.

Step 5. Checking of floor thickness by Khosla's theory

$$\text{Exit gradient} \quad G_e = \frac{1}{\pi \sqrt{\lambda}} \frac{H_s}{d_2}$$

$$\therefore \frac{1}{5} = \frac{1}{\pi \sqrt{\lambda}} \frac{2.44}{1.60}$$

IRRIGATION AND WATER POWER ENGINEERING

Since the discharge is more than 14 cumecs, trapezoidal crest will be adopted.
The discharge is given by

$$Q = 1.99 L H^{3/2} \left(\frac{H}{B} \right)^{1/4} \quad \dots(1)$$

Here, L = width of canal = 28 m
 Q = 40 cumecs.

$$B = 0.55 \sqrt{H+d}$$

But $H+d = D + \text{drop in the level}$

$$H+d = 1.8 + 1.5 = 3.3 \text{ m} \quad \dots(2)$$

$$B = 0.55 \sqrt{H+d} = 0.55 \sqrt{3.3} = 1.0 \text{ m} \quad \dots(3)$$

Substituting the value of B in (1), we get

$$40 = \frac{1.99 \times 28 H^{3/2+1/4}}{(1.0)^{1/4}}$$

$$H^{7/4} = \frac{40 \times 1}{1.99 \times 28} = 0.774$$

From which

$$H = (0.774)^{4/7} = 0.86 \text{ m} \quad \dots(4)$$

Substituting this in (2) we get

$$d = 3.3 - H = 3.3 - 0.86 = 2.44 \text{ m}$$

Height of crest above bed = $D - H = 1.8 - 0.86 = 0.94 \text{ m}$

Step 2. Design of crest

A trapezoidal crest will be adopted.

Top width B (found in step 1) = 1 m

d/s batter = 1:3 ; u/s batter = 1:8

Assuming canal side slope 1:1, velocity of approach is given by

$$V_a = \frac{40}{(27+1.8)1.8} = 0.8 \text{ m/sec}$$

$$\text{Velocity head} = \frac{V_a^2}{2g} = \frac{(0.8)^2}{2 \times 9.81} = 0.032 \text{ m}$$

$$\therefore \text{u/s T.E.L.} = \text{u/s F.S.L.} + \text{velocity head} = 218.30 + 0.032 = 218.332 \text{ m}$$

$$\text{R.L. of crest} = \text{u/s F.S.L.} - H = 218.30 - 0.86 = 217.44 \text{ m}$$

$$\text{Value of } E = \text{u/s T.E.L.} - \text{R.L. of crest} = 218.332 - 217.44 = 0.892 \text{ m}$$

Step 3. Design of cistern

$$\text{Depth of cistern} = x = \frac{1}{4} (E H_1)^{2/3} = \frac{1}{4} (0.892 \times 1.5)^{2/3} = 0.304 \text{ m}$$

$$\text{Length of cistern} = L = 5 (E H_1)^{1/2} = 5 (0.892 \times 1.5)^{1/2} = 5.8 \text{ m}$$

Keep the length of cistern = 8 m

$$\text{R.L. of bed of cistern} = \text{R.L. of d/s bed} - x = 215.00 - 0.304 = 214.696 \text{ m}$$

CANAL REGULATION WORKS

Keep the cistern at R.L. 214.69 m.

$$\therefore \text{Final value of } x = 215.00 - 214.69 = 0.31 \text{ m}$$

Step 4. Design of impervious floor

$$\text{Seepage head} = H_s = d = 2.44 \text{ m}$$

$$\text{Bligh's coefficient} = 8$$

$$\therefore \text{Length of impervious floor or Creep length} = C \times H_s = 8 \times 2.44 = 19.5 \text{ m}$$

$$\text{Provide u/s cut-off } d_1 = 1.0 \text{ m and d/s cutoff } d_2 = 1.6 \text{ m}$$

$$\text{The vertical length of creep} = 2(1+1.6) = 5.2 \text{ m}$$

$$\therefore \text{Length of horizontal impervious floor} = 19.5 - 5.2 = 14.3 \text{ m}$$

Provide 15 m length of impervious floor.

$$\text{Minimum length of impervious floor to the d/s of toe of the crest wall} \\ = L_d = 2(D+1.2) + H_L = 2(1.8+1.2) + 1.5 = 7.5 \text{ m}$$

Provide $L_d = 8 \text{ m}$. The balance of the length = $15 - 8 = 7 \text{ m}$ is to be provided under and u/s of the crest.

Calculation for uplift pressure and thickness

$$\text{Total creep length} = 15 + 2(1+1.6) = 20.2 \text{ m}$$

(i) The uplift pressure under the u/s floor will be counter-balanced by the weight of water, and hence no thickness is required. However, provide a minimum thickness of 0.4 m.

(ii) For other points, the maximum vertical ordinate between the Bligh's H.G. line and the floor level gives the uplift pressure.

$$\text{Maximum unbalanced head under the toe of the crest} = 1.66 \text{ m}$$

This can also be found by calculations as under :

$$\text{Static head} = 2.44 \left(1 - \frac{7+2 \times 1.0}{20.2} \right) + x = 2.44(1 - 0.446) + 0.31 = 1.66 \text{ m}$$

$$\therefore \text{Thickness required} = \frac{1.66}{\rho - 1} = \frac{1.66}{2.24 - 1} = 1.34 \text{ m}$$

Provide 1.40 m thick cement concrete floor overlaid by 0.2 m brick pitching.

(Note : While working out the thickness, the expression has not been multiplied by the safety factor 4/3, since the additional 0.2 m brick pitching will provide the additional safety.)

(iii) Provide a minimum thickness of 0.6 m overlaid by 0.2 m thick brick pitching, at the d/s end of the floor.

(iv) The thickness of the floor at intermediate points may be varied as per requirements of uplift pressure.

Step 5. Checking of floor thickness by Khosla's theory

$$\text{Exit gradient} \quad G_x = \frac{1}{\pi \sqrt{\lambda}} \frac{H_1}{d_2}$$

$$\therefore \quad \frac{1}{5} = \frac{1}{\pi \sqrt{\lambda}} \frac{2.44}{1.60}$$

$$\frac{1}{\pi \sqrt{\lambda}} = \frac{1}{5} \times \frac{1.60}{2.44} = 0.131$$

From Khosla's exit gradient curve; we get

$$\alpha = 10.5 \text{ corresponding to } \frac{1}{\pi \sqrt{\lambda}} = 0.131$$

Alternatively

$$\lambda = \left[\frac{1}{\pi (0.131)} \right]^2 = 5.891$$

$$\lambda = 5.891 = \frac{1 + \sqrt{1 + \alpha^2}}{2}, \text{ from which } \alpha = 10.73$$

$$b = \alpha \cdot d_2 = 10.73 \times 1.6 = 17.17 \text{ m.}$$

Length actually provided by Bligh's theory = 15 m only, which is less than required. Hence, either the length is to be increased by 2.17 m or the depth of cutoff is to be increased.

Let us increase the depth of cutoff d_2 to 1.80 m.

Keep $b = 15$ cm (same as before)

$$\alpha = \frac{b}{d_2} = \frac{15}{1.80} = 8.33$$

From the exit gradient curves, for $\alpha = 8.33$, we get

$$\frac{1}{\pi \sqrt{\lambda}} = 0.145$$

$$G_x = \frac{1}{\pi \sqrt{\lambda}} \frac{H_s}{d_2} = 0.145 \times \frac{2.44}{1.80} = \frac{1}{5.1}$$

Alternatively,

$$\lambda = \frac{1 + \sqrt{1 + \alpha^2}}{2} = \frac{1 + \sqrt{1 + (8.33)^2}}{2} = 4.695$$

$$G_x = \frac{1}{\pi \sqrt{4.695}} \times \frac{2.44}{1.80} = \frac{1}{5.02}$$

Permissible exit gradient = $1/5$. Hence, safe.

Calculation of pressures

(i) U/S cutoff

$$d_1 = 1.0 \text{ m; } b = 15 \text{ m}$$

$$\frac{1}{\alpha} = \frac{d_1}{b} = \frac{1}{15} = 0.0666$$

$$\phi_{c1} = 100 - 24 = 76 \%$$

$$\phi_{D1} = 100 - 17 = 83 \%$$

Assume a thickness of floor = 0.4 m at the w/s

$$\text{Corrected } \phi_{c1} = 76 + \frac{83 - 76}{1} \times 0.4 = 78.8 \%$$

(ii) D/S cutoff wall

$$d_2 = 1.8 \text{ m; } b = 15 \text{ m}$$

$$\frac{1}{\alpha} = \frac{d_2}{b} = \frac{1.8}{15} = 1.12$$

$$\phi_{c2} = 31 \%$$

$$\phi_{D2} = 21.5 \%$$

Assuming a thickness of d/s floor = 0.6 m.

$$\text{Corrected } \phi_{c2} = 31 - \frac{31 - 21.5}{1.8} \times 0.6 = 27.8 \%$$

Thus the pressure on the floor varies (linearly) from 78.8% at the w/s to 27.8% at the d/s.

Calculation of thickness

(i) Provide a minimum thickness of 0.4 m for the w/s floor.

(ii) Thickness at the d/s toe of crest :

$$\% \text{ Pressure} = \left(27.8 + \frac{78.8 - 27.8}{15} \times 8 \right) = 55 \%$$

$$\text{Maximum static head} = 0.55 H_s + x = 0.55 \times 2.44 + 0.31 = 1.67 \text{ m}$$

$$\text{Thickness required} = \frac{1.67}{\rho - 1} = \frac{1.67}{2.24 - 1} = 1.35 \text{ m}$$

Thickness provided by Bligh's theory is 1.40 m.

$\therefore$ Provide 1.40 m thick cement concrete floor overlaid by 020 m thickness brick pitching, for a length of 3 m.

(iii) At 3 m from the d/s toe of crest

$$\% \text{ Pressure} = 27.8 + \frac{78.8 - 27.8}{15} \times 5 = 44.8 \%$$

$$\text{Maximum static head} = 0.448 H_s + x = 0.448 \times 2.44 + 0.31 = 1.40 \text{ m}$$

$$\therefore \text{Thickness required} = \frac{1.40}{2.24 - 1} = 1.13 \text{ m.}$$

Hence, provide a thickness of 1.20 m cement concrete overlaid by 0.2 m thick pitching, for a length of 3 m (i.e. up to the end of cistern).

(iv) At 6 m from the d/s toe of crest

$$\% \text{ Pressure} = 27.8 + \frac{78.8 - 27.8}{15} \times 2 = 34.6 \%$$

$$\therefore \text{Maximum static head} = 0.346 \times 2.44 = 0.842 \text{ m}$$

(The value of x has not been added here since the cistern ends at 6 m.)

$$\therefore \text{Thickness required} = \frac{0.842}{2.24 - 1} = 0.59 \text{ m}$$

Hence, provide a thickness of 0.6 m cement concrete overlaid by 0.2 m thick pitching, for the last 2 m length of the floor.

Summary of floor thickness

- (i) U/s floor : 0.4 m.
 (ii) D/s toe of crest to 3 m d/s : 1.40 m.
 (iii) 3 m to 6 m from d/s toe : 1.20,
 (iv) Last 2 m : 0.6 m.

Step 6. Design of d/s wings

Keep the d/s wings vertical for a length of
 $6 \times \sqrt{H} = 6 \times \sqrt{0.892} \times 1.5 \approx 7$ m.

Then the wings may be warped to 1 : 1 slope, at a splay of 1 in 3. Height of top of d/s wings above the bed = water depth + F.B. = 1.8 + 0.5 = 2.3 m.
 Horizontal projection of this on 1 : 1 slope = 2.3 m. Using a splay of 1 : 3, length of warped wing, measured along the centre line of canal = $2.3 \times 3 = 6.9 \approx 7$ m.

Step 7. Downstream pitching

(a) **Bed pitching.** From Table 18.1, length of bed pitching for $H = 0.86$ m is equal to $(9 + 2 \times 1.5) = 12$ m. This may be provided horizontal upto the end of masonry wings and then sloping at 1 in 10. Thus length of sloping pitching = 7 m and length of horizontal pitching = 6 m.

(b) **Toe-wall.** Provide one toe wall of 1 m depth and 0.4 m width.

(c) **Side pitching.** The side protection has been provided, consisting of 0.2 m thick brick on edge. This has been warped to slope of $1 \frac{1}{2} : 1$. The side pitching is curtailed at an angle of 45° from the end of bed pitching in plan. Support the side pitching on a toe wall 0.4 m thick and 1 m deep.

Step 8. Energy dissipators

Energy dissipators are not provided for small discharges. However, for large discharges, additional dissipators should be provided. The dissipators consists of two rows of friction blocks in the cistern, and two rows of cube blocks just at the d/s end of impervious floor.

$$\text{Critical depth } d_c = \left(\frac{q^2}{g} \right)^{1/3}$$

$$\text{Here } q = \frac{40}{26} = 1.538 \text{ cumecs/m ; } d_c = \left[\frac{(1.538)^2}{9.81} \right]^{1/3} \approx 0.7 \text{ m}$$

(a) Size and position of friction blocks

- (i) Length of block $2d_c = 2 \times 0.7 = 1.4$ m
 (ii) Width of block $= d_c = 0.7$ m
 (iii) Height of block $= d_c = 0.7$ m
 (iv) Distance from toe of crest $= 1.5 d_c = 1.05 \approx 1$ m.

Provide two rows, staggered, at a distance of 1 m from the toe of the crest

(b) Cube blocks

- (i) Length $\frac{1}{10} D = \frac{1}{10} \times 1.8 = 0.18 \approx 0.2$ m

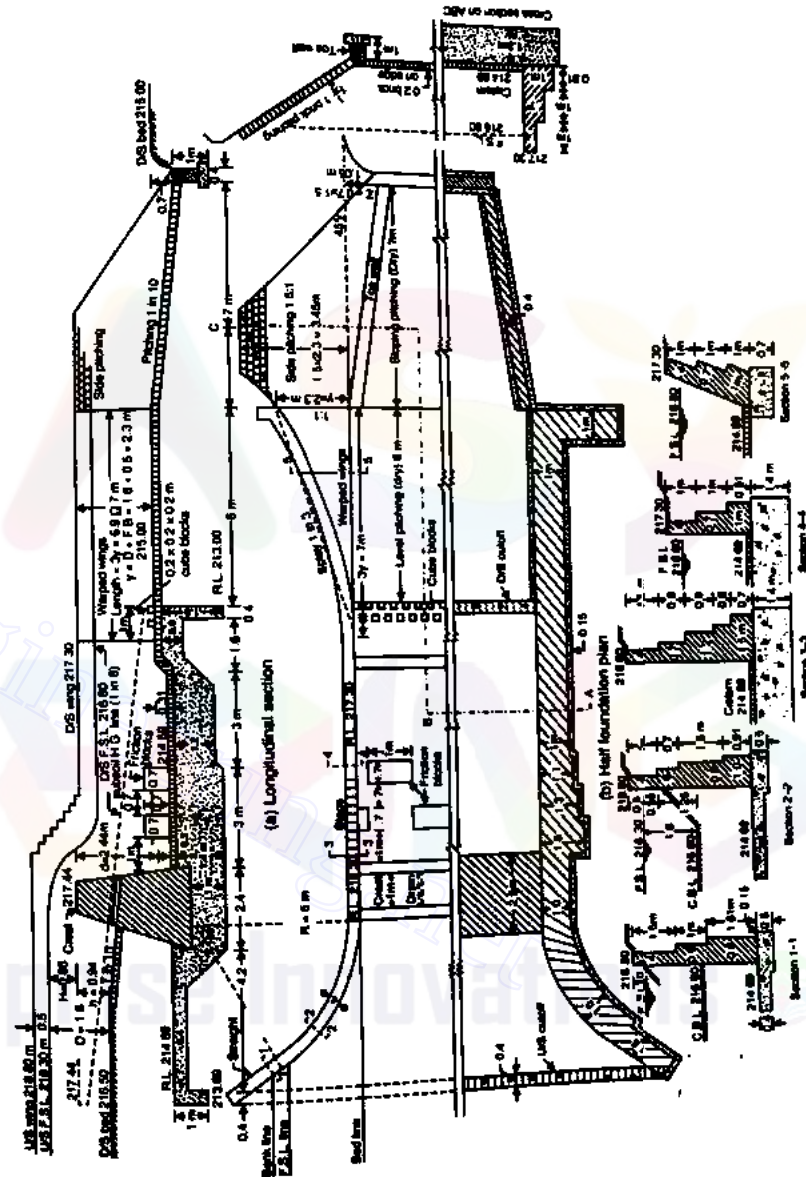


FIG. 18.10. SARDA TYPE FALL

$$(iii) \text{ Height} = \frac{1}{10} D = 0.2 \text{ m.}$$

$$(ii) \text{ Width} = \frac{1}{10} D = 0.2 \text{ m}$$

Provide two rows, staggered, at the end of impervious floor.

Step 9. U/S approach

$$\text{Radius of u/s wings} = 6H = 6 \times 0.86 = 5 \text{ m.}$$

Provide wing wall segmental with 5 m radius, subtending an angle of 60° at the centre, and the carried tangentially into the berm. The complete details of the fall are shown in Fig.18.10.

18.7. DESIGN OF STRAIGHT GLACIS FALL

(1) Crest design

(a) For non-meter falls, the u/s glacis is given a slope of $\frac{1}{2}:1$, and is joined tangentially to the crest with a radius of $E/2$, where E = height of u/s T.E.L. above crest.

(b) The d/s glacis is given a slope of $2:1$ and is joined tangentially to the cistern with a radius = E .

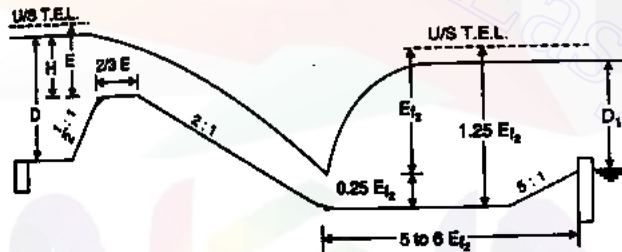


FIG. 18.11 STRAIGHT GLACIS FALL.

(c) The width of the crest is kept equal to $\frac{2}{3} E$.

(d) The glacis fall may sometimes be flumed if it is to be combined with a bridge etc. However, the minimum clear length of the crest, expressed in % of the bed width of canal, is kept according to the following recommendations :

Drop	Clear length of crest
Below 1.25 m	65%
1.25 to 3 m	75%
Above 3 m	85%

(e) The value of E is calculated from the discharge equation :

$$Q = 1.84 L_1 E^{3/2} \quad \dots(18.20)$$

where L_1 = length of the crest
 Q = discharge in cumecs.

If there are n number of piers, effective crest length to be substituted in Eq. 18.20 is equal to $(L_1 - 0.2n)$.

$$(f) \text{ Crest level} = \text{u/s T.E.L.} - E$$

$$\text{u/s T.E.L.} = \text{u/s F.S.L.} + \text{velocity head.}$$

2. Design of cistern

(a) Knowing the discharge intensity q per metre run and the drop H_L , the energy of flow (E_f) below the hydraulic jump is known.

$$\text{Then R.L. cistern} = \text{d/s T.E.L.} - 1.25 E_f$$

(b) Length of cistern = 5 to 6 E_f

3. Design of impervious floor

The total length of the impervious floor is found from the considerations of permissible exit gradient. The total length of the impervious floor may be provided in the following manner :

- (i) Length of cistern
- (ii) Horizontal length of d/s glacis
- (iii) Crest width
- (iv) Horizontal length of u/s glacis
- (v) Balance to be provided to the u/s.

4. Design of d/s protection

(a) Bed protection.

Since a deflector wall is provided at the u/s end of the floor, no bed pitching is provided.

The height of deflector wall above d/s bed, may be kept equal to $D_1/10$, where D_1 is the d/s water depth, shown in Fig. 18.11.

(b) Side protection.

Dry brick pitching of $1\frac{1}{2}$ brick thick is done upto a length $3D_1$. The pitching is supported on a toe wall 0.4 m wide and $D_1/2$ deep, subjected to minimum depth of 0.5 m .

(c) *Curtain wall.* Depth of curtain wall = $D_1/2$; width 0.4 m .

5. Design of energy dissipators

(a) Friction blocks

Four rows of friction blocks are provided in case of flumed glacis falls. The height of each block is kept equal to $D_1/8$ and length equal to three times the height of the block. The distance between the rows is kept equal to the height of the blocks. The u/s edge of the first block may be provided at a distance of 5 times the height of the blocks from the toe of glacis.

(b) Glacis blocks.

For falls more than 2 m , one row of glacis blocks, of the same dimensions as that of friction blocks, is provided at the d/s end of glacis.

(c) Deflector wall.

A deflector wall of height $D_1/10$ and width 0.4 m is provided at the d/s of cistern. The same wall may work as the curtain wall.

6. Design of d/s expansion

In a flumed fall, the d/s expansion starts from the toe of the glacis. A rectangular hyperbolic expansion is generally preferred.

The bed width B_x at any distance x from the d/s toe of the glacis is given by

$$b_x = \frac{B_1 \times B_2 \times L_x}{(L_x \times B_2) - (B_2 - B_1)x} \quad \dots(18.21)$$

where L_x = total length in which the expansion is to be done. For small falls, straight 1 in 3 expansion is done.

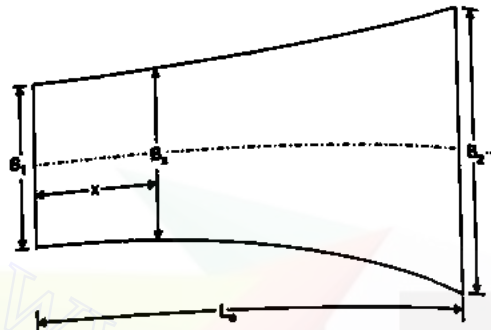


FIG. 18.12. HYPERBOLIC EXPANSION.

7. Upstream approach

For a non-meter fall, the side walls are splayed at an angle of 45° from the u/s edge of the crest and is carried straight into the berm by a length of at least 1 m.

DESIGN EXAMPLE OF UNFLUMED STRAIGHT GLACIS NON-METER FALL

Example 18.3. Design an unflumed straight glacis non-meter fall for the data of Example 18.1.

- (i) Full supply discharge : $\frac{u/s}{d/s} = 40$ cumecs
- (ii) Full supply level : $\frac{u/s}{d/s} = \frac{218.30}{216.80}$ m
- (iii) Full supply depth : $\frac{u/s}{d/s} = \frac{1.8 \text{ m}}{1.8 \text{ m}}$
- (iv) Bed width : $\frac{u/s}{d/s} = \frac{26 \text{ m}}{26 \text{ m}}$
- (v) Bed level : $\frac{u/s}{d/s} = \frac{216.50}{216.00}$ m
- (vi) Drop : 1.5 m
- (vii) Permissible exit gradient : $1/6$.

Solution**Step 1. Design of crest**

Length of crest = Bed width of canal = 26 m

Discharge is given by

$$Q = 1.84 L_c E^{3/2}$$

$$40 = 1.84 \times 26 \times E^{3/2}$$

$$E = \left(\frac{40}{1.84 \times 26} \right)^{2/3} = 0.89 \text{ m}$$

Velocity of approach, assuming canal side slopes of 1:1

$$= \frac{40}{(26 + 1.8) 1.8} = 0.8 \text{ m/sec}$$

$$\therefore \text{Velocity head} = \frac{(0.8)^2}{2 \times 9.81} = 0.03 \text{ m}$$

$$\therefore \text{u/s T. E. L.} = \text{u/s F. S. L.} + \text{velocity head} = 218.30 + 0.03 = 218.33 \text{ m} \quad \dots(iv)$$

$$\text{Crest level} = \text{u/s T. E. L.} - E = 218.33 - 0.89 = 217.44 \text{ m} \quad \dots(v)$$

$$\text{Width of the crest} = \frac{2}{3} E = \frac{2}{3} \times 0.89 \approx 0.6 \text{ m} \quad \dots(vi)$$

Provide u/s glacis at slope $\frac{1}{2}:1$, joined tangentially to the u/s bed with radius $= \frac{1}{2} \times 0.89 = 0.5$ m. Keep d/s glacis at slope 2:1 joined tangentially to the cistern bed with radius $= E \approx 0.9$ m.

Step 2. Design of cistern

$$q = \frac{Q}{L_c} = \frac{40}{26} = 1.54 \text{ cumecs/metre}$$

$$H_L = 1.5 \text{ m.}$$

Hence, from Blench curves corresponding to the value of $q = 1.54$ cumecs/metre and $H_L = 1.5$ m, we get $E_p = 1.44$ m.

$$\therefore \text{R. L. of cistern} = \text{d/s T. E. L.} - 1.25 E_p \\ = (216.80 + 0.03) - (1.25 \times 1.44) = 216.03 \text{ m}$$

This is higher than the d/s bed level which is 215.00 m

Keep the R.L. of cistern at 214.50 m

$$\text{Length of cistern} = 6 E_p = 6 \times 1.44 = 8.62 \text{ m}$$

$$\text{Depth of cistern below d/s bed} = 215.00 - 214.5 = 0.5 \text{ m}$$

Provide cistern of 9 m length. This cistern will be joined to d/s bed in a slope of 5:1.

Step 3. Design of impervious floor.

Minimum depth of u/s curtain wall d_1

$$= \frac{D_1}{3} = \frac{1.8}{3} = 0.6 \text{ m.}$$

$\therefore$ Provide 0.4 m wide and 1 m deep curtain wall.

$$\text{Depth of d/s curtain wall } d_2 = \frac{D_2}{2} = \frac{1.8}{2} = 0.9 \text{ m.}$$

$\therefore$ Provide 0.4 m wide and 1 m deep curtain wall at the d/s. The curtain wall will project above the d/s bed by

$$\frac{D_2}{10} = \frac{1.8}{10} = 0.18 \text{ m}$$

so as to act as a deflector.

Exit gradient. $G_x = \frac{1}{6}$

But $G_x = \frac{1}{\pi \sqrt{\lambda}} \frac{H_s}{d_2}$

where

$$H_s = \text{maximum seepage head} = \text{Crest level} - \text{d/s bed level}$$

$$= 217.44 - 215.00 = 2.44 \text{ m}$$

$$d_2 = \text{depth of d/s curtain wall} = 1 \text{ m.}$$

$$\frac{1}{6} = \frac{1}{\pi \sqrt{\lambda}} \frac{2.44}{1}$$

$$\frac{1}{\pi \sqrt{\lambda}} = \frac{1}{6 \times 2.44} = 0.068$$

From Khosla's exit gradient curve we get

$$\alpha = 40 \text{ for } \frac{1}{\pi \sqrt{\lambda}} = 0.068.$$

$$\text{Alternatively : } \lambda = \left[\frac{1}{\pi (0.068)} \right]^2 = 21.71$$

$$\lambda = 21.71 = \frac{1 + \sqrt{1 + \alpha^2}}{2}, \text{ from which } \alpha \approx 42$$

$$b = \alpha d_2 = 42 \times 1 = 42 \text{ m}$$

This length is too excessive.

Hence, increase the depth d_2 of the d/s curtain wall to 2 m

$$\frac{1}{6} = \frac{1}{\pi \sqrt{\lambda}} \frac{2.44}{2}$$

or $\frac{1}{\pi \sqrt{\lambda}} = \frac{2}{6 \times 2.44} = 0.137$

Hence, for $\frac{1}{\pi \sqrt{\lambda}} = 0.137$, we get $\alpha \approx 10 \text{ m}$

$$\text{Alternatively } \lambda = \left[\frac{1}{\pi (0.137)} \right]^2 = 5.43$$

$$\lambda = \frac{1 + \sqrt{1 + \alpha^2}}{2} = 5.43, \text{ from which } \alpha \approx 9.8$$

$$b = \alpha d_2 = 9.8 \times 2 \approx 20 \text{ m.}$$

This length of 20 m will be arranged as under:

(i) Length of cistern = 9 m

(ii) Length of d/s glacis

$$d = 2(217.44 - 214.50) = 2 \times 2.94 = 5.88 \text{ m.}$$

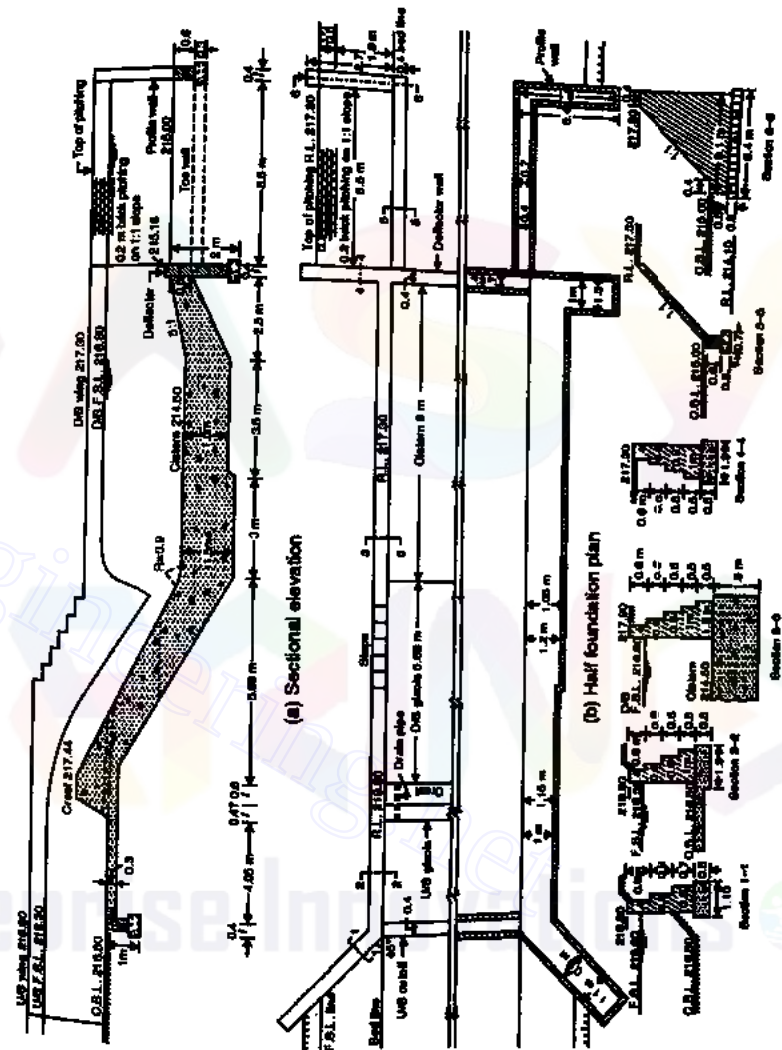


FIG. 18.13. GLACIS FALL.

- (iii) Width of crest = 0.60 m
 (iv) Length of w/s glacis = $\frac{1}{2}(217.44 - 216.50) = \frac{1}{2} \times 0.94 = 0.47$ m
 Total of (i) to (iv) = 15.95 m
 Balance to be provided to the w/s of the glacis
 = 20 - 15.95 = 4.05 m

Pressure calculations**(i) U/s curtain wall**

$$d_1 = 1 \text{ m}; b = 20 \text{ m}$$

$$\frac{1}{a} = \frac{d_1}{b} = \frac{1}{20} = 0.05$$

$$\phi_{D1} = 100 - 22 = 78\%; \phi_{D1} = 100 - 15 = 85\%$$

Assume a minimum thickness of 0.3 m at the upstream.

$$\therefore \text{Correction for thickness} = \frac{85 - 78}{1} \times 0.3 \pm 2\% (+)$$

$$\text{Corrected } \phi_{D1} = 78 + 2 = 80\%$$

(ii) D/s curtain wall

$$d_2 = 2 \text{ m}; b = 20 \text{ m}$$

$$\frac{1}{a} = \frac{d_2}{b} = \frac{2}{20} = 0.1$$

$$\phi_{D2} = 29\% \text{ and } \phi_{D1} = 21\%$$

Assuming a minimum thickness of 0.5 m at the d/s

$$\therefore \text{Correction to } \phi_{D2} = \frac{29 - 21}{2} \times 0.5 = 2\% (-ve)$$

$$\therefore \text{Corrected } \phi_{D2} = 29 - 2 = 27\%$$

(iii) Toe of d/s glacis

$$\begin{aligned} \text{Assuming a linear variation of pressure, the pressure at the d/s of the glacis} \\ = 27 + \frac{80 - 27}{20} \times 9 \pm 51\% \end{aligned}$$

Floor thickness

- (i) Provide a minimum thickness of 0.3 m at the u/s floor.
 (ii) Floor thickness at the toe of glacis :
 Minimum static head = $0.51 \times 2.44 + (215.0 - 214.5) = 1.75$ m
 $\therefore$ Floor thickness required

$$= \frac{1.75}{2.24 - 1} = 1.42 \text{ m}$$

Provide 1.5 m thick floor at the toe of the d/s glacis, for length of 3 m.

- (iii) Floor thickness at 3 m from the toe of glacis :

$$\% \text{ pressure} = 27 + \frac{80 - 27}{20} \times 6 \pm 43\%$$

$$\therefore \text{Maximum static head} = 0.43 \times 2.44 + (215.0 - 214.5) = 1.55 \text{ m}$$

$$\therefore \text{Floor thickness} = \frac{1.55}{2.24 - 1} = 1.25 \text{ m}$$

Provide a thickness of 1.30 m from 3 m to 6.5 m from the toe of the glacis.

- (iv) Floor thickness of d/s end of cistern:

% pressure at d/s curtain wall = 27%

$$\text{Thickness} = \frac{0.27 \times 2.44}{2.24 - 1} = 0.53 \text{ m}$$

Provide a thickness of 0.6 m.

Thickness at a point 6.5 m d/s of the toe of glacis = 1.30 m

Thus thickness at the d/s end of the floor = 0.6 m.

The thickness in between may be varied linearly as shown in Fig. 18.13.

Step 4. Design of d/s protection.

- (i) No bed protection is needed since the deflector wall has been provided.

- (ii) Side protection : Length of side protection
 = $3 D_2 = 3 \times 1.8 = 5.4$ m.

Hence, provide 20 cm thick brick pitching in 5.5 m length beyond the impervious floor at 1 : 1 slope. The pitching will rest on a toe wall 0.4 m wide and $1.8/2 = 0.9$ m deep. Provide a 0.4 m wide profile wall at the end of the pitching to protect the side pitching.

- (iii) No friction blocks are required since fluming has not been done.

Step 5. Design of u/s approach.

The u/s wing walls may be splayed at 45° from the u/s end of impervious floor and extended 1.0 m into the earthen banks from the line of F.S.I.

The design details are

shown in Fig. 18.13.

18.8. MONTAGU TYPE FALL

When hydraulic jump takes place on sloping glacis, the vertical component of velocity remains unaffected. Thus there is always the problem of residual energy below the jump. Hence, the profile of the glacis should be so designed that the maximum horizontal acceleration is imparted to the jet at all states of discharge. By imparting maxi-

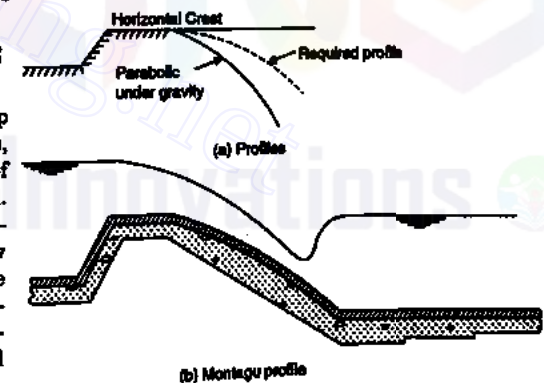


FIG. 18.14 MONTAGU TYPE FALL

imum horizontal acceleration, the maximum horizontal velocity is attained in any given length of the work.

When a stream is moving on a horizontal bed, there is no horizontal acceleration because the reaction is normal to the bed. Also, when the stream is flowing over a parabolic bed appropriate to the initial horizontal velocity U , there will be no horizontal acceleration because there is no reaction; the acceleration will be purely vertical under the action of gravity. In between these two, there lies such a glaciis profile upon which the horizontal component of the reaction imparts a maximum horizontal acceleration. Montagu derived analytically the following ideal glaciis profile :

$$x = U \sqrt{\frac{4y}{g}} + y \quad \dots(18.22)$$

where U = initial velocity of water leaving the crest,
 x = horizontal distance to any point on the profile, measured from the crest.
 y = vertical distance to that point, measured below the crest.

Dr. Boes, however, disagreed with the above equation, and gave the following expression for the glaciis profile:

$$x = U \sqrt{\frac{2y}{g}} + y \quad \dots(18.23)$$

Design Procedure

All falls of this type are flumed so that the width of the work is reduced. The width of the crest is kept equal to at least 2 to 2.5 times the head over the crest. The discharge is given by

$$Q = 1.7 L_c E^{3/2}$$

where E = head over the crest.

Assuming that the jump forms at the d/s bed level, the d/s specific energy E_2 is known. Knowing E_2 and H_1 , the discharge intensity q can be found from the Blench curves. Hence, $L_c = Q/q$, and can be found.

The depth of cistern, below the d/s bed, is kept equal to $\frac{1}{2} E_{f2}$ and the length of the cistern is kept four times the depth of the cistern.

The initial velocity U is found from the expression

$$U = \frac{q}{d_c}$$

where d_c is the critical depth.

Knowing U , the glaciis profile can be determined. The floor is designed on Khosla's theory. The minimum depth of u/s curtain wall is kept equal to $\frac{1}{2}$ F.S.D. and the depth of d/s curtain wall is kept equal to $\frac{1}{2}$ F.S.D. However, the bottom of d/s curtain wall is taken upto the lowest level of excavation. The d/s wings are flared. Brick pitching is provided in the bed and in the sides after the concrete floor and masonry wings. Friction blocks are provided to dissipate any residual energy. The thickness of the

glaciis is checked for the worst condition of the jump trough at different discharge intensities, as explained in Chapter 12 (design of sloping glaciis weir).

18.9. INGLIS TYPE FALL

Designed by C.C. Inglis, the fall makes use of horizontal impact for energy dissipation. The design consists of a standard long throated weir flume followed by a glaciis slope and a pavement on which a baffle is fixed to dissipate the energy. The baffle holds the jump stable on a horizontal platform. A cistern downstream of the baffle with a deflector at the d/s end of the cistern is provided. Pucca or impervious floor is provided only upto the end of the deflector. To the d/s of the deflector a second cistern is provided, which is only pitched with bricks or stones. (Fig. 18.15)

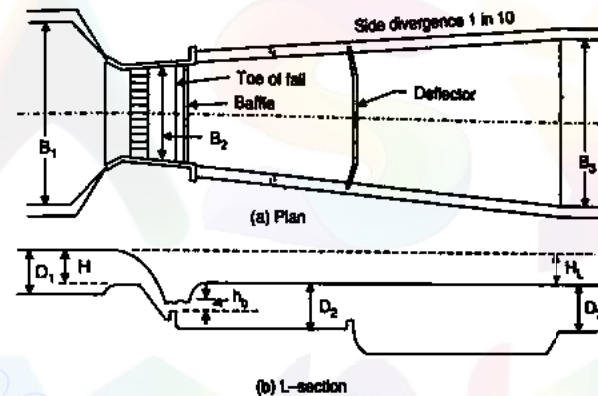


FIG. 18.15. INGLIS TYPE FALL.

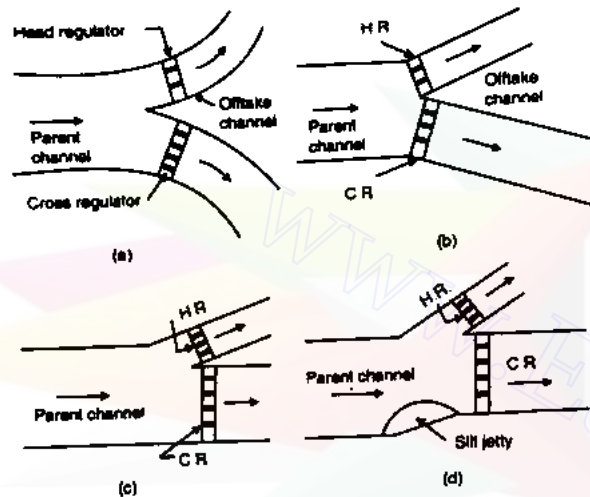
The maximum dissipation of energy by a hydraulic jump occurs when the jump forms at the toe of the glaciis. However, the jump may also form either on the glaciis, or at the d/s of the toe. If the jump forms on the sloping glaciis, the baffle should be fixed on a platform at a higher level than the canal bed level so that the natural wave will form at the toe of the platform. If the jump would form at the toe of the fall, the baffle should be fixed at pavement level. If, however, the jump would form at d/s of toe, the glaciis should be extended and a cistern provided of such depth as to bring the wave to the toe and the baffle fixed on the bed of the cistern. Even though the energy is effectively dissipated by baffle near the toe of a fall, the distribution of velocities is normal; hence, a cistern and deflector are provided. The cistern and deflector are meant to restore the normal distribution of velocities in the channel.

(B) CANAL REGULATORS

18.10. OFF-TAKE ALIGNMENT

When a distributing channel takes off from the parent channel, their off-take alignment should be very carefully designed. The best alignment of an off-take is when it makes zero angle with the parent channel, as shown in Fig 18.16(a). A transition

will have to be designed. If the transition curves are not given, both channels (i.e. parent channel and the distributing channel) should make an angle with the alignment of the parent channel upstream of the off-take Fig.18.16(b).



18.16. OFF-TAKE ALIGNMENT

Under the circumstances when the parent channel has to be kept straight both upstream and downstream of the off-take, the edge of the canal rather than the centre line should be considered in deciding the angle of off-take [Fig.18.16(c)]. In that circumstance, the section should not be narrowed down equally on both sides. An unbalanced off-take results in the formation of a jetty [Fig.18.16(d)]. The reduction in sectional area caused by the jetty would be made up by the scouring of the bed along the line of the deviated current.

18.11. HEAD REGULATORS AND CROSS-REGULATORS

Head regulator and cross-regulator regulate the supplies of the off-taking channel and the parent channel respectively. The distributary head regulator is provided at the head of the distributary and controls the supply entering the distributary. It is a necessary link between the parent channel and the distributing channel. A distributary head is a regulator, a metre of supply and a silt selective structure. A cross-regulator is provided on the main canal at the d/s of the off-take to head up the water level and to enable the off-taking channel to draw the required supply.

Functions of distributary head regulator

1. They regulate or control the supplies to the off-taking channel.
2. They serve as a meter for measuring the discharge entering into the off-taking canal.

CANAL REGULATION WORKS

3. They control the silt entry in the off-taking canal.
4. They help in shutting off the supplies when not needed in the off-taking canal, or when the off taking channel is required to be closed for repairs.

Functions of cross-regulator

1. The effective regulation of the whole canal system can be done with help of cross-regulator.
2. During the periods of low discharges in the parent channel, the cross-regulator raises water level of the u/s and feeds the off-take channel in rotation.
3. It helps in closing the supply to the d/s of the parent channels, for the purposes of repairs etc.
4. They help in absorbing fluctuation in various sections of the canal system, and in preventing the possibilities of breaches in the tail reaches.
5. Incidentally, bridges and other communication works can be combined with it.

18.12. DESIGN OF CROSS-REGULATOR AND DISTRIBUTARY HEAD REGULATOR

1. Design of crest

The discharge is determined by the drowned weir formula :

$$Q = \frac{2}{3} C_1 L \sqrt{2g} [(h + h_a)^{3/2} - h_a^{3/2}] + C_2 L d \sqrt{2g} (h + h_a) \quad \dots(18.24)$$

where Q = discharge, in cumecs

L = length of water-way, in metres

h = difference in water level u/s and d/s of the channel, in metres

h_a = head due to velocity of approach

d = depth of d/s water level in the channel, measured above the crest

C_1 = constant = 0.557

C_2 = constant = 0.80.

Generally the velocity of approach is small, and may be neglected while using Eq.18.24. Knowing the discharge Q , the length of water way L can be calculated.

For the cross-regulator, the crest level is kept equal to the upstream bed level of the parent channel. For the distributary head regulator, the crest level is kept 0.3 to 1 m higher than the crest level of the cross-regulator. The crest is joined to the d/s floor with a sloping glacis of 2 : 1.

2. Design of d/s floor

The level and length of the d/s floor is determined under two flow conditions:

- (i) full supply discharge passing through both the head regulator and cross-regulator, and
- (ii) the discharge in the parent channel being insufficient, the cross regulator gate is partially opened and the off-taking channel is running full. Or, the head regulator gate is fully open.

For both of these conditions, the discharge intensity q and the head loss $H_L (= h)$ are known. Hence, the value of E_p can be found from the Blench curves.

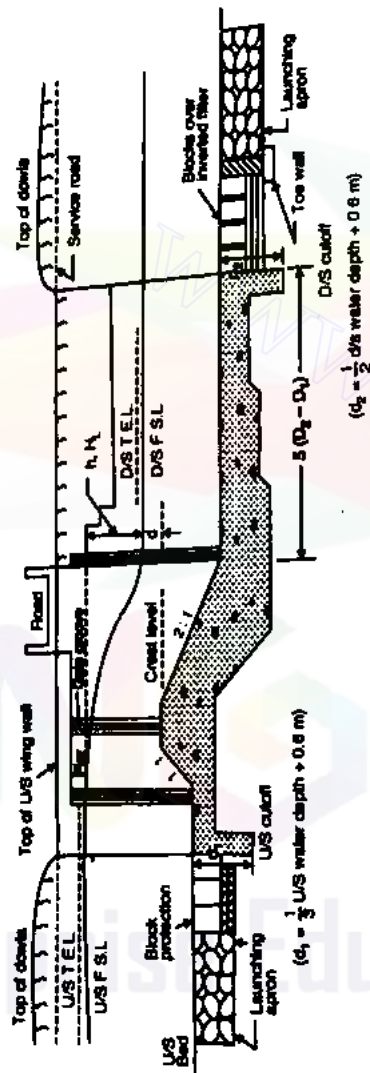


FIG. 18.17. DISTRIBUTARY HEAD REGULATOR

CANAL REGULATION WORKS

$$D/s \text{ floor level} = d/s \text{ T.E.L.} - E_n \approx d/s \text{ F.S.L.} - E_n$$

However, the d/s floor level, calculated from the above relation should never be provided higher than the d/s bed level.

$$\text{Now } E_n = E_{n1} + H_L$$

Hence, the depth D_1 and D_2 corresponding to E_{n1} and E_{n2} respectively are found from specific energy curves.

$$\text{Then length of d/s floor} = b(D_2 - D_1)$$

However, the d/s floor should be at least 2/3rd of the total impervious length of the floor.

3. Design of impervious floor

Total length of the impervious floor should be found from the consideration of permissible exit gradient.

$$\text{The depth of u/s cutoff } d_1 = \frac{1}{3} u/s \text{ water depth} + 0.6 \text{ m}$$

$$\text{The depth of d/s cutoff } d_2 = \frac{1}{2} d/s \text{ water depth} + 0.6 \text{ m}$$

$$\text{Maximum static head } H_s = u/s \text{ F.S.L.} - d/s \text{ floor level}$$

$$G_z = \frac{1}{\pi \sqrt{\lambda}} \frac{H_s}{d_2}, \text{ from which } \frac{1}{\pi \sqrt{\lambda}} \text{ is known.}$$

Then, from the exit gradient curves, $\alpha (=bd_2)$ is known. Hence, the total length b of the impervious floor is known.

The floor thickness is found from the considerations of uplift pressure. A minimum thickness of 0.3 to 0.5 m is provided from the practical considerations.

4. Design of u/s and d/s protection

U/s scour depth d_1 is taken equal $(\frac{1}{3} u/s \text{ water depth} + 0.6 \text{ m})$. The d/s scour depth d_2 is taken equal to $(\frac{1}{2} d/s \text{ water depth} + 0.6 \text{ m})$. These scour depths are below the corresponding bed levels, and protection works are to be designed corresponding to these.

(a) *U/s protection.* The u/s protection consists of a block protection having cubic contents $= d_1$ cubic metres/m. The cubic contents of u/s launching apron is kept equal to 2.25 d_1 cubic metre/metre width of regulator.

(b) *D/s protection.* The cubic contents d of d/s inverted filter is kept equal to d_2 cubic metre/metre. The cubic contents of d/s launching apron is kept equal to 2.25 d_2 cubic metre/metre width of regulator.

DESIGN EXAMPLE OF CROSS-REGULATOR AND HEAD REGULATOR

Example 18.3. Design a cross-regulator and the head regulator for a distributary channel taking off from the parent channel, for the following data.

IRRIGATION AND WATER POWER ENGINEERING

631

Discharge of parent channel	: = 100 cumecs
Discharge of distributary	: = 15 cumecs
F.S.L. of parent channel	: $\frac{u/s}{d/s} = \frac{218.10}{217.90}$
Bed width of parent channel	: $\frac{u/s}{d/s} = \frac{42 \text{ m}}{38 \text{ m}}$
Depth of water in parent channel	: $\frac{u/s}{d/s} = \frac{2.5 \text{ m}}{2.5 \text{ m}}$
F.S.L. of distributary	: 217.10 m
Bed width of distributary	: 15 m
Depth of water in distributary	: 1.5 m
Permissible exit gradient	: $= \frac{1}{5}$

Solution

(a) DESIGN OF CROSS-REGULATOR

Step 1. Design of crest and water-way

Crest level = u/s bed level of parent channel
 $= 218.10 - 2.50 = 215.60 \text{ m}$

Neglecting the velocity of approach, the discharge is given by

$$Q = \frac{2}{3} C_1 L \sqrt{2g} h^{3/2} + C_2 L d \sqrt{2gh}$$

Here $Q = 100 \text{ cumecs}$.

$$C_1 = 0.557; C_2 = 0.80$$

$$h = u/s \text{ F.S.L.} - d/s \text{ F.S.L.} = 218.10 - 217.90 = 0.20 \text{ m}$$

$$d = d/s \text{ F.S.L.} - \text{Crest level} = 217.90 - 215.60 = 2.30 \text{ m}$$

Substituting the values in the above expression, we get

$$100 = \frac{2}{3} \times 0.557 L \sqrt{2 \times 9.81} (0.20)^{3/2} + 0.8 L (2.30) \sqrt{2 \times 9.81 \times 0.20}$$

From which $L = 28.2 \text{ m}$.

Provide 4 bays of 7 m each with a clear water-way = 28 m

Provide 3 piers of 1.5 m width each.

Total width of cross-regulator = 28 + 4.5 = 32.5 m

Step 2. Design of d/s floor

$Q = 100 \text{ cumecs}$

$$q = \frac{100}{28} = 3.58 \text{ cumecs/m}$$

$$H_1 = h = u/s \text{ F.S.L.} - d/s \text{ F.S.L.} = 218.10 - 217.90 = 0.20 \text{ m}$$

$E_{p1} = 1.89 \text{ m}$ for $H_1 = 0.20 \text{ m}$ and $q = 3.58$ (from Blench curves)

$$d/s \text{ floor level} = d/s \text{ F.S.L.} - E_{p1} = 217.90 - 1.89 = 216.01 \text{ m}$$

This is higher than the d/s bed level.

CANAL REGULATION WORKS

633

Hence, adopt the d/s floor level = 216.40 = d/s bed level

$$E_{p1} = E_{p2} + H_1 = 1.89 + 2.20 = 2.09 \text{ m}$$

Hence from specific energy curves,

$$D_1 = 0.7 \text{ m and } D_2 = 1.65 \text{ m.}$$

$\therefore$ Cistern length (i.e. length of floor to the d/s of regulator gates)

$$= 5(D_2 - D_1) = 5(1.65 - 0.7) = 4.75 \text{ m.}$$

Total length of impervious floor = 16 m (see next step)

$$\frac{2}{3} \text{ length of impervious floor} = \frac{2}{3} \times 16 = 10.6 \text{ m}$$

Hence, provide length of d/s floor = 10.6 m

Step 3. Design of impervious floor

Depth of u/s cutoff = $d_1 = \frac{1}{3} u/s \text{ water depth} + 0.6 \text{ m}$

$$= \frac{1}{3} \times 2.5 + 0.6 = 1.43 \text{ m.}$$

Keep

$$d_1 = 1.50 \text{ m and width of cutoff} = 0.6 \text{ m}$$

Depth of d/s cutoff = $d_2 = \frac{1}{2} d/s \text{ water depth} + 0.6 \text{ m}$

$$= \frac{1}{2} \times 2.5 + 0.6 = 1.85 \text{ m}$$

Keep

$$d_2 = 2 \text{ m and width of cutoff} = 0.6 \text{ m.}$$

Worst case for under-seepage will be when full supply level is maintained in the parent channel u/s of cross-regulator, but no water to its d/s side.

Maximum static head = $H_s = u/s \text{ F.S.L.} - d/s \text{ floor level}$
 $= 218.10 - (217.90 - 2.5) = 2.7 \text{ m}$

$$G_s = \frac{1}{\pi \sqrt{\lambda}} \frac{H_s}{d_2}$$

$$\frac{1}{5} = \frac{1}{\pi \sqrt{\lambda}} \times \frac{2.7}{2}$$

$$\frac{1}{\pi \sqrt{\lambda}} = \frac{2}{5 \times 2.7} = 0.148$$

From exit gradient curves, we get

$$a = 8 \text{ when } \frac{1}{\pi \sqrt{\lambda}} = 0.148$$

$$\frac{b}{d_2} = 8$$

or

$$b = 8 d_2 = 8 \times 2 = 16 \text{ m.}$$

$\frac{2}{3}$ of this length = $\frac{16 \times 2}{3} = 10.6 \text{ m}$ which is more than 4.75 m of d/s length suggested earlier. Hence, the total length of 16 m will be provided as under :

IRRIGATION AND WATER POWER ENGINEERING

284

- (i) D/s floor length = 10.6 m
 (ii) D/s glacis length with 2 : 1 slope
 $= 2(215.60 - 215.40) = 0.4$ m
 (iii) Balance to be provided upstream = 5 m.

Pressure calculations

- (i) U/s cutoff :

$$d_1 = 1.5 \text{ m} ; b = 16 \text{ m}$$

$$\frac{1}{a} = \frac{d_1}{b} = \frac{1.5}{16} = 0.094$$

$$\phi_{U1} = 100 - 28 = 72 \%$$

$$\phi_{U2} = 100 - 19 = 81 \%$$

and Assuming a minimum floor thickness of 0.5 m at the u/s.

$$\text{Corrected } \phi_{U1} = 72 + \frac{81 - 72}{1.5} \times 0.5 = 75 \%$$

- (ii) D/s cutoff

$$d_2 = 2 \text{ m} ; b = 16 \text{ m}$$

$$\frac{1}{a} = \frac{d_2}{b} = \frac{2}{16} = 0.125$$

$$\phi_{D1} = 31 \% \text{ and } \phi_{D2} = 22 \%$$

Assuming a minimum thickness of 0.6 m at d/s,

$$\text{Corrected } \phi_{D1} = 31 - \frac{31 - 22}{2} \times 0.6 = 28.3 \%$$

- (iii) Toe of the glacis

$$\text{Percentage pressure} = 28.3 + \frac{75 - 28.3}{16} \times 10.6 = 59.3 \%$$

$$\text{Static pressure head} = 6.593 \times 2.7 = 1.6 \text{ m}$$

Calculations for thickness

- (i) Minimum thickness for u/s floor = 0.5 m.

- (ii) Thickness of floor near d/s cutoff

$$= \frac{0.283 \times 2.7}{2.24 - 1} = 0.61 \text{ m.}$$

Hence, provide 0.70 m thick floor for the last 2.1 m length.

- (iii) Thickness of floor at the toe of the glacis

$$= \frac{1.6}{2.24 - 1} = 1.29 \text{ m}$$

Provide 1.30 m thick floor at 4 m length.

- (iv) Thickness of floor at 4 m from toe of glacis

$$\text{Percentage pressure} = 28.3 + \frac{75 - 28.3}{16} \times 6.6 = 47.5 \%$$

CANAL REGULATION WORKS

285

$$\text{Thickness} = \frac{0.475 \times 2.7}{2.24 - 1} = 1.03 \text{ m}$$

Hence, provide 1.10 m thick floor for the next 2 m length of the floor.

- (v) Thickness of floor at 6 m from toe of glacis

$$\% \text{ pressure} = 28.3 + \frac{75 - 28.3}{16} \times 4.6 = 41.8 \%$$

$$\text{Thickness} = \frac{0.418 \times 2.7}{2.24 - 1} = 0.9 \text{ m}$$

Hence, provide 0.9 m thickness for the next 2.5 m.

Summary of floor thickness

- 0.5 m thickness of u/s floor.
- 1.30 m thick floor for a distance of 4 m from the toe of glacis.
- 1.10 m thick floor for the next 2 m length.
- 0.90 m thick floor for the next 2.5 m length.
- 0.70 m thick floor for the last 2.1 m length.
- Thickness under the gates, for a distance of 1 m = 1.0 m.

Step 4. Design of us protection

$$\text{Depth of u/s cutoff} = \frac{1}{3} \text{ F.S.D.} + 0.6$$

or

$$d_1 = \frac{1}{3} \times 2.5 + 0.6 = 1.43 \text{ m.}$$

- (i) Volume of block protection

$$= 1.43 \text{ cubic metre/m}$$

Keep the thickness of protection = 1 m

$$\therefore \text{Length} = 1.43 \text{ m}$$

Hence, provide 2 rows of 0.8 m $\times$ 0.8 m $\times$ 0.6 m thick concrete blocks over 0.4 m thick apron, in a length of 1.6 m.

- (ii) Cubic content of launching apron

$$= 2.25 d_1 = 2.25 \times 1.43 = 3.21 \text{ m}^3/\text{m.}$$

Hence provide 1 m thick and 3.5 m long launching apron.

Step 5. Design of d/s protection

$$\text{Depth of d/s cutoff} = \frac{1}{2} \text{ F.S.D.} + 0.6$$

or

$$d_2 = \frac{1}{2} \times 2.5 + 0.6 = 1.85 \text{ m}$$

- (i) Volume of inverted filter

$$= 1.85 \text{ m}^3/\text{m.}$$

Keep thickness of concrete blocks = 0.6 m kept over 0.6 m thick graded filter, making a total thickness of 1.2 m.

IRRIGATION AND WATER POWER ENGINEERING

596

$$\text{Length} = \frac{1.85}{1.2} = 1.54 \text{ m}$$

Hence, provide 2 rows of $0.8 \text{ m} \times 0.8 \text{ m} \times 0.6 \text{ m}$ c.c. block over 0.6 m graded filter for a length of 1.6 m .

$$(ii) \text{Launching apron volume} = 2.25 \times d_2 = 2.25 \times 1.85 = 4.16 \text{ m}^3/\text{m}.$$

Hence, provide 1 m thick launching apron for a length of 4.5 m .

Provide a 0.4 m thick and 1.5 m deep toe wall between the filter and the launching apron.

The design details have been shown in Fig. 18.18 (a).

(b) DESIGN OF DISTRIBUTARY HEAD REGULATOR

Step 1. Design of crest and water-way

$$\begin{aligned} \text{Crest level} &= \text{u/s bed level} + 0.5 \\ &= (218.10 - 2.5) + 0.5 = 216.10 \text{ m}. \end{aligned}$$

Neglecting the velocity of approach, the discharge is given by

$$Q = \frac{2}{3} C_1 L \sqrt{2g} h^{3/2} + C_2 L d \sqrt{2gh}$$

where $Q = 15 \text{ cumecs}$

$$C_1 = 0.577$$

$$C_2 = 0.80$$

$$h = \text{u/s F.S.L.} - \text{d/s F.S.L.} = 218.10 - 217.10 = 1.0 \text{ m}$$

$$d = \text{d/s F.S.L.} - \text{Crest level} = 217.10 - 216.10 = 1.0 \text{ m}.$$

Substituting these values in the discharge formula,

$$15 = \frac{2}{3} \times 0.577 \times L \sqrt{2 \times 9.81} (1.0)^{3/2} + 0.8 \times L \times 1.0 \sqrt{2 \times 9.81} \times 1.0$$

From which $L = 2.87 \text{ m}$.

This is too less in comparison to 15 m width of the offtaking channel.

Hence, provide 2 bays of 3.5 m each with 1 m thick pier in between.

$\therefore$ Over all width of regulator = 8 m .

The crest will have u/s glacis of $1 : 1$ slope and d/s glacis of $2 : 1$ slope.

Keep the width of crest = 1.0 m .

Step 2. Design of d/s floor

$$Q = 15 \text{ cumecs}$$

$$q = \frac{15}{7.5} = 2 \text{ cumecs/m}$$

$$H_1 = \text{u/s F.S.L.} - \text{d/s F.S.L.} = 218.10 - 217.10 = 1.0 \text{ m}.$$

Hence, for $q = 2$ and $H_1 = 1.0$, we get from Blench curves $E_{f2} = 1.58 \text{ m}$

$$\begin{aligned} \text{R.L. of d/s floor} &= \text{d/s F.S.L.} - E_{f2} \\ &= 217.10 - 1.58 = 215.52 \text{ m} \end{aligned}$$

CANAL REGULATION WORKS

837

Keep the R.L. of d/s floor = 215.50 m

$$\text{Again } E_{f1} = E_{f2} + H_1 = 1.58 + 1 = 2.58 \text{ m}$$

From specific energy curves, values of D_1 and D_2 corresponding to $E_{f1} = 2.58$ and $E_{f2} = 1.58 \text{ m}$ are :

$$D_1 = 0.42 \text{ m} \text{ and } D_2 = 2.55 \text{ m}$$

$$\therefore \text{Length of cistern} = 5(D_2 - D_1)$$

$$= 5(2.55 - 0.42) = 10.5 \text{ m}$$

Total Length of impervious floor

$$= 14 \text{ m (from Step 3)}$$

$$\frac{2}{3} \text{ of impervious floor} = 14.0 \times \frac{2}{3} = 9.3 \text{ m}$$

Hence, provide length of cistern (i.e. floor to the d/s of glacis) = 10.5 m .

Step 3. Design of impervious floor

Depth of u/s cutoff = $\frac{1}{3}$ u/s water depth = 0.6 m

$$d_1 = \frac{1}{3} \times 2.5 + 0.6 \approx 1.5 \text{ m}$$

Hence, provide 0.5 m wide and 1.5 m deep cutoff.

Depth of d/s cutoff = $\frac{1}{2}$ d/s water depth + 0.6 m

$$d_2 = \frac{1}{2} \times 1.5 + 0.6 = 1.35 \text{ m}$$

Now, in this case, the seepage head

$$= \text{u/s F.S.L.} - \text{d/s floor level}$$

$$H_s = 218.10 - 215.50 = 2.60 \text{ m}.$$

In the case of the cross-regulator, H_s was equal to 2.70 m and d_2 was equal to 2 m . Hence, in this case also, we should provide $d_2 = 2.00 \text{ m}$ (as against calculated value of 1.35) otherwise the floor length will come out to be excessive (i.e. much more than 16 m length of the previous case).

Hence, provide downstream cutoff of 0.5 wide and 2 m deep.

$$\text{Now } G_R = \frac{1}{\pi \sqrt{\lambda}} \frac{H_s}{d_2}$$

$$\frac{1}{5} = \frac{1}{\pi \sqrt{\lambda}} \frac{2.6}{2.0}$$

$$\frac{1}{\pi \sqrt{\lambda}} = \frac{2}{5 \times 2.6} = 0.154$$

From exit gradient curves, value of $\alpha = 7$ for $\frac{1}{\pi \sqrt{\lambda}} = 0.154$

$$b = \alpha d_2 = 7 \times 2 = 14 \text{ m}.$$

IRRIGATION AND WATER POWER ENGINEERING

838

The length of 14 m will be provided as under :

1. Length below the toe of glacis = 10.5 m
2. Length of d/s glacis at 2 : 1 slope = $2(216.10 - 215.60) = 1.2$ m
3. Width of crest = 1 m
4. Length of u/s glacis at 1 : 1 slope = $216.10 - 215.60 = 0.50$ m
5. U/s floor : balance = $14 - (10.5 + 1.2 + 1 + 0.5) = 0.8$ m.

However, provide a minimum length of 2.8 m to the u/s, making a total impervious length = 16 m. This increase of 2 m length will accommodate the piers and u/s wings.

Pressure calculations

(i) U/s cutoff $d_1 = 1.5$ m ; $b = 16$ m

$$\frac{1}{\alpha} = \frac{d_1}{b} = \frac{1.5}{16} = 0.094$$

Hence,

$$\phi_{C1} = 100 - 28 = 72\% \quad \text{and} \quad \phi_{D1} = 100 - 19 = 81\%$$

Assuming a minimum floor thickness of 0.5 m at u/s

$$\text{Corrected } \phi_{C1} = 72 + \frac{81 - 72}{1.5} \times 0.5 = 75\%$$

(ii) D/s cutoff $d_2 = 1.5$ m ; $b = 16$ m

$$\frac{1}{\alpha} = \frac{d_2}{b} = \frac{2}{16} = 0.125$$

Hence,

$$\phi_{C2} = 31\% \quad \text{and} \quad \phi_{D2} = 22\%$$

Assuming a minimum thickness of 0.6 m at d/s,

$$\text{Corrected } \phi_{C2} = 31 - \frac{31 - 22}{2} \times 0.6 = 28.3\%$$

(iii) Toe of glacis

$$\text{Pressure} = 28.3 + \frac{75 - 28.3}{16} \times 10.5 = 59.2\%$$

Calculations of thickness of floor

(i) Minimum thickness for u/s floor = 0.5 m

(ii) Thickness under the crest = 1 m

(iii) Thickness under the toe of the glacis

$$= \frac{0.592 \times 2.6}{2.24 - 1} = 1.24 \text{ m}$$

Provide a thickness of 1.30 m for a distance of 2 m from the toe of the glacis.

(iv) Thickness of floor at 2 m from the toe of glacis :

$$\% \text{ pressure} = 28.3 + \frac{75 - 28.3}{16} \times 9.5 = 53\%$$

$$\text{Thickness} = \frac{0.53 \times 2.6}{2.24 - 1} = 1.11 \text{ m}$$

Provide a thickness of 1.10 m for the next 4 m.

CANAL REGULATION WORKS

839

(v) Thickness of floor at 6 m from the toe of the glacis :

$$\% \text{ pressure} = 28.3 + \frac{75 - 28.3}{16} \times 4.5 = 41.4\%$$

$$\therefore \text{Thickness} = \frac{0.414 \times 2.6}{2.24 - 1} = 0.84 \text{ m}$$

∴ Provide a thickness of 0.90 m for the next 2.5 m.

(vi) Thickness at 8.5 m from the toe

$$\% \text{ pressure} = 28.3 + \frac{75 - 28.3}{16} \times 2 = 34.1\%$$

$$\therefore \text{Thickness} = \frac{0.341 \times 2.6}{2.24 - 1} = 0.71 \text{ m.}$$

Hence, provide 0.7 m thick floor for its 2 m length.

Step 4. Design of u/s protection

$$\text{U/s scour depth} = \frac{1}{3} \times 2.5 + 0.6 \approx 1.5 \text{ m}$$

Hence, provide the same protection as in the case of cross regulator.

Step 5. Design of d/s protection

$$\text{D/s scour depth} = \frac{1}{2} \text{ d/s water depth} + 0.6 \text{ m}$$

$$d_2 = \frac{1}{2} \times 1.5 + 0.6 = 1.35 \text{ m}$$

(i) Inverted filter

$$\text{Volume} = d_2 = 1.35 \text{ m}^3/\text{m}$$

Provide 0.5 m thick cement concrete blocks over 0.5 m thick graded filter.

$$\text{Length required} = \frac{1.35}{1} = 1.35 \text{ m.}$$

However, provide two rows of 0.8 m × 0.8 m × 0.5 m thick concrete blocks over 0.5 thick graded filter.

(ii) Launching apron

$$\text{Volume} = 2.25 d_2 = 2.25 \times 1.35 = 3.04 \text{ m}^3/\text{m.}$$

Provide 1 m thick launching apron for a length of 3.5 m. Provide a masonry toe wall 0.4 m wide and 1.20 m deep between the filter and the launching apron.

The design details have been shown in Fig. 18.18(b).

18.18. CANAL ESCAPES

A canal escape is a structure constructed on an irrigation canal for the purpose of wasting some of its water. Depending upon the purpose, there can be three types of escapes :

- (1) Canal scouring escape,
- (2) Surplus escape,
- and (3) Tail escape.



FIG. 18.19. CANAL ESCAPE.

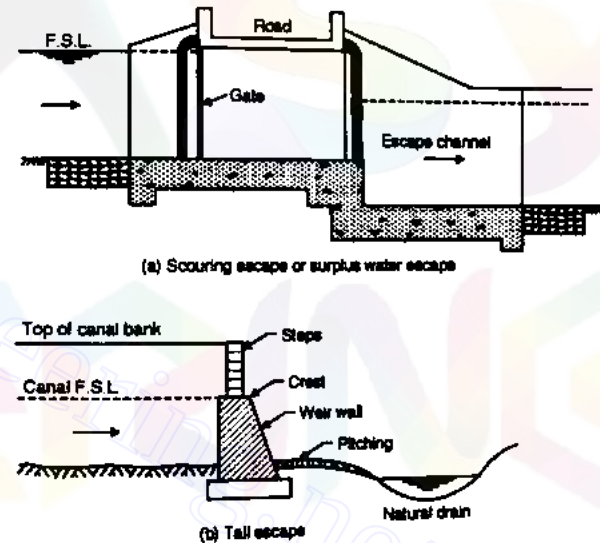


FIG. 18.19. CANAL ESCAPE.

PROBLEMS

1. What do you understand by a fall in a canal ? Why it is necessary? How do you select its location ?
2. Write a note on Notch type fall.
3. What is 'cistern element' in fall ? Give various expressions for its dimensions.
4. Explain the procedure of designing Sarda type fall.

5. Explain the procedure of designing straight glacis fall.
 6. Write a note on Montagu type fall ? How does it differ from glacis fall ?
 7. Design a Sarda type fall across a canal for the following data :

- (i) Full supply discharge $\frac{u/s}{d/s} = \frac{15}{15}$ cumecs
 (ii) Drop = 1.0 m
 (iii) Full supply level $\frac{u/s}{d/s} = \frac{101.00}{100.00}$ m
 (iv) Full supply depth $\frac{u/s}{d/s} = \frac{1.8}{1.8}$ m
 (v) Bed width $\frac{u/s}{d/s} = \frac{10}{10}$ m

Design the floor on Bligh's theory using coefficient of creep = 10. Sketch the longitudinal section of fall.

8. Design an unfummed straight glacis fall for the following data :

- (i) Full supply discharge $\frac{u/s}{d/s} = 12$ cumecs
 (ii) Drop = 1.5 m
 (iii) Full supply level $\frac{u/s}{d/s} = \frac{101.50}{100.00}$ m
 (iv) F.S. Depth $\frac{u/s}{d/s} = \frac{1.50}{1.50}$
 (v) Bed level $\frac{d/s}{d/s} = \frac{100}{98.5}$ m
 (vi) F.S. width $\frac{u/s}{d/s} = \frac{8.0}{8.0}$ m

Design the floor on Bligh's theory using coefficient of creep = 8.

Given : For linear discharge $q = 1.5$ cumecs/metre
 and $H_1 = 1.5$ m, $E_2 = 1.4$ m.

9. What do you understand by a head regulator ? State functions of a distributary head regulator and a cross-regulator.
 10. Explain the procedure for designing the head regulator of a distributary.
 11. Write a note on canal escape.
 12. What are the functions of regulators and falls on main canal? How are these works made safe against failure by (a) piping (b) bed scour?

Cross Drainage Works

19.1. INTRODUCTION

A cross drainage work is a structure carrying the discharge of a natural stream across a canal intercepting the stream. When a canal is to be taken to the watershed, it crosses a number of natural streams in the distance between the reservoir to the watershed. Once the canal is taken to the watershed, no cross-drainage works are normally necessary except when the canal leaves the watershed for some distance forming a loop. However, when the canal is aligned as a contour canal, a number of cross-drainage works are necessary.

A cross-drainage work is generally a very costly item, and should be avoided as far as possible by (i) diverting one stream into another, or (ii) changing the alignment of the canal so that it crosses below the junction of two streams.

19.2. TYPES OF CROSS-DRAINAGE WORKS

Depending upon the relative levels and discharges, cross-drainage works may be of the following types :

1. C.D. Works carrying canal over the drainage

In this type of C.D. work, the canal is carried over the natural drain (Fig. 19.1). The advantage of such arrangement is that the canal, running perennially, is above the ground and is open to inspection. Also, the damage done by floods is rare. However, sometimes during heavy floods the foundation can be scoured, or the waterway of the drain may be choked with trees etc. This is the usual type of work constructed when the drain is very big in comparison to the section of the canal. The structures that fall under this type are :

1. Aqueduct
2. Syphon aqueduct.

Fig. 19.1(b) and (c) show the aqueduct and syphon aqueduct respectively. As is clear from Fig. 19.1(b), the H.F.L. of the drain is much below the bottom of the canal trough in the case of aqueduct so that drainage water flows freely under gravity. However, in the case of a syphon aqueduct, the H.F.L. of the drain is much higher above the canal bed, and the water runs under syphonic action through the aqueduct barrels. In this case, the water surface level of the flood is depressed when it passes under the canal trough. The bed of the drainage is also lowered.

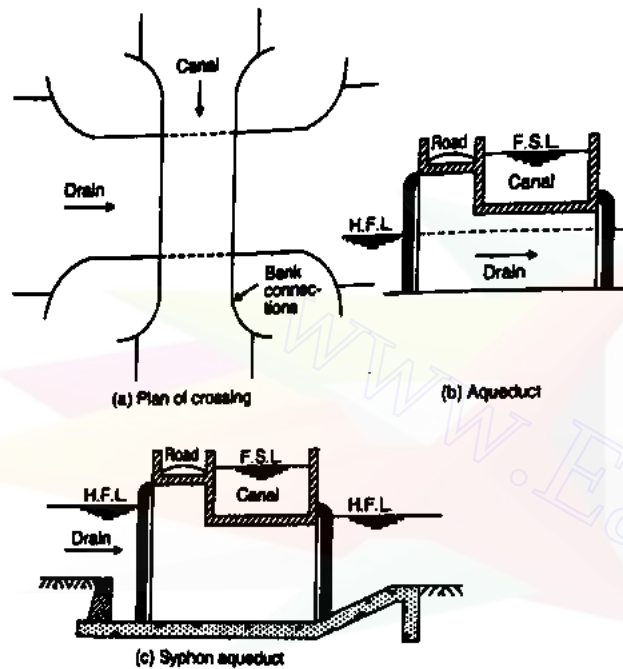


FIG. 19.1. AQUEDUCT AND SYPHON AQUEDUCT.

(II) C.D. Works carrying drainage over the canal

In this type of C.D. work, drainage is carried over the canal (Fig. 19.2). The advantage of this type is that the C.D. works themselves are less liable to damage than the earth-work of the canal.

The major disadvantage of this work is that the perennial canal is not open to inspection. Also, if the silt is deposited in the barrels of the work, it is difficult to clear it out.

The structures that fall under this type are

1. Super-passage
2. Canal syphon.

Fig. 19.2(b) shows a super-passage. A super-passage is similar to an aqueduct, except that in this case the drain is over the canal. The F.S.L. of the canal is lower under gravity. Fig. 19.2(c) shows a canal syphon, or simply *syphon*. In this, case, the trough, so that the canal runs under syphonic action under the trough. The canal bed is lowered and a ramp is provided at the exit so that the trouble of silting is minimised.

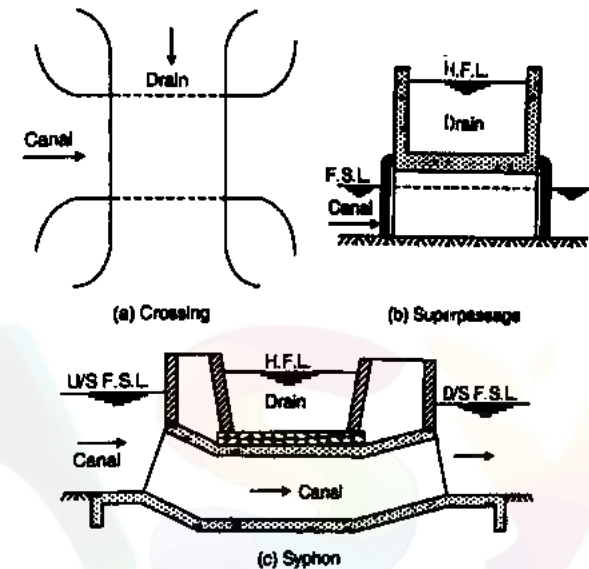


FIG. 19.2. SUPER-PASSAGE AND SYPHON.

(III) C.D. works admitting the drainage water into the canal

In this type of work, the canal water and the drainage water are permitted to intermingle with each other. The only advantage of this type of work is its low initial cost. Such type of works have the following disadvantages :

- (i) Regulation of such a work is difficult and requires additional staff.
- (ii) The canal has to be designed to carry the increased flood discharge of the drain.

(iii) The faulty regulation of gates may damage the canal.

(iv) There is additional expenditure of silt clearance.

Following are the structures under this type C.D. works :

1. Level crossing
2. Inlet and outlets.

Fig. 19.3(a) shows the schematic plan of a *level crossing* which is constructed in the circumstance when the beds of the canal and drainage are practically at the same level. In this type of work, the drainage water is passed into the canal and then taken out at the opposite bank. The work consists of (i) construction of a crest, with its top at the F.S.L. of the canal, at the u/s junction with the canal, and (iii) of the head regulator across the drainage at its d/s junction with the canal. When the drainage a cross-regulator across the canal at its d/s junction with the drainage. When the drainage does not carry any water, its regulator is closed while the cross-regulator of the canal is kept fully open so that the canal flows without any interruption. During the floods, however, the drainage regulator is opened so that the flood discharge, after spilling over the crest and mixing with the canal water, passes through it to the downstream

of the drainage. The accurate supplies in the canal are maintained by a cross-regulator. Level crossings are suitable for canals of all sizes. They are, however, specially suited to crossings of very large drains when the cost of other cross-drainage works would be very high.

A canal inlet (Fig. 19.3b) is constructed when cross-drainage flow is small, and its water may be absorbed into the canal without causing appreciable rise. However, if the canal is small, an outlet may be constructed to pass out the additional discharge which has entered the canal. It is not necessary that the number of inlets and outlets should be the same. There may be one outlet for two or three inlets.

19.3. SELECTION OF SUITABLE TYPE OF CROSS-DRAINAGE WORK

The factors which affect the selection of the suitable type of cross-drainage works are (i) relative bed levels and water levels of the canal and the drainage, and (ii) size of the canal and the drainage. The following considerations are important :

1. When the bed level of the canal is much above the H.F.L. of the drainage, so that sufficient headway is available for floating rubbish etc., and also for the structural elements of the work, an aqueduct is the obvious choice. Similarly, if the bed level of the drain is well above the F.S.L. of the canal, super-passage is provided.
2. The necessary headway between the canal bed level and the drainage H.F.L. can be increased by shifting the crossing to the d/s of the drainage. If, however, it is not possible to change the canal alignment, or if such a shifting does not give sufficient headway between the two levels, a syphon aqueduct may be provided. Thus, in the case of syphon aqueduct, the H.F.L. of the drain is above the bed of the canal.
3. When the canal bed level is much lower, but the F.S.L. of the canal is higher than the bed level of the drainage, a canal syphon is preferred.
4. When the drainage and the canal cross each other practically at the same level, a level crossing may be preferred. This type of work is avoided as far as possible.

The considerations governing the choice between aqueduct and syphon aqueduct (or super-passage and a syphon) are : (i) suitable canal alignment, (ii) suitable soil available for bank connections and (iii) nature of available foundation. As indicated

IRRIGATION AND WATER POWER ENGINEERING

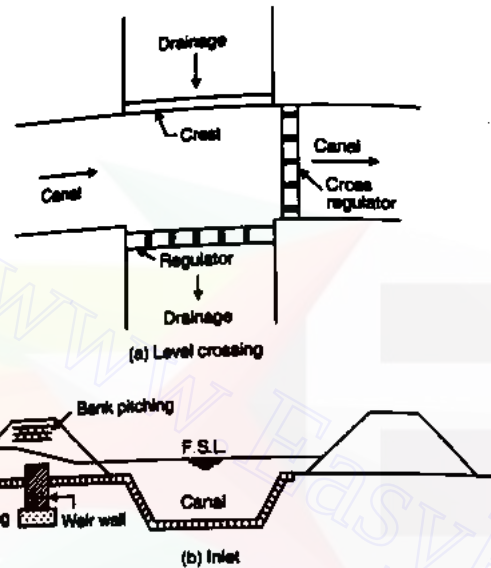


FIG. 19.3. LEVEL CROSSING AND INLET.

CROSS DRAINAGE WORKS

earlier, the relative difference between the bed level of the canal and H.F.L. of the drainage can be suitably altered by changing the canal alignment so that the point of crossing is shifted upstream or downstream of the drainage. For example, if the canal alignment is such that headway is not available between the H.F.L. of the drain and the bed of the canal, a syphon aqueduct is to be constructed at the crossing. But, however, if other conditions are not favourable for the construction of the syphon aqueduct, the canal alignment may be changed so that the crossing is shifted to the d/s and sufficient headway required for the construction of an aqueduct is available.

19.4. CLASSIFICATION OF AQUEDUCTS AND SYPHON AQUEDUCTS

Depending upon the nature of the sides of the aqueduct (or syphon aqueduct) it may be classified under three heads :

- (i) Sides of the aqueduct in earthen banks, with complete earthen slopes [Fig. 19.4 (b)].

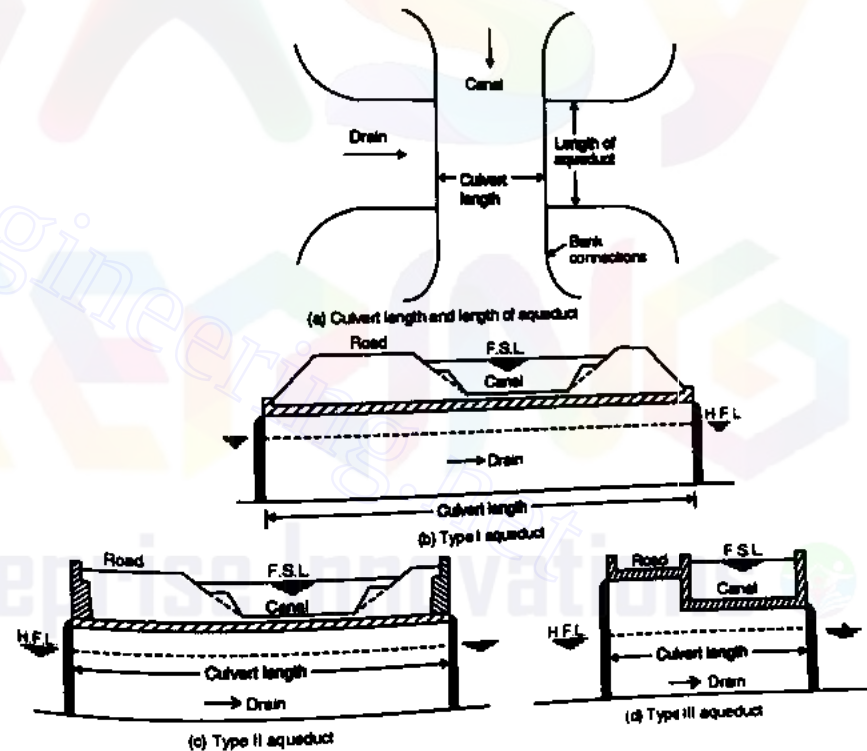


FIG. 19.4. CLASSIFICATION OF AQUEDUCTS

(ii) Sides of the aqueduct in earthen banks, with outer slopes supported by masonry walls [Fig. 19.4 (c)].

(iii) Sides of the aqueduct made of concrete or masonry [Fig. 19.4 (d)].

In the first type, the original canal section is retained and no fluming is done. The sides of the aqueduct are kept in earthen banks with complete earthen slopes. In the third type, on the contrary, the canal section is reduced, fluming is done and the sides of the canal are made of concrete or masonry instead of earth slopes. Type two is intermediate between the two in which the sides of the aqueduct are in earthen banks but the outer section is reduced by supporting it by masonry or concrete walls.

Selection of a suitable type.

In order to select a suitable type out of the three, it is necessary to understand three terms : (i) culvert length, (ii) length of the aqueduct, and (iii) bank connections. *Culvert length* is the width of the aqueduct, measured along the drain. The culvert length naturally depends upon the shape and size of the canal section. The *length of the aqueduct* is the length measured perpendicular to the drain. It is evidently equal to the width of the drain. *Bank connections* consist of masonry wings etc., required to connect the regular section of the canal to its modified section over the aqueduct.

In the first type, since canal is fully in earthen section, the culvert length is maximum. Hence, the cost per unit length of aqueduct will be maximum. However, bank connections are not required. Hence, the selection of this type depends upon the relative cost of bank connections and that of the aqueduct proper. In all cases, the cost of bank connection is independent of the length of the aqueduct. Hence, first type is suitable when the length of the aqueduct is small and the cost of bank connections would be large in comparison to the saving resulting from the reduction in the width of work if type III were adopted. On the contrary, in type III the culvert length is minimum. Hence, the cost per unit length of the aqueduct will be minimum. However, the cost of the bank connections will be additional. Therefore, type III is suitable where the length of culvert is large (i.e., a big drain). Type II is suitable for intermediate conditions. To sum up, for a very large drain, type III is more suitable while for a small drain, type I is more economical. The correct way of selecting a type is to work out cost of the three types and see which is cheaper.

19.5. FEATURES OF DESIGN OF CROSS-DRAINAGE WORKS

Following are some of the important features of design of cross drainage works.

(A) Hydraulic design :

1. Determination of maximum flood discharge and the high flood level (H.F.L.).
2. Fixation of waterway of the drain.
3. Contraction of canal waterway (for type III aqueducts)
4. Clearances and free board
5. Head loss through syphon barrels.
6. Determination of uplift pressure on the roof of trough.
7. Determination of uplift pressure on the floor.
8. Design of bank connections.

CROSS DRAINAGE WORKS

(B) Structural design

1. Design of the cross-section of the syphon barrels and the aqueduct trough.
2. Design of piers and abutments.
3. Design of foundations.

Determination of maximum flood discharge

Chapter 4 (Hydrology) deals with the various factors which affect the runoff from a catchment, and with the methods of determining the high flood discharge of a stream. For small drains, the empirical formulae may be used, while for large drains other reliable methods, such as hydrograph analysis etc. may be used.

IS Code recommendations (IS : 7784, Part I, 1975)

It is quite probable that a flood of higher magnitude than the design flood may pass through the structure posing greater danger to the stability of foundation and the structure. To take care of this unprecedented and unforeseen nature of flood intensities in cases of important structures, an adequate margin of safety is envisaged in the estimation of design discharge. For this purpose, the design discharge may be increased by the percentages given below for obtaining the *foundation and free board design* (Table 19.1)

TABLE 19.1

S.No.	Catchment area	Increase in design discharge
1.	Upto 500 km ²	35 percent to 25 percent decreasing with increase in area
2.	500 to 5000 km ²	25 percent to 20 percent decreasing with increase in area
3.	5000 to 25000 km ²	20 percent to 10 percent decreasing with increase in area
4.	Above 25000 km ²	Upto 10 percent

19.6. FIXATION OF WATERWAY OF THE DRAIN

The fixation of waterway of the drain is very important, as it may otherwise endanger the work. The waterway can be determined by Lacey's *P-Q* formulae :

$$P = 4.75 \sqrt{Q}$$

where P = wetted perimeter, in metre

Q = total discharge in cumecs.

For large drains, the wetted perimeter may be taken equal to the width of the river, and hence equal the waterway required. However, for small drains, width may be adjusted to provide the required perimeter. A contraction upto 20% of waterway is permissible in case of small drains. In large drains, however, no extra provision is made for the space occupied by the piers. After having known the waterway, the size of the barrels is so fixed that a minimum velocity of 2 and 3 meters per second is maintained. Afflux is also limited in such a way that there is no flooding of the country side. In case of syphon aqueduct, it is desirable to leave some clear headway (min. 1 m) between the bed level of the drainage downstream and the underside of the culvert roof.

IRRIGATION AND WATER POWER ENGINEERING

550

IS Code recommendations (IS : 7784, Part I, 1975)

In plains, the drainages are generally in alluvium and water way usually providing works without rigid floor is about 60 to 80 percent of perimeter, given by Lacey formula

$$P_w = C \sqrt{Q}$$

where P_w = wetted perimeter in m

C = a coefficient varying from 4.5 to 6.3 according to local conditions, the usual value adopted being 4.8 for regime channels

Q = design discharge in m^3/s .

The wetted perimeter obtained from the above formula is the total waterway between the two faces of the abutment.

Maximum permissible velocities

In works with rigid floors, however, water way can be further flumed within the permissible velocity negotiated through the available ventages. Ordinarily such velocities should be limited to the values given in Table 19.2.

TABLE 19.2. MAXIMUM PERMISSIBLE VELOCITIES

S. No.	Floor	Max. permissible velocity (m/s)
1.	Steel and C.I. lined face	10
2.	Concrete face	6
3.	Stone masonry face with cement pointing	4
4.	Stone or brick masonry face with cement plaster	4
5.	Brick masonry with cement pointing	2.5
6.	Hard rock	4
7.	Murram	2 to 3
8.	Soil : silt	1 to 2

For submountainous and mountainous terrains with flashy flows, the drainage water way is provided within the width of the existing stream. Where the slope of natural stream is quite steep, a slope flattening is usually required to bring down the velocity within the desired limits.

The minimum dimension of opening should be such as shall permit as far as possible manual clearing of deposits therein.

19.7. CLEARANCES AND FREEBOARD : IS CODE RECOMMENDATIONS

(a) Clearance for aqueduct

Clearance or headway in the case of aqueduct is the vertical distance between the u/s H.F.L. inclusive of afflux, of the drainage and the bottom surface of the trough.

CROSS DRAINAGE WORKS

551

Clearance for rectangular openings : The clearance will depend upon the relative levels of the canal bed and highest flood level of the drainage. The following values are suggested.

TABLE 19.3. MINIMUM VERTICAL CLEARANCE FOR RECTANGULAR OPENINGS.

S.N.	Discharge of drainage (m^3/s)	Minimum vertical clearance (mm)
1.	Below 30	600
2.	30 and above but below 300	900
3.	300 and above but below 3000	1200
4.	More than 3000	1500

If the minimum clearances specified above are not available, the safety of the superstructure shall be ensured against likely repercussions.

Clearance for arch openings : Minimum clearance measured to the crown of the arch should be normally be as recommended in Table 19.4.

TABLE 19.4. MINIMUM CLEARANCE FOR ARCH OPENINGS

S.No.	Arch opening (m)	Clearance
1.	Less than 3	Rise or 1.25 m whichever is more
2.	3 and above but less than 6	2/3 rise or 1.5 m whichever is more
3.	6 and above but less than 21	2/3 rise or 1.75 m whichever is more
4.	Above 21	2/3 rise

In the case of streams where a bed rise due to progressive silting is anticipated, the permissible clearance specified in Table 19.4 should be increased to allow for such aggradations depending upon the extent of silting.

(a) **Clearance for super passage :** In the case of super passage, clearance is the vertical distance between the u/s F.S.L. of the canal and the bottom surface of the drainage trough or the lowest point of the superstructure. Clearances of about 50% of those recommended for aqueducts may be provided in case of super passage.

(b) Free board

In the case of canals, free board is the difference between the F.S.L. of the canal to the top level of banks or the formation level of the guide banks. In the case of drainage, the free board is the difference between the H.F.L. including afflux, and the top of the embankment or guide banks.

The free board should not be less than 900 mm. Wherever heavy wave actions are anticipated, the free board should be increased suitably.

19.8. CONTRACTION OF CANAL WATERWAY

In type I aqueducts, the canal cross-section is not changed at the crossing, so that the culvert length is maximum. This is not economical on big drains. In such cases, type III aqueducts are adopted, where the original width of the canal is generally reduced. This requires the provision of extra transition wings for joining the flumed portion or the trough gradually to the normal section. The fluming or the contraction of the canal waterway should be done in such a way that the velocity in the trough is not more than 3 m/sec and the flow remains sub-critical without any formation

of hydraulic jump. The approach transition wings should not be steeper than 30° (corresponding to a splay of 2 : 1) and the departure transition should not be steeper than $22\frac{1}{2}^\circ$ (corresponding to a splay of 3 : 1). The transitions consist of curved and flared wing walls so that there is minimum loss of head and the flow is streamlined.

The transition can be designed under two different conditions:

- (1) Design of transitions when the water depth remains constant.
- (2) Design of transition when the water depth varies.

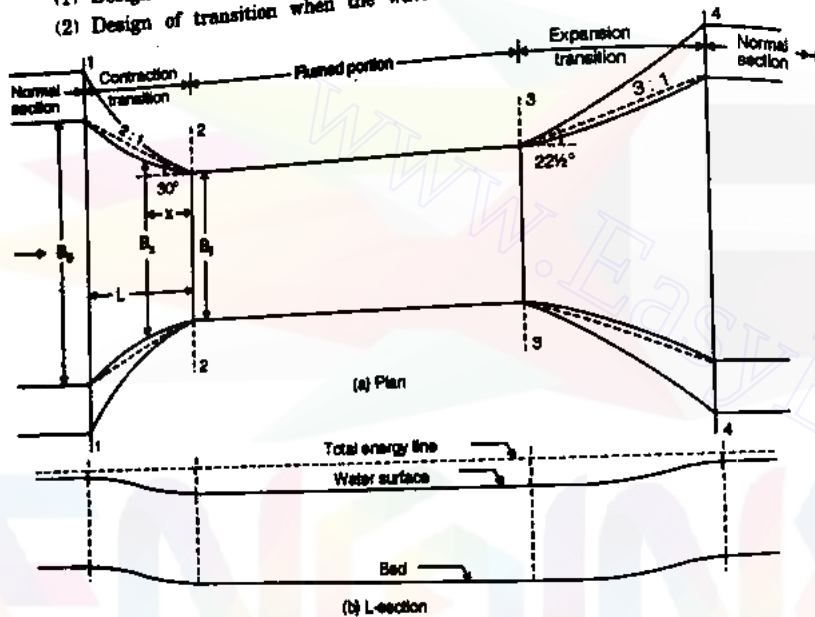


FIG. 19.5. CONTRACTION OF CANAL WATERWAY

(1) DESIGN OF TRANSITION WHEN THE WATER DEPTH REMAINS CONSTANT

(a) Mitra's hyperbolic transition

A.C. Mitra proposed hyperbolic transition for the water-depth in the trough to remain constant. Refer Fig. 19.5.

Let B_0 = original or normal width of canal section

B_f = flumed width of canal section

B_x = width of any distance x from the flumed section

Let V_0, V_f and V_x be the velocities at the corresponding sections

Let L_f = total length of transition

CROSS DRAINAGE WORKS

The transition is designed on the criterion that the rate of change of velocity per unit length of the transition is constant throughout the length of the transition. Thus

$$\frac{V_f - V_x}{x} = \frac{V_f - V_0}{L_f} \quad \dots (i)$$

Let Q = discharge (constant) and D = water length (constant)
 $V_f \times B_f \times D = V_x \times B_x \times D = V_0 \times B_0 \times D = Q$

$$V_f \times B_f = V_x \times B_x = V_0 \times B_0 = \frac{Q}{D} = \text{constant} = k$$

Hence, $V_f = \frac{k}{B_f}$; $V_x = \frac{k}{B_x}$ and $V_0 = \frac{k}{B_0}$

Substituting these values of velocities in (i) we get

$$\frac{\left(\frac{k}{B_f} - \frac{k}{B_x}\right)}{x} = \frac{\left(\frac{k}{B_f} - \frac{k}{B_0}\right)}{L_f}$$

or

$$B_x = \frac{B_0 B_f L_f}{L_f B_0 - (B_0 - B_f)x} \quad \dots (19.2)$$

This is Mitra's hyperbolic expression for bed width at any distance from the section.

(b) Chaturvedi's semi-cubical parabolic transition

Prof. R. S. Chaturvedi proposed the following equation of transition for constant depth, on the basis of his own experiments

$$x = \frac{L_f B_0^2}{B_0^{3/2} - B_f^{3/2}} \left\{ 1 - \left(\frac{B_f}{B_x}\right)^{3/2} \right\} \quad \dots (19.3)$$

Choosing various convenient values of B_x , the corresponding distance x can be computed from the above expression.

(2) DESIGN OF TRANSITION WHEN THE WATER DEPTH ALSO VARIES: HIND'S METHOD

When the water depth in the trough and the normal section of the canal vary, Hind's method given below may be used.

Fig. 19.5 shows the contraction and expansion transitions. The contraction starts at section 1-1 and ends at section 2-2. The canal section remains flumed from section 2-2 to 3-3. The expansion transition starts from section 3-3 and ends at section 4-4. Let D and V , with appropriate suffixes represent depth and velocity at different sections. The design is done in the following steps:

Step 1. Let the bed level and cross-section of the canal at section 4-4 be completely known.

Let the total energy level at any point be abbreviated as T.E.L.

Water surface elev. at section 4 = Bed level at section 4 + D_4 and

IRRIGATION AND WATER POWER ENGINEERING

854

T.E.L. at section 4 = Water surface elev. at section 3 + $\frac{V_4^2}{2g}$.

Step 2. Energy loss due to expansion between section 3 and 4 may be taken as $-0.3 \frac{V_3^2 - V_4^2}{2g}$. The friction loss between 3 and 4 is generally small and may be neglected. Since trough dimensions at 3 are known, V_3 is known.

Step 3. T.E.L. at section 3 = T. E. L. at 4 + $0.3 \frac{V_3^2 - V_4^2}{2g}$.

Water surface elev. at 3 = T.E.L. at 3 - $\frac{V_3^2}{2g}$.

Bed level at section 3 = Water surface elev. at 3 - D_3 .

Step 4. The channel section between 3 and 2 remains constant. The only loss of head in the trough is due to friction and can be worked out from the Manning's formula

$$Q = \frac{1}{N} A R^{2/3} S^{1/2}$$

T.E.L. at 2 = T. E. L. at 3 + Head loss

Water surface elev. at 2 = T.E.L. at 2 - $\frac{V_2^2}{2g}$.

Bed level at 2 = Water surface elev. at 2 - D_2 .

The depth and velocity in the trough are constant throughout and hence the T.E.L., water surface line and bed line are parallel to each other from section 2 to 3.

Step 5. Energy loss due to contraction between section 1 and 2 may be taken equal to $0.2 \left[\frac{V_2^2 - V_1^2}{2g} \right]$. The frictional loss may be neglected.

T.E.L. at 1 = T.E.L. at 2 + $0.2 \left[\frac{V_2^2 - V_1^2}{2g} \right]$

Water surface elev. at 1 = T.E.L. at 1 - $\frac{V_1^2}{2g}$.

Bed level at 1 = Water surface at 1 - D_1 .

Step 6. The bed level, water surface level and total energy level at all the four sections are known. The total energy line may be drawn straight between adjacent sections. The bed line is also drawn straight if the fall in level between the adjacent sections is small, and the corners can be rounded off. In case the drop in the bed is more, the sections are joined with a smooth reverse curve tangential to the bed.

Step 7. The drop in the water surface levels between any two adjacent sections will be due to (i) drop in the energy line between the two sections, and (ii) increased velocity head for contraction (or decreased velocity head for expansion). This drop is

CROSS DRAINAGE WORKS

855

achieved by two smooth parabolic curves, first convex and second concave upwards, meeting the water surface tangentially, as shown in Fig. 19.6.

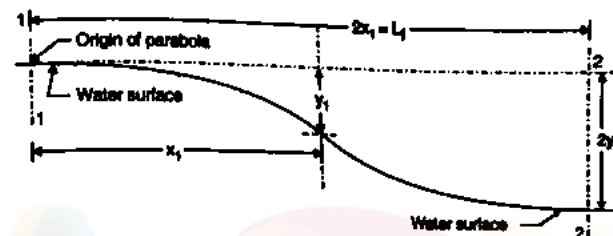


FIG. 19.6. WATER SURFACE CURVE FOR CONTRACTION TRANSITION.

Let $2x_1 = L_f$ = length in which fluming has been done.

$2y_1$ = total difference in water levels between 1 and 2.

The distance of the middle point of transition will be x_1 and drop in water surface will be y_1 . The equation of parabola, with origin at water surface of section 1, is given by

$$y = cx^2$$

When

$$x = x_1, y = y_1$$

and hence $c = \frac{y}{x^2} = \frac{y_1}{x_1^2}$

Hence the equation reduces to

$$y = \frac{y_1}{x_1^2} x^2 \quad \dots(19.4)$$

This is the equation of the first parabolic curve, which can now be plotted. Similarly, the second parabola can be plotted by taking the origin at section 2.

Similarly, the expansion transition water surface between section 3 and 4 can be plotted. There will be rise in the water surface.

Step 8. After having plotted the water surface profile over the whole length, the velocity head h_v can be found by measuring the vertical distance between the T. E. L. and water surface line at any point. The velocity head can then be converted into equivalent velocity by the relationship

$$V = \sqrt{2gh_v} \quad \dots(19.5)$$

Thus the velocity at each point can be known.

Step 9. The cross-sectional area at any point, required to the given discharge is given by

$$A = \frac{Q}{V} \quad \dots(19.6)$$

In trapezoidal channel, with water depth D , bed width B and side slopes $s:1$, the area is given by

$$A = BD + sD^2 \quad \dots(19.7)$$

In flared wings, the side slopes are gradually brought to vertical from an initial slope and, therefore, the side slope s at any point can be interpolated in proportion to the length of transition undergone. Thus, at any point, A , D and s are known. Substituting these in Eq. 19.7, the bed width B at any point is known. Thus all the dimensions of the transition are fully known.

19.9. HEAD LOSS THROUGH SYPHON BARRELS

In case of canal syphon and syphon aqueducts, the following formula by Unwin is generally used to calculate the head loss :

$$h = \left(1 + f_1 + f_2 \frac{L}{R}\right) \frac{V^2}{2g} - \frac{V_a^2}{2g} \quad \dots(19.8)$$

where h = head loss through the syphon, in metres

L = length of the barrel, in metres

R = hydraulic mean depth of the barrel

V = velocity of flow in metres/second

V_a = velocity of approach, and is generally neglected

f_1 = coefficient of head loss at entry

= 0.505 for unshaped mouth

= 0.08 for bell mouth.

and

$f_2 = a \left(1 + \frac{b}{R}\right)$, where the values of a and b for different materials may be taken as follows :

Nature of surface of barrel	a	b
1. Smooth iron pipe	0.00497	0.025
2. Encrusted pipe	0.00996	0.025
3. Smooth cement plaster	0.00316	0.030
4. Ashlar or brickwork	0.00401	0.070
5. Rubble masonry or stone pitching	0.00507	0.250

The velocity through the syphon barrel is limited between 2 to 3 metres per second. Thus, knowing the velocity, the head required to generate the same can be found from Eq. 19.8. The H.F.L. of the drain d/s of work remains unchanged, but the H.F.L. of the u/s side increases by the amount of afflux or total head loss as calculated by Unwin's equation. The top of guide banks and other marginal bunds etc., should be governed by this raised H.F.L. The extent of bed protection in the drain to d/s of the work depends upon the value of this afflux. Thus, by permitting a higher value of afflux (and thereby higher velocity through the syphon barrel) the cross-sectional area of the syphon barrel is reduced, but there is corresponding increase in the cost due to higher guide banks, etc., and greater length of protection d/s of the work. Also in order to reduce the afflux, the entry should be smooth, by providing

bell mouth shaped piers. The inside surface of the barrel should be made as smooth as possible.

Example 19.1. Design an expansion transition for a canal for the following data, using Mitra's method.

Length of flume = 16 m

Width of throat = 9 m

Width of canal = 15 m

Solution : Given : $B_f = 9$ m ; $B_0 = 15$ m ; $L_f = 16$ m

Let B_x = width at any distance x from the flumed section.

Now

$$B_x = \frac{B_0 B_f L_f}{L_f B_0 - (B_0 - B_f) x}$$

$$B_x = \frac{15 \times 9 \times 16}{16 \times 15 - x(16 - 9)} = \frac{2160}{240 - 7x}$$

Let us find the values of B_x at $x = 2, 4, 6, 8, 10, 12, 14$, and 16 m, as tabulated below :

x (m)	2	4	6	8	10	12	14	16
B_x (m)	9.47	10	10.59	11.25	12	12.86	13.86	15

Example 19.2. Solve Example 19.1 using Chaturvedi's method.

Solution

$$x = \frac{L_f B_0^{3/2}}{B_0^{3/2} - B_f^{3/2}} \left[1 - (B_f/B_x)^{3/2} \right]$$

$$x = \frac{16(15)^{3/2}}{15^{3/2} - 9^{3/2}} \left[1 - \left(\frac{B_f}{B_x} \right)^{3/2} \right] = 29.893 \left[1 - (B_f/B_x)^{3/2} \right]$$

$$(B_f/B_x)^{3/2} = 1 - (x - 29.893)$$

The computations are tabulated below.

x (m)	2	4	6	8	10	12	14	16
$B_f/B_x)^{3/2}$	0.933	0.8662	0.7983	0.7324	0.6666	0.5986	0.5317	0.4648
B_f/B_x	0.9549	0.9087	0.8613	0.8126	0.7622	0.7102	0.6563	0.6000
B_x	9.43	9.90	10.48	11.08	11.81	12.67	13.71	15.000

19.10. UPLIFT PRESSURE ON THE ROOF OR TROUGH

The hydraulic gradient line can be drawn when once afflux is known by Unwin's formula. Fig. 19.7 shows the hydraulic gradient line, along with the entry loss, the frictional loss and the exit loss. The uplift pressure at any point under the roof or the trough carrying canal water will be equal to the vertical ordinate between the hydraulic gradient line and the underside of the trough at that point. Maximum uplift pressure will be at the u/s point, near the entry. The uplift pressure on the underside

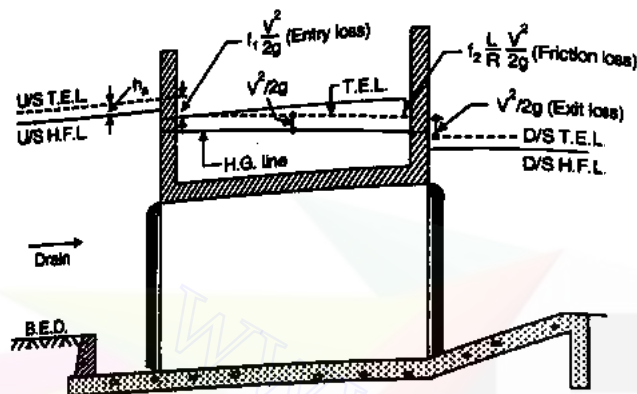


FIG. 19.7. UPLIFT PRESSURE ON THE TROUGH.

of the trough at the d/s end and will be equal to the difference in the d/s water and the under side of the trough.

The trough is to be designed under two conditions of flow :

Condition I. There is high flood in the drain but no water is flowing in the canal. This case will give maximum uplift pressure which should be resisted both by gravity action as well as the bending action. The roof thickness will be excessive if the uplift is resisted by gravity action alone. The increase in thickness will result in lowering the levels of both the underside of the trough and the floor. This, in turn, increases the uplift pressure on the trough as well as the floor. The best course, therefore, would be to provide the thickness of the trough for condition (II), and resist the remaining uplift by bending action by providing reinforcement at the top. In that case, the trough has to be anchored down to the bottom floor through piers by steel bolts etc.

Condition II. The second critical condition will be when the canal is running full, but there is no uplift from the underside. The trough is to be designed to take this water load, by providing a reinforced concrete roof with reinforcement at its bottom. A better alternative will be to provide a box culvert of reinforced concrete construction.

19.11. UPLIFT PRESSURE ON THE FLOOR OF THE AQUEDUCT

The floor of the aqueduct is subjected to uplift pressure due to two causes :

1. **Uplift pressure due to seepage of water from the canal to the drainage**
The maximum uplift pressure due to the seepage occurs when the canal is running full, but there is no water flowing in the drain.

2. **Uplift pressure due to subsoil water table in the drainage bed.** This acts when the floor is depressed below the bed of the drainage, specially in the case of siphon aqueducts. The worst condition here also will be when there is no water flowing in the drain, but the water table has risen upto the drainage bed. The uplift

pressure due to this static head will be equal to the difference in the level between the canal bed and the underside of the floor.

The determination of uplift pressure at any point under the floor due to cause (1) is highly complicated since the flow pattern is three dimensional. No approximation to a two dimensional case is possible as there is no typical plane across which the flow is practically two dimensional. As an approximate guide, Bligh's creep theory can be used to find the uplift pressure and to design the floor. For major works, this can be tested by model studies. Bligh's method is described below with reference to Fig. 19.8.

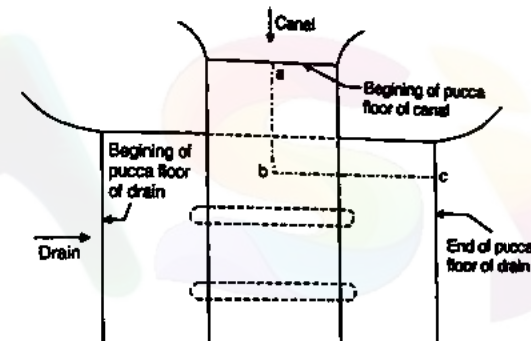


FIG. 19.8. UPLIFT PRESSURE UNDER THE FLOOR.

The flow starts from a point *a*, underside of the canal trough at its u/s end and then comes out at the point *c* under the centre of floor of the first culvert at its end. Point *b* is the point under the centre of the floor of first culvert.

The seepage path from *a* to *b* and from *b* to *c* can be known.

The total creep length ($\hat{L}$) will then be equal to the sum of the seepage path L_1 from *a* to *b* and the seepage path L_2 from *b* to *c*.

If H = total seepage head = F.S.L. of canal - d/s bed level of the drain, the residual seepage head at the point *b* is given by (Bligh's assumption) :

$$H_r = \frac{H}{\hat{L}} \times L_2 \quad \dots(19.9)$$

The floor of the aqueduct is to be designed for the total uplift pressure, which is equal to the sum of the uplift pressure due to seepage head plus the uplift pressure due to static head. The total uplift should be partly resisted by the weight of the floor and partly by the bending action.

Methods of reducing uplift on the floor

The uplift pressure on the floor can be reduced by the following two methods:

1. By extending the impervious length of the canal bed on either side, the creep length is increased. If a concrete floor is uneconomical, a puddle apron may be provided.

IRRIGATION AND WATER POWER ENGINEERING

880

2. By providing drainage holes or relief holes in the floor of the aqueduct, the uplift pressure can be released. An inverted filter may be provided below the floor so that the subsoil particles are not lifted up. It would also be necessary to provide flap valves on the top of relief holes, so that seepage water from beneath may come up but the silt laden water may not enter the filter and choke it.

19.12. DESIGN OF BANK CONNECTIONS

The bank connections consist of :

- (i) River Wings (ii) Canal Wings

(i) River Wings

The river wings or the drainage wings should be so designed that

- there is smooth entry and exit.
- they retain the earth behind it when no water is flowing in the drain.
- their foundation is taken below the scour level.

(ii) Canal Wings

The canal wings are generally warped to change the canal section from trapezoidal to rectangular in the trough. The wings are extended to the end of splay. They should be designed as retaining walls, with no water in the canal. They should be taken deep enough to give safe creep length.

19.13. DESIGN EXAMPLE OF A SYPHON AQUEDUCT

Design a siphon aqueduct for the following data :

- Discharge of the canal : 25 cumecs.
- Bed width of the canal : 20 m.
- Depth of water in the canal : 1.5 m.
- Bed level of the canal : 160.00 m.
- High flood discharge of the drainage : 400 cumecs.
- High flood level of the drainage : 160.50 m.
- Bed level of the drainage : 158.00 m.
- General ground level : 160.00 m.

Solution.

Since the size of the drainage is quite large, type III aqueduct will be economical and, hence, is adopted. The canal section will be flumed and the canal water will be taken in a concrete trough.

Step 1. Design of drainage water-way

The wetted perimeter of the river is found by Lacey's $P-Q$ formula :

$$P = 4.75 Q = 4.75 \sqrt{400} = 95 \text{ m.}$$

Provide 13 spans of 6 m each, separated by 12 piers each of 1.25 m thick

Length of clear water way = $13 \times 6 = 78 \text{ m}$

Length occupied by piers = $12 \times 1.25 = 15 \text{ m}$

Total length of water way = $78 + 15 = 93 \text{ m}$

CROSS DRAINAGE WORKS

881

Let the velocity through syphon barrels = 2 m/sec.

$$\therefore \text{Height of barrels required} = \frac{400}{78 \times 2} = 2.56 \text{ m}$$

Provide rectangular barrels 6 m wide and 2.50 m high

$$\text{Actual velocity through barrels} = \frac{400}{6 \times 2.50 \times 1.3} = 2.05 \text{ m/sec}$$

Step 2. Design of canal water-way :

Type III aqueduct will be adopted.

Bed width of the canal = 20 m.

Let the width be reduced to 10 m

Providing a splay of 2 : 1 in contraction the length of contraction transition

$$= \frac{20 - 10}{2} \times 2 = 10 \text{ m}$$

Providing a splay of 3 : 1 in expansion, the length of expansion transition.

$$= \frac{20 - 10}{2} \times 3 = 15 \text{ m}$$

Length of the flumed portion from abutment to abutment = 93 m.

In the transitions the side slopes of the canal section will be warped from the original slopes of $1\frac{1}{2} : 1$ to vertical.

Step 3 : Design of levels at different sections

Refer Fig. 19.5.

(a) At Section 4-4

Area of section with $1\frac{1}{2} : 1$ side slopes

$$= (B + 1.5 D) D = (20 + 1.5 \times 1.5) 1.5 = 33.4 \text{ m}^2$$

$$\text{Velocity of flow} = V = \frac{Q}{A} = \frac{25}{33.4} = 0.75 \text{ m/sec}$$

$$\text{Velocity head} = \frac{V^2}{2g} = \frac{(0.75)^2}{2 \times 9.81} = 0.029 \text{ m}$$

R.L. of bed = 160.00 m (given)

R.L. of water surface = 160.00 + Water depth = 160.00 + 1.5 = 161.5 m

R.L. of T.E.L. = 161.5 + 0.029 = 161.529 m

(b) At Section 3-3

Area of trough = $10 \times 1.5 = 15 \text{ m}^2$

$$\therefore \text{Velocity} = \frac{Q}{A} = \frac{25}{15} = 1.665 \text{ m/sec.}$$

IRRIGATION AND WATER POWER ENGINEERING

362

$$\text{Velocity head} = \frac{V^2}{2g} = \frac{(1.665)^2}{2 \times 9.81} = 0.141 \text{ m.}$$

Loss of head in expansion, from Section 3-3 to 4-4

$$= 0.3 \left[\frac{V_3^2 - V_4^2}{2g} \right] = 0.3 (0.141 - 0.029) = 0.034 \text{ m}$$

$$\text{Elev. of T.E.L. at 3} = \text{T.E.L. at 4} + \text{Head loss} = 161.529 + 0.034 = 161.563 \text{ m}$$

$$\text{R.L. of water surface at 3} = \text{T.E.L.} - \text{Velocity head} = 161.563 - 0.141 = 161.422 \text{ m.}$$

∴ R.L. of bed, to maintain constant water depth

$$= \text{R.L. of water surface} - \text{Water depth} = 161.422 - 1.5 = 159.922 \text{ m.}$$

(c) At Section 2-2

Hydraulic mean depth in the trough

$$= R = \frac{A}{P} = \frac{15}{10 + 2 \times 1.5} = 1.155$$

$$\text{Now } V = \frac{1}{N} R^{2/3} S^{1/2} \text{ (Manning's formula)}$$

$$\text{or } S = \frac{V^2 N^2}{R^{4/3}}$$

where V = Velocity of flow in the trough

S = Required slope

N = Rugosity coefficient = 0.016

$$\therefore S = \frac{(1.665)^2 (0.016)^2}{(1.155)^{4/3}} = 5.85 \times 10^{-4}$$

Length of trough (flumed portion) = 93 m

$$\therefore \text{Head loss in trough} = 93 \times 5.85 \times 10^{-4} = 0.054 \text{ m.}$$

∴ R.L. of T.E.L. at 2 = R.L. of T.E.L. at 3 + Head loss in trough

$$= 161.563 + 0.054 = 161.617 \text{ m}$$

$$\text{R.L. of water surface at 2} = 161.617 - 0.141 = 161.476 \text{ m}$$

R.L. of bed, to maintain constant water depth

$$= \text{R.L. of water surface at 2} - \text{Water depth} = 161.476 - 1.5 = 159.976 \text{ m.}$$

(d) At Section 1-1

Loss of head in contraction transition from section 1-1 to section 2-2

$$= 0.2 \frac{V_2^2 - V_1^2}{2g} \text{ where } V_1 = V_4 \text{ and } V_2 = V_3$$

CROSS DRAINAGE WORKS

363

$$= 0.2 \left[\frac{(1.665)^2 - (0.75)^2}{2 \times 9.81} \right] = 0.022 \text{ m}$$

$$\text{R.L. of T.E.L. at 1} = \text{R.L. of T.E.L. at 2} + \text{Head loss} = 161.617 + 0.022 = 161.639 \text{ m.}$$

$$\text{R.L. of water surface} = \text{R.L. of T.E.L.} - \frac{V_1^2}{2g}$$

$$= 161.639 - \frac{(0.75)^2}{2 \times 9.81} = 161.639 - 0.029 = 161.610 \text{ m.}$$

R.L. of bed, to maintain constant water depth

$$= 161.610 - 1.5 = 160.110 \text{ m.}$$

Step 4. Design of contraction transition

The transition will be designed on the basis of Chaturvedi's formula :

$$x = \frac{L B_0^{3/2}}{B_0^{3/2} - B_f^{3/2}} \left\{ 1 - \left(\frac{B_f}{B_x} \right)^{3/2} \right\}$$

where $B_0 = 20 \text{ m}$; $B_f = 10 \text{ m}$; $L = 10 \text{ m}$

$$x = \frac{10 (20)^{3/2}}{(20)^{3/2} - (10)^{3/2}} \left\{ 1 - \left(\frac{10}{B_x} \right)^{3/2} \right\}$$

or

$$x = 15.45 \left\{ 1 - \left(\frac{10}{B_x} \right)^{3/2} \right\}$$

The values of x for various values of B_x are calculated from the above relation, and are tabulated below :

B_x	10	11	12	13	14	15	16	17	18	19	20
x	0	2.12	3.73	4.97	6.06	7.06	7.84	8.5	9.00	9.60	10

The distance x is measured from the flumed section, as shown in Fig. 19.5.

Step 5. Design of expansion transition

The expansion transition will also be designed on the basis of Chaturvedi's formula:

$$x = \frac{L B_0^{3/2}}{B_0^{3/2} - B_f^{3/2}} \left\{ 1 - \left(\frac{B_f}{B_x} \right)^{3/2} \right\}$$

Here

$$L = 15 ; B_f = 10 ; B_0 = 20$$

$$x = \frac{15 (20)^{3/2}}{(20)^{3/2} - (10)^{3/2}} \left\{ 1 - \left(\frac{10}{B_x} \right)^{3/2} \right\} = 23.15 \left\{ 1 - \left(\frac{10}{B_x} \right)^{3/2} \right\}$$

The value of x for various values of B_x are calculated from the above relation, and are tabulated below :

IRRIGATION AND WATER POWER ENGINEERING

884

S_1	10	11	12	13	14	15	16	17	18	19	20
x	0	3.18	5.60	7.46	9.12	10.6	11.75	12.75	13.60	14.4	15

Step 6. Design of the trough

Flumed water way of canal = 10 m. The trough carrying the canal water will be divided into two compartments, each 5 m wide separated by a 0.3 m thick partition wall. The road way, 5 m wide shall be carried over one of the compartments. The height of the trough will be kept equal to 1.5 m + 0.5 = 2 m. The entire trough section will be constructed in monolithic reinforced concrete and can be designed by the usual structural methods. The outer walls and the bottom slab of the trough may be tentatively kept equal to 0.4 m thick. Thus, the outer width of the trough (and, hence, the length of the syphon barrel) = $10 + 0.3 + (2 \times 0.4) \approx 11.1$ m.

Step 7. Head loss through syphon barrels

The loss of head through the syphon barrels can be calculated from the Unwin's formula:

$$h = \left(1 + f_1 + f_2 \frac{L}{R} \right) \frac{V^2}{2g}$$

where V = Velocity through barrels = 2.05 m/sec.

f_1 = Coefficient of loss of head at entry = 0.505

$$f_2 = a \left(1 + \frac{b}{R} \right)$$

$a = 0.00316$ and $b = 0.030$ for cement plaster.

$$R = \frac{A}{P} = \frac{8 \times 2.5}{2(8 + 2.5)} = 0.765 \text{ m}$$

L = Length of barrel = 11.1 m.

Substituting these values, we get

$$h = \left[1 + 0.505 + 0.00316 \times \left(1 + \frac{0.030}{0.765} \right) \left(\frac{11.1}{0.765} \right) \right] \frac{(2.05)^2}{2 \times 9.81} = 0.333 \text{ m}$$

Downstream H.F.L. = 160.50 (given)

$$\therefore \text{u/s H.F.L.} = \text{d/s H.F.L.} + \text{Loss of head} \\ = 160.50 + 0.333 = 160.833 \text{ m.}$$

Step 8. Uplift pressure on the roof

$$\begin{aligned} \text{R.L. of bottom of trough} &= \text{R.L. of canal bed} - \text{Slab thickness} \\ &= 160.00 - 0.40 = 159.60 \text{ m.} \end{aligned}$$

Loss of head at the entry of the barrel

$$= 0.505 \frac{V^2}{2g} = 0.505 \frac{(2.05)^2}{2 \times 9.81} = 0.108 \text{ m.}$$

$$\text{Uplift on the roof} = 160.833 - 0.108 - 159.60 = 1.125 \text{ m.}$$

CROSS DRAINAGE WORKS

885

or Uplift pressure = $1.125 \times 9.81 \approx 11.04 \text{ kN/m}^2$.

The trough slab is 0.4 m thick, and exerts a downward load of $0.4 \times 2.4 \times 9.81 = 9.42 \text{ kN/m}^2$.

The balance of the uplift pressure = $11.04 - 9.42 = 1.62 \text{ kN/m}^2$ has to be resisted by the bending action of the trough slab, and for this, suitable reinforcement has to be provided at the top of the slab. Since the trough as a whole is heavier than the total upward force, no anchorage to piers is necessary. When the water level in the drainage is low, trough slab is to be designed for the full water load of the canal, and suitable reinforcement should be provided at the bottom of the slab.

Step 8. Uplift on the floor of the barrel and its design**(a) Static head**

$$\begin{aligned} \text{R.L. of barrel floor} &= \text{R.L. of bottom of trough slab} - \text{Height of barrel} \\ &= 159.60 - 2.50 = 157.10 \text{ m.} \end{aligned}$$

Let the tentative thickness of the floor be equal to 0.8 m.

$$\therefore \text{R.L. of bottom of floor} = 157.10 - 0.8 = 156.30 \text{ m}$$

Drainage bed level = 158.00 m (given)

Assuming the subsoil level water upto the bed level of the drain, the static uplift, on the floor

$$= 158.00 - 156.30 = 1.70 \text{ m}$$

(b) Seepage head

The residual seepage head at the centre of the first barrel (point B) can be calculated with reference to Fig. 19.9. The seepage line, according to Bligh's theory,

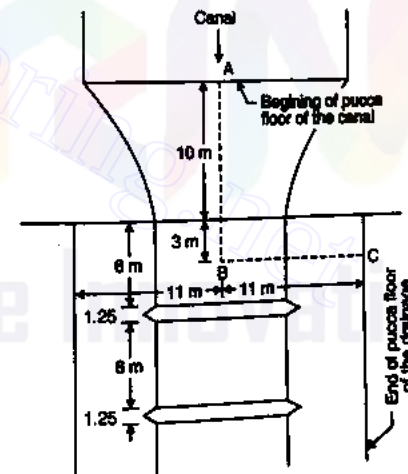


FIG. 19.9. SEEPAGE HEAD

(a) Length of wave transition = 10 m.
Length of wave = 3 m.

- (i) Length of the transverse
- (ii) Half the barrel span = 3 m.
- (iii) End of drainage floor from the centre of barrel, assuming a total length of 22 m = 11 m.

Total creep length = 24 m.

Creep length upto the centre of the barrel = 13 m.

Total seepage head = Canal F.S.L. - R.L. of drainage bed
 $= 161.5 - 158.00 = 3.50 \text{ m}$

Residual seepage head at B = $3.50 \left(1 - \frac{31}{24} \right) = 1.6 \text{ m}$

$$\begin{aligned}\text{Total uplift} &= \text{Static head} + \text{Seepage head} \\ &= 1.70 + 1.6 = 3.3 \text{ m of head of water} \\ &= 3.3 \times 9.81 = 32.37 \text{ kN/m}^2\end{aligned}$$

Provide thickness of the floor equal to 0.8 m ; the uplift resisted by the self weight = $0.8 \times 2.24 \times 9.81 = 17.58 \text{ kN/m}^2$.

∴ Balance to be resisted by the bending action

$$= 33.37 - 17.58 = 14.79 \text{ kN/m}^2.$$

Suitable reinforcement will have to be provided at the top of the floor to counteract the bending action.

Note. The length of the floor has been kept as 22 m on the following considerations:

- Note: The length of the floor has been kept as 11.1 m.
- | | |
|---|----------|
| (i) Length of the floor under the barrel | = 11.1 m |
| (ii) Extra floor to accommodate the piers on both sides, keeping the projection of the piers to be 0.9 m | = 1.8 m |
| (iii) Horizontal length of the d/s ramp joining to the bed level, at a slope 1 : 5 = $5 \times (158.00 - 157.10)$ | = 4.5 m |
| (iv) Width of d/s cut off beyond ramp | = 0.5 m |
| (v) Length of the extra floor to the u/s of the piers | = 4.1 m |

Total = 22 m

The cross-section of the siphon aqueduct is shown in Fig. 19.10.

The scour depth $R = 0.47 \left(\frac{Q}{f} \right)^{1/3} = 0.47 \left(\frac{400}{1} \right)^{1/3} = 3.46 \text{ m}$

Depth of w/s cutoff, below H.F.L.

$$\approx 1.5 R = 1.5 \times 3.46 \approx 5.2 \text{ m}$$

R.L. of bottom of w/s cutoff

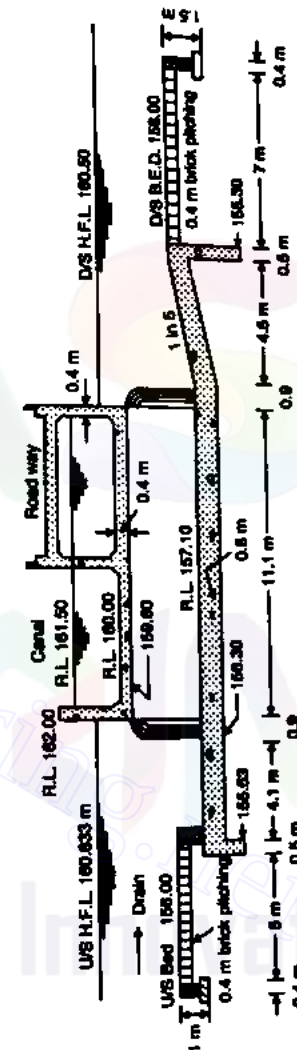
$$= \text{w/o. H.F.L.} - 5.2 = 160.833 - 2.5 = 155.633 \approx 155.63 \text{ m}$$


FIG. 19.10. CROSS-SECTION OF SYPHON AQUEDUCT.

IRRIGATION AND WATER POWER ENGINEERING

664

Depth of d/s cutoff, below H.F.L.

$$= 1.5 R = 1.5 \times 3.48 = 5.2 \text{ m}$$

R.L. of bottom of d/s cutoff

$$= \text{d/s H.F.L.} - 5.2 = 160.50 - 5.2 = 155.30 \text{ m}$$

Length of d/s protection consisting of 40 cm brick pitching

$$= 2.5 (\text{R.L. of d/s bed} - \text{R.L. of bottom of d/s cutoff})$$

$$= 2.5 (158.00 - 155.30) = 6.76 \text{ m} \approx 7 \text{ m.}$$

The pitching may be supported by a toe wall 0.4 m wide and 1.5 m deep at its d/s end. Length of d/s protection consisting of 0.4 cm brick pitching

$$= 2 (\text{R.L. of u/s bed} - \text{R.L. of bottom of u/s cutoff})$$

$$= 2 (158.00 - 155.62) = 4.76 \text{ m} \approx 5 \text{ m.}$$

This pitching may be supported by a toe wall 0.4 m wide and 1 m deep at its u/s end.

The cross-section of the syphon aqueduct is shown in Fig. 19.10.

PROBLEMS

1. Describe with the help of sketches various types of cross-drainage works.
2. Differentiate between (i) syphon aqueduct and canal syphon, (ii) aqueduct and super passage.
3. What do you understand by level crossing ?
4. Write a note on selection of suitable type of cross-drainage works.
5. Classify aqueducts and explain under what circumstances each one is used.
6. Explain the method of fixation of water-way of drain in an aqueduct.
7. Explain Mitra's method of hyperbolic transition. Derive the equation you use.
8. Explain the Hind's method of designing canal transitions.
9. Explain the method of determining uplift pressure on the roof of a syphon aqueduct.
10. Give Bligh's approximate method of determining uplift pressure under the floor of a cross-drainage work.
11. What is meant by bank connections ? How are they designed ?

SECTION-IV

MISCELLANEOUS TOPICS

Chapters :

20. RIVER ENGINEERING
21. WATER POWER ENGINEERING
22. WATER RESOURCES PLANNING
23. IMPORTANT DAMS OF INDIA

River Engineering

20.1. CLASSIFICATION OF RIVERS

Rivers can be divided into two parts according to the topography of river basin, namely :

- (a) Upper reaches in the hilly region.
- (b) Lower reaches in the alluvial plain.

Rivers in the hilly region are characterised by the steepness of their slope, the swiftness of flow, the occurrence of land slides and the formation of rapids along their courses.

Alluvial rivers are characterised by the fact that the alluvia, on which the rivers flow, are built by rivers themselves. Rivers on alluvial plains may be broadly classified into three types :

- (1) The meandering type,
- (2) The aggrading type,
- (3) The degrading type.

All the three types may be found on a single river from its uppermost point on alluvial plain to its mouth. A particular section of the river may also be aggrading, degrading or meandering at different times depending upon the variation of sediment load and discharge. Among the three-types of rivers mentioned above, the meandering type is the full and final stage of river development while the other two are interim type.

20.2. MEANDERING

A river meandering through an alluvial plain has a series of consecutive curves of reversed order connected with short straight stretches called *crossing*. *Width of meandering belt M_B* is the transverse

distance between the apex point of one curve and apex point on reverse curve. *Length of meander M_L* . It is the air distance along river between tangent point of one curve and tangent point of other curve of same order.



FIG 20.1 MEANDERING OF RIVER.

Degree of Sinuosity. It is the ratio between the curved length and the straight air distance.

20.3. CAUSES OF MEANDERING

Primary cause of meandering is the excess of total charge during floods, when excess of turbulence is developed. Experiments have shown that when charge is in excess of that required for stability, a river tends to build a steeper slope by depositing excess of that required for stability, a river tends to increase, in its turn, the width the charge on the bed. This increase in slope tends to increase, in its turn, the width of the channel if the banks are not resistant. Only a slight deviation from uniform axial flow is then necessary to cause more flow towards one bank than towards the other. Additional flow is immediately attracted towards the former banks, leading to shoaling along the latter, accentuating the curvature of flow and producing, finally, meander in its wake.

The cause given in the paragraph above is due to the Inglis. According to J.S. Firdkin, meandering results from the local bank erosion and consequent over loading deposition by the river of the heavier sediments moving along the bed.

20.4. BASIC FACTORS CONTROLLING PROCESS OF MEANDERING

The basic factors controlling process of meandering are given below :

1. **Valley slope.** This is overall slope of terrain traversed by the stream and is measured down the axis of the valley. A change in the valley slope always produces change in the meander pattern.
2. **Stream load.** Composition of stream load as well as its rate of movement materially affects the meander pattern.
3. **Discharge.** This concerns seasonal stream flow variations as represented by the average hydrograph. There is a close relationship between the rate of discharge and the rate of bed load movement and, therefore, it affects meandering.
4. **Bed and side resistance.** Boundary resistance are characterised by the nature of materials composing the alluvium, more particularly by its resistance to erosion. Grain size, specific gravity, cohesion and roughness are important factors. Grain sizes may range from large boulders and pebbles in head reaches to fine sand and silt in lower reaches of a river. In a river with gravel and boulder bed, no definite meander pattern is observed as the river has a tendency to form interlaced channels.

20.5. GENERAL FEATURES OF MEANDERING

The general features of meandering have been worked out by various experiments on meandering rivers. The same are discussed below :

1. A fully developed meander of homogeneous valley material with a constant discharge has a definite pattern of curvature, length, width and depth of the channel is reproducible in models.
2. The width and length of meander as well as width of river vary roughly with square root of discharge.
3. The pattern of meander is altered by variation in discharge slope or bank and bed forming material.

4. Increase in discharge or slope increases the meandering feature viz size of bends, their length, width and degree of sinuosity.
 5. Increase of sediment charge increases the slope as well as width of the river, while the depth of the river is reduced.
- It can be summarised that every phase of meandering depends upon the following three closely related factors :

- (i) The discharge and hydraulic properties of the channel
- (ii) The sediment load.
- (iii) The relative erodibility of bed and bank.

Generally speaking, where a river has enough capacity to carry the incoming sediment downstream with forming large deposits, the whole or a part of it would be meandering type. The relations between M_1 , M_2 , W and Q as arrived at by Inglis after analysing data on Indian rivers are given in Table 20.1.

TABLE 20.1. INTER-RELATIONSHIP BETWEEN M_1 , M_2 , W AND Q

M_1 , M_2 or W (metre)	Rivers in flood plain	Inland rivers (Rivers flowing in a cross-section cut below the natural ground surface)
M_1	$6.06 W$	$11.46 W$
M_2	$17.40 W$	$27.30 W$
M_2	$2.86 M_1$	$2.3 M_1$
M_1	$53.25 \sqrt{Q}$	$48 \sqrt{Q}$
M_2	$153.75 \sqrt{Q}$	$102.28 \sqrt{Q}$
W	$8.87 \sqrt{Q}$	$4.54 \sqrt{Q}$

Q is discharge in cumecs.

20.6. THE AGGRADING TYPE OF RIVER

The aggrading type of river is building up its bed to a certain slope. This may be because of :

1. Excessive sediment entering a river with a sudden diminution of slope on the plain.
 2. Excessive sediment entering a river with the slope flattened because of some obstruction like barrage or a dam across it.
 3. Extension of delta at the river mouth.
 4. The sudden inrush of sediment from a tributary.
- A river of aggrading type or a section of it, has usually a straight or wide reach with shoals in the middle.

20.7. DEGRADING TYPE OF RIVER

The degrading type of river is in the process of losing its bed gradually in the form of sediment load of the stream. The reason associated with the degradation of river is sudden diminution of sediment load due to deposition of bed upstream of a barrage or a dam. The water falling downstream of the dam scours the bed and replenishes the sediment load. For the same reason, meandering type of river develops

2. Groynes or spurs,
3. Levees or embankment,
4. Bank protection and pitched banks,
5. Pitched islands,
6. Miscellaneous methods,

- (i) River training without embankments
- (ii) Bandalling.

20.12. GUIDE BANK SYSTEM

Guide banks are made for guiding the stream near a structure so as to confine it in a reasonable width of the river. It was first designed by Bell in whose honour it is also sometimes known as *Bell's Bund*. The design was further developed by Spring and is known as guide bank.

The guide bank usually consists of a heavily built embankment in the shape of bell mouth on both sides of constricted channel. Usually only one embankment is required if the other side is a high and stable bank. The remaining side of the river *khadir* is covered by ordinary embankment.

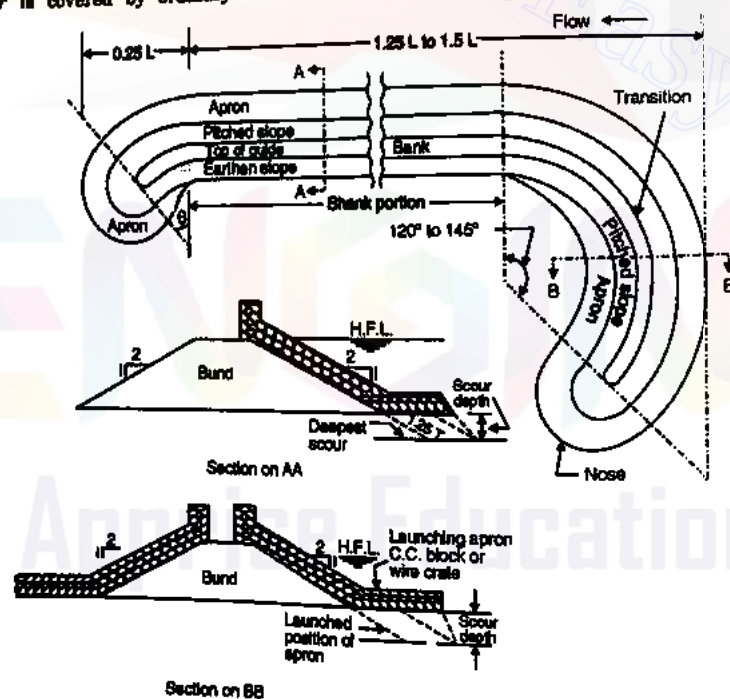


FIG. 20.3. GUIDE BANK.

Design considerations

The design of a guide bank involves the following considerations:

- (a) The ultimate width to which an alluvial river can be constricted may be computed from the relation

$$L = 4.75 \sqrt{Q}$$

where Q is estimated maximum discharge in cumecs; L is the width of the channel in metres.

An extra allowance of 20% may be made for thickness of bridge piers etc.

- (b) Length of upstream guide bank $\approx 1.25 L$ to $1.5 L$.

- (c) Length of downstream guide Bank $= 0.25 L$.

A symmetrical layout of the guide bank in plan is very essential for the successful performance of the bund. The two guide banks on both sides of stream can be made (a) parallel, (b) diverging, (c) converging at upper end, (d) single guide bank depending upon local conditions, although Bell recommended a diverging form with an area of narrowest part of upstream end equal to the effective area at bridge site. Both upstream and downstream ends of the banks are curved gently so as to lead the flow along smooth lines.

- (d) The main parts of a guide bank are:

- (i) U/S curved head or impregnable head
- (ii) D/S curved head
- (iii) Shank or a straight portion which join two curved heads
- (iv) Slope and bed protection.
- (i) *U/S curved head*

On the u/s end the curvature has a central angle of 120° to 140° . The radius of curvature R can be provided as suggested by Spring or Gale or as per an empirical relationship. Spring suggested a value of R equal to 180 m to 250 m for rivers having velocities 2.4 metres per second to 3.1 respectively. Gale on the other hand suggested a value of 250 m for rivers having flood discharge between 7000 to 20,000 cumecs; a value of 580 sharper curves are permissible for discharge less than 7000 cumecs. A value of 580 m is recommended for discharge varying from 40,000 to 70,000 cumecs. The radius may be obtained by interpolation for discharge between 40,000 to 20,000 cumecs.

It has also been suggested that the value of R can be calculated from the relation

$$R = 0.45 L$$

- (ii) *D/S curved head*

On the d/s the river fans out to attain the normal width. A proper shape ensures safety of approach embankment. Radius of curvature of D/S head may be kept as half of the u/s radius with a central angle of 45° to 60° .

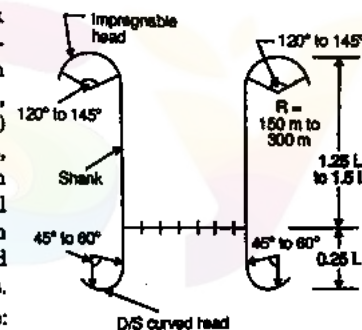


FIG. 20.4. LAYOUT OF GUIDE BANK.

(iii) *Section of guide bund and material of construction*

The material of construction of guide bank is the river sand available locally. Side slope is generally kept 2 : 1 to 3 : 1. It should have sufficient top width which should not be less than 4 m so as to provide sufficient carriage way. The free board usually adopted for the design varies from 1.25 to 1.50 m. The inside slope is protected by stone pitching, the total thickness of which varies from 0.4 m to 0.6 m. The stone pitching is 1 m above H.F.L. The thickness of stone pitching (T) as recommended by Inglis is given by

$$T = 0.06 (Q)^{1/3}$$

The thickness of the pitching should be 25% more at the impregnable head than for the rest of the bund.

The rear side of the shank portion is not pitched, but is generally coated with 0.3 m to 0.6 m earth for encouraging vegetation growth so as to make it resistant against wind and rain.

(iv) *Slope and toe protection*

The toe of the slope is protected by the so called "falling apron" or "launching apron". The quantity of stone in the launching apron should be sufficient to cover a scoured face fully after the apron fans out while launching at a slope of 2 : 1 with a thickness of 1.25 T . It is generally laid in a width equal to 1.5 D where D is the depth of maximum anticipated scour below the bed, and has a thickness of 1.9 T . The maximum anticipated scour below H.F.L. is taken as $C.R$ where R is the Lacey's normal scour depth given by the relation

$$R = 0.47 \left(\frac{Q}{f} \right)^{1/3}$$

where Q is the discharge and f is the silt factor. The value of C is generally assumed as 2.25 at the nose of the guide banks, 1.50 at the transition from nose to the straight portion and 1.25 in the straight reaches of the guide bund. Inglis and Joglekar gave the following values for maximum scour depth :

(i) Maximum scour depth downstream of bridge :

$$D_s = 1.9 \left(\frac{Q}{f} \right)^{1/3}$$

(ii) Maximum scour depth around bridge piers :

$$D_s = 0.95 \left(\frac{Q}{f} \right)^{1/3}$$

(iii) Maximum scour depth at nose of guide banks of a large radius :

$$D_s = 1.3 \left(\frac{Q}{f} \right)^{1/3}$$

The usual types of guide banks provided in India are as given below (Table 20.2) :

TABLE 20.2. TYPES OF GUIDE BANKS

S.No.	Type of Bank	Place
1.	Guide Bank parallel but not perpendicular to Weir	Rasul Weir on Jhelum
2.	Parallel guide banks with curved heads	Punjab Weir on Sutlej
3.	Bottle neck guide banks	Sulemani Weir on Sutlej Khanki Weir on Chenab
4.	Splayed guide banks	Islam Weir on Sutlej
5.	Parallel straight guide bank	Ferozpur Weir on Sutlej

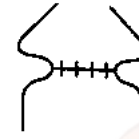


FIG. 20.6

20.13. GROYNES OR SPURS

Groynes are structures constructed transverse to the river flow and extend from the bank into river upto a limit. These groynes are also known as spurs, dikes and transverse dikes.

Type of Groynes : Groynes can be classified in the following ways :

1. *Classification according to the material of construction*

- Permeable groyne.
- Solid impermeable groyne.

2. *Classification according to its height below high water*

- Submerged groyne.
- Non-submerged groyne.

3. *Classification according to the function it serves*

- Attracting groyne.
- Deflecting groyne.
- Repelling groyne.
- Sedimenting groyne.

4. *Special types of groynes, such as*

- Denehy's T-headed groyne.
- Hockey type groyne.
- Burma type groyne.

Choice of a groyne : Factors which influence the choice and design of groynes are

- Fall velocity of river.
- Width of water way, at high water, mean water, and low water.
- Depth of water way, height of flood rise and nature of flood hydrograph.
- Character of bed material such as shingles, boulders, sand or silt.

- (5) Available material and funds.
 (6) Amount of silt carried in stream.

Permeable groynes are suitable for silt laden river. If the river has to be confined in a defined channel, generally impermeable groynes are used.

Purpose of a groyne : Groynes serve a four-fold purposes, namely :

- (1) Contracts a river channel to improve its depth.
- (2) Protects the river bank.
- (3) Silts up the area in the vicinity by creating a slack flow.
- (4) Trains the flow along a certain course.

Depending upon the purpose, groynes can be used with other training measures.

20.14. REPELLING GROUYNE

The repelling groyne is constructed in such a way that it is pointing towards upstream at an angle of 10° to 30° to the line normal to the bank. The head of groyne causes the current to be deflected in a direction nearly perpendicular to itself. The current coming into contact with still water area adjacent to the spur causes vertical eddies and deep scour. The head of groyne should be strong to resist the swirling action of the current. In case of repelling groyne, a still water pocket is formed upstream of it and suspended load brought down by the river gets deposited in the pocket. Rapid success of the groynes depends upon how quickly still water pockets between groynes are filled with sediment, which can be accomplished only when they are sufficiently long.

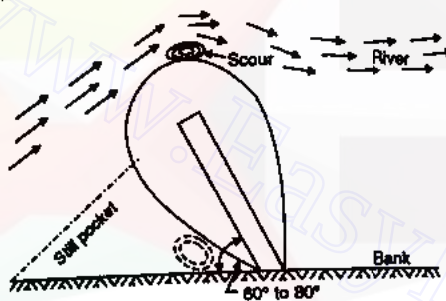


FIG. 20.6. REPELLING GROUYNE.

20.15. DEFLECTING GROYNES

A deflecting groyne has a much shorter length than a repelling groyne and it is generally taken in a river perpendicular to the bank it only deflects the flow.

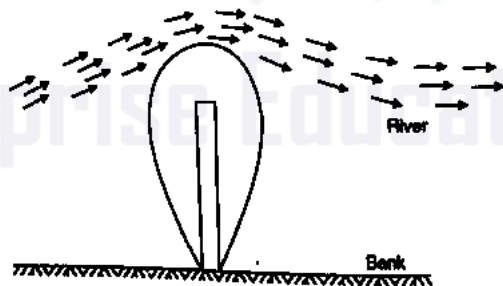


FIG. 20.7. DEFLECTING GROUYNE.

20.16. ATTRACTING GROUYNE

The attracting groyne is constructed in such a way that it points downstream the direction of normal flow. When a groyne points downstream, it causes scour holes to form closer to the bank than the groyne inclined at right angles ; therefore, they tend to maintain the deep current close to the bank. The attracting groyne bears the full fury of the frontal attack of the river on its upstream face and has, therefore, to be adequately strong. An equally heavy protection is not necessary on the downstream slope.

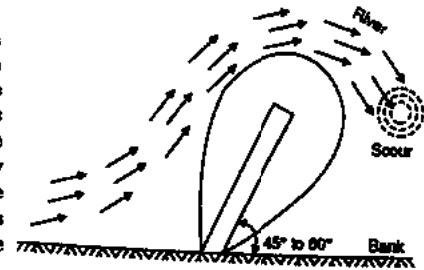


FIG. 20.8. ATTRACTING GROUYNE.

20.17. LENGTH OF GROYNES

The length of groynes depends upon the magnitude of river training required. If the current is to be thrown back on the other bank, naturally longer length of groyne is required. A long groyne constructed out of easily erodible material, however, may fall. Hence, it may be worthwhile to start with a short length and extend the groynes gradually as silting between them proceeds.

20.18. SPACING OF GROYNES

It is general practice to space the groynes by 2 to 2.5 times the length of groyne. The spacing may be altered under the following conditions :

- (1) The spacing can be farther apart on convex bund than on concave bund.
- (2) The spacing is also farther apart for wider rivers than for narrow ones of the same discharge.

- (3) Permeable spurs are spaced farther and impermeable spurs comparatively closer.

Section of Groynes

The section of groyne is similar to that shown in Fig. 20.9.

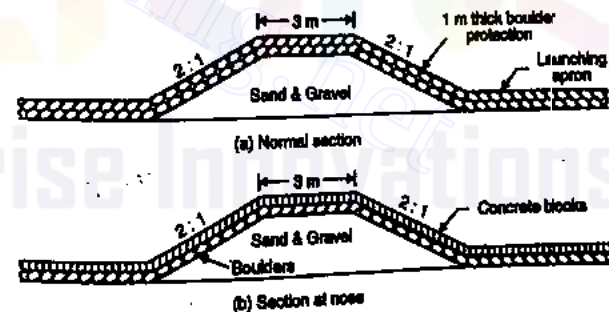


FIG. 20.9. SECTION OF GROUYNE.

20.19. IMPERMEABLE GROYNES

Impermeable or solid groynes do not permit any flow through them. They are constructed by a core of locally available material like sand, clay and gravel and heavily protected on the sides by resistant materials like stone pitching or concrete blocks. The groyne section has a side slope varying from 2 : 1 to 3 : 1 and front slope varies from 3 : 1 to 5 : 1. An apron must be provided at the bottom of the slope to prevent slipping of stones. They can be designed as any one of the repelling, deflecting or attracting groynes.

20.20. PERMEABLE GROYNES

They permit the flow of water through them. They dampen the velocity and thus reduce the erosive action of the stream. In case river water carries sediment load, the groyne cause deposition of sediment near it because of reduction of velocity.

Advantages of permeable groyne. The basic advantages associated with a permeable groyne are given below :

1. The groyne is very cheap in cost of construction.
2. Small quantity of stone is required for its construction and hence, it is useful where stone is scarce.
3. The permeable groyne has a better performance as compared to solid impermeable groynes.
4. The groyne does not change the flow abruptly, hence, there is no serious formation of eddy and scour.
5. It is more suitable for deep narrow rivers.
6. Submerged permeable groynes do not create turbulence and eddy conditions as is the case with submerged solid groynes.

Disadvantages. They are not strong to resist shock and pressures from debris of floating ice logs.

Types of permeable groynes. The permeable groyne is either a

1. Tree groyne or
2. Pile groyne.

Tree Groyne. A tree groyne consists of trees held in position by thick weir ropes anchored firmly on the bank and tied to a heavy buoy. The trees used for groynes should be very leafy with abundant branches like the babul. Roots, branches and twigs help to trap sediment by reducing the flow velocity. A tree groyne should be completed before the early floods. Tree groynes have been successfully used :

1. To divert or deflect the current which is directly threatening erosion of a bank.
2. For closing a river channel and opening the other.
3. To silt up a channel of the river at its source by checking the flow in it.

Pile Groyne. These groynes are constructed with timber piles driven down the river bed upto 9 metres depth. The groynes consist of main verticals in two or three rows braced together by transverses and diagonals. The timber main verticals are spaced 2 to 3 m apart and in 2 or 3 similar rows separated 1.5 to 2 m apart. Between

the main verticals there can be two intermediates embedded at least 1.5 m below the bed. These are filled with alternate layers of brush woods and stones. Sometimes they catch silt, become sand bound and start acting as solid groyne. To safeguard it against scour in such a case it is protected by stone apron 1 m thick having a length of 3 m along shank and 6 m along the nose.

20.21. DENEHY'S GROYNES

These are special groynes first provided at Okhla barrage and then at Narora Weir. At Narora Weir the groynes are earthen embankment projecting from the marginal bund. The head of groynes has T shape of which the front perpendicular arm about 100 m in length was parallel to current. A longer portion of the head is on upstream

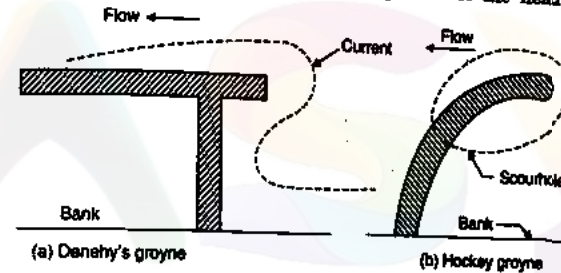


FIG. 20.10. DENEHY'S GROYPE AND HOCKEY GROYPE.

side so that it looks more like a reversed L [Fig. 20.10(c)]. The head has a slope on both sides and is heavily protected by stone. They are spaced 800 m apart. The T heads are on a straight line or on a regular curve along 6.4 km length upstream and 25.6 km length downstream of headworks at Narora. These groynes aim at an economy in the use of protection stone since only 100 m length of the groyne would suffice for a length of 800 m.

20.22. HOCKEY GROYPE

Amongst the various designs for groyne heads, a groyne with a curved head [Fig. 20.10(b)], is termed as a hockey groyne. It increases the attracting tendency of the groyne and is not likely to be helpful for bank protection.

20.23. MARGINAL BUND OR LEVEES

Marginal bund or Levee is an earthen dike constructed roughly parallel to the river rather than across the channel.

Effect of Levees on flood flows

The effect of levees on flood is summarised in the following points :

1. Increases the intensity of flood water travel downstream.
2. Increases the water surface elevation of the river at flood.
3. Increases maximum discharge at all points downstream.
4. Increases the velocity and scouring action through leveed section.

IRRIGATION AND WATER POWER ENGINEERING

584

5. Decreases the surface slope of the stream above the leveed portion.
6. Increases the flood peak and decreases the valley storage.

Planning of levees

A levee should be planned with following points under consideration :

1. Levees should be provided on areas which are lying low and, hence, the danger from the flood is pronounced.
2. A sufficient channel must be provided in levees to transmit design flow with reasonable freeboard against wave action. The cost of levee system consists of costs of land for levees and channel plus the cost of levee construction. Hence, the channel width and levee height combination which gives minimum cost is to be determined by trial.
3. Levees should be placed on the high ground. Full advantage should be taken of ridges, which permit lower levees and better foundation condition.
4. The levee should be tied down to the high grounds.
5. The levee is roughly spaced equidistant from midstream.

Design of Levee

The design of a levee is discussed in the following steps :

(a) **Spacing and height** : Spacing and height of a levee are interdependent. More height will have levees closer together while lower height will keep them apart. A provision of free board of 1 to 1.25 m should be made over H.F.L.

(b) **Top width** : Top width is decided as per the use to which it is put to. Minimum width = 3 m.

(c) **Side slope** : The side slope provided in a levee depends upon :

- (i) Nature of material of which levee is composed.
- (ii) Method of construction.
- (iii) Length of time the levee is likely to be subjected to a wave action.

The usual slop provided is as given below :

1. Slope on river side 3 : 1 to 5 : 1.
2. Land side 4 : 1 to 7 : 1.

(d) **Banquette** : A banquette is a terrace of earth added to the base of a high levees on the land side to prevent danger of slouging from seepage.

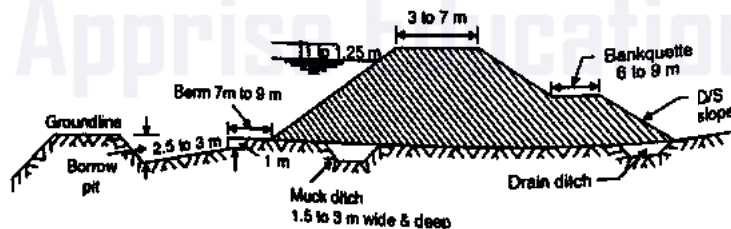


FIG. 20.11. SECTION OF A LEVEE.

RIVER ENGINEERING

585

(e) **Muck ditch** : They are trenches filled with most impervious material at hand, to form cutoff to check the seepage along the foundation plane.

(f) **Drain Ditch** : Any seepage through the levee is collected and disposed off through the drain ditch.

Causes of failure of a Levee. There may be many reasons of the failure of a levee, important amongst them are :

1. Caving of the stream bank may result in undercutting of the river-ward toe of the levee.
2. Seepage through foundation material at high stage of river can cause a sand boil and removal of foundation material by piping.
3. Overtopping during high floods may cause its ultimate collapse.
4. The foundation may fail due to infiltration.
5. Piping as a result of insufficient cross-section leaks and cracks due to shrinkage of soil and rat holes.

Advantages of river training by embankment

1. They are the only means of preventing the inundation by tides.
2. They are initially cheaper in the cost of construction though subsequent raising may make it more expensive.
3. The construction and maintenance is easy.
4. They can be executed in parts, but the ends should be well protected.

Disadvantages

The various disadvantages of river training by embankment are as mentioned below :

1. Embankment cause a rise in high flood level.
2. Gopher holes or channels left by decay or roots may permit the start of destructive erosion leading to eventual levee failure or crevasse.
3. They require heavy maintenance.
4. They fall suddenly and inundate the low lying areas.
5. They are susceptible to direct attack of river flow which can erode or undermine them.
6. The interior drainage of the area is enclosed by embankment which may give rise to material climate.
7. They check the deposition of silt on flood plain, thus hastens delta formation resulting in increase of flood stages and rate of aggradation of river bed.

20.24. ARTIFICIAL CUTOFF

To improve upon the flood capacity of a given reach and thereby reduce flood level in certain reaches an artificial cutoff may be introduced as a river training measure. This is generally done when the loop length exceeds $1\frac{1}{2}$ to $2\frac{1}{2}$ times the chord length. In the beginning a pilot cut is introduced. The pilot cut should carry 8 to 10% discharge in the beginning and are permitted subsequently to develop the carrying to about 40

to 50% of the total river discharge. Its design alignment and cross section is governed by the following considerations :

1. The direction of cut should be tangential to the main direction of flow.
2. Alignment of pilot cut should preferably be made at a curvature much flatter than the curvature of river itself.
3. Entrance to cutoff should be given bellmouth shape.
4. Sometimes if the cut is not excavated to develop by scour, they should be excavated initially as a main river channel.

5. According to the regime formula by Lacey, RS^2 depends wholly upon velocity. So that to have scouring velocities, RS^2 for cutoff should be higher than the river. Since head difference is same for river and cutoff, the slope is inversely proportional to the length of channel. Hence, R/L^2 for the pilot cut should be much larger than R/L^2 for the river.

6. The pilot section should be made very deep as deeper section develop rapidly.

20.25. PITCHED ISLANDS

This consists generally of a sand core pitched with boulders along its side and slopes and protected at the toe by a falling apron. Sometimes, advantages can be taken of an existing natural island (a bela) which can be re-shaped and pitched. The pitched island (Fig. 20.12) causes redistribution of velocity and tractive force. The tractive force near a pitched island begins to increase rapidly after the construction of island, with the result that deep scour begins to form round the island gradually draws the river

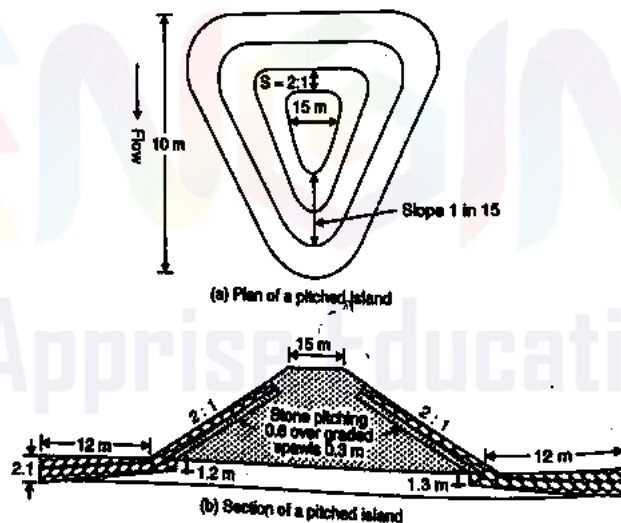


FIG. 20.12. PITCHED ISLAND.

channel into itself and ultimately holds it permanently. As deep scour occurs into its vicinity the boulder pitching and apron have to be designed for maximum scour.

$$\text{Maximum scour} = 2R = 2.70(q^2/f)^{1/3}$$

It is used for the following purposes :

- (i) Training of the river to have an axial flow.
- (ii) Rectification of adverse curvature for effective sand exclusion.
- (iii) Improving channel for navigation.

The device is not likely to be successful in shallow and wide rivers where obstruction caused by island is so small that scour cannot form. Similarly, in case of very flashy rivers the scour formed while rising is filled quickly during falling stage.

20.26. BANK PROTECTION

In a broad sense bank protection includes any protection work that aims at maintaining the stability of land against the action of water.

Purpose of Bank Protection

The purpose of bank protection is to save the banks from forces of flood. The purpose thus can be brought out in the following points :

- (i) To check erosion of bank by current or waves.
- (ii) To check sliding of slope due to its gradual steepening because of erosion.
- (iii) To prevent undermining of the toe of the lower bank by currents, waves, swirls or eddies.
- (iv) To prevent sloughing or sliding of slope when saturated with water.
- (v) To prevent piping due to various causes.
- (vi) To check sliding due to drawdown of the flood.

Methods of Bank Protection

The following methods of bank protection are in general practice :

1. Protection of bank by vegetal cover, either by turfing or sodding. Low growth of shrubs and willows is by far the most effective cover. The method should be used if the slope is not subjected to strong current.
2. Pavement of bank slopes by such material which can resist quick erosion. Along with pavement the slope is given a temporary covering with rush wood or lumber mattress weighed down by stones for energy purposes. The method is utilized if the current is strong.
3. When the current is very strong, protection is provided by stone revetment or various types of mattresses such as willow, lumber or asphalt or articulated concrete.

20.27. PITCHED BANK

A pitched bank may be created in part or full to attract the flow and hold the river at a desired place. The positioning of pitched bank has to be determined with the consideration of river meanders upstream and downstream. An artificial control point such as groyne upstream of pitched bank may be necessary to make the measure

permanent. The pitched bank has a tendency to pull the river to itself, though this tendency depends upon the extent of scour which occurs at toe. The pitched bank must be protected by falling stone apron.

20.28. BANDALLING

Bandalling is an impervious system of river training. It is designed to confine the low water flow in a single channel with a view to maintain suitable depth for navigation.

A *bandal* consists of a frame-work of bamboos driven in the river bed and set 0.6 m apart with horizontal ties supported by struts placed at every 1.2 m. To this bamboo frame-work, *Jhamps* are tied with coire rope to the horizontal ties. The bamboo used on the frame-work are usually 3 to 6 m in length and *Jhamps* are made of bamboo mats from 0.75 to 1.0 m wide strengthened at the edges by strips of split bamboo. Bandals are placed at an angle 30° to 40° inclined downstream to direct an additional volume of water into the inclined downstream to direct an additional volume of water into the desired channel and to increase the velocity in it. The bandals do not stop the flow of water but merely provides an impediment to it. Their action leads to the deposition of silt in ridges parallel to and behind the bandals. Thus a channel confined between bandals is formed with sand banks on either side.

20.29. NANAL PLANTATION

Nanal is a plant like cane. It is 3 m to 4.5 m height of which bottom 1.5 m is about 12 and 25 mm in diameter and hollow like bamboo. Nanal grass planted along rivers will spread roots and sprouts quickly. The growth of nanal checks the velocity and induces deposition of sediment along the river bank.

Example 20.1. Design a guide bank required for a bridge on a river having the following particulars :

Design flood discharge	= 50000 cumecs
Silt factor	= 1.10
Bed level of river	= 130.00 m
High flood level	= 140.00 m

Also find the volume of stone required per m length of the guide bank.

Solution

$$Q = 50000 \text{ cumecs} ; f = 1.10$$

$$\text{Lacey's water way} = 4.75 \sqrt{50000} = 1062 \text{ m.}$$

Provide 20% more length to account for thickness of piers and end contractions due to piers and abutments

$$= 212 \text{ m}$$

$$\text{Gross length between banks} = L = 1274 \text{ m}$$

$$\text{Upstream length of guide bund} = \frac{5}{4} L = \frac{5}{4} \times 1274 = 1592 \text{ m}$$

$$\text{Downstream length of guide bund} = \frac{L}{4} = \frac{1}{4} \times 1274 = 318 \text{ m}$$

$$\text{Radius of u/s curved head} = 0.45 L = 573 \text{ m.}$$

The u/s end of the guide bank may, therefore, be curved by 145° with a radius of 573 m.

Radius of d/s curved head may be kept as 287 m with an angle of 60° at the centre.

Cross section of guide bund

The given H.F.L. at the bridge site = 140.0 m

Assuming free board = 1.5 m.

Level of top of guide bund equals 141.5 m. To be more safe and making an allowance for further settlement etc., let us adopt the top level of bund as 142.00 m.

$$\text{Height of bund above river bed} = 142.0 - 130.0 = 12.0 \text{ m.}$$

Keep the top width of bund as 4 m and the side slope may be kept as 2:1.

The stone pitching and a launching apron must be provided on the slope on water side for the entire length of the bund. However the rear side also will be pitched for curved portions of the bund.

Design of stone pitching and apron

The thickness of stone pitching is given by

$$T = 0.06 [Q]^{1/3} = 0.06 [50000]^{1/3} = 2.21 \text{ m.}$$

This can be kept 1.0 m above H.F.L. i.e. upto level 141.00 m.

$$\text{Depth of scour } R = 0.47 [Q/f]^{1/3} = 0.47 \left(\frac{50000}{1.1} \right)^{1/3} = 16.77 \text{ m.}$$

For straight reach of guide bund,

$$\text{Maximum scour} = 1.25 R = 1.25 \times 16.77 = 20.97 \text{ m}$$

$$\text{R.L. at maximum anticipated scour} = 140 - 20.97$$

$$= 119.03 \text{ m}$$

$$\text{Depth of maximum scour } D = 130 - 119.03$$

$$= 10.97 \text{ m}$$

$$\text{Length of apron} = 1.5 D = 1.5 \times 10.97$$

$$= 16.45 \text{ m}$$

For curvilinear transition portion of guide bund,

$$\text{Maximum scour} = 1.5 R = 25.16 \text{ m}$$

$$\text{R.L. of maximum scour} = 140 - 25.16 = 114.84 \text{ m}$$

$$\text{Depth of maximum scour } D = 130 - 114.84 \text{ m}$$

$$= 15.16 \text{ m}$$

$$\text{Length of apron} = 1.5 D = 1.5 \times 15.16$$

$$= 22.74 \text{ m}$$

$$\text{Thickness of launching apron} = 1.9 T = 1.9 \times 2.21 = 4.2 \text{ m}$$

Volume of Stones

$$\begin{aligned}\text{At shank : on slope} &= \sqrt{5} (141 - 130) \times T \times 1 \\ &= \sqrt{5} \times 11 \times 2.21 = 54.36 \text{ m}^3/\text{m}\end{aligned}$$

On apron with a slope 2 : 1

In a vertical length of D and a thickness of 1.25 T

$$\begin{aligned}\text{Volume} &= \sqrt{5} \times D \times 1.25 T \times 1 \\ &= \sqrt{5} \times 10.97 \times 1.25 \times 2.21 = 66.76 \text{ m}^3/\text{m}\end{aligned}$$

UIS and DIS curved portions

$$\text{Volume of apron} = \sqrt{5} \times 15.16 \times 1.25 \times 2.21 \times 1 = 93.65 \text{ m}^3/\text{m}$$

Volume on slope = Same as for shank

$$= 54.36 \text{ m}^3/\text{m}$$

Thickness of launching apron

$$= \frac{\text{Volume of apron}}{\text{Width}} = \frac{93.65}{1.5 \times 15.16}$$

$$= 4.12 \text{ m} \triangleq 1.9 T.$$

PROBLEMS

1. Describe various types of river training and protection works.
2. What do you understand by meandering ? What are its causes?
3. Sketch a suitable cross-section of a guide banks as used in river training works. Explain the process of launching of aprons in such works.
4. Describe in brief various types of groynes used for river training. Draw a section of a groyne.
5. Explain with sketches the (i) Denshy's groyne, (ii) Bell's bund. What considerations determine the length of both ?
6. Write a note on the design of a level.
7. What do you understand by a pitched island ?

Water Power Engineering**21.1. GENERAL**

The various sources from which energy can be had are :

- (i) Fuels, such as coal, fuel oils and atomic fuel,
- (ii) Flowing water in rivers and canals,
- (iii) Wind, and
- (iv) Ocean waves and tides.

The use of water power by crude devices dates back to ancient times. The primitive wheels, including the Chinese nora, or float wheel, built of bamboo, with woven paddles are still in use along with the modern development of hydraulic turbines.

India has water power potential of about 41 million kW, out of which 4.4 million kW, (i.e., about 11%) was exploited by 1966. Perhaps the first hydroelectric work in India was undertaken in 1897 near Darjeeling. The State of Mysore constructed, in 1902, the Siva Samudram hydroelectric work with an installed capacity of 4500 kW which was later increased to 42,000 kW. Tatas constructed Khapoli hydroelectric works (1914) with an installed capacity of 50,000 kW, Bhivpuri hydroelectric works (1920) with a capacity 72,000 kW and Bhira scheme (1927) with an installed capacity of 110,000 kW. During the period between the two World Wars, some hydroelectric schemes were taken by various states. They include Jogindernagar hydroelectric scheme (48,000 kW), Ganga Canal power station (17,400 kW), Pykara hydroelectric scheme (38,760 kW), Mettur hydroelectric scheme (30,000 kW) and Pillivasal scheme (13,500 kW). After Second World War and before Independence, only two hydroelectric installations, i.e., Papanasam (21,000 kW) and Jog (48,000 kW) were made. After independence, India has made significant progress in Hydro power development, increasing from 0.5 million kW in 1947 to 4.4 million kW by the end of 1966. The planned capacity by type of plant, by the end of 1966 are as follows.

TABLE 21.1. INSTALLED CAPACITY BY THE TYPE OF PLANT

	1950	1955	1960-61	1965-66
Hydro	0.56	0.94	1.85	4.4
Steam	1.69	2.27	3.40	0.62
Oil	0.15	0.21	0.33	0.4
Total	2.3	3.42	5.58	11.0

(891)

21.2. TYPES OF WATER POWER DEVELOPMENT

A water power development is essentially to utilize the available power in the fall of a river or a canal, through a portion of its course, by means of hydraulic turbines. A water power development scheme is known as *hydroelectric scheme* or briefly as *Hydel-scheme*. The hydel plants can be classified according to their function as follows:

- (1) Run-Off-River Plants. (2) Storage Plants (3) Pumped Storage Plants.

1. Run-off-River Plants

Run-off-river plants are those which utilize the river flow having no pondage at its upstream. A weir or a barrage is constructed across the river, simply to raise the water level slightly. Such a scheme is essentially a low head scheme, and is adopted in the case of a perennial river which has minimum dry weather flow of such magnitude which makes the development worthwhile. Run-off-river plants are of two types :

- (i) those that utilize the varying flow, and
- (ii) those that operate on the minimum available discharge and work throughout the year.

In some cases, small storage or *pondage* is provided in such plants, for tiding over hourly fluctuations of load. When the available rate of flow at the site is in excess of demand, the excess water can be temporarily stored in the pond on the upstream side of the weir. This *pondage* can then be utilized during the peak demand.

2. Storage Plants

Such a plant has a storage reservoir at its upstream. A dam is constructed across the river and water is stored during the periods of excess supply. The storage to take care of fluctuation of the river supply or that the load enhances the utility of a hydel plant. In India, most of the major plants are of this category.

3. Pumped Storage Plants

A pumped storage plant stores power in the form of potential energy of water. The scheme consists of a storage reservoir at a higher level, and a turbine-cum-pump installation. The scheme is used to generate power only during the peak hours of demand, and is meant for assisting an existing thermal plant. During the peak demand, water flows from the reservoir to the turbine and power is generated. During off-peak hours, excess power is available from the thermal plant or from any other hydro-plant. This power is utilised in pumping the water from the low reservoir (Fig.21.1) back to the storage reservoir at the higher elevation and water is thus stored. This stored water is utilised again during hours of peak demand, thus supplementing the power supply of the existing thermal power station or any other hydroplant. In such a scheme, no water is wasted.

Additional cost of pumping plant can be saved by using the newly developed machines which are designed to work as turbine as well as pump. Similarly, generator-cum-motor is provided instead of separate motor to run the pump and generator to generate electricity from the available mechanical energy of the turbine. By such a scheme, the load factor of the system is very much improved.

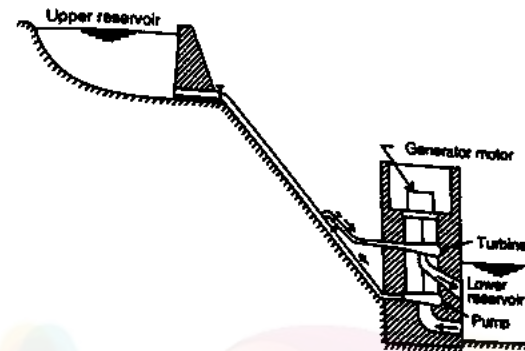


FIG. 21.1. PUMPED STORAGE PLANT.

Classification of hydroelectric schemes

Depending upon the available head, the hydel schemes may be classified as under :

- (1) Low head scheme. (2) Medium head scheme. (3) High head scheme.

A *low head scheme* is the one which uses head of less than about 15 metres. It is essentially a run-off river scheme. Fig. 21.2 shows the schematic plan of such a scheme. A weir or barrage is constructed to raise the water level in the river. The power-house may be constructed in continuation with the barrage. Sometimes, the barrage is constructed some distance upstream of powerhouse and water is conveyed to the power house through an intake canal.

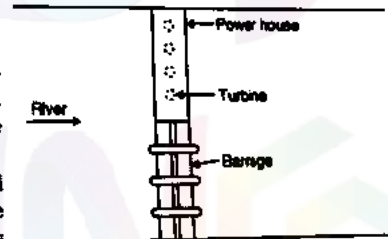


FIG. 21.2 LOW HEAD SCHEME.

A *medium head scheme* uses head between 15 to 60 metres. Such a scheme is essentially a storage scheme. A dam is constructed to store some quantity of water and also to obtain the required head. A medium head scheme has the features mid-way between the low head scheme and the high head scheme.

Conventionally, a *high head scheme* uses a head of more than 60 metres. A dam of sufficient height is constructed to store the water to ensure the supplies throughout the year, and to attain sufficient head. Fig. 21.3 shows the schematic plan for such a scheme. High head scheme using heads upto about 1800 metres have been developed.

The high heads needed for high head schemes can be obtained by one of the following methods :

1. *Construction of high dam.* The examples of Bhakra dam in Punjab, Rihand dam in U.P. or Hoover dam in U.S.A. fall under this method. The high dam also helps in creating sufficient storage

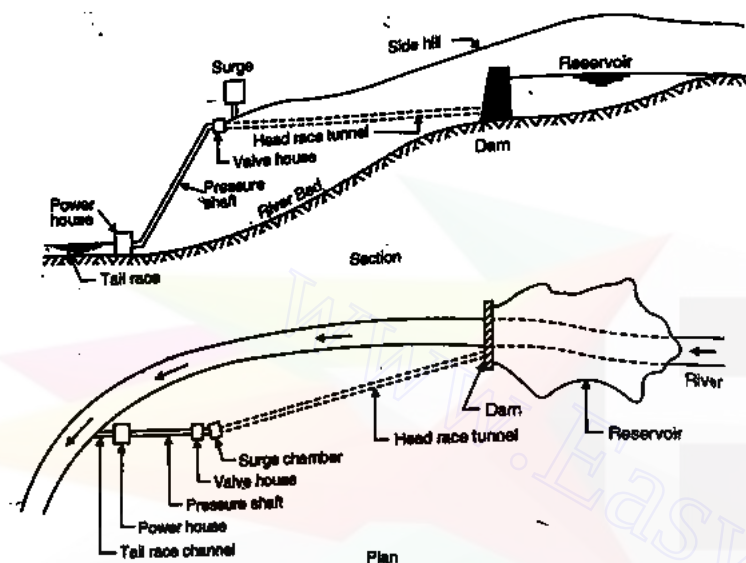
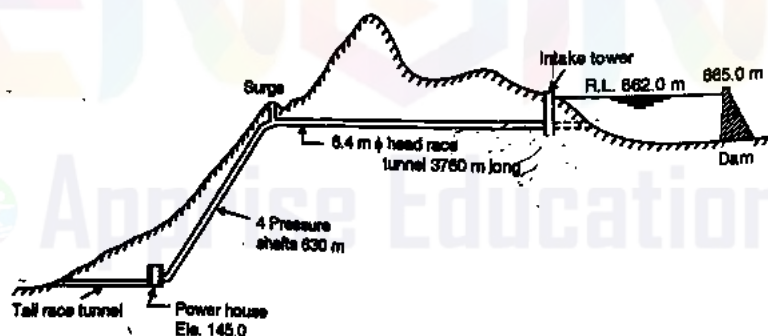


FIG. 21.3. HIGH HEAD SCHEME.

2. *Utilisation of natural fall.* The natural high falls may be utilised for power generation. Examples under this are Jog Falls in India and Niagara Falls in U.S.A.

3. *Conveying water to a lower valley.* The water may be conveyed to a lower valley, by cutting across the watershed, so that high head is available. The Koyna hydel scheme is one such example. Fig 21.4 shows the schematic section of Koyna scheme.

FIG. 21.4. A SCHEMATIC SECTION OF KOYNA SCHEME
(TAIL RACE DEVELOPMENT STAGE III NOT SHOWN)

21.3. PRINCIPAL COMPONENTS OF HYDROELECTRIC SCHEME

A hydroelectric work comprises of the following components :

1. Forebay
2. Surge tank
3. Intake structure
4. Penstocks
5. Power-house
6. Turbines and governors
7. Generators
8. Transformers
9. Transmission lines.

21.4. FOREBAY

A forebay is an enlarged body of water just in front of the intake. The main function of the forebay is to store, temporarily, the water rejected by the plant when the load is reduced and to meet the instantaneous increased demand when the load is instantaneously increased. Thus, the forebay absorbs the short interval variations of intake of water into turbines in accordance with the fluctuating loads. When a canal leads water to the turbine, the canal itself serves as a forebay. When the penstocks directly take water from the reservoir, the reservoir acts as the forebay. Often, the canal near the power-house is enlarged to provide an effective forebay.

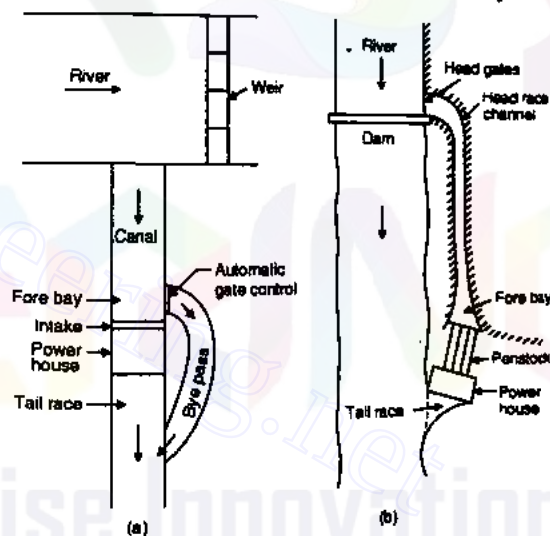


FIG. 21.5. FOREBAY.

Sometimes, a bypass is provided to convey water from the forebay to tail race when the load is reduced. For maximum efficiency, the water level in the forebay is automatically controlled within narrow limits by providing automatic gates at the head of the bypass. When the load is reduced, the governors of the turbines reduce the entry of water. This in turn, has the tendency to increase the water level in the forebay.

However in order to keep the constant water level in the forebay, the gates at the head of the bypass are automatically operated so that the spillage into the bypass is increased. Similarly, during the increased load, more water is required. The automatic gates then move up and reduce the spillage into the bypass.

21.5. INTAKE STRUCTURE

The water is conveyed from the forebay to the penstocks through the intake structure. Following are the accessories of an intake structure

1. *Trash rack* : There rack prevents the debris getting into the penstock. See § 11.13 and Fig. 11.45 for details.
2. *Rakes* to clear the trash rack.
3. Ice removal equipment.
4. Penstock closing gates with hoists.

21.6. PENSTOCKS

Water from the storage reservoir is carried through penstocks or canal to the power-house. Penstocks are the pipes of large diameter, usually made of steel in various forms, reinforced concrete or wood stave, which carry water under pressure from the storage reservoir to the turbine. Penstocks may be subjected to water hammer pressure due to fluctuations in the load. Short length penstocks are designed to take this extra pressure. However, in the case of long penstocks, surge tank is provided to reduce the water hammer.

The intake of the penstock at the forebay must be at a level low enough to provide an adequate water seal under all conditions, particularly at low water, so that the entry of air is prevented. If there is too little depth of cover, whirlpools will tend to form and carry into the penstock and to the wheels, tending to lessen the power output. In addition to proper depth of water seal, the entrance to the penstock should be flared to avoid any loss of head by contraction. Sharp bends cause loss of head and require special anchorages, and should be avoided.

An air vent, and stand pipe connecting the top of the penstock with the open air, should be provided below the gate. The air vent permits the air to enter the penstock when the head gates are closed and water drawn off through the wheel. In the absence of this, dangerous collapse pressure may be exerted on the penstocks. The water in the vent pipe should not get frozen, otherwise it will prevent entrance of air. Before opening the main head gate, the penstock, if empty, should be filled through a bypass so that the surge of water may not damage the turbine.

21.7. SURGE TANK

When the load on the turbine is reduced, the governor automatically closes the inlet gates partially, to reduce the inflow to the turbines. This retards the water flowing in the penstocks, leading to water hammer in the penstocks. The excessive inertia pressure so caused may burst the penstock. A surge tank may be provided to reduce the water hammer pressures. The functions of surge tank are as follows :

1. The surge tank furnishes a reservoir surface very near to the discharge end of the penstock to dampen the water hammer pressure.

2. It temporarily stores water when the load is reduced until such time that the velocity has fallen to the steady value.
3. It temporarily supplies more water when the load is increased until such time that the velocity has increased to a fresh steady value.
4. It calms down effectively and rapidly the surging in the water level.

An elementary surge tank consists of cylindrical open-top storage reservoir connected by a vertical branch pipe to the penstock at a point as close to the turbine as possible (Fig. 21.6). Surge tanks are almost always built high enough so that the water cannot over flow, even with a full load change. Where topography permits, the surge tank may be placed on the surface of the ground, above the penstocks line, at the point where the latter drops rapidly to the power house. The minimum height of a surge tank is determined by the restriction that in no case must the water level in it drop to such a point that air is drawn into the penstock.

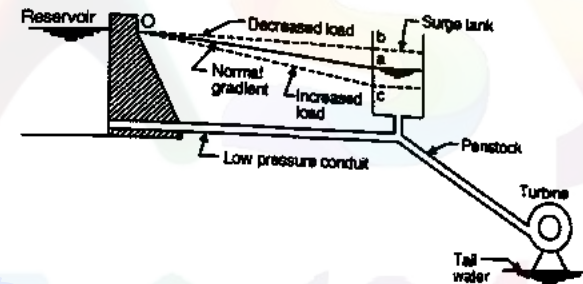


FIG. 21.6. SURGE TANK.

Under normal load condition, a normal hydraulic gradient oa (Fig.21.6) exists. When load is reduced, water level in the surge tanks rises and the hydraulic gradient is flattened. This decelerates the flow from the reservoir to the surge tank. When the load increases, more water is drawn and the water level to the surge tank drops. This makes the hydraulic gradient steeper, thus accelerating the flow from the reservoir to the surge tank. Other types of surge tanks are : (i) Surge shaft with expansion chamber, (ii) Throttled or restricted orifice surge tank, (iii) Differential surge tank and (iv) Multiple surge tank. A differential surge tank has a central riser pipe having small ports or holes at its lower end. The main advantage of this type is that for some stabilising effects its capacity may be less than an elementary surge tank. This is so because in a differential surge tank retarding and accelerating heads are developed more promptly than in the simple surge tank in which the head build up gradually as the tank fills.

21.8. TURBINES

Hydraulic turbines are the machines which convert hydraulic energy into mechanical energy. The mechanical energy developed by a turbine is used in running an electric generator which is directly coupled to the shaft of the turbine. The generator thus

develops electric power which is also sometimes known as *hydro-electric power*. In general, a water turbine consists of a wheel called *runner* which is provided with specially designed blades or buckets. The water possessing large hydraulic energy when strikes the runner and causes it to rotate.

Hydraulic turbines may be classified under two heads :

- (i) Impulse or velocity turbines.
- (ii) Reaction or pressure turbines.

1. Impulse turbine

In the case of impulse turbine, all the available potential energy or head is converted into kinetic energy or velocity head by passing it through a contracting nozzle or by guide vanes before it strikes the buckets of the turbine. The wheel revolves free in air and water is in contact with only a part of the wheel at a time. The pressure of water all along is atmospheric. In order to prevent splashing and to guide the water discharged from the buckets to the tail race, a casing is provided. Thus, an impulse turbine has the following characteristic features :

- (i) The wheel passages are not completely filled.
- (ii) The water acting on the wheel vanes is under atmospheric pressure.
- (iii) The water is supplied at a few points at the periphery of the wheel (usually one point, but occasionally two or more points).
- (iv) Energy applied to the wheel is wholly kinetic.

An impulse turbine is essentially a low-speed wheel and is used for relative high heads, ordinarily beginning in the neighbourhood of 150 m and so far being limited to about 1000 m, though a head of over 1770 m has been utilized by impulse wheels in Switzerland. Examples of the impulse turbines are: Pelton wheel, Turgo-impulse wheel, Girard turbine, Banki turbine, Jonval turbine etc. Out of these, the Pelton wheel is predominantly used at present.

2. Reaction Turbine

In the case of reaction turbine, only a part of the available potential energy is converted into velocity head, at the entrance to the runner, and the balance that forms a substantial portion remains as a pressure head. The pressure at the inlet to the turbine is much higher than the pressure at the outlet, and it varies throughout the passage of water through the turbine. Major part of the power is derived from the difference in pressure acting on front and back of runner blades and only a minor part from the dynamic action of velocity. The water being under pressure, the entire flow from head race to tail race takes place in a closed conduit system and the atmospheric air has no access at any point. To summarize, a reaction turbine has the following characteristic features :

- (i) The wheel passages are completely filled with water.
- (ii) The water action on the wheel vanes is under pressure greater than atmospheric.
- (iii) The water enters all around the periphery of the wheel.
- (iv) Energy in the form of both pressure and kinetic is utilized by the wheel.

The examples for reaction turbines are : Fournayon turbine, Tompcon turbine, Propeller turbine, Francis turbine, Kaplan turbine etc. Out of these, Francis and Kaplan turbines are predominantly used at present.

The turbines may also be classified according to the direction of flow of water in the runner, as follows :

- (i) Tangential flow turbine.
- (ii) Radial flow turbine
- (iii) Axial flow turbine
- (iv) Mixed flow turbine.

In a *tangential flow*, the water strikes the runner along the tangent to the part of rotation of the runner, such as in the Pelton wheel. In a *radial flow*, the water flows along the radial direction and remains in a plane normal to the axis of rotation, as it passes through the runner. The radial flow may be *inward* (i.e. water entering the outer circumference and flowing radially inward) such as in the Francis turbine, Thomson turbine, or Girard turbine, or the flow may be *outward* (i.e. water entering at the centre and flow radially outward), such as in Fournayon turbine or Boyden turbine. In an *axial flow* turbine or *parallel flow* turbine, water enters and leaves the runner along the direction parallel to the axis of the turbine shaft. Examples of this type of flow are in Jonval turbine, Girard turbine, propeller turbine and Kaplan turbine. In the *mixed flow turbine*, water enters the runner at the outer periphery in the radial direction and leaves it at the centre in the direction parallel to the axis of the shaft. Modern Francis turbine is one such example.

Specific speed

The turbines may also be classified according to their specific speed. The specific speed of a turbine is the speed of a geometrically similar turbine that would develop one metric horse power under a head of one metre. Table 24.2 shows the specific speeds of various turbines.

TABLE 24.2

S.No.	Head (meters)	Type of turbine	Specific speed
1	300 or more	Pelton wheel (single jet)	10 to 35
2	150 to 300	Pelton or Francis	35 to 100
2(a)	200 to 300	Pelton wheel (Multiple jets)	35 to 55
2(b)	150 to 200	Francis turbine	80 to 100
3	60 to 150	Francis turbine	100 to 220
4	Less than 60	Francis, Kaplan or Propeller	220 to 1000
4(a)	30 to 60	Francis turbine	220 to 300
4(b)	16 to 30	Kaplan turbine	300 to 1000
4(c)	Less than 16	Kaplan or Propeller (for no load variation)	300 to 1000

21.9. SELECTION OF SUITABLE TYPE OF TURBINE

The selection of the suitable type of turbine depends primarily upon the available head and the quantity of water required. On this basis, turbines may be classified as follows :

1. Low head turbine
2. Medium head turbine
3. High head turbine.

IRRIGATION AND WATER POWER ENGINEERING

800

Low head turbines are those which are capable of working under heads less than 30 m. Examples of such turbines are : propeller turbine and Kaplan turbine. These turbines use large quantity of water. *Medium head turbines* are capable of working under the heads ranging from about 30 to 150 m. The example under this type is that of Modern Francis turbine. *High head turbines* are those which are capable of working under very high head ranging from 150 to over 1000 metres. Impulse turbines are high head turbines. These turbines require relatively less quantity of water. Table 21.2. gives the range of head and specific speed for which each type of turbine is most suitable. As far as possible, a turbine with highest permissible specific speed should be chosen, which will not only be cheapest in itself but its relatively small size and high rotational speed will reduce the size of the generation as well as the power-house. But the specific speed cannot be increased indefinitely because higher specific speed turbine is generally more liable to cavitation.

Part Load Operation

The turbines may be required to work with considerable load variations. As the load deviates from the normal working load the efficiency would also vary. At part load, the performance of Kaplan and Pelton turbines are better in comparison to that of Francis and Propeller turbines. The variability of load will influence the choice of type of turbine if the head lies between 150 m to 300 m, or lies below 30 m. For higher range of heads, Pelton wheel is preferable for part load operation in comparison to Francis turbine, though former involves higher initial cost. For heads below 30 m, Kaplan turbine is preferable for part load operation in comparison to Propeller turbine.

In addition to the above points, there are certain other points to be considered for the selection of right type of a turbine. The *rotational speed* of the turbine should be such that it will give the synchronous speed of the generator. The *overall cost* which includes initial cost and the running cost should also be considered. *Cavitation* affects the installation of a reaction turbine. The *disposition of turbines shaft* may be vertical which is better for large size reaction turbines or horizontal which is suitable for large size impulse turbine. In the 150 to 300 m range of head adopt either to the Pelton wheel or the Francis turbine ; the presence of large amounts of sand in water would rule out the Francis type, since its runner will not be able to withstand the erosive action.

21.10. THE POWER-HOUSE

A power-house of a hydel scheme serves as a protective covering for the hydraulic and electrical equipment. The power-house is the conspicuous and vital part of a hydroelectric development. It should, therefore, be of satisfactory appearance, harmonizing with the surroundings. The following items of equipment should be provided for the layout of the power-house :

1. Hydraulic Equipment

- (i) Turbines
- (ii) Gates or gate valves
- (iii) Relief valves for penstock settings
- (iv) Governors

WATER POWER ENGINEERING

801

- (v) Flow measurement equipment.

2. Electrical Equipment

- (i) Generators :
Air ducts
- (ii) Exciters
- (iii) Transformers :
Pumps and cooling system
Connections
Runs and platforms
- (iv) Switching equipment :
Low tension buses
Switch board panels
Switch board equipment and instruments
Oil switches
- (v) High tension system :
Buses
Oil circuit breakers
Lightening arresters
Outgoing connections
- (vi) Auxiliaries :
Storage batteries
Station lighting.

3. Miscellaneous Equipment :

- (i) Crane
- (ii) Shop
- (iii) Office rooms
- (iv) Lavatory, etc.

It is becoming more and more the practice to place the transformers and high tension equipment outdoors near the power-house which results in a material saving in cost. A power-house has two main parts : (i) super-structure and (ii) sub-structure. A super-structure is that portion of power house which is above the ground level. A big-size power station may be a multi-storeyed building having the main machine hall at the ground floor. Sometimes, in a mountainous country, underground power-house may be constructed. The sub-structure of the power-house mainly consists of a foundation block of concrete immediately below the machine hall. The concrete block has a number of spaces and passages for fixing the turbine scrolls, their draft tubes and power cables.

General arrangement of hydroelectric units in the power-house

In the case of vertical single runner water-wheel unit of reaction turbines and direct connected generators, the most convenient and economical arrangement is to place the units in a line parallel to the length of the power-house. In the case of horizontal units of reaction turbines, the best arrangement is to place each unit on a line normal to the axis of length of the power-house. In both the cases the spacing between the

units is controlled by the flume of scroll casing, draft tube, tail race or generators dimensions. For horizontal setting of impulse turbines, the best arrangement is to place the wheel shaft parallel to the big axis of the power-house. The spacing of units in this case is determined by their dimensions and necessary clearance rather than by penstock, flume or tail race widths. For vertical settings of impulse-wheels, the units should lie with centres on a line parallel to the axis of length of the house, both penstock and tail race channel running in the same direction.

The above arrangements require a rectangular building with main floor occupied by the turbines and their accessories, the governors, oil pumps, exciters etc. The switch board and operating platform are generally placed on a mezzanine or second floor. A travelling crane is an important part of the power-house equipment. It will span the width of the house and be arranged to travel for its entire length. In fixing the elevation of the crane rail above the floor, it is essential that sufficient head room be provided for lifting and carrying along any of the various machine parts.

21.11. SCROLL CASING, DRAFT TUBE AND TAIL RACE

1. Scroll Casing

Scroll casing surrounds the turbine runner and the guide mechanism. The water from the penstock is delivered to the scroll casing. The spiral casing is spiral shaped to avoid the formation of eddies and to distribute water evenly around the guide vanes. The casing is made of cast steel, plate steel or concrete, depending upon the head. The main purpose of the scroll casing is to maintain nearly uniform velocity at the entry to the guide vanes. The use of the concrete scroll casing is necessarily confined to low heads, usually not over 20 m. Plate-steel scroll casing is used for head upto 110 metres.

2. Draft-Tube

A draft-tube is a pipe or passage of gradually increasing cross-sectional area, which connects the runner exit to the tail race. It serves the following purposes :

(1) It permits a negative or suction head to be established at the runner exit, thus making it possible the placing of wheel and connecting machinery at a level above that of water in the tail race under high-water flow-conditions of river, without loss of head.

(2) It reduces the high velocity of the water discharged by the turbine by gradually enlarging the cross-section and thus converts the kinetic energy of the water leaving the runner, into mechanical energy of the wheel. Thus it acts as a recuperator of pressure energy.

Fig. 21.7 shows the various types of draft tubes in common use. The conical tube or vertical tapered pipe [Fig. 21.7(a)] expanding at the rate of 4 to 8 degrees is made of steel plate and is the most efficient of all the types of draft tubes. It is the most efficient in reducing exit losses, but with large units this would mean excessive excavation and sub-structure cost. Consequently, for large units, the elbow tube [Fig. 21.7(b)] has become conventional as it affords benefit in cost of excavation. The hydracone draft tube [Fig. 21.7 (c)] patented in 1917 by W.N. White utilizes the so-called 'hydracone action' of jet of water impinging on a plate and being deflected

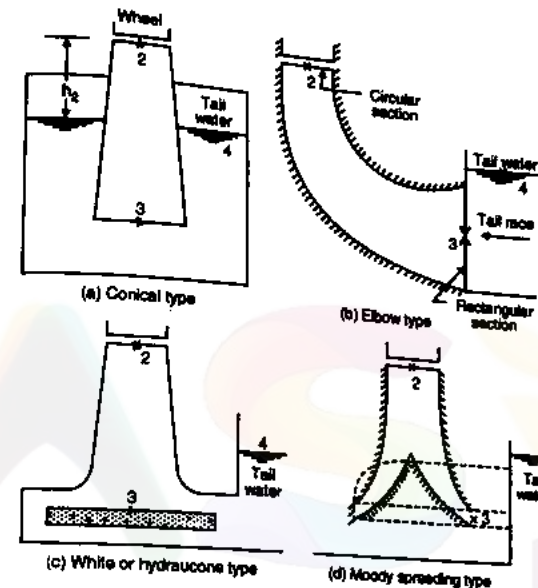


FIG. 21.7. TYPES OF DRAFT TUBES.

there from in all directions. The Moody or spreading draft tube [Fig. 21.7 (d)] obtains substantially the same effect as the hydracone by an actual cone constructed symmetrically in tube. The hydracone and Moody draft tubes also permit the better utilization of the 'whirl component' of water entering the draft tube. For impulse turbines, no draft tubes are provided and the head from nozzle to tail race is lost.

Applying Bernoulli's equation between point 2 (at the entrance of the draft tube) and point 4 (at tail race level), we get

$$\frac{V_2^2}{2g} + \frac{p_2}{w} + h_2 = \frac{V_4^2}{2g} + 0 + 0 + h_f + k \frac{V_3^2}{2g} \quad \dots(1)$$

where h_f represents friction losses in the draft tube, and k is approximately unity. Transposing and noting that $\frac{V_4^2}{2g}$ is usually small and may be neglected, we get

$$\frac{p_2}{w} = -h_2 - \left(\frac{V_2^2}{2g} - \frac{V_3^2}{2g} \right) + h_f \quad \dots(21.1)$$

and

$$h_2 = -\frac{p_2}{w} - \left(\frac{V_2^2}{2g} - \frac{V_3^2}{2g} \right) + h_f \quad \dots(21.2)$$

From the above approximate analysis, it is clear that the pressure in the draft tube next to the turbine blades is negative. Except for the losses, the residual head

(h_2) and the large proportion of velocity head $\left(\frac{V_2^2}{2g} - \frac{V_3^2}{2g}\right)$ are converted by the draft tube into a negative pressure head. Since, to avoid cavitation, the negative pressure has to be limited to such a value that the absolute pressure in the draft tube remains above the vapour pressure of water, it is evident that for a given height of draft tube there is a limiting value of V_2 . Or, conversely, a given value of V_2 limits the height of draft tube. The height h_2 of the wheel above the tail race is limited to about 8 m as an extreme.

8. Tail race

Tail race is the channel into which the draft tube discharges. Where the power-house is close to the river, the tail race may be the river itself. In other cases, an artificial race channel may connect the power-house with the river. Moody spreading tubes, and some forms of hydracone, where there is no open channel under the power-house, the sub-structure becomes practically a mass of concrete with water passages. The depth of tail race depends upon the assumed design velocity. Usually a velocity of 0.8 to 1.5 m/sec will be suitable. The tail race channel should be maintained properly so that excessive aggradation and degradation is avoided. The excessive aggradation raises the tail water level, thus reducing the head. The excessive degradation may lower the tail water levels, and may uncover the draft tube. This may, in turn, cause unsatisfactory flow conditions, increases the losses and may damage the turbine blade by way of cavitation.

21.12. ASSESSMENT OF WATER POWER POTENTIAL

Let Q = discharge in cumecs, and
 H = height of fall, in metres.

Then the rate of work done is

$$= QWH = 9.81 QH \text{ kN-m or kilo-watt}$$

(Since 1 kN-m = 1 kW)

Let η_t be the efficiency of turbines, η_g be the efficiency of generators and $\eta (= \eta_t \times \eta_g)$ be the overall efficiency. The electrical energy or power (P) generated by the hydro-electric plant is given by

$$P = 9.81 \eta_t \eta_g QH = 9.81 \eta QH \text{ kW} \quad \dots(21.3)$$

$$\text{Taking } \eta = 0.8, \quad P = 9.81 \times 0.8 QH = 7.84 QH \approx 8 QH \text{ kW} \quad \dots(21.3 a)$$

For the usual values of efficiencies, the electrical power available can be roughly estimated to be $7.5 QH$ to $8.5 QH$ kW.

Since 1 metric H.P. = 0.735 kW, the metric H.P. generated is given by

$$\text{Metric H.P.} = \frac{9.81 \eta QH}{0.735} = 13.33 \eta QH \quad \dots(21.4)$$

Gross head, operating head and effective head

Gross head is the difference in the elevation of the point of diversion of water for a hydel scheme and the point of return of water back to the stream. **Operating head** is the simultaneous difference between the elevations of the water surface in

the forebay and the tail race after making due allowance for approach and exit velocity head. The **effective or net head** is the head applied to the turbine and denotes the difference of head at the points of entry and exit of the turbine taking into account both the velocity and the pressure heads.

Primary or Continuous power and Secondary power

The **primary or firm power** is that which must be always available for carrying load or the corresponding to the minimum stream flow, with due consideration of the effects of pondage and load factor. It is the minimum continuous power available from a plant under the most adverse hydraulic conditions. **Secondary power or surplus power** is the power other than primary generated at the plant. The term surplus power is also sometimes used to distinguish the excess power available particularly at night, when owing to low or no-load conditions, there is no use for the power.

Installed capacity, dependable capacity

The **installed capacity** of a power station shows the total capacity of all the turbine-generator units installed in the power station. **Dependable capacity** is the load carrying ability of the power station with respect to the characteristic of the load to be supplied during a specified time interval and the period, and is determined by the power factor, capacity and the load on a station.

Load factor, utilization factor and capacity factor

The **load factor** is defined as the ratio of the average load over a certain period to the peak load occurring during the same period.

$$\text{i.e. } L.F. = \frac{\text{Average load over a certain period}}{\text{Peak load occurring during the same period}}$$

If the period chosen is one day, the load factor is known as the **daily load factor**. The **annual load factor** for a power station is the annual load factor for the entire area served by the power supplied from the power station. The load factor for industrial area will be higher than that for domestic area since the power demand by industry is more uniform than that by domestic area.

The **utilization factor** is the ratio of water actually utilized for power to that available in the river, this latter being limited as shown by the flow-duration curve and (in the higher part of the curve) the discharge capacity of the wheels. With constant head, utilization factor would also be the ratio of power utilised to that available.

Capacity factor or plant use factor or plant factor is the ratio the average output of the plant to the plant capacity i.e.

$$\text{Plant capacity factor} = \frac{\text{Average load output}}{\text{Installed capacity}} = \frac{\text{Average load}}{1.1 (\text{peak load})}$$

if we take the reserve capacity as 10% of the peak load.

If the average capacity is zero, load factor on a particular day = capacity factor for that day. Thus, capacity factor will be identical with load factor when the maximum just equals the plant capacity. The capacity factor for a hydroelectric plant commonly varies from about 0.25 to 0.75, depending on load factor etc. The capacity factor does

not exceed 0.75 for the usual plant capacity, unless in addition to high load factor and complete pondage at the plant, a large degree of regulation by storage is possible.

Example 21.1. Three turbo-generators each of capacity 10000 kW have been installed at a hydel power station. During a certain period of load, the load on the plant varies from 12000 kW to 26000 kW. Calculate (i) total installed capacity, (ii) load factor (iii) plant factor and (iv) utilisation factor.

Solution

(i) Total installed capacity = $3 \times 10000 = 30000$ kW

(ii) Load factor = $\frac{\text{Average load}}{\text{Peak load}}$

$$\text{Average load} = \frac{1}{2}(12000 + 26000) = 19000 \text{ kW}$$

$$\text{Peak load} = 26000 \text{ kW.}$$

$$\therefore \text{Load factor} = \frac{19000}{26000} = 0.7308 \approx 73.1\%$$

(iii) Plant factor = $\frac{\text{Energy actually produced}}{\text{Max. energy which can be produced}}$

Taking any time duration t hours,

$$\text{Energy actually produced} = \frac{1}{2}(12000 + 26000)t = 19000t \text{ kWh}$$

$$\text{Max. energy that could be produced} = 3 \times 10000 \times t = 30000t \text{ kWh.}$$

$$\therefore \text{Plant factor} = \frac{19000t}{30000t} = 0.633 \approx 63.3\%$$

(iv) Utilisation factor = $\frac{\text{Max. power developed}}{\text{Total power that can be developed}}$

$$= \frac{26000}{30000} = 0.867 = 86.7\%$$

Example 21.2. A run-off river hydroelectric power station is proposed across a river at a site where a net head of 30 m is available on the turbine. The river carries a sustained minimum flow of 40 cumecs in dry weather and behind the power station, sufficient pondage is provided to supply daily peak load of demand with a load factor of 73%. Assuming the plant efficiency of 60%, determine the maximum generating capacity of the generator to be installed at the power house. If the daily load pattern indicates 20 hours average load and 4 hours peak load, determine the pondage to be provided to supply the daily demand.

Solution

The average electric energy is given by

$$P = 9.81 Q H \eta = 9.81 \times 40 \times 30 \times 0.6 = 7063.2 \text{ kW}$$

$$\text{Load factor} = \frac{\text{Average load}}{\text{Peak load}}$$

$$\therefore \text{Peak load} = (\text{Average load}) / \text{L.F.} = 7063.2 / 0.73 = 9675.6 \text{ kW}$$

$\therefore$ Max. capacity of generators to be installed = 9675.6 kW

During 3 hours of peak load, the excess power to be developed from the pondage = $9675.6 - 7063.2 = 2612.4$ kW.

The discharge required for developing the above excess power is

$$Q = \frac{P}{9.81 H \eta} = \frac{2612.4}{9.81 \times 30 \times 0.6} \approx 14.8 \text{ cumecs.}$$

$\therefore$ Pondage required = $(14.8 \times 4 \times 3600) \approx 21.3 \times 10^6 \text{ m}^3$

Example 21.3. A run-off-river plant has a installed capacity of 15000 kW and operates at 30% load factor when it serves as a peak load station. What should be the minimum discharge in the stream so that it may serve as a peak load station? The plant efficiency may be taken as 82% when working under a head of 25 m. Also calculate the maximum load factor of the plant when the discharge in the stream is $32 \text{ m}^3/\text{s}$.

Solution

Average power developed when the plant operates as peak load plant, with a L.F. of 30%

$$= 15000 \times 0.3 = 4500 \text{ kW}$$

$\therefore$ Discharge Q required to generate the above power is

$$Q = \frac{P}{9.81 H \eta} = \frac{4500}{9.81 \times 25 \times 0.82} = 22.38 \text{ m}^3/\text{s}$$

If the stream has a flow of $32 \text{ m}^3/\text{s}$, power developed is

$$P = 9.81 \times 32 \times 25 \times 0.82 = 6435.36 \text{ kW}$$

$$\therefore \text{Maximum load factor} = \frac{6435.36}{15000} = 0.429 = 42.9\%$$

PROBLEMS

1. Describe various types of hydel schemes.
2. Classify the hydel schemes according to available head. Draw a sketch to show a general layout and section of a high head scheme.
3. Write a note on pumped storage plants.
4. Distinguish clearly between run-off river plants and storage plants.
5. Enumerate principle components of a hydroelectric scheme.
6. Write notes on (i) Forebay, (ii) Intake structure, (iii) Penstock.
7. What is surge tank? Discuss its function and working.
8. Write notes on : (i) Scroll casing, (ii) Draft-tube, (iii) Tailrace.
9. Write a note on the selection of suitable type of turbine for a hydroelectric scheme.
10. How do you assess the water potential of a hydroelectric scheme?

Water Resources Planning

22.1. INTRODUCTION

Water resources are planned and developed either to use or to control ground and surface water flows. In the process of development, important questions that may arise are :

1. How much water is needed?
2. How much water is available?
3. How are the requirements satisfied by the supplies?
4. How is the used up water disposed of.

A developmental activity for the use of water comprises studying the availability and demand of water and contemplating a project that can meet the expected needs from available supplies by means of engineering works and non structural measures.

The development of water resources involves the conception, planning, designing, construction and operation of facilities to control and utilise water, with the *national objective of improving the quality of the life of the people*. It should be recognized that planning and design studies provide only the conceptual framework of a water development project. The complete process of development includes the construction, operation and management of the project as well.

22.2. INDIA'S WATER RESOURCES

India, with a geographical area of nearly 3.3 million square kilometers experiences extremes of climate. Normal annual rainfall varies from 100 mm in Western Rajasthan to over 11000 mm at Cherrapunji in Meghalaya. Variability of rainfall from season to season is also very high. The average annual rainfall over the country is of the order of 1170 mm depth which is nearly 4000 km³ (where 1 km³ = 10⁹ m³). Of the total precipitation a part goes towards increasing ground water storage, a part is lost as evapo-transpiration and the remaining appears as *surface water*.

The average flow in the river systems of the country has been estimated to be 1880 km³. But over 90% of the annual runoff in peninsular rivers and over 80% of the annual runoff in Himalayan rivers occur during the four monsoon months of June to September. Also, the amount of water that can be actually put to beneficial use is much less. The major constraint in exploiting the available water resources is the fact that the major part of the river flows occur as flood flows during three or four

(1908)

WATER RESOURCES PLANNING

months and consequent necessity for storing the flood flows in reservoirs for their utilisation. is about 690 km³ only, which is about 38% of the total.

Ground water is another important phase of water. Quantum of water which can be extracted economically from the ground water aquifers every year is generally reckoned as *ground water potential*. The Central Ground Water Board (CGWB) has under taken scientific assessment of the quantum of replenishable ground water in the country. The preliminary estimates made by the Board indicate that the utilisable ground water is about 450 km³.

22.3. SCENARIO OF WATER USE

Being faced with a rainfall season with few raining days, agriculture in India is heavily dependent on irrigation. The principal consumptive use of water is in irrigation. The quantum of water, being utilised for other purposes, such as that for domestic use, energy, industrial use etc etc is far less than that used for irrigated structures.

Table 22.1 shows the projected water demands for various purposes, upto the year 2025.

TABLE 22.1 WATER DEMANDS (km³)

Purpose	1990	2000	2025
1. Domestic use	25	33	52
2. Irrigation	480	630	770
3. Energy	19	27	71
4. Industrial use	15	30	120
5. Others	33	30	37
Total	552	750	1050
(a) Surface water	362	500	700
(b) Ground water	190	250	350

22.4. PURPOSES OF WATER RESOURCES DEVELOPMENT

Water resources development projects are planned to serve the following purposes:

1. **Municipal and Industrial water supply**
 - (i) Surface water development
 - (ii) Ground Water development
 - (iii) Water desalination
2. **Irrigation**
 - (i) Crop growth including cultivation of cash crops
 - (ii) Land reclamation
 - (iii) Salinity Control
3. **Hydropower generation**
 - (i) Run-of-river and storage projects
 - (ii) Hydropower from pumped storage
 - (iii) Tidal power

4. Flood control

- (i) Reservoirs and detention basins
- (ii) Embankments or dikes
- (iii) Flood diversion
- (iv) River channel improvements
- (v) Watershed management
- (vi) Flood zoning
- (vii) Flood forecasting and warning

5. Water transport

- (i) Navigation
- (ii) Port or harbour

6. Soil conservation

- (i) Erosion control
- (ii) Sedimentation control
- (iii) Watershed management

7. Pollution Control

- (i) Prevention of salt water intrusion
- (ii) Waste water dilution.

8. Scenic Preservation

- (i) Fish and wild life
- (ii) Recreation
- (iii) Public health and sanitation

22.5. CLASSIFICATION OF WATER RESOURCES DEVELOPMENT PROJECTS

Water resources development projects can be classified following types categories:

1. Single purpose project
2. Multipurpose project

A *single purpose project* is the one which serves only one purpose, such as that for irrigation, or for water power generation, or for flood control. A *multipurpose project* is the one which serves two or more purposes such as (i) storage and control of water for irrigation (ii) storage and diversion of water for domestic uses (iii) water supplies for industrial uses (iv) development of hydroelectric power (v) inland navigation (vi) flood control (vii) reclamation of land (viii) debris control (ix) preservation and cultivation of aquatic life and (x) recreation.

The *multipurpose development* is aimed at fulfilling the following objectives:

1. To enhance national economic development by *increasing the output* of goods and services.
2. To enhance the quality of the environment by *improving the quality* of the natural and cultural resources and the ecological system.
3. To enhance *social well being* through developmental activities.
4. To enhance *regional development* through increase in region's income, employment, economic base and other components of the regional objective.

22.6. FUNCTIONAL REQUIREMENTS IN MULTI-PURPOSE PROJECTS

A *multipurpose project* has uses of water for several purpose. The water stored in the reservoir has to be jointly used for these purposes, with proper coordination and compatibility.

1. Requirements for irrigation :

Water requirement for irrigation is not uniform. Maximum demand for irrigation is during winter season when Kharif crops are grown. However, if Kharif crops are grown, irrigation water will be required for the period prior to monsoon. Low rainfall years require more irrigation water than the high rainfall years. Since a storage reservoir protects the area against drought, some storage should be reserved to meet the irrigation demands during drought period.

2. Requirements for domestic supply :

The water requirement for domestic use is more or less constant, though the requirement is slightly more during summer than in winter. With increase in standards of living, the requirements of water for domestic use is gradually increasing.

3. Requirements for industrial use :

The water requirements for industrial use is constant throughout the year. However with the steady growth of industries and power plants, the requirements of water for these are steadily increasing. Paper, petrochemicals and steel industries are some of the highly water incentive industries. The water use by thermal and nuclear power plants is also very high.

4. Requirements for hydro-electric power :

Water requirement for water-power generation may vary with the seasonal variation in the power demand which in turn may depend upon the type of area served. It should be noted that water power production is not a consumptive use of water. The water discharged through turbines may be picked-up and used for irrigation purposes.

5. Requirements for flood control :

For a reservoir used for flood control, the basic requirement is that sufficient empty space should be available for absorption of flood.

6. Requirements for navigation :

The water required to maintain a desired downstream flow for navigation has marked seasonal variation. Peak releases are required during the later period of dry season.

22.7. PROCESS OF PROJECT FORMULATION

The development process for water resources projects comprises of formulation, construction and operation of a project. It is a dynamic process, comprising of the following steps :

1. Problem identification and statement of objectives :

This includes determination of need for the project on the context of use/control of water, market demand for the product and political incentive. The planner must have a clear statement of objectives of the project, before he starts the planning. The objectives of a project depends upon the availability of funds.

2. Solution identification :

This consists of data collection, inventory of all resources, analysis of data and developing solutions. The data so collected should be highly reliable for the realistic planning of a water resources project. Apart from hydrologic and topographic data of the site, data pertaining to other fields such as geological, economic and social data are required.

3. Projection for planning :

The water resources projects are not planned for present needs only, but they are also planned to cater the future needs. Hence projections of future needs compatible to the population growth should be made. However, unrealistic water use for future needs may lead overdesign and excessive investments.

4. Definition of alternatives :

This consists of formulation of alternative systems of structural and nonstructural measures in the light of constraints on the resources.

5. Evaluation of alternative projects :

This is achieved by performing economic, financial, environmental and social analyses.

6. Comparison of alternative projects :

This is done by displaying all relevant beneficial and adverse effects of each proposal.

7. Selection of a project :

This consists of picking up a project based on the sole objectives of optimum net economic benefit consistent with protecting the nation's environment.

22.8. PROJECT EVALUATION

A number of alternatives may exist, and the cost of each alternative can be worked out in steps of the previous article. The next step is to select the unit or combination of units which is economically most efficient. The financial analysis provides a complete picture of monetary conditions of a project. The financial analysis to compare project alternatives may be performed in several ways as follows :

1. Determination of net benefits, which is equal to the economic benefits (B) minus economic costs (C).
2. Determination of benefit cost ratio (B/C ratio),
3. Determination of rate of return on investment, which is equal to $(\text{annual benefits} - \text{annual costs}) / (\text{investment cost})$
4. Determination of the internal rate of return, which is the rate of discount applied to annual costs and benefits so that the present worth of all costs equals the present worth of all benefits.

Out of these the benefit cost ratio is the most common indicator adopted in economic analysis.

A unit or combination of units may be (i) physically independent unit (ii) economic independent unit.

A physically independent unit is the one for which there is no other unit, situated either upstream or downstream which would affect the inflow to this unit or outflow from this unit. An economically independent unit has no economic interconnection with other units. If there are two streams in an area, and there are two hydro power plants, one on each stream, the power plants are physically independent units. However, they are not economically independent units because the power produced by these two units is integrated into the same power system.

The financial analysis can be graphically presented in the form of three alternative curves:

(a) Benefits (B) v/s costs (C) : Curve 1 of Fig 22.1.

(b) Benefit cost ratio (B/C ratio) v/s costs (C) : Curve 2 of Fig 22.1.

(c) Benefit minus costs (B - C) v/s costs (C) : Curve 3 (dotted) of Fig 22.1.

From curve 1 of Fig 22.1, the benefits are more than costs between points P and Q, and hence Benefit Cost Ratio is more than 1 during this range of cost of project. This is also reflected by curve 2. However, the B/C ratio is maximum at point R; hence this should be the size the project. The curve of (B - C) v/s C has maximum net benefit at S. Thus, though the maximum B/C ratio is at R, increment from R to S will also be economical since the rate of return on the increment exceeds the minimum attractive rate of return. The decision on the size of the project should therefore be based on a minimum acceptable rate of return as well as on benefit cost ratio.

22.9. STRATEGIES FOR THE FUTURE

The present scenario of water use for various purposes in India and the future needs indicate that the country is fast approaching the stage when all her water resources would have been put to beneficial use and no additional source will be available to meet any further increase in demand. Strategies should be formulated, discussed and adopted well before that critical stage is reached. The strategies can be divided into two heads :

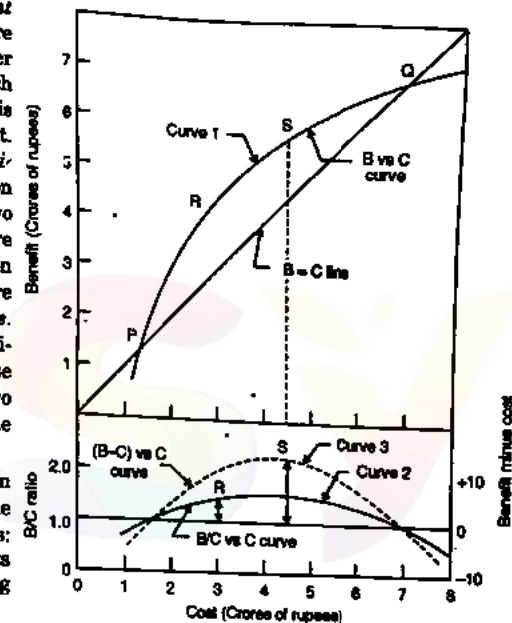


FIG 22.1 FINANCIAL ANALYSIS OF A PROJECT

- (a) Planning strategies
- (b) Management strategies

22.10. PLANNING STRATEGIES

Water resources development planning in India has traditionally been attempted *State wise*, although it was well known that *water does not obey political administrative boundaries*. In the initial stages of water resources development, projects were formulated to serve largely the irrigation requirements or irrigation combined with hydel power to serve largely the irrigation requirements. As the number of projects were relatively few, inter project considerations were relatively absent and each project were, thus, not viewed as a part of overall river basin system and integrated development and management of river basins to achieve maximum benefit out of the available water resources for various other purposes such as drinking, meeting industrial needs and navigation, was not seen as an immediate necessity. However, water is already scarce in many of the basin/sub-basins and will fast become scarce in several other basins also, since the demand for various purposes is increasing day by day. The process of planning is, therefore, becoming very complex, since several competing and sometimes conflicting demands are to be met. *Unless water resources development is planned in an integrated manner at the river basin level, optional use of the available waters in meeting the various demands cannot be achieved.*

A river basin must be the basic unit for water resources planning. The National Water Policy adopted in 1987 stresses this in clear terms when it declares: 'Resource planning in the case of water has to be done for a hydrological unit such as a drainage basin as a whole, or for a sub basin. All individual developmental projects and proposals should be formulated by the States and considered within the framework of such an overall plan for a basin or sub basin, so that the best possible combination of option can be made.' It is therefore imperative that planners and managers of water resources should start thinking in terms of river basins or sub basins as far as water resources development is concerned.

In India, there are 12 major river basins with drainage area exceeding 20000 sq. km. There are : (i) Indus, (ii) Ganga-Brahmaputra- Meghna, (iii) Brahmani Baitarani, (iv) Mahanadi, (v) Godavari, (vi) Krishna, (vii) Pennar, (viii) Cauvery, (ix) Tapi, (x) Narmada (xi) Mahi, and (xii) Sabarmati. Subarnarekha basin having a drainage area of 19296 sq. km is also an important river basin. The remaining river basins can be conveniently clubbed together into seven river systems clusters for the purpose of planning and management. *Therefore in all, India comprises twenty river basin units from the point of water management.*

River basin organisations

Treating basin as the basic unit, planning and preparation of master plans have been emphasized in the *National water Policy*. In the future, water resources planning will become so complex that without a basic master plan, it would be difficult and erroneous to hasten the decisions on proposed projects. Preparation of master plan for a river basin is a multi-disciplinary activity. Since most of the rivers of India are interstate ones, individual states can prepare master plans only for the portion of the

river basin falling within their states. There is, thus a need for setting up of multi-disciplinary units in the states for preparation of master plans and as the national water policy suggests, appropriate inter-state organisations at the basin level for the planned development and management of a river basin as a whole. *Without the establishment of river basin authorities, future prospects in optimised development of water resources for various beneficial purposes appear bleak.*

Inter basin transfer of water

In order to remedy, atleast to a certain extent, the *regional imbalances* in the availability of water resources in the country, inter-basin transfer of water from surplus basins to deficit basins has been advocated. Transfer are already taking place in some of such a mass transfer of water to thirsty water short river basins. Elsewhere also, large scale transfers will become inevitable in the future to meet the growing demands in the deficit basin.

The Government of India has framed National Perspectives of Water Resources Development with two broad components, viz. (i) *Himalayan Rivers Development* and (ii) *Peninsular Rivers Development* for transfer of waters from surplus basins to deficit basins. Studies have indicated that linking of peninsular rivers could be more easily attempted to meet the water requirement of peninsular regions because of limited lifts involved in such a link. The *Peninsular Rivers Development* again has four major parts. One is the inter linking of Mahanadi-Godavari-Krishna-Pennar-Cauvery, the second is the interlinking of west flowing rivers north of Bombay and South of Tapi, the third is the interlinking of Ken with Chambal and the fourth is the diversion of west flowing rivers to the east.

22.11. MANAGEMENT STRATEGIES

1. Basin Management

There is an intense interaction between water and land. In the river basin it will be essential to understand the linkages between land uses and soil profile on one hand and the question of water and distribution over time and space on the other hand. *Land budgeting* will have to be done at the river basin level considering the unique hydrological and soil erosion responses of various types and land uses. But the sectoral planning process being followed cannot achieve scientific land management. *Effective basin wise programmes have to be evolved considering the inter relationships of soil conservation, afforestation, land development, controlled grazing etc.* The strategy for the future should be to have forests in upland, where slopes are steep, which will minimise land slides and lead to lesser sediment transport. In *water short catchments*, selective and planned cutting of trees will decrease the interception and evapotranspiration losses leading to higher yields. Lower reaches of river basins with mild slopes and flat lands are suitable for grass lands. Developmental activities such as mining, setting up of industries, urbanisation, setting up of infrastructural facilities like roads and railway embankments, cannot be looked at in isolation but should be considered as a part of basin management.

Environmental monitoring of the catchment area should be between as sustained activity in the context of basin management. Further, *soil conservation measures* like contour bunding, planting of vetiver grass, check dams, afforestation measures will have to be taken up in degraded catchments for rejuvenation of rivers.

In *water short basins* careful selection of crops is necessary. Water is required by crops more at their critical stages of growth. Water stress during other periods has negligible impact on yield. Additional waterings do not add proportionately more to the yield. In case of dwarf wheat, the yield level has been found to be 70 % of the optimum of 5 tons/ha even after reducing the waterings from six to three. The position is more favourable in case of lentils, gram, linseed, sunflower, mustard or safflower. These crops should therefore, be preferred for irrigation in water short areas. Even under rainfed conditions, millets and pulses possess higher tolerance to rainfall deficiency compared to wheat and rice and should be preferred.

2. Water Conservation

Water conservation measures comprise of the following :

- (i) Creation of reservoirs for storing the water
 - (ii) Prevention of losses
 - (iii) Improving use efficiency
 - and (iv) Recycling and reuse of waste water.
- Apart from these, conservation strategy may also include introduction of rational price structure that would reflect the true value of water.

(i) Creation of storage reservoirs :

Since bulk of the flows in most of the river systems occurs during the 3 to 4 monsoon months, storage dams are inevitable if the seasonal flows are to be conserved and put to beneficial use for irrigation or power generation. The total storage capacity created, under creation and under consideration (upto 1991) put together is only 304 km³ where as the average annual flow in the rivers is 1880 km³. *The major constraints for the country may perhaps be the lack of storage sites.* Bulk of the river flows are yet to be stored in Ganga Brahmaputra Meghna basin. In Ganga river system, most of the large reservoir sites lie in Nepal. Brahmaputra valley has some excellent sites which are yet to be developed. Similarly, in peninsular rivers also there is still scope for development of several large storage sites. The country has to go in for creation of storages in future—small as well as large in the basins where surface water still remain untapped. In addition of this there are a large number of old small tanks. The capacities of these tanks are showing downward trends mainly due to *heavy siltation*. Most of these tanks need modernisation measures such as desilting, strengthening of bund, improving of surplussing arrangement and clearing of the inlet water ways/channels. It would be advantageous if these minor tanks are integrated with the canal systems of major projects. *Tank modernisation programmes should be integrated with watershed treatment programmes.*

(ii) Control of water loss through evaporation :

From the surface of the reservoirs, from the open channel irrigation systems and from the irrigated farms, large quantities of water are lost by evaporation. In

the vast arid and semi arid regions of the country the annual loss through evaporation is as high as 3 metres depth of water. Large storages in deep valleys conserve water much better than small storages since large storages expose much less area for evaporation. Irrigation system particularly in water short areas should be so planned that the reservoirs are depleted well before summer sets in. *Irrigation in summer months may have to be essentially from ground water.* Research and development to involve practical, safe and cost effective methods of evaporation control of reservoirs in the country is essential. The loss of soil moisture in irrigated areas can be controlled by *mulching of soil*, i.e. direct protection of soil by surface cover of various materials such as leaves, straw, sawdust etc.

(iii) Improving water use efficiency :

Improving water use efficiency means getting a higher output for the same quantity of input in the various processes of water use, be it irrigation or industrial manufacturing processes. In irrigation for instance, the efficiency of water use can be increased by going in for improved methods of irrigation such as *drip and sprinkler irrigation*. *Irrigation water use efficiency can be increased by careful planning of conjunctive use of ground water with surface water.* Lining of canals, distributaries and water courses are effective for controlling losses in transmission systems of irrigation projects. The efficiency obtained in the field application of water needs closer examination. The efficiency obtained to be compelled by statutory regulations to implement in-plant control measures including recycling and improved technology for reducing water consumption.

(iv) Recycling and reuse of wastewater :

If it is optimistically assumed that by 2025 AD, 50% of the effluents from the industries can be recovered by suitable treatments for reuse, this itself would amount to about 12 km³—enough to support a population of 33 crores or irrigate 1.5 million hectares. Waste water from urban domestic use after proper treatment could also be harnessed for irrigation and industrial use. Domestic waste water also provides additional nutritional value for agricultural crop production if necessary precautions are taken in designing and operating such irrigation systems.

(v) Restructuring of pricing policy :

It has been emphasised time and again in many forums that there is a need for a complete restructuring of the pricing system in water supply, be it for irrigation, domestic supply or industrial supply, it will discourage wasteful use. The National Water Policy recommends that water rates should be such as to convey the scarcity value of the resource to the users and to foster the motivation for economy in water use.

Important Dams of India

IMPORTANT DAMS OF INDIA

TABLE 23.1. WATER AVAILABILITY PER CAPITA AND PER HECTARE OF CULTURABLE AREA IN THE RIVER BASINS OF INDIA

Sl. No	River Basin	Drainage Area (km ²)	Average annual runoff (Mm ³)	Population (million)	Culturable area (Tha)	Average annual runoff per capita (m ³)	Average annual runoff per ha of culturable area (m ³)
1	2	3	4	5	6	7	8
1	Indus	321399	72305	38.58	9638	2015	7690
2	Ganga-Brahmaputra-Meghna						
	(a) Ganga	661404	536022	301.27	90161	1742	9727
	(b) Brahmaputra	194412	537240	25.51	12146	21080	44202
	(c) Barak & others	78180	68800	8.90	1124	7475	53680
3	Subarnarekha	29196	10794	7.94	1368	1369	5687
4	Brahmani-Baitarani	51222	38227	10.25	2201	2600	11217
5	Mahanadi	141589	66879	24.45	7964	2735	5386
6	Godavari	312812	118082	47.34	10821	2612	6226
7	Krishna	268948	87790	47.09	20298	1440	2339
8	Pennar	55213	6856	8.91	3551	770	1991
9	East flowing rivers between Mahanadi and Pennar	74364	16948	12.41	4328	1254	2616
10	Cauvery	87900	21268	27.45	5797	778	2784
11	East flowing rivers between Pennar and Kanyakumari	100128	17725	36.28	6875	487	2673
12	West flowing rivers from Tadri to Kanyakumari	112117	198554	53.70	6279	2702	2170
13	West flowing rivers from Tapi to Tadri						
14	Tapi	65146	18289	11.75	4536	1885	4954
15	Narmada	98796	41273	14.92	5901	2785	6884
16	Mahi	24842	11629	7.13	2210	1658	5352
17	Sabarmati	21674	4079	9.99	1548	618	2625
18	West flowing rivers of Kutch and Saurashtra	221551	5085	30.94	23447	185	217
19	Area of inland drainage in Rajasthan desert	-	-	-	-	-	-
20	Minor rivers draining to Bangladesh and Burma	-	21000	-	-	-	-

Dam

Type Straight concrete gravity dam
Bed rock Sandstone interspersed with claystone

23.1. MAJOR RIVER BASINS OF INDIA

In India, there are following 12 major river basins with drainage area exceeding 20000 sq. km :

(i) Indus	(ii) Ganga-Brahmaputra-Meghna
(iii) Brahmani-Baitarani	(iv) Mahanadi
(v) Godavari	(vi) Krishna
(vii) Pennar	(viii) Cauvery
(ix) Tapi	(x) Narmada
(xi) Mahi	and (xii) Sabarmati

Subarnarekha basin having a drainage area of 19296 sq. km. is also an important river basin. The remaining river basins can be conveniently clubbed together into seven river system clusters for purposes of planning and management. Therefore in all, India comprises of basic twenty river basin units from the point of water management.

Table 23.1 shows water availability per capita and per hectare of culturable area in the river basins of India.

23.2. BHAKRA DAM

Bhakra Nangal Project is located near village Bhakra, Bilaspur District, Himachal Pradesh. Bhakra dam has been constructed for Irrigation and Hydro power, across river Satluj. When constructed it was the tallest concrete gravity dam of the World, surpassing the existing height of 726' of Hoover dam. However, presently, the tallest dam of the World is the Grand Dixence Dam (284 m high) in Switzerland.

The dam, reservoir and the power house have the following data :

River.....	Satluj
Area of catchment.....	56,876 km ²
Annual runoff in the catchment.....	14,803,430,000 m ³ to 20,971,526,000 m ³
Rainfall.....	70 cm
Snow.....	239 cm
Design flood discharge.....	11,327 m ³ /s
Year of commencement of construction work.....	1948
Year of completion.....	1963

IRRIGATION AND WATER POWER ENGINEERING

289

Maximum height above the lowest point of foundation.....	226 m
Length at the top of the dam.....	518 m
Total volume content.....	4,130,000 m ³
Spillway	Centrally located overflow spillway
Type.....	4 Nos. Radial gates of size (15.24 m × 14.47 m)
Type and number of gates.....	8,372 m ³ /s
Maximum discharge capacity.....	
Reservoir	
Area at F.R.L.....	166 km ²
Gross storage capacity.....	9,621,000,000 m ³
Effective storage capacity.....	7,191,000,000 m ³
Available drawdown.....	69.5 m
Power Plant	Vertical radial-axial Francis turbine
Type.....	Maximum - 158 m
Hydraulic head.....	Minimum - 80 m
Maximum discharge.....	102 m ³ /s
Installed capacity.....	1204 MW
Generating units	Vertical radial axial Francis
Type.....	16 Nos. (5 + 5 + 3 + 3)
Number.....	1204 MW (600 + 450 + 77 + 77)
Capacity.....	407 MW
Firm.....	148 MW
Secondary.....	

Plate 1 shows the photographic view of Bhakra dam.
Fig. 23.1 shows the maximum spillway section.

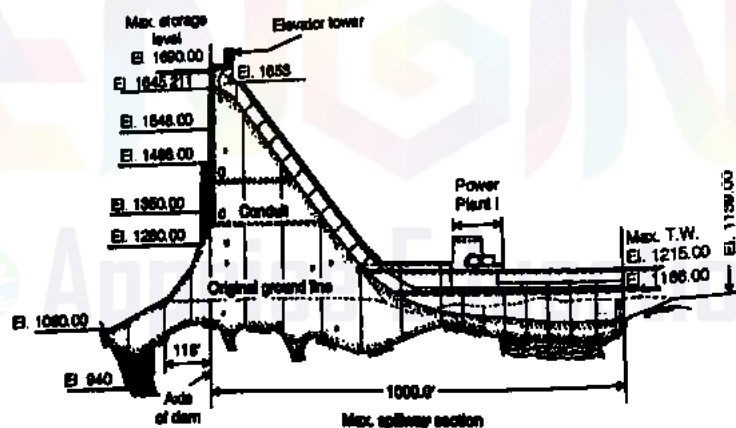


FIG. 23.1 MAXIMUM SPILLWAY SECTION OF BHAKRA DAM.

IMPORTANT DAMS OF INDIA

23.3. KOYNA DAM

292

Koyna dam is situated at Deshmukhwardi, Koyna, in Satara District of Maharashtra. It is constructed on Koyna river, and is for hydro-power generation. The salient data about the project are as follows :

Area of catchment.....	881 km ²
Rainfall.....	389.4 cm
Design flood discharge.....	7,864 m ³ /s
Year of commencement of construction work.....	1966
Year of completion.....	1961
Dam	
Type.....	Gravity
Bed rock.....	Decan trap
Maximum height above the lowest point of foundation.....	103 m
Length at the top of the dam.....	306 m
Total volume content.....	1,556,000 m ³
Spillway	
Type.....	Controlled Spillway
Type and number of gates.....	Tainter gates (12.5 m × 7.62 m)
Maximum discharge capacity.....	3,823 m ³ /s
Reservoir	
Area at F.R.L.....	115.35 km ²
Gross storage capacity.....	2,796,500,000 m ³
Effective storage capacity.....	2,663,000,000 m ³
Power Plant	
Type.....	Underground Power House
Hydraulic head.....	Stage I-475 m. Stage II-490 m
Installed capacity.....	640 MW

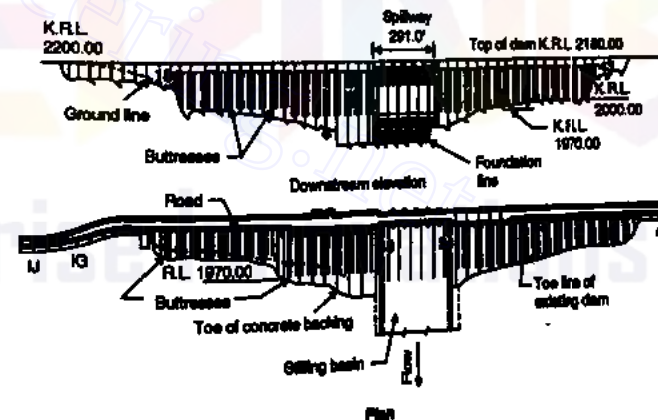


FIG. 23.2. PLAN AND D/E ELEVATION OF KOYNA DAM

IRRIGATION AND WATER POWER ENGINEERING

Generating units

Type	Palton
Number	8 Nos.
Capacity	4 Nos. - 75 MW
	4 Nos. - 60 MW

Plate 2 shows the photographic view of Koyna dam. Fig. 23.2 shows the plan and down stream elevation of the dam. Fig. 23.3 shows the non-overflow section and overflow section of the dam.

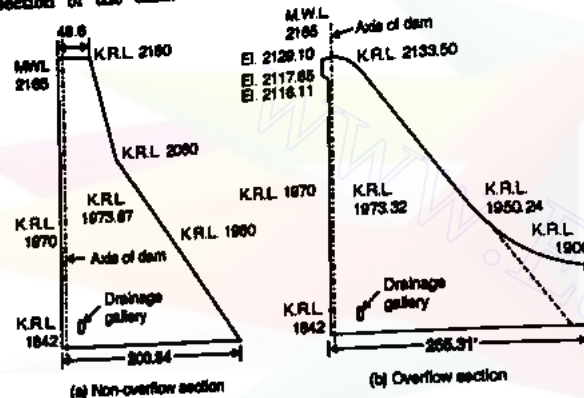


FIG. 23.3 KOYNA DAM.

23.4. NAGARJUN SAGAR DAM

Nagarjun Sagar Dam is located near Nandikonda village Dist. Nalgonda, Andhra Pradesh. It is a combined irrigation and hydro power project, constructed across Krishna river. The salient data of the project are given below.

Area of catchment	215,192 km ²
Mean annual runoff in the catchment	42,360,000,000 m ³
Rainfall	89 cm
Design flood discharge	53,450 m ³ /s
Year of commencement of construction work	1967
Year of completion	1974

Dam

Type	Earth-cum-gravity
Bed rock	Archaean granite-gneiss
Maximum height above the lowest point of foundation	125 m
Length at the top of the dam	
(i) Non overflow	
(a) Earth dam	5,415 m
(b) Masonry dam	978 m
(ii) Spillway	473 m
Total volume content	
(i) Concrete and masonry	5,510,000 m ³
(ii) Earth	2,880,000 m ³

IMPORTANT DAMS OF INDIA

Spillway

Type	Ogee Spillway
Type and number of gates	Radial gates-13.72 m x 13.41 m-26 Nos.
Maximum discharge capacity	53,450 m ³ /s

Reservoir

Area at F.R.L.	285 km ²
Gross storage capacity	11,550,000,000 m ³
Effective storage capacity	6,940,000,000 m ³
Available drawdown	24.38 m

Power Plant

Type	Conventional	Pumped Storage
Hydraulic head	Francis vertical	Francis vertical
Installed capacity	86 m	56 m
Generating units	110 MW	4 x 110 MW-440 MW
Number	1	4
Capacity	110 MW	440 MW

Plate 3 shows Nagarjun-Sagar dam. Fig. 23.4 (a) shows the non-overflow section of gravity dam, while Fig. 23.4 (b) shows the overflow section. Fig. 23.5 (a) shows left earth dam section while Fig. 23.5 (b) shows right earth dam section.

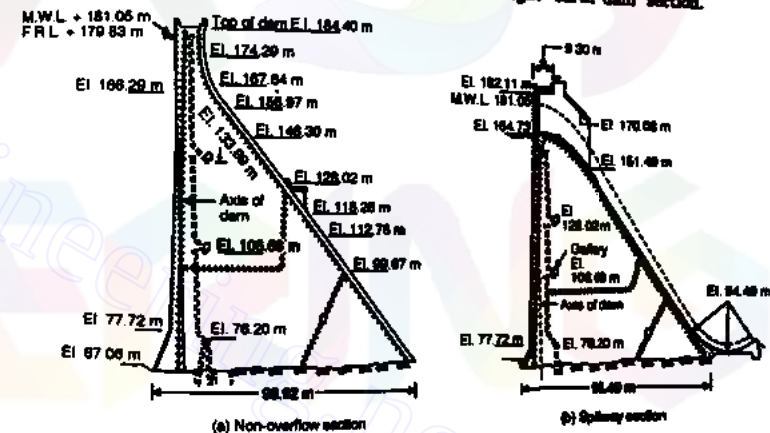


FIG. 23.4. NAGARJUN SAGAR DAM : GRAVITY DAM SECTIONS.

23.5. GANDHI SAGAR DAM

Gandhi Sagar dam is situated near Chaurasgarh Fort, Dist. Mandla, Madhya Pradesh. It is an irrigation and hydropower project, constructed across river Chambal. The salient data about the project are as follows.

Area of catchment	23,140 km ²
Rainfall	86 cm
Design flood discharge	21,940 m ³ /s
Year of commencement of construction work	1954

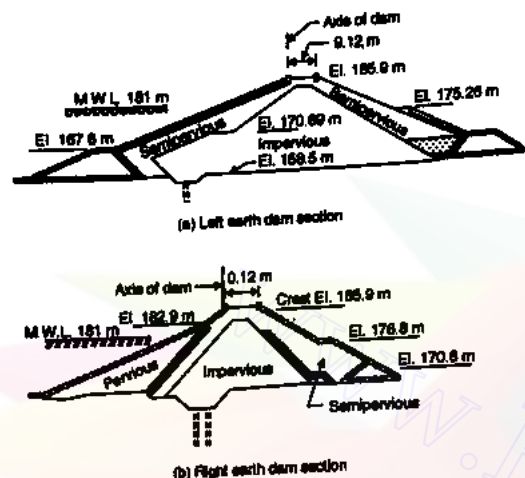


FIG. 23.5. NAGARJUN SAGAR DAM : EARTH DAM SECTIONS

Year of completion	1960
Dam	Masonry dam (Straight gravity stone masonry)
Type	
Maximum height above the lowest point of foundation	64 m
Length at the top of the dam	
(i) Non overflow	280 m
(ii) spillway	254 m
Total volume content	
(i) concrete	168,920 m ³
(ii) Masonry	550,000 m ³
Spillway	
Type	Gated
Type and number of gates	Vertical gates-10 Nos
Maximum discharge capacity	21,240 m ³ /s
Reservoir	
Area at F.R.L.	723 km ²
Gross storage capacity	7,746,000,000 m ³
Effective storage capacity	6,911,000,000 m ³
Power Plant	
Hydraulic head	45 m
Installed capacity	115 MW
Generating units	
Number	5
Capacity	23 MW each
Firm	80 MW at 60% load factor

Plate 4 shows the view of Gandhi Sagar dam. Fig. 23.6 shows the plan and downstream elevation of the dam. Fig. 23.7 shows the section of spillway and non-spillway

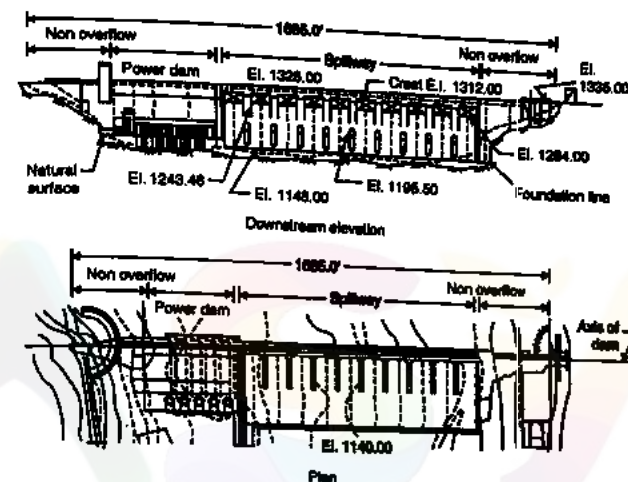


FIG. 23.6. GANDHI SAGAR DAM.

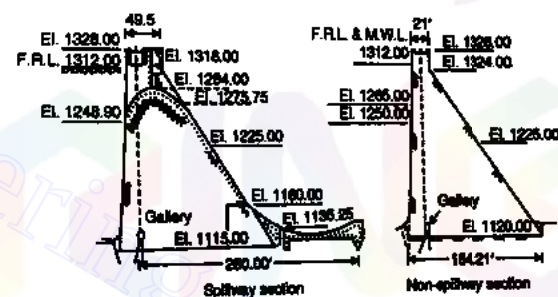


FIG. 23.7. GANDHI SAGAR DAM.

23.6. RIHAND DAM

Rihand dam has been constructed across Rihand river, at Pipri, Mirzapur district, Uttar Pradesh. It is a hydro power project. The salient data of the project are given below :

Area of catchment	13,383 km ²
Rainfall	142 cm
Design flood discharge	17,275 m ³ /s
Year of commencement of construction work	1963
Year of completion	1963

IRRIGATION AND WATER POWER ENGINEERING

Dam	Straight concrete gravity
Type	
Maximum height above the lowest point of foundation	91 m
Length at the top of the dam	744 m
(i) Non overflow	190 m
(ii) Spillway	

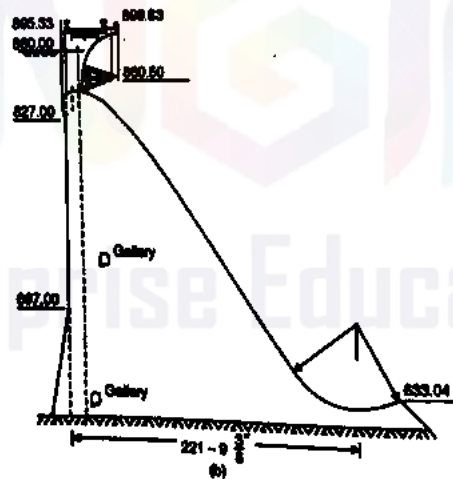
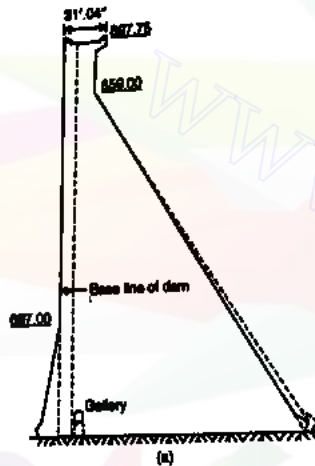


FIG. 23.8 RIHAND DAM.

IMPORTANT DAMS OF INDIA

Total volume content	1,880,000 m ³
Spillway	
Type	Ogee type
Type and number of gates	Tainter gates 13 Nos.
Maximum discharge capacity	13,339 m ³ /s
Reservoir	
Area at F.R.L.	469 km ²
Gross storage capacity	10,600,000,000 m ³
Effective storage capacity	8,900,000,000 m ³
Available drawdown	82 m
Power Plant	
Hydraulic head	44 m to 76 m
Maximum discharge	510 m ³ /s
Installed capacity	300 MW
Generating units	
Types	Umbrella type
Number	60 Nos
Capacity	50 MW each

Fig. 23.8 (a) shows non-overflow section while Fig. 23.8(b) shows the overflow section of Rihand dam. Plate 5 gives the view of the project.

23.7. LOWER BHAVANI DAM

Lower Bhavani Dam is located near Tettupallayam, Tamil Nadu. It is an irrigation dam constructed across river Bhavani which is a tributary of Cauvery. The salient data of the dam are as follows:

Area of catchment	4200 km ²
Rainfall	182.4 cm
Design flood discharge	4040 m ³ /s
Year of commencement of construction work	1948
Year of completion	1955
Dam	
Type	Masonry cum earth dam
Bed rock	Biotite gneiss
Max. height above the lowest point of foundation	62 m
Length at the top of the dam	8,677 m
(i) Non overflow	8,677 m
(ii) Spillway	121 m
Total volume content	
(i) Concrete and masonry	400,000 m ³
(ii) Earth	4,560,000 m ³
Spillway	
Type	Ogee type
Type and number of gates	Lift gates - 9 Nos (10.97 m x 6.10 m)
Maximum discharge capacity	3,141 m ³ /s

Reservoir

Area at F.R.L.	78.75 km ²
Gross storage capacity	928,240,000 m ³
Effective storage capacity	928,240,000 m ³

Plate 5 gives the photographic view of the dam. Fig. 23.9 shows the typical section of earth dam portion. Fig. 23.10 shows the maximum section of spillway.

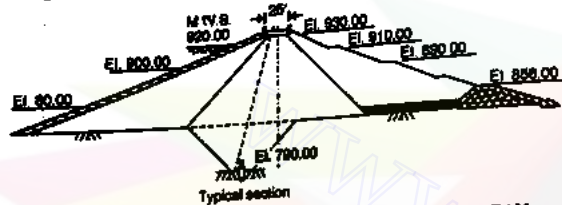


FIG. 23.9. EARTH DAM SECTION : LOWER BHAVANI DAM.

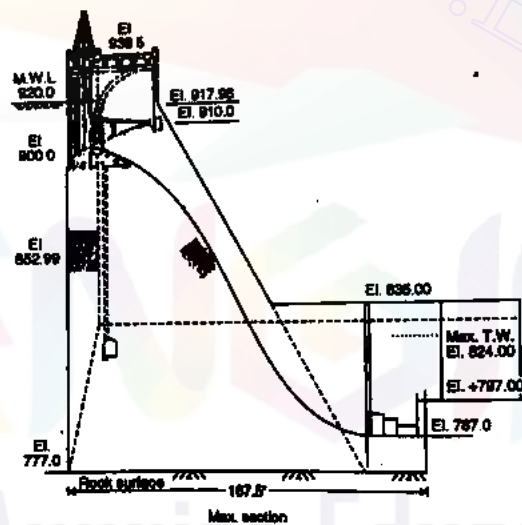


FIG. 23.10. MAX. SPILLWAY SECTION : LOWER BHAVANI DAM.

23.8. HIRAKUD DAM

Hirakud dam has been constructed at Hirakud, Sambalpur district, Orissa. It is a multipurpose project, meant for irrigation, hydropower and flood control, constructed across river Mahanadi. The salient data of the project are given below :

Area of catchment.....82,880 km²

IMPORTANT DAMS OF INDIA

Mean annual runoff in the catchment	37,000,000,000 m ³
Rainfall	137 cm
Design flood discharge	38,785 m ³ /s
Year of commencement of Construction work	1948
Year of completion	1967
Dam	
Type	Composite (earth, masonry & concrete)
Bed rock	Granite rocks with banks of schistose rocks mainly quartz siltite & medium grained granite gneiss, mylonite slate & shales
Maximum height above the lowest point of foundation	59 m
Length at the top of the dam	
(i) Non overflow	3,652 m
(ii) Spillway	1,148 m
(iii) Earthen dykes	20,660 m
Total volume content	
(i) Concrete and masonry	1,400,000 m ³
(ii) Earth	23,700,000 m ³
Spillway	
Type	Ogee (overflow) gated spillway
Type and number of gates	34 crest bays with radial crest gates and 64 undersluices with vertical gates
Maximum discharge capacity	4586 m ³ /s in each crest bay and 465.25 m ³ /s in each undersluice
Reservoir	
Area at F.R.L.	727 km ²

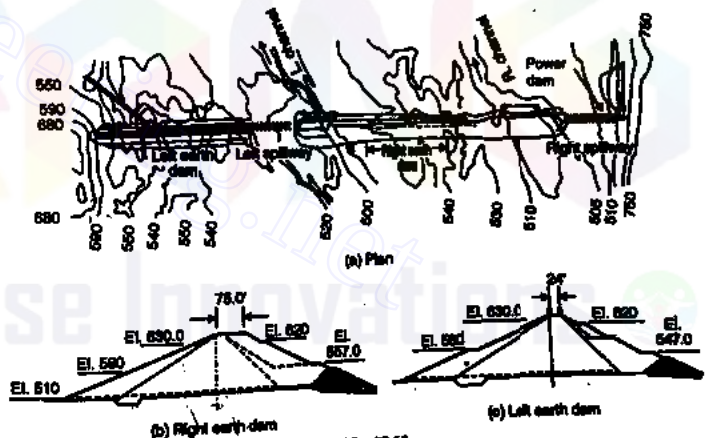


FIG. 23.11.

IRRIGATION AND WATER POWER ENGINEERING

Gross storage capacity	8,105,000,000 m ³
Effective storage capacity	5,948,000,000 m ³
Available drawdown	12.19 m
Power Plant	
Hydraulic head	Average head 26.6 m
Maximum discharge	800 m ³ /s
Installed capacity	270 MW
Generating units	
Type	vertical alternator
Number	9 Nos (7 Kaplan & 2 Francis)
Capacity	37.5 MW-4 Nos. & 24 MW - 5 Nos.
Firm	120 MW

Plate 7 shows the view of Hirakud dam. Fig. 23.11 (a) gives the plan of the dam showing the position of the two spillways, power dam, and the two earth dams. Fig. 23.11 (b) shows the section of the right earth dam while Fig. 23.11 (c) show the section of left earth dam.

23.9. KRISHNARAJA SAGAR DAM

Krishnaraja Sagar dam is located at Krishnaraja Sagar, Karnataka. Constructed across main Cauvery (tributaries Hemavathy and Lakshmanathirtha), it is meant for Irrigation and hydropower, apart from recreation because of location of famous Brindawan Gardens at its down stream side. The salient data about the dam are as follows :

Area of catchment	10,619 km ²
Rainfall	635 cm
Design flood discharge	8,215 m ³ /s
Year of commencement of construction work	1911
Year of completion	1932

Dam	
Type	Masonry

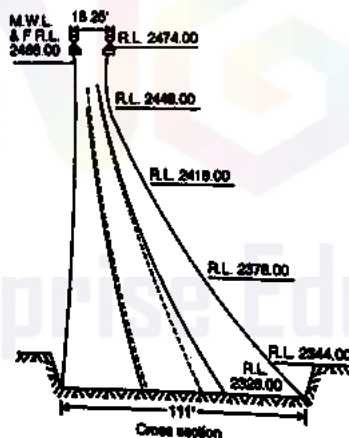


FIG. 23.12. KRISHNARAJA SAGAR DAM.

IMPORTANT DAMS OF INDIA

931

Bed rock	Hard rock
Maximum height above the lowest point of foundation	43 m
Length at the top of the dam	2,821 m
Total volume content	849,800 m ³
Spillway	
Type and number of gates	162 lift gates
Maximum discharge capacity	9,911 m ³ /s
Reservoir	
Area at F.R.L.	129 km ²
Gross Storage capacity	1,868,847 m ³
Effective storage capacity	1,244,210,800 m ³

Plate 8 shows the view of Krishnaraja Sagar dam. Fig. 23.12 shows the section of Krishnaraja Sagar dam.

23.10. UKAI DAM

Ukai dam, constructed at Fort Songardh, Gujarat, across river Tapi is a multipurpose dam, meant for hydro-power, irrigation and flood control. The salient data about the dam are as under.

Area of catchment	62,236 km ²
Mean annual runoff in the catchment	17,220,000,000 m ³
Rainfall	79 cm
Design flood discharge	49,490 m ³ /s
Year of commencement of construction work	1964
Year of completion	1972

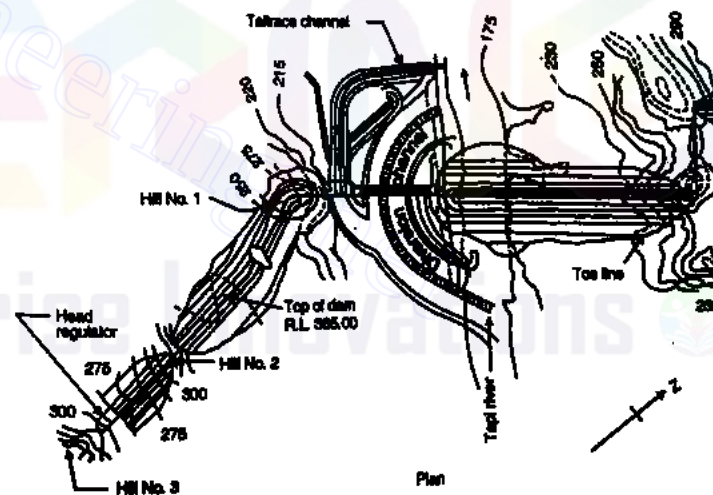
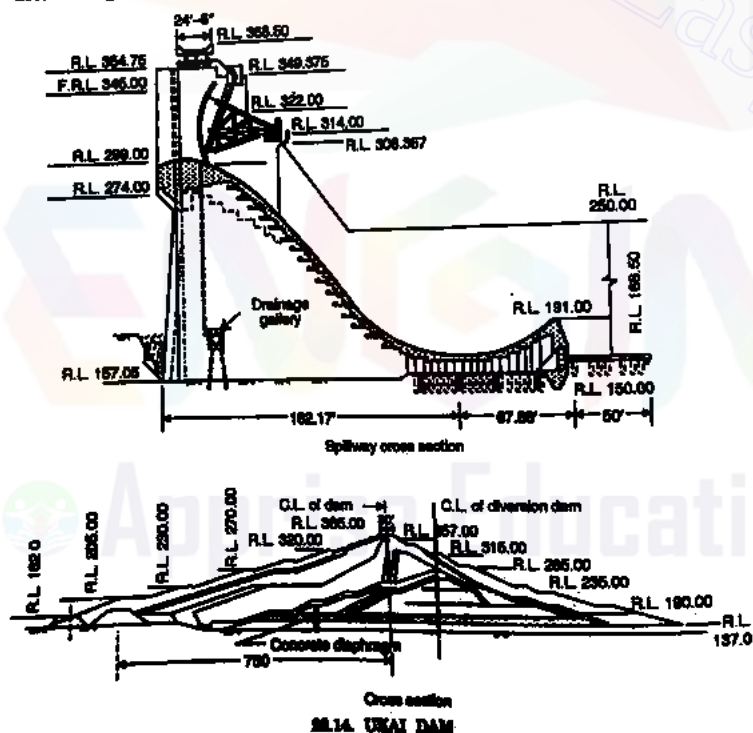


FIG. 23.13 UKAI DAM : PLAN.

IRRIGATION AND WATER POWER ENGINEERING

283

Dam	Earth-cum-gravity
Type	Basalt
Bed rock	
Maximum height above the lowest point of foundation	81 m
Length at the top of dam	4640 m
(i) Non overflow	425 m
(ii) Spillway	
Total volume content	
(i) Concrete	1,510,000 m ³
(ii) Earth	23,670,000 m ³
Spillway	Gated Ogee with ski-jump
Type	Radial crest gates-23 Nos.
Type and number of gates	
Maximum discharge capacity	35,960 m ³ /s
Reservoir	
Area at F.R.L.	601 km ²
Gross storage capacity	8,511,000,000 m ³



IMPORTANT DAMS OF INDIA

288

Effective storage capacity	7,092,000,000 m ³
Power Plant	
Type	Underground, River bed power house
Hydraulic head	57 m to 34 m
Maximum discharge	213 m ³ /s
Installed capacity	300 MW
Generating units	
Type	Generators-Umbrella type, vertical shaft, salient pole Turbines-Vertical shaft Kaplan turbine
Number	4 Nos
Capacity	75 MW each
Firm	193 MW at 35% load factor continuously

Fig. 23.13 shows the layout plan of the dam, showing various components.
Fig. 23.14 shows the spillway cross-section and earthdam cross section.

23.11. THEIN DAM

Thein Dam is located across river Ravi, Gurdaspur district, Punjab. It is a multipurpose project, meant for irrigation, hydropower and flood control. It is an earth core rock fill dam, with salient data as follows :

Area of catchment	6087 km ²
Mean annual runoff in the catchment	8,142,000,000 m ³
Design flood discharge	21,238 m ³ /s
Year of commencement of Construction work	1978
Year of completion	Under construction

Dam	
Type	Earth core rockfill dam
Bed rock	Alternate bands of sandstone and clay/silt stone
Maximum height above the lowest point of foundation	147 m
Length at the top of the dam	
(i) Non overflow	661 m
(ii) Spillway	217 m
Total volume content	
(i) Concrete and masonry	1,070,000 m ³
(ii) Earth	20,850,000 m ³
Spillway	
Type	Controlled gated ogee
Type and number of gates	12 Nos radial gates (15.25 m × 10.67 m)
Maximum discharge capacity	19,339 m ³ /s
Reservoir	
Area at F.R.L.	83 km ²
Gross storage capacity	3,670,000,000 m ³
Effective storage capacity	2,340,000,000 m ³
Available drawdown	40.23 m
Power Plant	
Hydraulic head	131 m

Maximum discharge.....	600 m ³ /s
Installed capacity.....	480 MW
Generating units	
Type.....	Vertical axis Francis reaction turbine
Number.....	4 Nos.
Capacity.....	120 MW each
Firm.....	138 MW at 100% load factor or 222 MW at 60% load factor

Fig. 23.15 shows the layout of various components of the project. Fig. 23.16 shows the typical earth dam section at maximum height.

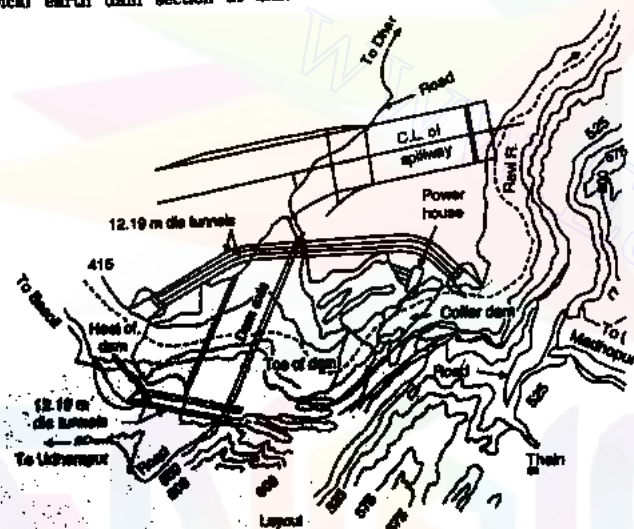


FIG. 23.15. TEHRI DAM : LAYOUT

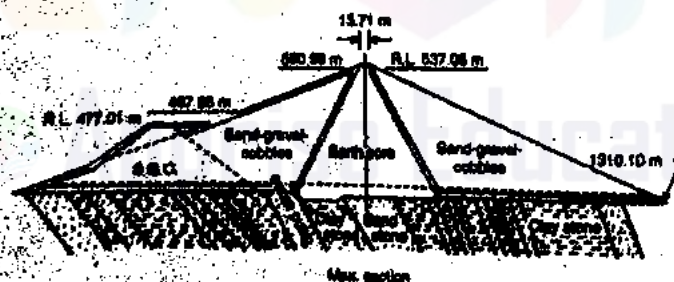


FIG. 23.16. TEHRI DAM : EARTH DAM SECTION

23.12. TEHRI DAM

Tehri dam is located at new Tehri town, Tehri district, Uttar Pradesh. Constructed across river Bhagirathi, it is a combined irrigation and hydropower project. The salient data of the project are as follows :

Area of catchment.....	7,511 km ² out of which 2,323 km ² is snow bound
Rainfall.....	Min. 102 cm Max. 178 cm
Design flood discharge.....	15,300 m ³ /s

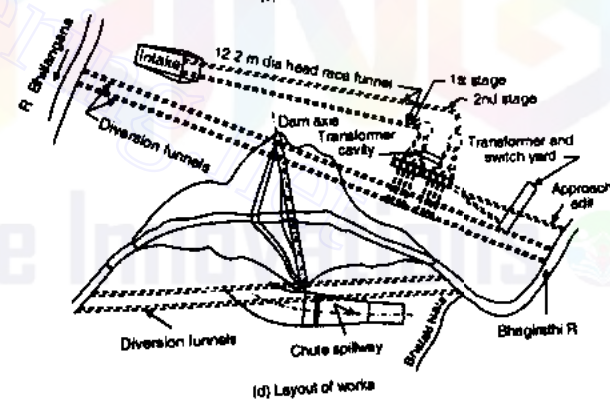
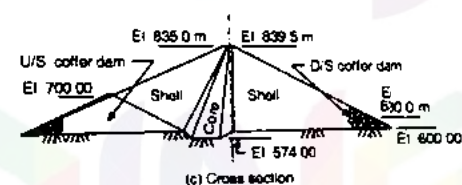
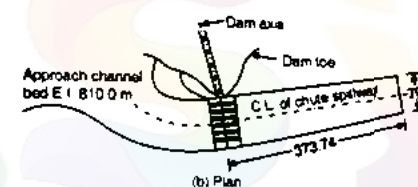
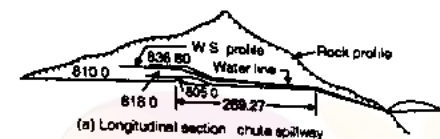


FIG. 23.17. TEHRI DAM

Maximum discharge.....	500 m ³ /s
Installed capacity.....	480 MW
Generating units.....	Vertical axis Francis reaction turbine
Type.....	4 Nos.
Number.....	120 MW each
Capacity.....	138 MW at 100% load factor or 222 MW
Firm.....	at 60% load factor

Fig. 23.15 shows the layout of various components of the project. Fig. 23.16 shows the typical earth dam section at maximum height.

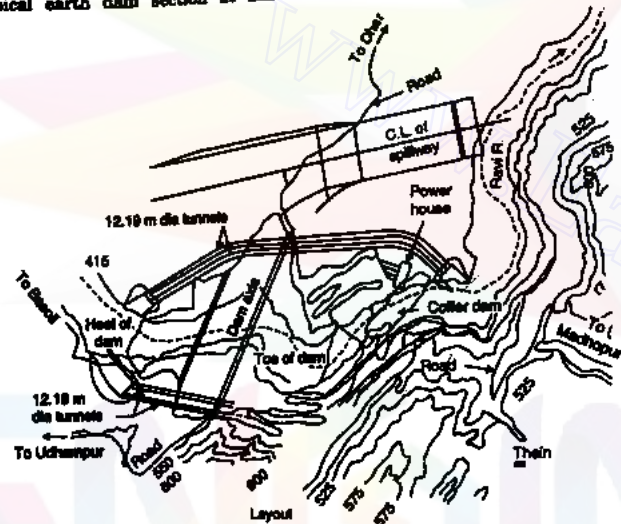


FIG. 23.15. TEHRI DAM : LAYOUT

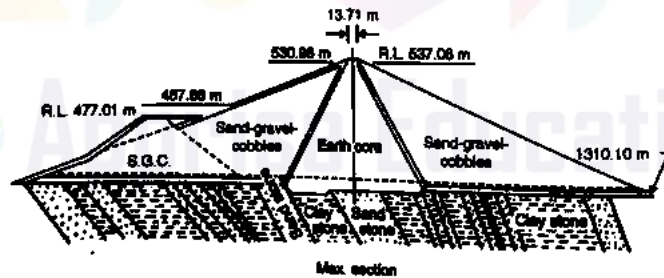


FIG. 23.16. TEHRI DAM : EARTH DAM SECTION

23.12. TEHRI DAM

Tehri dam is located at new Tehri town, Tehri district, Uttar Pradesh. Constructed across river Bhagirathi, it is a combined irrigation and hydropower project. The salient data of the project are as follows :

Area of catchment.....	7,511 km ² out of which 2,323 km ² is snow bound
Rainfall.....	Min. 102 cm Max. 178 cm
Design flood discharge.....	15,300 m ³ /s

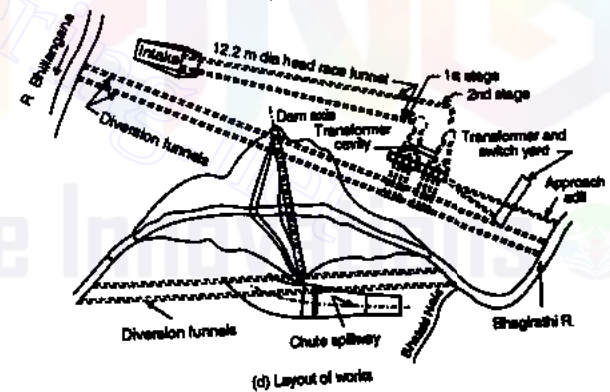
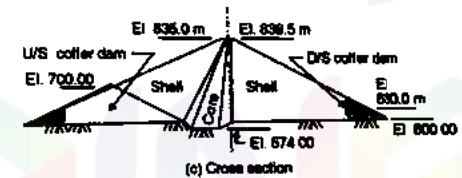
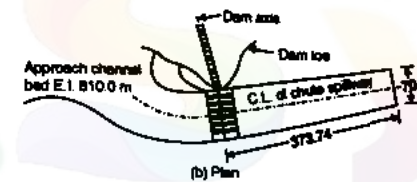
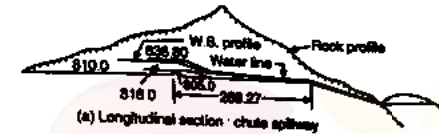


FIG. 23.17. TEHRI DAM

IRRIGATION AND WATER POWER ENGINEERING

936

Year of commencement of construction work	1978
Year of completion	Under Construction
Dam	
Type	Earth and rockfill
Bed rock	Slightly jointed phyllites of different grades
Maximum height above the lowest point of foundation	261 m
Length at the top of the dam	570 m
Total volume content	
(i) Concrete and masonry	550,000 m ³
(ii) Earth	22,000,000 m ³
Spillway	
Type	Open channel chute spillway
Type and number of gates	7 Nos. tainter gates
Maximum discharge capacity	13,150 m ³ /s
Reservoir	
Area at F.R.L.	42 km ²
Gross storage capacity	3,539,000,000 m ³
Effective storage capacity	2,615,000,000 m ³
Available drawdown	90 m
Power Plant	
Type	Underground power house on left bank
Hydraulic head	218 m
Maximum discharge	550 m ³ /s
Installed capacity	600 MW
Generating units	
Type	Synchronous vertical umbrells type
Number	4 Nos.
Capacity	150 MW each

Fig 23.17 (a) shows the layout of the works. Fig. 23.17 (b) shows the cross-section of earth dam. Fig. 23.17 (c) shows the plan and longitudinal section of chute spillway.

23.18. RAMGANGA DAM

Ramganga dam, constructed across river Ramganga, at Kalagarh (Bijnor), Uttar Pradesh, is a combined irrigation and hydro-power project. It is an earth and rockfill dam, with a chute spillway. It has a surface power house designed for submerged condition, utilising a hydraulic head of 98 m. The salient data about the project are as follows :

Area of catchment	3,134 km ²
Rainfall	155 cm
Design flood discharge	12,121 m ³ /s
Year of commencement of construction work	1964
Year of completion	1978

Dam

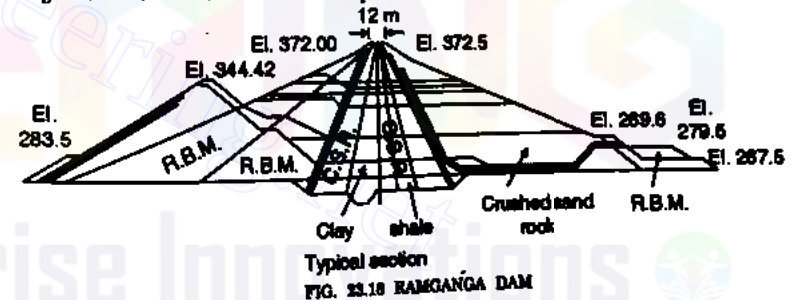
Type	Earth and rockfill dam
Bed rock	Alternate clay and sand rock bands with bands of silt stone

IMPORTANT DAMS OF INDIA

937

Maximum height above the lowest point of foundation	Main dam 128 m	Saddle dam 71 m
Length at the top of the dam		
(i) Non overflow	630 m	562 m
(ii) Spillway	113 m	
Total volume content		
(i) Concrete and masonry	195,000 m ³	
(ii) Earth	10,818,000 m ³	
Spillway		
Type	Chute spillway	Auxiliary spillway
Type and number of gates	5 Nos tainter gates	3 Nos tainter gates
Maximum discharge capacity	8,467 m ³ /s	1,926 m ³ /s
Reservoir		
Area at F.R.L.	78 km ²	
Gross storage capacity	2,442,000,000 m ³	
Effective storage capacity	2,186,500,000 m ³	
Power Plant		
Type	Surface Power House, designed for submerged condition	
Hydraulic head	98 m	
Maximum discharge	226 m ³ /s	
Installed capacity	198 MW	
Generating units		
Type	Francis turbine	
Number	3 Nos	
Capacity	66 MW each	

Fig 23.18 shows a typical section of the earth and rockfill dam having a max. height of 128 m above the lowest point of foundation.

**23.14. MAHI RAJAJ SAGAR DAM**

Mahi Bajaj Sagar dam is located in Banswara district, Rajasthan, constructed across Mahi river. It is a combined irrigation and hydro-power project, having dam sections of masonry and earth. The salient feature of the project are given below.

Area of catchment	6,149 km ²
Rainfall	82 cm

IRRIGATION AND WATER POWER ENGINEERING

Design flood discharge.....	25,595 m ³ /s
Year of commencement of construction work.....	1971
Dam	
Type.....	Masonry and earth
Bed rock.....	Alluvium consisting of sand gravel/cobble deposit
Maximum height above the lowest point of foundation.....	68 m
Length at the top of the dam.....	2862 m
(i) Non-overflow.....	300 m
(ii) Spillway.....	300 m
Total volume content.....	
(i) Concrete and masonry.....	781,000 m ³
(ii) Earth.....	5,324,000 m ³
Spillway	
Type.....	W.E.S. Type
Type and number of gates.....	16 radial gates (15.0 m × 12.25 m)
Maximum discharge capacity.....	23,785 m ³ /s
Reservoir	
Area at F.R.L.....	133 km ²
Gross storage capacity.....	2,069,000,000 m ³
Effective storage capacity.....	1,715,000,000 m ³
Power Plant	
Type.....	Two indoor type surface power houses
Hydraulic head.....	At Power House 1-40 m. At Power House 2-54 m
Maximum discharge.....	186 m ³ /s
Installed capacity.....	140 MW
Generating units	
Type.....	Francis turbine
Number and capacity.....	Power House No 1-2 units of 25 MW each Power House No 2-2 units of 45 MW each
Firm.....	29 MW

Fig. 23.19 shows the typical section of earthdam

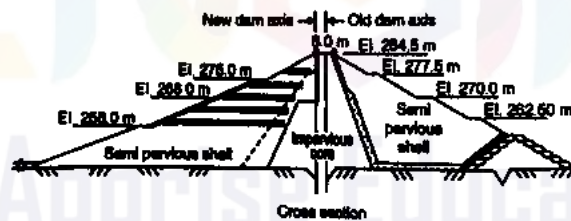


FIG. 23.19 MAHI BAJAJ SAGAR EARTH DAM

23.15. UJJANI DAM

This dam is located at Ujjani Village, Village, Sholapur district, Maharashtra. Constructed across river Bhima, it is a combined irrigation and hydro-power project. The dam is earth-cum-gravity. The salient data of the project are given below.

IMPORTANT DAMS OF INDIA

Area of catchment.....	14,860 km ²
Mean annual runoff in the catchment.....	8,712,000,000 m ³
Rainfall.....	69 cm
Design flood discharge.....	18,013 m ³ /s
Year of commencement of construction work.....	1979

Dam	
Type.....	Earth-cum-gravity
Bed rock.....	Massive basalt
Point of foundation.....	56 m
Length at the top of the dam.....	
(i) Non overflow.....	1,864 m
(ii) Spillway.....	603 m
Total volume content.....	
(i) Concrete and masonry.....	680,000 m ³
(ii) Earth.....	1,116,000 m ³
Spillway	
Type.....	Controlled
Type and number of gates.....	Radial gates - 41 Nos. of size 18 m × 6.5 m
Maximum discharge capacity.....	18,013 m ³ /s
Reservoir	
Area at F.R.L.....	337 km ²

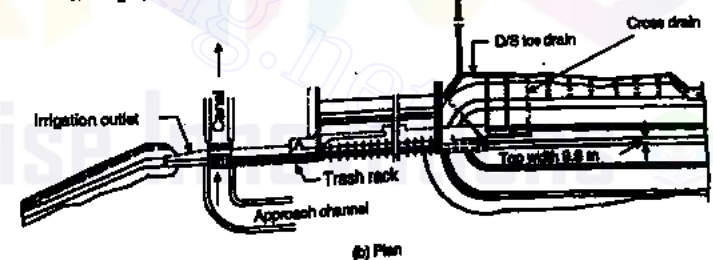
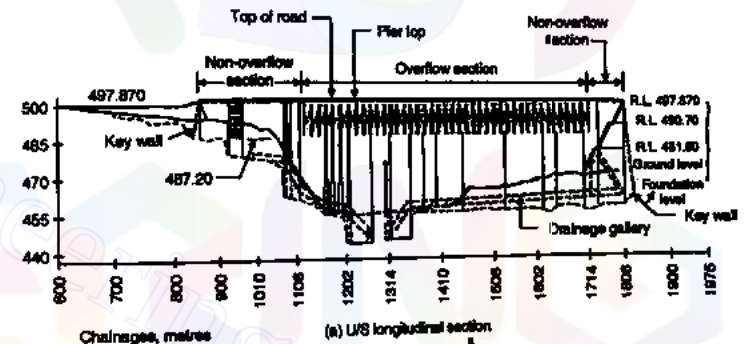


FIG. 23.20. UJJANI DAM.

Gross storage capacity.....3,140,000,000 m³
 Effective storage capacity.....1,440,000,000 m³
 Power Plant.....There is provision of one penstock of 3.2 m dia. in the dam. It is meant for generating 12 MW power seasonally

Fig. 23.20 shows the plan and longitudinal section of the dam. Fig. 23.21 shows typical section of earth dam while Fig. 23.22 show maximum over flow section.

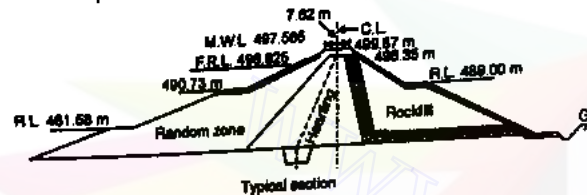


FIG. 23.21. TYPICAL EARTH DAM SECTION: UJANI DAM.

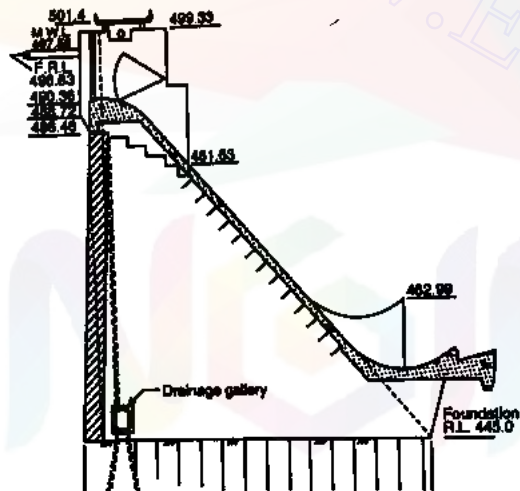


FIG. 23.22. MAX. OVERFLOW SECTION UJANI DAM.

23.16. TUNGBHADRA DAM.

Tungbhadra dam is located near Hospet town Karnataka and has been constructed across river Tungbhadra. It is a combined irrigation and hydropower project. It has masonry dam with central spillway section, and earth dam and composite dam on the left side. The salient data of the project are as under.

Area of catchment.....27,853 km²
 Rainfall.....104 cm

IMPORTANT DAMS OF INDIA

Design flood discharge.....18,408 m³/s
 Year of commencement of construction work.....1945
 Year of completion.....1968

Dam

Type.....Masonry dam with central spillway section
 Earthen dam and composite dam on the left side

Bed rock.....Hard rock

Maximum height above the lowest point of foundation.....49 m

Length at the top of the dam

(i) Non overflow.....1,748 m

(ii) Spillway.....701 m

Total volume content.....1,155,000 m³

Spillway

Type.....Ogee type spillway

Type and number of gates.....33 Nos.

Maximum discharge capacity.....18,408 m³/s

Reservoir

Area at F.R.L.....378 km²

Gross storage capacity.....3,787,000,000 m³

Effective storage capacity.....3,701,000,000 m³

Available drawdown.....25 m

Power plant

Generating Units

Number.....14 units of 9 MW each

Capacity.....126 MW

Fig. 23.23 shows the non-spillway section and spillway section of the Tungbhadra dam.

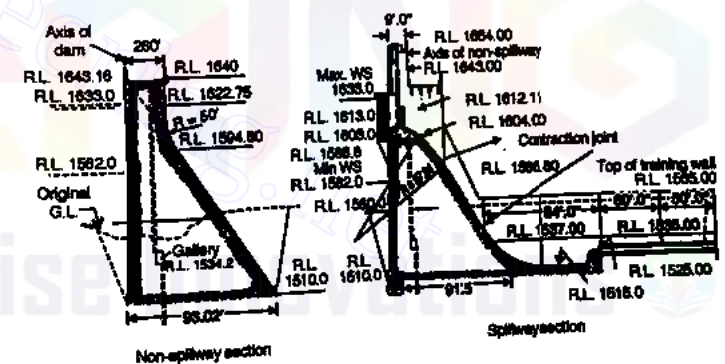


FIG. 23.23. TUNGBHADRA DAM

23.17. DANTIWADA DAM

This dam is located at Dantiwada village, Banaskantha District, Gujrat. It is mainly an irrigation dam, constructed across river Banas. It is a composite rolled filled earth dam with masonry spillway having radial gates. The salient data about the project are as under.

Area of catchment.....	2,862 km ²
Mean annual runoff in the catchment.....	447,830,000 m ³
Rainfall.....	94 cm
Design flood discharge.....	6,555 m ³ /s
Year of commencement of construction work.....	1958
Year of completion.....	1968

Dam	
Type.....	Composite rolled filled earthen dam with masonry spillway
Bed rock.....	Prampura granite
Maximum height above the lowest point of foundation.....	61 m
Length at the top of the dam	
(i) Non overflow.....	4,719 m
(ii) Spillway.....	182 m
Total volume content	
(i) Concrete.....	32,000 m ³
(ii) Earth.....	6,200,000 m ³ /s

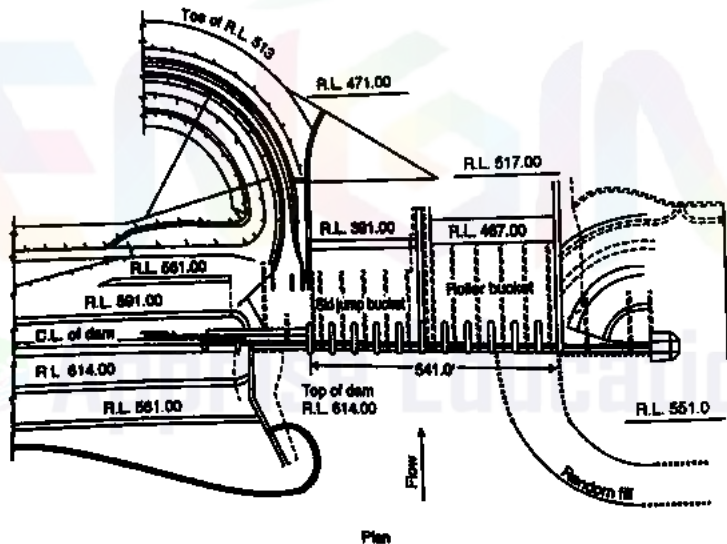


FIG. 23.24. LAYOUT OF DANTIWADA DAM.

IMPORTANT DAMS OF INDIA**Spillway**

Type.....	Masonry gated spillway
Type and number of gates.....	11 Nos radial gates
Maximum discharge capacity.....	6,865 m ³ /s

Reservoir

Area at F.R.L.....	42 km ²
Gross storage capacity.....	484,450,000 m ³
Effective storage capacity.....	444,770,000 m ³

Fig. 23.24 shows the layout of various components of the dam and spillway.

Fig. 23.25 shows the typical earth dam cross-section and the elevation along the length of the dam.

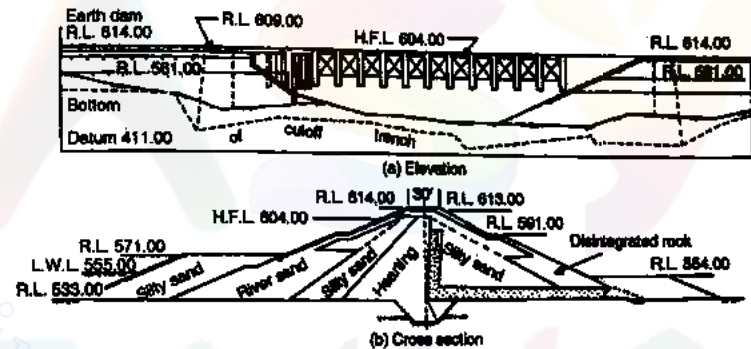


FIG. 23.25. DANTIWADA DAM.

APPENDIX-1

Khosla's Theory

1. COMPOSITE FLOOR PROFILE

Fig. 1 shows a composite floor profile, the most general case analysed by A.N.Khosla, Dr. N.K.Boes and Dr. E.M.Taylor (1936) by method of Schwarz-Christoffel transformation. The flow lines and equipotential lines of the actual section are no longer ellipses and hyperbolas. By the mathematical process of transformation, the distorted flow net can be brought back to the normal elliptical and hyperbolic curves.

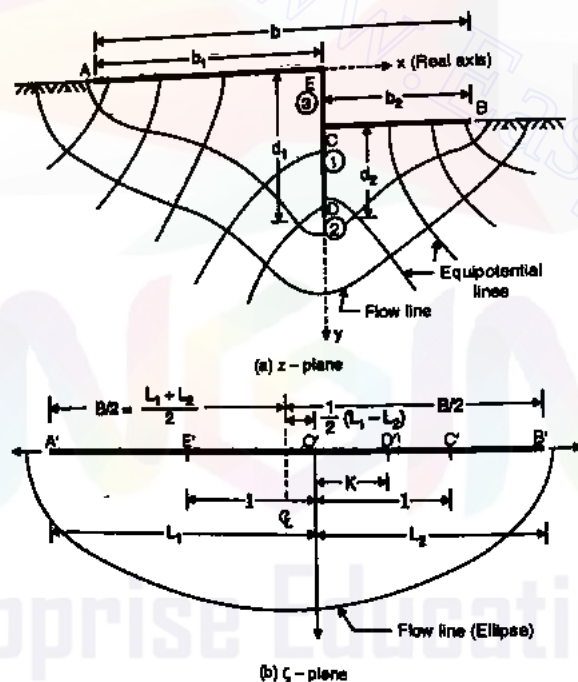


FIG. 1.

If a polygon is located in the Z-plane, then the Schwarz-Christoffel equation, that maps it conformally on to ζ -plane is given by

(944)

APPENDIX-1

945

$$z = A \int \frac{d\zeta}{(\zeta - \zeta_1)^{\lambda_1} (\zeta - \zeta_2)^{\lambda_2} \dots (\zeta - \zeta_n)^{\lambda_n}} \quad \dots(1)$$

where $\pi\lambda_1, \pi\lambda_2, \dots, \pi\lambda_n$ = change in the angles of the polygon which are required to transform it into a straight line.

$\zeta_1, \zeta_2, \dots, \zeta_n$ = co-ordinates of its vertices.

A = complex constant.

Step 1. Transformation Equation $z = f(\zeta)$: Let us now apply this equation to transform the composite profile in Fig. 1(a) into straight line profile of Fig. 1(b). The key points A, E, D, C and B of Fig. 1(a) fall into points A', E', D', C', and B', respectively in Fig. 1(b). It will be seen that the angles at vertices E, D and C are respectively $\frac{\pi}{2}$, 2π and $\frac{\pi}{2}$, and these have been changed to π each at E', D' and C' on transformation. The Schwarz-Christoffel formula is :

$$z = A \int \frac{d\zeta}{(\zeta - \zeta_1)^{\lambda_1} (\zeta - \zeta_2)^{\lambda_2} (\zeta - \zeta_3)^{\lambda_3}} \quad \dots(1)$$

where

$\pi\lambda_1$ = change in the angle at C = $\pi - \pi/2 = +\pi/2$

$\pi\lambda_2$ = change in the angle at D = $\pi - 2\pi = -\pi$

$\pi\lambda_3$ = change in the angle at E = $\pi - \pi/2 = +\pi/2$

Hence $\lambda_1 = \frac{1}{2}$; $\lambda_2 = -1$ and $\lambda_3 = \frac{1}{2}$.

Also, selecting the origin O' midway between C' and E', O'E' and O'C' will be equal, and can be selected to be equal to unity. Let O'D' = K, where K is an absolute number.

Then,

ζ_1 = co-ordinate of C = +1

ζ_2 = co-ordinate of D = K

ζ_3 = co-ordinate of E = -1

Substituting these in (1), we get

$$z = A \int \frac{d\zeta}{(\zeta - 1)^{1/2} (\zeta - K)^{-1} (\zeta + 1)^{1/2}} \quad \dots(2)$$

or

$$z = A \int \frac{(\zeta - K) d\zeta}{\sqrt{(\zeta^2 - 1)}}$$

Integrating the above, we get

$$z = A \sqrt{\zeta^2 - 1} - AK \log_e (\zeta + \sqrt{\zeta^2 - 1}) + B \quad \dots(3)$$

Step 2. Determination of constants A and B : In order to find the constant B, let us compare the co-ordinates of point C and C' in the two planes. Evidently, the co-ordinate z of C = $i(d_1 - d_2)$ while that of C', $\zeta = 1$.

Substituting these in (2), we get

$$B = i(d_1 - d_2)$$

Similarly, the co-ordinates of E and E' are : $z = 0$ and $\zeta = -1$.

Substituting these in (3), we get
 $0 = B - AK \log_e(-1)$

$$\text{or } AK\pi i = B = i(d_1 - d_2)$$

[Since $\log_e(-1) = \pi i$]

$$\therefore AK = \frac{d_1 - d_2}{\pi} \quad \text{and} \quad A = \frac{d_1 - d_2}{\pi K} \quad \dots(4)$$

Step 3. Determination of Parameter K

Comparing the co-ordinates of points D and D' , we get
 $z = id_1$ and $\zeta = K$

Substituting these in Eq. (3), we have

$$id_1 = A\sqrt{K^2 - 1} - AK \log_e(K + \sqrt{K^2 - 1}) + i(d_1 - d_2)$$

or

$$id_1 = A i \sqrt{1 - K^2} - AK \log_e(K + i \sqrt{1 - K^2}) + i(d_1 - d_2)$$

Putting $K = \cos \theta$, we get

$$id_1 = A i \sin \theta - AK \log_e(\cos \theta + i \sin \theta) + i(d_1 - d_2)$$

or

$$id_1 = A i \sin \theta - AK i \theta + i(d_1 - d_2)$$

$$d_2 = A \sin \theta - AK \theta$$

Substituting the values of A and AK from Eq. 4, we get

$$d_2 = \frac{d_1 - d_2}{\pi K} \sin \theta - \frac{d_1 - d_2}{\pi} \theta$$

or

$$d_2 = \frac{d_1 - d_2}{\pi} \tan \theta - \frac{d_1 - d_2}{\pi} \theta$$

$$\text{Hence, } \frac{d_2}{d_1 - d_2} \pi = \tan \theta - \theta$$

or

$$\pi \delta = \tan \theta - \theta = \frac{\sqrt{1 - K^2}}{K} - \cos^{-1} K \quad \dots(5)$$

where

$$\delta = \frac{d_2}{d_1 - d_2}$$

Thus, from Eq. 5 the values of K for various values of δ can be determined.

Step 4. Final Transformation Equation : Substituting the values of A and AK in Eq. 3, we get the final transformation formula

$$z = \frac{d_1 - d_2}{\pi K} \sqrt{\zeta^2 - 1} - \frac{d_1 - d_2}{\pi} \log_e(\zeta + \sqrt{\zeta^2 - 1}) + i(d_1 - d_2) \quad \dots(1)$$

where K is now a known parameter.

Step 5. Determination of co-ordinates of A' and B'

Equation (1) is applicable to all points on the profile of the floor. Let the ζ co-ordinates of point A' and B' be $-L_1$ and $+L_2$ respectively. The z co-ordinates of the corresponding points A and B are $-b_1$ and $[b_2 + i(d_1 - d_2)]$ respectively. Substituting these co-ordinates, separately in Eq.(1), we get

$$-b_1 = \frac{d_1 - d_2}{\pi K} \sqrt{L_1^2 - 1} - \frac{d_1 - d_2}{\pi} \log_e(-L_1 + \sqrt{L_1^2 - 1}) + i(d_1 - d_2)$$

or

$$-b_1 = \frac{d_1 - d_2}{\pi K} \sqrt{L_1^2 - 1} - \frac{d_1 - d_2}{\pi} \log_e(L_1 - \sqrt{L_1^2 - 1}) \quad \dots(6)$$

and

$$b_2 + i(d_1 - d_2) = \frac{d_1 - d_2}{\pi K} \sqrt{L_2^2 - 1} - \frac{d_1 - d_2}{\pi} \log_e(L_2 + \sqrt{L_2^2 - 1}) + i(d_1 - d_2)$$

or

$$b_2 = \frac{d_1 - d_2}{\pi K} \sqrt{L_2^2 - 1} - \frac{d_1 - d_2}{\pi} \log_e(L_2 + \sqrt{L_2^2 - 1}) \quad \dots(7)$$

Assuming $L_1 = \cosh \gamma_1$ and $L_2 = \cosh \gamma_2$ and substituting these in the above, we get

$$-b_1 = \frac{d_1 - d_2}{\pi K} \sinh \gamma_1 + \frac{d_1 - d_2}{\pi} \gamma_1$$

or

$$\sinh \gamma_1 + K\gamma_1 = -\frac{\pi K b_1}{d_1 - d_2}$$

and

$$\sinh \gamma_2 - K\gamma_2 = \frac{\pi K b_2}{d_1 - d_2}$$

Again let $\delta_1 = \frac{b_1}{d_1 - d_2}$ and $\delta_2 = \frac{b_2}{d_1 - d_2}$. Substituting in the above, we get

$$\sinh \gamma_1 + K\gamma_1 = -\pi K \delta_1 \quad \dots(8)$$

$$\sinh \gamma_2 - K\gamma_2 = \pi K \delta_2 \quad \dots(9)$$

Thus, from Eqs.(8) and (9), γ_1 and γ_2 can be calculated. Hence, $L_1 = \cosh \gamma_1$ and $L_2 = \cosh \gamma_2$ can be known. Thus, corresponding to given set of values of floor dimensions b_1, b_2, d_1 and d_2 we can first get K , and then L_1 and L_2 .

Step 6. Pressure distribution below the floor : Now, in order to get the pressure distribution under the floor, let us consider Eq. 1 in § 12.8, we have seen that for the case of a horizontal floor (Fig. 12.9)

$$x + iy = z = \frac{b}{2} \cosh w = \frac{b}{2} \cosh(u + iv).$$

Applying this to the transformed floor of Fig. 1(b) where $\frac{b}{2}$ = half the width of the floor $= \frac{1}{2}(L_1 + L_2)$ and $z = \zeta + \frac{1}{2}(L_1 - L_2)$, we get

$$\zeta + \frac{1}{2}(L_1 - L_2) = \frac{1}{2}(L_1 + L_2) \cosh w$$

$$\text{Putting } \frac{L_1 + L_2}{2} = \lambda \text{ and } \frac{L_1 - L_2}{2} = \lambda_1, \text{ we get} \quad \dots(11)$$

$$\zeta = \lambda \cosh w - \lambda_1$$

Substituting this in Eq. (1), we get a general equation for the flow net diagram as under :

$$z = \frac{d_1 - d_2}{\pi K} \sqrt{(\lambda \cosh w - \lambda_1)^2 - 1} - \frac{d_1 - d_2}{\pi} \log_e \left[\frac{|\lambda \cosh w - \lambda_1|}{\sqrt{(\lambda \cosh w - \lambda_1)^2 - 1}} \right] + i(d_1 - d_2)$$

For the first stream line which hugs to the base, we have $u = 0$ and hence $w = u + iv = iv$.

Also $z = x + iy$. Hence

$$x + iy = \frac{d_1 - d_2}{\pi K} \sqrt{(\lambda \cos v - \lambda_1)^2 - 1} - \frac{d_1 - d_2}{\pi} \log_e \left\{ \frac{(\lambda \cos v - \lambda_1)}{\sqrt{(\lambda \cos v - \lambda_1)^2 - 1}} \right\} + i(d_1 - d_2) \quad \dots(III)$$

From Eq. (III), we can get the pressure function v for any point whose co-ordinate is $(x + iy)$. We shall apply this equation to different regions of the floor of Fig. 1(a).

Step 7. Pressure Distribution for Length AE

In this length, $y = 0$. Also, introducing a new variable defined by the equation

$$\lambda \cos v - \lambda_1 = -\cosh \beta_1' \quad \dots(10)$$

we get, from Eq. (III),

$$x = \frac{d_1 - d_2}{\pi K} \sinh \beta_1' + \frac{d_1 - d_2}{\pi} \log_e \left[\frac{-\cosh \beta_1' + \sinh \beta_1'}{\sinh \beta_1'} \right] + i(d_1 - d_2)$$

or

$$x = \frac{d_1 - d_2}{\pi K} \sinh \beta_1' + \frac{d_1 - d_2}{\pi} \beta_1' - \frac{d_1 - d_2}{\pi} \log_e (-1) + i(d_1 - d_2)$$

or

$$x = \frac{d_1 - d_2}{\pi K} \sinh \beta_1' + \frac{d_1 - d_2}{\pi} \beta_1' \quad \dots(11)$$

$$\text{Hence } \frac{x}{d_1 - d_2} \pi K = \sinh \beta_1' + K \beta_1'$$

From this equation, β_1' can be calculated. Hence from Eq. (10), the pressure function v is found to be

$$v = \cos^{-1} \left(\frac{\lambda_1 - \cosh \beta_1'}{\lambda} \right) = \pi \frac{P}{H}$$

$$P = \frac{H}{\pi} \cos^{-1} \frac{\lambda_1 - \cosh \beta_1'}{\lambda} \quad \dots[IV (a)]$$

$$\phi = \frac{P}{H} = \frac{1}{\pi} \cos^{-1} \frac{\lambda_1 - \cosh \beta_1'}{\lambda} \quad \dots[IV (b)]$$

This gives pressure distribution under AE

Step 8. Pressure distribution for Length CB : Introducing a new variable β_2' defined by the equation $\lambda \cos v - \lambda_1 = \cosh \beta_2'$, we get from Eq. (III)

$$\sinh \beta_2' - K \beta_2' = \pi K \frac{x}{d_1 - d_2} = \delta_x \pi K \quad \dots(12)$$

From this equation β_2' can be known. Hence the pressure distribution is given by

$$v = \cos^{-1} \frac{\lambda_1 + \cosh \beta_2'}{\lambda} = \pi \frac{P}{H}$$

APPENDIX-I

$$\text{Hence } P = \frac{H}{\pi} \cos^{-1} \frac{\lambda_1 + \cosh \beta_2'}{\lambda} \quad \dots[V(a)]$$

$$\phi = \frac{1}{\pi} \cos^{-1} \left(\frac{\lambda_1 + \cosh \beta_2'}{\lambda} \right) \quad \dots[V(b)]$$

Combining Eqs. [IV(a)] and [V(a)] in one common form, we get

$$P = \frac{H}{\pi} \cos^{-1} \left(\frac{\lambda_1 + \cosh \beta_{1,2}'}{\lambda} \right) \quad \dots(VI)$$

where - sign is used for upstream and + sign for the downstream part of the floor.

Step 9. Pressure distribution along ED and DC : For the pressure distribution along the u/s and d/s faces of the pile, we have $x = 0$. Hence from the Eq. (III)

$$iy = \frac{d_1 - d_2}{\pi K} \sqrt{(\lambda \cos v - \lambda_1)^2 - 1} - \frac{d_1 - d_2}{\pi} \log_e \left[\frac{|\lambda \cos v - \lambda_1|}{\sqrt{(\lambda \cos v - \lambda_1)^2 - 1}} \right] + i(d_1 - d_2)$$

Introducing a new variable β defined by the equation

$$\lambda \cos v - \lambda_1 = \cos \beta, \text{ we get}$$

$$iy = \frac{d_1 - d_2}{\pi K} i \sin \beta - \frac{d_1 - d_2}{\pi} i \beta + i(d_1 - d_2)$$

or

$$\left(\frac{y}{d_1 - d_2} - 1 \right) \pi K = \sin \beta - K \beta \quad \dots(13)$$

From which β can be calculated.

$$\text{Hence } v = \cos^{-1} \left(\frac{\lambda_1 + \cos \beta}{\lambda} \right) = \pi \frac{P}{H}$$

$$P = \frac{H}{\pi} \cos^{-1} \left(\frac{\lambda_1 + \cos \beta}{\lambda} \right) \quad \dots(VII)$$

Step 10. Pressure at key points

(i) **Pressure at E**

For E' A' ,

At E' ,

Hence

$$\zeta = \lambda \cos v - \lambda_1 \text{ (From Eq. II)}$$

$$\zeta = -1$$

$$-1 = \lambda \cos v - \lambda_1$$

$$v = \pi \frac{P}{H} = \cos^{-1} \frac{\lambda_1 - 1}{\lambda}$$

$$P_E = \frac{H}{\pi} \cos^{-1} \frac{\lambda_1 - 1}{\lambda} \quad \dots[VIII(a)]$$

or

or

(ii) **Pressure at C**

For C' B' ,

At C' ,

$$\zeta = \lambda \cos v - \lambda_1 \text{ (From Eq. II)}$$

$$\zeta = +1$$

$$+1 = \lambda \cos v - \lambda_1$$

$$v = \pi \frac{P}{H} = \cos^{-1} \frac{\lambda_1 + 1}{\lambda}$$

or

IRRIGATION AND WATER POWER ENGINEERING

$$P_C = \frac{H}{\pi} \cos^{-1} \frac{\lambda_1 + 1}{\lambda}$$

...[VIII(b)]

(iii) Pressure at D

At point D

$$\zeta = \lambda \cos v - \lambda_1 = K$$

[VIII(c)]

or

$$P_D = \frac{H}{\pi} \cos^{-1} \frac{\lambda_1 + K}{\lambda}$$

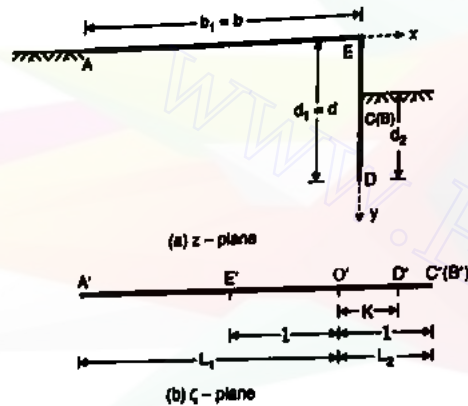
2. SPECIAL CASE : $b_2 = 0$ 

FIG. 2

Let us now take the case when $b_2 = 0$ and $b_1 = b$ as shown in Fig. 2.

$$\delta_2 = \frac{b_2}{d_1 - d_2} = 0$$

Hence from Eq. (9),

$$\sinh \gamma_2 - K \gamma_2 = \pi K \delta_2 = 0$$

or

$$\gamma_2 = 0$$

$$L_2 = \cosh 0 = 1$$

This could also be found from Fig. 2.

Now,

$$\lambda = \frac{L_1 + 1}{2}$$

$$\lambda_1 = \frac{L_1 - 1}{2} = \lambda - 1$$

$$\lambda_1 - 1 = \frac{L_1 - 3}{2}$$

$$\lambda_1 + 1 = \frac{L_1 + 1}{2} = \lambda$$

APPENDIX-I

261

Hence from Eq. (VIII),

$$P_z = \frac{H}{\pi} \cos^{-1} \left[\frac{\lambda_1 - 1}{\lambda} \right] = \frac{H}{\pi} \cos^{-1} \frac{L_1 - 3}{L_1 + 1}$$

...[IX (a)]

$$P_C = \frac{H}{\pi} \cos^{-1} \left[\frac{\lambda_1 + 1}{\lambda} \right] = \frac{H}{\pi} \cos^{-1} (1)$$

or

$$P_C = 0$$

...[IX (b)]

$$P_D = \frac{H}{\pi} \cos^{-1} \left[\frac{\lambda_1 + K}{\lambda} \right] = \frac{H}{\pi} \cos^{-1} \left[\frac{2K + L_1 - 1}{L_1 + 1} \right]$$

...[IX (c)]

Exit Gradient.

Along the sheet pile, we get from Eq. 13,

$$\left[\frac{y}{d_1 - d_2} - 1 \right] \pi K = \sin \beta - K \beta$$

or

$$y = \frac{d_1 - d_2}{\pi K} \sin \beta - \frac{d_1 - d_2}{\pi} \beta + (d_1 - d_2)$$

To find the exit gradient at C, differentiate the above with respect to y :

$$\frac{dy}{dy} = 1 = \frac{d_1 - d_2}{\pi K} \cos \beta \frac{d\beta}{dy} - \frac{d_1 - d_2}{\pi} \frac{d\beta}{dy}$$

But

$$\lambda \cos v - \lambda_1 = \cos \beta$$

$$\lambda \sin v \frac{dv}{dy} = \sin \beta \frac{d\beta}{dy}$$

or

$$\frac{d\beta}{dy} = \lambda \frac{\sin v}{\sin \beta} \frac{dv}{dy}$$

...[14]

Substituting this, we get

$$1 = \frac{d_1 - d_2}{\pi K} \cos \beta \cdot \lambda \frac{\sin v}{\sin \beta} \frac{dv}{dy} - \frac{d_1 - d_2}{\pi} \lambda \frac{\sin v}{\sin \beta} \frac{dv}{dy}$$

...[15]

At the exit end $v = 0$

$$\cos \beta = \lambda \cos v - \lambda_1 = \lambda - \lambda_1 = 1.$$

$$\beta = 0$$

Hence when $v = 0$, $\beta = 0$

or

$$\frac{\sin v}{\sin \beta} = \frac{0}{0}$$

It is, therefore, essential to determine the limit of $\left(\frac{\sin v}{\sin \beta} \right)_{v=0, \beta=0}$

$$\begin{aligned} \text{Lt} \left(\frac{\sin v}{\sin \beta} \right)_{v=0, \beta=0} &= \text{Lt} \left(\frac{\frac{d}{dy} (\sin v)}{\frac{d}{dy} (\sin \beta)} \right)_{v=0, \beta=0} = \text{Lt} \left(\frac{\cos v \cdot \frac{dv}{dy}}{\cos \beta \cdot \frac{d\beta}{dy}} \right)_{v=0, \beta=0} \\ &= \text{Lt} \left(\frac{\cos v}{\cos \beta} \cdot \frac{\sin \beta}{\lambda \sin v} \right)_{v=0, \beta=0} \end{aligned}$$

From (14)

$$= \text{Lt} \left(\frac{\sin \beta}{\lambda \sin \nu} \right)_{\nu=0, \beta=0}$$

$$\therefore \left[\text{Lt} \left(\frac{\sin \nu}{\sin \beta} \right) \right]_{\nu=0, \beta=0} = \frac{1}{\lambda}$$

$$\text{or} \quad \text{Lt} \left(\frac{\sin \nu}{\sin \beta} \right)_{\nu=0, \beta=0} = \frac{1}{\sqrt{\lambda}}$$

Substituting this in 15, we get

$$1 = \frac{d_1 - d_2}{\pi K} \cdot \frac{\lambda}{\sqrt{\lambda}} \left[\frac{dv}{dy} \right]_{\nu=0} - \frac{d_1 - d_2}{\pi} \frac{\lambda}{\sqrt{\lambda}} \left(\frac{dv}{dy} \right)_{\nu=0}$$

$$\left[\frac{dv}{dy} \right]_{\nu=0} = \frac{\pi K}{(d_1 - d_2) \sqrt{\lambda}} \cdot \frac{1}{1 - K}$$

$$\text{Now} \quad \left[\frac{dP}{dy} \right]_{\nu=0} = G_R = \frac{H}{\pi} \left[\frac{dv}{dy} \right]_{\nu=0} = \frac{H}{\pi} \cdot \frac{\pi K}{(d_1 - d_2) \sqrt{\lambda}} \cdot \frac{1}{1 - K}$$

$$G_R = \frac{H}{d_2} \cdot \frac{d_2}{d_1 - d_2} \cdot \frac{K}{1 - K} \cdot \frac{1}{\sqrt{\lambda}}$$

...(X)

End Sheet pile with $d_1 = d_2 = d$

For case where $d_1 = d_2$ we have, $K = 0$

$$\therefore \frac{K}{d_1 - d_2} = \frac{0}{0}$$

Let us find the value of the above limit. From Eq. (V),

$$\tan \theta - \theta = \pi \delta = \frac{\pi d_2}{d_1 - d_2}; \text{ also } \cos \theta = K$$

Hence multiplying left side by $\cos \theta$ and right side by K , we get

$$\sin \theta - \theta \cos \theta = \pi d_2 \frac{K}{d_1 - d_2}$$

For $K = 0$, $\theta = \pi/2$

Hence the above equation becomes

$$1 - 0 = \pi d_2 \left[\frac{K}{d_1 - d_2} \right]_{d_1=d_2}$$

$$\therefore \left[\text{Lt} \left(\frac{K}{d_1 - d_2} \right) \right]_{d_1=d_2=d} = \frac{1}{\pi d_2} = \frac{1}{\pi d}$$

...(15)

Hence from Eq. X,

$$G_R = \left(\frac{H}{1 - K} \right) \left(\frac{K}{d_1 - d_2} \right) \cdot \frac{1}{\sqrt{\lambda}} = \frac{H}{1} \cdot \frac{1}{\pi d} \cdot \frac{1}{\sqrt{\lambda}}$$

or

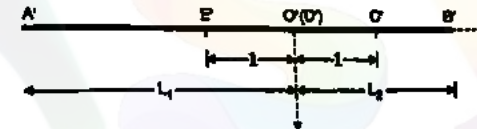
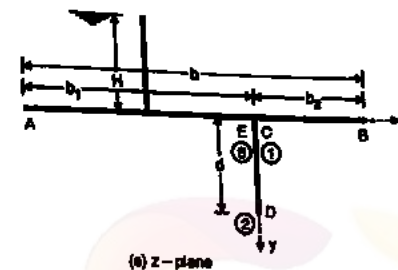
$$G_R = \frac{H}{d} \cdot \frac{1}{\pi \sqrt{\lambda}}$$

...(XI)

This is very important equation for exit gradient. When floor is absent (i.e. $b = 0$), $\lambda = 1$

$$G_R = \frac{H}{\pi d}$$

...(XII)



(a) Z-plane

(b) ζ-plane

FIG. 3.

3. ECCENTRIC SHEET PILE: INDEPENDENT SOLUTION

The Schwarz-Christoffel transformation equation is :

$$z = A \int \frac{d\zeta}{(\zeta - \zeta_1)^{1/2} (\zeta - \zeta_2)^{1/2} (\zeta - \zeta_3)^{1/2}} = A \int \frac{d\zeta}{(\zeta - 1)^{1/2} (\zeta - 0)^{-1} (\zeta + 1)^{1/2}} = A \int \frac{\zeta d\zeta}{\sqrt{\zeta^2 - 1}} \quad \dots(16)$$

or

$$z = A \sqrt{\zeta^2 - 1} + B$$

In order to determine constant B, consider co-ordinates of points C and C'

$$z = 0 \text{ and } \zeta = 1.$$

Hence from Eq. (16),

$$B = 0$$

In order to determine constant A, consider co-ordinates of D and D'

$$z = id \text{ and } \zeta = 0$$

Hence from Eq. (16),

$$id = A \sqrt{-1} = iA$$

or

$$A = d$$

Hence the transformation equation becomes,

$$z = d \sqrt{\zeta^2 - 1} \quad \dots(17)$$

Now, for the horizontal floor in ζ plane, we have

$$\zeta + \frac{1}{2}(L_1 - L_2) = \frac{1}{2}(L_1 + L_2) \cosh w$$

Putting $\frac{L_1 + L_2}{2} = \lambda$ and $\frac{L_1 - L_2}{2} = \lambda_1$

we get $\zeta = \lambda \cosh w - \lambda_1$

For the floor profile, $u = 0$, hence $\cosh w = \cosh iv = \cos v$

$$\zeta = \lambda \cos v - \lambda_1$$

Substituting this in Eq. (17), we get

$$z = d \sqrt{(\lambda \cos v - \lambda_1)^2 - 1} \quad \dots(19)$$

From the above equation, pressure distribution below the floor can be found.

For finding co-ordinates of A' and B', we get from Eq. (17),

$$-b_1 = d \sqrt{L_1^2 - 1}$$

$$b_2 = d \sqrt{L_2^2 - 1}$$

and

$$L_1 = \sqrt{\left(\frac{b_1}{d}\right)^2 + 1} = \sqrt{1 + \alpha_1^2} \quad \dots[20(a)]$$

and

$$L_2 = \sqrt{\left(\frac{b_2}{d}\right)^2 + 1} = \sqrt{1 + \alpha_2^2} \quad \dots[20(b)]$$

where

$$\alpha_1 = \frac{b_1}{d} \quad \text{and} \quad \alpha_2 = \frac{b_2}{d}$$

Hence

$$\lambda = \frac{L_1 + L_2}{2} = \frac{\sqrt{1 + \alpha_1^2} + \sqrt{1 + \alpha_2^2}}{2} \quad \dots[21(a)]$$

and

$$\lambda_1 = \frac{L_1 - L_2}{2} = \frac{\sqrt{1 + \alpha_1^2} - \sqrt{1 + \alpha_2^2}}{2} \quad \dots[21(b)]$$

For the floor, $y = 0$. Hence from Eq. (19),

$$z = x + iy = x = d \sqrt{(\lambda \cos v - \lambda_1)^2 - 1}$$

or

$$\lambda \cos v - \lambda_1 = \pm \sqrt{\frac{d^2 + x^2}{d^2}}$$

$$\cos v = \frac{\lambda_1 d \pm \sqrt{d^2 + x^2}}{\lambda d}$$

$$v = \pi \frac{P}{H} = \cos^{-1} \frac{\lambda_1 d \pm \sqrt{d^2 + x^2}}{\lambda d}$$

or

$$P = \frac{H}{\pi} \cos^{-1} \frac{\lambda_1 d \pm \sqrt{d^2 + x^2}}{\lambda d} \quad \dots(22)$$

where -ve sign is to be used for u/s floor and +ve sign is to be used for d/s floor.

For point E,

$$x = 0$$

$$P_E = \frac{H}{\pi} \cos^{-1} \left[\frac{\lambda_1 - 1}{\lambda} \right] \quad \dots[23(a)]$$

For point C,

$$x = 0$$

$$P_C = \frac{H}{\pi} \cos^{-1} \left[\frac{\lambda_1 + 1}{\lambda} \right] \quad \dots[23(b)]$$

For pressure along the sheet pile, put $x = 0$ in Eq.(19),

$$z = x + iy = iy = d \sqrt{(\lambda \cos v - \lambda_1)^2 - 1}$$

$$\text{or} \quad \lambda \cos v - \lambda_1 = \pm \sqrt{\frac{d^2 - y^2}{d^2}}$$

$$\cos v = \frac{\lambda_1 d \pm \sqrt{d^2 - y^2}}{\lambda d}$$

$$v = \pi \frac{P}{H} = \cos^{-1} \left(\frac{\lambda_1 d \pm \sqrt{d^2 - y^2}}{\lambda d} \right)$$

or

$$P = \frac{H}{\pi} \cos^{-1} \left(\frac{\lambda_1 d \pm \sqrt{d^2 - y^2}}{\lambda d} \right)$$

From point D,

$$y = d$$

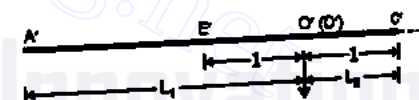
$$P_D = \frac{H}{\pi} \cos^{-1} \left(\frac{\lambda_1}{\lambda} \right) \quad \dots[23(c)]$$

The curves of Fig. 12.13 are based on Eqs. 23(a), (b), (c).

4. FLOOR WITH END SHEET PILE



(a) z-plane



(a) z-plane

FIG. 4

Schwarz-Christoffel transformation equation becomes :

$$z = A \int \frac{d\zeta}{(\zeta - 1)^{1/2} (\zeta - 0)^{-1} (\zeta + 1)^{1/2}} = A \int \frac{\zeta d\zeta}{\sqrt{\zeta^2 - 1}}$$

or

$$z = A\sqrt{\zeta^2 - 1} + B$$

For point C, $z = 0$ and $\zeta = 1$; $\therefore B = 0$ For point D, $z = id$ and $\zeta = 0$

$$\therefore id = A\sqrt{-1} = Ai$$

Hence $A = d$

Thus, the transformation equation becomes :

$$z = d\sqrt{\zeta^2 - 1} \quad \dots(24)$$

From Fig. 4, $L_2 = 1$. For L_1 , consider co-ordinates of A and A'

$$\therefore -b_1 = d\sqrt{L_1^2 - 1} \quad (\text{From 24}).$$

$$\therefore L_1 = \sqrt{\left(\frac{b}{d}\right)^2 + 1} = \sqrt{1 + \alpha^2}$$

where

$$\alpha = \frac{b}{d}$$

Now for the horizontal floor of ζ -plane

$$\zeta + \frac{1}{2}(L_1 - L_2) = \frac{1}{2}(L_1 + L_2) \cosh w$$

$$\lambda = \frac{L_1 + L_2}{2} = \frac{L_1 + 1}{2} = \frac{\sqrt{1 + \alpha^2} + 1}{2} \quad \dots[26(a)]$$

and

$$\lambda_1 = \frac{L_1 - L_2}{2} = \frac{L_1 - 1}{2} = \frac{(L_1 + 1) - 2}{2} = \lambda - 1 = \frac{\sqrt{1 + \alpha^2} - 1}{2} \quad \dots[26(b)]$$

$$\zeta = \lambda \cosh w - \lambda_1$$

For the floor profile, $u = 0$. Hence $\cosh w = \cosh iv = \cos v$.

$$\zeta = \lambda \cos v - \lambda_1$$

Substituting this in Eq. (24),

$$z = d\sqrt{(\lambda \cos v - \lambda_1)^2 - 1} \quad \dots(27)$$

For the floor, $y = 0$, and hence $z = x$,

$$x = d\sqrt{(\lambda \cos v - \lambda_1)^2 - 1} \quad \dots[27(a)]$$

$$\lambda \cos v - \lambda_1 = \pm \sqrt{\frac{d^2 + x^2}{d^2}}$$

From which

$$v = \pi \frac{P}{H} = \cos^{-1} \frac{\lambda_1 d \pm \sqrt{d^2 + x^2}}{\lambda d}$$

(where negative sign is used for w/s floor)

$$P = \frac{H}{\pi} = \cos^{-1} \frac{\lambda_1 d \pm \sqrt{d^2 + x^2}}{\lambda d} \quad \dots(28)$$

For point E,

$$x = 0$$

$$P_E = \frac{H}{\pi} \cos^{-1} \frac{\lambda_1 d - d}{\lambda d} = \frac{H}{\pi} \cos^{-1} \frac{\lambda_1 - 1}{\lambda}$$

or

$$P_E = \frac{H}{\pi} \cos^{-1} \frac{\lambda - 2}{\lambda} \quad (\text{since } \lambda_1 = \lambda - 1) \quad \dots[29(a)]$$

For point C,

$$x = 0$$

$$P_C = \frac{H}{\pi} \cos^{-1} \frac{\lambda_1 d + d}{\lambda d} = \frac{H}{\pi} \cos^{-1} \frac{\lambda_1 + 1}{\lambda} = \frac{H}{\pi} \cos^{-1} \frac{\lambda}{\lambda} = 0 \quad \dots[29(b)]$$

For the sheet pile, $x = 0$. Hence from Eq. 27, we get

$$iy = d\sqrt{\lambda \cos v - \lambda_1}^2 - 1 \quad \dots[27(b)]$$

or

$$\lambda \cos v - \lambda_1 = \pm \sqrt{\frac{d^2 - y^2}{d^2}}$$

$$\therefore v = \pi \frac{P}{H} = \cos^{-1} \frac{\lambda_1 d \pm \sqrt{d^2 - y^2}}{\lambda d}$$

or

$$P = \frac{H}{\pi} \cos^{-1} \frac{\lambda_1 d \pm \sqrt{d^2 - y^2}}{\lambda d}$$

From point D,

$$y = d$$

$$P_D = \frac{H}{\pi} \cos^{-1} \frac{\lambda_1 d}{\lambda d} = \frac{H}{\pi} \cos^{-1} \frac{\lambda - 1}{\lambda} \quad \dots[29(c)]$$

The curves of Fig. 12.14 are based on Eqs. 29 (a) and (c).

Exit Gradient.

For finding exit gradient, consider Eq. 27(b)

$$iy = d\sqrt{\lambda \cos v - \lambda_1}^2 - 1$$

Let

$$\lambda \cos v - \lambda_1 = \cos \beta$$

 $\therefore$

$$iy = d\sqrt{\cos^2 \beta - 1} = id \sin \beta$$

 $\therefore$

$$y = d \sin \beta$$

Differentiating it with respect to y :

$$1 = d \cos \beta \frac{d\beta}{dy} \quad \dots[30(b)]$$

But from Eq. 30(a), $\lambda \sin v \frac{dv}{dy} = \sin \beta \frac{d\beta}{dy}$ Substituting the value of $\frac{d\beta}{dy}$ in Eq. 30(b).

$$1 = d \cos \beta \frac{\lambda \sin v}{\sin \beta} \cdot \frac{dv}{dy} \quad \dots[30(d)]$$

At the exit end, $v = 0$. Hence from Eq. 30(a)

$$\cos \beta = \lambda \cos v - \lambda_1 = \lambda \cos 0 - \lambda_1 = \lambda - \lambda_1 = 1$$

$$\beta = 0 \text{ when } v = 0$$

$$\frac{\sin v}{\sin \beta} = \frac{0}{0}$$

Let us determine the limit of $\left(\frac{\sin v}{\sin \beta}\right)_{v=0, \beta=0}$

$$\text{Lt} \left(\frac{\sin v}{\sin \beta}\right)_{v=0, \beta=0} = \text{Lt} \left(\frac{\frac{d}{dy} \sin v}{\frac{d}{dy} \sin \beta}\right)_{v=0, \beta=0} = \text{Lt} \left(\frac{\cos v \frac{dv}{dy}}{\cos \beta \frac{d\beta}{dy}}\right)_{v=0, \beta=0}$$

From Eq. [30(c)]

But $\frac{dv}{dy} / \frac{d\beta}{dy} = \frac{\sin \beta}{\lambda \sin v}$

$$\text{Lt} \left(\frac{\sin v}{\sin \beta}\right)_{v=0, \beta=0} = \text{Lt} \left(\frac{\sin \beta}{\lambda \sin v}\right)_{v=0, \beta=0}$$

Hence $\text{Lt} \left(\frac{\sin v}{\sin \beta}\right)_{v=0, \beta=0} = \frac{1}{\sqrt{\lambda}}$

Substituting the value in Eq. 30(d), we get, for $v = 0$ and $\beta = 0$

$$1 = \frac{d}{\sqrt{\lambda}} \left(\frac{dv}{dy}\right)_{\text{exit}} \quad \dots(32)$$

or

$$\left(\frac{dv}{dy}\right)_{\text{exit}} = \frac{1}{d \sqrt{\lambda}} \quad \dots(32)$$

Now

$$\left(\frac{dP}{dy}\right)_{\text{exit}} = G_E = \frac{H}{\pi} \left(\frac{dv}{dy}\right)_{\text{exit}} = \frac{H}{\pi} \cdot \frac{1}{d \sqrt{\lambda}}$$

$$G_E = \frac{H}{d} \cdot \frac{1}{\pi \sqrt{\lambda}} \quad \dots(33)$$

This is the required expression for exit gradient. In Fig. 12.13 (b), the exit gradient curve is based on the above equation.

Example 1. A horizontal impervious floor of length 20 m is provided with a cutoff of 4 m depth at its downstream end (Fig. 5). Determine analytically the uplift pressure at point E and D, and also the exit gradient, if the head causing seepage is 2 m.

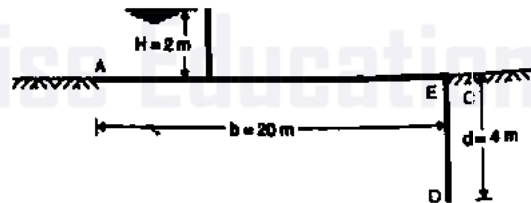


FIG. 5

Solution. (Fig. 5) :
Given : $b = 20$ m ; $d = 4$ m ;
 $H = 2$ m.

APPENDIX-I

The pressure at point E is given by

$$P_E = \frac{H}{\pi} \cos^{-1} \left(\frac{\lambda - 2}{\lambda} \right)$$

where

$$\lambda = \frac{1 + \sqrt{1 + \alpha^2}}{2}, \text{ and } \alpha = b/d = \frac{20}{4} = 5$$

$$\lambda = \frac{1 + \sqrt{1 + 25}}{2} = 3.05$$

$$P_E = \frac{2}{\pi} \cos^{-1} \left(\frac{3.05 - 2}{3.05} \right) = \frac{2}{\pi} (1.2195) = 0.78 \text{ m}$$

Also

$$P_D = \frac{H}{\pi} \cos^{-1} \left(\frac{\lambda - 1}{\lambda} \right) = \frac{2}{\pi} \cos^{-1} \left(\frac{3.05 - 1}{3.05} \right) = \frac{2}{\pi} (0.8337) = 0.53 \text{ m}$$

and

$$G_E = \frac{H}{d} \cdot \frac{1}{\pi \sqrt{\lambda}} = \frac{2}{4} \cdot \frac{1}{\pi} \cdot \frac{1}{\sqrt{3.05}} = \frac{1}{11}$$

Example 2. A horizontal impervious apron of length 10 m is provided with a cutoff of 4 m depth, placed eccentrically as shown. Determine analytically the uplift pressure at points E, D and C.

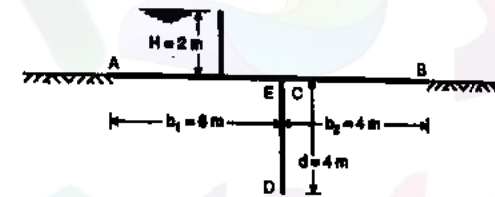


FIG. 6

Solution. Given

$$b_1 = 6 \text{ m} ; b_2 = 4 \text{ m} ; d = 4 \text{ m} ; b = 10 \text{ m}$$

$$\alpha_1 = \frac{b_1}{d} = \frac{6}{4} = 1.5 ; \alpha_2 = \frac{b_2}{d} = \frac{4}{4} = 1$$

$$\lambda = \frac{\sqrt{1 + \alpha_1^2} + \sqrt{1 + \alpha_2^2}}{2} = \frac{\sqrt{1 + (1.5)^2} + \sqrt{1 + (1)^2}}{2} = 1.61$$

$$\lambda_1 = \frac{\sqrt{1 + \alpha_1^2} - \sqrt{1 + \alpha_2^2}}{2} = \frac{\sqrt{1 + (1.5)^2} - \sqrt{1 + (1)^2}}{2} = 0.194$$

$$P_E = \frac{H}{\pi} \cos^{-1} \frac{\lambda_1 - 1}{\lambda} = \frac{2}{\pi} \cos^{-1} \left(\frac{0.194 - 1}{1.61} \right) = \frac{2}{\pi} (2.095) = 1.33 \text{ m}$$

$$P_C = \frac{H}{\pi} \cos^{-1} \frac{\lambda_1 + 1}{\lambda} = \frac{2}{\pi} \cos^{-1} \left(\frac{0.194 + 1}{1.61} \right) = \frac{2}{\pi} (0.735) = 0.47 \text{ m}$$

$$P_D = \frac{H}{\pi} \cos^{-1} \left(\frac{\lambda_1}{\lambda} \right) = \frac{2}{\pi} \cos^{-1} \left(\frac{0.194}{1.61} \right) = \frac{2}{\pi} (1.45) = 0.92 \text{ m}$$

Example 3. For the impervious floor shown in Fig. 7, determine the uplift pressure at points C_1 and E_2 .

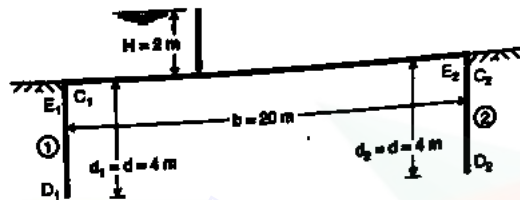


FIG. 7.

Solution : (a) *Pile No. 2.*

$$\alpha = \frac{b}{d} = \frac{20}{4} = 5.$$

Neglecting presence of upstream pile,

$$P_{m2} = \frac{H}{\pi} \cos^{-1} \left(\frac{\lambda - 2}{\lambda} \right)$$

where

$$\lambda = \frac{1 + \sqrt{1 + \alpha^2}}{2} = \frac{1 + \sqrt{1 + 25}}{2} = 3.05$$

$$P_{m2} = \frac{2}{\pi} \cos^{-1} \left(\frac{3.05 - 2}{3.05} \right) = \frac{2}{\pi} (1.2195) = 0.78 \text{ m.}$$

The correction due to interference of pile No. 1 is (Eq. 12.12)

$$C = 19 \sqrt{\frac{D}{b'}} \left(\frac{d + D}{b} \right)$$

where

$$D = d = 4 \text{ m}; b = b' = 20 \text{ m.}$$

$$C = 19 \sqrt{\frac{4}{20}} \left(\frac{4 + 4}{20} \right) = 3.4\% \text{ (subtractive)}$$

∴ Corrected $P_{m2} = 0.78 - 0.034 \times 2 = 0.71 \text{ m.}$

(b) *Pile No. 1.*

Both piles 1 and 2 are of same depth. By the principle of reversibility of flow,

$$P_{C1} = H - P_{m2} = 2 - 0.78 = 1.22 \text{ m.}$$

Also, correction due to interference of pile No. 2 is

$$C = 19 \sqrt{\frac{D}{b'}} \left(\frac{d + D}{b} \right) = 19 \sqrt{\frac{4}{40}} \left(\frac{4 + 4}{20} \right) = 3.4\% \\ = 0.034 \times 2 = 0.068 \text{ m (additive)}$$

Corrected $P_{C1} = 1.22 + 0.068 \approx 1.29 \text{ m.}$

Index

A			
Abyssinian tubewell	256	Cavity wells	268
Afflux	584	Centrifugal pumps	272
Air-lift pump	272	Channels—Alignment	645
Air vent	530	—, cross section	718
Apron	508	—, lining	787
Aqueducts	844	—, design	653, 710
Arch dams	349, 434	Chaturvedy, R.S.	354
Arrows	807	Cistern	804
Ashford Strainers	257	Commanded area	60
Asphaltic lining	760	Consumptive use	70
Assessment, irrigation water	86	Critical slip circle	476
B		Crops type	56
Baby Syphon	531	Crop rotation	85
Bandhara irrigation	650	Crop seasons	56
Balancing—depth	712	Cross-drainage works	843
Barrage	568	Cross-regulator	828
Basin flooding	26	Cutoff	572
Berms, channel	723	D	
Blench curves	613	Dams, arch	349, 434
Bligh's creep theory	573	—, buttress	350, 440
Borrow pits	728	—, earth	354, 445
Branch canal	644	—, gravity	348, 359
Breaching section	512	—, Rockfill	354, 501
Brick lining	760	Deep well	266
Bucket, Roller	543	Delta	58
—, Skijump	548	Dentated sill	808
Buttress dams	350, 440	Dicken's formula	196
C		Discharge measurement	734
Canal, branch	644	Distributary	643
—, distributary	644	Diversion headwork	566
—, inundation	643	Draft tube	802
—, main	644	Drainage system	458, 752
—, permanent	643	Duty of water	58
Canal falls	799	E	
Canal head regulator	828	Earth dams	354, 445
Canal losses	713, 760	Earthquake effect	362, 496
Capillary water	49	Ejectors	631
Casagrande, A.	465	Elementary profile	383
		Energy dissipation	543, 804
		Equipotential lines	462

864

IRRIGATION AND WATER POWER ENGINEERING

Shaft spillway	529	Terzaghi filter criteria	459
Shallow well	266	Theissen polygon	114
Shear friction factor	384	Timber dam	353
Shear stress in gravity dams	384	Training of river	875
Shotcrete concrete	769	Transitions	852
Shrouding	266	Trash rack	561
Side channel spillway	527	Trial load analysis	392
Silt excluder	629	Trough spillway	528
Silt extractor	630	Tubewell, artesian	242
Silt pressure	374	—, boring	262
Siphon spillway	630	—, cavity	257
Siphon, canal	843	—, shrouding	266
Siphon, volute	537	Turbines	897
Ski jump bucket	548		
Slab analogy method	392	U	
Slotted wells	269	Under sluices	560
Soil cement lining	760	Unit hydrograph	174
Specific retention	239	Uplift pressure, C.D. works	557
Specific yield	238	—, Dams	360
Spillway, chute	528	—, Weirs	573
—, emergency	511	V	
—, Gates	556	Vertical drop weir	599
—, Ogee	518	Vertical lift gates	558
—, Shaft	529	Volute syphon	537
—, Side channel	527		
—, Siphon	630	W	
—, Tunnel	528	Water course	644
Spoil banks	728	Water jet boring	260
Spurs	879	Water logging	746
Stability analysis	390, 473	Water power engineering	891
Stilling basin	549	Weirs, causes of failure	571
Strainer wells	256	—, parabolic	568
Strange's curves	156	—, sloping glacis	611
Stream lines	461, 577	Weirs, vertical drop	599
Sudden draw down	479	Well hydraulics	239
Super passage	844	Wings, canal	860
Surge tank	896	Wood's table	656
Syphon aqueduct	843, 860		
T		Y	
Tail race	904	Yield of wells	245, 270
Tainter gates	558		
Tej strainer	257		

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