SEDIMENT CONTROL
FROM A DIVERSION HEADWORKS SYSTEM
WITH SPECIAL REFERENCE TO TEESTA BARRAGE

A Thesis

Submitted to the Department of Water Resources Engineering of Bangladesh University of Engineering and Technology in Partial Fulfillment of the Requirements for the Degree of Master of Science in Engineering (Water Resources).

by

Dewan Md. Hasan Sayeed

May, 1988

CERTIFICATE

This is to certify that this work has been done by me and neither this thesis nor part thereof has been submitted elsewhere for the award of any degree or diploma.

(Professor Dr. M. Shahjahan)
Countersigned by Supervisor

(Dewan Md. Hasan Sayeed)
Signature of Candidate

BANGLADESH UNIVERSITY OF ENGINEERING AND TECHNOLOGY

DEPARTMENT OF WATER RESOURCES ENGINEERING

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We hereby recommended that the thesis presented by Mr. Dewan Md. Hasan Sayeed entitled "SEDIMENT CONTROL FROM A DIVERSION HEADWORKS SYSTEM WITH SPECIAL REFERENCE TO TEESTA BARRAGE" be accepted as fulfilling this part of the requirements for the degree of master of science (Water Resources).

Chairman of the Committee (Supervisor)	Dr. M. Shahjahan
Member	Dr. A. Nishat
Member	Dr. A. Hannan
Member (External)	Dr. Shajahan Kabir Chowdhury
Head of the Department	Dr. A. Nishat

ABSTRACT

The present study is mainly devoted to the evolvement of a methodology to design a silt excluder considering not only the hydraulic parameters but also the sediment discharge including bed and suspended load. The study also includes the estimation procedure of the sediment entrainment through the canal head regulator into the main canal.

In developing the design procedure of sediment excluder, a modification over Garde and Pande Method (1976) was carried out. The study also contains the best size, shape and location of different component structures of a barrage situated in alluvial plains. A literature survey of the existing barrages particularly in the Indo-Pak subcontinent together with the available scale models has been also reviewed.

Considering above, the design of the Teesta Barrage Silt Excluder has been evaluated and modifications suggested. It has been observed that the present sediment excluder containing discharge of 88.75% of canal discharge will have difficulty in efficient functioning due to nonavailability of hydraulic head for various flow conditions.

Teesta River carries 31 million tons of bed material load yearly for 75% dependable flow, of which 75% is from suspended load and 28% is from bedload. Teesta River carries sediment particle range 0.074 to o.15mm, 0.15 to o.30mm, 0.30 to 0.60mm and >0.60mm of 42, 42, 14, 2 and 17, 46, 33 and 4 percent

respectively from suspended load and bed material. A procedure to estimate bed and suspended sediment load to be entrained into the main canal through the Canal Head Regulator of the Teestsa Barrage has also been given. Quantitative values of bed and suspended sediment load with their various grain-size ranges are also estimated.

Suggested rectification measures are also advised to incorporate in the Teesta Barrage during the construction phase of the project. Particularly a corrective measure was suggested for no entrainment of bed load and a reduction of 52% of the suspended load entrainment per year for 10-day average 75% dependable flow of the Teesta River based on the analysis of 33 years data.

The possibility of the installation of a sediment ejector in the Teesta Main Canal was also studied and found to be suitably arranged provided some modifications in the crest level of CHR, FSL and canal bed of main canal are considered.

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TABLE OF CONTENTS

CONTENT	PAGE
ABSTRACT	i
ACKNOWLEDGEMENT	iii
LIST OF TABLES	vi
LIST OF FIGURES	vii
LIST OF SYMBOLS	ix
LIST OF ABBREVIATIONS	xii
Chapter	
I. INTRODUCTION	
1.1 Background and Definition of the Problem	1
1.2 Diversion Headworks and Sediment Control	2
1.3 Objectives of the Study	7
II. MEASURES TO CONTROL SEDIMENT	
2.1 Location of Offtake	9
2.2 Orientation of Offtake	12
2.3 Divide Wall	14
2.4 Width of Pocket	15
2.5 Location of Undersluices	18
2.6 Crest Level of Undersluice and Head Regulator	18
2.7 Shape of Guide Bunds	19
2.8 Barrage Regulation	20
2.9 Tunnel Type Sediment Excluder	23
III. REVIEW OF EXISTING SEDIMENT CONTROLLING MEASURES	
3.1 Location of Offtake	26
3.2 Orientation of Offtake	30
3.3 Divide Wall	34
3.4 Width of Pocket	34
3.5 Location of Undersluices	34
3.6 Crest Level of Undersluice,Weir and Head Regulator	35
3.7 Shape of Guide Bunds	38
3.8 Barrage Regulation	38
3.9 Tunnel Type Sediment Excluder	39
IV. SEDIMENT MOVEMENT IN ALLUVIAL CHANNELS	
4.1 Resistance to Flow in Alluvial Streams	41
4.2 Bed Forms	44
4.3 Mechanics of Sediment Transportation	48
4.4 Critical Review of Sediment Transport Equations	50
V. DEVELOPMENT OF CRITERIA FOR DESIGN OF SEDIMENT EXCLUDER	
5.1 Pond Level	63
5.2 Upstream Floor Level of Undersluice and Other Barrage Bays	64
5.3 Excluder Discharge	65
5.4 Tunnel Dimension	66
5.5 Staggering of Excluder Tunnel	68
5.6 Entrance of Tunnel	68
5.7 Excluder Velocity	70
5.8 Tunnel Blockage	74

5.9 Width of Excluder and Clear Waterway................ 75
5.10 Head Loss in Tunnel and Operating Head.............. 76
5.11 Barrage Regulation.................................. 79
5.12 Entrainment of Sediment Discharge into Tunnel........ 81
5.13 Sediment Concentration Carrying Capacity and Efficient Exclusion of Tunnel......................... 91
VI. DEVELOPMENT OF PREDICTION FUNCTION OF SEDIMENT INTO MAIN CANAL
6.1 Entrainment of Sediment Discharge into Main canal.... 94
6.2 Entrainment of Different Grain-Size Range into Main Canal.. 97
VII. EVALUATION OF SEDIMENT CONTROL IN TEESTA HEADWORKS
7.1 Selection of Project for Critical Review and Sources of Data.................................... 103
7.2 Teesta River: Sediment Transport and Stage-Discharge Relation.. 104
7.3 Evaluation of Sediment Controlling Measures Used in the Headworks.................................... 114
7.4 Entrainment of Sediment Load into the Main Canal..... 125
7.5 Possibility of Sediment Ejector in the Main Canal.... 150
VIII. CONCLUSSIONS AND RECOMMENDATIONS
8.1 Conclussions... 157
8.2 Recommendations...................................... 159
8.3 Suggestions for Future Study......................... 160

REFERENCES... 161

Appendix
1. Design Procedure of Sediment Excluder................. 170
2. Design of Sediment Excluder for Teesta Headworks...... 174
3. Analysis of Entrainment of Sediment Load into the Excluder Tunnel and into the Teesta Main Canal Under Design Condition.. 181
4. Analysis of Entrainment of Grain-Size Range Group into the Teesta Main Canal Under Design Condition.......... 184
5. Design of Sediment Ejector for Teesta Main Canal....... 187

LIST OF TABLES

TABLE		PAGE
3.1	Showing Orientation of off-takes With Respect to Axis Barrage/Weir at Various Headworks	31
3.2	Width of Undersluice Pocket and Length of Divide Wall in Barrage	33
3.3	Statement Showing Sill Levels of Weir, Undersluice and Head Regulator	36
3.4	Sediment Excluders	40
4.1	Summary Description of Bed Forms and Configurations	45
4.2	Bedload Equations	54
4.3	Suspended Load Equations	56
4.4	Values of Integrals J_1 and J_2	57
4.5	Additional Integral Values Calculated in Closed Form	58
4.6	Variation of K_s With τ_o/τ_{oc}	59
4.7	Variation of L_s With M	59
7.1	10 Day Average 75% Dependable Discharge(1952-85) by Log Normal Distribution	113
7.2	Different Conditions for Teesta Headworks	126
7.3	Tunnel Dimensions of Existing Excluder(Khanki Type)	128
7.4	Head Loss in the Excluder Tunnel Under Existing Condition	129
7.5	Parameters for the Analysis of Entrainment of Sediment into the Excluder Tunnel and into the Main Canal, Under Existing Condition	130
7.6	Entrainment of Sediment Load into the Excluder Tunnel and into the Main Canal, Under Existing Condition	131
7.7	Tunnel Dimensions of Designed Excluder(CWPC Type)	133
7.8	Head Loss in the Designed Excluder(CWPC Type)	134
7.9	Parameters for the Analysis of Entrainment of Sediment into the Excluder Tunnel and into the Main Canal, Under Design Condition	135
7.10	Entrainment of Sediment Load into the Excluder Tunnel and into the Main Canal, Under Design Condition	136
7.11	Head Loss in the Excluder Tunnel, Under Suggested Condition	138
7.12	Parameters for the Analysis of Entrainment of Sediment into the Excluder Tunnel and into the Main Canal, Under Suggested Condition	139
7.13	Entrainment of Sediment Load into the Excluder Tunnel and into the Main Canal, Under Suggested Condition	140
7.14	Entrainment of Sediment Discharge into the Main Canal for 10 Day Average 75% Dependable Discharge	141

LIST OF FIGURES

FIGURE		PAGE
1.1	Location of Headworks in Bangladesh	4
2.1	General Layout of a Diversion Headworks	10
2.2	Schematic Diagram of Flow in Curved Channel	10
2.3	Distribution of Water and Sediment	13
2.4	Distribution of Bedload as Related to θ	13
2.5	Flactuations in the Amount of Suspended Sediment Versus Amount of Water Diverted	13
2.6	Sediment Entry in Canal Versus Angle of Offtake as Observed in Model	13
2.7	Optimum Width of Pocket	17
2.8	Sediment Excluder for Lower Chenab Canal at Khanki	24
2.9	Sediment Excluder in the Left Pocket of Emerson Barrage at Trimmu	24
2.10	Sediment Excluder for Lower Sarda Canal	24
3.1	Sukkur Barrage on the Indus River	27
3.2	Layout of Kotri Diversion Dam as Constructed	27
3.3	Layout of Curved Wall at Narora Barrage	28
3.4	Gandak Barrage Showing Undersluices, River Sluices and Main Barrage	28
3.5	Location Map of Teesta Barrage	29
3.6	Variation of Length of Divide Wall With Width of Head Regulator	32
3.7	Concave Convex Guide Bund at Kosi Barrage	37
3.8	Elliptical Guide Bund at Lower Sarda Barrage	37
4.1	Flat-Bed Resistance	43
4.2	Bed Form Resistance	43
4.3	Bed Forms	46
4.4	Albertson-Simons-Richardson's Criteria for Resistance to Flow	46
4.5	Bogardi's Criteria for Regimes of Flow	47
4.6	Garde and Ranga Raju's Criteria	47
4.7	Garde-Albertson's Regime Criteria	47
4.8	Simons-Richardson's Regime Criteria	47
4.9	Sediment Coefficient and Critical Tractive Force for DuBoys Bedload Equation	60
4.10	Variation of $\bar{U}_g/U$ With τ_{oc}/τ_o	60
4.11	Factor x	60
4.12	Factor ξ	60
4.13	Factor Y	61
4.14	Integration Curves for Suspended Load Discharge	61
4.15	Variation of τ_s/τ_o With $2D/K_s$ and K	62
4.16	Variation of $K_s L_s \xi_s$ With $\tau_o/\Delta\gamma_s d_i$	62
5.1	Sediment Excluder at Narora Headworks	69
5.2	Projected Top Slab of Tunnels	69
5.3	Limit Deposit Velocity for Nonuniform Sediment	72

5.4 Ratio of Discharge Coefficients Due to Tail Water Effect.. 80
5.5 Integration Curves for Suspended Load Discharge........ 87
5.6 Percentage of Suspended Load in the Tunnel to the Suspended Load in the Pocket.............................. 90
7.1 Discharge VS. Suspended Load From Measured Values...... 105
7.2 Year-1985, Discharge VS. Bedload, Rottners Bedload Equation.. 106
7.3 Percentage Analysis of Suspended Material............. 108
7.4 Average Grain Size Distribution of Suspended Material.. 109
7.5 Average Grain Size Distribution of Bed Material........ 109
7.6 Stage-Discharge Curve................................. 111
7.7 Stage-Discharge Curve With Confidence Band............ 112
7.8 General Layout Plan of Headworks(Teesta Barrage Project)...................... 115
7.9 Profile of Undersluice and Weir(Teesta Barrage Project)........................... 118
7.10 Plan of Sediment Excluder(Teesta Barrage Project)...... 120
7.11 Sections of Sediment Excluder(Teesta Barrage Project).. 121
7.12 Plan of Designed Sediment Excluder.................... 123
7.13 Sections of Designed Sediment Excluder................ 124
7.14 Deposition of Suspended Load Discharge in the Pocket... 142
7.15 Deposition of Bedload Discharge in the Pocket.......... 143
7.16 Entrainment of Suspended Load Discharge into the Main Canal.. 144
7.17 Entrainment of Bedload Discharge into the Main Canal... 145
7.18 Entrainment of Maximum Grain Size of Suspended Load into the Main Canal................................. 146
7.19 Entrainment of Maximum Grain Size of Bedload into the Main Canal.. 147
7.20 Entrainment Percentage of Grain-Size Range of Suspended Load into the Main Canal.................. 148
7.21 Entrainment Percentage of Grain-Size Range of Bedload into the Main Canal......................... 149
7.22 Plan and Sections of Sediment Ejector With Main Canal and Escape Channel.................................. 153
7.23 Plan and Elevation of Sediment Ejector................ 154
7.24 Sections and Details of Sediment Ejector.............. 155
1 Sediment Removal Function for Settling Basin........... 196
2 Relation Between λ and β for Particle Settling.......... 196

LIST OF SYMBOLS

A	L^2	projected area of a particle
B	L	stream width
B_{EJ}	L	clear water way of ejector tunnel at exit
B_{EX}	L	clear water way of excluder tunnel at exit
B_R	L	width of the river at barrage
B_{RE}	L	width of the river contributing discharge to the pocket
C	$L^{1/2}T^{-1}$	Chezy's coefficient
C_a	–	suspended sediment concentration at a distance a above the bed
C_{2d}	–	suspended sediment concentration at a distance 2d above the bed
C_{EJ}	–	actual concentration in the ejector tunnel
C_{EX}	–	actual concentration in the excluder tunnel
C_L	–	lift coefficient of particle
C_{md}	–	suspended sediment concentration at mid-depth
C_T	–	sediment concentration carrying capacity of tunnel
d	L	sieve diameter of particle
d_a	L	arithmetic mean diameter of sediment
d_i	L	any size of sediment in a sample
D	L	depth of flow
D_{FEJ}	L	depth of flow in ejector tunnel based on free flow area available
D_{FEX}	L	depth of flow in excluder tunnel based on free flow area available
f	–	friction factor, factor
f'	–	friction factor due to grain resistance
f''	–	friction factor due to bed undulations
f_b	–	friction factor for base curves
f_m	–	mixture friction factor
g	LT^{-2}	acceleration due to gravity
g_b	$ML^{-1}T^{-1}$	bedload rate per unit width
g_s	$ML^{-1}T^{-1}$	suspended load rate per unit width
G_B	MT^{-1}	bedload
G_s	MT^{-1}	suspended load
h	L	available head
h_o	L	clear water head loss in tunnel
h_b	L	loss due to bend
h_c	L	loss due to contraction
h_{en}	L	loss due to entry
h_{ex}	L	loss due to exit
h_f	L	loss due to friction
i_b	–	fraction of bed sediment of a given grain size
i_s	–	fraction of suspended load of a given grain size
I_1, I_2	–	integrals
J	–	hydraulic gradient in a pipe carrying sediment-water mixture
J_o	–	hydraulic gradient in a pipe carrying clear water

J_1, J_2	–	integrals
K	–	Vonkarman's constants, constant
K_s	L	equivalent sand grain roughness of the boundary
L	L	length
n	$L^{1/6}$	Manning's roughness coefficient
P	–	prosity, percent
P_w	$ML^{-1}T^{-1}$	stream power
P_w'	$ML^{-1}T^{-1}$	excess stream power
P_{wc}	$ML^{-1}T^{-1}$	critical stream power
q	L^2T^{-1}	fluid discharge rate per unit width
q_B	L^2T^{-1}	bedload discharge rate per unit width
q_s	L^2T^{-1}	suspended load discharge rate per unit width
Q_B	L^3T^{-1}	bedload discharge
Q_{BC}	L^3T^{-1}	entrainment of bedload discharge into the canal
Q_{BP}	L^3T^{-1}	entrainment of bedload discharge into the pocket
Q_{BPB}	L^3T^{-1}	bedload discharge remains as bedload in the pocket
Q_{BPD}	L^3T^{-1}	bedload discharge deposited in the pocket
Q_{BPI}	L^3T^{-1}	initial entrainment of bedload discharge into the pocket
Q_{BT}	L^3T^{-1}	bedload discharge in the tunnel
Q_C	L^3T^{-1}	water discharge in the main canal
Q_{CI}	L^3T^{-1}	water discharge in the main canal of India
Q_{EJ}	L^3T^{-1}	water discharge in the sediment ejector
Q_{EX}	L^3T^{-1}	water discharge in the sediment excluder
Q_P	L^3T^{-1}	water discharge in the pocket
Q_R	L^3T^{-1}	water discharge at the upstream of barrage
Q_{RO}	L^3T^{-1}	original water discharge
Q_S	L^3T^{-1}	suspended load discharge
Q_{SC}	L^3T^{-1}	entrainment of suspended load discharge into the canal
Q_{SP}	L^3T^{-1}	entrainment of suspended load discharge into the pocket
Q_{SPD}	L^3T^{-1}	suspended load discharge deposited in the pocket
Q_{SPI}	L^3T^{-1}	initial entrainment of suspended load discharge into the pocket
Q_{SPS}	L^3T^{-1}	suspended load discharge remains as suspended load in the pocket
Q_{ST}	L^3T^{-1}	suspended load discharge in the tunnel
Q_T	L^3T^{-1}	sediment discharge in the tunnel
Q_U	L^3T^{-1}	water discharge in the undersluice portion of the barrage
Q_W	L^3T^{-1}	water discharge in the weir portion of the barrage
R	L	radius, hydraluic radius
R_b	L	hydraulic radius of the bed
R_b'	L	hydraulic radius of the bed corresponding to grain resistance
$R_c{*}$	–	critical Reynold's number
R_{EJ}	L	hydraulic radius of ejector tunnel
R_{EX}	L	hydraulic radius of excluder tunnel
R_{FEJ}	L	hydraulic radius of ejector for the free flow area available

R_{FEX}	L	hydraulic radius of excluder for the free flow area available
S	–	water surface or bed slope
S'	–	slope corresponding to grain resistance
S"	–	slope corresponding to bed undulations
S_s	–	specific gravity of sediment
t	L	depth of tunnel
T	–	transport function
U	LT^{-1}	velocity at a certain level
U	LT^{-1}	average velocity of flow
U_{cr}, U_c	LT^{-1}	critical velocity
U_{cp}	LT^{-1}	velocity distribution over the width
U_d	LT^{-1}	velocity at the particle level
U_{EJ}	LT^{-1}	ejector velocity
U_{EX}	LT^{-1}	excluder velocity
U_{FEJ}	LT^{-1}	ejector velocity based on the free flow area available
U_{FEX}	LT^{-1}	excluder velocity based on the free flow area available
U_L	LT^{-1}	limit deposit velocity
U_y	LT^{-1}	velocity component in the upward direction at a distance y from the bottom of flow
U_*	LT^{-1}	shear velocity
U_*'	LT^{-1}	shear velocity corresponding to grain roughness
U_{*c}	LT^{-1}	critical shear velocity
V	L^3	volume
V_b	LT^{-1}	velocity of sand wave
y	L	reference level from the bottom of flow
z	–	actual exponent in suspended sediment distribution equation
γ_f	ML^{-3}	specific weight of fluid
γ_s	ML^{-3}	specific weight of sediment
Δ	L	average amplitude of sand wave
$\Delta\gamma_s$	ML^{-3}	difference of specific weight of sediment and fluid
η	–	dimensionless y distance
ν	L^2T^{-1}	kinematic viscosity of fluid
ρ_f	$ML^{-4}T^2$	mass density of fluid
ρ_s	$ML^{-4}T^2$	mass density of sediment
τ_o	ML^{-2}	average shear stress
τ_{oc}	ML^{-2}	critical shear stress
τ_*	–	dimensionless shear stress
τ_*'	–	dimensionless shear stress for bed roughness
τ_{*c}	–	dimensionless critical shear stress
w_o	LT^{-1}	fall velocity

LIST OF ABBREVIATIONS

AGU	American Geophysical Union
ASCE	American Society of Civil Engineers
CBIP	Central Board of Irrigation and Power
CHR	Canal Head Regulator
CWPRS	Central Water and Power Research Station
FSL	Full Supply Level
IAHR	International Association of Hydraulic Research
ICID	International Commission on Irrigation & Drainage
JHR	Journal of Hydraulic Division
MPO	Master Plan Organisation
UPIRI	Uttar Pradesh Irrigation Research Institute
USBR	United States Department of the Interior Bureau of Reclamation
USDA	United States Department of Agriculture
US Geol. Survey	United States Geological Survey

CHAPTER I

INTRODUCTION

1.1. BACKGROUND AND DEFINITION OF THE PROBLEM

The main canal forming the primary part of a direct irrigation project, takes off from a diversion weir or a barrage. In fact, these permanent canals take off from rivers and the arrangements are so made at their heads, that a continuous water flow is ensured into the canal, even during the period of low flow. Dams are generally constructed across the river where the upstream location is suitable to act as reservoir. On the other hand, barrage is constructed to just head up water level at an upstream location mainly for irrigation purpose. Different parts of a barrage including sediment exclusion devices and canal head regulator is known as the diversion headworks.

Due to the construction of a barrage sediment transport of the river is disturbed. This distrubance may cause aggradation at the upstream of headworks and degradation at the downstream. In addition sediment may enter into the main canal. The success of an irrigation project depends to a large extent on the degree of control achieved on the deposition of sediment at the upstream of headworks and on the sediment entry into the offtaking canal.

Irrigation project built in the last century, when sediment flow was not sufficiently controlled, suffered from frequent silting with consequential need for closure of the canals for silt clearance. In the year 1954 Upper Bari Doab Canal got silted (CBIP,1966) up by 2.4m due to excessive sediment entry into the canal.

Advancement in the knowledge of sediment control is the recent product of science. Sediment control is divided into two categories- preventive method and curative method. In the former, coarser sediment is excluded at the head of canal before it enters while in the latter the finer material is removed after its entry into the canal.

The present work concerns mainly on the preventive measures as control of sediment from a diversion headworks system. Attempt will be made to review the past works on the control of sediment entry into the main canal through the canal head regulator. Attempt will also be made to review the design works of the sediment excluder for Teesta Barrage.

1.2 DIVERSION HEADWORKS AND SEDIMENT CONTROL

Use of surface water by constructing diversion headworks for irrigation, is the cheapest way though the initial cost is high. At present Bangladesh is using only 24,925 hectares of cultivable land for irrigation by using diversion headworks.

Each Year approximately 124 million hectare-meter of surface water (MPO,1986) is discharged through the rivers into the Bay of Bengal. Sediment transport through the two main rivers- Brahmaputra and Ganges are 739 million tons per year (MPO,1986). Hence construction of diversion headworks for diverting water through offtaking canal possess a great problem for sediment control.

In Bangladesh river water for many cases is directly pumped to the main canal and is allowed to flow through the secondary and tertiary canals by gravity for irrigation. Where the topography of the irrigated land is comparatively high, secondary lifting from canals through Low Lift Pumps is also allowed. Since the pump intakes are placed very close to the water surface of channel flow, normally sediment free water enters into the main canal and hence the siltation problem in the canals is not so sevre. But in some cases where river water is allowed to flow by gravity into the main canal by constructing a barrage across the river, larger quantity of sediment may enter into the main canal and thus great care is needed to control sediment. At present two barrages are under operation and the third one i.e., Teesta Barrage is under construction at Doani, Nilphamari. The existing two barrages are Buri Teesta and Manu Barrage. The former is located at Kaliganj, Nilphamari while the latter is at Moulvibazar. The locations of the barrages are shown in Figure 1.1.

<u>Buri Teesta</u>

Buri Teesta Barrage was constructed during the period 1958 -64 to irrigate net area of 13,360 hectares of Nilphamari district by diverting discharge of 7.08 m^3/s through two canals (BWDB,1986). Discharge through the river is inadequate during dry season but during monsoon the quantum of flow is quite high. The BWDB, has designed the project for supplementary irrigation

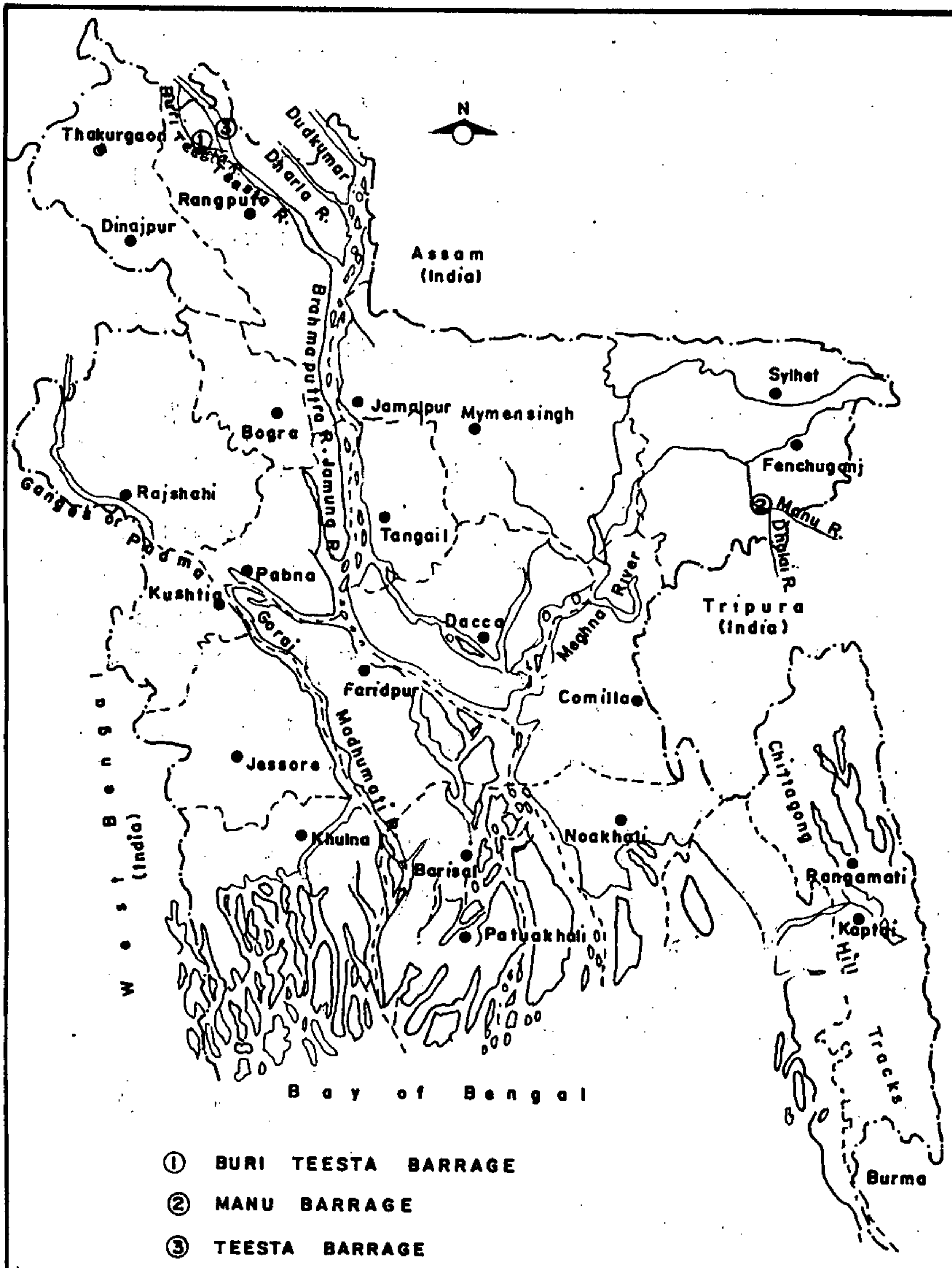


Fig. 1.1 LOCATION OF HEADWORKS

during second Kharif season (June to October). The provision of a reservoir of area 492.7 hectare has been kept to guard against sudden flood water. No silt exclusion device has been introduced in the Buri Teesta Barrage. It has been assumed that during dry season sediment will be deposited and in monsoon high flood will flush the sediment load, so there is no need of sediment exclusion device. But it has been observed that the reservoir capacity is reduced and the beds of the two canals get silted up due to the deposition of sediment load.

<u>Manu Barrage</u>

Manu Barrage was constructed during the period 1976-83 to irrigate net area of 11,565 hectares of Moulvibazar district by diverting discharge of 14.42 m^3/s through the canal. The barrage was designed with 6 weir bays and 2 undersluice bays. A model study of the barrage was carried out at the River Research Institute (BWDB,1976) and crest level for the undersluice was fixed 0.305m below the weir crest. Suspended sediment concentration in the river varied between 20 to 2000 ppm and the annual average sediment load was 156 hectare-meter. The project was mainly for lean period supplementary irrigation when sediment concentration in the river is very low. No sediment exclusion device was introduced in the diversion headworks due to lower sediment concentration of the river in the lean period. The barrage started functioning recently and no data about sediment deposition is available.

Teesta Barrage

Teesta Barrage Project is now under construction. The command area of the project is 20.85 million hectares of greater districts of Rangpur, Dinajpur and Bogra with a diverting capacity of 226.7 m^3/s through the main canal. Bangladesh Water Development Board has designed the barrage with 37 weir bays and 7 undersluice bays of 12.19m each where crest level of undersluice is lowered by 1.829m from the crest level of weir bay(details are shown in Figures 7.8 and 7.9). The project is mainly for supplementary irrigation from July to October when sediment concentration in the river is very high. Due to higher sediment concentration in the river during this period a sediment exclusion device has been introduced. The sediment excluder contains 12 tunnels of the following sizes at the exit, in the diversion headworks (details are shown in Figures 7.10 and 7.11 and in Table 7.3).

NO	Size
1	3.05m by 1.68m
3	2.13m by 1.68m
8	2.29m by 1.68m

The design of sediment excluder has been made only on the basis of hydraulic conditions without consideration of sediment aspects. The capacity of the excluder is 88.75% of canal discharge.

1.3 OBJECTIVES OF THE STUDY

The design of the sediment excluder has included only the hydraulic conditions. This does not seem to be sufficient for the larger sizes of the bed material to move through the tunnels and ultimately may make blockage of the tunnels. So there is a need to have a set of criteria to be fixed to design sediment excluder considering not only the hydraulic parameters but also the sediment factors for effective functioning.

Even after introduction of sediment excluder, larger quantity of sediment with coarse particles may enter into the offtaking canal. Entrainment of sediment load of larger volume (greater than the carrying capacity of canal) may gradually get silted up causing the canal to reduce its capacity. In addition, entrainment of sediment particles of larger sizes may settle a long distance upstream of the sediment ejector. So it is necessary to evolve some methodology by which entrainment of sediment load and entrainment of grain-size range into main canal can be determined.

Considering above factors the present work concerns following objectives.

1) Development of criteria for design of sediment excluder
2) Development of a prediction function for entrainment of sediment load into main canal
3) Development of a prediction function for entrainment of grain-size range into main canal

Furthermore, present study also involves the following works for Teesta Barrage Project.

1) Critical review of Teesta Headworks
2) Design of sediment excluder considering both hydraulic and sediment factors. This will be carried out using the data of Teesta River
3) Suggestions for the probable modification of Teesta Headworks
4) Comparison of entrainment of sediment load with grain-size range into the offtaking canal for existing condition, design condition and for suggested condition

CHAPTER II

MEASURES TO CONTROL SEDIMENT

The success of an irrigation project depends upon the minimum entrainment of sediment into the offtaking canal (Figure 2.1). To ensure this, different methods are used against sand entering a canal taking off from an aluvial river. The methods to control sediment for an irrigation system can be broadly classified as:

a) Preventive method

and b) Curative method.

In the former, coarser sediment is excluded at the head of the canal before it enters while in the latter the finer material is removed after its entry into the canal.

The present work concerns mainly on the preventive measures as control of sediment from a diversion headworks system (Figure 2.1). Control of sediment from a diversion headworks system can be done by applying the measures described below:

2.1 LOCATION OF OFFTAKE

Amongst the preventive measures, proper location of an offtake is the most important. The offtake may be either from a straight channel or from a curved channel. In the case of a straight channel, the sand charge decreases from midstream towards the bank. Thus for a straight channel, a canal taking off

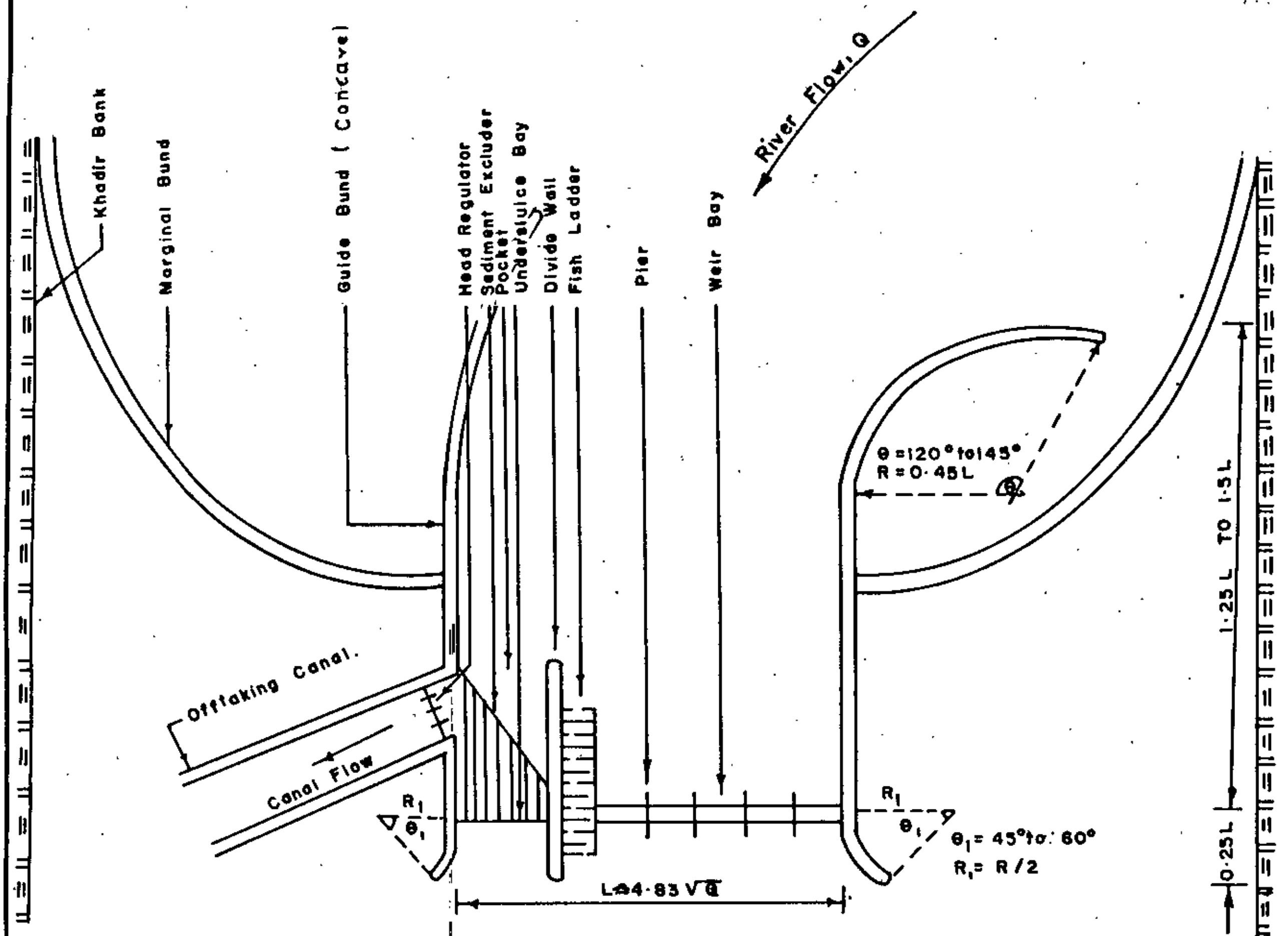


Fig: 2.1 GENERAL LAYOUT OF A DIVERSION HEADWORKS

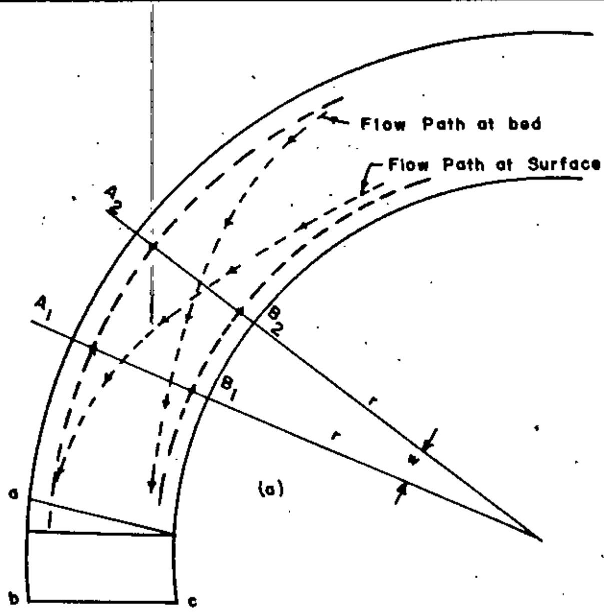


Fig: 2.2 SCHEMATIC DIAGRAM OF FLOW IN CURVED CHANNEL
(AFTER VANONI, ed, 1977)

over a raised sill will not draw a considerable proportion of sediment, provided the discharge drawn is small as compared to that of the channel (Joglekar,1971).

Though an offtake from a straight channel would work satisfactorily when discharge drawn is small, but it is very seldom to have in nature a sufficiently long and stable straight channel. In such cases, an offtake can be located on the concave bend of the channel. The curvature will produce a helicoidal flow (Vanoni,1977) which sweeps the bedload towards the convex bend (Figure 2.2) and thus reduces the sediment entry into the canal. The location on the concave bend can best be determined by a model study. Normally the diversion should be located two-thirds to three-fourths of the length of the curve from the beginning of the curvature. It is expected that secondary currents will be developed there and the upstream of the canal headworks will be fully effective (Vanoni,1977).

For single offtake, head regulator is constructed on the concave bend, where island will be formed due to deposition of sediment on the other side (Convex bend). Thus bedload will be again deflected towards the head regulator and will enter if other measure such as sediment exluder is not provided. For double offtake, concave curvatures are produced by upstream control on each side of the channel for positioning head regulators and sediment excluders are also needed as island will be formed in the middle.

2.2 ORIENTATION OF OFFTAKE

The orientation of an offtake with respect to the axis of barrage influences entrainment of sediment into the offtaking canal, diversion discharge and sediment exclusion.

Bulle (1926, after Vanoni,1977), conducted a model study and the results are shown in Figure 2.3, where various divisions of flow between the stream and branch have plotted against the amount of bedload entering each for a constant angle of diversion of 30°. Figure 2.4 shows the distribution of bedload with change in the diversion angle for a constant 50% diversion of water discharge (Garde and Ranga Raju,1985). Both the attempts were made by them to have some parameters by means of which the optimum angle of diversion could be made. However, it has been observed that there is no such thing as an optimum angle, because this angle would vary with the change of diversion ratio. Where diversion ratio is the percentage of total discharge passes through the branch canal. It can be seen from Figures 2.3 and 2.4 that at 30° diversion angle and for 50% diversion discharge, (Q_c) bedload transport, (Q_{BC}) in the canal is 97%. It can also be concluded from the above mentioned figures that for every diversion angle a large amount of bedload moves towards the head regulator, which needs exclusion. Otherwise, large volume of bedload will enter into the offtaking canal and will ultimately decrease the capacity of the canal.

For suspended load entrainment into the offtaking canal, Ethem and Ozden (1975), recommended frontal intake to be better over lateral intake for minimum sediment entry (Figure 2.5).

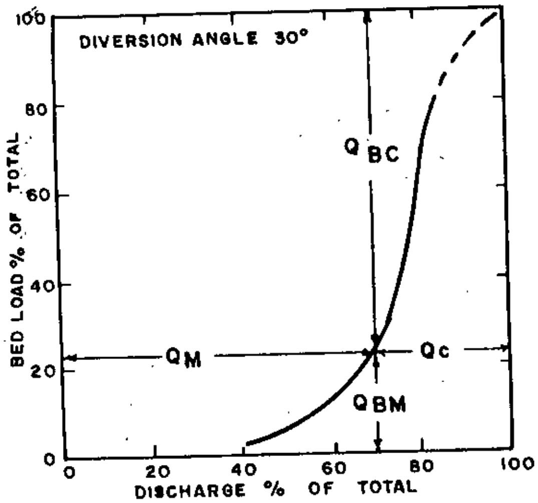


Fig:2.3 DISTRIBUTION OF WATER & SEDIMENT

(AFTER GARDE & RANGA RAJU,1985)

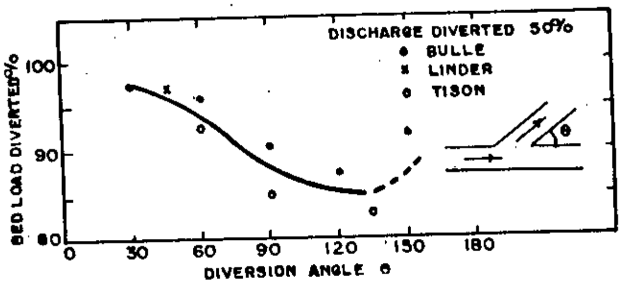


Fig:2.4 DIVERSION OF BED LOAD AS RELATED TO θ

(AFTER GARDE & RANGA RAJU,1985)

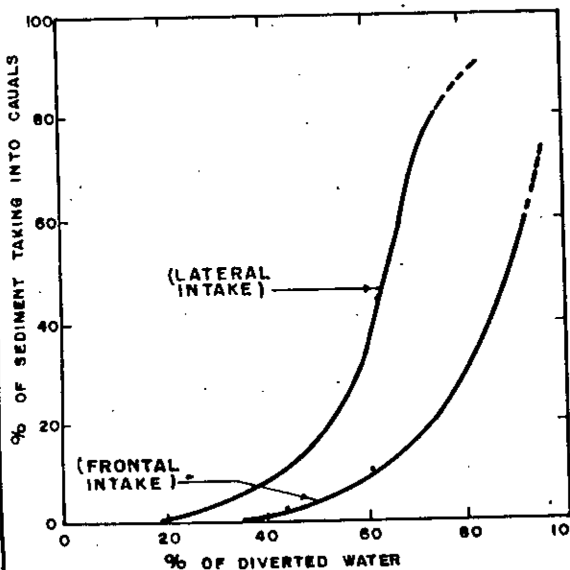


Fig:2.5 FLUCTUATIONS IN THE AMOUNT OF SUSPENDED SEDIMENT TAKEN VERSUS AMOUNT OF WATER DIVERTED

(AFTER ETHEM-OZDEN, 1975)

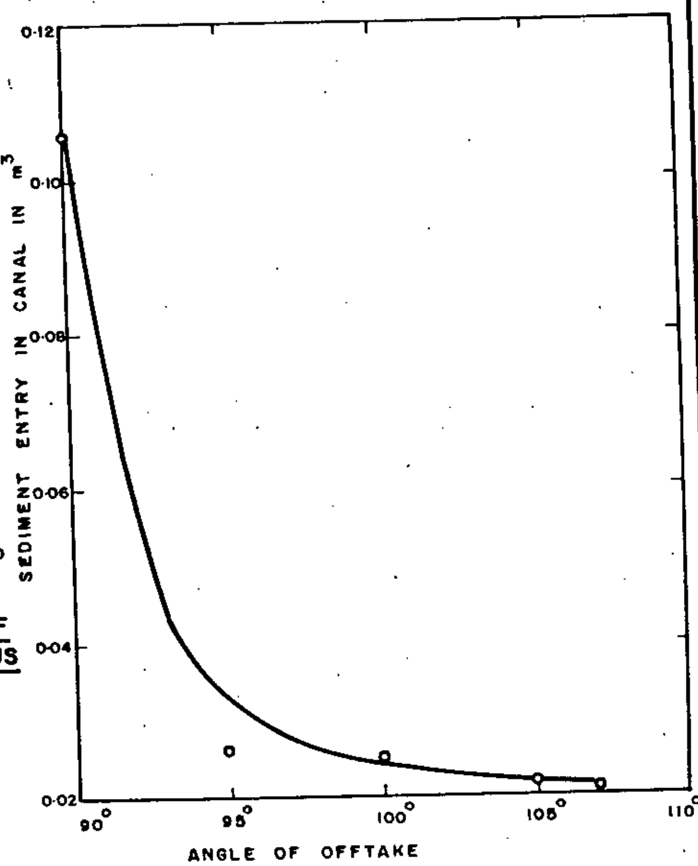


Fig:2.6 SEDIMENT ENTRY IN CANAL VERSUS ANGLE OF OFFTAKE AS OBSERVED IN MODEL

(AFTER DHILLON, 1980)

Where lateral intake is the diversion of water making the diversion angle of 30°-60° and in frontal intake, a parallel and uniform flow is established infront of the intake. The above recommendation was made by them after model study. From Figure 2.5, it can be seen that for 50% of discharge to be diverted, lateral intake takes 18% of suspended load whereas, for the same diversion discharge frontal intake takes only 5%.

For total sediment load, UPIRI (1973, after Dhillon,1980) recommended the optimum offtaking angle to be in between 105° to 110° (Figure 2.6). The above recommendation was made by UPIRI, after conducting a model study for lower Sarda Barrage, taking sediment exclusion into consideration.

2.3 DIVIDE WALL

It is a long wall constructed at right angle to the main barrage axis, extending upstream so as to form a pocket infront of the canal intake. Divide wall isolates canal head regulators from the main flow and has been useful in effecting sand exclusion. The improvement in exclusion due to a divide wall owes to the difference in discharge intensities per meter run in the pocket and in the weir during high floods. As lower velocity is maintained in the pocket, coarser bed material gets trapped.

A very long divide wall can cause the coarser bed material to settle upstream of excluder tunnel, which may require long time (after canal closure) for exclusion (Joglekar,1971). Uppal and Sharma from their study on "Functioning of Divide Wall " (CBIP,1966), and Uppal and Gulati from a model study on "Harike

Barrage" (CBIP,1966), suggested the divide wall length to be little beyond the canal head regulator for best results. But divide wall convering two-third width of head regulator gives generally good result as concluded by Joglekar (1971), Vanoni (1977), and Sharma, Sharma and Jain (1977, after Dhillon 1980) for single head regulator. In situation where more than one canal takes off from the same bank it is essential to extend the divide wall to the point opposite the upstream abutment of the last regulator (Joglekar,1971).

The top of the divide wall will be kept above the pond level or high flood level whichever is higher so as to avoid spilling over it and formation of hydraulic jump (Sharma, Sharma and Jain, 1977, after Dhillon,1980).

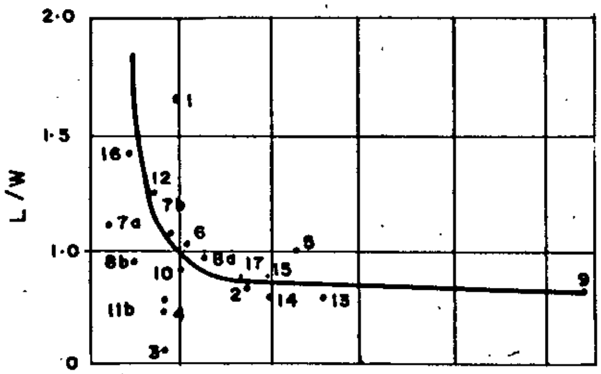
Design of divide wall nose also influences sediment exclusion to some extent. Steeper slope of the nose creates greater depth and extent of scour hole arround it. Exclusion is better effected with a scour hole round the nose of the divide wall, due to the twist of the bed flow caused at the nose. On the other hand, flat slopes tend to reduce scour depth, and better design safety; but the beneficial effect on sediment exclusion is lost to some extent. Advantages of this fact can be taken by adopting a flat nose slope on the pocket side and a steep slope on the river side (CWPRS,1946, after Joglekar,1971).

2.4 WIDTH OF POCKET

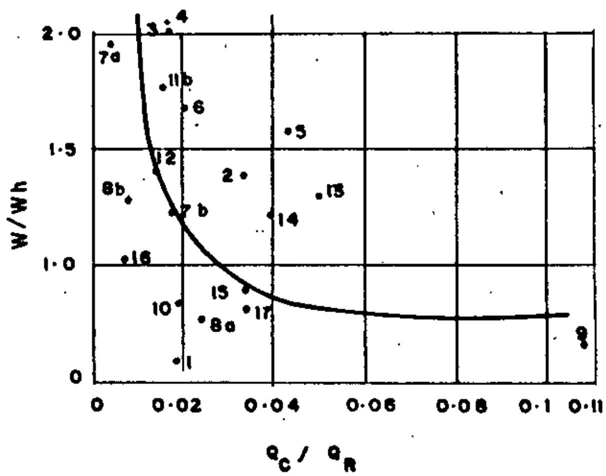
The pocket created between the divide wall and the head regulator also influences sediment control on the offtaking canal. Earlier it was indicated that (Sharma,1959, after Dhillon, 1980) the width of the pocket depended on the capacity of the undersluices and it was to be at least double the canal discharge to accommodate about 10% to 15% of maximum flood discharge. Experience from both models and prototypes have shown that a pocket with smaller width generates higher velocity causing higher sediment entry into the offtaking canal while an unduly wide pocket develops parallel flow along the divide wall and the undersluices. After model study at the Central Water and Power Research Station, Poona, India (Joglekar,1971), it was suggested that the width of the pocket to be such that for dominant discharge the ratio $U_R/U_P > 1$. Where U_R is the velocity of flow on the river side and U_P is velocity of flow on the pocket side. But this criterion has not found to hold good in the case of Sukkur Barrage (Joglekar,1971). For efficient functioning of a barrage it was then concluded that the width of the pocket should be optimum and this should be investigated in conjunction with the length of the divide wall as the two were inter dependent.

Sharma, Sharma and Jain (1977, after Dhillon,1980), analysed data of 17 barrages and developed two curves as shown in Figure 2.7, from which the width of the pocket can be approximated. At present these curves can be used till other better methods are developed.

A pocket slightly converging towards the downstream or towards the undersluices has seen to be preferable (Joglekar, 1971) than a straight pocket, since convergence is helpful in



a. OPTIMUM RATIO OF LENGTH OF DIVIDE WALL & WIDTH OF POCKET.



b. RATIO OF APPROXIMATE WIDTH OF POCKET & WIDTH OF HEAD REGULATOR

1. Sarda Barrage
2. Harike Barrage
3. Narora Barrage (L.G.L.)
4. Dakpather Barrage
5. Ashan Barrage
6. Gandak Barrage
7. Kosi Barrage
   (a) Right Bank Canal
   (b) Left Bank Canal
8. Girija Barrage
   (a) Right Bank Canal
   (b) Left Bank Canal.
9. Lower Sarda Barrage
10. Ramganga Barrage
11. Durgapur Barrage
    (a) Right Bank Canal
    (b) Left Bank Canal
12. Kosi Barrage (U.P.)
13. Rishikesh Barrage
14. Hindon Barrage
15. Narora Barrage
16. Giri Barrage
17. Shah Nehar Barrage.

Fig:2.7 OPTIMUM WIDTH OF POCKET

(AFTER DHILLON, 1980)

scouring operations. But the splay should not exceed 1 in 10 as this reduces the barrage spans adjacent to the divide wall.

2.5 LOCATION OF UNDERSLUICES

From the consideration of sediment control the most suitable location of undersluices is adjacent to the canal head regulator. When canal offtakes from both the banks then two undersluices pockets, one near each head regulator are necessary. A new sediment control measure (Joglekar,1971) evolved by CWPRS, Poona, India, with the aid of hydraulic model studies comprises provision of a second pocket or river sluices adjacent to the pocket sluices. This measure has seen to be equally applicable to the situations where river curvature is whether unfavourable (head regulator is on convex bend of river) or favourable (head regulator is on concave bend of river).

2.6 CREST LEVEL OF UNDERSLUICE AND HEAD REGULATOR

Crest levels have influence on sediment control in canal headworks. For proper flushing of the sediment deposited in the pocket and to have well defined channel to the undersluices, the undersluice crest is kept lower than the weir crest and the crest of head regulator.

The crest level of a barrage is fixed from the consideration of existing river bedlevel at the proposed site. Average deeper channel level is generally considered as the upstream floor level (see subsection 5.2) of weir bays. In order to have a deeper channel in the undersluice, the crest level should be 1m below the upstream floor level of weir portion. The crest level of weir bay is kept higher than the upstream floor level of weir bay, and is generally 1m to 1.5m above. For selecting crest level of head regulator, flow condition in the barrage is important. For alluvial channel, if the flow in the barrage is under free fall condition, higher percentage of sediment may enter into the excluder tunnel i.e., suspended sediment load to enter into the canal remains low. In such a condition crest level of head regulator should be at top level of sediment excluder (Garde and Pande,1976). On the other hand, if the river is alluvial, submerge flow condition occurs in the barrage and diversion discharge is small in comparison to the total discharge of the river; a small percentage of sediment may enter into the excluder tunnel i.e., suspended sediment load to enter into the canal remains high. In such a condition a raised crest of head regulator of about 1m to 4m (Sharma and Asthana,1975) above the bed level of the channel may be used to reduce sediment entry into the offtaking canal. The raised crest of head regulator is also helpful for higher floor level of the offtaking canal for efficient ejection of sediment through sediment ejector (curative measure).

2.7 SHAPE OF GUIDE BUNDS

The shape of the guide bunds helps secure artificially suitable approach of the flow to the pocket and thus exercises control on the entry of sediment into the offtaking canal. Proper alignment of the guide bunds are converging, bottleneck, parallel, diverging, concave, concave-convex etc, depending to a large extent on the river approach condition prevailing upstream of the headworks. Sharma and Asthana (1975), and Dhillon (1980) indicated that converging or bottleneck type guide bund could make a large island at the upstream of pocket, while the parallel and diverging types though being economical to maintain also ensured a smooth entrance by avoiding churning up of flow at the head regulator. From model study and field experience it has been seen that the diverging guide bunds are suitable for wide and shallow alluvial rivers. Concave guide bunds tend to reduce sediment entry into the offtaking canal by forming helicoidal flow when the river approach condition is suitable (head regulator is on the concave bend of river), concave-convex type guide bunds reduces sediment entry into the offtaking canal when the river approach condition is not suitable (head regulator is on the convex bend of the river). Recent model studies for Girija and Lower Sarda Barrage in India, (UPIRI,1973, after Dhillon, 1980) have brought out the superiority of guide bunds with gradually changing curvature in the form of an ellipse at all river discharges.

2.8 BARRAGE REGULATION

The regulation of river supplies at a barrage requires operation of the gates of the undersluices and weir bays and the head regulator in a systemetic manner depending on the river stage so as to keep the sediment entry into the offtaking canal minimum. Some barrage regulation methods are described below:

a) Still Pond Regulation

In still pond regulation all the gates of the undersluices are closed and still pond is produced in the pocket. Thus in still pond regulation pocket discharge, Q_P is always equal to the canal discharge, Q_C.

Still pond regulation has inherent advantage (Dhillon,1980) due to lesser percentage of sediment entry into the offtaking canal. But in this system a great disadvantage may arise due to the deposition of a huge percentage of sediment in the pocket. For flushing the deposited sediment, interuption of supplies of the offtaking canal may require. Thus still pond regulation may not give better result for sediment control.

b) Semi-Still Pond Regulation

Semi-still pond regulation receives some excess discharge in the pocket ($Q_P > Q_C$) and is escaped either through sediment excluder tunnels or through undersluice bays by partial opening of the gates. In this system the remaining discharge is allowed to pass through the barrage by opening the gates of the weir portion away from the head regulator more than those near to it. However, if the canal offtakes from both the sides of the barrage, opening of the gates should be maximum in the centre, decreasing gradually towards the divide wall.

By continuous flushing through sediment excluder, deposition of sediment in the pocket may be reduced in this system. This type of regulation can give good result for high discharges (Joglekar,1971). This is due to the fact that at high discharge ($U_R/U_P > 1$) maximum sediment moves away from the head regulator.

Thus opening of the gates away from the head regulator is helpful.

c) Wedge from Right or Left Regulation

Like that of semi-still pond regulation it also receives some excess discharge in the pocket ($Q_R > Q_C$) and is escaped either through sediment excluder tunnel or through undersluice bays by partial opening of the gates. In this system, the remaining discharge is allowed to pass through the barrage by opening the gates of the weir portion adjacent to the divide wall more than those away from it. However, if the canal offtakes from both the sides of the barrage, opening of the gates should be minimum in the centre increasing gradually towards the divide wall, and is called "Double-Wedge Regulation".

By continuous flushing through the sediment excluder, it also reduces sediment deposition in the pocket. This type of regulation can give good result for low discharges (Joglekar, 1971). At low discharges ($U_R/U_P < 1$) maximum sediment moves towards the head regulator. Thus opening of gates closer to the divide wall is helpful.

d) Regulation During High Flood

During high floods sediment concentration in the river may increase so much that the sediment exclusion measures available may not be enough to cope with the problem. So during high floods if the offtaking canals are opened, water with higher sediment concentration may enter the canal causing it to be choked up. Hence for high floods the canals are generally closed.

2.9 TUNNEL TYPE SEDIMENT EXCLUDER

Inspite of all the methods described earlier, a large quantity of coarse material may find its way into the pocket. Elsden (1922, after Dhillon,1980), proposed a diaphragm at suitable height without disturbing the sediment distribution. This arrangement is known as the tunnel type sediment excluder. The tunnels are placed along the canal head regulator. There are three types of sediment excluders, viz.,

a) Khanki type (Figure 2.8)

b) Trimmu type (Figure 2.9)

and c) CWPC type (Figure 2.10)

In Khanki type sediment excluder, the tunnels are of different lengths covering the whole length of head regulator. In Trimmu type, all the tunnels starts from the same line and at the same distance from the axis of the barrage. In CWPC type sediment excluder, various length of tunnels are maintained approximately at a slope of 1:1, i.e., the tunnel openings covers a certain length of canal head regulator.

The choice of the type of an excluder depends on many factors, such as:

a) river approach condition

b) barrage regulation

c) sediment characteristics

and d) river stages.

The river approach condition (Sharma and Asthana,1975) is the most important factor which generally keeps on changing. The Khanki type sediment excluder tunnels are more effective for

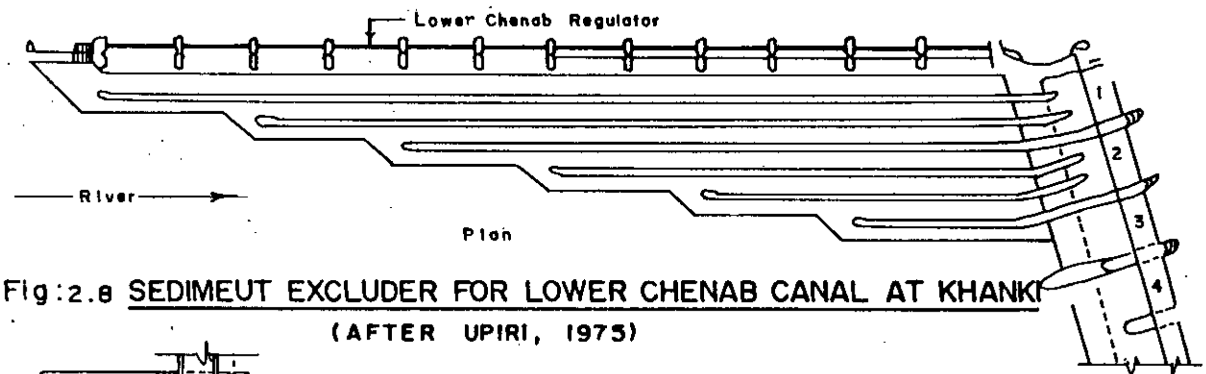


Fig:2.8 SEDIMEUT EXCLUDER FOR LOWER CHENAB CANAL AT KHANKI
(AFTER UPIRI, 1975)

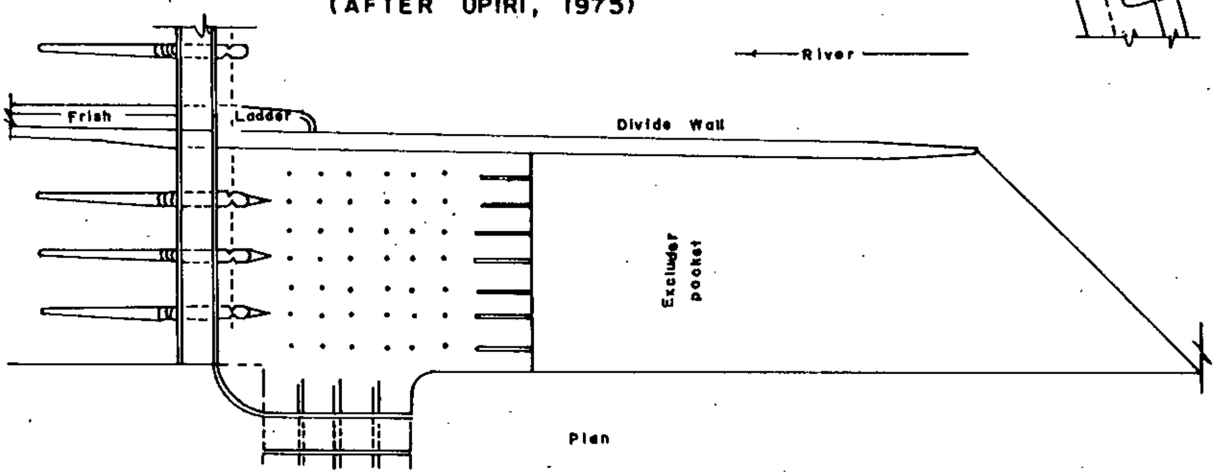


Fig: 2.9 SEDIMEUT EXCLUDER IN THE LEFT POCKET OF EMERSON BARRAGE AT TRIMMU (AFTER UPIRI, 1975)

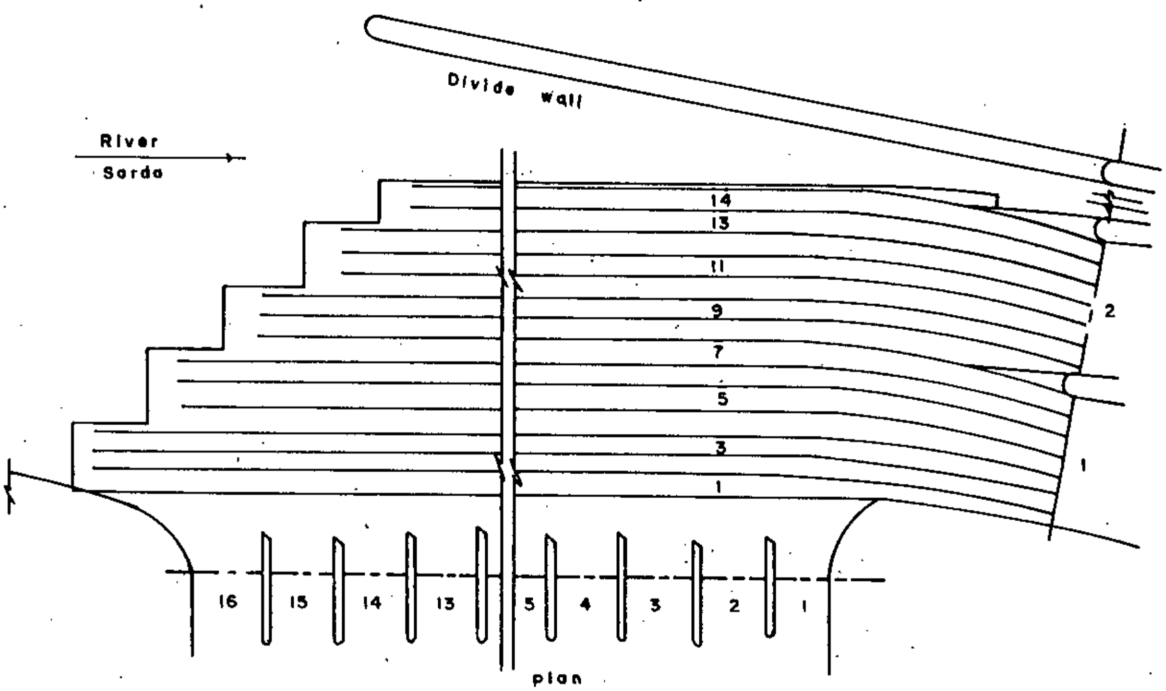


Fig:2.10 SEDIMENT EXCLUDER FOR LOWER SARDA CANAL
(AFTER UPIRI, 1975)

oblique river flow, but the efficiency is greatly affected if any subsequent changes occur due to swinging of the river caused by developing meander (UPIRI,1975). It is also mentioned in UPIRI (1975), that the Trimmu type sediment excluders are very sensitive to changes in river approach condition and become in-effective for oblique flow (Sharma and Asthana,1975). The CWPC type sediment excluder is effective with straight river approach condition (Sharma and Asthana,1975). However, UPIRI (1975) has seen that in CWPC type sediment excluder turbulence at the entrance of the tunnel is confined to a narrow region and works satisfactorily under oblique flow condition also. Furthermore, the extent of oblique flow can be controlled by providing a suitable length of divide wall and adopting a proper regulation. Hence, CWPC type may be concluded as the better form of sediment excluder and are being used extensively in the barrages constructed in India.

CHAPTER III

REVIEW OF EXISTING SEDIMENT CONTROLLING MEASURES

Adoption of the preventing measures to control sediment entry into the main canal can only be justified by reviewing the excluders so far constructed as part of barrages. Maximum number of barrages in the world have been constructed in the Indo-Pak subcontinent. Since the excluder is close to the canal headworks, the location, orientation and other pertinent parameters are described below. These factors are very important pertaining to the efficient functioning of the silt excluder.

3.1 LOCATION OF OFFTAKE

The downstream end of a concave curve is the suitable location for the head regulator for controlling sand entry into canal. After extensive studies on the networks of canal system developed in Punjab, Sind, Pakistan and Uttar Pradesh, India the above recommendation was made by Joglekar (1971). A few examples of the location of head regulator which have constructed in different parts of the world are given below.

In Sukkur Barrage (Figure 3.1), Sind, Pakistan (CWPRS,1941-42,1943 after Joglekar,1971) the canal of the left bank takes off from concave curves while those on the right bank takes off from convex sides. It has been observed that the left bank canal is working satisfactorily while the right bank canal has been

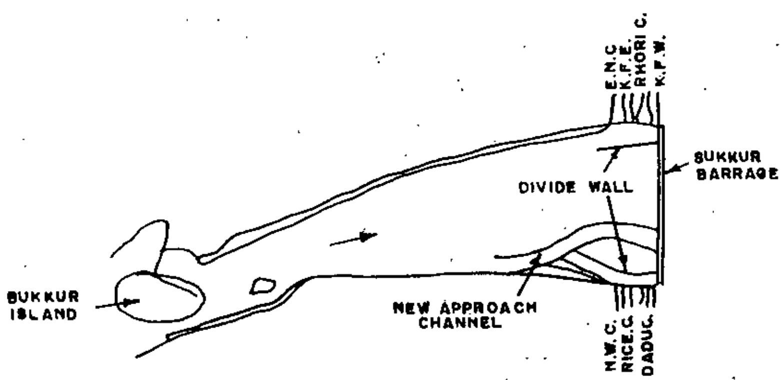


Fig: 3.1 SUKKUR BARRAGE ON THE INDUS RIVER
(AFTER GARDE & RANGA RAJU, 1985)

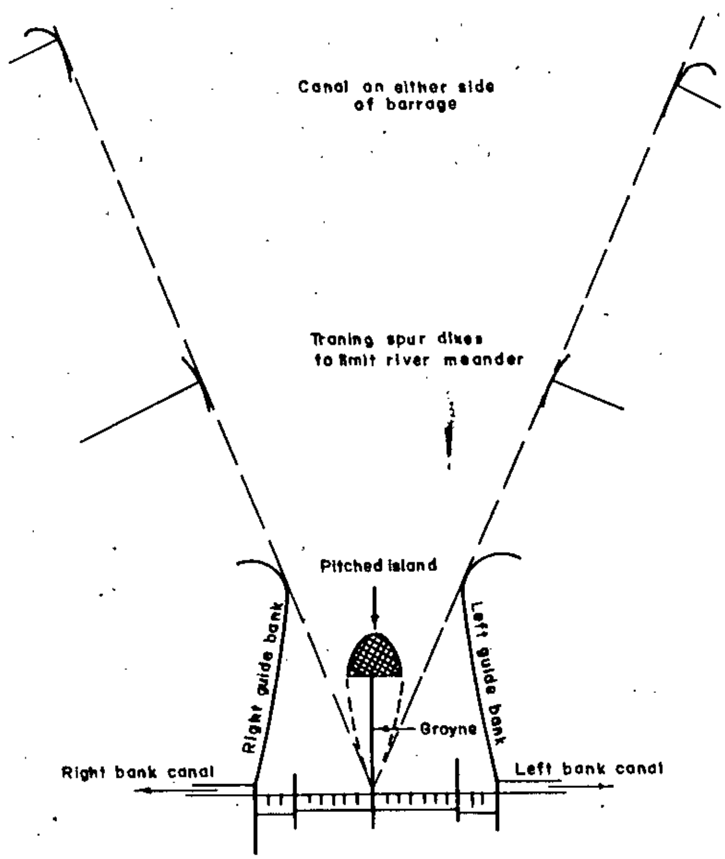


Fig: 3.2 LAYOUT OF KOTRI DIVERSION DAM (PAKISTAN) AS CONSTRUCTED
(AFTER VANONI, ed, 1977)

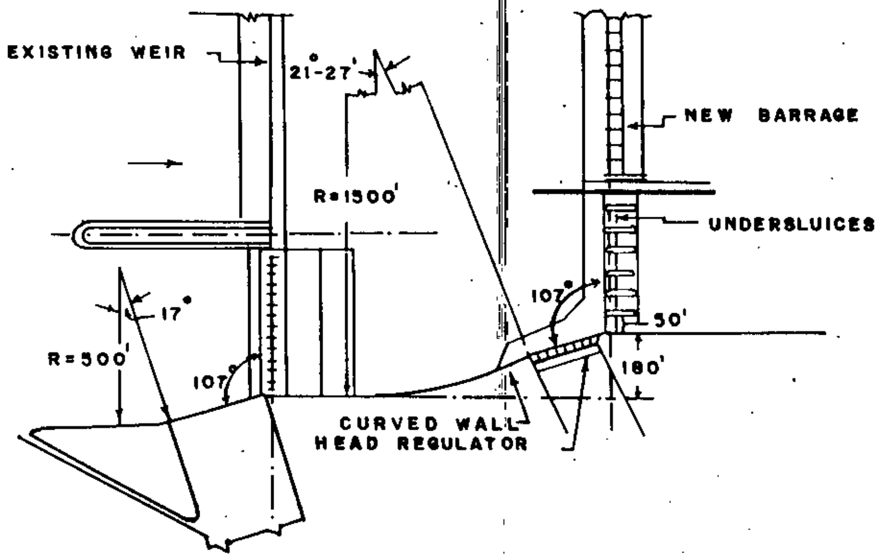


Fig: 3.3 LAYOUT OF CURVED WALL AT NARORA BARRAGE (U.P)
(AFTER SHARMA - ASTHANA, 1975)

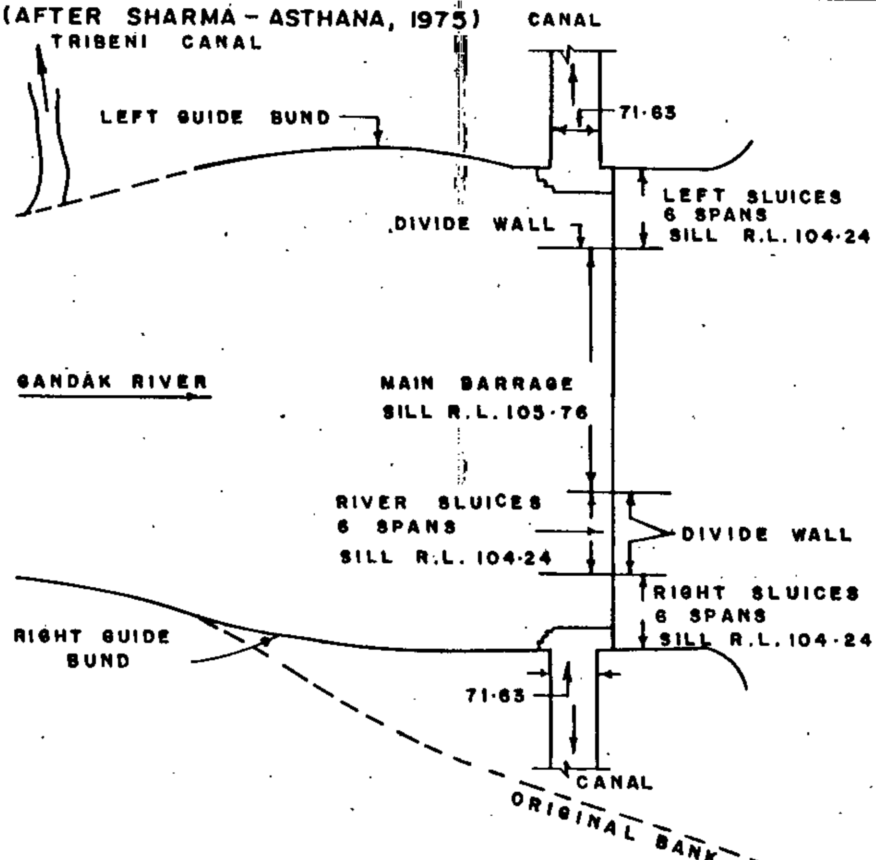


Fig: 3.4 GANDAK BARRAGE SHOWING UNDERSLUICES, RIVER SLUICES AND MAIN BARRAGE.
(AFTER SHARMA - ASTHANA, 1975)

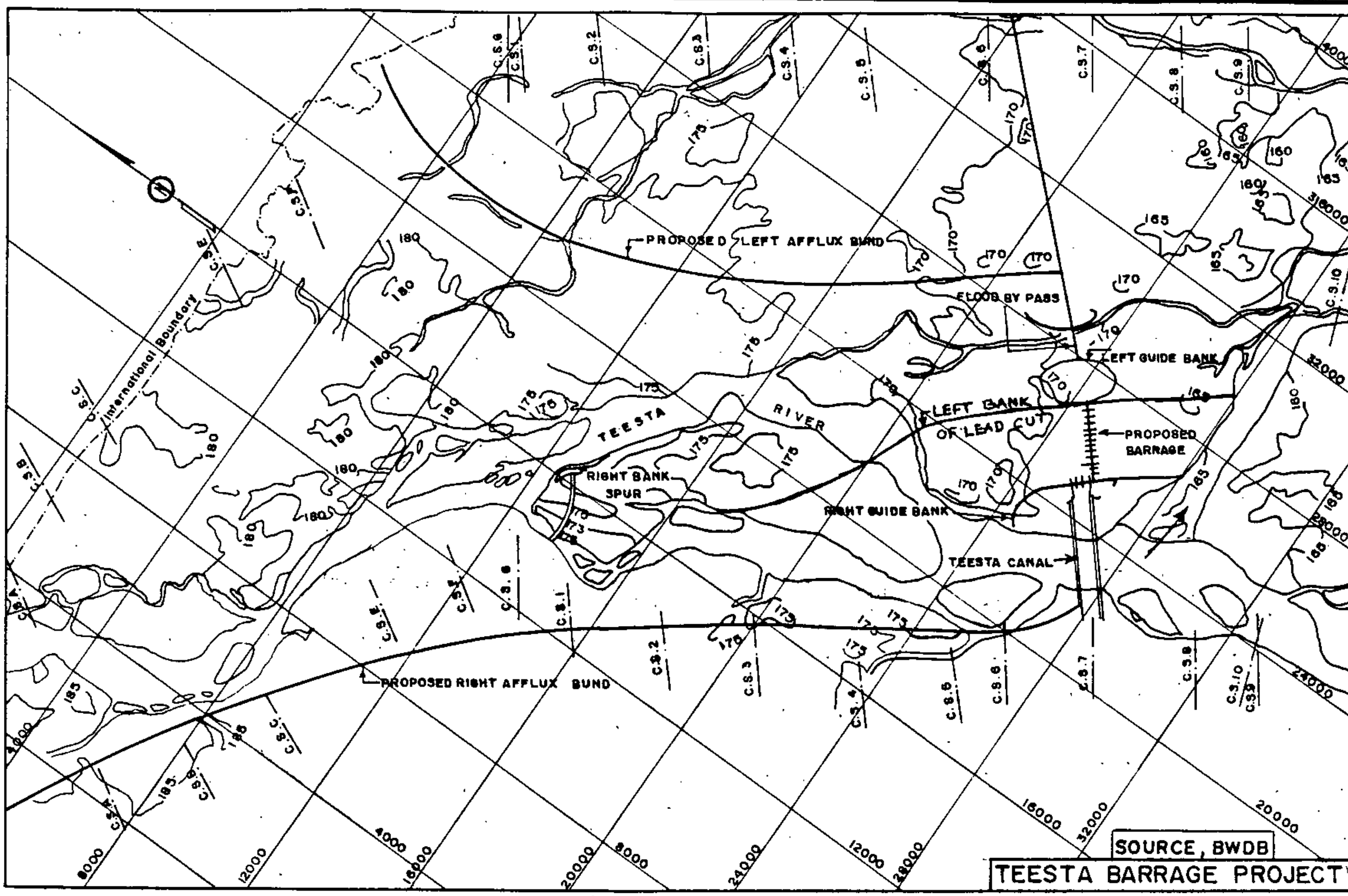


Fig: 3.5 LOCATION MAP OF TEESTA BARRAGE

silted up as much as 1.5m. A concave curvature has been introduced on the right bank after model study made by Central Water and Power Research Station, Poona (CWPRS,1941-42,1943, after Joglekar,1971). The canals are at present working satisfactorily as reported by Jogelkar (1971).

Head regulators constructed on artificially created or natural concave bends such as Kotri Diversion Dam (Figure 3.2) in Pakistan (Vanoni,1977), Narora Barrage (Figure 3.3) in Uttar Pradesh, India (Joglekar,1971) and Gandak Barrage (Figure 3.4) in India (Sharma and Asthana,1975) have found to function satisfactorily. The Teesta Barrage in Bangladesh has its canal headowrks on the convex bend (Figure 3.5) of the river (after imposing lead cut) and has been expected to work satisfactorily.

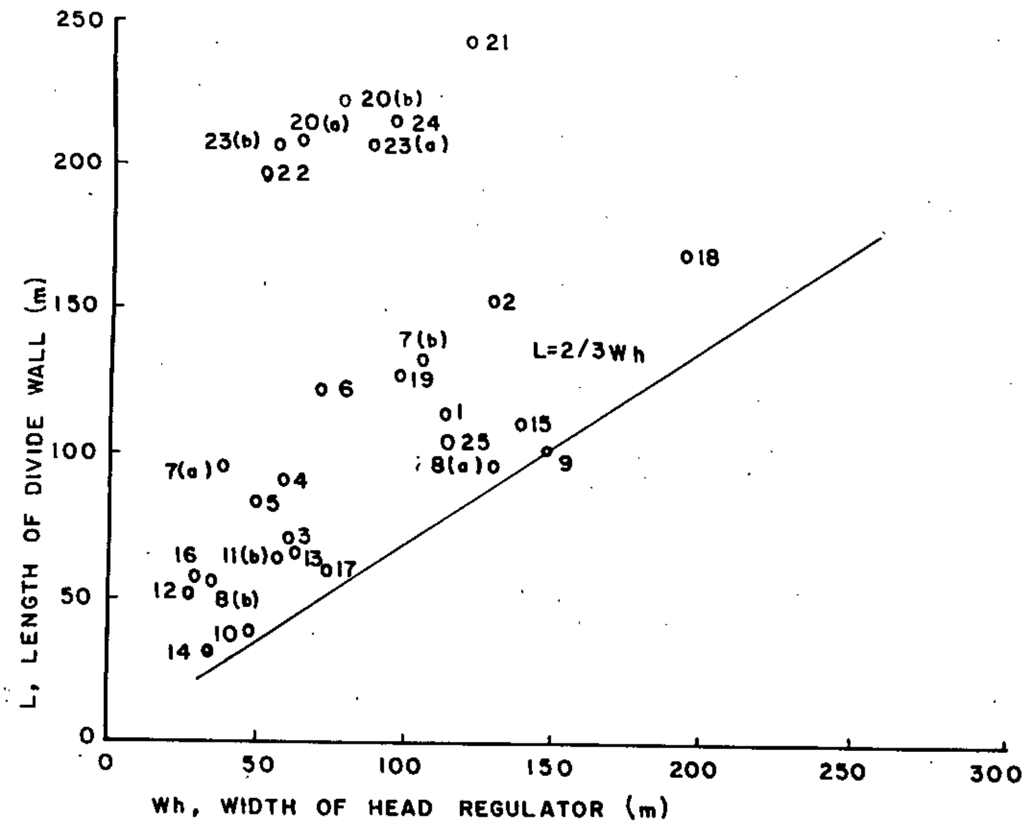
3.2 ORIENTATION OF OFFTAKE

Earlier it was indicated that (CBIP,1966) the offtakes were generally aligned at an angle of 90° to 105° with the barrage axis and this is shown in Table 3.1. This table also contains the offtake angle used for recent barrages. The latest practice in India (Sharma and Asthana,1975) is to orient the head regulator at 107° to 110° with the axis of barrage and is followed in the recently constructed headworks e.g., Narora, Dakpathar, Ashan, Ramganga, Kho etc. In Teesta Barrage, Bangladesh, offtaking angle of 112° has been used by BWDB (Figure 7.8) which nearly coincides with the recent works in India.

Table 3.1 Showing Orientation of Off-takes with Respect to Axis Barrage/Weir at Various Headworks(Dhillon,1980)

Sl.No.	Name of Headworks	Angle of Off-takes in degrees
1.	Sulemanki Headworks:	
	Left Pocket Regulator	104.0
	Right Pocket Regulator	104.0
2.	Ferozepur Headworks:	
	Left Pocket Regulator	104.0
	Right Pocket Regulator	104.0
3.	Islam Headworks:	
	Left Pocket Regulator	104.5
	Right Pocket Regulator	104.5
4.	Punjnad Headworks:	
	Left Pocket Regulator	103.5
	Right Pocket Regulator	–
5.	All-American Canal (U.S.A.)	111.0
6.	Rasul Headworks:	
	Lower Jhelum Canal	103.5
7.	Khanki Headworks:	
	Lower Chenab Canal	105.0
8.	Marala Headworks:	
	Upper Chenab Canal	90.0
9.	Kala bagh Barrage	90.0
10.	Emerson Barrage:	
	Haveli Canal	90.0
11.	Nangal Hydel Canal	102.25
12.	Madhopur Headworks:	
	Upper Bari Doab Canal	90.0
13.	Rupar Headworks:	
	Sirhind Canal Regulator	105.0
	Bist Doab Canal	–
14.	Harike Barrage:	
	Rajasthan Canal	101.1
15.*	Narora Barrage	107.0
16.	Gandak Barrage:	
	(i) Left	90.0
	(ii) Right	90.0
17.*	Kosi Barrage:	
	(i) West Kosi	102.5
	(ii) East Kosi	102.5
18.*	Dakpathar	110.0
19.*	Ahsan	107.0
20.*	Ram Ganga	105.0
21.	Shah Nehar Feeder Barrage	100.0
22.*	Teesta Barrage (Bangladesh)	112.0

*Indicates recent works



1. Sarda Barrage
2. Harike Barrage
3. Narora Barrage (L.G.C.)
4. Dakpathar Barrage
5. Ahgan Barrage
6. Gandak Barrage
7. Kosi Barrage (Bihar)
   (a) Right Bank Canal
   (b) Left Bank Canal
8. Girija Barrage
   (a) Right Bank Canal
   (b) Left Bank Canal
9. Lower Sarda Canal
10. Ram Ganga Barrage
11. Durgapur Barrage
   (a) Right Bank Canal
   (Data Not Available)
   (b) Left Bank Canal
12. Kosi Barrage (U.P.)
13. Rishi Kesh Barrage
14. Hindon Barrage
15. Narora Barrage (L.G.C.& PLGC)
16. Giri Barrage
17. Shah Nehar Barrage
18. Khanki Head Works (Lower Chenab Canal)
19. Marala Head Works (Upper Chenab Canal)
20. Ferozepur Barrage
   (a) Bikaner Eastern Canal
   (b) Dipalpur Canal
21. Panjnad Head Works (Abbasia Canal)
22. Islam Head Works
   (Mailsi Canal)
23. Sulemanki Head Works
   (a) Sadiqi Canal
   (b) Pakpattan Canal
24. Rupar Barrage
   (Sirhind Canal)
25. Teesta Barrage (Bangladesh).

Fig :3.6 VARIATION OF LENGTH OF DIVIDE WALL WITH WIDTH OF HEAD REGULATOR

(AFTER DHILLON, 1980)

Table 3.2 Width of Undersluice Pocket and Length of Divide Wall in Barrage(Dhillon,1990).

Sl.No.	Name of Structure	Design discharge m^3/s	Total Water way m	Canal discharge m^3/s	Width of canal head regulator m	Length of divide wall from barrage axis m	Width of undersluice pocket m	Width of regulator covered by the divide wall
1.	Sarda Barrage	16,997	598.63	328.4	115.82	112.78	68.28	Full
2.	Harike barrage	18,414	636.12	849.0	131.37	153.40	181.66	Beyond Regulator
3.	Narora Barrage(L.G C.)	14,164	924.15	240.7	62.48	67.00	124.97	Full
4.	Dakpathar Barrage	14,400	516.33	244.7	60.65	88.39	121.92	About 95 percent
5.	Ahsan Barrage	4,500	287.73	200.00	51.00	80.00	80.47	Full
6.	Gandak Barrage (both canals are equal)	24,079	742.80	509.8	71.63	121.92	120.40	Beyond regulator
7.	Kosi Barrage(Bihar)							
	(a) Right Bank canal	26,912	1490.10	127.40	40.83	83.39	79.55	Beyond regulator
	(b) Left Bank canal	26,912	1490.10	495.60	98.15	126.68	120.40	Full
8.	Girija Barrage.							
	(a) Right Bank canal	19,700	721.50	510.0	131.50	97.60	100.00	Half
	(b) Left Bank canal	19,700	721.50	195.00	46.00	55.00	59.00	2/3 rd.
9.	Lower Sarda Barrage.	7,200	407.50	780.0	150.50	100.00	100.00	Half
10.	Ramganga Barrage	7,360	408.00	151.8	50.00	37.10	41.00	2/3 rd.
11.	Durgapur Barrage.							
	(a) Right Bank canal	15,581	692.20	64.3	-	38.56	38.71	Beyond regulator
	(b) Left Bank canal	15,581	692.20	260.2	56.39	61.57	99.97	About 90 percent
12.	Kosi Barrage(U.P.)	5,100	142.50	73.7	28.5	50.00	40.00	Full
13.	Rishikesh Barrage	13,200	310.70	680.0	63.0	64.00	81.00	2/3 rd.
14.	Hindon Barrage	2.830	162.00	113.0 (56.5 m each)	32.0	30.00	38.50	2/3 rd.
15.	Narora Barrage (LGC&PLGC)	14,164	924.15	495.7 (225 PLGC & 240.7 LGC)	140.00	110.00	124.97	3/4th.
16.	Giri Barrage	5,180	160.93	47.0	37.18	53.88	37.74	Full
17.	Shah Nehar Barrage	11,320	561.75	382.05	75.64	58.86	43.0	2/3rd.
18.	Teesta Barrage (Bangladesh)	9,918.5	615.24	226.7	110.37	95.73	96.34	2/3rd.

3.3 DIVIDE WALL

In the past works, it was the practice to provide a very long divide wall with a length of 1.29 to 7.75 times (CBIP,1974, after Dhillon,1980) the width of head regulator (Figure 3.6). In Figure 3.6 a line defined by $L=2/3\ W_h$ has been drawn, where L is the length of the divide wall and W_h is the width of the head regulator. It is seen that majority of the divide wall lengths pertaining to recent works fall above the $L=2/3\ W_h$ line. For the barrage across Teesta River, the length of the divide wall is 95.73m which is $2/2.3\ W_h$ and falls above the $2/3\ W_h$ line.

3.4 WIDTH OF POCKET

Table 3.2 shows the width of the undersluice pocket and length of the divide wall for different barrages. It is observed that either a very wide or a very narrow pocket has not been recommended except for a very few cases. For Teesta Barrage (Table 3.2) the width of the pocket, W is 96.34m for $Q_c/Q_R=0.023$. From Figure 2.7a (for $Q_c/Q_R=0.023$) optimum ratio of L/W is 0.95, which gives the optimum width of the pocket as 100.77m. From Figure 2.7b (For $Q_c/Q_R=0.023$), approximate ratio of W/W_h is 1.08, which gives the approximate width of the pocket of 119.2m. Comparison of the values indicates that the pocket width is inadequate for Teesta Barrage.

3.5 LOCATION OF UNDERSLUICES

All the barrages already constructed in India and Pakistan contain single undersluice pocket adjacent to the canal head regulator for single offtake and two undersluice pockets near each head regulator for double offtakes for effective sediment control. A second pocket for river sluices has been reported (Dhillon,1980) to be successfully employed at Gandak Barrage, India (Figure 3.4). In Teesta Barrage a single offtake takes off from the right bank of the channel (Figure 3.5).

3.6 CREST LEVEL OF UNDERSLUICE, WEIR AND HEAD REGULATOR

Table 3.3 contains crest levels of undersluice, weir and head regulator for different barrages. It is observed that the difference of crest level between undersluice and weir is between 1.2m to 4m. The table also shows that this differences are 1.54m and 1.83m for Kosi and Teesta Barrage respectively. In the early works, crest level height for the head regulator from the crest of undersluice varies from 0.98 to 5.18m. In Japan it is normal practice to use raised crest (Hiroyasu and Koichi,1975) of head regulator sufficiently above the stream bed as a preventive measure to control sediment. It has been mentioned that though the Fukuyama and Aimoto Water Intake of Japan are running effectively but low sediment laden flow from top of alluvial channel could not be prevented from entering the offtaking canal. In barrage over river Chenab, Pakistan (PIPD,1978) the crest level of head regulators has been kept higher from the top level of excluder by nearly 1m considering the river being alluvial. In Teesta Barrage crest level of head regulator is only 0.305m above the top level of excluder tunnel.

Table 3.3 Statement Showing Sill Levels of Weir, Undersluice and Head Regulator (Dhillon, 1980)

Sl.No.	Name of Headworks	R - L (m) of Weir crest	R - L (m) of Undersluice crest	R - L (m) of Head Regulator sill	Difference (m) between undersluice and Head regulator crest
1.	Madhopur Headworks Upper Bari Doab Canal		344.272	346.405	2.133
2.	Tajewala Headworks Western Yamuna Canal.		320.650	322.478	1.828
3.	Ferozepur Barrage				
	(i) Dipalpur Canal		192.176	194.462	-
	(ii) Eastern Canal		192.176	194.462	2.286
4.	Khanki Headworks Lower Chenab Canal	221.646	217.627	221.285	3.658
5.	Quadirabad Barrage	208.689	207.317	210.366	3.049
6.	Marala Headworks Upper Chenab Canal	-	241.402	242.386	0.984
7.	Trimmu Barrage	145.579	143.902	147.409/146.646	3.507/2.744
8.	Rupar Headworks Sirhind Canal	-	261.214	263.957	2.743
9.	Punjnad Headworks				
	(i) Main Line Canal		99.060	101.346	2.286
	(ii) Abbasia Canal		99.060	100.584	1.524
10.	Islam Headworks				
	(i) Mailsi Canal		134.517	135.636	1.119
	(ii) Bahawalpur Canal		134.517	135.636	1.119
	(iii) Quainspur Canal		134.517	135.941	1.424
11.	Madhopur Addl. Regulator and underlsuice		344.424	346.710	2.286
12.	Resul Headworks		213.665	215.494	1.829
13.	Harkie Barrage		204.826	-	-
14.	Nangal Hydel Channel	338.94	337.718	342.900	5.182
15.	Dakpathar		449.928	451.348	1.420
16.*	Narora Barrage		174.589	176.327	1.738
17.	Gandak Barrage				
	(i) Left		104.242	106.375	2.133
	(ii) Right		104.242	106.375	2.133
18.*	Kosi Barrage				
	(i) West Kosi	71.646	70.104	71.933	1.829
	(ii) East Kosi	71.646	70.104	71.933	1.829
19.*	Ahsan		395.2	397.5	2.3
20.*	Ram Ganga		223.098	224.497	1.398
21.	Shah Nehar Feeder Barrage		324.225	336.441	2.216
22.*	Teesta Barrage	48.7805	46.9512	49.2378	1.2866

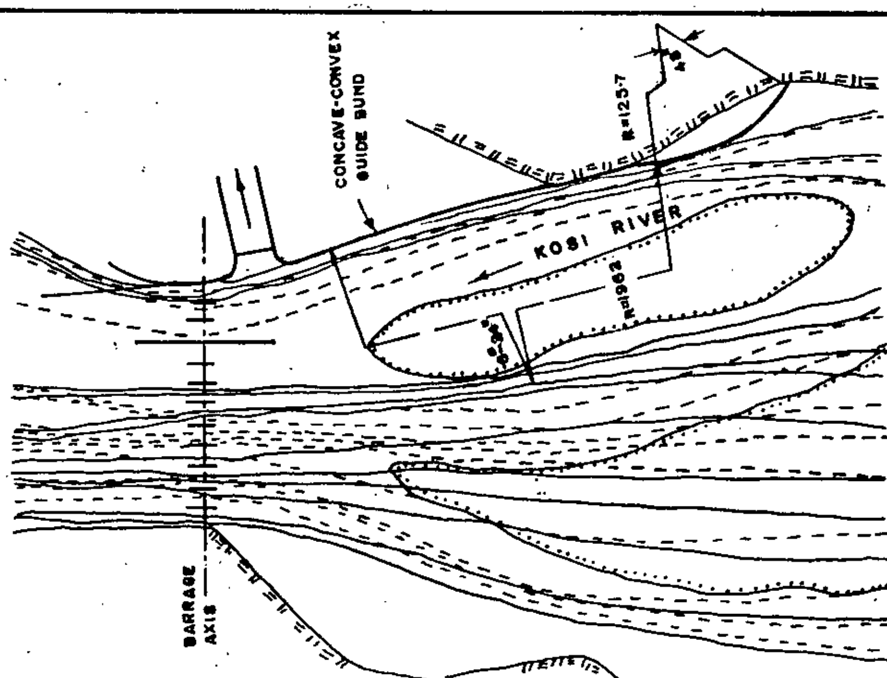


Fig: 3.7 CONCAVE CONVEX GUIDE BUND AT KOSI BARRAGE (U.P). (AFTER SHARMA - ASTHANA, 1975)

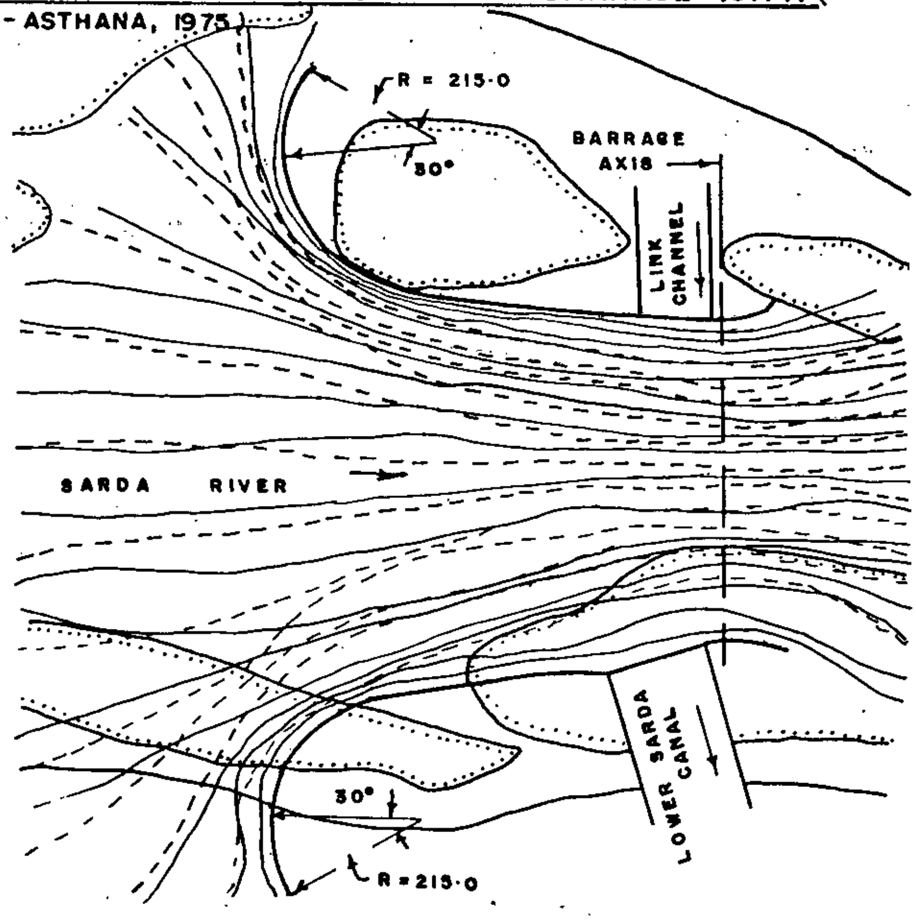


Fig: 3.8 ELLIPTICAL GUIDE BUND AT LOWER SARDA BARRAGE. (AFTER SHARMA - ASTHANA, 1975)

3.7 SHAPE OF GUIDE BUNDS

In Sind Barrage, Sulemanki, Pakistan converging or bottleneck type guide bunds (Joglekar,1971) were used and it was reported that the guide bunds did not provide effective measure for sediment control. Large islands were formed at the upstream of the undersluice pocket. In Kotri Diversion Dam, Pakistan (Figure 3.2) diverging type guide bunds were employed and the canals had been found working satisfactorily (Joglekar,1971). Concave guide bunds are also functioning well in Sukkur Barrage (Figure 3.1), Pakistan (Dhillon,1980). Model study of Kosi Barrage, India (Figure 3.7) has indicated the superiority of concave-convex guide bund. Recently UPIRI (1973, after Dhillon, 1980) recommended the elliptical guide bunds to be superior at all river discharges and has been applied in the Lower Sarda Barrage, India (Figure 3.8) though nothing regarding workability for post barrage condition is available. In Teesta Barrage, Bangladesh the left guide bund has straight shank (perpendicular with barrage axis) with a curved head while the right guide bund has straight shank (splayed at an angle of 106° with the barrage axis) with a curved head of gradually varying radius. Thus converging type guide bunds have been employed in Teesta Barrage.

3.8 BARRAGE REGULATION

Ghosh (1975) has tested various alternative regulations in the Kosi Barrage and found still pond regulation to be suitable for lower flow in dry months and Semi-still pond regulation for higher flow in monsoon period. Semi-still pond regulation has been extensively used (Sharma and Asthana,1975) in the headworks in India. During high flood when the concentration exceeds 3000 ppm in the river, the offtake canal is closed (Ghosh,1975) for Kosi Barrage. The method of regulation for Teesta Barrage has not yet been decided as the barrage is yet to be operated.

3.9 TUNNEL TYPE SEDIMENT EXCLUDER

The first sediment excluder has been constructed in the pocket of Lower Chenab Canal at Khanki Headworks (Figure 2.8), Pakistan in 1934 (Dhillon,1980). Subsequently Punjab Irrigation Research Institute, Lahore, Pakistan conducted a model study and has found (Dhillon,1980) that (1) the side openings provided in the tunnel were not very effective and (2) the three tunnels discharging in bay no.2 of the undersluice were not very effective as the other three. When the side openings and the three ineffective tunnels were blocked in the field, it was found to be functioning well. Later an excluder had been constructed at Trimmu Headworks (Figure 2.9) for Haveli Main Canal, Pakistan. After studying the behaviour of the Khanki and Trimmu types of excluders the Central Water and Power Commission, India has evolved an excluder which is more or less a combination of the above two types and is known as CWPC type (Figure 2.10). A number of excluders of this type have been recently constructed on a number of headworks e.g. Gandak, Sone, Kosi, Farrakka and Lower Sarda Barrage. BWDB has used Khanki type sediment excluder for Teesta Headworks. Use of different types of excluders with their efficiency is shown in Table 3.4.

Table 3.4 Sediment Excluders(Dhillon,1980)

Sl. No.	Excluder	Year of construction	Stage of river	Type of excluder	Numbers of tunnels	Length of regulation covered by excluder	Efficiency Model	Efficiency Prototype
1.	W.J.C. at Tajewala	1942-43	Boulder, (sand and grave)	Khanki	2	2/3rd length	98.0	98.0
2.	Thal Canal at Kala bagh Headworks	1944	-do-	Modified Khanki	6	Full length	-	-
3.	Remodelled sediment Excluder at W.J.C. at Tajewala	1945	-do-	Khanki	5	5/6th length	95.0	93-98
4.	Nangal Hydel Channel at Nangal	1954	-do-	-do-	6	Full length	-	-
5.	Lower Chenab Canal Khanki	1933-34	Alluvial	-do-	6	-do-	65.0	80.0 (for size).2 mm)60-70)
6.	Haveli Main Line Canal at Emersun Barrage	1937	-do-	Emersun	4	-do-	70.0	72
7.	Rajasthan Canal at Harike	1952-53	-do-	Khanki	12	-do-	70.0	-
8.	Sediment Excluder in the left pocket at Tilpara Barrage (West Bengal)	1949-50	-do-	-do-	2x4	-do-	-	-
9 .	Lower Ganga Canal at Nirora	1967	-do-	-do-	6	-do-	91	50.87
10.	Lower Sarda Canal at Sarda Barrage	1974	-do-	C.W.P.C	14	-do-	50	-
11.	Teesta Barrage (Bangladesh)	Under construction	-do- (Silt to Sand)	Khanki	12	-do-	-	-

CHAPTER IV

SEDIMENT MOVEMENT IN ALLUVIAL CHANNELS

4.1 RESISTANCE TO FLOW IN ALLUVIAL STREAMS

During the past two hundred years or so, several emperical formulae have been suggested for channel resistance. Among these Chezy (Simons and Senturk,1976) and Manning (Simons and Senturk, 1976) equations are most commonly used which are respectively,

$$\bar{U} = C\sqrt{RS} \tag{4.1}$$

$$\bar{U} = 1/n \; R^{2/3}S^{1/2} \tag{4.2}$$

Where $\bar{U}$ is the average velocity, C is the Chezy's coefficient, n is the Manning's roughness coefficient, R is the hydraulic radius and S is the slope. Chezy and Manning equations do not take into account the effect of viscocity on resistance. Hence these equations can be dependable only when viscous effects are negligible or when boundary is hydrodynamically rough.

It is well known that alluvial bed deforms into ripples, dunes etc. when the sediment moves due to increase of discharge. Thus for alluvial channels total resistance can be considered to be the sum of the grain resistance and form resistance due to the bed undulations. Considering the total resistance, Lacey (1930, after Garde and Ranga Raju,1985) developed resistance relationship on the basis of Indian stable channel data.

He suggested an equation for mean velocity as follows;

$$\bar{U} = 10.8\ R^{2/3}\ S^{1/3} \qquad 4.3$$

Lacey also tested the applicability of the equation for river data and found the equation to be valid for rivers at dominant discharge. However he stated that the equation may not be applicable for rivers at all stages.

Alam and Kennedy (1969, after Vanoni,1977) divided the slope into two components as S = S'+S" to have seperate resistance for grain roughness and bed form roughness. These authors defined S' as the slope at which the flow would have if the bed were plane and S" is the additional slope resulting from bed undulations. Such a separation technique has been earlier adopted by Meyer-Petter and Müller (1948, after Garde and Ranga Raju,1985) and the velocity relation is

$$\bar{U} = \sqrt{8gRS/f} \qquad 4.4$$

and $f = f' + f''$ 4.5

where $f' = 8gRS'/\bar{U}^2$ 4.6

and $f'' = 8gRS''/\bar{U}^2$ 4.7

Here f' and f" are the friction factors associated with grain roughness and bed roughness respectively. Lovera and Kennedy (1969, after Vanoni,1977) studied the variation of f' for a plane bed with sediment motion and obtained relationship between

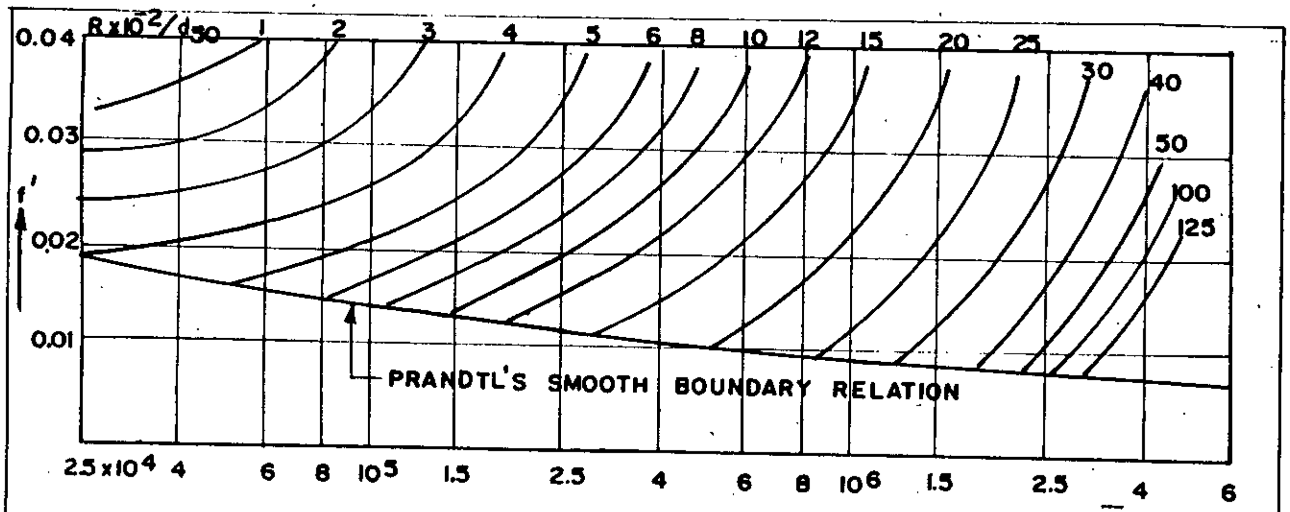


Fig. 4.1 FLAT - BED REGISTANCE (AFTER LOVERA-KENNEDY, 1969) $Re = \frac{\bar{U}R}{\nu}$

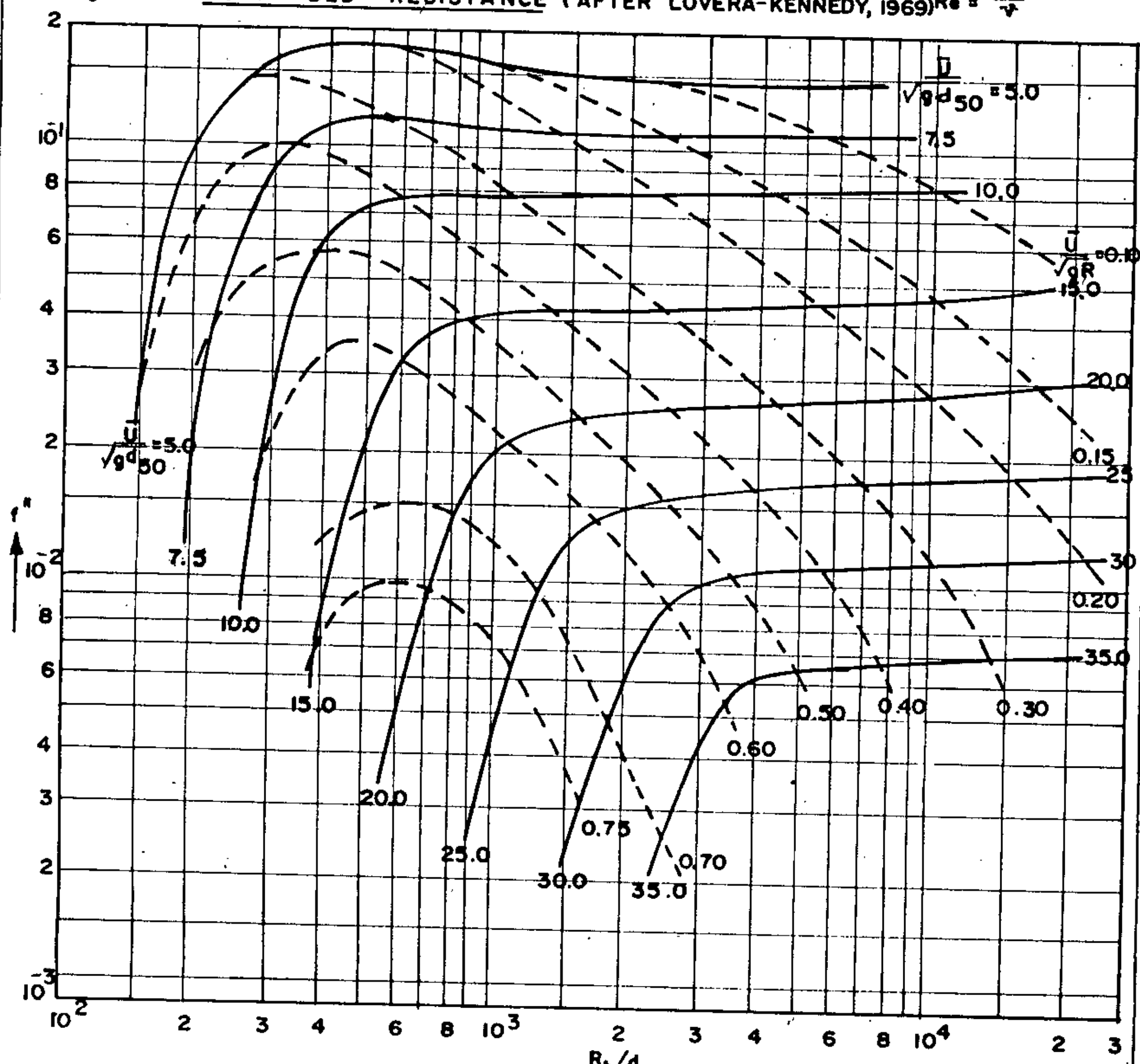


Fig. 4.2 BED FORM REGISTANCE (AFTER ALAM-KENNEDY, 1969)

f', $\bar{U}R/\nu$ and R/d. Figure 4.1 shows this relationship from where f' can be obtained. Alam and Kennedy (1969, after Vanoni,1977) postulated that f" will be a function of $\bar{U}/\sqrt{gd}$, $\bar{U}/\sqrt{gR}$ and R/d. Figure 4.2 shows this relationship from where f" can be obtained.

4.2 BED FORMS

As the sediment characteristics, the flow characteristics and fluid characteristics are changed in alluvial channels, the nature of the bed surface and the water surface changes accordingly. These types of the bed and water surfaces are classified according to their characteristics and are called bed forms (Garde-Albertson,1959, after Garde and Ranga Raju,1985).

Bed forms are generally classified as ripples, dunes, transition, plane bed, antidune, chutes and pools, and bars. Figure 4.3 and Table 4.1 gives detail description about common bed forms. From Table 4.1, a qualitative information about sediment discharge may also be available for different bed forms.

Prediction of bed forms can be done by using Figures 4.4 to 4.8. Figure 4.4 represents Albertson-Simons-Richardson's (Garde and Ranga Raju,1985) criteria developed on the basis of flume data and it does not hold good for natural streams where depths are large and slopes are flat. Figure 4.5 represents Bogardi's (Garde and Ranga Raju,1985) criteria developed on the basis of a few field data and flume data, but does not hold good for natural streams of larger depth and flat slopes. Figure 4.6 represents Garde-Ranga Raju's (1963, after Garde and Ranga Raju,1985) criteria which does not involve velocity of

Table 4.1 Summary Description of Bed Forms and Configurations

Bed Form	Dimensions	Shape of bed form and water surface	Occurrence, Behaviour and Sediment Transport
Ripples	Wave length less than approximately 30 cm and height less than approximately 5cm.	Roughly triangular in profile, with gentle, slightly convex upstream slopes and downstream slopes nearly equal to angle of repose.Generally short crested and three dimensional. water surface in phase with the bed form.	Occured at low shear stress. Move downstream with velocity much less than that of flow. Suspended load is minimum and main load of transport is bedload and total load is small.
Dunes	Wave length greater than 60cm and less than 3m; and height greater than 6cm and less than 30cm.	Shape of bed form is similar to ripple. Water surface is out of phase of bed form.	Upstream slopes of dunes may be covered with ripples. Dunes migrate downstream in manner similar to ripples. Suspended and bedload is greater than ripple bed form, but bedload is still main load of transport.
Transition	Wave length increases than the wave length of dune, but the height decreases.	Shape of bedform may vary widely.Water surface in phase with the bed form.	A configuration consisting of a heterogenous array of bed forms, primarily low amplitude ripples and dunes interspersed with flat region. Suspended and bedload increases and may be of equal proportion.
Plane bed	Wave length is much higher and height is much lower and may seen to be without bed form.	Water surface in phase with the bed form.	May not occur for some ranges of depth and sand size. Suspended load is the main load of transport.
Antidune	Wave length = $2\pi \bar{U}^2/g$ (approx)[a], Height depends on depth and velocity of flow.	Nearly sinusoidal in profile. Water surface in phase with the bed form.	Standing wave or breaking wave antidune may occur. Antidune may move upstream. Suspended load of transport is the main load of transport.
Chutes and pools	-	-	For supercritical flow chutes and pools are formed. Suspended load is the main load of transport.
Bars	Wave length comparable to the channel width and height comparable to mean flow depth.	Profile similar to ripples and variable plan form.	Four types of bars are distinguished: (1) Point; (2) alternating; (3) transverse; and (4) tributary. Ripples may occur on the upstream slopes.

[a]Reported by Kennedy (1969, after Vanoni,ed.,1977)

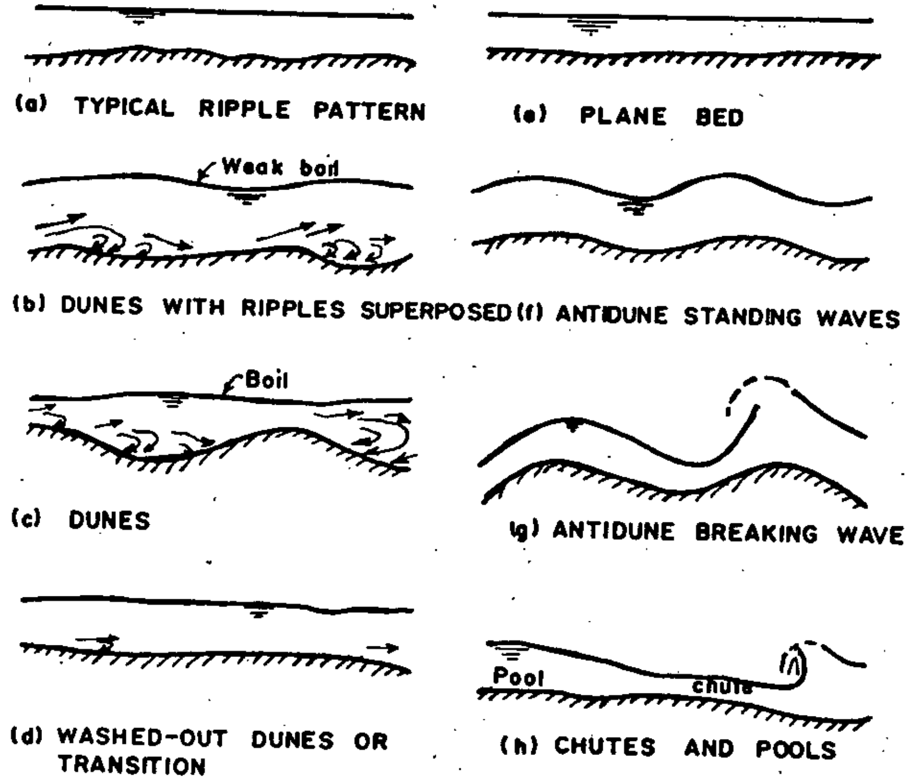


Fig. 4.3 BED FORMS

(AFTER SIMONS-RICHARDSON, 1964)

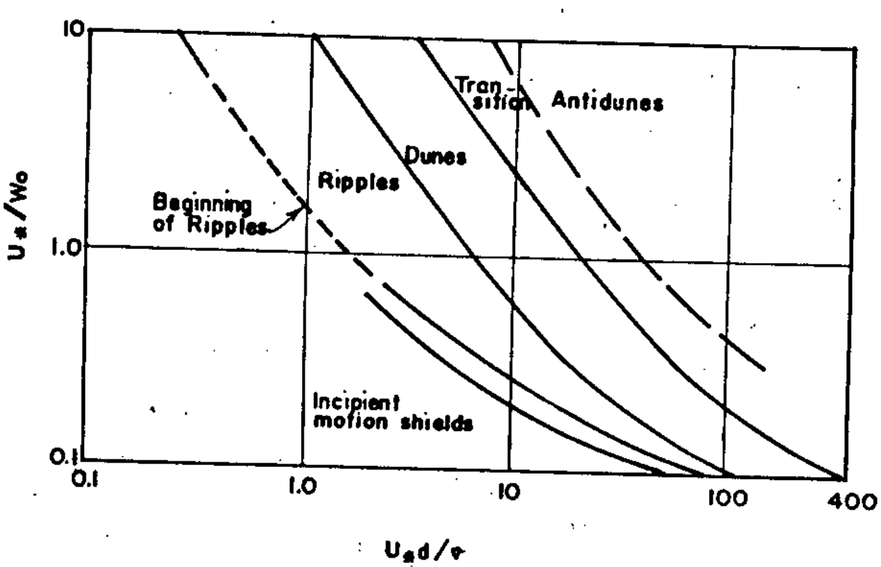


Fig. 4.4 ALBERTSON - SIMONS - RICHARDSON'S CRITERION FOR REGIMES OF FLOW (AFTER GARDE & RANGA RAJU, 1985)

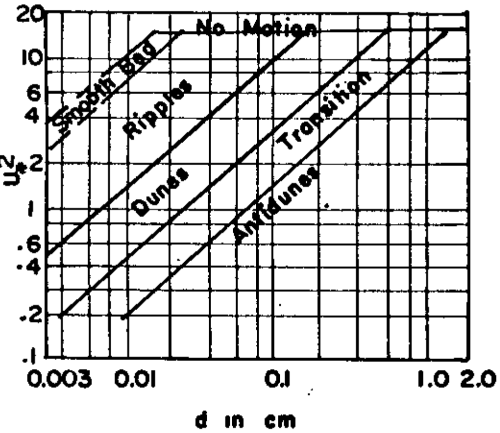


Fig. 4.5 BOGARDI'S CRITERION FOR REGIMES OF FLOW (AFTER GARDE & RANGA RAJU, 1985)

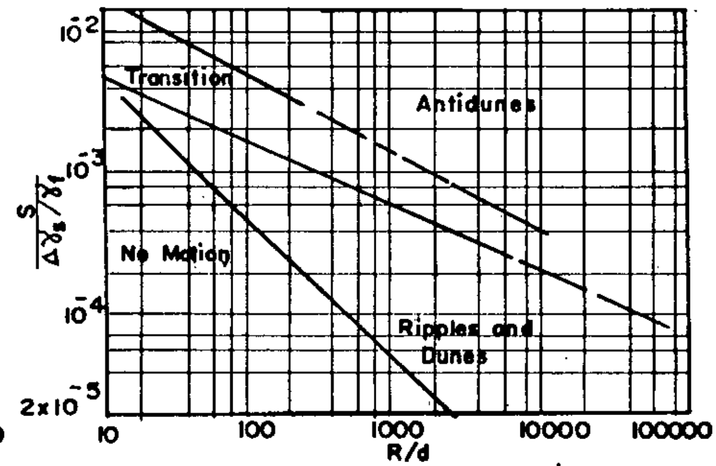


Fig. 4.6 GRADE AND RANGA RAJU'S CRITERIA

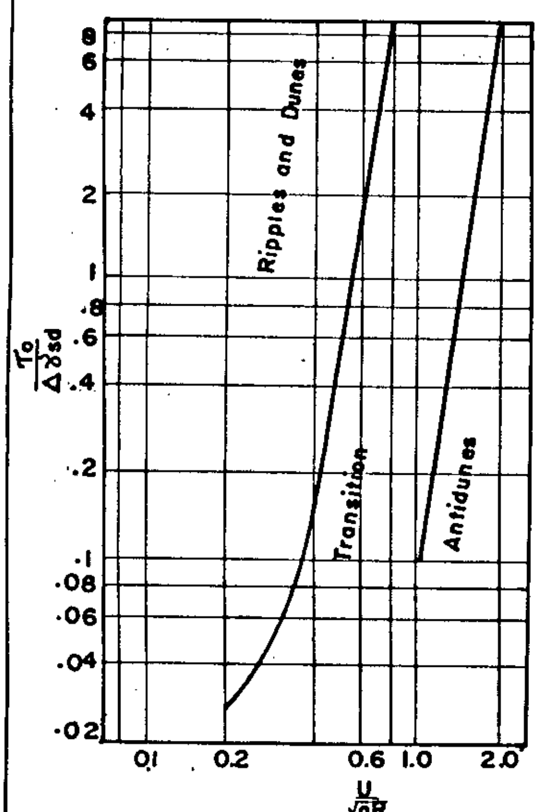


Fig. 4.7 GARDE-ALBERTSON'S REGIME CRITERION (AFTER GARDE & RANGA RAJU, 1985)

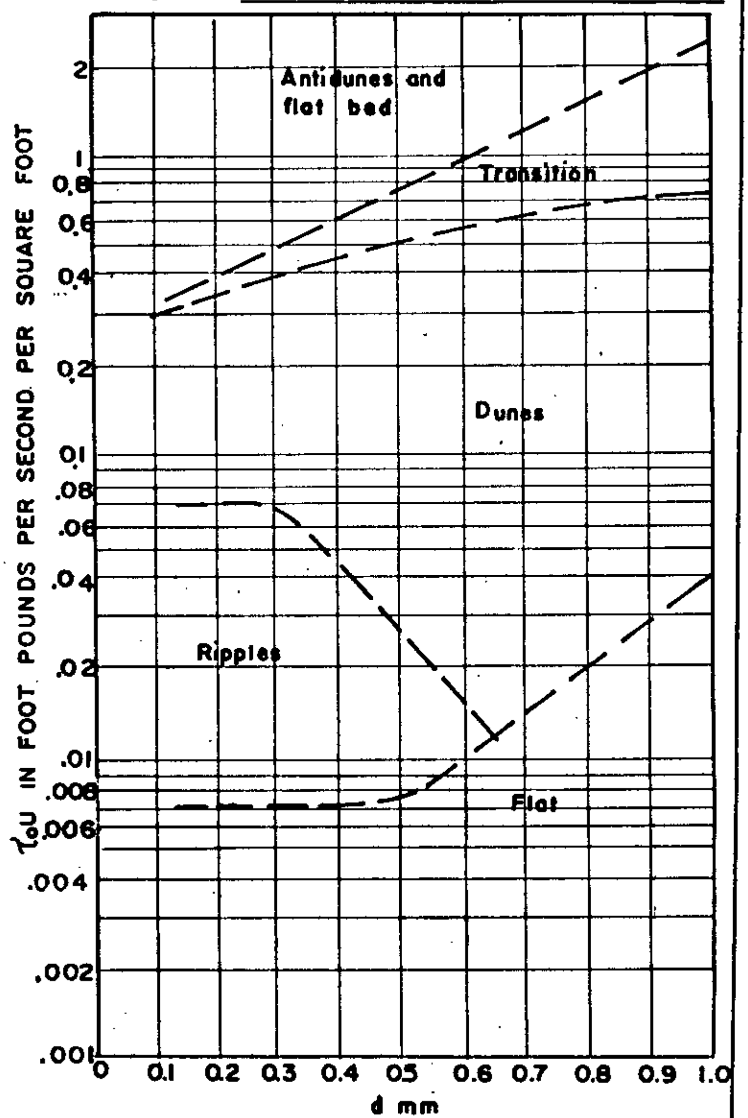


Fig. 4.8 SIMONS-RICHARDSON'S REGIME CRITERIA (AFTER SIMONS-RICHARDSON, 1962)

flow and can be used for prediction of regimes in problems related to resistance. Figure 4.7 represents Garde-Albertson's (1959, after Garde and Ranga Raju,1985) criteria developed on the basis of several sets of field and flume data, where a clear demarcation line has been observed between ripple and dune regime from transition regime. The antidune regime on the other hand has not been found to be predicted on the same degree of accuracy. It may be concluded from Figure 4.7 that $\bar{U}/\sqrt{gR} > 1$ gives a fairly good criteria for antidune formation. Figure 4.8 represents Simons-Richardson's (1966, after Vanoni,1977) criteria developed on the basis of field and flume data. Nordin (1964, after Vanoni,1977) observed that Figure 4.8 could have been suitable as regime predictor for natural streams where the depths were less than 5 ft and the velocity were relatively high.

4.3 MECHANICS OF SEDIMENT TRANSPORTATION

Water flowing over a bed of sediment exerts hydrodynamic forces on the grains. These forces tend to lift or entrain the particle in the direction of flow. When the hydrodynamic force has reached a value that, if increased even slightly, will put the sediment particles into motion, threshold conditions or conditions of incipient motion are said to have been reached. At such conditions of critical motion there is a balance between the restraining forces arising from the immersed particle weight and interparticle friction, which tend to keep the particle at rest, and the fluid forces of drag and lift which tend to dislocate the particles..

There are three different approaches which have been used to establish the condition for incipient motion of sediment particles as:

i) Competency

ii) Lift concept

and iii) Critical tractive force approach

Among the three approaches to the problem of defining the hydraulic conditions at incipient motion, it is critical tractive force approach which has gained wide acceptance and seems to be rational (Garde and Ranga Raju,1985). Vanoni(1977), after analysing the works of different researchers recommended to use the data on critical shear stress wherever possible.

Several investigators such as Kramer (Chang,1939), USWES (Iwagaki,1956 and Shulits,1937), Change (1939), Krey (Iwagaki, 1956), Indri (Iwagaki,1956), Schoklitsch (Shulits,1937), Aki and Sato (Iwagaki,1956) and Sakai (Iwagaki,1956) developed several emperical equations for critical tractive stress. But Garde and Ranga Raju(1985) have suggested not to use the emperical equations blindly, as these equations do not take into account the viscous effect, mechanics of lift and flow condition etc.

Theoretical and semitheoretical equations for critical tractive stress have been developed by several investigators such as Shield (1936), White (1940), Iwagaki (1956), Yalin and Karahan (1979), and Jansen (1979). The computed value of critical tractive stress by different authors differs due to different critical Reynold's number, $R_c{}^* = U_{*c} d/\nu$ for laminar and turbulent flow. Moreover the influence of lift force and gradation of material have not been considered equally by these investigators.

Various investigators have taken the critical Reynold's number, R_c* as the criterion for fully developed turbulent flow. Shield (1936) had fixed the limit $R_c* \geqslant 200$, while Yalin and Karahan (1979), Iwagaki (1956) and White (1940) had set the value at 70, 51.1, and 3.5 respectively. From their works it may be observed that the value of the non-dimensional critical stress, $\tau_{oc}/[(S_s-1)\gamma_f d]$ varies from 0.037 to 0.06. Garde and Ranga Raju (1985) have suggested to use $\tau_{oc}/[(S_s-1)\ \gamma_f d\ \tan\phi\ ] = 0.045$ to 0.05 for fully turbulent flow at $R_c* > 100$, where τ_{oc} is the critical shear stress.

Similarly the value of the critical Reynold's number set for development of laminar flow in mobile bed channel are equal to or less than 2, 1, 6.83 and 3.5 by Shield (1936), Yalin and Karahan (1979), Iwagaki (1956) and White (1940) respectively. From their works it may be observed that the value of non-dimensional critical stress lies between 0.06 to 0.2. Garde and Ranga Raju (1985) have agreed to use White's (1940) criterion i.e., $\tau_{oc}/[(S_s-1)\gamma_f d] = 0.18$.

4.4 CRITICAL REVIEW OF SEDIMENT TRANSPORT EQUATIONS

a) Modes of Sediment Movement in Alluvial Streams

The analysis of transport of sediment is usually grouped into three parts: bedload, suspended load and wash load. Bedload is the general name given to the material transported along the bottom of a channel by sliding and rolling and is essentially in contact with the bottom except for very short distances i.e., saltation. Suspended load is that part of the sediment load that goes in suspension and is composed of those sizes of sediment found in abundance in the bed. Wash load is that part of the sediment load which remains in suspension and is composed of particle sizes smaller than those found in appreciable quantity in the bed. Bedload and suspended load are expressed by a single expression called bed material load and all the three are termed by a single expression called total load. Einstein (1950) recommended that the limiting size for the wash load may be arbitrarily chosen from the mechanical analysis of bed material, as that particle size for which 10 percent of the bed material is finer. As a rule of thumb, many engineers assume that the size less than 0.0625mm of bed material produces wash load. Middle Loup river, USA, carries wash load of about 10 percent (Benedict and Matejka,1962) of total load. In Bangladesh suspended load is the main load of transport (more than 70%) and wash load may be higher than 10 percent of total load.

b) Review of Various Approaches of Sediment Transport Equations

Engineers engaged in river regulation and design and operation of canal system have great need for a reliable method of computing sediment discharge. Unfortunately, available methods or relations for computing sediment dicharge are far from completely satisfactory. Raudkivi (1971) states that errors of the order of 100 percent are to be expected and recommends that more than one formula should be used in any given circumstances.

In the design of sediment excluder and to have the entrainment of gain-size range into the main canal, seperate assessment of bedload and suspended load is necessary. For the beneficial use, different bedload and suspended load equations with their critical remarks are shown in Tables 4.2 and 4.3 respectively.

There are three different approaches to the problem of bedload transport computations. They are

1) the DuBoys type equation deriving from a shearing stress relationship
2) the Schoklitsch type equation deriving from a discharge relationship
3) the Einstein type equation based upon statistical consideration of the lift force

Bedload equations are also available based on the consideration of bed form motion and energy concept. The different approaches adopted by different investigators led to the same functional relationship when reduced to a simpler form (Herbertson,1969 and Garde and Ranga Raju,1985). In fact, most of sediment discharge formulas predicting bedload may be reduced to one of the following forms:

$$g_b = Af\ (\tau_o - \tau_{oc})$$

$$g_b = Af\ (q - q_c)$$

$$g_b = Af\ (U - U_{cr})$$

in which the quantities with subscripts c or cr refer to the incipient condition and A is a constant related to sediment and fluid characteristics. In their review of sediment transport equations of White et alia (Hossain,1984) concluded that Rottner's (1959) bedload equation can be used with confidence. Gole, Tarapore and Dexit (1973) have compared 10 formulae and have concluded that the DuBoys-straub and Einstein-Brown formula were better than others. BUET (1987) have found Rottner's equation to be in good agreement for Teesta River while estimating bedload.

Suspended load equations have been developed mainly on the assumption of fully developed flow in which equilibrium was maintained between sediment inflow and sediment outflow for any reach. Einstein (1950) developed a theoretically sound method for estimating suspended sediment load carried by the stream. It is to integrate few (concentration X velocity) curves after knowing the sediment concentration and the velocity. To avoid the complexity of knowing the concentration at a particular level, atempts have been made to relate suspended sediment load directly to water discharge considering the individual fractions in a mixture. The hypothesis is that only a fraction of the total shear stress is responsible for that fraction of the material to be in suspension.

Table 4.2 Bedload Equations

No.	Investigation	Bedload Equation	Remarks
1.	Duboys (1879, after Graf,1971)	$q_B = K\tau_o\ (\tau_o - \tau_{oc})$	It is a theoretical equation, where q_B is the bedload discharge rate, τ_o and τ_{oc} are the average and critical shear stress respectively and K is a constant. Value of τ_{oc} and K (Straub, 1935) is shown in Figure 4.9.
2.	O'Brien and Rindlaub (1934, after Graf,1971)	$q_B = K'(\tau_o - \tau_{oc})^m$	It is an emperical equation based on excess shear consideration, where K' is a constant, m varies from 1.5 to 1.8 for sediment size of 0.025 to 0.56 mm.
3.	Shields (1936, after Vanoni,ed.,1977)	$g_B = 10qS(\tau_o - \tau_{oc})/[(S_s-1)^2*d_{50}]$	It is a dimensionally homogeneous equation, where g_B bedload rate, S is the slope, S_s is the specific gravity. Using flume data of $1.06 < S_s < 4.25$ and $1.5 < d_{50} < 2.47$mm, the equation was derived.
4.	Kalinske (1947,after Garde,1985)	$g_B = 2.57U_*d\gamma_s\bar{U}_g/\bar{U}$	It is a semitheoretical equation, where U_* is the shear velocity, γ_s is the specific weight of sediment. $\bar{U}_g/\bar{U} = f(\tau_{oc}/\tau_o, r)$ is shown in Figure 4.10, where $r = \sqrt{(U-\bar{U})^2}/\bar{U}$, U and $\bar{U}$ are the instantaneous and average velocity respectively.
5.	Meyer-Peter Müller(1948, after Garde,1985)	$0.25(\gamma_f/g)^{1/3}(g_B/\gamma_s)^{2/3}/[(\gamma_s - \gamma_f)^{1/3}d_a] =$ $(K/K')^{3/2}\gamma_f RS/[(\gamma_s - \gamma_f)d_a] - 0.047$	It is a dimensionally homogeneous equation, where g is the acceleration due to gravity, γ_f is the specific weight of water, d_a varies between d_{50} to to d_{60}, R is the hydraulic radius, K/K' varies between 0.5 to 1 and is 0.5 and 1 for strong bed form and for no bed form respectively. The author used flume data of sediment size from 0.4 to 30mm and $1.25 < S_s < 4.22$ for developing the equation.
6.	Schoklitsch (Graf,1971)	$g_B = 2500S^{3/2}(q-q_c)$ in which $q_c = 0.26(S_s-1)^{5/2}d_{40}^{3/2}/S^{7/6}$	The equation was developed on the excess discharge consideration, where q and q_c are the water discharge rate and the same under critical condition respectively.
7.	Einstein-Brown(Brown,1950 after Vanoni, ed, 1977)	$q_B = 40[\tau_o/\{\gamma_f(S_s-1)d_{50}\}]^3\ [\sqrt{\{(S_s-1)gd_{50}^3\}}*$ $\sqrt{[2/3+36\nu^2/\{gd_{50}^3(S_s-1)\}]}-$ $\sqrt{[36\nu^2/\{gd_{50}^3(S_s-1)\}]}]$	It is a dimensionally homogeneous equation, where ν is the kinematic viscosity of fluid. The equation was based on the flume data of sediment median sizes from 0.3 to 7mm, gravel of size 5.21mm and 28.6mm and barite and coal.
8.	Einstein(1950,after Vanoni,ed., 1977)	$1-1/\sqrt{\pi}\int_{-(1/7)\Psi_{*i}-2}^{(1/7)\Psi_{*i}^{-2}} e^{-t^2}\,dt$ $= 43.5\Phi_{*i}/(1+43.5\Phi_{*i})$ in which $\Psi_{*i} = \xi_i Y[\mathrm{Log}10.6/\mathrm{Log}(10.6xX/d_{65})]^2 *$ $(S_s-1)d_{si}/(R_b'S)$ $\Phi_{*i} = [g_{Bsi}/(P_i\gamma_s)]*\sqrt{[1/\{(S_s-1)gd_{si}^3\}]}$	It is a semi-theoretical equation and is dimensionally homogeneous, where g_{Bsi} is the bedload rate of mean size d_{si}, P_i is the percent fraction by weight of mean size d_{si}, R_b' is the hydraulic radius of bed due to sand grain roughness and t is the only variable of integration. Value of x, ξ and Y are shown in Figures 4.11, 4.12 and 4.13. The equation was developed by using flume data of d_{50} = 28.5mm and 0.785mm.

9. Rottner (1959)	$g_B/[\gamma_s\sqrt{(S_s-1)gD^3}]=[\{0.667(d/D)^{2/3}+0.14\}*\bar{U}/\sqrt{(S_s-1)gD}-0.788(d/D)^{2/3}]^3$	The formula is based on dimensional consideration, where D is the depth of flow. The equation was developed by using flume data of 0.205 < d_{50} < 15.49mm.
10. Shinohara and Tsabuki (Hossain, 1984)	$q_B = (1-P)V_b\Delta/2 + a$	The formula is based on bed form motion, where P is the porosity of sand, V_b is the velocity of sand wave at the bed in the direction of flow, Δ is the average amplitude of the sand wave and a is a consonant accounts for that part of the bedload which does not enter into the the propagation of ripples or dunes.
11. Barekyan(1962,after Graf,1971)	$g_B=0.187\gamma_f[\gamma_s/(\gamma_s-\gamma_f)]qS(\bar{U}-U_c)/U_c$	The equation was derived on velocity consideration, where U_c is the the critical velocity.
12. Yalin(1977,after Garde,1985)	$g_B/(\Delta\gamma_s U_* d)=0.635(\tau_*/\tau_{*c}-1)[1-2.3Log\{1+2.45\sqrt{\tau_{*c}}*(\tau_*/\tau_{*c}-1)/S_s^{0.4}\}/\{2.45\sqrt{\tau_{*c}}*(\tau_*/\tau_{*c}-1)/S_s^{0.4}\}]$	It is a theoretical equation, where τ_* is the dimension less shear stress, τ_{*c} is the dimensionless critical shear stress. The formula is restricted to plane bed for fully developed turbulent flow and to large value of relative roughness D/d.
13. Chang-Simons-Richardson (1967,after Waliuzzaman, 1986)	$g_B = K_T\bar{U}(\tau_o - \tau_{oc})$	The equation was developed on excess shear stress, where K_T is a constant and varies between 0.27 to 1.10 when applied to the Colorado, Middle loup and Niobraba rivers to have the result in F.P.S system.
14. Misri et al (1980, after Garde, 1985)	$\phi = g_B\sqrt{[1/\{(S_s-1)gd^3\}]}/\gamma_s$ in which $\phi = 3.62*10^{-7}\tau_*'^{8}$ for $\tau_*' < 0.065$; $\phi = 8.5\tau_*'^{1.8}/[1+5.95*10^{-6}\tau_*'^{-4.7}]^{1.45}$ for $\tau_*' > 0.065$	The equation is based on dimensional consideration. For sediment size from .49mm to 4.94mm and regimes from ripple to plane bed this function was found to be uniquely related. Where, τ_*' dimensionless shear stress for bed roughness.
15. Engel and Lau (1981, after Hossain, 1984)	$g_B = K\gamma_s(1-P)\bar{\xi}V_b$	The formula is based on bed form motion, where $\bar{\xi}$ is the average departure of the bed elevations about the average of all the elevations. For sand size 0.62, 1.20 and 2.60mm it varies from 0.0147 to 0.0207m with an average value of 0.0179m. K=1.32 for dunes having an average value of height-length ratio of 0.06.
16. Mantz(1983,after Hossain,1984)	$g_B=6.17*10^{-4}(P_w')^{1.5}/(D\sqrt{d})$ for d = 0.2 to 300mm, D = 0.12 to 12m	The formula is based on stream power theory, where P_w' is the excess stream power = ($P_w - P_{wc}$), P_{wc} is the critical stream power. Good correlation were found when applied to field data.

Table 4.3 Suspended Load Equations

No.	Investigator	Suspended load equation	Remarks
1.	Einstein (1950, after Garde 1985 and Hubbell and Matejka, 1959)	$g_s = 0.01\gamma_s * 11.6U'_* C_{2d}[2.3 \log(30.2D_x/d_{65}) I_1 + I_2]$ in which, $I_1 = 0.216\, \eta_o^{(z-1)}/(1-\eta_o)^z * J_1$, $I_2 = -0.216\, \eta_o^{(z-1)}/(1-\eta_o)^z * J_2$, $\eta_o = 2d/D, z = w_o/KU_*$	The equation was developed by integrating curves of (concentration X velocity), where g_s is the suespended load rate, C_{2d} is the ceoncentration in percentage by volume at an elevation 2d from bottom, x is the correction factor (Figure 4.11), J_1 and J_2 are the integrals can be obtained from Tables 4.4 and 4.5, for for η_o and z.
2.	Velikanov(Waliuzzaman, 1986)	$q_s = 1/(S_s-1) * \{\tau_o \bar{U}^2/w_o - b\bar{U}^4/(g w_o)\}$	The equation was developed on gravitational theory, where q_s is the suspended load discharge rate, b is a coefficient and w_o is fall velocity.
3.	Brooks (1963)	$q_s/(qC_{md}) = T(K\bar{U}/U_*, z)$	The equation was developed by integrating curves of (concentration x velocity), where C_{md} is the middepth concentration, $T(KU/U_*, z)$ is the transport function can be taken from Figure 4.14.
4.	Bagnold(1966, after Waliuzzaman,1986)	$g_s(S_s-1) = 0.01\tau_o \bar{U}^2/w_o$	The equation was developed on stream power approach.
5.	Chang et al(1967,after Waliuzzaman,1986)	$g_s = DC_a(\bar{U}I_1 - 2U_* I_2/K)$	Where I_1 and I_2 are integrals.
6.	Engelund (1970,after Garde, 1985)	$g_s/(\gamma_f q) = 5.10*10^{-5}(U_*/w_o)^4$	The equation was developed by plotting laboratory data to relate water discharge with sediment load.
7.	Mantz(1983, after Hossain, 1984)	$g_s = 1.26*10^{-2}(P'_w)^{1.03} * \bar{U}/w_o$	The formula is based on stream power theory.
8.	HoLtorff(1983,after Garde, 1985)	$g_s \Delta\gamma_s/(\tau_o \bar{U}) = 0.055 \sum i_b (\tau_s/\tau_o)_i (\bar{U}/w_o)_i$	The formula is based on the hypothesis that only a fraction of the total shear stress is responsible for the particle to be in motion. The value of (τ_s/τ_o) may be obtained from Figure 4.15.
9.	Samaga (1984, after Garde, 1985)	$[g_s i_s/(\gamma_s d_i i_b)] * \sqrt{[1/\{(S_s-1) g d_i\}]} = 30[\xi_s \tau_o/(\Delta\gamma_s d_i)]^6$	The formula is based on the consideration of inidividual fractions in a mixture. The value of interference coefficient, ξ_s may be obtained from Figure 4.16 and Tables 4.6 and 4.7, where M is the the Kramer's uniformity coefficient.

TABLE 4.4 Values for the integrals $J_1=\int_{\eta}^{1}\left(\frac{1-y}{y}\right)^{z}dy$ and $J_2=-\int_{\eta}^{1}\left(\frac{1-y}{y}\right)^{z}\log_e(y)dy$ as determined by the Simpson formula

η	$z=0.2$		$z=0.4$		$z=0.6$		$z=0.8$		$z=1.0$		$z=1.2$		$z=1.5$		$z=2.0$	
	J_1	J_2	J_1	J_2	J_1	J_2	J_1	J_2	J_1	J_2	J_1	J_2	J_1	J_2	J_1	J_2
1.000	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
.90	.047736	.003029	.027451	.0017824	.015853	.0010544	.0091963	.0006271	.005361	.000375	.0031405	.0002256	.0014223	.0001063	.000371	.000032
.80	.11823	.014639	.077254	.010061	.051113	.0069698	.034211	.0048617	.023144	.003412	.015808	.0024074	.009065	.0014394	.00372	.000622
.70	.19844	.037872	.14166	.028791	.10287	.022085	.075852	.017071	.056676	.013282	.042832	.010394	.028654	.0072628	.01523	.004076
.60	.28678	.076114	.21975	.062683	.17195	.052142	.13699	.043745	.11083	.036968	.090824	.031440	.068744	.024911	.04501	.01727
.50	.38286	.13380	.31211	.11826	.26079	.10571	.22249	.095397	.19315	.086806	.17014	.079551	.14382	.070593	.11370	.05927
.40	.48700	.21733	.42062	.20545	.37391	.19678	.34048	.19058	.31630	.18633	.29873	.18367	.28118	.18159	.26742	.18462
.30	.60027	.33628	.54901	.34122	.51953	.35108	.50574	.36606	.50399	.38603	.51204	.41108	.53992	.45829	.62537	.56917
.20	.72505	.51118	.70487	.55950	.71441	.62474	.74963	.70947	.80955	.81741	.89522	.95351	1.0791	1.2246	1.5811	1.9350
.16	.77925	.60429	.77833	.68576	.81399	.79601	.88466	.94185	.99269	1.1328	1.1437	1.3816	1.4719	1.9020	2.4248	3.3921
.12	.83681	.71775	.86118	.84921	.93330	1.0316	1.0565	1.2815	1.2404	1.6226	1.5008	2.0884	2.0905	3.1279	3.9728	6.4656
.100	.86720	.78490	.90737	.95129	1.0035	1.1868	1.1633	1.5175	1.4027	1.9817	1.7476	2.6346	2.5535	4.1528	5.2948	9.3937
.090	.88290	.82186	.93201	1.0093	1.0422	1.2779	1.2240	1.6606	1.4981	2.2063	1.8974	2.9873	2.8481	4.8469	6.2052	11.539
.080	.89898	.86153	.95787	1.0731	1.0838	1.3806	1.2910	1.8258	1.6059	2.4722	2.0708	3.4152	3.2022	5.7205	7.3685	14.410
.070	.91551	.90438	.98520	1.1440	1.1290	1.4977	1.3657	2.0177	1.7294	2.7925	2.2751	3.9448	3.6366	6.8473	8.8972	18.376
.060	.93256	.95100	1.0143	1.2235	1.1786	1.6333	1.4502	2.2490	1.8736	3.1869	2.5210	4.6178	4.1844	8.3469	10.980	24.080
.050	.95023	1.0023	1.0455	1.3141	1.2338	1.7935	1.5477	2.5321	2.0459	3.6875	2.8256	5.5028	4.9006	10.428	13.959	32.741
.040	.96866	1.0595	1.0795	1.4196	1.2964	1.9881	1.6633	2.8911	2.2590	4.3499	3.2188	6.7252	5.8863	13.494	18.522	46.942
.030	.98809	1.1247	1.1172	1.5464	1.3698	2.2347	1.8060	3.3710	2.5367	5.2838	3.7592	8.5433	7.3538	18.435	26.290	73.121
.020	1.00893	1.2018	1.1607	1.7073	1.4605	2.5707	1.9954	4.0729	2.9323	6.7514	4.5860	11.614	9.8556	27.735	42.156	132.20
.016	1.0178	1.2376	1.1805	1.7870	1.5047	2.7483	2.0937	4.4686	3.1514	7.6332	5.0743	13.580	11.480	34.276	54.214	180.77
.012	1.0272	1.2777	1.2025	1.8810	1.5562	2.9687	2.2146	4.9858	3.4351	8.8472	5.7402	16.429	13.876	44.537	74.476	267.61
.01000	1.0321	1.2999	1.2146	1.9356	1.5860	3.1031	2.2879	5.3165	3.6155	9.6612	6.1840	18.433	15.590	52.276	90.780	341.25
.0090	1.0347	1.3117	1.2210	1.9655	1.6022	3.1788	2.3291	5.5084	3.7198	10.147	6.4484	19.664	16.657	57.244	101.68	392.04
.0080	1.0372	1.3240	1.2277	1.9975	1.6196	3.2618	2.3742	5.7233	3.8366	10.704	6.7511	21.108	17.919	63.267	115.34	457.18
.0070	1.0399	1.3370	1.2348	2.0320	1.6384	3.3536	2.4240	5.9674	3.9691	11.353	7.1033	22.832	19.446	70.741	132.93	543.32
.0060	1.0426	1.3508	1.2423	2.0697	1.6589	3.4567	2.4800	6.2494	4.1223	12.125	7.5224	24.944	21.343	80.301	156.43	661.79
.0050	1.0455	1.3655	1.2503	2.1114	1.6815	3.5746	2.5441	6.5830	4.3036	13.069	8.0356	27.617	23.787	93.032	189.40	833.56
.0040	1.0484	1.3814	1.2589	2.1583	1.7071	3.7129	2.6194	6.9907	4.5258	14.271	8.6905	31.160	27.103	110.98	238.95	1101.9
.0030	1.0515	1.3990	1.2686	2.2127	1.7369	3.8816	2.7118	7.5141	4.8125	15.895	9.5802	36.202	31.971	138.57	321.71	1571.3
.0020	1.0548	1.4189	1.2796	2.2788	1.7735	4.1014	2.8335	8.2451	5.2170	18.327	10.926	44.300	40.151	187.82	487.57	2570.7
.0016	1.0562	1.4279	1.2846	2.3105	1.7912	4.2137	2.8964	8.6428	5.4398	19.738	11.716	49.293	45.416	221.13	612.12	3359.1
.0012	1.0577	1.4377	1.2901	2.3470	1.8119	4.3497	2.9734	9.1499	5.7271	21.627	12.787	56.347	53.136	271.98	819.88	4728.0
.00100	1.0585	1.4430	1.2932	2.3678	1.8238	4.4310	3.0200	9.4675	5.9092	22.869	13.499	61.199	58.638	309.49	986.18	5862.0
.00090	1.0589	1.4458	1.2948	2.3791	1.8303	4.4763	3.0462	9.6497	6.0145	23.601	13.922	64.147	62.054	332.93	1097.1	6634.1
.00080	1.0593	1.4487	1.2965	2.3911	1.8373	4.5255	3.0748	9.8520	6.1322	24.434	14.406	67.570	66.093	361.50	1235.7	7614.8
.00070	1.0597	1.4517	1.2983	2.4038	1.8448	4.5794	3.1064	10.080	6.2656	25.394	14.969	71.621	70.970	396.60	1414.0	8898.4
.00060	1.0602	1.4549	1.3002	2.4177	1.8530	4.6394	3.1419	10.340	6.4196	26.525	15.638	76.530	77.021	441.03	1651.8	10645.0
.00050	1.0606	1.4583	1.3022	2.4328	1.8620	4.7073	3.1825	10.645	6.6019	27.893	16.456	82.675	84.808	499.43	1984.8	13146.0
.00040	1.0611	1.4619	1.3044	2.4496	1.8722	4.7860	3.2303	11.013	6.8249	29.614	17.499	90.721	95.359	580.83	2484.4	17001.0
.00030	1.0616	1.4658	1.3068	2.4689	1.8841	4.8807	3.2887	11.478	7.1125	31.905	18.915	102.00	110.82	704.16	3317.1	23642.0

TABLE 4.4 *Continued*

η	$z=0.2$ J_1	$z=0.2$ J_2	$z=0.4$ J_1	$z=0.4$ J_2	$z=0.6$ J_1	$z=0.6$ J_2	$z=0.8$ J_1	$z=0.8$ J_2	$z=1.0$ J_1	$z=1.0$ J_2	$z=1.2$ J_1	$z=1.2$ J_2	$z=1.5$ J_1	$z=1.5$ J_2	$z=2.0$[1] J_1	$z=2.0$[1] J_2
0.00020	1.0621	1.4702	1.3095	2.4919	1.8987	5.0019	3.3656	12.118	7.5180	35.276	21.054	119.80	136.78	920.55	4983.0	37516.0
.00016	1.0624	1.4721	1.3108	2.5027	1.9057	5.0629	3.4053	12.460	7.7411	37.201	22.307	130.61	153.47	1064.6	6232.5	48303.0
.00012	1.0626	1.4742	1.3122	2.5151	1.9140	5.1361	3.4539	12.893	8.0288	39.757	24.005	145.72	177.93	1282.0	8315.3	66821.0
.00010	1.0627	1.4753	1.3130	2.5221	1.9187	5.1794	3.4833	13.161	8.2111	41.420	25.135	156.02	195.35	1440.9	9951.6	82024.0
.000090	1.0628	1.4759	1.3134	2.5259	1.9213	5.2034	3.4998	13.314	8.3165	42.396	25.806	162.24	206.17	1541.1	11093.0	92312.0
.000080	1.0628	1.4765	1.3138	2.5299	1.9241	5.2294	3.5179	13.484	8.4342	43.500	26.574	169.44	218.95	1661.0	12481.0	105332.0
.000070	1.0629	1.4772	1.3142	2.5341	1.9271	5.2578	3.5379	13.673	8.5678	44.769	27.467	177.93	234.39	1807.7	14267.0	122296.0
.000060	1.0630	1.4778	1.3147	2.5387	1.9303	5.2892	3.5603	13.889	8.7219	46.255	28.528	188.16	253.54	1992.4	16647.0	145261.0
.000050	1.0630	1.4785	1.3152	2.5436	1.9339	5.3245	3.5859	14.141	8.9042	48.044	29.825	200.89	278.19	2234.2	19980.0	177974.0
.000040	1.0631	1.4793	1.3158	2.5491	1.9380	5.3652	3.6160	14.442	9.1274	50.279	31.479	217.45	311.57	2568.6	24980.0	228000.0
.000030	1.0632	1.4801	1.3164	2.5554	1.9427	5.4138	3.6529	14.821	9.4151	53.233	33.723	240.51	360.49	3071.3	33313.0	313709.0
.000020	1.0633	1.4809	1.3171	2.5627	1.9485	5.4754	3.7014	15.336	9.8206	57.539	37.115	276.53	442.60	3943.2	49978.0	490870.0
.000016	1.0633	1.4813	1.3174	2.5662	1.9514	5.5062	3.7265	15.610	10.044	59.979	39.101	298.24	495.39	4520.8	62478.0	627560.0
.000012	1.0633	1.4817	1.3177	2.5701	1.9546	5.5429	3.7572	15.954	10.332	63.197	41.797	328.40	572.75	5386.5	83311.0	860760.0
.000010	1.0634	1.4820	1.3179	2.5723	1.9565	5.5645	3.7758	16.166	10.514	65.279	43.588	348.86	627.85	6016.0	99977.0	1051160.0

[1] Integrals calculated in closed form.

TABLE 4.5 *Additional integral values calculated in closed form*

η	$\int_\eta^1 \left(\frac{1-y}{y}\right)^z dy$ $z=0$	3.0	4.0	5.0	$\int_\eta^1 \log_e(y)\left(\frac{1-y}{y}\right) dy$ 0	3.0	4.0	5.0
1.0	0	0	0	0	0	0	0	0
.1	.90000	$.2851\cdot10^{2}$	$.1758\cdot10^{3}$	$.1237\cdot10^{4}$	.66974	$.5560\cdot10^{2}$	$.3632\cdot10^{3}$	$.2602\cdot10^{4}$
.01	.99000	$.4715\cdot10^{4}$	$.3136\cdot10^{6}$	$.2338\cdot10^{8}$	.94395	$1.948\cdot10^{4}$	$1.343\cdot10^{6}$	$1.0198\cdot10^{8}$
.001	.99900	$.4970\cdot10^{6}$	$.3313\cdot10^{9}$	$.2483\cdot10^{12}$	.99209	$3.187\cdot10^{6}$	$2.177\cdot10^{9}$	$1.6535\cdot10^{12}$
.0001	.99990	$.4997\cdot10^{8}$	$.3331\cdot10^{12}$	$.2498\cdot10^{16}$	.99898	$4.353\cdot10^{8}$	$2.955\cdot10^{12}$	$2.239\cdot10^{16}$
.00001	.99999	$.5000\cdot10^{10}$	$.3333\cdot10^{15}$	$.2500\cdot10^{20}$	.99987	$5.508\cdot10^{10}$	$3.723\cdot10^{15}$	$2.816\cdot10^{20}$

Table 4.6 Variation of K_s with τ_o/τ_{oc} (Samaga, 1984)

τ_o/τ_{oc}	<2	3	4	5	7	9	10	11	14	>17
K_s	2.2	2.1	1.9	1.8	1.65	1.5	1.35	1.25	1.1	1.0

Table 4.7 Variation of L_s with M (Samaga, 1984)

M	<0.2	0.25	0.3	0.4	>0.5
L_s	0.8	0.86	0.90	0.97	1.0

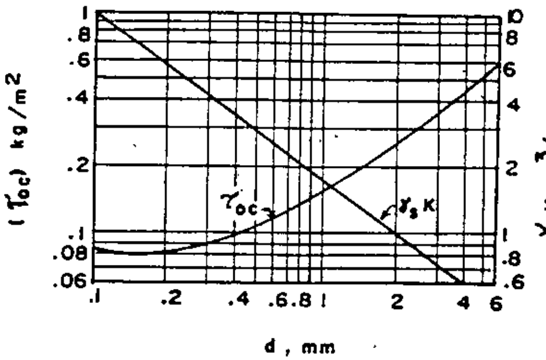


Fig. 4.9 SEDIMENT COEFFICIENT AND CRITICAL TRACTIVE FORCE FOR DUBOYS' BED-LOAD EQUATION (AFTER STRAUB, 1935)

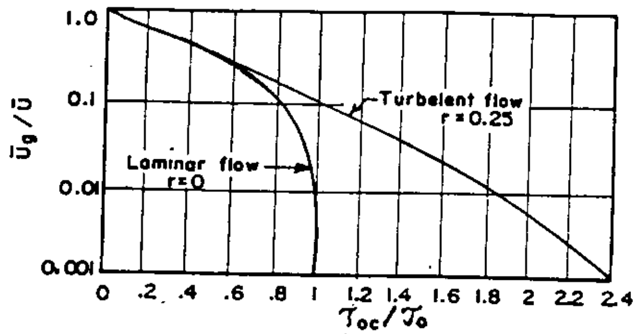


Fig. 4.10 VARIATION OF $\bar{U}_g/\bar{U}$ with τ_{oc}/τ_o (AFTER KALINSKE, 1947)

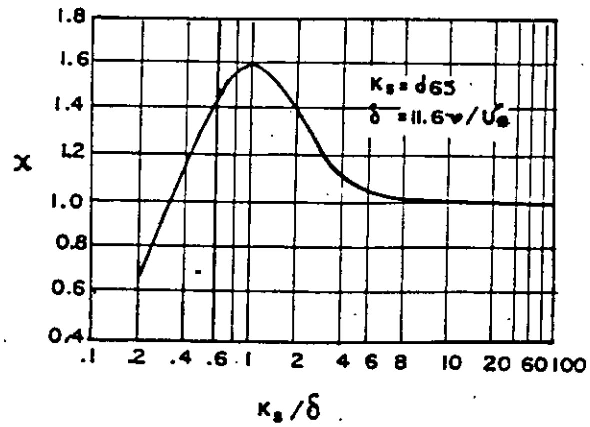


Fig. 4.11 FACTOR X (AFTER EINSTEIN, 1950)

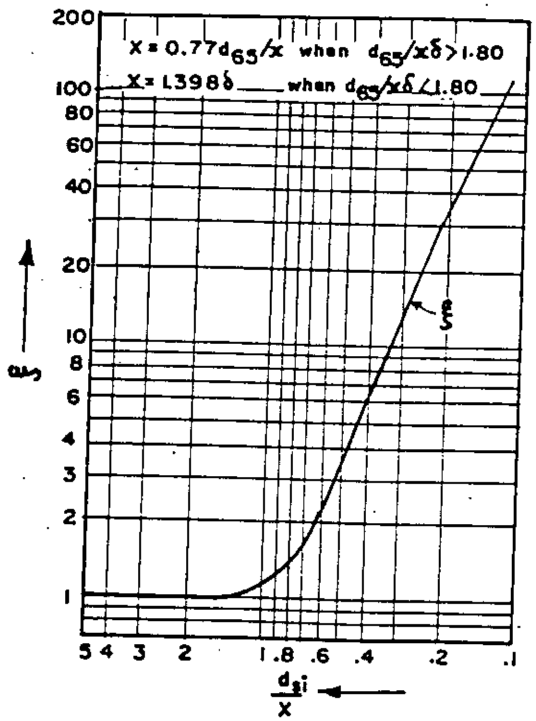


Fig. 4.12 FACTOR ξ (AFTER EINSTEIN, 1950)

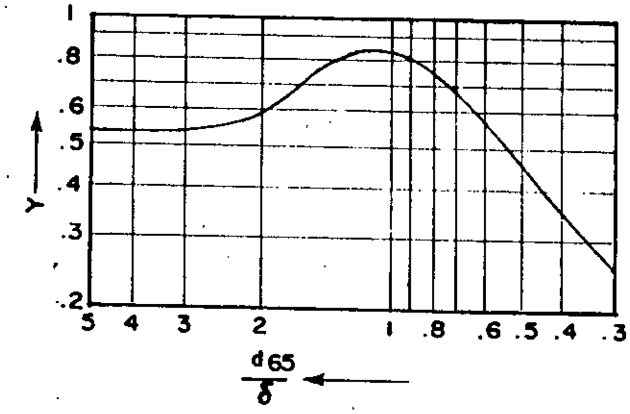


Fig. 4.13 FACTOR Y (AFTER EINSTEIN, 1950)

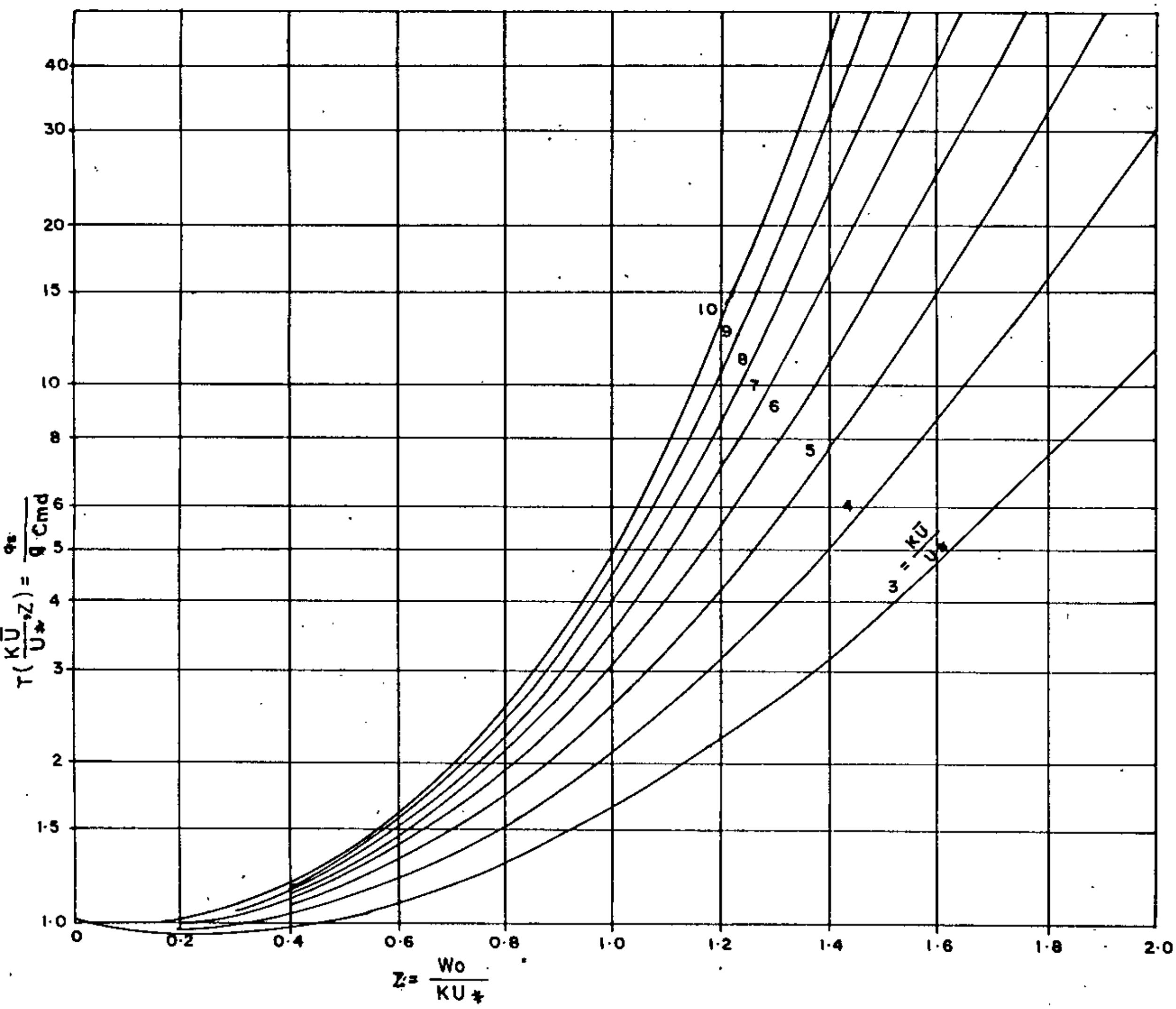


Fig: 4.14 INTEGRATION CURVES FOR SUSPENDED LOAD DISCHARGE (AFTER BROOKS, 1963)

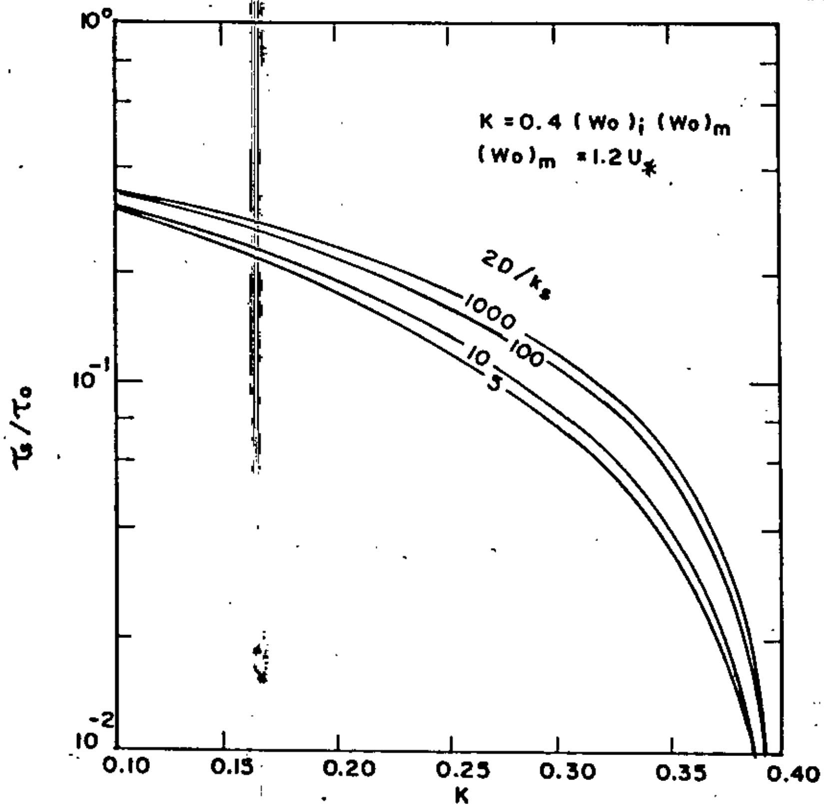


Fig. 4.15 VARIATION OF τ_s/τ_0 WITH $2D/k_s$ AND K
(AFTER HOLTORFF, 1983)

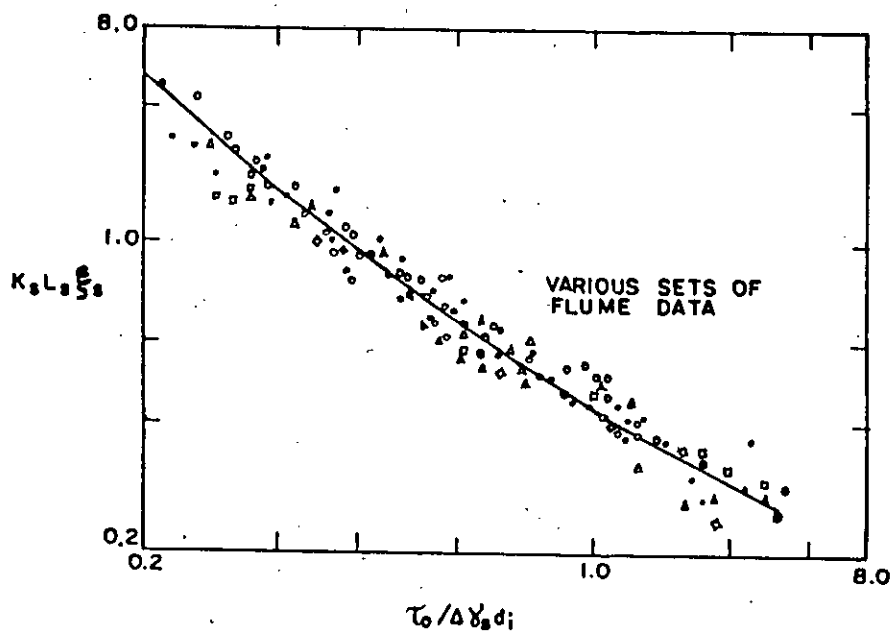


Fig. 4.16 VARIATION OF $K_s L_s \xi_s$ WITH $\tau_0/\Delta\gamma_s d_i$
(AFTER SAMAGA, 1984)

CHAPTER V

DEVELOPMENT OF CRITERIA FOR DESIGN OF SEDIMENT EXCLUDER

Most of the design methods of sediment excluder are mainly based on hydraulic and hydrologic data of the channel. Recently Garde and Pande (1976) developed a method for the Narora Headworks in India which includes not only the hydraulic data but also the sediment data of the channel. It is observed after critically reviewing the performances of the existing excluders that there is further scope to modify the existing procedure. The detailed design procedure is shown in Appendix-1 while the essential criteria needed to be fixed up before design of an excluder are given below.

5.1 POND LEVEL

Pond level is the raised water level maintained at the upstream of the barrage to facilitate supply through the main canal. For better performance of the excluder tunnel and to supply irrigation water for a long distance through the main canal, pond level should be set at a higher level.

In Punjab, Pakistan (PIPD,1978) normally the pond level is fixed as the downsream water level at maximum design discharge or a little below it for the case of nonconservance of water in the upstream of barrage. In the case of Kosi Barrage, India (CBIP, 1981) it has been maintained slightly lower than the downstream retrogressed water level for maximum design discharge of barrage. In Teesta Barrage, BWDB has fixed the pond level at a level 1.85m below the downstream water level for maximum design discharge of barrage.

In the case of storage type barrages the pond level is maintained higher than the downstream maximum design discharge level. For example in the barrages over river Indus, Pakistan (PIPD,1978) such as Kalabagh Barrage, Chasma Barrage and Taunsa Barrage the pond levels have been maintained in between 1 foot to 9 feet above the downstream designed flood level.

Hence for barrages for raising the water level only, the pond level should be equal to or slightly less than the downstream highest flood level and for the storage type reservoirs this should be higher than the downstream highest flood level.

5.2 UPSTREAM FLOOR LEVEL OF UNDERSLUICE AND OTHER BARRAGE BAYS

To attract a deep current near head regulator, upstream floor level of undersluice is generally kept lower than the other barrage bays. This will facilitate lean period flow also to remain near the head regulator.

In some cases both the levels are maintained same and in many cases the upstream floor level of undersluice has been kept 1m below the upstream floor level of the other barrage bays. Varshney and Gupta (1982) and Garg (1983) have suggested the crest level or upstream floor level of undersluice to be at or slightly above the deepest river bed. But Sehgal (1982) has suggested the winter bed level to be the crest level or the upstream floor level of the undersluice for minimum excavation. In lower Sarda Barrage, India (CBIP,1981) the upstream floor level of undersluice has been maintained as the level for deeper channel and for other barrage bays the upstream floor level has been kept 1m higher, which is also the average bed level of the river at the proposed site. In Teesta Barrage, BWDB has fixed the upstream floor level of undersluice to be 0.67m below the recorded lowest bed level which is also 2.66m below average deeper channel level. For other barrage bays, it has been set 0.55m above the lowest bed level which is also 1.43m below the average deeper channel level.

For sufficient sediment entry into the excluder tunnel, higher velocity in the upstream is necessary (see subsection 5.12). Thus lowering of upstream floor levels to a great extent is harmful in the context of sediment exclusion. For efficient sediment exclusion it may be recommended that the upstream floor level of weir bay should be the average deeper channel level and for undersluice it should be 1m below.

5.3. EXCLUDER DISCHARGE

For the design of sediment excluder, it is necessary to select the design discharge, Q_{EX}. Higher efficiency of the excluder and optimal use of water through main canal may ensure by allowing minimum water as escape discharge through the excluder. During flood season, sufficient discharge can be made available for sediment excluder but it is not advisable to select high design discharge. This is due to the fact that higher discharge causes churning up and heavy turbulence in the upstream of pocket which ultimately reduces the efficiency of the excluder.

Experiments conducted by Joglekar (1959, after Vanini, 1977), had shown that about 15% to 20% of the canal discharge should pass through excluder for best performance.In the past works in India the excluder discharge was in the range of 15% to 20% of canal discharge but in recent works it goes to some higher value (Dhillon,1980) such as 22, 25 and 30% for Narora, Farrakka and Kosi Barrage respectively. In Teesta Barrage 88.75% of canal discharge has been assumed to pass through the excluder tunnels which is a marked deviation from the standard practice. Prakash (1962) and Garde and Ranga Raju (1985) have suggested the excluder discharge to be about 30% of canal discharge. Uppal (1951, after Vanoni,1977), has indicated that the efficiency may vary widely with discharge through the tunnels and has advised to have a model study for best efficiency, if time and funds permit.

It is recommended that the excluder discharge should be equal to or in the neighbourhood of 30% of canal discharge.

5.4 TUNNEL DIMENSION

Sediment concentration in the bottom one-third of the water flow contains maximum sediment than in the middle or upper one-third portion (Vanoni,1977). To exclude this dense sediment layer in the downstream in an undisturbed way, tunnels are used. The tunnels are rectangular in cross section throughout the length. The exit section is the controlling section of a tunnel. In order to draw equal discharge through each tunnel the exit section of all the tunnels should be equal. The lengths of all the tunnels are different but the head loss in each tunnel is kept equal by suitably changing the width.

The height of the tunnel, t is generally kept equal to the height of the canal head regulator crest from the crest level of undersluice minus the thickness of the top slab. Depths used on tunnels in India vary greatly but Garde and Ranga Raju (1985) have suggested it to be 1.8m to 3m as a satisfactory range. In Teesta Barrage the BWDB used tunnel depths of 2.287m at entrance and 1.677m at exit. UPIRI (1975) has brought out a design monograph for sediment excluders and ejectors where the exit section was suggested to be 2m by 2m. Varshney and Gupta (1982) suggested the same section to be 2m by 3m for the covenience of maintenance and repair works.

However, the tunnel depth should be such that the blockage in the tunnel is within the permissible limit (see subsection 5.8). In order to have the blockage in the permissible limit, tunnel depth may come out to be very small and maintenance work may be disturbed. Hence the minimum tunnel depth of one man height may be recommended for the excluder. Once the depth of the tunnel, excluder velocity (see subsection 5.7) and excluder discharge are known, total water way, B_{EX} required for the excluder can be easily calculated. This width is divided into number of tunnels by taking suitable width of each tunnel.

For the length of the tunnel, L the tunnel closest to the head regulator should start from some distance upstream of the head regulator and extend upto the crest of undersluice. The other tunnels may be of varying lengths and can be chosen after selecting staggering of the tunnels (see subsection 5.5). Varshney and Gupta (1982) have suggested that the radius of the bend to be 10 to 15 times the tunnel width, if there exist any bend in the tunnel.

5.5 STAGGERING OF EXCLUDER TUNNEL

The opening system of excluder tunnels at inlet is the staggering of excluder tunnels. Staggering of excluder tunnels can be of

a) Khanki type (Figure 2.8)

b) Trimmu type (Figure 2.9)

and c) CWPC type (Figure 2.10)

In Teesta Barrage, BWDB has used Khanki type staggering. In subsections 2.9 and 3.9, it has been shown that by using CWPC type staggering, turbulence at the entrance of the tunnel can be confined to a narrow region and can work satisfactorily under oblique flow condition also. Hence CWPC type staggering may be recommended for sediment excluder design.

5.6 ENTRANCE OF TUNNEL

The entrance to the tunnels should be such that the head loss due to entry is reduced and the zone of influence (area influenced by each tunnel from where sediment may enter) can be

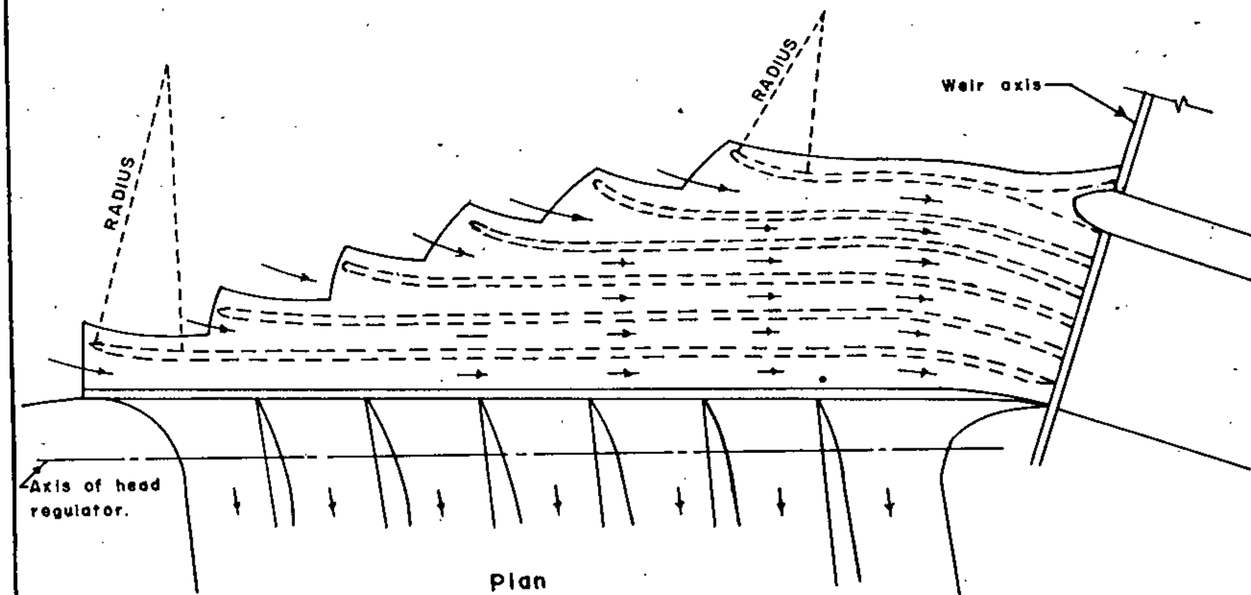
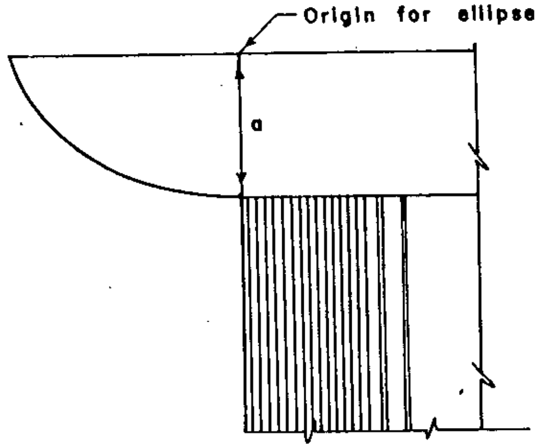


Fig: 5.1 SEDIMENT EXCLUDER AT NARORA HEADWORKS
(AFTER GRADE - PANDE, 1976)



$$\frac{X^2}{4a^2}+\frac{Y^2}{a^2}=1$$

Fig: 5.2 PROJECTED TOP SLAB OF TUNNELS
(AFTER GARG, 1983)

increased. In order to avoid disturbance in the flow due to partition walls of the tunnels, the top slab is generally projected beyond the tunnels by 0.5m to 0.6m and the bottom surface of the projected slab is made elliptical (Figure 5.2, Sharma and Asthana,1975 and CBIP,1966). To increase the zone of influence, bell mouthing (Figure 5.1) of the entrance is necessary. The partition walls between the tunnels at entry should also be elliptical. The radius of the bell mouthing varies from 2 to 6 times the tunnel width (Garg,1983). The radius increases gradually from the tunnel away from the head regulator to the tunnel closer to the head regulator i.e., smaller radius for smaller tunnel and larger radius for larger tunnel.

5.7 EXCLUDER VELOCITY

Excluder velocity, U_{EX} through the tunnel should be carefully calculated for transporting sediment water mixture through the tunnel. Sediment moving with water through the excluder tunnel is two-phase flow and is dependent on the characteristics of flow, liquid, slid and the conduit.

At high velocity the particles move by rolling, sliding or saltation and a few particles may go into suspension and the phenomenon is termed as regime with a movable bed. A further increase in velocity may result all the transported sediment in suspension and this is termed as heterogeneous regime. In the heterogeneous regime sediment concentration is varying over the depth and large concentration gradient is produced. The heterogeneous regime occurs in the case of fine sediment and at high velocity where all the sediment is transported in suspension. In homogeneous regime the sediment concentration over the depth is constant i.e., zero concentration gradient. Thus for movement of all particles of larger sizes in suspension, a demarcation line of velocity between the heterogeneous regime and the regime with the movable bed is necessary. The term limit deposit velocity, U_L was introduced by Durand (1953) as the passage from the regime of the movable bed to the heterogeneous regime. Durand and Condolios (1952) proposed relation for limit deposit velocity for nonuniform sediment (Figure 5.3). Nonuniformity of sediment is the actual condition in the excluder tunnel. Hence the relation given by Durand and Condolios (1952) as shown below may be chosen as the relation for limit deposit velocity.

$$U_L = F_L \sqrt{[8gR_{EX}(S_s-1)]} \qquad 5.1$$

where R_{EX} is the hydraulic radius of excluder and F_L depends on particle size and sediment concentration. For sediment size greater than 0.5mm, F_L varies between 0.8 to 1.0 and can be taken as unity.

If the velocity in the excluder is greater than the limit deposit velocity there will be no blockage in the tunnel. However, in paractice it is not possible to have excluder velicity equal to limit deposit velocity as it is quite large. So naturally a lower limit of excluder velocity is arising. For lower limit it should be taken in consideration that the maximum

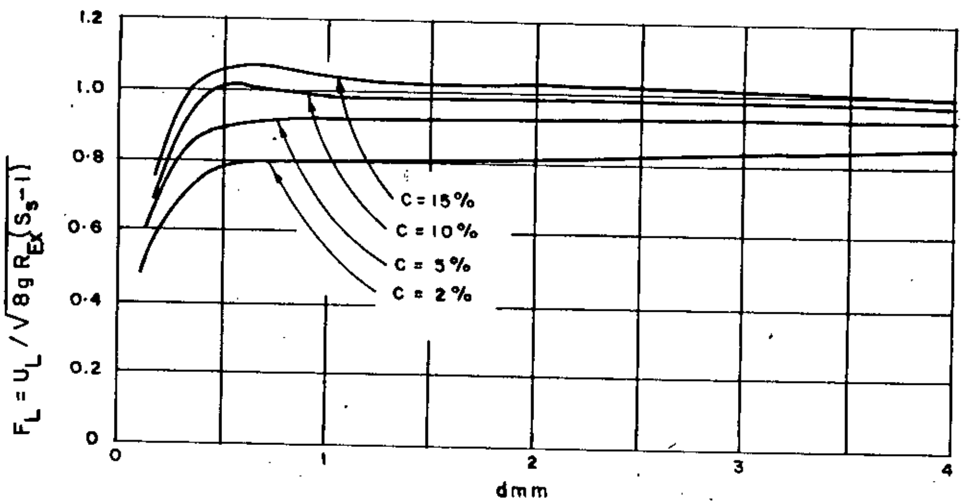


Fig:5.3 LIMIT DEPOSIT VELOCITY FOR NONUNIFORM SEDIMENT, DURAND & CONDOLIOS (1952).

size of the sediment is needed to be in motion. Garde (1970, after Garde and Pande,1976), has analysed the available data and suggested the critical velocity as

$$U_c = 1.6(R_{EX}/d)^{1/8} * \sqrt{(\Delta\gamma_s d/\rho_f)} \qquad 5.2$$

where U_c is the critical velocity, $\Delta\gamma_s$ is the difference between the specific weight of sediment and water and d is the largest sediment size that can move by the shear stress developed by the pond level.

Any velocity less than limit deposit velocity if exist in the excluder tunnel, blockage in the tunnel may take place. In such a position, flow will occur through the free area available above the blockage. Thus another velocity named excluder velocity based on the free flow area available, U_{FEX} will occur and was postulated by Gibert (1960, after Vanoni,1977), as

$$U_{FEX}/\sqrt{R_{FEX}} = U_L/\sqrt{R_{EX}} \qquad 5.3$$

where U_{FEX} and R_{FEX} are the excluder velocity and hydraulic radius respectively based on the free flow area available and are given as

$$U_{FEX} = Q_{EX}/(B_{EX}*D_{FEX}) \qquad 5.4$$

$$\text{and } R_{FEX} = B_{EX}*D_{FEX}/[2(B_{EX}+D_{FEX})] \qquad 5.5$$

where B_{EX} is the clear width of excluder and D_{FEX} is the depth of flow based on free flow area available.

There are numerous works about excluder velocity. After conducting research in the Punjab Irrigation Research Institute, India (Joglekar,1959) it has been suggested that the excluder velocity of 3mps to be maintained for efficient functioning. To prevent choking in the excluder tunnel Garde and Ranga Raju (1985) and Vanoni (1977) also suggested the excluder velocity to be 3mps. The UPIRI (1975) recommends excluder velocity between 2 and 2.5mps in alluvial reach and 3-4mps in shingle reach is adequate. The lower Sarda Barrage in India was located in the alluvial stage of the river where the flushing velocity was considered as 2.5mps and the Nangal Sediment Exluder was designed as shingle excluder where the flushing velocity of about 6.43mps was considered (CBIP, 1966). The excluder velocity is considered as 4.23mps (considering sandy bed with pebbles), for the design of sediment excluder in Teesta Barrage of Bangladesh.

It is recommended that the chosen excluder velocity will be greater than the critical velocity and the developed excluder velocity based on the free flow area available should be in the neighbourhood of limit deposit velocity. For design purpose, a chosen velocity of 2 to 2.5mps for alluvial reach, 2.5 to 3mps for sandy reach and 3 to 4mps shingle reach may be adequate.

5.8 TUNNEL BLOCKAGE

As described in subsection 5.7 that for any velocity less than limit deposit velocity blockage in the tunnel will take place. Flow will occur through the free flow depth available,

D_{FEX}. Blockage in the tunnel can be defined as the difference between the original depth of tunnel, t and the free flow depth available, D_{FEX}. After knowing the blockage depth, percentage blockage can be known. For efficient functioning, this blockage percentage should not exceed 30% to avoid permanent blockage. For Teesta Barrage the blockage of 44.6% has been provided. The blockage in Narora Headworks in India is about 52% (after Garde and Pande,1976).

By keeping the tunnel depth of one man height and the excluder discharge of 30% of canal discharge it may not be possible to maintain the tunnel blockage within 30%. Hence higher blockage percentage may be allowed.

5.9 WIDTH OF EXCLUDER AND CLEAR WATERWAY

Width of excluder is the total width of the excluder tunnels plus the partition wall thickness at exit (over the axis of barrage). The total width of excluder tunnels at exit is the clear waterway for excluder, B_{EX}.

Usually the excluder width covers about two bays of the undersluices but it may sometimes cover the entire width of the undersluices (Grade and Ranga Raju ,1985). UPIRI (1975), mentioned 1 to 4 bays of undersluices as excluder width for efficient functioning. It has been also suggested to have a model study for selecting excluder width to avoid exceesive sediment entry in the pocket. Varshney and Gupta (1982) after model as well as the prototype observation have shown that the excluder tunnels should cover the minimum width of undersluice pocket consistent with the requirement to cover the width of the approaching flow for feeding the canal.

In Teesta Barrage, BWDB has supplied 3 undersluice bays as the width of the excluder and the clear waterway of excluder is of 28.354m. For sediment excluder in Lower Chenab Canal at Khanki, excluder width covers 2 undersluice bay while in Trimmu Barrage it covers the entire width of the undersluice pocket i.e., it covers 4 undersluice bays (Dhillon,1980). In lower Sarda Barrage it covers 2 undersluice bays and has found to be adequate for effective sediment exclusion (varshney and Gupta,1982).

For recommendation, the clear waterway for sediment excluder at exit, B_{EX} may be equal to the width of one undersluice bay. By choosing suitable width of tunnel, total number of tunnels may be found out. This total number of tunnels should be accommodated in 2 number of undersluice bays.

5.10 HEAD LOSS IN TUNNEL AND OPERATING HEAD

a) Head Loss in Tunnel

Losses of head in the tunnel for clear water flow is the head loss in the tunnel, h_o. This is the summation of all the losses i.e., loss due to friction, entry loss, bend loss, transition loss and exit loss. The tunnels are of different lengths but the sizes of the tunnels should be so fixed that the losses in every tunnel should be same in order to have constant head throughout the width of the excluder in the downstream.

i) Frictional loss

Frictional loss in tunnel can be written from the Manning's equation (Brater and King,1976), and is

$$h_f = \bar{U}_{Ex}^2 n^2 L / R_{Ex}^{4/3} \tag{5.6}$$

where h_f is the head loss due to friction, $\bar{U}_{Ex}$ is the mean velocity in the tunnel, L is the length of tunnel, R_{Ex} is the hydraulic radius of the tunnel and n is the coefficient of reoughness of the tunnel surface.

ii) Loss due to bend

Loss due to bend of the tunnel can be written from Weisbach's formula of rectangular section and is

$$h_b = F U_{Ex}^2/(2g)*\theta/180 \tag{5.7}$$

where h_b is the head loss due to bend, g is the acceleration due to gravity, θ is the angle of deviation and $F=0.124+3.104*[W/(2r)]^{1/2}$, where W is the width of tunnel and r is the radius of bend along the centre line of tunnel.

iii) Transition loss in contraction

Loss of head in transition for contraction can be written as:

$$h_c = 0.1[U_{Ex2}^2/(2g) - U_{Ex1}^2/(2g)] \tag{.5.8}$$

where h_c is the head loss due to contraction, U_{Ex2} and U_{Ex1} are the velocity in the tunnel at smaller and larger section respectively.

iv) Entry and Exit loss

Loss due to entry and exit can be written as follows where the coefficients are taken from Straub and Morris (1950, after Brater and King, 1976):

$$h_{en} = 0.1\bar{U}_{EX}^2/(2g) \qquad 5.9$$

and $$h_{ex} = 1.0\bar{U}_{EX}^2/(2g) \qquad 5.10$$

where h_{en} is the head loss due to entry, h_{ex} is the head loss due to exit and $\bar{U}_{EX}$ is the mean velocity in the tunnel.

Thus clear water head loss in tunnel, h_o can be written as:

$$h_o = h_f + h_b + h_c + h_{en} + h_{ex} \qquad 5.11$$

Head loss at various headworks in India varies from 0.6m to 1.2m (Sharma and Asthana,1975) while for Teesta Barrage the average clear water head loss is 1.314m in the tunnels.

b) Operating Head

The difference between the upstream and downstream water level of a barrage is the operating head, h for sediment excluder. For efficient functioning of the excluder by better flushing, it is preferable to provide more head to pass extra discharge through the tunnels. To ensure reasonably satisfactory efficiency, free jump formation in the downstream of the undersluice bays is necessary even during high floods. But it is not always possible to maintain free jump formation in the downstream of barrage. The alluvial river flowing through flat topography generally develops submerged flow condition. Maintaining the pond level for maximum designed discharge of the barrage, operating head will be maximum for lower discharges and decreases gradually as the discharge increases.

UPIRI (1975) has suggested the operating head of about 0.6m to 1.2m as sufficient for flood discharge. However, Grade and Ranga Raju (1985) have suggested a minimum head of 0.9m to 1.2m necessary for the operation of tunnel. But it should be taken into consideration that the minimum operating head should be greater than the clear water head loss in the tunnel.

In the Teesta Barrage available operating head for every discharges goes below clear water head loss in the tunnel(Table 7.5). So there is likelihood that the excluder tunnels will be choked up with sediment.

Considering the above fact it is reommended that the minimum operating head should be greater than the clear water head loss in the tunnel and in the range of 0.9m to 1.2m.

5.11 BARRAGE REGULATION

It has already been discussed that for higher discharges semi-still pond regulation is suitable. Thus semi-still pond regulation may be considered as a criteria of barrage regulation for the design of sediment excluder.

For semi-still pond regulation, pocket discharge for different discharges of the river can be obtained from the following relations:

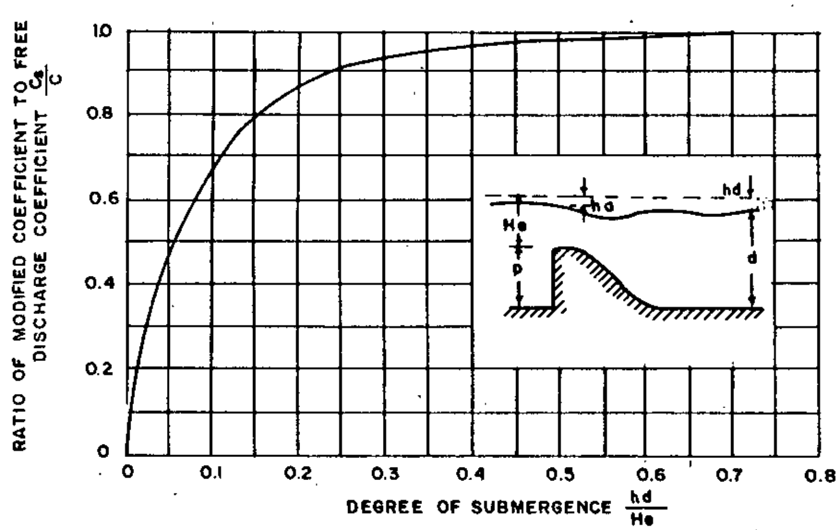


Fig: 5.4 <u>RATIO OF DISCHARGE COEFFICIENTS DUE TO TAILWATER EFFECT</u>

(AFTER VARSHNEY & GUPTA, 1982)

$$Q_P = Q_{EX} + Q_C \quad \text{for } Q_R - (Q_{EX} + Q_C) \leqslant Q_W \qquad 5.12$$

$$Q_P = Q_U \quad \text{for } Q_R - (Q_{EX} + Q_C) > Q_W \qquad 5.13$$

where Q_R, Q_C, Q_{EX}, Q_W, Q_U and Q_P represents the discharges of the river at the upstream of the barrage (high discharge is assumed to flow, see subsection 5.12), canal discharge, excluder discharge, weir portion discharge of barrage, undersluice portion discharge of barrage and the pocket discharge respectively. It is important to note here that the discharge coefficients for free fall condition can be taken as 1.84 (in metric unit) for sharp crested weir (the width of crest at top is less than 0.667 H_e, where H_e is the head over crest) and 1.705 for broad crested weir (the width of crest at top is greater than 2.5 H_e). On the other hand if submerged flow takes place in the barrage, the coefficients should be reduced by a factor C_s/C, shown in Figure 5.4. In order to calculate pocket discharge, Q_P from Equation 5.12 or 5.13, it should be taken in consideration that for below pond level discharge at the upstream of barrage, Q_W and Q_U will be for pond level discharge and for above pond level discharge at the upstream of barrage, Q_W and Q_U will be for respective discharges.

5.12 ENTRAINMENT OF SEDIMENT DISCHARGE INTO TUNNEL

Entrainment of bedload discharge and suspended load discharge into the tunnel should be considered for the design of sediment excluder. Rivers carry maximum load at high flood discharge but this need not be considered for the design of sediment excluder. This is due to the fact that canals are generally closed for high flood discharge to avoid excessive sediment entry into the canal. Selection of water discharge, Q_R is vital for quantitative entrainment of sediment discharge into the excluder tunnel. For free fall condition in the barrage, high average velocity $\bar{U}$ is developed at the upstream of the barrage and consequently higher percentage of sediment discharge enters into the excluder tunnel. In such a case water discharge, Q_R may be the dominant discharge. On the other hand if submerged flow occurs in the barrage, low average velocity $\bar{U}$ is developed at the upstream of the barrage and thus lower percentage of sediment discharge enters into the excluder tunnel. For submerged flow condition with a reasonable entrainment of sediment discharge into the excluder tunnel, the water discharge Q_R should be such that minimum operating head (for which exclusion through excluder may be possible) is available.

a) Entrainment of Bedload Discharge into Tunnel

The entrainment of bedload discharge into the tunnel depends on the quantity of sediment in the pocket between the divide wall and the canal head regulator. Bedload discharge coming into the pocket is a part of the bedload discharge moving through the entire river bed. The division of the bedload discharge depends upon the discharge through the pocket, weir portion discharge, curvature of flow and size and shape of divide wall. It is reasonable to assume that the quantity of sediment in the pocket is proportional to the discharge entering into the undersluice pocket. The calculation of bedload discharge entering into the pocket can be obtained by the following procedure:

Let B_R and B_{RE} represent the width of the river at barrage and the width of the river contributing discharge into the undersluice pocket respectively; Q_R and Q_P represent river and pocket discharge respectively. The width of the river contributing discharge into the pocket can be written as

$$B_{RE} = (Q_P/Q_R)*B_R \qquad 5.14$$

The initial bedload discharge into the pocket is

$$Q_{BPI} = (Q_B/B_R)*B_{RE} \qquad 5.15$$

where Q_{BPI} is the initial entrainment of bedload discharge into the pocket, Q_B is the bedload discharge through the entire river bed and all other terms as described earlier.

If velocity is not sufficient to move the sediment particles at the mouth of the pocket, some particles will be deposited. Garde (1970, after Garde and Ranga Raju,1985), has analyzed available data and proposed critical velocity requirement for the sediment particle to be in motion as

$$U_{cr}/\sqrt{[(S_s-1)gd]}=0.5\mathrm{Log}(D/d)+1.63 \qquad 5.16$$

where U_{cr} is the critical velocity to start the particle in motion, D is the depth of flow, S_s is the specific gravity of sediment, g is the acceleration due to gravity and d is the particle diameter. Using average velocity as critical velocity in Equation 5.16, maximum diameter of particle in motion can be obtained and all the particles above that size will settle down. The deposited percentage of sediment particles thus is obtained from the grain size distribution curve. The bedload discharge deposited at the upstream of the excluder tunnel, Q_{BPD} can be written as

$$Q_{BPD} = B_{BPI} * (\text{Percentage of bedload discharge deposited}) \tag{5.17}$$

and entrainment of bedload discharge into the pocket, Q_{BP} is

$$Q_{BP} = (Q_{BPI} - Q_{BPD}) \tag{5.18}$$

The bedload discharge which enters the tunnel is

$$Q_{BT} = Q_{BP} \tag{5.19}$$

and moves downstream provided the concentration carrying capacity of the tunnels permit (see subsection 5.13).

For the design of sediment excluder, the assumed river discharge, Q_R is much higher and a higher velocity is developed at the upstream of barrage causing all the particles in motion. That is

$$Q_{BP} = Q_{BPI} \tag{5.20}$$

If the concentration carrying capacity of the tunnels permit, this bedload discharge may enter into the tunnel i.e.,

$$Q_{BT} = Q_{BP} = Q_{BPI} \tag{5.21}$$

and excluded downstream.

b) Entrainment of Suspended Load Discharge into Tunnel

The following equations can be written for the suspended load entrainment similar to that of bedload entrainment:

$$Q_{SPI} = (Q_S/Q_R)*B_{RE} \tag{5.22}$$

$$Q_{SPD} = Q_{SPI}*(\text{Percentage of suspended load discharge deposited}) \tag{5.23}$$

$$Q_{SP} = (Q_{SPI} - Q_{SPD}) \quad \text{for lower upstream velocity} \tag{5.24}$$

and $$Q_{SP} = Q_{SPI} \quad \text{for higher upstream velocity} \tag{5.25}$$

where Q_{SPI} is the initial entrainment of suspended load discharge into the pocket, Q_S is the suspended load discharge throughout the river section, Q_{SPD} is the suspended load discharge deposited at the upstream of the excluder tunnel, Q_{SP} is the entrainment of suspended load discharge into the pocket and all other terms as described earlier.

It is seen in the case of entrainment of bedload discharge into the tunnel that total bedload discharge into the pocket may enter into the tunnel. But suspended load entered into the pocket, Q_{SP} will not entirely enter into the tunnel. From the vertical distribution of the suspended sediment concentration, a denser bottom layer will enter into the tunnel. For quantitative analysis, entrainment of suspended load discharge into the tunnel depends upon the parameters $K\bar{U}/U_*$, z and t/D (Garde and Pande, 1976). The procedure of finding the percentage of suspended sediment entry into the tunnel is given by the method used by Brook (1963).

Brooks (1963) developed total suspended load discharge rate equation as

$$q_s/(qC_{md}) = T(K\bar{U}/U_*, z) = J_1(z, e^{-K\bar{U}/U_*} - 1) + [J_1(z, e^{-K\bar{U}/U_*} - 1) - J_2(z, e^{-K\bar{U}/U_*} - 1)]U_*/(K\bar{U}) \quad 5.26$$

where q_s is the total suspended load discharge rate, q is the water discharge rate, C_{md} is the middepth concentration, T is the transport function, K is the VonKarman's constant, $\bar{U}$ is the average velocity of flow, U_* is the shear velocity, z is the exponent and J_1 and J_2 are the integrals.

Brooks (1963) developed the integration curves (Figure 4.14) by Equation 5.26 for suspended load discharge against the parameter $K\bar{U}/U_*$ upto a value of 3 and z. The curves for the lower range upto $K\bar{U}/U_* = 0.25$ is shown in Figure 5.5. In developing the curves for lower range the values of the integrals are taken from the results published by Einstein (1950) and are shown in Tables 4.4 and 4.5. A sample calculation is shown below:

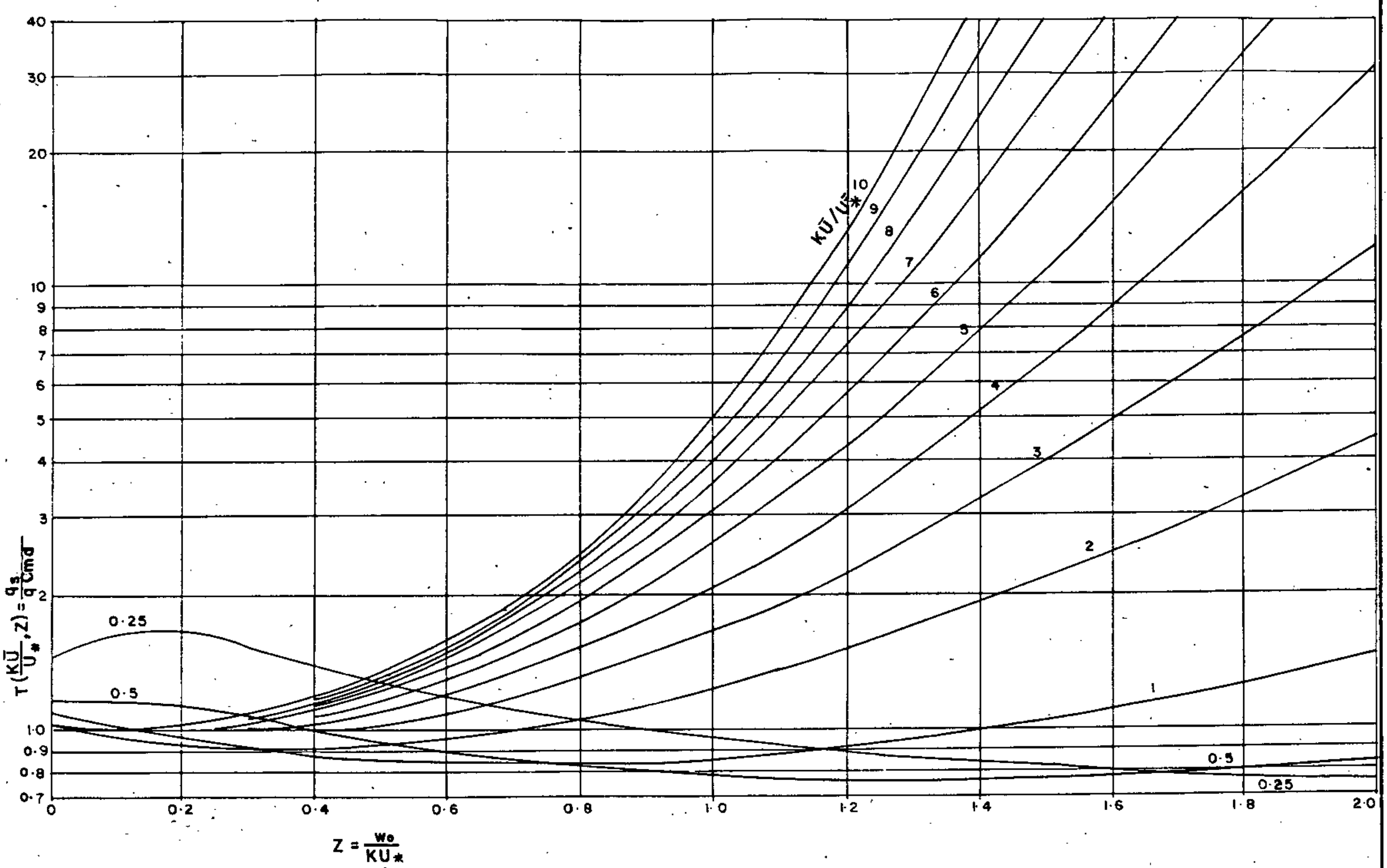


Fig: 5.5 INTEGRATION CURVES FOR SUSPENDED LOAD DISCHARGE.

Taking $K\bar{U}/U_* = 2$ and $z = 1.2$, Equation 5.26 becomes

$$q_s/(qC_{md}) = T(K\bar{U}/U_*, z) = J_1(1.2, e^{-3}) + [J_1(1.2, e^{-3}) - J_2(1.2, e^{-3})]1/2$$

$$= 2.8335 + [2.8335 - 5.5272]1/2$$

$$= 1.4867$$

Suspended load discharge rate above the tunnel depth, t can be written from Brooks (1963) as

$$q_s = C_{md} \int_t^D [(D-y)/y]^z \, [\bar{U} + U_*/K*\{1+Ln(y/D)\}]dy \tag{5.27}$$

Let $\eta = y/D$. For lower limit, $y = t$, $\eta_1 = t/D$; for upper limit, $y = D$, $\eta = 1$ and $dy = Dd\eta$. Putting these values in Equation 5.27, it becomes

$$q_s = C_{md}\,\bar{U}D \int_{\eta_1}^{1} [(1-\eta)/\eta]^z \, [1 + U_*/K\bar{U} *(1+Ln\,\eta)]d\eta$$

or

$$q_s/(qC_{md}) = \int_{\eta_1}^{1} [(1-\eta)/\eta]^z \, d\eta + U_*/K\bar{U} *[\int_{\eta_1}^{1} \{(1-\eta)/\eta\}^z \, d\eta + \int_{\eta_1}^{1} \{(1-\eta)/\eta\}^z \, Ln\,\eta \, d\eta\}] \tag{5.28}$$

Equation 5.28 can be written in terms of transport function, T as

$$q_s/(qC_{md}) = T(K\bar{U}/U_*, z, \eta_1) = J_1(z, \eta_1) + [J_1(z, \eta_1) - J_2(z, \eta_1)]U_*/(K\bar{U}) \tag{5.29}$$

where $\eta_1 = t/D$,

$$J_1(z, \eta_1) = \int_{\eta_1}^{1} [(1-\eta)/\eta]^z \, d\eta \tag{5.30}$$

$$J_2(z, \eta_1) = -\int_{\eta_1}^{1} [(1-\eta)/\eta]^z \, Ln\,\eta \, d\eta \tag{5.31}$$

The values of the integrals J_1 and J_2 can be taken from Tables 4.4 and 4.5 as given by Einstein (1950).

Dividing Equation 5.29 by Equation 5.26, we obtain the percentage of suspended sediment passing above the tunnel depth, t. The ratio in functional form is

$$T(K\bar{U}/U_*, z, \eta_1)/T(K\bar{U}/U_*, z)$$

The percentage of suspended load passing through the tunnel can be wirtten as

$$Q_{ST}/Q_{SP} = [1 - T(K\bar{U}/U_*, z, \eta_1)/T(K\bar{U}/U_*, z)] \qquad 5.32$$

Garde and Pande (1976) have developed four sets of curves for $\eta_1 = t/D = 0.2, 0.3, 0.4$ and 0.5 against the parameter $K\bar{U}/U_*$ upto a value of 3 and z. Three sets of curves are developed by Equation 5.32 for $\eta_1 = 0.2, 0.3$ and 0.4 against the parameter $K\bar{U}/U_*$ upto lower values of 0.25 and z and are shown in Figure 5.6. A sample calculation is shown below:

Taking $\eta_1 = t/D = 0.3$, $K\bar{U}/U_* = 2$ and $z = 1.2$

$T(K\bar{U}/U_*, z) = 1.4867$ (from Figure 5.5)

and $T(K\bar{U}/U_*, z, \eta_1) = 0.51204 + (0.51204 - 0.41108)1/2$

$= 0.56252$ (from Equation 5.29)

Now $Q_{ST}/Q_{SP} = (1 - 0.56252/1.4867) = 62\%$ (from Equation 5.32)

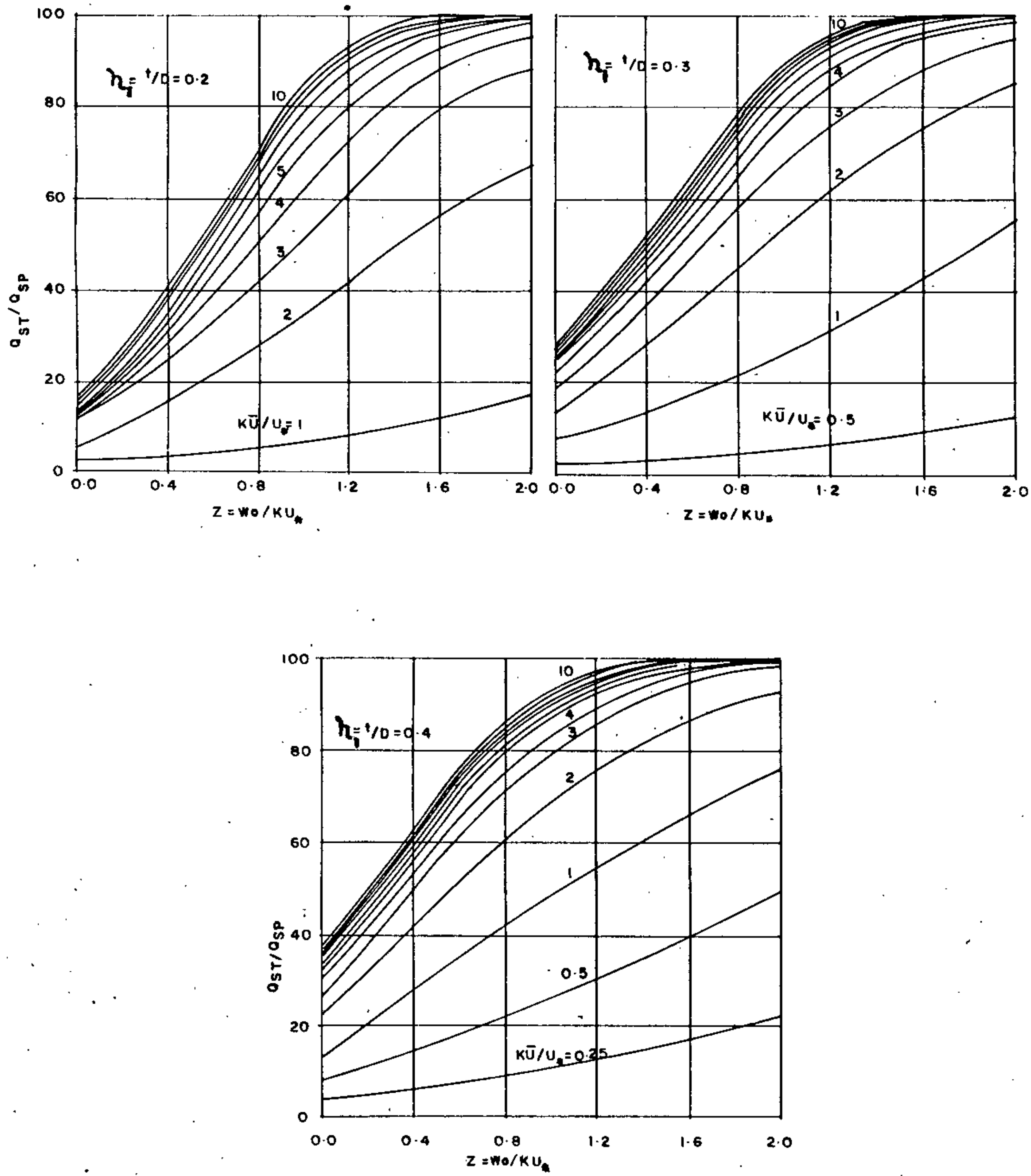


Fig: 5.6 PERCENTAGE OF SUSPENDED LOAD IN THE TUNNEL TO THE SUSPENDED LOAD IN THE POCKET

Suspended load discharge entering into the tunnel is

$$Q_{ST} = Q_{SP} * [1 - T(K\bar{U}/U_*, z, h_1)/T(K\bar{U}/U_*, z)] \qquad 5.33$$

This suspended load discharge may pass through the tunnel and can be excluded downstream provided the sediment concentration carrying capacity of the tunnel permits (see subsection 5.13).

5.13 SEDIMENT CONCENTRATION CARRYING CAPACITY AND EFFICIENT EXCLUSION OF TUNNEL

Durand (1953) and others (Newitt,1955; Zandi and Govatos, 1967; Rose,1969; Charles,1970; Babcock,1970,1971; Heyden,1971; Vocadlo,1972) have given many formulas for friction head loss gradient for solid fluid mixture in pipeline. The variation of results from these formulas may be attributed to the varied sources of laboratory and flume data where experiments were carried out under different conditions. Later Lazarus and Neilson (1978) developed emperical equations for sliding bed and moving dune, heterogeneous regime and pseudo homogeneous regime conditions by dimensional analysis. Change of available head may cause the regimes of flow to change from one to another. Hence general formula developed by Lazarus and Neilson (1978) is duly mentioned below:

$$f_m/f_b = 1+2.1[\exp\{-2(\lambda-0.32)\}]+5.85[\exp\{-12(\lambda-0.32)\}] \qquad 5.34$$

where f_m = mixture friction factor = $2gR_{EX}h/(LU_{EX}^2)$

f_b = friction factor for base curves

$$=10^{\{-\exp[0.835-\{6.3-\log(4U_{EX}R_{EX}/\nu)\}^2/24] +\exp[\{7-\log(4U_{EX}R_{EX}/\nu)\}^2/28-1.6]*S_sC_T -[\{\log(4U_{EX}R_{EX}/\nu)-5.2\}^2/24.5+0.2]*\exp(-2S_sC_T)\}}$$

$$\lambda = [U_{EX}^2/(4gR_{EX})]*\sqrt{[\nu}/(4U_{EX}R_{EX}S_s^3C_T)] *[1000\{d/(4R_{EX})\}^{[0.444\mathrm{Log}(d/(4R\ \ ))+1.31]}]^{\tanh(1+S_sC_T)}$$

C_T=sediment concentration carrying capacity of tunnel (volume without part of voids)=Q_s/Q

d = d_{50} of bed material and all other terms are described earlier.

The various equations developed so far do not explicitly consider the equivalent sandgrain roughness of the boundary into account. Kazanskij (1978) for the first time introduced sandgrain roughness, K_s for the determination of friction head loss gradient for solid-fluid mixture in pipeline and proposed empirical equation. The inaccuracies resulted from the use of some early developed well known methods have also been brought to light by Kazanskij (1978).

$$(J-J_o)/(J_oC_T)=1.58*10^4[K_s/(4R_{FEX})]^{2/3} * (4gR_{FEX}/U_{FEX}^2)*(S_s-1)^{-1}*(w_o/\sqrt{gd}) \qquad 5.35$$

where J is the hydraulic gradient for sediment laden flow (for excluder design it may be assumed to be the available head,h), J_o is the hydraulic gradient for clear water flow (for excluder design it may be assumed to be the clear water head loss in tunnel,h_o), w_o is the fall velocity of d_{50}, K_s the equivalent sand grain roughness (Stricler's equation in metric unit) = $(24n)^6$, n = Manning's roughness coefficient for concrete pipe (USBR,1978) = 0.013 and all other terms are same as described earlier.

Efficient exclusion of the excluder can only be achieved if the sediment concentration carrying capacity of the tunnel, C_T goes higher than the actual concentration, C_{EX} entering into the tunnel. Actual concentration,C_{EX} depends upon the entrainment of bed and suspended load into the tunnel and can be written as

$$C_{EX} = (Q_{BT}+Q_{ST})/Q_{EX} \tag{5.36}$$

CHAPTER VI

DEVELOPMENT OF PREDICTION FUNCTION OF SEDIMENT INTO MAIN CANAL

Prediction of sediment into main canal is the amount of sediment and the percentages of their different grain-size ranges which enter into the main canal through canal head regulator. This can be done either by scale modelling or through analytical method. Analytical method can be applied if field data are available and here an analytical approach is developed.

6.1 ENTRAINMENT OF SEDIMENT DISCHARGE INTO MAIN CANAL

Sediment load in the form of suspension and or bedload may enter into the canal depending on the hydraulic conditions. A large quantity of this load can reduce the canal capacity. On the other hand, if the canal discharge does not contain any sediment load, degradation may take place at the downstream of head regulator, causing the full supply level of main canal to go down. Thus estimation of entrainment of sediment discharge into main canal is important.

a) Entrainment of Bedload Discharge into Main Canal

When available head of the excluder tunnel is less than the clear water head loss in the tunnel, sediment carrying capacity of the tunnel will be zero. In such a case bedload discharge may block the entrance of the excluder tunnel and remains as bedload in the pocket infront of the canal head regulator. This bedload has every possibility of entering into the main canal.

Bedload discharge remains as bedload in the pocket may be written as:

$$Q_{BPB} = (Q_{BPI} - Q_{BPD}) \quad \text{for low upstream velocity} \qquad 6.1$$

$$Q_{BPB} = Q_{BPI} \quad \text{for high upstream velocity} \qquad 6.2$$

Where Q_{BPB}, Q_{BPI} and Q_{BPD} are the bedload discharge remains as bedload in the pocket, initial entrainment of bedload discharge in the pocket and bedload discharge deposited in the pocket respectively.

Bedload discharge remains as bedload in the pocket, Q_{BPB} may not totally enter into the main canal. This is due to the fact that entrainment is dependent upon the pocket discharge, Q_P excluder discharge, Q_{EX} and canal discharge, Q_C. Entrainment of bedload discharge into the main canal can be written as:

$$Q_{BC} = Q_{BPB} * Q_C / (Q_P - Q_{EX}) \qquad 6.3$$

where Q_{BC} is the entrainment of bedload discharge into the main canal.

For different water discharges, Q_{RO} variation may occur in the bedload discharge remains as bedload in the pocket, Q_{BPB} canal discharge, Q_C and pocket discharge, Q_P. To have a general curve for the prediction of entrainment of bedload discharge into the main canal Q_{BC} and Q_{RO} may be plotted on a log-log paper and total entrainment for the whole year may be obtained by summation of the individual 10-day value.

$$Q_{BC} = (\Sigma Q_{BCi})*10*24*60*60 \text{ m}^3 \qquad 6.4$$

Where Q_{BC} is the total entrainment of bedload discharge into the main canal for the whole year and Q_{BCi} is the entrainment of bedload discharge into the main canal for the respective 10-day average 75% dependable water discharge, Q_{RO}.

b) Entrainment of Suspended Load Discharge into Mian Canal

Even after satisfactory working of sediment excluder suspended load may enter into the main canal. This is due to the fact that a portion of the suspended load may enter into the tunnel (as described in subsection 5.12b) and the remaining portion will remain in suspension in the pocket. This suspended load will enter into the main canal with canal flow.

Suspended load discharge remains as suspended load in the pocket may be written as:

$$Q_{SPS} = (Q_{SPI} - Q_{SPD} - Q_{ST}) \quad \text{for low upstream velocity} \qquad 6.5$$

$$Q_{SPS} = (Q_{SPI} - Q_{ST}) \quad \text{for high upstream velocity} \qquad 6.6$$

Where Q_{SPS}, Q_{SPI}, Q_{SPD} and Q_{ST} are the suspended load discharge remains as suspended load in the pocket, initial entrainment of suspended load discharge in the pocket, suspended load discharge deposited in the pocket and entrainment of suspended load discharge into the excluder tunnel respectively.

As described in subsection 6.1a, entrainment of suspended load discharge into the main canal is dependent upon the pocket discharge, Q_P excluder discharge, Q_{EX} and canal discharge, Q_C. Entrainment of suspended load discharge into the main canal, Q_{SC} can thus be written as:

$$Q_{SC} = Q_{SPS} * Q_C / (Q_P - Q_{EX}) \qquad 6.7$$

To have a general curve for the prediction of suspended load entrainment into the main canal Q_{SC} and Q_{RO} may be plotted on a log-log paper (as described in subsection 6.1a) and total entrainment for the whole year may be obtained as:

$$Q_{SC} = (\Sigma Q_{SCi})*10*24*60*60 \text{ m}^3 \qquad 6.8$$

Where Q_{SC} is the total entrainment of suspended load discharge into the main canal for the whole year and Q_{SCi} is the entrainment of suspended load discharge into the main canal for the respective 10-day average 75% dependable water discharge, Q_{RO}.

6.2 ENTRAINMENT OF DIFFERENT GRAIN-SIZE RANGE INTO MAIN CANAL

Entrainment of coarser particles into main canal may cause early settlement closer to the downstream of the canal headworks. The location of sediment ejector (curative measure) tends to be very critical. This is due to the fact that closer location of sediment ejector from the head regulator catches coarser sediment particles while the finer particles in suspension moves downstream of the canal. Thus estimation of entrainment of different grain-size range is important.

a) Entrainment of Different Grain-Size Range of Suspended Load Material

After construction of the barrage, water level in the upstream rises causing reduction of flow velocity (due to ponding effect) and steady flow is assumed to develop in the pocket due to the presence of divide wall. In the post barrage condition, coarser particles fall to the bottom layer and pass through the sediment excluder while finer particles in the top layer enter into the main canal.

For a particular discharge, settlement of minimum diameter of sediment particles may be obtained by the following equation (Garde,1970 after Garde and Ranga Raju,1985).

$$U_{cr}/\sqrt{[(S_s-1)gd]} = 0.5\ \mathrm{Log}(D/d)+1.63 \qquad 5.16$$

Where U_{cr} is the critical velocity, S_s is the specific gravity of sediment, g is the accleration due to gravity, d is the diameter of sediment particle and D is the depth of flow.

By using this equation percentages of materials with maximum grain size in suspension may be available. From this suspended material a percentage will be excluded by the sediment excluder and is given by Equation 5.32 (Figure 5.6).

$$Q_{ST}/Q_{SP} = [1-T(K\bar{U}/U_*, z, \eta_1)/T(K\bar{U}/U_*, z)] \qquad 5.32$$

where Q_{ST}/Q_{SP} is the percentage of suspended load discharge excluded by the tunnel and all other terms are as described earlier.

By deducting the percentage excluded, Q_{ST}/Q_{SP} from the percentage of material in suspension, entrainment of maximum diameter into the main canal can be estimated by using the grain size distribution curve.

For variable discharges, a general curve for the entrainment of maximum grain size of suspended load into the main canal is to be drawn.

For the entrainment of different grain-size range, total sediment diameter range should be divided into several groups. For a particular discharge maximum grain size entrainment from the above said general curve is found out. Knowing this maximum size on the grain size distribution curve, different grain-size range entrainment percentages are obtained. For different discharges and for different grain-size range different general curves are drawn for the entrainment of grain-size range into the main canal.

Alternate Method Given by Rozovskii (1957)

Rozovskii (1957) has also given a detail analysis for the velocity component in the vertical direction in a two-dimensional turbulent flow on a circular bend assuming smooth bottom and logarithmic velocity distribution. His equation for vertical component of velocity is given below:

$$U_y = -(1.5/K^2) * U_{cp}(D/R)^2 * [y/D - (y/D)^2] \qquad 6.9$$

Where U_y is the velocity component in the vertical direction for a two-dimensional turbulent flow in a bend of radius R, y is the reference level from the bottom of flow, D is the depth of flow, U_{cp} is the velocity distribution over the width of the channel and K is VonKarman's constant and equalled 0.5 for flow in bend.

The upward velocity component for a two-dimensional turbulent flow obtained by Equation 6.9 can be assumed to produce lift force. This lift force when equated to the submerged weight of the particle gives the maximum diameter of the particle that is lifted up. The equation is shown below:

$$C_L A \rho_f U^2/2 = \pi d^3 (\gamma_s - \gamma_f)/6 \qquad 6.10$$

Where C_L is the lift coefficient and is equal to 0.178 (Einstein and El-Samni,1949 after Garde and Ranga Raju,1985), A is the projected area of sediment particle, ρ_f is the mass density of water, U is the velocity of flow at a distance $0.35d_{35}$ from the theoretical bed, d is the diameter of the sphere and γ_s and γ_f are the specific weight of sediment and water respectively.

Replacing velocity U by U_y from Equation 6.9, Equation 6.10 becomes

$$0.178*(\pi d^2/4)*(\gamma_f/2g)*[-(1.5/0.5^2)*U_{cp}(D/R)^2\{y/D-(y/D)^2\}]^2$$
$$=\pi d^3*(\gamma_s - \gamma_f)/6$$

or, $$d = 2.969*10^{-1}*[U_{cp}(D/R)^2\{y/D - (y/D)^2\}]^2 \qquad 6.11$$

To determine the diameter of a particle that may be lifted upto a height equal to the tunnel depth, t replace y by t in Equation 6.11.

$$d = 2.969*10^{-1}*[U_{cp}(D/R)^2\{t/D - (t/D)^2\}]^2 \qquad 6.12$$

For variable discharges, different maximum diameter lifted upto the tunnel depth, t may be obtained by Equation 6.12 and may be plotted on a log-log paper against water discharges to obtain general curve for the entrainment of maximum diameter into the main canal. General curves for the entrainment of different grain-size range can also be drawn as described earlier.

b) Entrainment of Different Grain-Size Range of Bedload Material

For a particular discharge, settlement of minimum diameter of sediment particle may be obtained by the Equation 5.16 (Garde, 1970 after Garde and Ranga Raju,1985).

$$U_{cr}/\sqrt{[(S_s-1)gd]} = 0.5\ \mathrm{Log}(D/d) + 1.63 \qquad 5.16$$

The particle size in the above equation is also the maximum diameter in motion. This moving particles enter into the excluder to be discharged downstream provided the excluder works satisfactorily. If excluder does not work due to smaller operating head, deposited particles will close the excluder tunnel entrances and the moving particle will enter into the main canal. For different discharges, a general curve for maximum grain size and general curves for different grain-size ranges entrainment into the main canal can be drawn following the same procedure as described earlier.

CHAPTER VII

EVALUATION OF SEDIMENT CONTROL IN TEESTA HEADWORKS

7.1 SELECTION OF PROJECT FOR CRITICAL REVIEW AND SOURCES OF DATA

Teesta Project at present under construction in Bangladesh where sediment excluder is intorduced has been taken as a case study for evaluation. For critical review of the Teesta Project, data concerning river flows, river stages for different flows, sediment transport of the river, grain size distribution of sediment particles and information about shapes, locations and elevations of different parts of a diversion headworks are necessary.

The river discharge data of the Teesta River at Dalia and Doani gage stations were collected from Bangladesh Water Development Board (BWDB, F-123).

The suspended load transport and the grain-size range (for both suspended and bed material) were taken from the following reports of BWDB:

a) BWDB, Reports SED-164,168,179,194,198,204,208,217 and 222

b) BWDB, RRI, File no. S-139/66-82 Part II

Water discharge versus bedload transport relation for Teesta River (Figure 7.2) has been developed by BUET and BWDB (1988) and was collected for the study.

Uptodate information about the shapes, locations and elevations of different parts of Teesta Headworks were made available from BWDB (Figures 7.8 to 7.11).

7.2 TEESTA RIVER: SEDIMENT TRANSPORT AND STAGE-DISCHARGE RELATION

Teesta River carries 31 million tons (Table 7.14) of bed material load yearly for 75% dependable flow (Table 7.1). About 22.52 million tons i.e., 72% of the bed material load is from suspension. These quantification is made after developing water discharge versus suspended load discharge relation (Equation 7.1 and Figure 7.1) by considering sediment data obtained from the field (BWDB Reports SED-164,168,179,194,198,204,208,217,222 and BWDB File no. S-139/66-82 Part II). The relation for suspended load is

$$Q_s = 5.213784*10^{-6}*(Q_{Ro})^{1.65683} \qquad 7.1$$

where suspended load discharge, Q_s and water discharge, Q_{Ro} are in m^3/s. Bedload of the river turns out to be 8.48 million tons i.e., 28% of the bed material load. These quantification is made from the water discharge against bedload plot (Equation 7.2 and Figure 7.2) collected from BUET and BWDB (1988). The relation for bedload is

$$G_B = 1.41312*(Q_{Ro})^{1.49325} \qquad 7.2$$

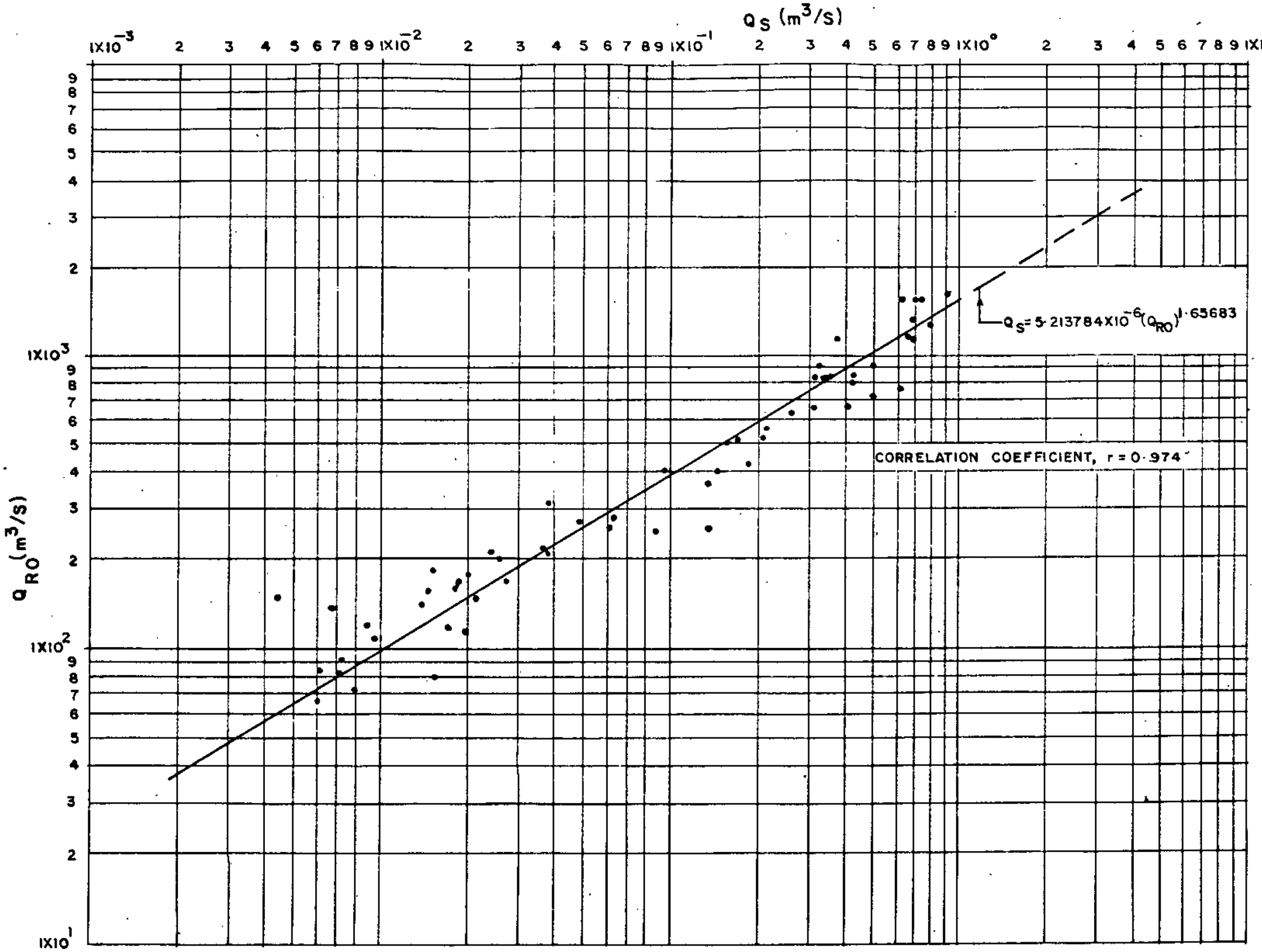


Fig: 7.1 DISCHARGE VS SUSPENDED LOAD, FROM MEASURED VALUES (1981-85).

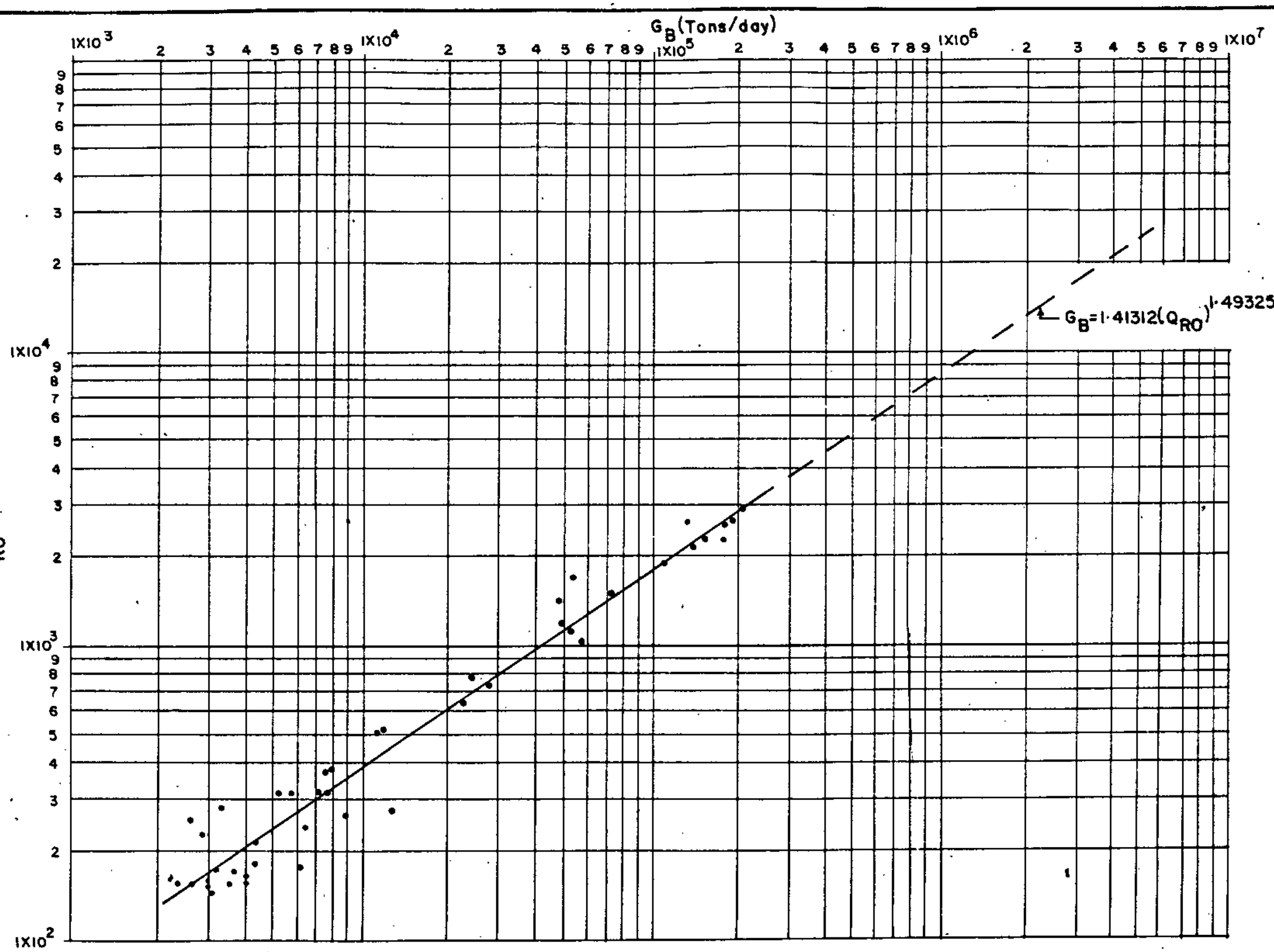


Fig: 7.2 YEAR-1985, DISCHARGE VS BED LOAD, ROTTNERS BED LOAD EQUATION (AFTER BUET AND BWDB, 1988)

Where bedload, G_B is in tons/day and water discharge, Q_{RO} in m^3/s.

The movement of grain sizes for different discharges is correlated (Figure 7.3). The correlation seems to be very weak indicating that the flow is unsteady in the river. Under this condition a smaller discharge can entrain even larger particles in the flow. From available data (BWDB, Reports SED-164,168, 179,194,198,204,208,217,222 and BWDB File No. S-139/66-82 Part II) average grain size distribution curves for both suspended (Figure 7.4) and bed materials (Figure 7.5) are developed. Figures 7.4 and 7.5 indicate that the Teesta River carries the bed material of grain-size range between 0.074mm to 0.6mm and even larger. The percentages of different particle size range are shown below:

Particle range	Percent	
	Suspended material	Bed material
0.074 to 0.15mm	42	17
0.15 to 0.30mm	42	46
0.30 to 0.60mm	14	33
> 0.60mm	2	4

The theoretical analysis of the stage-discharge relationship is made on the basis of a procedure given by Jansen (1979) and is shown below:

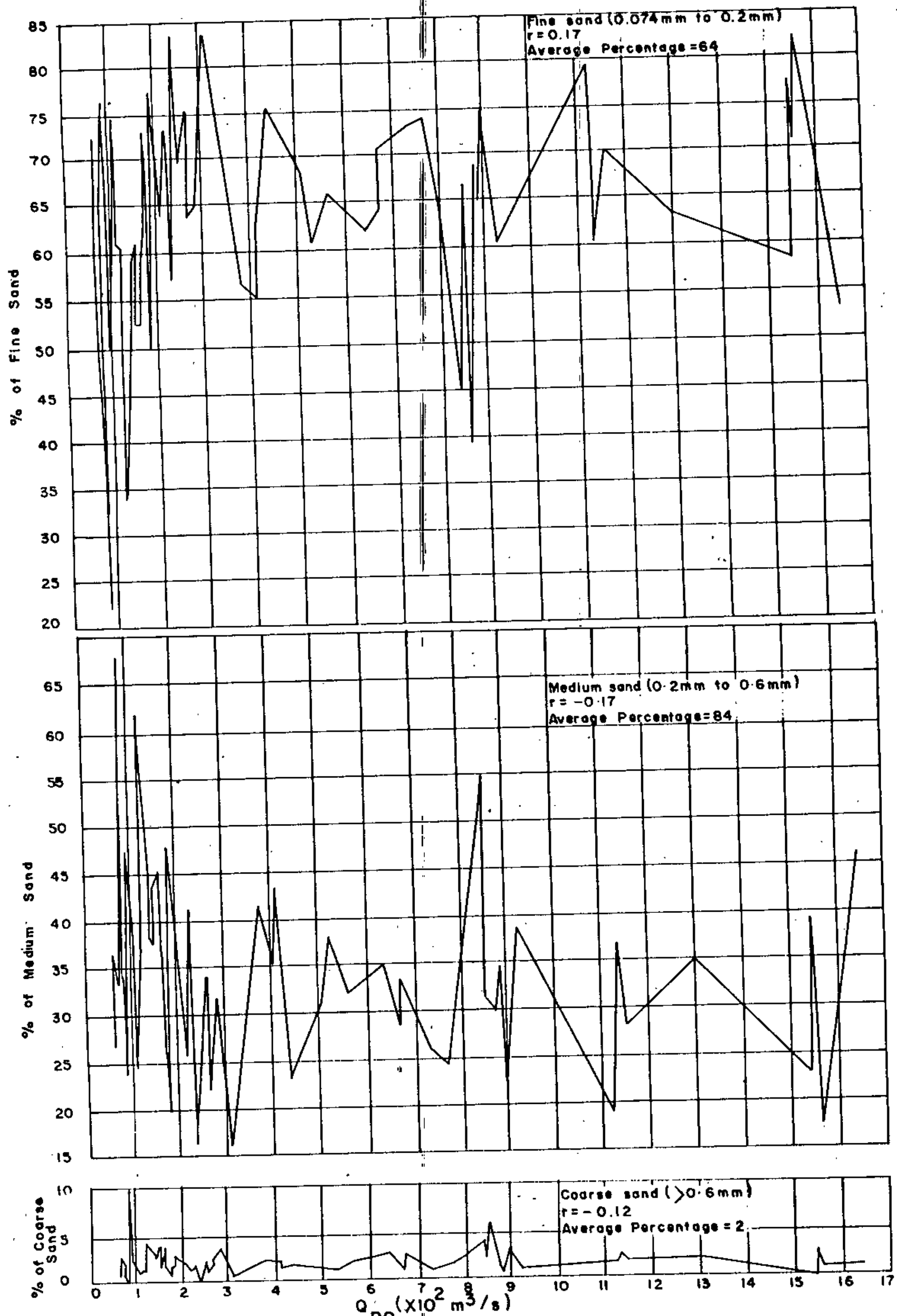


Fig: 7.3 PERCENTAGE ANALYSIS OF SUSPENDED MATERIAL

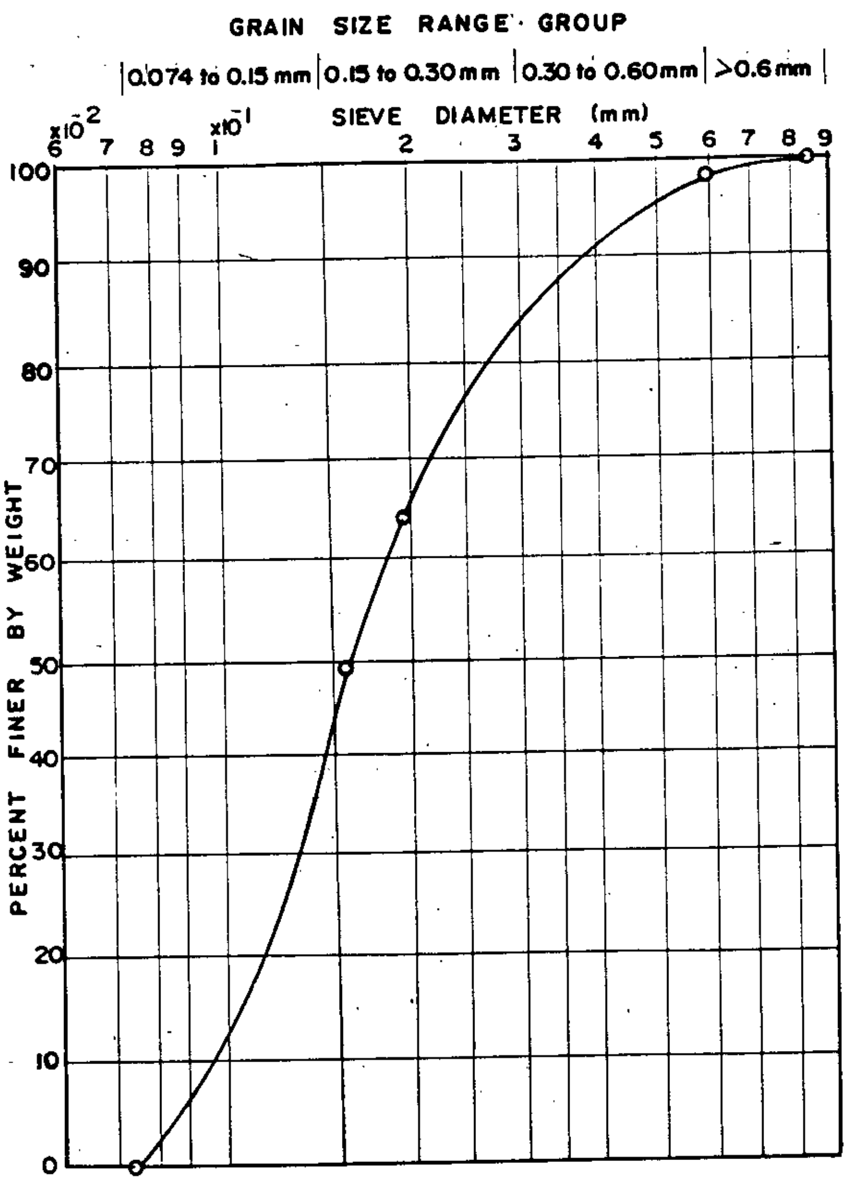


Fig. 7.4 AVERAGE GRAIN SIZE DISTRIBUTION OF SUSPENDED MATERIAL (1981-86)

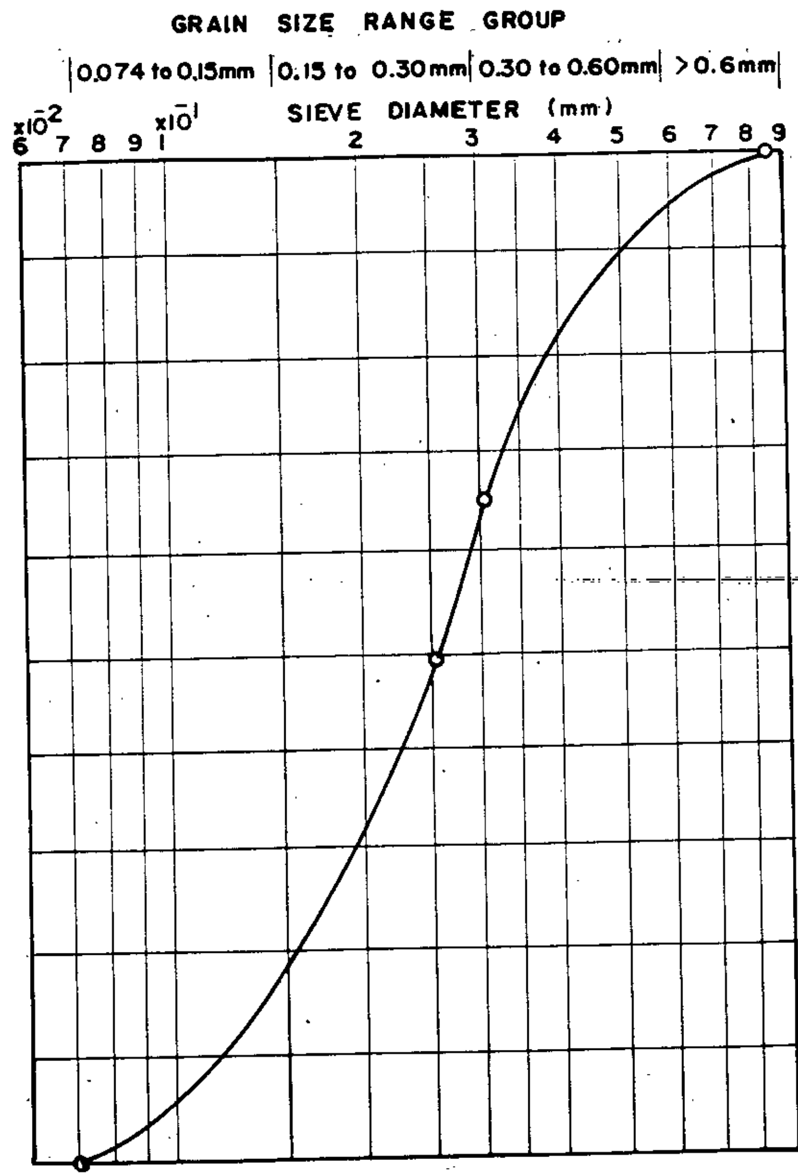


Fig. 7.5 AVERAGE GRAIN SIZE DISTRIBUTION OF BED MATERIAL (1981-86)

$$Q_{RO} = a(Z_W - Z_o)^b \tag{7.3}$$

$$\therefore \quad Y = a^* + bx \tag{7.4}$$

Where $Y = \log(Q_{Ro})$; $a^* = \log a$, where a is a constant; b is a constant; $x = \log(Z_W - Z_o)$, where Z_o is the stage at zero discharge $= (Z_1 * Z_3 - Z_2^2)/(Z_1 + Z_3 - 2Z_2)$ and Z_1, Z_2 and Z_3 are the three stages from the smooth curve drawn by visual estimation such that the corresponding discharges Q_{RO1}, Q_{RO2} and Q_{RO3} could make $Q_{RO2}^2 = Q_{RO1} * Q_{RO3}$.

The data used for plotting is for the period between 1979 and 1986 of Teesta River (BWDB, File F-123). The stage-discharge relation (Equation 7.5) is shown in Figure 7.6.

$$Q_{Ro} = 185.06(Z_W - 50.19)^{3.18468615} \tag{7.5}$$

Where water discharge, Q_{Ro} in m^3/s and stage Z_W is in m.

The confidence over the selected stage-discharge curve at 95% confidence band was found out (Figure 7.7) by applying t-test for individual observation as suggested by Jansen (1979), as

$$Y = a^* + bX \pm t_{0.975}^{N-2} \; S_D \sqrt{[1/N + (X-\bar{X})^2 / \sum_{i=1}^{N} (X_i - \bar{X})^2]} \tag{7.6}$$

where N is the number of observation; $\bar{X} = \sum_{i=1}^{N} X_i / N$; $S_D = \log\sqrt{[1/(N-1) * \sum_{i=1}^{N}(Q_{mi} - Q_{ri})^2]}$, where Q_m is the measured discharge and Q_r is the read value from stage-discharge curve for the corresponding stage of Q_m.

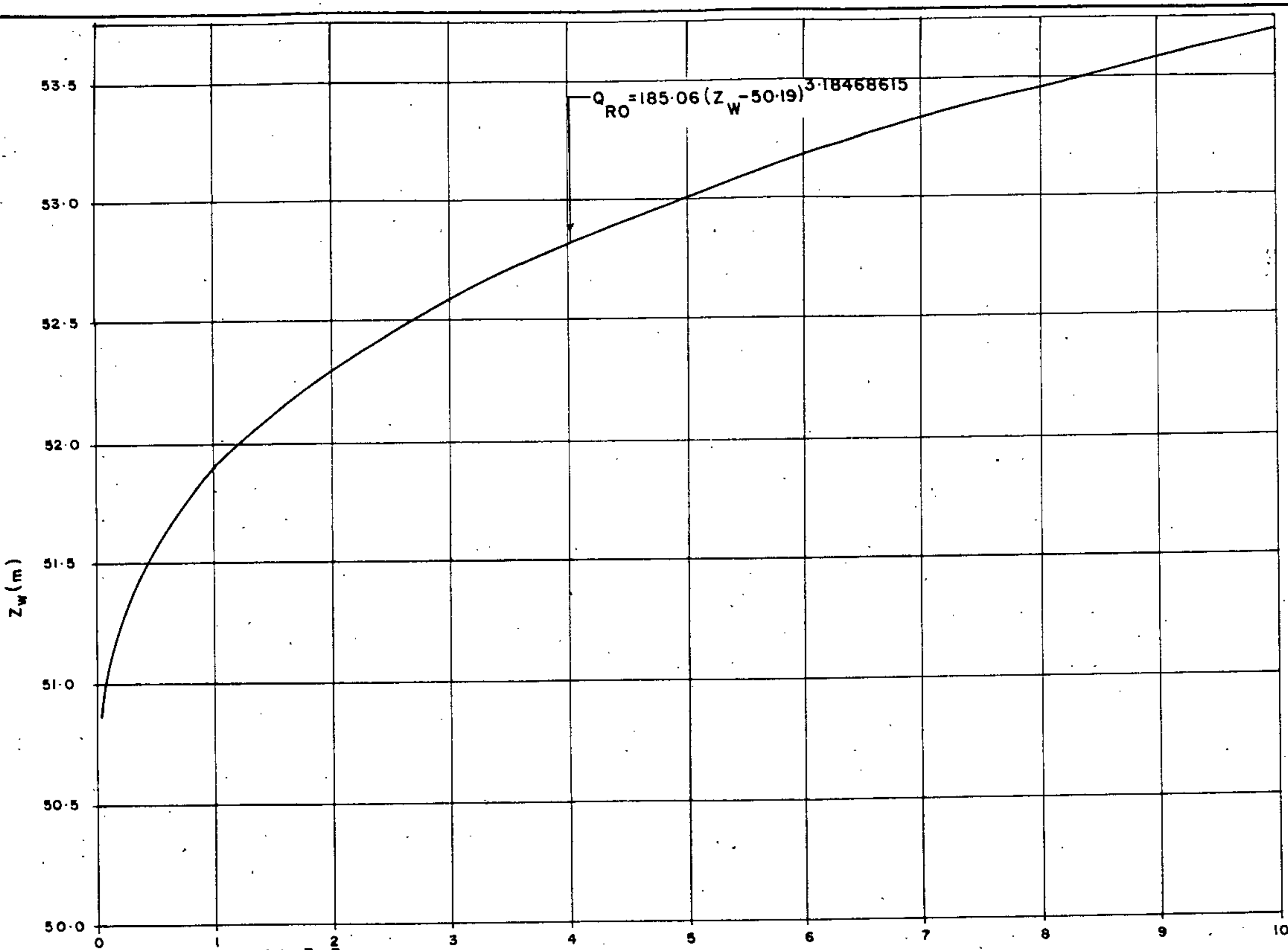


Fig: 7.6 STAGE DISCHARGE CURVE (1979-86).

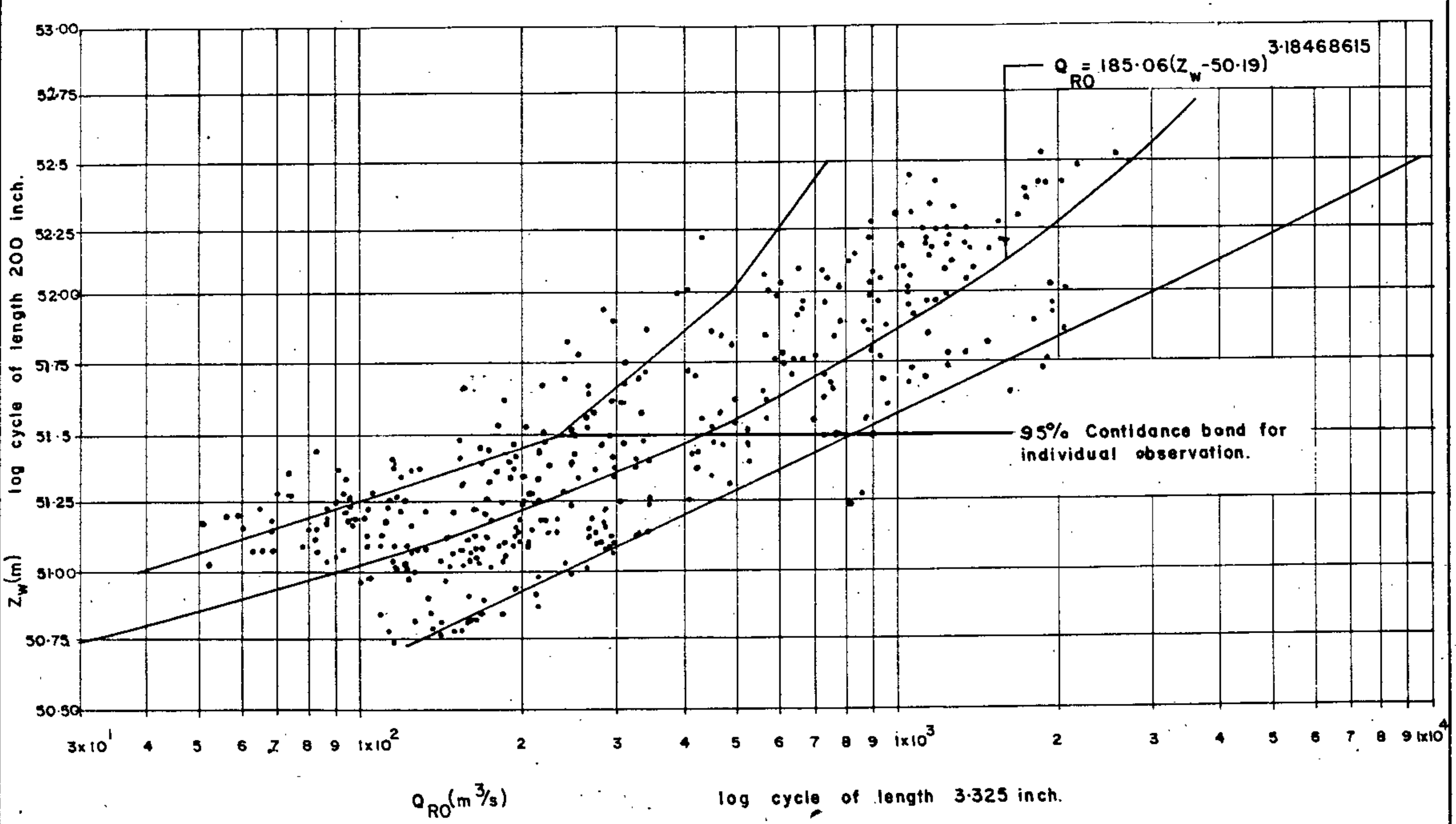


Fig: 7.7 STAGE DISCHARGE CURVE (1979-86) WITH CONFIDENCE BAND

Table 7.1 10 Day Average 75% Dependable Discharge(1952-85) by Log Normal Distribution

Month	Period	75% dependable discharge m^3/s	Month	Period	75% dependable discharge m^3/s
	1-10	131		1-10	1375
January	11-20	128	July	11-20	1580
	21-31	119		21-31	1725
	1-10	112		1-10	1435
February	11-20	109	August	11-20	1325
	21-28/29	112		21-31	1385
	1-10	122		1-10	1268
March	11-20	128	September	11-20	1211
	21-31	131		21-30	1037
	1-10	155		1-10	716
April	11-20	172	October	11-20	517
	21-30	198		21-31	423
	1-10	248		1-10	264
May	11-20	312	November	11-20	228
	21-31	438		21-30	177
	1-10	553		1-10	148
June	11-20	863	December	11-20	131
	21-30	1142		21-31	113

From Figure 7.7 it is observed that over 83 percent of the whole data remains within the 95% confidence band.

75% dependable discharge is that water discharge whose exceeding probability of occurence is 75%. 10 day average dicharges of July and August (1952-85) of Teesta River (BWDB, R-156 and rest data from Surface Water Hydrology, BWDB,Dhaka) are plotted on Log Probability Papers and found to be fitted by Log Normal Distribution. Log Normal Distribution equation is thus used for determining 75% dependable discharge and results are shown in Table 7.1. Log Normal Distribution equation for 75% dependable discharge is

$$Q_{75\%} = 10^{(\mu + z * \sigma)} \tag{7.7}$$

Where $Q_{75\%}$ is the 10 day average 75% dependeable discharge, μ is the mean of 10 day average log discharges of particular 10 day period, σ is the standard deviation of 10 day average log discharges of particular 10 day period and Z = - 0.675 (for area (0.5-0.75) = - 0.25 from standard normal curve).

7.3 EVALUATION OF SEDIMENT CONTROLLING MEASURES USED IN THE HEADWORKS

Tremendous amount of sediment discharge flowing through the Teesta River, if not controlled properly, may create serious probelm by deposition at the upstream of headworks. Moreover, a larger volume of coarser material of the sediment particle can enter into the main canal creating problem of serious nature.

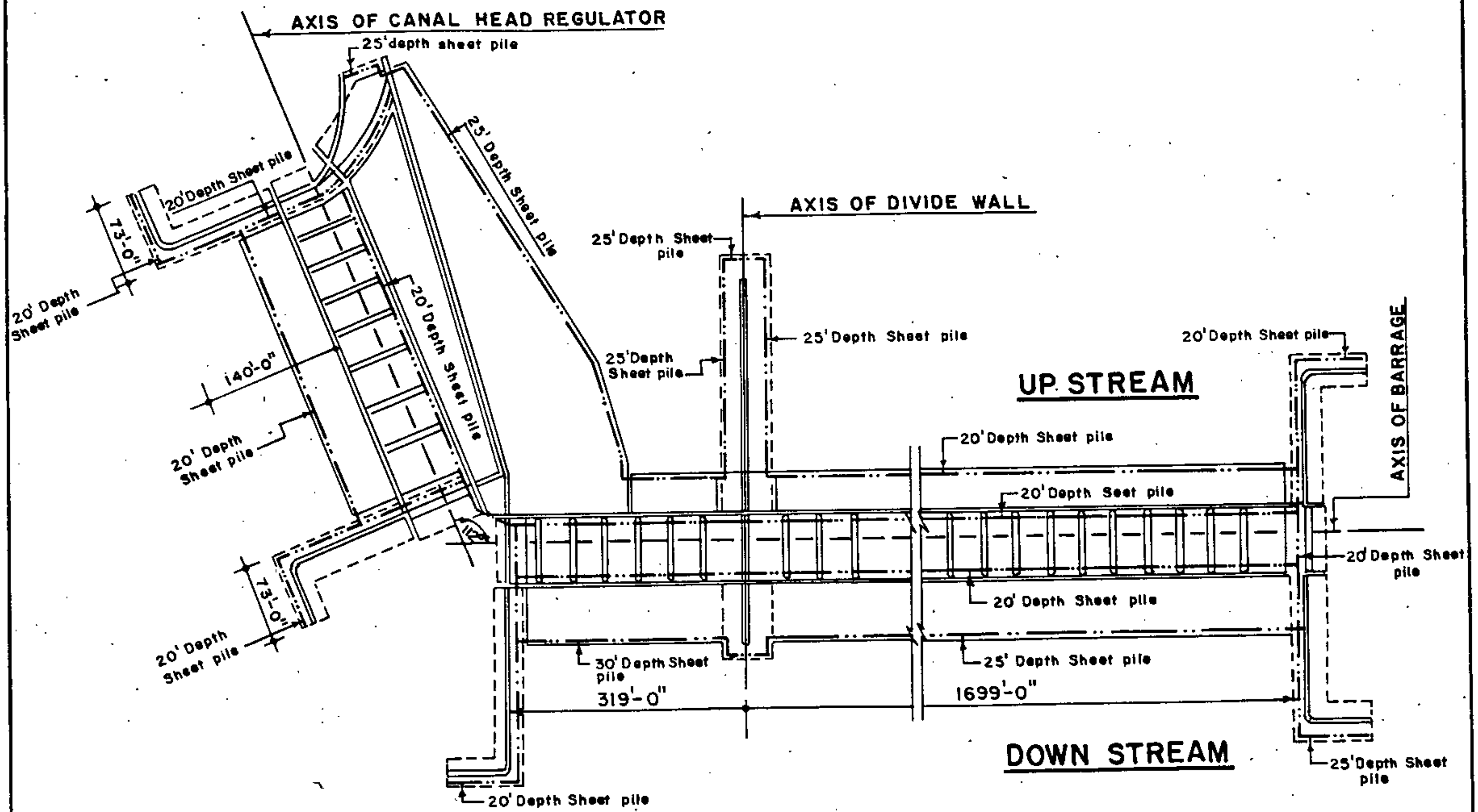


Fig: 7.8 GENERAL LAYOUT PLAN OF HEAD WORKS

SOURCE, BWDB

TEESTA BARRAGE PROJECT

BWDB has used different sediment controlling measures in Teesta Headworks to control sediment. It is now necessary to evaluate different sediment controlling measures used in Teesta Headworks.

a) Location of Offtake

For single offtake head regulator should be located at the concave bend of the river, but in the Teesta Project head regulator has been located on the convex bend (after imposing lead cut). Thus helicoidal flow may deflect larger volume of bedload to move towards the head regulator creating formation of island in front of the pocket and enhance more probability entrainment of bedload into the main canal.

b) Orientation of Offtake

BWDB has used the angle of offtake to be 112° (Figure 7.8) which is in agreement with the angles provided with the canal head regulator constructed recently in India (Dhillon,1980).

c) Divide Wall

Divide wall covering two-third width of head regulator gives generally good result and in Teesta it is 95.73m which is 2/2.3 W_h and is closer to the marginal line (Figure 3.6).

Exclusion is beter effected if steeper slope is provided on the divide wall nose but design safety is lost to some extent due to scour hole. On the other hand flat slopes tend to reduce scour depth, and better design safety but beneficial effect on sediment exclusion is lost to some extent. Advantages of this fact can be taken by adopting a flat nose slope on the pocket side and a steep slope on the river side (CWPRS,1946, after Joglekar,1971). But BWDB has provided steep slopes on both sides of the divide wall nose which will facilitate sediment exclusion but design safety may loss to some extent. A flatter slope on the pocket side could have given better result.

d) Width of the Pocket

The design discharges of barrage and main canal were kept at 9918.5m^3/s and 226.7m^3/s respectively. While the legnth of the divide wall, width of head regulator and pocket width have been maintained at 95.73m, 110.37m and 96.34m respectively. But according to the design standard (Figure 2.7) the width of the pocket should be in between 100.77m and 119.2m. Comparision of the values indicates that the pocket width is inadequate.

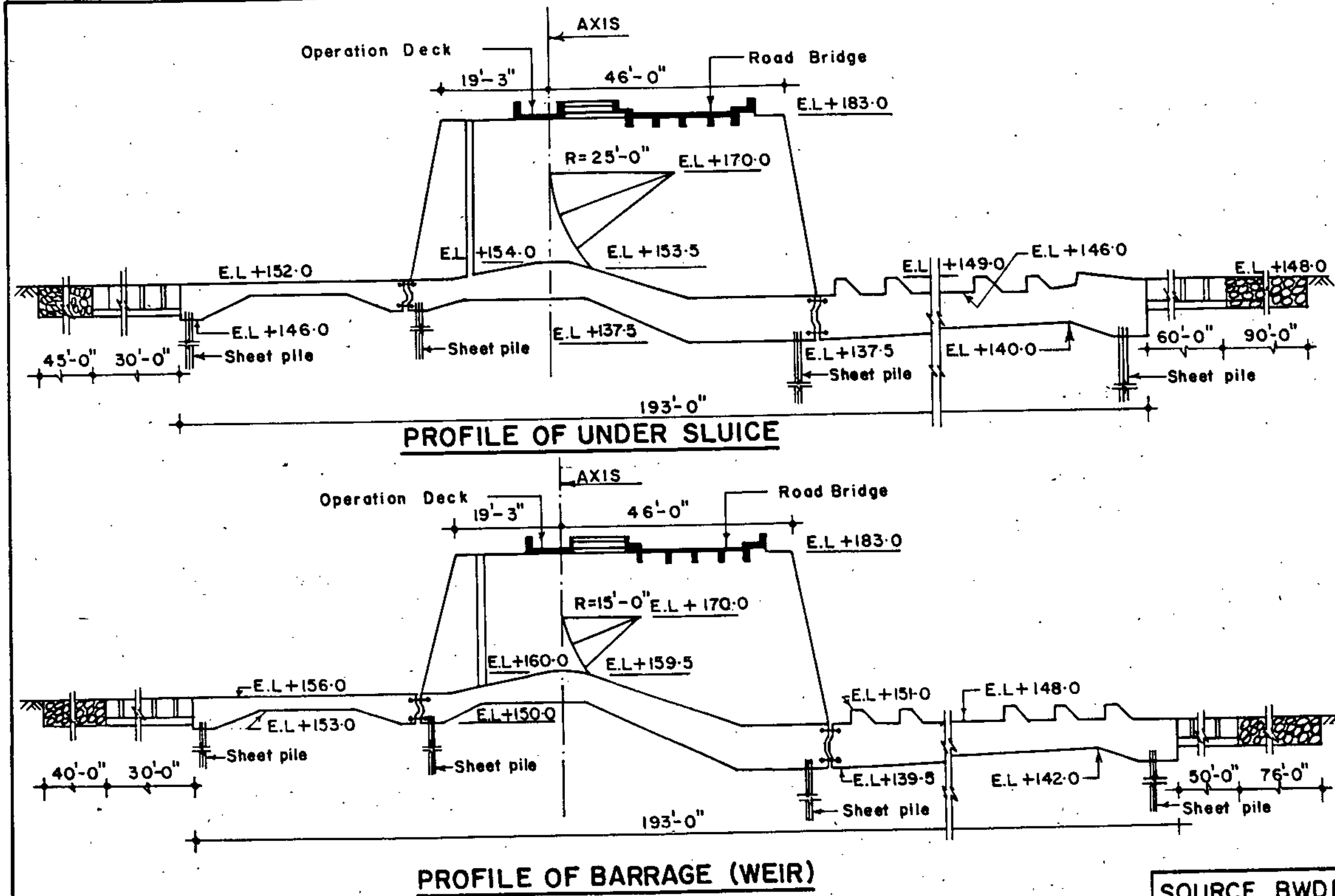
A converging pocket towards the downstream has been designed in Teesta Barrage which will give better performance for sediment exclusion.

e) Location of Undersluices

Most suitable location of undersluices is adjacent to the canal head regulator and is properly positioned in the Teesta Barrage (Figure 7.8). But a second pocket or river sluices (Figure 3.4) adjacent to the undersluice pocket should have been positioned in Teesta Barrage for better sediment control as the river curvature is unfavourable.

f) Crest Level of Undersluices and Head Regulator

Crest level of undersluice should be 1m below the average deeper channel level at barrage site but in Teesta Barrage (Figure 7.9) BWDB has fixed it to be 2.05m below the average deeper channel level and 0.06m below the recorded lowest bed level. Lowering of undersluice crest level to a great extent may



SOURCE, BWDB

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Fig. 7.9 PROFILE OF UNDERSLUICE AND WEIR.

create lower velocity of flow at upstream and is harmful in the context of sediment exclusion.

When submerged flow condition occurs in a barrage sufficient amount of suspended load may enter into the main canal. To compensate this, crest level of head regulator should be 1 to 4m higher than the average deeper channel level. But in Teesta Headworks crest level of head regulator is only 0.55m above the average deeper channel level at barrage site. This may create larger volume of suspended load to enter into the main canal.

g) Shape of Guide Bunds

Diverging guide bunds may be used for wide and shallow river and where head regulator is situated on the convex bend of the river, concave-convex guide should be used. Teesta is a wide and shallow river where convex bend of the river (after imposing lead cut) has been selected for the position of head regulator. Hence either of the diverging guide bunds or concave-convex guide bunds will be suitable for approach protection of the Teesta Headworks. But in Teesta Headworks BWDB have used converging guide bunds with gradually varying radius, which may create an island in front of the pocket and sediment exclusion may be hampered.

h) Barrage Regulation

Semi-still pond regulation is generally followed and will be suitable for Teesta Barrage also. The reason for the use of this method have been elaborated in subsection 3.8.

i) Tunnel Type Sediment Excluder

BWDB has used Khanki type sediment excluder (Figures 7.10 and 7.11) for headworks which is quite effective for oblique

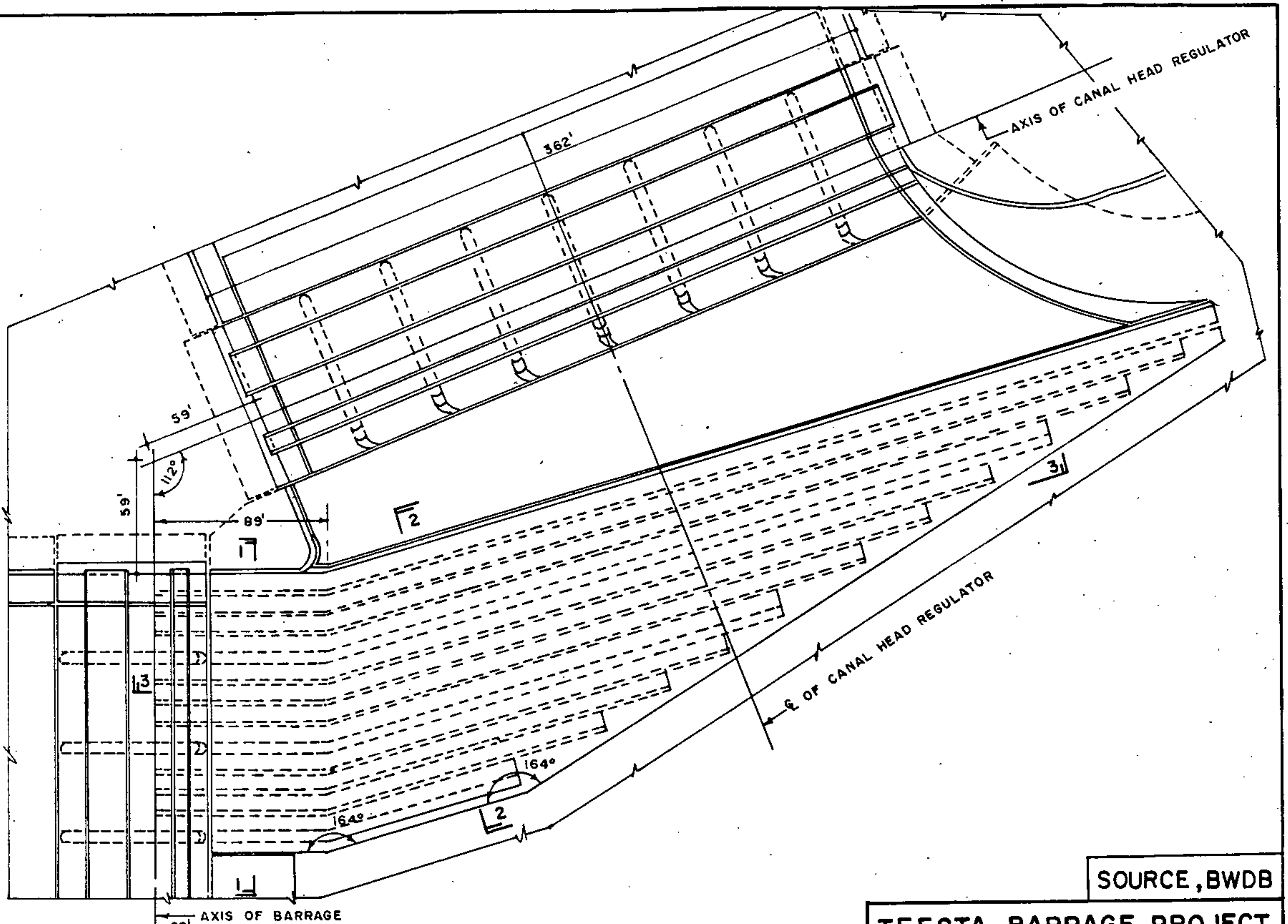
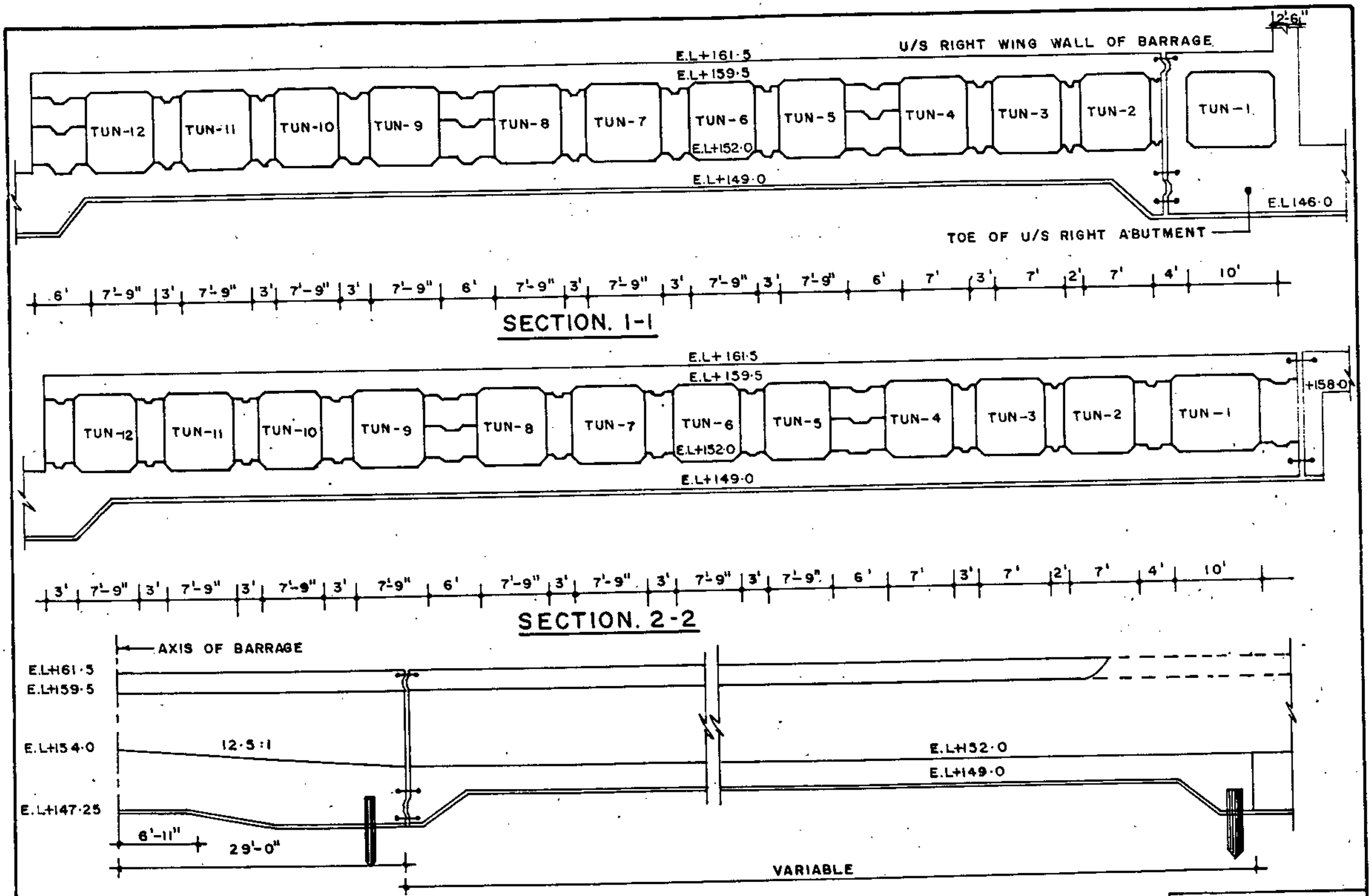


Fig: 7.10 PLAN OF SEDIMENT EXCLUDER



SECTION. 3-3

Fig: 7.11 SECTIONS OF SEDIMENT EXCLUDER

SOURCE, BWDB

TEESTA BARRAGE PROJECT

river flow but the efficiency may be greatly affected if any subsequent changes occur due to swinging of the river caused by meander. It is mentioned here that the CWPC type sediment excluder can work satisfactorily by confining the turbulence at the entrance of the tunnel for straight river approach and for oblique flow condition also. Hence for Teesta Headworks CWPC type sediment excluder might be necessary for better sediment exclusion. The design of the CWPC type sediment excluder is given in Appendix-2 considering the data of Teesta River at Barrage site and size and shape of barrage and head regulator as existing. The design is shown in Figures 7.12 and 7.13.

BWDB has fixed different criteria for the design of sediment excluder, the evaluation of the parameters are given below:

Pond level should be equal to or slightly less than the downstream water level for maximum design discharge of barrage like Teesta. But BWDB has fixed the level at 1.85m below the downstream water level for maximum design discharge of the barrage. Due to this reason net head for sediment excluder may not be sufficient for better exclusion.

Excluder discharge should be equal to or in the neighbourhood of 30% of canal discharge. In Teesta Barrage 88.75% of canal discharge has been assumed to pass through the excluder tunnels which is a marked deviation from the standard practice and may cause extra turbulence in the pocket.

BWDB has used tunnel depths of 2.287m at entrance and 1.677m at exit which will act satisfactorily for maintenance and repair works. BWDB used different sections at tunnel exit (Figure 7.11)

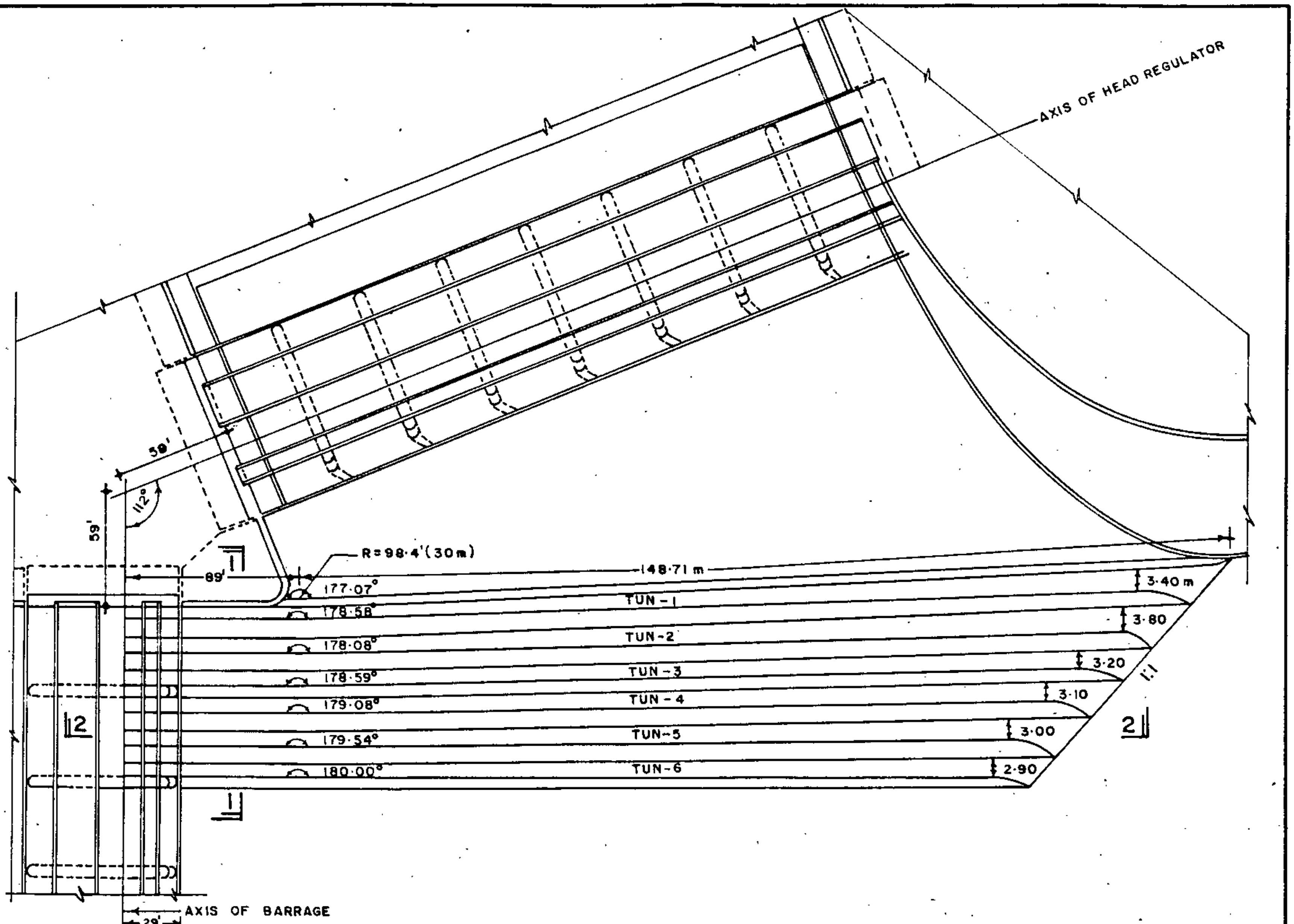


Fig: 7.12 PLAN OF DESIGNED SEDIMENT EXCIUDER (C.W.P.C. TYPE).

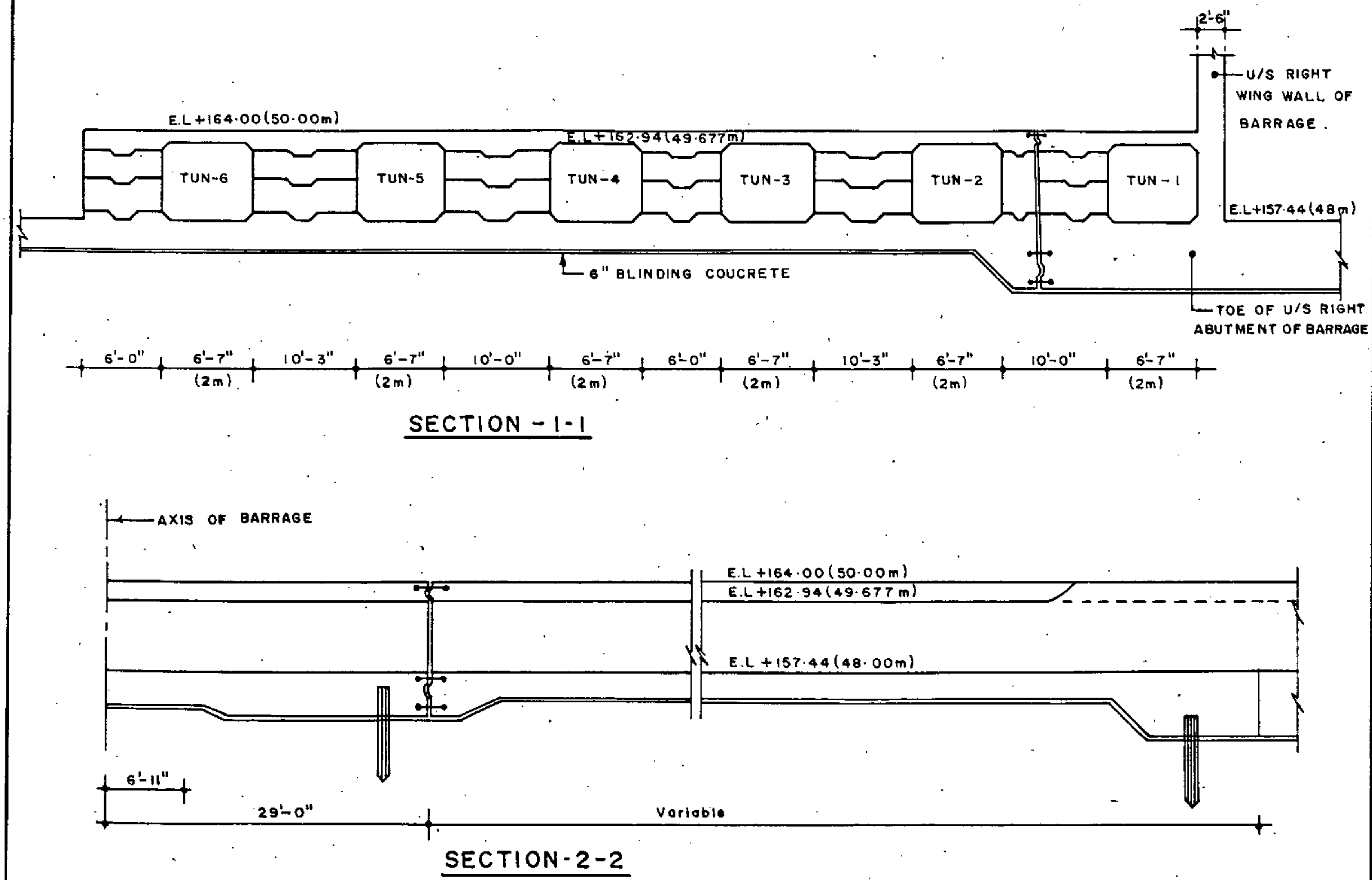


Fig: 7.13 SECTIONS OF DESIGNED SEDIMENT EXCLUDER (C.W.P.C TYPE).

instead of having same section. This may cause unequal water level at the downstream of excluder. Normally the widths of the tunnels are adjusted for nearly equal head loss in all the tunnels but BWDB has used constant width throughout the length and thus considerable variation of head loss (1.02m to 1.63m) occurs in the tunnels (Table 7.4). Due to non-availability of operating head, sediment excluder may block totally. The radius of the bend should be 10 to 15 times the tunnel width but the radius used is only 3 times i.e. instead of using the radius of 21-46m the present works used only 7.47m and hence smooth passing through the bend may be hampered.

Excluder velocity of 3m/s may be sufficient for sandy river like Teesta to pass sediment through excluder. But the excluder velocity used is 4.23m/s which seems to be high and applicable for rivers carrying boulders.

The present design has used 3 undersluice bays comprising of 12 tunnels as the total widths of excluder at exit (38.41m). According to Varshney (1982) 2 undersluice bays seem to be sufficient to pass the sediment. In addition, the clear width of an excluder at exit should be the width of 1 undersluice bay but BWDB has used 2.3 times the normal requirement i.e. instead of 12.195m the present use in 28.354m.

7.4 ENTRAINMENT OF SEDIMENT LOAD INTO THE MAIN CANAL

a) Existing Condition

For the existing barrage condition (Table 7.2) analysis is carried out for the deposition of sediment discharge in the

Table 7.2 Different Conditions for Teesta Headworks

Item	Existing Condition	Design Condition	Suggested Condition
Pond level	(m) +51.8293	+ 53.6	+ 53.6
Upstream floor level of weir	(m) +47.561	+ 49	+ 47.561
Crest level of weir	(m) +48.7805	+ 50.1	+ 48.7805
Upstream floor level of undersluice	(m) +46.3415	+ 48	+ 46.3415
Crest level of undersluice	(m) +46.9512	+ 48	+ 46.9512
Excluder discharge	(m^3/s) 201.2	60	60
Total width of barrage	(m) 615.24(44@12.195)	615.24	615.24
Width of undersluice bays	(m) 96.34(7@12.195)	96.34	96.34
Width of other barrage bays	(m) 517.07(37@12.195)	517.07	517.07
Number of tunnel	12 (Khanki type) Covering 3 bays of underlusice	6(CWPC type) Covering 2 bays of undersluice	5 Existing tunnel nos.1,2,4,6 and 8; (Khanki type)Covering 2 bays of undersluice

pocket and the entrainment of sediment discharge into the main canal. These are shown in Tables 7.3 to 7.6 and in Figures 7.14 to 7.21. For 75% dependable discharge, deposition of bed material load in the pocket becomes 1.626 million tons (Table 7.14) of which 59% is for suspended load and 41% is for bedload deposition. Entrainment of bed material load into the main canal is found to be 6.5 million tons (Table 7.14) of which 73% is for suspended load and 27% is for bedload. Sizes of particles entering into the main canal for bed material load are shown below:

Particle range	Percent
0.074 to 0.15mm	36.13
0.15 to 0.30mm	43.20
0.30 to 0.60mm	18.35
>0.60mm	2.32

Entrainment of bedload into the main canal is due to the fact that under existing condition sediment excluder does not work. Clear water head loss through the tunnel, h_o = 1.314m (Table 7.4). The available operating head, h for few discharges are shown below (for detail see Table 7.5).

Table 7.3 Tunnel Dimensions of Existing Excluder (Khanki Type)

Tunnel	Belmouth				Straight			Bend, =16°				Straight			Transition			
	Length	Width u/s	Width d/s	Depth	Length	Width	Depth	Length	Radius	Width	Depth	Length	Width	Depth	Length	Width	Depth u/s	Depth d/s
	m	m	m	m	m	m	m	m	m	m	m	m	m	m	m	m	m	m
1	2.134	5.488	3.049	2.287	146.17	3.049	2.287	2.086	7.47	3.049	2.287	16.98	3.049	2.287	8.841	3.049	2.287	1.677
2	0.915	2.744	2.134	2.287	140.24	2.134	2.287	2.086	7.47	2.134	2.287	17.52	2.134	2.287	8.841	2.134	2.287	1.677
3	1.524	3.049	2.134	2.287	129.37	2.134	2.287	2.086	7.47	2.134	2.287	17.91	2.134	2.287	8.841	2.134	2.287	1.677
4	3.5	3.963	2.134	2.287	113.99	2.134	2.287	2.086	7.47	2.134	2.287	18.34	2.134	2.287	8.841	2.134	2.287	1.677
5	1.524	3.277	2.363	2.287	105.08	2.363	2.287	2.086	7.47	2.363	2.287	18.9	2.363	2.287	8.841	2.363	2.287	1.677
6	1.524	3.277	2.363	2.287	94.1	2.363	2.287	2.086	7.47	2.363	2.287	19.36	2.363	2.287	8.841	2.363	2.287	1.677
7	1.524	3.277	2.363	2.287	83.114	2.363	2.287	2.086	7.47	2.363	2.287	19.63	2.363	2.287	8.841	2.363	2.287	1.677
8	3.5	4.19	2.363	2.287	66.96	2.363	2.287	2.086	7.47	2.363	2.287	20.29	2.363	2.287	8.841	2.363	2.287	1.677
9	1.524	3.277	2.363	2.287	58.07	2.363	2.287	2.086	7.47	2.363	2.287	20.88	2.363	2.287	8.841	2.363	2.287	1.677
10	1.524	3.277	2.363	2.287	47.09	2.363	2.287	2.086	7.47	2.363	2.287	21.33	2.363	2.287	8.841	2.363	2.287	1.677
11	1.524	3.277	2.363	2.287	36.1	2.363	2.287	2.086	7.47	2.363	2.287	21.8	2.363	2.287	8.841	2.363	2.287	1.677
12	1.524	3.277	2.363	2.287	25.11	2.363	2.287	2.086	7.47	2.363	2.287	22.26	2.363	2.287	8.841	2.363	2.287	1.677

Table 7.4 Head Loss in the Excluder Tunnel Under Existing Condion

Tunnel	h_f					h_{en}	h_c		h_b	h_{ex}	h_o	U_{EX}	U_{EX}	Q_{EX}
	Bell-mouth	Straight	Bend	Straight	Contraction		Bell mouth	Contra-ction				Avg	Max.	
	m	m	m	m	m		m	m	m	m	m	m/s	m/s	m^3/s
1	.004	.635	.009	.0738	.0573	.0465	.0339	.0422	.0666	.4645	1.4328	3.019	4.23	21.64
2	.0035	.763	.011	.0954	.0705	.0538	.0194	.0422	.0566	.5380	1.6334	3.249	4.23	15.14
3	.005	.704	.011	.0975	.0705	.0512	.0251	.0422	.0566	.5119	1.6277	3.169	4.23	15.14
4	.0075	.620	.011	.0998	.0705	.0459	.0349	.0422	.0566	.4593	1.4476	3.002	4.23	15.14
5	.0049	.534	.0106	.0961	.0661	.0519	.0236	.0422	.0593	.5187	1.4073	3.19	4.23	16.77
6	.0049	.478	.0106	.0984	.0661	.0519	.0236	.0422	.0593	.5187	1.3536	3.19	4.23	16.77
7	.0049	.422	.0106	.1008	.0661	.0519	.0236	.0422	.0593	.5187	1.300	3.19	4.23	16.77
8	.0076	.340	.0106	.1031	.0661	.0467	.0335	.0422	.0593	.4673	1.1747	3.028	4.23	16.77
9	.0049	.295	.0106	.1061	.0661	.0519	.0236	.0422	.0593	.5187	1.1783	3.19	4.23	16.77
10	.0049	.239	.0106	.1084	.0661	.0519	.0236	.0422	.0593	.5187	1.1246	3.19	4.23	16.77
11	.0049	.183	.0106	.1108	.0661	.0519	.0236	.0422	.0593	.5187	1.071	3.19	4.23	16.77
12	.0049	.128	.0106	.1131	.0661	.0519	.0236	.0422	.0593	.5187	1.0183	3.19	4.23	16.77

Average total head loss, h_o=1.314m

Table 7.5 Parameters for the Analysis of Entrainment of Sediment into the Excluder Tunnel and into the Main Canal, Under Existing Condition

Q_{RO}	Q_R	Q_C	Q_{EX}	Q_P	D	U	h (total discharge downstream)	h (required flow through main canal)	B_{EX} and no. tunnel	h_o
m^3/s	m^3/s	m^3/s	m^3/s	m^3/s	m	m/s	m	m	m	m
163.90	100.00	59.0	41.00	100.00	5.488	0.050	0.8150	1.0163	5.183 No.2	1.533
327.87	200.00	118.0	82.00	200.00	"	0.100	0.6146	0.8648	11.814 No.5	1.510
655.74	400.00	226.7	173.30	400.00	"	0.200	0.3655	0.6597	23.629 No.10	1.368
983.60	600.00	"	201.20	427.90	"	0.300	0.1925	0.3928	28.354 No.12	1.314
1368.85	835.00*	"	"	"	"	0.418	0.0343	0.1863	"	"
1566.80	1000.00	"	"	"	5.589	0.484	0.0410	0.1732	"	"
2566.80	2000.00	"	"	"	6.019	0.851	0.0585	0.1368	"	"
3566.80	3000.00	"	"	575.00	6.319	1.177	0.0718	0.1303	"	"
4566.80	4000.00	"	"	806.00	6.569	1.474	0.0951	0.1427	"	"
5566.80	5000.00	"	"	1043.00	6.784	1.750	0.1196	0.1603	"	"
6566.80	6000.00	"	"	1237.00	6.989	2.005	0.1587	0.1945	"	"
7566.80	7000.00	"	"	1389.00	7.159	2.254	0.1808	0.2131	"	"
8566.80	8000.00	"	"	1608.00	7.339	2.481	0.2268	0.2562	"	"
9566.80	9000.00	"	"	1828.00	7.519	2.691	0.2839	0.3109	"	"
10485.30	9918.50	"	"	2006.00	7.669	2.880	0.3290	0.3542	"	"

$Q_{CI} = 0.39 * Q_{RO} < 566.8 m^3/s$ $\quad Q_{EX} = (Q_R - Q_C) < 212.2 m^3/s$

$Q_R = (Q_{RO} - Q_{CI}) m^3/s$ $\quad Q_P = (Q_{EX} + Q_C) m^3/s$ $\quad$ when $Q_R - (Q_{EX} + Q_C) < Q_W$

$Q_C = 0.36 * Q_{RO} < 226.7 m^3/s$ $\quad Q_P = Q_U m^3/s$ $\quad$ when $Q_R - (Q_{EX} + Q_C) > Q_W$

*Pond level discharge ($Q_U = 170 m^3/s$ and $Q_W = 665 m^3/s$)

Table 7.6 Entrainment of Sediment Load into the Excluder Tunnel and into the Main Canal, Under Existing Condition

Q_{RO}	Q_{BPI}	Q_{SPI}	Q_{BPD}	Q_{SPD}	$Q_{BP}=Q_{BT}$	Q_{SF}	Q_{ST}	C_{EX}	C_T	Q_T	Q_{BPB}	Q_{SPS}	Q_{BC}	Q_{SC}
m^3/s	m^3/s	m^3/s	m^3/s	m^3/s	m^3/s	m^3/s	m^3/s	m^3/m^3	m^3/m^3	m^3/s	m^3/s	m^3/s	m^3/s	m^3/s
163.90	5.98×10^{-3}	1.07×10^{-3}	5.98×10^{-3}	1.07×10^{-2}	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
327.87	1.68×10^{-2}	3.385×10^{-2}	1.68×10^{-2}	3.385×10^{-2}	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
655.74	4.74×10^{-2}	1.067×10^{-1}	3.745×10^{-2}	5.122×10^{-2}	9.954×10^{-3}	5.548×10^{-2}	2.441×10^{-3}	7.152×10^{-5}	0.0	0.0	9.954×10^{-3}	5.548×10^{-2}	9.954×10^{-3}	5.548×10^{-2}
983.60	6.196×10^{-2}	1.49×10^{-1}	1.115×10^{-2}	1.49×10^{-2}	5.081×10^{-2}	1.341×10^{-1}	1.341×10^{-2}	3.026×10^{-4}	0.0	0.0	5.081×10^{-2}	1.341×10^{-1}	5.081×10^{-2}	1.341×10^{-1}
1368.85	7.29×10^{-2}	1.852×10^{-1}	0.0	0.0	7.29×10^{-2}	1.852×10^{-1}	3.148×10^{-2}	4.919×10^{-4}	0.0	0.0	7.29×10^{-2}	1.852×10^{-1}	7.29×10^{-2}	1.852×10^{-1}
1566.80	7.97×10^{-2}	2.084×10^{-1}	0.0	0.0	7.97×10^{-2}	2.084×10^{-1}	3.751×10^{-2}	5.524×10^{-4}	0.0	0.0	7.97×10^{-2}	2.084×10^{-1}	7.97×10^{-2}	2.084×10^{-1}
2566.80	1.12×10^{-1}	3.286×10^{-1}	0.0	0.0	1.12×10^{-1}	3.286×10^{-1}	9.431×10^{-2}	9.722×10^{-4}	0.0	0.0	1.12×10^{-1}	3.286×10^{-1}	1.12×10^{-1}	3.286×10^{-1}
3566.80	1.84×10^{-1}	5.764×10^{-1}	0.0	0.0	1.84×10^{-1}	5.764×10^{-1}	1.942×10^{-1}	1.782×10^{-3}	0.0	0.0	1.84×10^{-1}	5.764×10^{-1}	1.116×10^{-1}	3.496×10^{-1}
4566.80	2.97×10^{-1}	9.759×10^{-1}	0.0	0.0	2.97×10^{-1}	9.759×10^{-1}	3.679×10^{-1}	3.133×10^{-3}	0.0	0.0	2.97×10^{-1}	9.759×10^{-1}	1.113×10^{-1}	3.658×10^{-1}
5566.80	4.298×10^{-1}	1.462	0.0	0.0	4.298×10^{-1}	1.462	5.906×10^{-1}	4.809×10^{-3}	0.0	0.0	4.298×10^{-1}	1.462	1.157×10^{-1}	3.394×10^{-1}
6566.80	5.576×10^{-1}	1.9549	0.0	0.0	5.576×10^{-1}	1.9549	7.370×10^{-1}	6.101×10^{-3}	0.0	0.0	5.576×10^{-1}	1.9549	1.22×10^{-1}	4.279×10^{-1}
7566.80	6.756×10^{-1}	2.4289	0.0	0.0	6.756×10^{-1}	2.4289	9.230×10^{-1}	7.533×10^{-1}	0.0	0.0	6.756×10^{-1}	2.4289	1.269×10^{-1}	4.636×10^{-1}
8566.80	8.354×10^{-1}	3.069	0.0	0.0	8.354×10^{-1}	3.069	1.182	9.507×10^{-3}	0.0	0.0	8.354×10^{-1}	3.069	1.346×10^{-1}	4.946×10^{-1}
9566.80	1.0065	3.770	0.0	0.0	1.0065	3.770	1.429	1.148×10^{-2}	0.0	0.0	1.0065	3.770	1.403×10^{-1}	5.254×10^{-1}
10485.30	1.1599	4.4149	0.0	0.0	1.1599	4.4149	1.644	1.321×10^{-2}	0.0	0.0	1.1599	4.4149	1.457×10^{-1}	5.546×10^{-1}

Water discharge at upstream of barrage (m^3/s)	Available head, h(m)	
	Total discharge downstream	Required flow through the main canal
100	0.8150	1.0163
1,000	0.0410	0.1732
9,918.5	0.3290	0.3542

Now it is observed that the available head,h is always less than the clear water head loss of 1.314m. Due to non availability of net head sediment excluder under existing condition cannot function and no sediment will move through excluder tunnel (Table 7.6).

b) Design Condition

To eliminate bedload entry into the offtaking canal, operating head of sediment excluder should be greater than the clear water head loss of the tunnel. This can be made only by raising the pond level in such a way that the head difference between the upstream and downstream water level is greater than the clear water head loss in the excluder tunnel. To have greater velocity of flow at the upstream of barrage, the upstream floor levels should not be much lower. The upstream floor level of weir portion and undersluice portion should be the average deeper channel and 1m below the average deeper channel level respectively. For the design condition (Table 7.2) analysis is

Table 7.7 Tunnel Dimensions of Designed Excluder (CWPC Type)

Tunnel	Bellmouth				Contraction				Bend					Straight		
	Length	Width u/s	Width d/s	Depth	Length	Width u/s	Width d/s	Depth	Length	Radius	Width	Depth	Angle of deviation	Length	Width	Depth
	m	m	m	m	m	m	m	m	m	m	m	m		m	m	m
1	6.1	6.32	3.4	1.677	136.42	3.4	2	1.677	1.40	30	2	1.677	2.675	26.43	2	1.677
2	6.25	6.32	3.3	1.677	129.85	3.3	2	1.677	1.136	30	2	1.677	2.17	26.56	2	1.677
3	3.66	4.99	3.2	1.677	126.10	3.2	2	1.677	0.87	30	2	1.677	1.665	26.69	2	1.677
4	6.1	6.10	3.1	1.677	118.76	3.1	2	1.677	0.61	30	2	1.677	1.165	26.82	2	1.677
5	6.25	6.10	3.0	1.677	112.62	3.0	2	1.677	0.36	30	2	1.677	0.69	26.95	2	1.677
6	3.66	4.73	2.9	1.677	109.28	2.9	2	1.677	0.12	30	2	1.677	0.23	27.07	2	1.677

Average length of tunnel, L = 155.01m

Table 7.8 Head Loss in the Designed Excluder (CWPC Type)

Tunnel	h_f				h_{en}	h_c		h_b	h_{ex}	h_o	U_{EX}	U_{EX}	Q_{EX}
	Bellmouth	Transition	Bend	Straight		Bellmouth	Contraction				Average	Maximum	
	m	m	m	m	m	m	m	m	m	m	m/s	m/s	m^3/s
1	0.0044	0.4103	0.0091	0.1713	0.0233	0.0111	0.0296	0.0016	0.2334	0.8941	2.14	2.98	10
2	0.0046	0.4093	0.0074	0.1722	0.0238	0.0121	0.0287	0.0013	0.2378	0.8972	2.16	2.98	10
3	0.0040	0.4170	0.0056	0.1730	0.0256	0.0104	0.0276	0.0010	0.2557	0.9199	2.24	2.98	10
4	0.0050	0.4125	0.0040	0.1739	0.0247	0.0140	0.0264	0.0007	0.2467	0.9079	2.20	2.98	10
5	0.0053	0.4113	0.0023	0.1747	0.0249	0.0153	0.0252	0.0004	0.2489	0.9083	2.21	2.98	10
6	0.0047	0.4201	0.0008	0.1755	0.0277	0.0134	0.0238	0.0001	0.2767	0.9428	2.33	2.98	10

Average total head loss, h_o = 0.9117m.

Table 7.9 Parameters for the Analysis of Entrainment of Sediment into the Excluder Tunnel and into the Main Canal, Under Design Condition

Q_{RO}	Q_R	Q_C	Q_{EX}	Q_P	D	U	h (total discharge dowsnstream)	h (required flow through main canal)	B_{EX} and no. of tunnel	h_o
m^3/s	m^3/s	m^3/s	m^3/s	m^3/s	m	m/s	m	m	m	m
163.90	100.00	59.0	41	100.00	5.60	0.037	2.5857	2.7870	8m 4 No	0.9048
327.87	200.00	118.0	60	178.0	"	0.074	2.3853	2.6355	12m 6 No	0.9117
491.80	300.00	177.0	"	237.0	"	0.112	2.2462	2.5304	"	"
655.74	400.00	226.7	"	286.7	"	0.149	2.1362	2.4304	"	"
983.60	600.00	"	"	"	"	0.223	1.9632	2.1635	"	"
1566.80	1000.00	"	"	"	"	0.372	1.7115	1.8432	"	"
2566.80	2000.00	"	"	"	"	0.745	1.2985	1.3768	"	"
3566.80	3000.00	"	"	"	"	1.117	1.0118	1.0703	"	"
4566.80	4000.00	"	"	"	"	1.489	0.7851	0.8327	"	"
5566.80	5000.00	"	"	"	"	1.862	0.5946	0.6353	"	"
6766.80	6200.00*	"	"	1355.0	"	2.309	0.3978	0.4329	"	"
7566.80	7000.00	"	"	1526.0	5.80	2.484	0.4808	0.5130	"	"
8566.80	8000.00	"	"	1751.0	6.07	2.670	0.6168	0.6462	"	"
9566.80	9000.00	"	"	1934.0	6.33	2.841	0.7539	0.7809	"	"
10485.30	9918.50	"	"	2104.0	6.565	2.985	0.8840	0.9092	"	"

$Q_{CI} = 0.39 * Q_{RO} < 566.8\ m^3/s$ $\quad Q_{EX} = (Q_R - Q_C) < 60\ m^3/s$

$Q_R = (Q_{RO} - Q_{CI})\ m^3/s$ $\quad Q_P = (Q_{EX} + Q_C)\ m^3/s$, when $Q_R - (Q_{EX} + Q_C) < Q_W$

$Q_C = 0.36 * Q_{RO} < 226.7\ m^3/s$ $\quad Q_P = Q_U\ m^3/s$, when $Q_R - (Q_{EX} + Q_C) > Q_W$

*Pond level discharge(Q_U=1355m^3/s and Q_W=4845m^3/s)

Table 7.10 Entrainment of Sediment into the Excluder Tunnel and into the Main Canal, Under Design Condition

Q_{RO} m^3/s	Q_{BPI} m^3/s	Q_{SPI} m^3/s	Q_{BPD} m^3/s	Q_{SPD} m^3/s	$Q_{BP}=Q_{BT}$ m^3/s	Q_{SP} m^3/s	Q_{ST} m^3/s	C_{EX} m^3/s	C_T m^3/s	Q_T m^3/s	Q_{BPB} m^3/s	Q_{SPS} m^3/s	Q_{BC} m^3/s	Q_{SC} m^3/s
163.90	5.98×10^{-3}	1.07×10^{-2}	5.98×10^{-3}	1.07×10^{-2}	0.0	0.0	0.0	0.0	1.416×10^{-1}	0.0	0.0	0.0	0.0	0.0
327.87	1.499×10^{-2}	3.013×10^{-2}	1.499×10^{-2}	3.013×10^{-2}	0.0	0.0	0.0	0.0	$1.301*10^{-1}$	0.0	0.0	0.0	0.0	0.0
491.80	2.438×10^{-2}	5.235×10^{-2}	2.438×10^{-2}	5.235×10^{-2}	0.0	0.0	0.0	0.0	$1.222*10^{-1}$	0.0	0.0	0.0	0.0	0.0
655.74	3.399×10^{-2}	7.65×10^{-2}	3.33×10^{-2}	7.38×10^{-2}	6.8×10^{-4}	2.68×10^{-3}	0.0	1.133×10^{-5}	$1.146*10^{-1}$	6.798×10^{-4}	0.0	2.68×10^{-3}	0.0	2.68×10^{-3}
983.60	4.15×10^{-2}	9.986×10^{-2}	2.739×10^{-2}	3.395×10^{-2}	1.411×10^{-2}	6.591×10^{-2}	2.61×10^{-3}	2.787×10^{-4}	$9.447*10^{-2}$	1.672×10^{-2}	0.0	6.33×10^{-2}	0.0	6.33×10^{-2}
1566.80	5.34×10^{-2}	1.397×10^{-1}	1.335×10^{-3}	1.397×10^{-3}	5.207×10^{-2}	1.383×10^{-1}	1.743×10^{-2}	1.158×10^{-3}	$7.024*10^{-2}$	6.948×10^{-2}	0.0	1.209×10^{-1}	0.0	1.209×10^{-1}
2566.80	7.518×10^{-2}	2.202×10^{-1}	0.0	0.0	7.518×10^{-2}	2.202×10^{-1}	6.386×10^{-2}	2.317×10^{-3}	$3.517*10^{-2}$	1.39×10^{-1}	0.0	1.563×10^{-1}	0.0	1.563×10^{-1}
3566.80	9.18×10^{-2}	2.87×10^{-1}	0.0	0.0	9.18×10^{-2}	2.87×10^{-1}	1.134×10^{-1}	3.42×10^{-3}	$1.203*10^{-2}$	2.052×10^{-1}	0.0	1.736×10^{-1}	0.0	1.736×10^{-1}
4566.80	1.06×10^{-1}	3.46×10^{-1}	0.0	0.0	1.06×10^{-1}	3.46×10^{-1}	1.583×10^{-1}	4.405×10^{-3}	0.0	0.0	1.06×10^{-1}	3.46×10^{-1}	1.06×10^{-1}	3.46×10^{-1}
5566.80	1.18×10^{-1}	4.02×10^{-1}	0.0	0.0	1.18×10^{-1}	4.02×10^{-1}	1.99×10^{-1}	5.28×10^{-3}	0.0	0.0	1.18×10^{-1}	4.02×10^{-1}	1.18×10^{-1}	4.02×10^{-1}
6766.80	6.208×10^{-1}	2.188	0.0	0.0	6.208×10^{-1}	2.188	1.141	2.936×10^{-2}	0.0	0.0	6.208×10^{-1}	2.188	1.087×10^{-1}	3.83×10^{-1}
7566.80	7.42×10^{-1}	2.669	0.0	0.0	7.42×10^{-1}	2.669	1.358	3.5×10^{-2}	0.0	0.0	7.42×10^{-1}	2.669	1.147×10^{-1}	4.127×10^{-1}
8566.80	9.097×10^{-1}	3.343	0.0	0.0	9.097×10^{-1}	3.343	1.658	4.28×10^{-2}	0.0	0.0	9.097×10^{-1}	3.343	1.22×10^{-1}	4.482×10^{-1}
9566.80	1.065	3.989	0.0	0.0	1.065	3.989	1.869	4.89×10^{-2}	0.0	0.0	1.065	3.989	1.29×10^{-1}	4.826×10^{-1}
10485.30	1.215	4.626	0.0	0.0	1.215	4.626	2.161	5.63×10^{-2}	0.0	0.0	1.215	4.626	1.348×10^{-1}	5.13×10^{-1}

carried out. Appendix-3 and Appendix-4 contain the analysis procedure for developing curves (Figures 7.14 to 7.21) and Table 7.10 for Teesta Headworks. Tables 7.7 to 7.9 contain data for the analysis obtained after proper calculation. For 75% dependable discharge, bed material load deposited in the pocket becomes 2.25 million tons (Table 7.14) of which 65% is for suspended load and 35% is for bedload deposition. Entrainment of suspended load into the main canal is found to be 2.54 million tons (Table 7.14) of which the percentage of different grain-size ranges are shown below:

Particle range	Percent
0.074 to 0.15mm	57.6
0.15 to 0.30mm	41.0
0.30 to 0.60mm	1.4
> 0.60mm	0.0

c) Suggested Condition

The calculation shows that the excluder will not function due to non-availability of flow head. The author has suggested (Table 7.2) the raising of the pond level at + 53.6m instead of + 51.8293m. The construction of the excluder containing 12 tunnels is near about completion but for excluder discharge of $60m^3/s$ instead of $201.2m^3/s$, the calculation shows that so many tunnels would not be necessary. It is also suggested (Table 7.2) to take only 5 tunnels of existing tunnels no.1,2,4,6 and 8 to be

Table 7.11 Head Loss in the Excluder Tunnel, Under Suggested Condition

Tunnel no. Suggested	Tunnel No. Existing	h_f Bell mouth	h_f Straight	h_f Bend	h_f Straight	h_f Contraction	h_{en}	h_c Bell mouth	h_c Contraction	h_b	h_{ex}	h_o	U_{EX} Average	U_{EX} Maximum	Q_{EX}
		m	m	m	m	m	m	m	m	m	m	m	m/s	m/s	m^3/s
1	1	.0020	.3132	.0045	.0364	.0282	.0229	.0167	.0208	.0328	.2290	.7065	2.1199	2.9709	15.19
2	2	.0017	.3763	.0056	.0470	.0348	.0265	.0096	.0208	.0279	.2652	.8154	2.2812	2.9709	10.63
3	4	.0037	.3059	.0056	.0492	.0348	.0226	.0172	.0208	.0279	.2264	.7141	2.1075	2.9709	10.63
4	6	.0024	.2483	.0055	.0511	.0326	.0256	.0116	.0208	.0292	.2558	.6829	2.2401	2.9709	11.77
5	8	.0037	.1678	.0052	.0508	.0326	.0230	.0165	.0208	.0292	.2304	.5800	2.1260	2.9709	11.77

Average head loss, h_o = 0.6998m.

Table 7.12 Parameters for the Analysis of Entrainment of Sediment into the Excluder Tunnel and into the Main Canal, Under Suggested Condition.

Q_{RO}	Q_R	Q_C	Q_{EX}	Q_P	D	U	h (total discharge downstream)	h (required flow through main canal)	B_{EX} and no. of tunnel	h_o
m^3/s	m^3/s	m^3/s	m^3/s	m^3/s	m	m/s	m	m	m	m
163.9	100	59	41	100	7.259	.0316	2.5857	2.7870	9.68m 4 No.	.7297
327.87	200	118	60	178	"	.0633	2.3853	2.6355	12.043m 5 No.	.6998
491.80	300	177	"	237	"	.0949	2.2462	2.5304	"	"
655.74	400	226.7	"	2867	"	.1265	2.1362	2.4304	"	"
983.60	600	"	"	"	"	.1898	1.9632	2.1635	"	"
1566.80	1000	"	"	"	"	.3164	1.7115	1.8432	"	"
2566.80	2000	"	"	"	"	.6327	1.2985	1.3768	"	"
3566.80	3000	"	"	"	"	.9491	1.0118	1.0703	"	"
4566.80	4000	"	"	"	"	1.2655	0.7851	0.8327	"	"
5566.80	5000	"	"	"	"	1.5818	0.5946	0.6353	"	"
6566.80	6000	"	"	"	"	1.8982	0.4287	0.4645	"	"
7966.80	7400*	"	"	1502	"	2.3411	0.2258	0.2567	"	"
8566.80	8000	"	"	1616	7.369	2.4740	0.2568	0.2862	"	"
9566.80	9000	"	"	1806	7.639	2.6378	0.4039	0.4309	"	"
10485.30	9918.5	"	"	1967	7.869	2.7831	0.5290	0.5542	"	"

$Q_{CI} = 0.39 * Q_{RO} < 566.8\ m^3/s$

$Q_R = (Q_{RO} - Q_{CI})\ m^3/s$

$Q_C = 0.36 * Q_{RO} < 226.7\ m^3/s$

$Q_{EX} = (Q_R - Q_C) < 60\ m^3/s$

$Q_P = (Q_{EX} + Q_C)\ m^3/s$, when $Q_R - (Q_{EX} + Q_C) < Q_W$

$Q_P = Q_U\ m^3/s$, when $Q_R - (Q_{EX} + Q_C) > Q_W$

*Pond level discharge ($Q_U = 1505\ m^3/s$ and $Q_W = 5895\ m^3/s$)

Table 7.13 Entrainment of Sediment Load into the Excluder Tunnel and into the Main Canal, Under Suggested Condition

Q_{RO} m^3/s	Q_{BPI} m^3/s	Q_{SPI} m^3/s	Q_{BPD} m^3/s	Q_{SPD} m^3/s	$Q_{BP}=Q_{BT}$ m^3/s	Q_{SP} m^3/s	Q_{ST} m^3/s	C_{EX} m^3/s	C_T m^3/s	Q_T m^3/s	qBPB m^3/s	Q_{SPS} m^3/s	Q_{BC} m^3/s	Q_{SC} m^3/s
163.90	5.98×10^{-3}	1.07×10^{-2}	5.98×10^{-3}	1.07×10^{-2}	0.0	0.0	0.0	0.0	1.903×10^{-1}	0.0	0.0	0.0	0.0	0.0
327.87	1.499×10^{-2}	3.013×10^{-2}	1.499×10^{-2}	3.013×10^{-2}	0.0	0.0	0.0	0.0	1.867×10^{-1}	0.0	0.0	0.0	0.0	0.0
491.80	2.438×20^{-2}	5.235×10^{-2}	2.438×10^{-2}	5.235×10^{-2}	0.0	0.0	0.0	0.0	1.766×10^{-1}	0.0	0.0	0.0	0.0	0.0
655.74	3.399×10^{-2}	7.65×10^{-2}	3.399×10^{-2}	7.65×10^{-2}	0.0	0.0	0.0	0.0	1.669×10^{-1}	0.0	0.0	0.0	0.0	0.0
983.60	4.15×10^{-2}	9.986×10^{-2}	3.53×10^{-2}	6.391×10^{-2}	6.23×10^{-2}	3.595×10^{-2}	0.0	1.038×10^{-4}	1.412×10^{-1}	6.228×10^{-3}	0.0	3.595×10^{-2}	0.0	3.595×10^{-2}
1566.80	5.34×10^{-2}	1.397×10^{-1}	7.48×10^{-3}	8.103×10^{-3}	4.59×10^{-2}	1.316×10^{-1}	4.606×10^{-1}	8.418×10^{-4}	1.103×10^{-1}	5.051×10^{-2}	0.0	1.270×10^{-1}	0.0	1.27×10^{-1}
2566.80	7.518×10^{-2}	2.202×10^{-1}	0.0	0.0	7.518×10^{-2}	2.202×10^{-1}	2.488×10^{-2}	1.668×10^{-3}	6.531×10^{-2}	1.001×10^{-1}	0.0	1.953×10^{-1}	0.0	1.953×10^{-1}
3566.80	9.18×10^{-2}	2.87×10^{-1}	0.0	0.0	9.18×10^{-2}	2.87×10^{-1}	6.027×10^{-2}	2.535×10^{-3}	3.575×10^{-2}	1.521×10^{-1}	0.0	2.267×10^{-1}	0.0	2.267×10^{-1}
4566.80	1.06×10^{-1}	3.46×10^{-1}	0.0	0.0	1.06×10^{-1}	3.46×10^{-1}	9.446×10^{-2}	3.341×10^{-3}	1.282×10^{-2}	2.005×10^{-1}	0.0	2.515×10^{-1}	0.0	2.515×10^{-1}
5566.80	1.18×10^{-1}	4.02×10^{-1}	0.0	0.0	1.18×10^{-1}	4.02×10^{-1}	1.286×10^{-1}	4.11×10^{-3}	0.0	0.0	1.18×10^{-1}	4.02×10^{-1}	1.18×10^{-1}	4.02×10^{-1}
6566.80	1.29×10^{-1}	4.53×10^{-1}	0.0	0.0	1.29×10^{-1}	4.53×10^{-1}	1.586×10^{-1}	4.793×10^{-3}	0.0	0.0	1.29×10^{-1}	4.53×10^{-1}	1.29×10^{-1}	4.53×10^{-1}
7966.80	7.509×10^{-1}	2.724	0.0	0.0	7.509×10^{-1}	2.724	1.035	2.977×10^{-2}	0.0	0.0	7.509×10^{-1}	2.724	1.18×10^{-1}	4.282×10^{-1}
8566.80	8.396×10^{-1}	3.085	0.0	0.0	8.396×10^{-1}	3.085	1.203	3.404×10^{-2}	0.0	0.0	8.396×10^{-1}	3.085	1.22×10^{-1}	4.495×10^{-1}
9566.80	9.944×10^{-1}	3.725	0.0	0.0	9.944×10^{-1}	3.725	1.453	4.079×10^{-2}	0.0	0.0	9.944×10^{-1}	3.725	1.29×10^{-1}	4.837×10^{-1}
10485.3	1.136	4.325	0.0	0.0	1.136	4.325	1.708	4.74×10^{-2}	0.0	0.0	1.136	4.325	1.35×10^{-1}	5.141×10^{-1}

Table 7.14 Entrainment of Sediment Discharge into the Main Canal for 10 Day Average 75% Dependable Discharge.

				Existing Condition				Design Condition				Suggested Condition			
Sediment discharge of the river (Hectare-meter per year)	Suspended load, Q_s	850 (22.52*10^6 tons)	1170 (31.00*10^6 tons)	Sediment size range group											
	Bed Load, Q_B	320 (8.48*10^6 tons)		0.074 to 0.15 (mm)	0.15 to 0.30 (mm)	0.30 to 0.60 (mm)	>0.60 (mm)	0.074 to 0.15 (mm)	0.15 to 0.30 (mm)	0.30 to 0.60 (mm)	>0.60 (mm)	0.074 to 0.15 (mm)	0.15 to 0.30 (mm)	0.30 to 0.60 (mm)	>0.60 (mm)
Sediment discharge deposited in the pocket (Hectare-meter per year)	Suspended load deposition in the pocket, Q_{SPD}			36.47 (9.66*10^5 Tons)				55.42 (1.47*10^6 Tons)				72.38 (1.92*10^6 Tons)			
	Bed load deposition in the pocket, Q_{BPD}			24.90 (6.60*10^5 Tons)				29.24 (7.75*10^5 Tons)				36.90 (9.78*10^5 Tons			
Entrainment of sediment discharge in the main canal (Hectare-meter per year)	Suspended load entry into the canal, Q_{SC}			179.3(4.75*10^6 Tons)				95.80(2.54*10^6 Tons)				86(2.28*10^6 Tons)			
				75.3	75.3	25.1	3.6	55.2	39.3	1.3	0	83.7	2.3	0	0
				42%	42%	14%	2%	57.6%	41%	1.4%	0	97.3%	2.7%	0	0
	Bed load entry into the canal, Q_{BC}			65.9(1.75*10^6 Tons)				0				0			
				13.28	30.65	19.88	2.09	0	0	0	0	0	0	0	0
				20%	47%	30%	3%	0	0	0	0	0	0	0	0

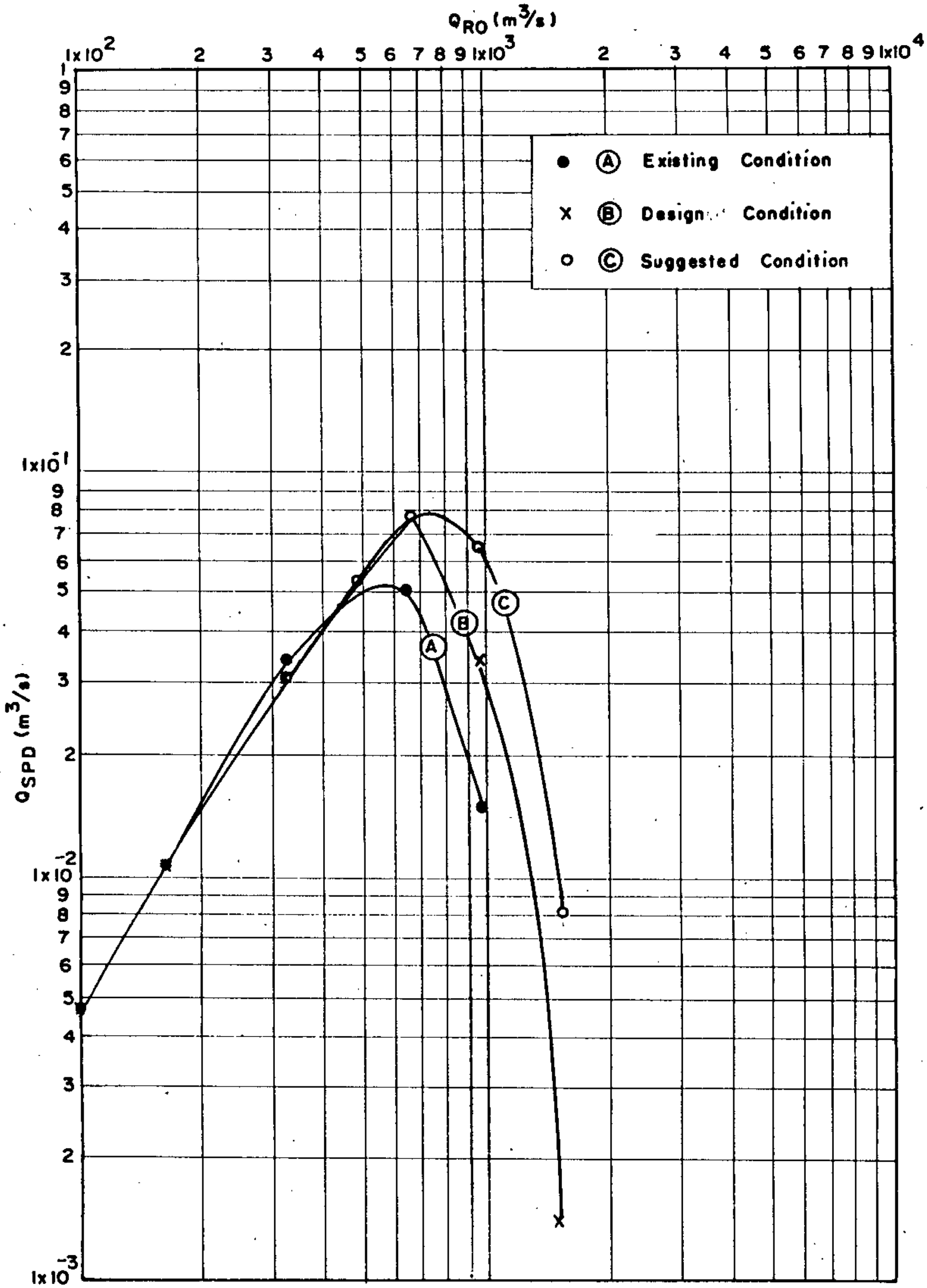


Fig. 7.14 DEPOSITION OF SUSPENDED LOAD DISCHARGE IN THE POCKET.

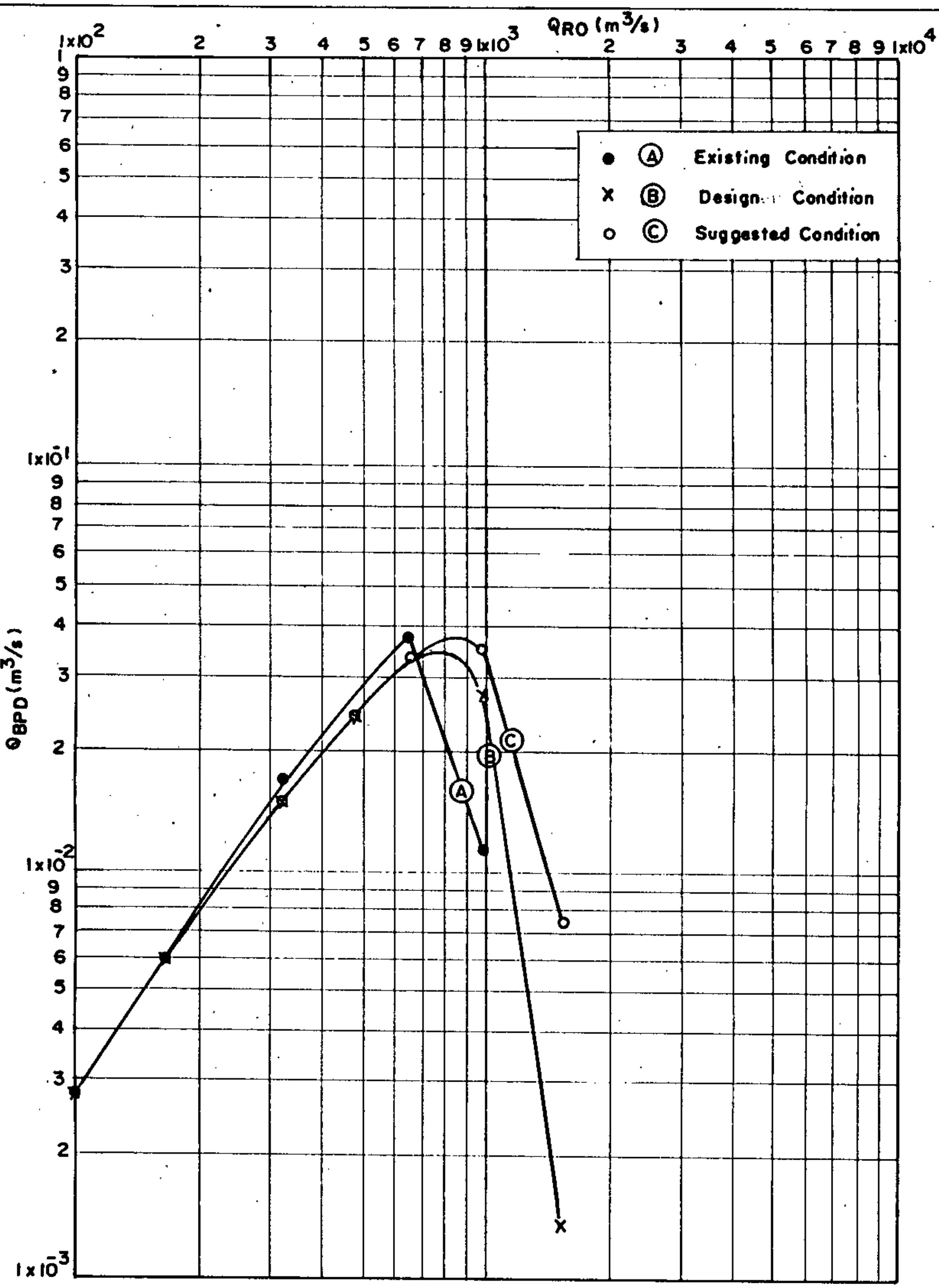


Fig. 7.15 DEPOSITION OF BEDLOAD DISCHARGE IN THE POCKET.

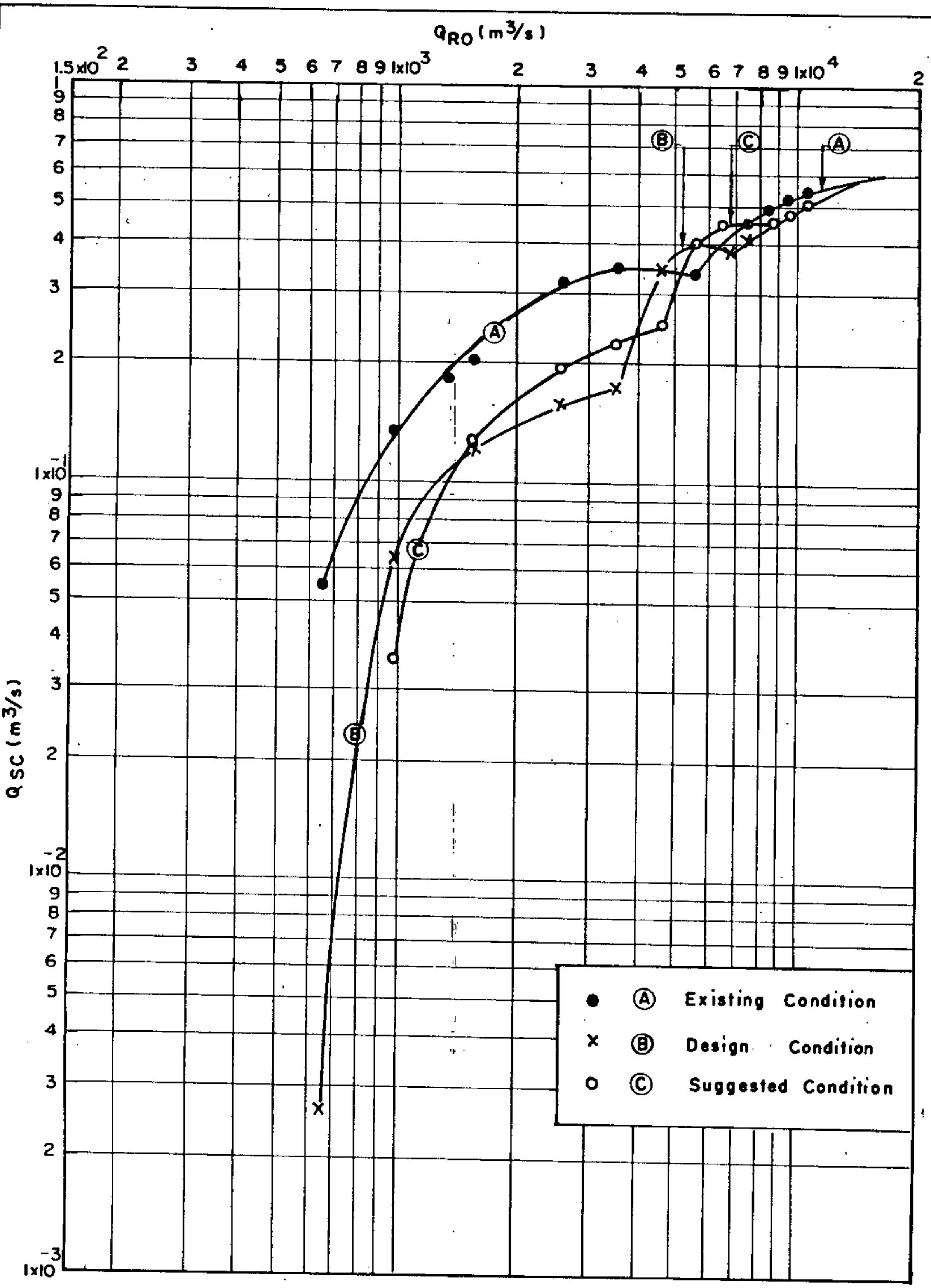


Fig. 7.16 ENTRAINMENT OF SUSPENDED LOAD DISCHARGE INTO THE MAIN CANAL.

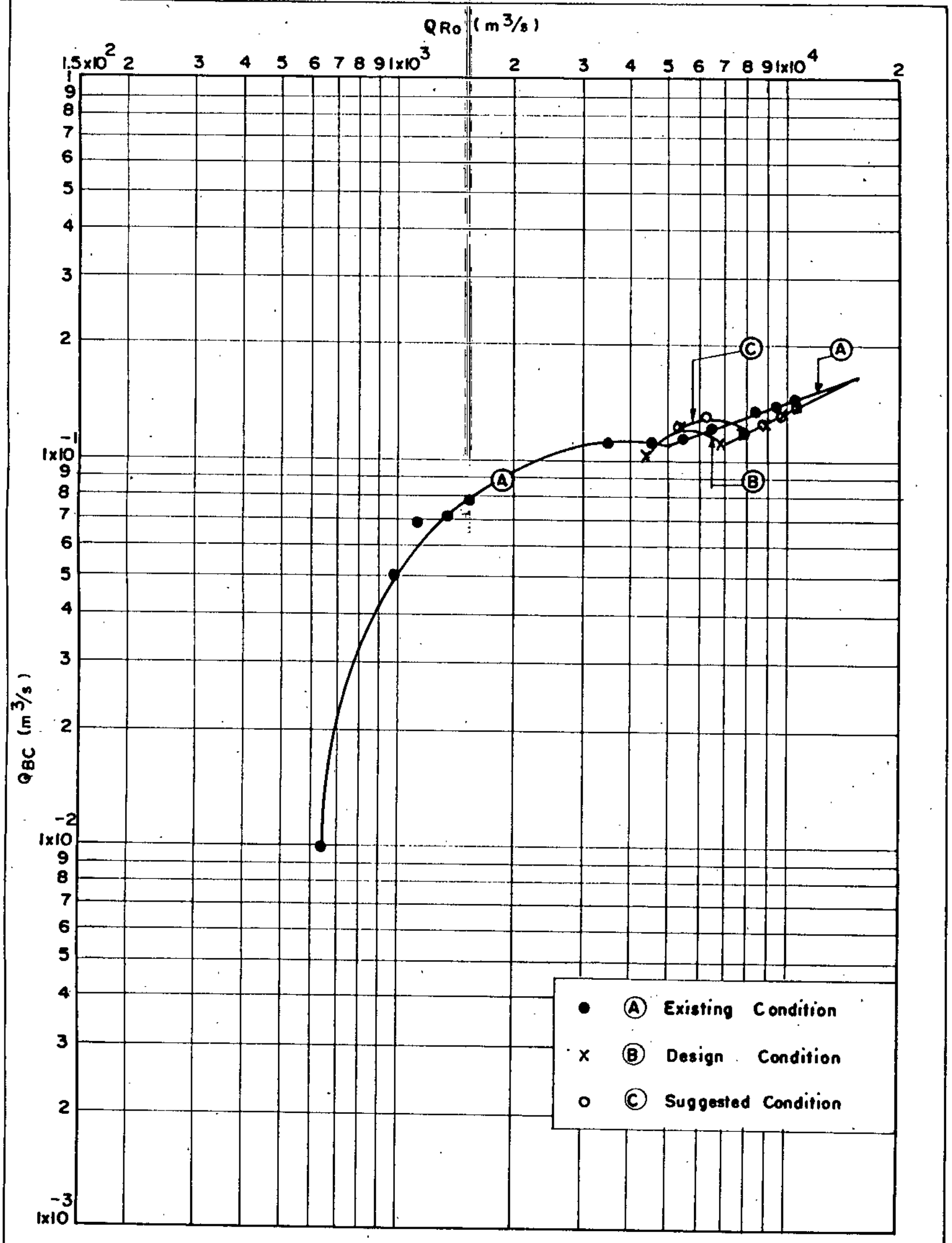


Fig. 7.17 ENTRAINMENT OF BEDLOAD DISCHARGE INTO THE MAIN CANAL.

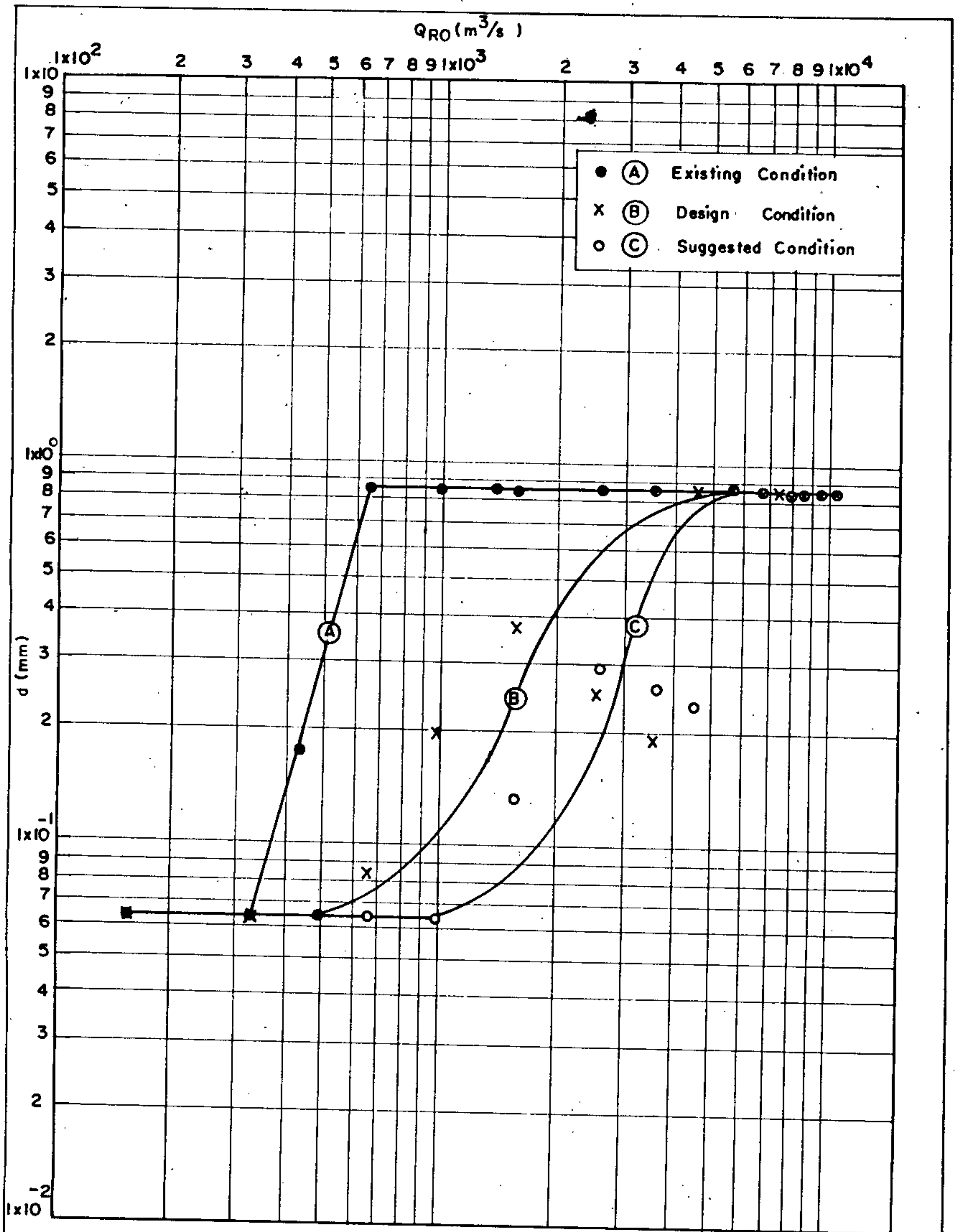


Fig. 7.18 ENTRAINMENT OF MAXIMUM GRAIN SIZE OF SUSPENDED LOAD INTO THE MAIN CANAL.

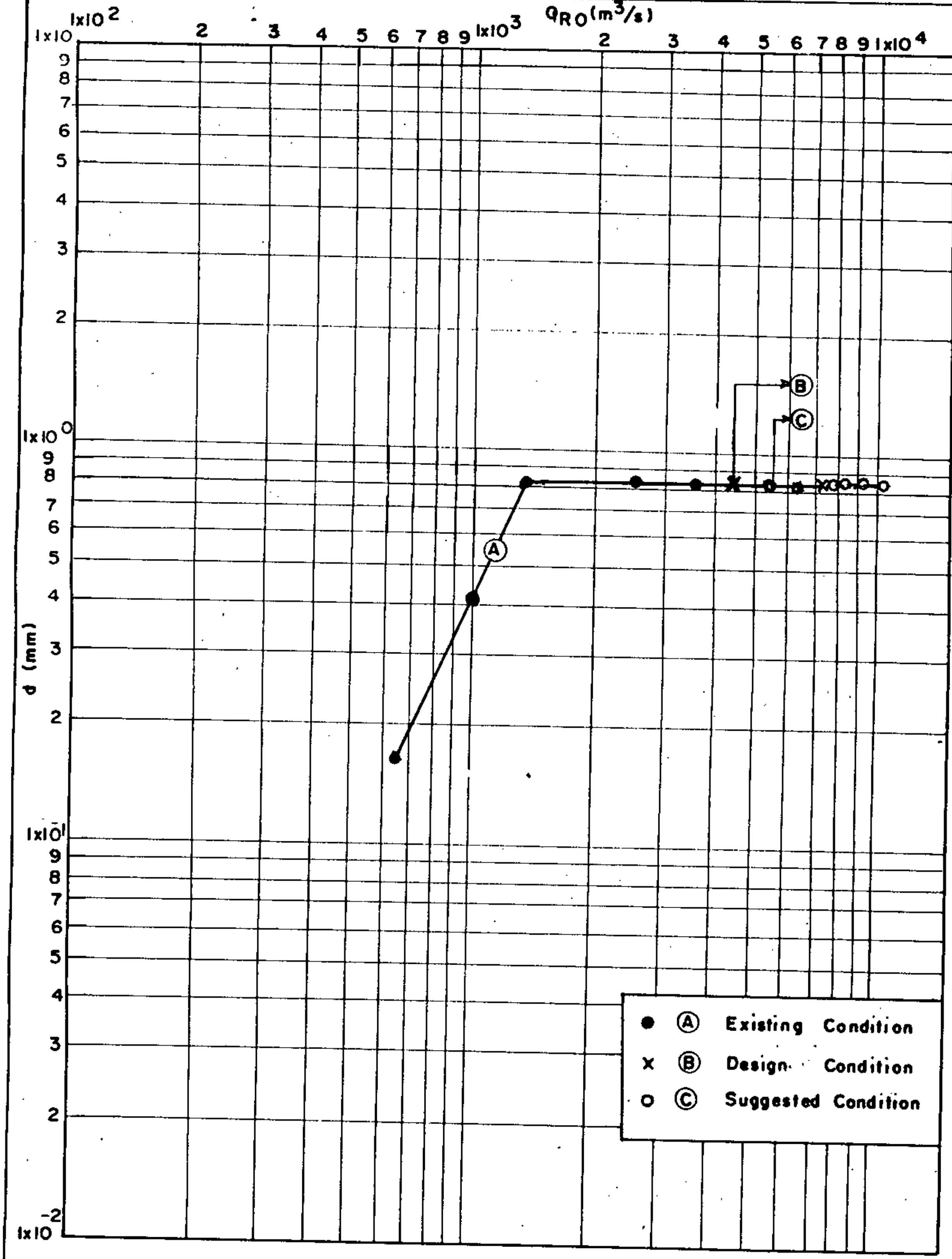


Fig. 7.19 ENTRAINMENT OF MAXIMUM GRAIN SIZE OF BEDLOAD INTO THE MAIN CANAL.

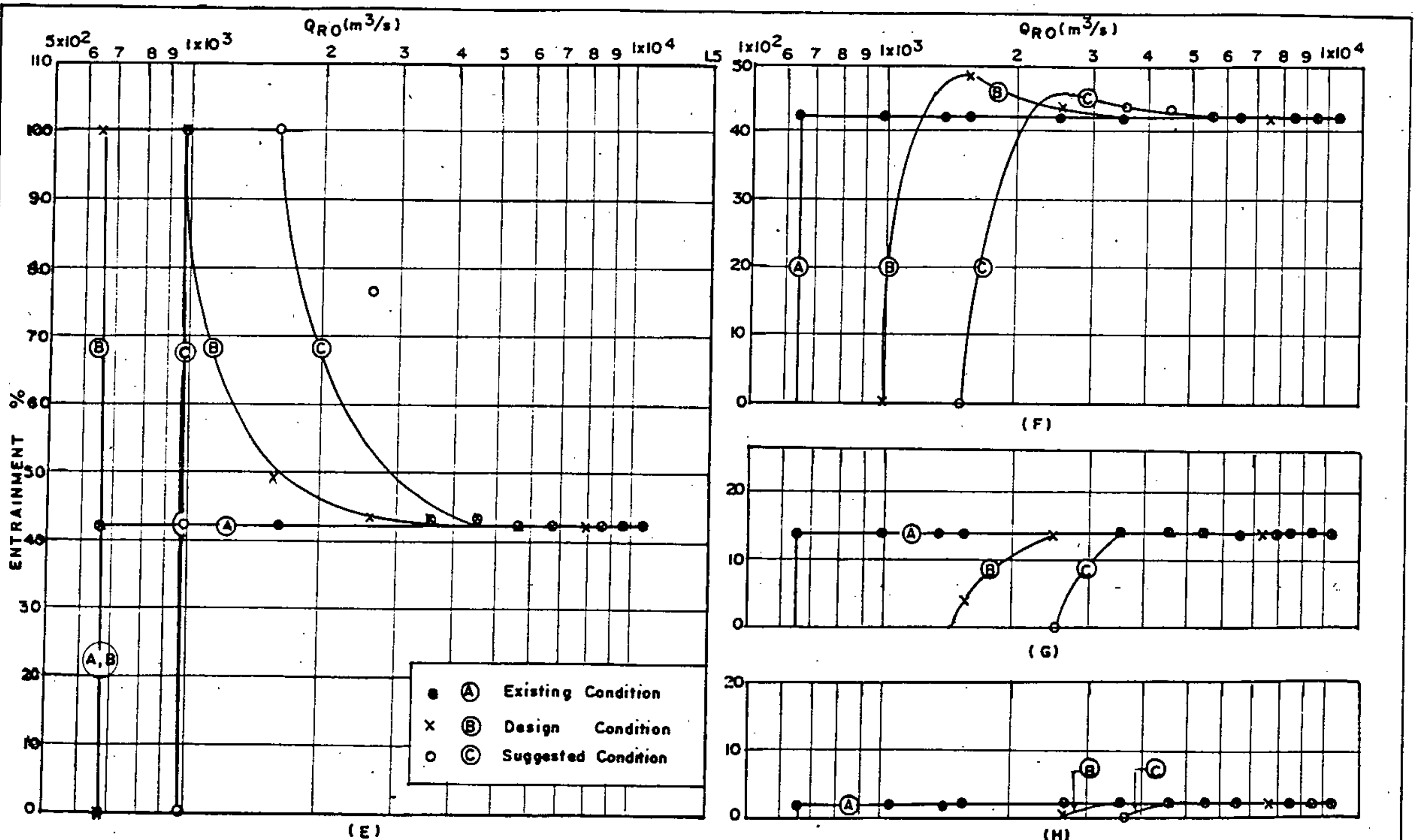


Fig. 7.20 (E), (F), (G) AND (H) SHOWS ENTRAINMENT PERCENTAGE OF GRAIN SIZE RANGE (0.074 TO 0.15mm), (0.15mm TO 0.30mm), (0.30mm TO 0.60mm) AND (>0.60mm) RESPECTIVELY OF SUSPENDED LOAD INTO THE MAIN CANAL.

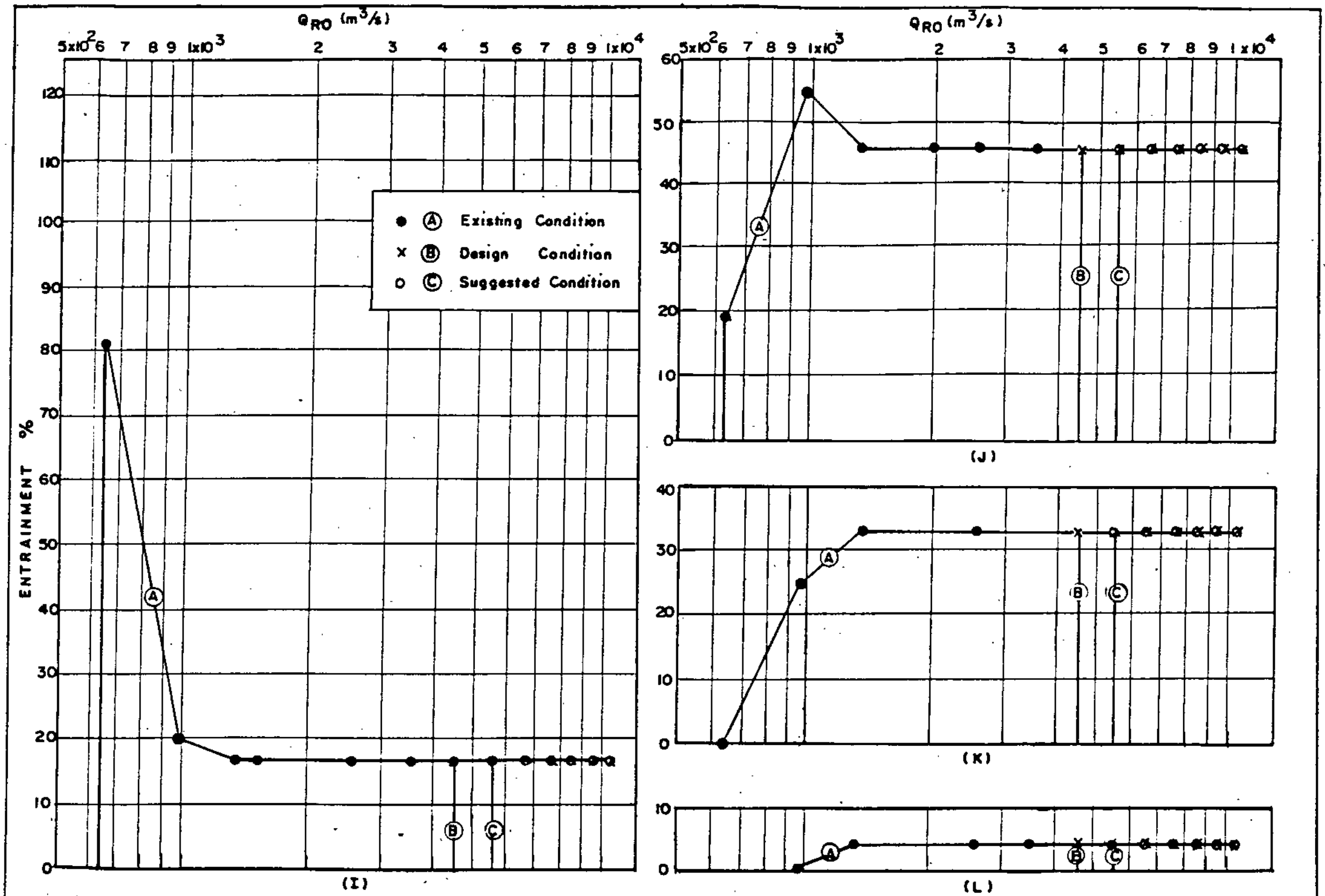


Fig. 7.21 (I), (J), (K) AND (L) SHOWS ENTRAINMENT PERCENTAGE OF GRAIN SIZE RANGE (0.074 TO 0.15mm), (0.15 TO 0.30mm), (0.30 TO 0.60mm) AND (>0.60 mm) RESPECTIVELY OF BEDLOAD INTO THE MAIN CANAL.

in operation by blocking all other tunnels. Analyses for the deposition of sediment in the pocket and entrainment of sediment into the main canal are carried out and are shown in Tables 7.11 to 7.13 and in Figures 7.14 to 7.21. Deposition of bed material load in the pocket for 75% dependable discharge becomes 2.9 million tons (Table 7.14) of which 66% is for suspended load and 34% is for bedload deposition. Entrainment of suspended load into the main canal is found to be 2.28 million tons (Table 7.14) of which the percentages of different grain-size ranges are shown below:

Particle range	Percent
0.074 to 0.15mm	97.3
0.15 to 0.30mm	2.7
0.30 to 0.60mm	0.0
> 0.60mm	0.0

7.5 POSSIBILITY OF SEDIMENT EJECTOR IN THE MAIN CANAL

No provision of silt ejector for the Teesta Barrage has been provided for the non-availability of hydraulic head between the pond level and the water level in the river at a distance one mile downstream of barrage site, which is the site for sediment disposal into the main flow of the river.

Under existing condition, full supply level of main canal has been fixed at +51.2195m. To position a sediment ejector in Teesta Main Canal, water level requirement at 1 mile downstream of barrage is

51.2195 (F.S.L. at downstream of head regulator) - 600/12000 (head lose due to slope assuming approach channel length of 600m and slope of 1:12000) - 0.63 (assumed head loss in ejector tunnel) - 1609.76/1800 (head loss due to slope assuming escape channel length of 1 mile and slope of 1:1800) = + 49.645m.

But water level at 1 mile downstream of barrage for every discharge is higher than + 49.645m which is shown below:

Discharge(original)	Water level (1 mile downstream of barrage).
163.9	50.008
819.67	50.515
1556.80	50.952
2066.80	51.217
2587.50	51.426
3066.80	51.583
3566.80	51.725

From the above comparison it is clear that sediment ejector cannot be provided in the Teesta Main Canal due to non-availability of net head. At present it is under construction to have a silt trap from which sediment is to be removed by dredger into the Teesta River. But removal of sediment from the main canal by using dredger is a costly process. If existing condition (Table 7.2) is allowed to maintain in the Teesta Headworks, 6.5 million tons (Table 6.14) of bed material load per year will have to be removed from silt trap by dredger. On the other hand if the suggested condition (Table 7.2) is allowed to incorporate then the removal may be only 2.28 million tons (Table 6.14) instead of 6.5 million tons. In a large irrigation project like Teesta, there is still scope for inclusion of a sediment ejector for removal of silt from the head reach of the main canal.

There is a need to raise the pond level to +53.6m from the present level of +51.8293m. This raised level will also help in the smooth functioning of the silt excluder. Appendix-5 contains the design of sediment ejector for Teesta Main Canal where it is suggested to increase the canal discharge from 226.7m^3/s to 272m^3/s and also to raise the full supply level (at + 53m instead of + 51.2195m) by 1.78m upto the ejector. The bed level of the main canal should also be raised (at +49.45m instead of +47.78) by 1.67m upto the ejector while the crest level of head regulator should be raised (at +51.32m instead of +49.54m) by 1.78m. At the downstream of the sediment ejector on the main canal a fall strcuture is necessary to construct to have a fall of 2.553m. The detail drawings of the sediment ejector is shown in Figures 7.22 to 7.24. Sediment ejector will work efficiently with escape channel slope of 1:1800. The table shown below indicates that for a maximum discharge of 2020.7m^3/s only at the downstream of barrage the sediment ejector with escape channel will work.

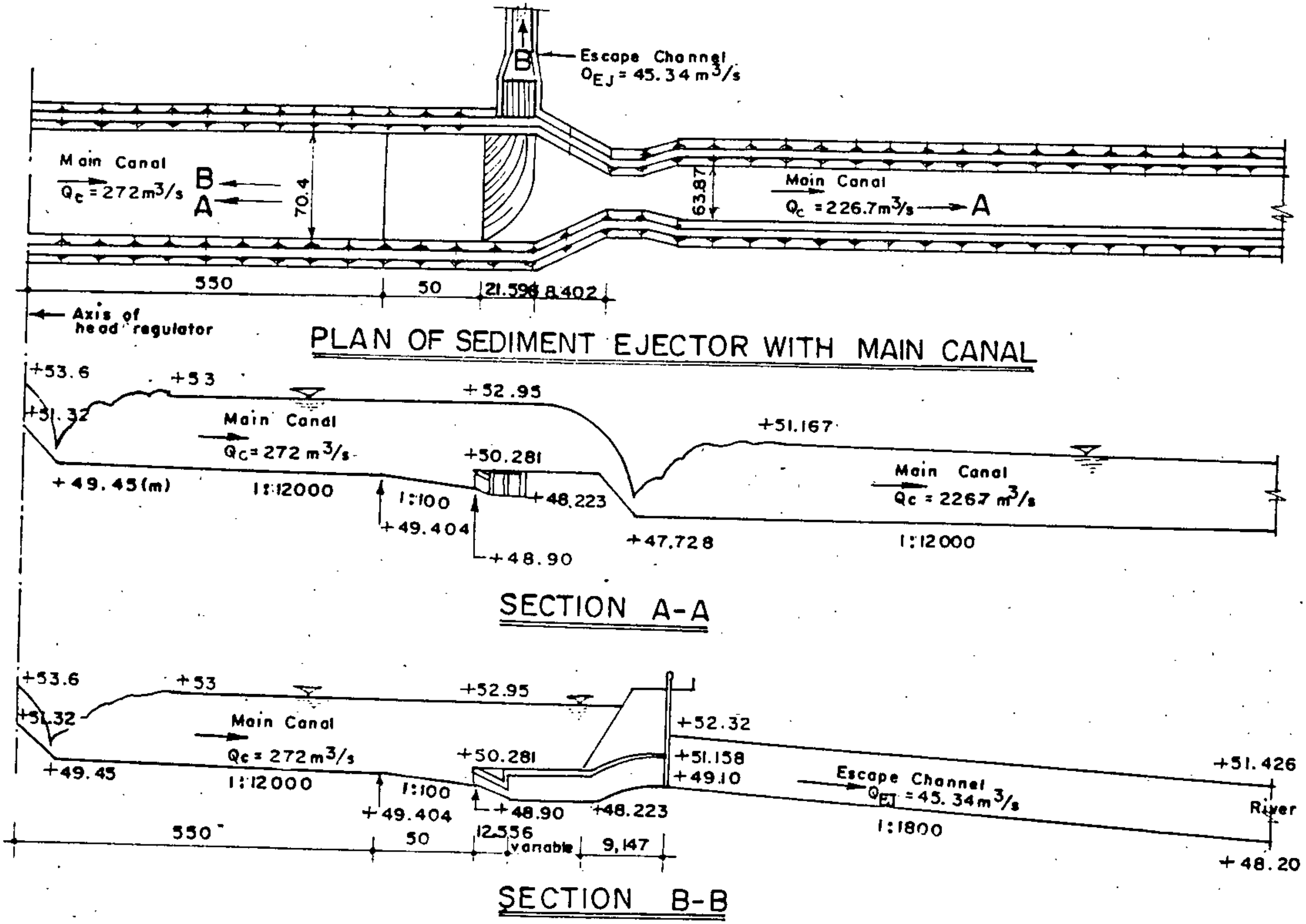


Fig. 7.22 PLAN AND SECTIONS OF SEDIMENT EJECTOR WITH MAIN CANAL AND ESCAPE CHANNEL (DIMENSIONS IN METER)

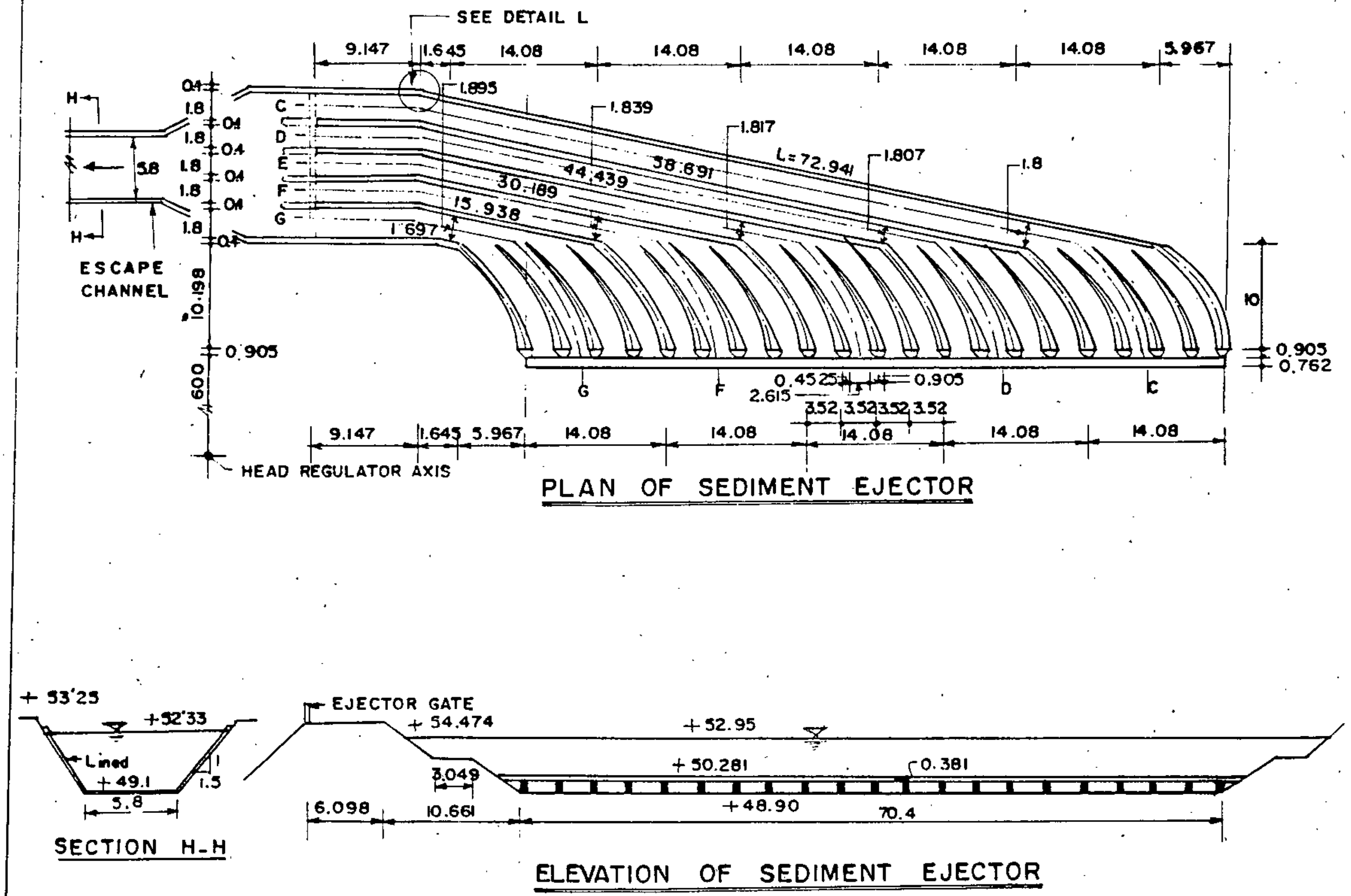


Fig. 7.23 PLAN AND ELEVATION OF SEDIMENT EJECTOR (DIMENSIONS IN METER)

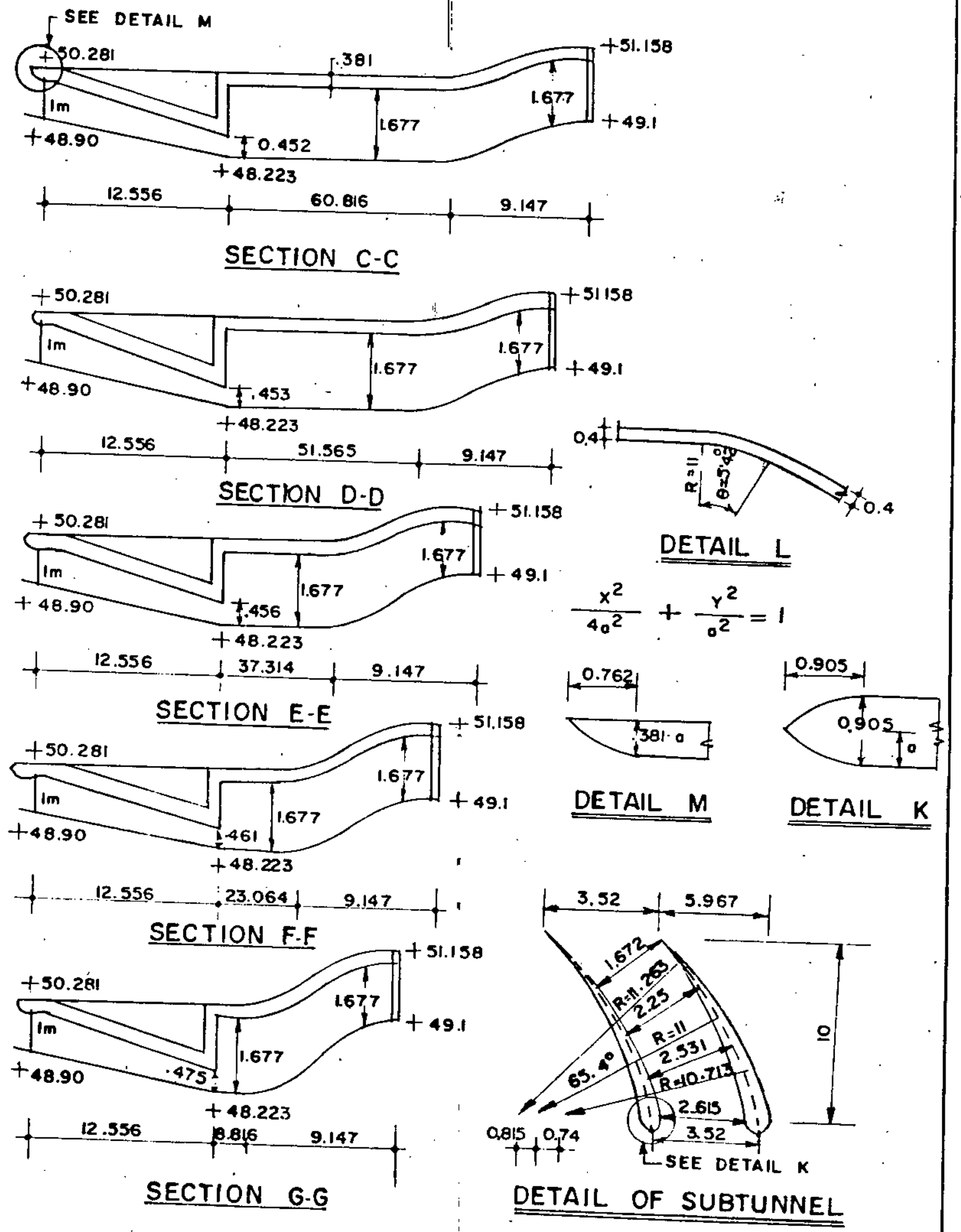


Fig 7.24 SECTIONS AND DETAILS OF SEDIMENT EJECTOR
(DIMENSIONS IN METER)

Discharge original	Discharge at u/s of barrage	Discharge at d/s of barrage	Water level at d/s of ejector	Water level at 1 mile d/s of barrage	Escape channel slope
163.90	100.0	41.0	52.32	50.008	1/696
819.67	500.0	273.3	"	50.515	1/892
1566.80	1000.0	773.3	"	50.952	1/1177
2066.80	1500.0	1273.3	"	51.217	1/1459
2587.50	2020.7	1794.0	"	51.426	1/1800
3066.80	2500.0	2273.3	"	51.583	1/2184
3566.80	3000.0	2773.3	"	51.725	1/2705

CHAPTER VIII

CONCLUSIONS AND RECOMMENDATIONS

8.1 CONCLUSIONS

For a barrage across an alluvial channel where sediment movement is considerable an excluder for sediment bypass is essential. The following salient points clearly emerge as concluding remarks from the study of the present work.

1) Fixation of the levels and dimensions of different parameters of the sediment excluder should be fixed in conjunction with the fixation of the datum for the main barrage parts and the canal headworks. Without synchronisation of the levels of the appurtenant structures of the barrage, the silt excluder may not work efficiently and even works adversely. To avoid the complexity, a complete design procedure of sediment excluder is shown in Appendix-1.

2) In barrages for the case of raising the water level, the pond level should be equal to or slightly less than the downstream highest flood level. The present pond level (+51.8293m) of Teesta Barrage is lowered by 1.85m than the downstream water level (+53.68m) for maximum design discharge. Under the present situation, the sediment excluder may face difficulty in functioning due to nonavailability of net head.

3) Normally the crest level of undersluice should be 1m below the average deeper channel level at barrage site but it has

been kept 2.05m below the average deeper channel level and 0.06m below the recorded lowest bed level. Lowering of crest level of the undersluice to a great extent reduces the flow velocity causing reduction of entrainment of sediment load into the excluder tunnel. This may enhance additional sediment deposition at the upstream of barrage.

4) Width of undersluice pocket between divide wall and canal head regulator for Teesta Barrage seems to smaller compared to the standard design practice (Figure 2.7).

5) When submerged flow condition occurs in a barrage the crest level of head regulator should be 1m to 4m higher than the average deeper channel level. But in Teesta Headworks, crest level of head regulator is only 0.55m above the average deeper channel level at barrage site. As sediment excluder does not work and crest level of head regulator goes much lower entrainment of bed material load into the main canal goes higher by 6.5 million tons, 73% of which is suspended load and 27% is for bedload. Size of particles entering into the main canal varies between 0.074mm to 0.6mm. Only 2.32% of the particles have diameter more than 0.6mm. Particles size range between 0.074mm to 0.15mm, 0.15mm to 0.3mm and 0.3mm to 0.6mm have 36.13, 43.20 and 18.35 percent respectively.

6) The pond level and the full supply level of the main canal have been kept lower than the required level to install a silt ejector behind the head regulator.

8.2 RECOMMENDATIONS

An indepth methodology for the design of sediment excluder and sediment entrainment through the CHR into the main canal has been evolved and may be applied for the barrage projects that will be executed in the coming future.

The following recommendations have been made for Teesta Barrage Project.

1) There is a need to raise the pond level of Teesta Barrage by 1.77m for minimum entrainment of suspended sediment into the main canal. This will also help the excluder for efficient functioning.

2) Instead of using all excluder tunnels only five tunnels (Tunnels 1,2,4,6 and 8) seem to be sufficient to bypass the sediment flow of the Teesta River. This will ensure excluder velocity greater than the critical velocity of settling particles and also reduce churning action and turbulence in the upstream pocket of the undersluice bays.

3) There is a need to modify the shape of the leadcut so that the CHR remains on the concave bend of the river.

4) Semi-still pond method of regulation of the barrage gates can be adopted for Teesta Barrage to minimise entrainment of sediment into the main canal during the normal flow condition of the Teesta River. When the flow at the upstream of barrage exceeds $4000 m^3/s$ excessive sediments (both bed and suspended load) may enter through the CHR into the main canal. For this situation it is better to close the head regulator of the main canal for such high discharges.

5) The crest level of CHR can be raised by 1.78m. The FSL and bed level of canal behind the CHR can also be raised by 1.78m and 1.67m respectively at least upto the location of sediment ejector. Due to non-availability of sufficient head the concept of a silt trap behind the CHR has been evolved at present. This also need sediment disposal system from the silt trap through dredgers. Though this method looks attractive but will be costly for operation and maintenance agaist a hydraulically operated sediment ejector.

8.3 SUGGESTIONS FOR FUTURE STUDY

The methodology for the preventive approach as evolved in the present study needs field checking through curative measures. This involves the use of scale models in open sand plains. Though simulation of the sediment is difficult but there is need to develop small or intermediate scale models to check the functioning of the excluders, CHR and sediment ejector considering the variable parameters as discharge, sediment flow and channel geometry.

There is also a need to study the behaviour of the river in post barrage condition i.e., when the barrage is fully operational. Present literature study clearly indicates that there is a great scope to study in this area. The aid of Computers may be sought to partially solve the problem particularly in the areas of aggradation and degradation.

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APPENDIX — 1

DESIGN PROCEDURE OF SEDIMENT EXCLUDER

To design sediment excluder, the following steps should be performed:

1) Select the pond level of the barrage as the downstream water level or a level little below it for maximum design discharge of barrage, Q. For storage type reservoir, select the level to be higher than the downstream water level for maximum design discharge of barrage.

2) Select the waterway of barrage, $B \approx 4.83\sqrt{Q}$, where B is the lacey's wetted perimeter. If barrage location is on the boulder reach of the river it would be economical to reduce the waterway of the barrage to about 0.6 to 0.8 times Lacey's wetted perimeter. In plains where silt factor is in the neighbourhood of unity it should be 1.0 to 1.2 times the Lacey's wetted perimeter. Find the total width of barrage, B_R after adding the pier widths with Lacey's wetted perimeter.

3) Select the upstream floor level of weir bay as average deeper channel level.

4) Select the upstream floor level and crest level of undersluice as 1m below the average deeper channel level.

5) Select the crest level of head regulator as 1 to 4m higher than the average deeper channel level for submerged flow condition in the barrage. For free fall condition it should be 1m

higher than the average deeper channel level.

6) Select the canal discharge, Q_c from availability and requirement of the command area of canal and considering the sediment ejector discharge, Q_{EJ}.

7) Find the width of the head regulator, W_h by using equation

$$Q_c = 2/3\ C_1 l \sqrt{2g}\ \{(h+h_a)^{3/2} - h_a^{3/2}\} + C_2 l d \sqrt{2g(h+h_a)}$$

where C_1 = 0.577, C_2 = 0.80, h = pond level - water level of main canal at downstream of head regulator (should be higher than F.S.L for plain land), h_a = velocity head at approach, d= water level of the canal at downstream of head regulator - crest level of head regulator, and l is the clear width of head regulator. After allowing the piers with l, W_h may be obtained.

8) Find the length of divide wall, L = 2/3 W_h

9) Find the width of pocket, W by using Figure 2.7

10)Find the crest level of weir bay as 1 to 1.5m above the average deeper channel level by using weir formula to pass the design discharge, Q of the barrage.

11) Find the average shear stress, $\tau_o = \gamma_f DS$, where D is the depth of flow for undersluice and S is the slope of the energy line.

12) Find the coarsest material which can move at this stage by $\tau_{oc}/[(S_s-1)\gamma_f d] = 0.06$, where d is the coarsest material and it should be greater than coarsest material existing in the river bed.

13) Consider minimum depth of tunnel, t=1.677m (one man height)

14) Assume excluder discharge, $Q_{EX} = 0.30*Q_C$

15) Find the critical velocity, U_C for sediment size obtained in step (12) by using Equation 5.2.

16) Find the limit deposit velocity, U_L for this size by using equation 5.1.

17) Chose excluder velocity, U_{EX} as $U_C < U_{EX} < U_L$ and it should be 2 to 2.5 m/s for alluvial reach, 2.5 to 3 m/s for sandy reach and 3 to 4 m/s for boulder reach.

18) Find the clear width of excluder, $B_{EX} = Q_{EX}/(t*U_{EX})$ and it should be equal to the width of one or two undersluice bay.

19) Find the hydraulic radious of the excluder, $R_{EX}=B_{EX}*t/[2(B_{EX}+t)]$

20) Find the tunnel blockage = t − D_{FEX}, where D_{FEX} is the depth of the tunnel for the free flow aresa available and can be obtained by using Equations 5.3, 5.4 and 5.5.

21) Find the excluder velocity for free flow area available, U_{FEX} by using Equation 5.4.

22) Find the hydraulic radius of the excluder for the free flow area available, R_{FEX} by using equation 5.5.

23) Repeat steps (14) to (22) for different Q_{EX}, U_{EX} and B_{EX} and find the best design in such a way that U_{EX} follows step (17), U_L in the neighbourhood of U_{FEX} and tunnel blockage as minimum as possible.

24) Divide clear waterway of excluder, B_{EX} into few numbers of tunnels by taking suitable width of each tunnel at exit to

cover two or three bays of undersluice.

25) Select the length of each tunnel by CWPC type staggering.

26) Select the width of each tunnel so that clear water head loss in each tunnel is equal and determine average clear water head loss, h_o and average length of tunnel, L.

27) Select maximum water discharge, Q_R as dominant discharge where free fall condition occurs in the barrage. For submerged flow condition select the water discharge for which minimum operating head, $h \simeq 1.1m > h_o$ is available.

28) Find the width of the river contributing discharge to the undersluice pocket, B_{RE} by using Equation 5.14.

29) Find the bedload discharge in the tunnel, Q_{BT} by using Equation 5.21.

30) Find the suspended load discharge in the pocket, Q_{SP} by using Equation 5.25.

31) Find the suspended load discharge entering into the excluder tunnel, Q_{ST} by using equation 5.33

32) Find the actual concentration developed in the excluder tunnel, C_{EX} by using ewquation 5.36

33) Find the concentration carrying capacity of the excluder tunnel, C_T by using equations 5.34 and 5.35

If $C_T > C_{EX}$, design is OK.

APPENDIX-2

DESIGN OF SEDIMENT EXCLUDER FOR TEESTA HEADWORKS

Data:-

1) Pond level =+53.6m

   (Downstream water level for design discharge of barrage)

2) Water discharge at the upstream of barrage, Q_R=2850m^3/s

   (Submerged flow-discharge for which minimum operating head is available

   Free fall -dominant discharge)

3) Excluder discharge,Q_{EX} = 68m^3/s (assumed)

   (30% of canal discharge)

4) Upstream floor level of weir bay = + 49m

   (Average deeper channel level at barrage site)

5) Upstream floor level and crest level of undersluice = +48m

   (Average deeper channel level - 1m)

6) Crest level of weir bay = + 50.1m

   (Average deeper channel level + 1.1m)

7) Bed width, B_R = 615.24m

   (Width of barrage as existing)

8) Depth of water for undersluice, D = 5.6m

   (Pond level - upstream floor level of undersluice)

9) Slope of the river, S= 1/2000

10) Available head, h = 1.05m

(Pond level - Downstream water level for Q_R considering total discharge downstream) > clear water head loss of tunnel

11) Average velocity at the upstream of barrage, $\bar{U}$ = 1.061mps [Q_R/(Average depth * Average width)]

12) Tunnel depth, t = 1.677m (One man height)

13) d_{50} of bed material = 0.26mm

Average shear stress, $\tau_o = \gamma_f DS$=1000*5.6/2000=2.8kg/m^2

For coarsest material to move, $\tau_{oc}/[(S_s - 1)\gamma_f d] = 0.06$

∴ Coarsest material that can move, d=2.8/[(2.65-1)*1000*0.06]

= 0.0283m = 28.3mm

But coarsest material present in Teesta River is 10mm, which is less than 28.3mm. Hence O.K.

Let the hydraulic radius of excluder, R_{EX} = 0.75(assumed)

Critical velocity for the coarsest material that can move,

$U_c = 1.6*(R_{EX}/d)^{1/8} * \sqrt{(\Delta\gamma_s d/\rho_f)}$

$= 1.6 *(0.75/0.283)^{1/8} * \sqrt{[(2650-1000)*0.0283*9.81/1000]}$

=1.63m/s

Limit deposit velocity for the corasest material that can move, $U_L = F_L \sqrt{[8gR_{EX}(S_s - 1)]}$

$= 1* \sqrt{[8*9.81*0.75*(2.65-1)]}$

= 9.85m/s

Chose excluder velocity, U_{EX} = 2.75 m/s ($U_c < U_{EX} < U_L$, and for sandy river 2.5 $< U_{EX} <$ 3m/s)

Width of clear waterway of excluder for the chosen velocity,

$B_{EX} = Q_{EX}/(t*U_{EX})$

$= 68/(1.677*2.75)$

$= 14.75m$

Developed hydraulic radius,

$R_{EX} = B_{EX}*t/[2(B_{EX}+t)]$

$= 14.75*1.677/[2(14.75+1.677)]$

$= 0.753m$

Revised limit deposit velocity for the developed hydraulic radius, $U_L = 1*\sqrt{(8*9.81*0.753*1.65)}$

$= 9.87$ m/s

Tunnel blockage = $t - D_{FEX}$

$Q_{EX}/(B_{EX}*D_{FEX})/\sqrt{[(B_{EX}*D_{FEX})/\{2(B_{EX}+D_{FEX})\}]} = U_L/\sqrt{R_{EX}}$

or, $68/(14.75*D_{FEX})/\sqrt{[(14.75*D_{FEX})/\{2(14.75+D_{FEX})\}]}$

$= 9.87/\sqrt{0.753}$

By trial and error, $D_{FEX} = 0.7m$

Tunnel blockage = 1.677-0.7=0.977m i.e., 58%.

Developed excluder velocity for the free flow area available, $U_{FEX} = Q_{EX}/(B_{EX}*D_{FEX})$

$= 68/(14.75*0.7)$

$= 6.586$ m/s

and this should be in the neighbourhood of revised U_L.

Developed hydraulic radius of excluder for the free flow area available, $R_{FEX} = B_{EX}*D_{FEX}/[2(B_{EX}+D_{FEX})]$

$= 14.75*0.7/[2(14.75+0.7)]$

$= 0.334m$

Computations for different Q_{EX}, t and B_{EX} are tabulated below:

No. of trial	Q_{EX} m³/s	t m	B_{EX} m	U_{EX} m/s	R_{EX} m	U_L m/s	D_{FEX} m	blockage %	R_{FEX} m	U_{FEX} m/s
a			10	4.05	0.718	9.64	0.921	45	0.422	7.38
1 b	68	1.677	12	3.38	0.736	9.87	0.801	52	0.375	7.07
c			16	2.53	0.759	9.91	0.663	60	0.318	6.42
a			10	3.58	0.718	9.64	0.845	50	0.390	7.10
2 b*	60	1.677	12	2.98	0.736	9.87	0.737	56	0.347	6.78
c			16	2.24	0.759	9.91	0.609	64	0.293	6.16
a			10	3.40	0.718	9.64	0.816	51	0.377	6.99
3 b	57	1.677	12	2.83	0.736	9.87	0.712	58	0.336	6.67
c			16	2.12	0.759	9.91	0.588	65	0.284	6.06
a			10	6.80	0.455	7.67	0.921	8	0.422	7.38
4 b	68	1.00	12	5.67	0.462	7.73	0.809	19	0.379	7.00
c			16	4.25	0.471	7.81	0.659	34	0.316	6.45
a			10	6.00	0.455	7.67	0.845	16	0.390	7.10
5 b	60	1.00	12	5.00	0.462	7.73	0.743	26	0.350	6.73
c			16	3.75	0.471	7.81	0.662	34	0.318	5.66

* Best design

From the above comparison it may be concluded that smaller depth and smaller clear waterway produce smaller blockage of the tunnel. In design 4 and 5 nearly all the tunnel blockage are within the permissible limit and the developed excluder velocity for the free flow area available are in the neighbourhood of limit deposit velocity, but where the excluder velocity is very high and the tunnel depth of 1m may not be sufficient for maintenance work. For maintenance work tunnel depth of one man height of 1.677m may be adequate. Clear waterway for the excluder may be considered as one undersluice bay and is 12.195m for Teesta Barrage. Thus clear waterway of 12m may be taken as adequate. Hence comparison may be done between the designs 1(b), 2(b) and 3(b), for selecting best design. In the design 1(b), excluder velocity is high but in designs 2(b) and 3(b)it is within the limit. The blockage of tunnel in design 2(b) is lower than the design 3(b), thus design 2(b) may be considered as the best design though the blockage is high, where Q_{EX} = 60m^3/s, t = 1.677m and B_{EX} = 12m.

Clear waterway of excluder is then divided into 6 tunnels which is shown in Figures 7.12 and 7.13. The length of the larger tunnel is selected to cover the width of head regulator, whereas lengths of other tunnels and their varying widths (Table 7.7) are selected for CWPC type staggering and to have nearly equal head loss (Table 7.8) in each tunnel. Average clear water head loss in the tunnels, h_o = 0.9117m (Table 7.8) and average length of tunnel, L=155.01m (Table 7.7).

Width of river contributing discharge to the excluder,

$B_{RE} = (Q_P/Q_R)*B_R = [(Q_{EX} + Q_C)/Q_R]*B_R$

$= [(60+226.7)/2850]*615.24$

$= 61.89m$

Bedload discharge to the pocket,

$Q_{BP} = Q_{BPI} = B_{RE} * Q_B/B_R$

$= B_{RE}*1.41312*(Q_R)^{1.49325}*1000/(24*60*60*2650)/B_R$

$=61.89*1.41312*(2850)^{1.49325}*1000/(24*60*60*2650)/615.24$

$= 8.95*10^{-2}\ m^3/s$

As available head of 1.05m is greater than clear water head loss in the tunnel, bedload discharge in the pocket will be equal to the bedload discharge in the tunnel i.e., $Q_{BP} = Q_{BT}$

$Q_{BT} = 8.95 * 10^{-2}\ m^3/s$

Suspended load discharge in the pocket,

$Q_{SP} = Q_{SPI} = B_{RE} * Q_s/B_R$

$= B_{RE}*5.213784*10^{-6}\ (Q_R)^{1.65683}/B_R$

$=61.89* 5.213784*10^{-6}*(2850)^{1.65683}/615.24$

$= 2.78*10^{-1}m^3/s$

Entrainment of suspended load into the tunnel,

$Q_{ST} = Q_{SP}*f[K\bar{U}/U_*, z, t/D]$ $\qquad K=0.4, U_* = \sqrt{(\tau_o/\rho_f)}=0.166m/s$

$= Q_{SP}*f[2.56, 0.512, 0.30]$ $\qquad w_o=3.4*10^{-2}m/s(\text{for } d=0.26mm)$

$= 2.78*10^{-1}*0.425$ [Figure 5.6] $\qquad K\bar{U}/U_*=0.4*1.061/0.166=2.56$

$= 1.182*10^{-1}\ m^3/s$ $\qquad z=w_o/(KU_*)=0.512$

$t/D=1.677/5.6=0.3$

Actual concentration in the tunnel,

$$C_{EX} = (Q_{BT}+Q_{ST})/Q_{EX}$$

$$= (8.95*10^{-2} + 1.182*10^{-1})/60$$

$$= 3.46*10^{-3}\ m^3/m^3$$

Concentration carrying capacity of tunnel:-

Lazarus and Neilson (1978), Equation 5.34.

$$f_m=2*9.81*0.736*1.05/(155.01*2.98^2)$$

$$f_b=10^{\{-\exp[0.835-\{6.3-Log(4*2.98*0.736/1*10^{-6})\}^2/24]}$$
$$+\exp[\{7-Log(4*2.98*0.736/1*10^{-6})\}^2/28-1.6]*2.65*C_T$$
$$-[\{Log(4*2.98*0.736/1*10^{-6})-5.2\}^2/24.5+0.2]*\exp(-2*2.65*C_T)\}$$

$$\lambda=[2.98^2/(4*9.81*0.736)]*\sqrt{[1*10^{-6}/(4*2.98*0.736*2.65^3*C_T)]}$$
$$*[1000\{.26/1000/(4*0.736)\}^{(0.444*Log(.26/1000/(4*0.736))+1.31)}]$$
$$\tanh(1+2.65*C_T)$$

$$\text{and } f_m/f_b = 1+2.1[\exp\{-2(\lambda-0.32)\}]+5.85[\exp\{-12(\lambda-0.32)\}]$$

By trial and error, $C_T = 5.848*10^{-1} m^3/m^3 > C_{EX}$ O.K.

Kazanskij (1978),Equation 5.35.

$$K_s=(24n)^6 = (24*0.013)^6 = 9.224*10^{-4}\ m$$

$$C_T=(1.05-0.9117)/0.9117/[1.58*10^4*\{9.224*10^{-4}/(4*0.347)^{2/3}$$
$$*(4*9.81*0.347/6.78^2)*(2.65-1)^{-1}*\{3.4*10^{-2}/\sqrt{(9.81*.26/1000)}\}]$$
$$= 1.043*10^{-2}\ m^3/m^3 > C_{EX}\ \text{O.K.}$$

APPENDIX-3

ANALYSIS OF ENTRAINMENT OF SEDIMENT DISCHARGE INTO THE EXCLUDER TUNNEL AND INTO THE TEESTA MAIN CANAL UNDER DESIGN CONDITION

Data (Tables 7.7, 7.8 and 7.9):-

Q_{Ro}=1566.8m^3/s, Q_R=1000m^3/s, Q_c=226.7m^3/s, Q_{Ex}=60m^3/s, Q_P=286.7m^3/s D=5.6m, $\bar{U}$=0.372m/s, h(required flow through the main canal) =1.8432m, h_o=0.9117m, B_{Ex}=12m, B_R=615.24m, S=1/2000, t=1.677m, U_{Ex}=2.98m/s, R_{Ex}=0.736m, U_{FEx}=6.78m/s, R_{FEx}=0.347m, L=155.01m, d_{50}=0.26mm.

Width of river contributing dsicharge to the excluder,

$B_{RE} = B_R*Q_P/Q_R$=615.24*286.7/1000=176.39m

Initial entrainment of bedload discharge into the pocket,

$Q_{BPI}=B_{RE}*Q_B/B_R$=176.39*1.41312(1000)$^{1.49325}$*1000/(24*60*60*2650)

/615.24 = 5.34*10^{-2}m^3/s

Initial entrainment of suspended load discharge into the pocket, $Q_{SPI}=B_{RE}*Q_S/B_R$=176.39*5.213784*10^{-6}(1000)$^{1.65683}$/615.24

= 1.397*10^{-1}m^3/s

Diameter of particle in motion,

$U_{cr}/\sqrt{[(S_s-1)gd]} = 0.5 \text{ Log } (D/d) + 1.63$

or $0.372/\sqrt{[(2.65-1)*9.81*d]}=0.5 \text{ Log } (5.6/d) + 1.63$

By trial and error, d = 0.66mm

From grain size distribution curve of bed material (Figure 7.5) and suspended material (Figure 7.4), 2.5% of bedload and 1% of suspended load will deposit at the upstream of sediment excluder.

Deposited bedload discharge in the pocket,

$Q_{BPD}=Q_{BPI}$*% deposited = $5.34*10^{-2}*0.025=1.335*10^{-3}m^3/s$

Deposited suspended load discharge in the pocket,

$Q_{SPI}=Q_{SPI}$*% deposited = $1.397*10^{-1}*0.01=1.397*10^{-3}m^3/s$

Entrainment of bedload discharge into the pocket,

Q_{BP} = Q_{BT} (as available head is greater than clear water head loss of tunnel) = $(Q_{BPI}-Q_{BPD})$

$= (5.34*10^{-3}-1.335*10^{-3}) = 5.207*10^{-2}m^3/s$

Entrainment of suspended load discharge into the pocket,

$Q_{SP} = (Q_{SPI}-Q_{SPD})=(1.397*10^{-1}-1.397*10^{-3})=1.383*10^{-1}m^3/s$

Entrainment of suspended load discharge into the tunnel,

$Q_{ST}=Q_{SP}*f[K\bar{U}/U*,z,t/D]=1.383*10^{-1}*0.126=1.743*10^{-2}m^3/s$

Actual concentration in the excluder tunnel,

$C_{EX}=(Q_{BT}+Q_{ST})/Q_{EX}=(5.207*10^{-2}+1.743*10^{-2})/60=1.158*10^{-3}m^3/m^3$

Concentration carrying capacity:-

Lazarus and Neilson (1978),

$C_T=1.0m^3/m^3>C_{EX}$ Ok.

Kazanskij (1978),

$C_T=7.024*10^{-2}m^3/m^3>C_{EX}$ OK.

Sediment load discharge excluded downstream through the tunnel, $Q_T = Q_{EX} * C_{EX} = 60*1.158*10^{-3} = 6.948*10^{-2} m^3/s$

Bedload discharge remains as bedload in the pocket, $Q_{BPB} = 0$ (Since sediment concentration carrying capacity of the tunnel is higher than the actual concentration in the tunnel)

Suspended load discharge remains in suspension in the pocket, $Q_{SPS} = (Q_{SP} - Q_{ST}) = (1.383*10^{-1} - 1.743*10^{-2}) = 1.209*10^{-1} m^3/s$

Entrainment of bedload discharge into the main canal, $Q_{BC} = Q_{BPB} * Q_C / (Q_P - Q_{EX}) = 0.0*226.7/(286.7-60) = 0.0 m^3/s$

Entrainment of suspended load discharge into the main canal, $Q_{SC} = Q_{SPS} * Q_C / (Q_P - Q_{EX}) = 1.209*10^{-1} * 226.7/(286.7-60) = 1.209*10^{-1}\ m^3/s$

APPENDIX—4

ANALYSIS OF ENTRAINMENT OF GRAIN-SIZE RANGE GROUP INTO THE TEESTA MAIN CANAL UNDER DESIGN CONDITION

Data (Tables 7.7, 7.8 and 7.9):-

$Q_{Ro}=1556.8m^3/s$, $Q_R=1000m^3/s$, $Q_C=226.7m^3/s$, $Q_{Ex}=60m^3/s$, $Q_P=286.7m^3/s$, $D=5.6m$, $\bar{U}=0.372m/s$, h(required flow through the main canal) $=1.8432m$, $h_o=0.9117m$, $B_{Ex}=12m$, $B_R=615.24m$, $S=1/2000$, $t=1.677m$, $U_{Ex}=2.98m/s$, $R_{Ex}=0.736m$, $U_{FEx}=6.78m/s$, $R_{FEx}=0.347m$, $L=155.01m$, $d_{50}=0.26mm$.

a) Entrainment of Different Grain-Size Range of Suspended Load Material

From the average grain size distribution curve of suspended material (Figure 7.4), the total grain size-range is divided into four groups of 0.074 to 0.15mm, 0.15 to 0.30mm, 0.30 to 0.60mm and > 0.6mm, where the percentage of different groups are 42,42,14 and 2 respectively.

From Appendix-3, it may be observed that for U=0.372m/s, maximum diameter of sediment particle in motion is 0.66mm. Then by using grain size distribution curve of suspended material (Figure 7.4), it may be concluded that 99% of the suspended material will be in motion with maximum diameter of 0.66mm.

Percentage of material excluded by the excluder tunnel,

$Q_{ST}/Q_{SP}=f(K\bar{U}/U*,z,t/D)=f(0.90,0.51,0.30)=12.6\%$ (Figure 5.6)

Entrainment percentage of suspended material into the main canal = 99-99*0.126 = 86.5%.

Entrainment of maximum grain size of suspended load into the main canal is 0.36mm (From grain size distribution curve of suspended material (Figure 7.4) for 86.5%).

Now the entrainment percentages of different grain-size range groups are as follows,

Grain-Size Range	Entrainment %	
0.074 to 0.15mm	= 42/86.5	= 48.5%
0.15 to 0.30mm	= 42/86.5	= 48.5%
0.30 to 0.60mm	= (86.5-42-42)/86.5=3%	
and >0.60mm		= 0%.

Alternate Method Given by Rozovskii (1957).

Considering two-dimensional turbulent flow (Rozovskii,1957) the material of diameter, d lifted over the tunnel depth t (Vertical opening + thicness of the top slab) can be obtained by using Equation 6.12. Where R is the radius of curvature of flow for the offtaking canal and is 288m for Teesta Main Canal.

$$d=2.969*10^{-1}[0.372(5.6/288)^2\{2.287/5.6-(2.287/5.6)^2\}]^2$$

$$=3.428*10^{-10}\ \mathrm{m}$$

$$=3.428*10^{-7}\ \mathrm{mm}$$

Thus only wash load can enter into the main canal.

But for diversion headworks system, entrainment of sediment particle into the main canal is dependent upon the workability of sediment excluder, turbulence created by flow, turbulence created by different parts of the barrage and radius of curvature of flow for the offtaking canal. The procedure contains only the turbulence created by flow and radius of curvature of flow for the offtaking canal and may not be used as good predictor for the entrainment of sediment particle into the offtaking canal.

b) Entrainment of Different Grain-Size Range of Bedload Material

For the river discharge, Q_{RO}=1566.8m^3/s the sediment excluder can exclude downstream (Appendix-3) the total sediment load entered into the tunnel i.e., no bedload will remain as bedload in the pocket. In such a position no sediment of bedload material will enter into the main canal.

APPENDIX — 5

DESIGN OF SEDIMENT EJECTOR FOR TEESTA MAIN CANAL

Data:-

Canal discharge, $Q_C = 226.7 m^3/s$

Ejector discharge, Q_{EJ} = 20-25% of Q_C

Water level at downstream of head regulator = +53m

River bed at one mile downstream of barrage = +48.20m

Entrainment of suspended and bedload discharge into the main canal are available in Figures 7.16 and 7.17

Stage-Discharge relation of the river,

$$Q_{RO}=185.06(Z_W-50.19)^{3.18468615}$$

Main canal slope 1:12,000

River slope 1:2000

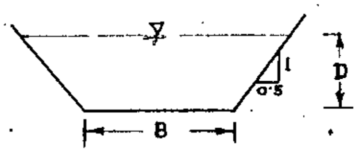
Design:-

Canal discharge upto the ejector

Ejector discharge, $Q_{EJ} = 226.7*0.2 = 45.34 m^3/s$

$\therefore$ Canal discharge upto ejector = $Q_C+Q_{EJ}=226.7+45.34=272 m^3/s$

Main canal design upto the ejector by Lacey's regime theory



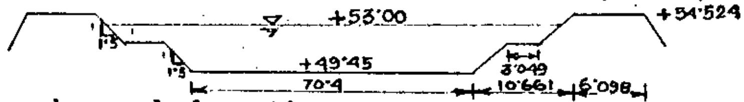
$$\bar{U} = (Q_C*f^2/140)^{1/6} = (272*0.8^2/140)^{1/6} = 1.037 m/s$$

$$P = 4.75\sqrt{Q_C} = 4.75\sqrt{272} = 78.34 m = B + 2.236D$$

$A = Q_c/\bar{U} = 272/1.037 = 262.30 m^2 = D(B+D)$

$\therefore B = 70.40m$, and $D = 3.55m$

$S = f^{5/3}/(3340*Q_c^{1/6}) = 0.8^{5/3}/(3340*272^{1/6}) = 1/12,332 \simeq 1/12000$



Approach channel length

Rozovskii (1957) has suggested the legnth on which circulation deminishes as:

$$L = 2.3\ CD/\sqrt{g}$$

where L is the legnth of approach channel, C is the cezy's coefficient = $R^{1/6}/n$, R is the hydraulic radius, n is the Manning's roughness coefficient, D is the depth of flow and g is the acceleration due to gravity.

$$L = 2.3*48.92*3.55/\sqrt{9.81} \quad \text{as, } C = (262.3/78.34)^{1/6}/0.025$$

$$= 127.5m \qquad\qquad = 48.92$$

Champ's (Rouse Hunter,1950) chart (Figure 1) can also be used to determine the length of approach channel. If we consider 0.1mm particle to settle,

$$w_o D^{1/6}/n\bar{U}\sqrt{g} = 7*10^{-3}*3.55^{1/6}/(0.025*1.037*\sqrt{9.81}) = 0.106$$

Now to have 90% settlement $[1-(qs)e/(qs)i]$ of 0.1mm particle (Figure 1)

$$w_o L/\bar{U}D = 1.75$$

or, $L = 1.75\bar{U}D/w_o = 1.75*1.037*3.55/7*10^{-3} = 920m$

Summer (1977) has introduced a system by which length of approach channel can be determined. The settling velocity parameter,

$$\beta = w_o/KU_* = 7*10^{-3}/(0.4*\sqrt{9.81*3.55/12000}) = 0.325$$

From Figure 2 for $\beta = 0.325$, $\lambda = 3.8$

$$L = -\,[6*(\bar{U}/U_*)D/K\lambda]*Ln(1-r),$$ where r is the settlement percentage

$$= -\,[6*\{1.037/\sqrt{(9.81*3.55/12000)}\}*3.55/(0.4*3.8)]*Ln(1-0.9)$$

$$= 621m.$$

UPIRI (1975) suggested the approach channel length as 150 to 300m for boulder stage river and for alluvial stage river it should be increased to about 600m or more.

By using shield's tractive force criteria maximum diameter of sediment particle can reach upto the sediment ejector can be obtained by

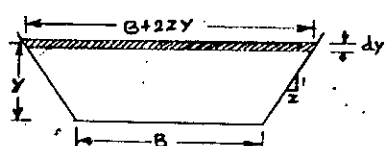
$$d = \tau_{oc}/[0.06(\gamma_s-\gamma_f)] = \gamma_f DS/[0.06(\gamma_s-\gamma_f)]$$

$$= (1000*3.55/12000)/[0.06(2650-1000)] = 3mm > 0.84mm \text{ O.K.}$$

Teesta is a sandy river and the entrainment of sediment particles for ranges 0.074 to 0.30mm can occur into the main canal. Thus an approach channel length of 600m may be adequate for sediment ejector in Teesta Main Canal.

Subtunnel depth, t at entry

To avoid extra turbulence, depth of tunnel should be so chosen that the discharge in the bottom layers below the diaphragm equals the discharge through the ejector. This can be obtained by using integrated discharge equation with Vanoni's (1941) logarithmic velocity distribution.



$$Q_{EJ} = \int dQ_{EJ}$$

$$= \int_0^t (B + 2Zy)[\bar{U} + \sqrt{gDS}/K * \{1+Ln(y/D)\}]dy$$

$$= \int_0^t \{B\bar{U} + B\sqrt{gDS}/K + B\sqrt{gDS}/K * Ln(y/D) + 2Zy\bar{U}$$

$$+ 2Zy\sqrt{gDS}/K + 2Zy\sqrt{gDS}/K * Ln(y/D)\}dy$$

$$= [B\bar{U}y + By\sqrt{gDS}/K + B\sqrt{gDS}/K * \{yLn(y/D)-y\} + Z\bar{U}y^2 + Zy^2\sqrt{gDS}/K + 2Z\sqrt{gDS}/K * \{y^2/2 * Ln(y/D) - y^2/4\}]$$

$$Q_{EJ} = [B\bar{U}t + Bt\sqrt{gDS}/K + B\sqrt{gDS}/K * \{tLn(t/D)-t\} + Z\bar{U}t^2 + Zt^2\sqrt{gDS}/K + 2Z\sqrt{gDS}/K * \{t^2/2 * Ln(t/D) - t^2/4\}] \quad (1)$$

In our case, Q_{EJ} = 45.34m^3/s, B = 70.4m, Z = 1.5, S = 1/12000, D = 3.55m, $\bar{U}$ = 1.037m/s

By trial and error (Equation 1) subtunnel depth at entry, t = 0.77m. But from prototype observation it has found that the concentration of coarser sediment usually occurs in 1/3rd to 1/4th of the depth of flow from the bottom i.e., t should be equal to 0.89m to 1.18m

Here depth of subtunnel at entry, t = 1.0m is chosen.

In order to ensure satisfactory performance of the ejector for lower discharges also, the bed of main canal at upstream of ejector should be depressed by 10-15 percent (UPIRI,1975) of the normal depth of canal. The top slab of the ejector should also be depressed by the same amount to have the subtunnel depth at entry equals to 1m. The depression of the bed is generally connected with the upstream floor by a slope of 1:100.

Upstream depressed floor length = 0.5*100 = 50m

Upstream floor level of ejector =(49.45-600/12000)-0.5=48.9m

Top level of ejector tunnel = (48.9+1+0.381) = 50.281m

(slab thicness=0.381m)

Subtunnel width at entry

At entry the ejector spans the entire 70.4m width of main canal.

It has five main tunnels of each 14.08m wide (centre to centre). Each main tunnel is again subdivided into four subtunnels, each 3.52m wide (centre to centre).

Clear width of each subtunnel should be so fixed that the average velocity at the entry equal to the average velocity of flow in the main canal over the depth of the tunnel.

Average velocity occurs at 0.37D using Vanoni's (1941) logarithmic velocity distribution equation. Thus over the depth of tunnel at entry, t average velocity will occur at depth 0.37t $= 0.37*1 = 0.37m$, and is

$= 1.037 + \sqrt{(9.81 * 3.55/12000)}/0.4 * \{1+Ln(0.37/3.55)\} = 0.867m/s.$

Thus total waterway at entry$=Q_{EJ}/(\bar{U}*t)=45.34/(0.867*1)=52.3m$

Each subtunnel clear width at entry $= 52.3/20 = 2.615m$

Each partition wall thickness at entry $=(70.4-52.3)/20=0.905m$

Critical velocity to move the largest material of 0.84mm which can enter into the ejector tunnel is

$$U_c = 1.6*(R_{EJ}/d)^{1/8} * \sqrt{(\Delta \gamma_s d/\rho_f)}$$

$$= 1.6*(0.4906/0.84*10^{-3})^{1/8} * \sqrt{(1650*0.84*10^{-3}*9.81/1000)}$$

$$= 0.414m/s < 0.867m/s \text{ O.K.}$$

Subtunnel clear width at various sections

The radius of vanes for subtunnels should be 3-4 times the width of subtunnel (Varshney and Gupta,1982).

$$R = 3.52 * 3 = 10.56 \simeq 11m.$$

The radius of inner and outer face and clear width at various sections are determined and are shown in Figure 7.24.

Main tunnel depth and width

Main tunnel section at exit should be same, so that equal discharge could pass through each main tunnel. Depth of one man height of 1.677m is necessary for maintenance work. For sandy river the exit velocity should be 3m/s.

Total clear width at eixt = $Q_{EJ}/(U_{EJ}*t)$

$= 45.34/(3*1.677) = 9.0m$

Clear width of each main tunnel at exit = 9/5 = 1.8m

and total width of main tunnels at exit = (1.8*5+0.4*4) = 10.6m

(partition wall thickness = 0.4m)

The lengths and widths of main tunnels at entry are determined so that the velocity in all the tunnels are in the neighbourhood of 3m/s, and are shown in Figure 7.23.

Subtunnel depth at exit

Considering the exit velocity of subtunnel to be same as the entrance velocity of main tunnel, the depths subtunnels at exit (Figure 7.24) are determined

Depth at exit of subtunnels (1- 4) = 0.452m

(5- 8) = 0.453m

(9-12) = 0.456m

(13-16) = 0.461m

(17-20) = 0.475m

Headloss in ejector tunnel

	h_c	h_f	h_b	h_{en}	h_{ex}	total	average	total
Sub-tunnel(1-20)	0.0437	0.0628	0.0769	0.0191	-	0.2025	0.2025	
Main-tunnel(1)	-	0.3470	0.0134	-	0.1906	0.5510		
Main-tunnel(2)	0.0003	0.2801	0.0134	-	0.1906	0.4844		0.63
Main-tunnel(3)	0.0007	0.2127	0.0134	-	0.1906	0.4174	0.4180	
Main-tunnel(4)	0.0018	0.1452	0.0134	-	0.1906	0.3510		
Main-tunnel(5)	0.0044	0.0778	0.0134	-	0.1906	0.2862		

Escape channel design

Slope requirement for the movement of 0.84mm

Shield (1936)

$$\tau_{oc} = 0.06*(S_s-1)\gamma_f d \text{ and } \tau_o = \gamma_f DS$$

$$\text{or, } \gamma_f DS = 0.06(S_s-1)\gamma_f d$$

$$\text{or, } S = 0.06(S_s-1)d/D$$

$$= 0.06(2.65-1)*0.84*10^{-3}/3.22$$

$$= 1/38,721.$$

Lacey's regime theory

$$S = [f^{5/3}/(3340*Q_{EJ}^{1/6})]$$

$= [0.8^{5/3}/(3340*45.34^{1/6})]$

$= 1/9,148.$

Sharma and Asthana (1975) have suggested several escape channel slopes viz. 1/300, 1/1200 and 1/1800 to be adequate for transporting boulder, shingle and sand respectively. Thus escape channel slope of 1/1800 may be chosen for Teesta river.

Section of escape channel

Lacey equation, $\bar{U} = 10.8\ R^{2/3}S^{1/3}$

or, $45.34/[3.22(B+3.22)] = 10.8*[3.22(B+3.22)/(B+3.22*2.236)]^{2/3} * (1/1800)^{1/3}$

or, $B = 5.8m$, $D = 3.22m$ and $\bar{U} = 1.55$ m/s

Section of escape channel is shown in Figure 7.23.

Actual concentration developed in the ejector tunnel

Sediment ejector will function effectively as long as the escape channel slope goes higher than 1/1800. The original river discharge at which the escape channel slope comes to 1/1800 is $2587.5 m^3/s$

At $Q_{RO} = 2587.5 m^3/s$, $Q_{BC} = 0$ and $Q_{SC} = 1.95*10^{-1} m^3/s$

$C_{EJ} = (Q_{BC} + Q_{SC})/Q_{EJ}$

$= (0 + 1.95 \times 10^{-1})/45.34$

$= 4.3 * 10^{-3}\ m^3/m^3$

Sediment carrying capacity of ejector tunnel

Hydraulic radius at exit section, $R_{EJ}=(B_{EJ}*t)/[2(B_{EJ}+t)]$

$=(9*1.677)/[2(9+1.677)]=0.707m$

Limit deposit velocity, $U_L = 1 * \sqrt{(8*9.81*0.707*1.65)} = 9.57m/s$

Tunnel blockage = t - DFEX

$Q_{EJ}/(B_{EJ}*D_{FEJ})/\sqrt{[(B_{EJ}*D_{FEJ})/\{2(B_{EJ}+D_{FEJ})\}]} = U_L/\sqrt{R_{EJ}}$

or, $45.34/(9*D_{FEJ})/\sqrt{[(9*D_{FEJ})/\{2(9+D_{FEJ})\}]} = 9.57/\sqrt{0.707}$

By trial and error $D_{FEJ} = 0.752m$

Tunnel blockage = 1.677-0.752 = 0.925m i.e. 55 percent.

$U_{FEJ} = Q_{EJ}/(B_{EJ}*D_{FEJ}) = 45.34/(9*0.752) = 6.7m/s$

and $R_{FEJ} = B_{EJ}*D_{FEJ}/\{2(B_{EJ}+D_{FEJ})\}$

$= 9*0.752/[2(9+0.752)]$

$= 0.347m.$

Sediment carrying capacity (Kazauskij, 1978)

$C_T = (h - h_o)/[h_o * 1.58 * 10^4 (K_s/4R_{FEJ})^{2/3} * (4gR_{FEJ}/U_{FEJ}^2)$

$* (S_s-1)^{-1} * (w_o/\sqrt{gd})]$

For $Q_{RO} = 2587.5m^3/s$, $h = (53 - 51.426) = 1.524m$

$h_o = 0.63m$, $d = 0.1mm$, $w_o = 7*10^{-3}m/s.$

$C_T = (1.524-0.63)/[0.63*1.58*10^4/4/0.347)^{2/3}$

$*(4*9.81*0.347/6.7^2)*1.65^{-1}*\{7*10^{-3}/\sqrt{(9.81*0.1/1000)}\}]$

$= 2.87*10^{-1} m^3/m^3 > C_{EJ}$ O.K.

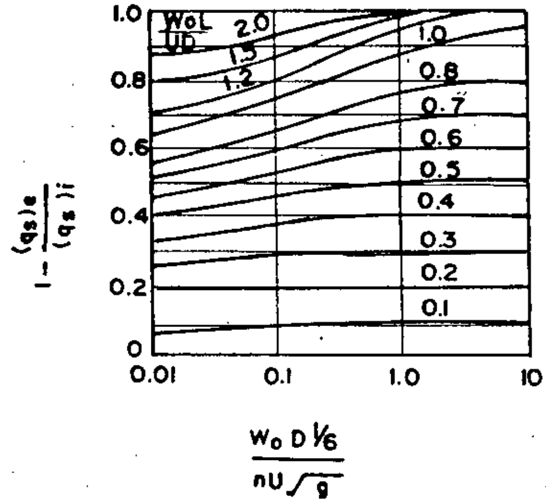


Fig. 1 SEDIMENT REMOVAL FUNCTION FOR SETTLING BASINS (AFTER ROUSE HUNTER, 1950)

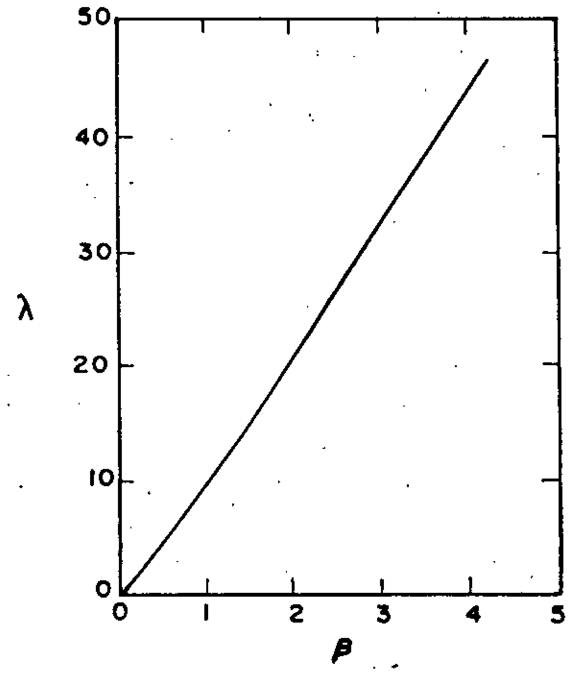


Fig. 2 RELATION BETWEEN λ AND β FOR PARTICLE SETTLING (AFTER SUMER, 1977)