COMPARATIVE ANALYSIS OF DESIGN PRACTICES FOR LINED CANALS WITH REFERENCE TO INDIRA GANDHI MAIN CANAL

A DISSERTATION

Submitted in partial fulfillment of the requirements for the award of the degree

of

MASTER OF TECHNOLOGY

in

WATER RESOURCES DEVELOPMENT

By

SAMPURNO

WATER RESOURCES DEVELOPMENT TRAINING CENTRE
INDIAN INSTITUTE OF TECHNOLOGY ROORKEE
ROORKEE -247 667 (INDIA)

December, 2002

CANDIDATE'S DECLARATION

I hereby certify that the work which is being presented in the dissertation entitled, **"COMPARATIVE ANALYSIS OF DESIGN PRACTICES FOR LINED CANALS WITH REFERENCE TO INDIRA GANDHI MAIN CANAL"** in partial fulfillment of the requirements for the award of degree of MASTER OF TECHNOLOGY in WATER RESOURCES DEVELOPMENT (CIVIL) submitted in Water Resources Development Training Centre, Indian Institute of Technology Roorkee, is an authentic record of my own work carried out during the period July, 2002 to November, 2002 under the supervision of **V.K. Bairathi**, Visiting Professor of WRDTC, Indian Institute of Technology Roorkee, India.

The matter embodied in this dissertation has not been submitted by me for the award of any other degree.

Dated : December 5, 2002

Place : Roorkee

(SAMPURNO)

This is to certify that the above statement made by the candidate is correct to the best of my knowledge.

5.12.2002

(V.K. Bairathi)
Visiting Professor, WRDTC
Indian Institute of Technology Roorkee
Roorkee, INDIA

ACKNOWLEDGEMENT

I would like to express my deep gratitude to **V.K. Bairathi**, Visiting Professor, WRDTC, Indian Institute of Technology, Roorkee, for his valuable guidance, advice and encouragement during the preparation of this dissertation.

I am very much grateful to Professor **D.D. Das**, Head of WRDTC and Government of India for extending various facilities for my study at IIT-Roorkee.

Thanks are due to other entire faculty members specifically Dr. B.N. Asthana Emeritus Fellow and staff of WRDTC for their co-operation and help extended to me.

I have no word to adequately express my most sincere and heart-felt gratitude to my parent, my parent in law, my wife *Nurul Huda Wardian*, and my children *Firzi Maulana Sani, Bhanat Aliyan Afsari, Bhanat Alin Afsari and Sonia Aliyah Tardha*, who have sacrificed by living lonely in Indonesia during my entire course of study at IIT-Roorkee. Without their encouragement and moral support, it would have been impossible for me to complete the course.

Roorkee, December 5, 2002

SAMPURNO
Trainee Officer
46th WRD Course

ABSTRACT

Canal design is perhaps the simplest and most common amongst Irrigation Engineers. With a large number of irrigation projects constructed in the world, the design of canals is well known.

Design of canal depends on discharge, topography, inner side slopes lined and unlined, operation and maintenance.

Following are important aspectsin a canal :

(i) Silting

(ii) Scouring/non scouring

(iii) Weed growth

(iv) Maximum permissible velocity

(v) Minimum and maximum bed slopes

(vi) Operation and maintenance cost

(vii) Seepage losses

(viii) Cost economics of canal section

There are a number of design practices and it is not that simple as is being practiced.

The design of lined canals particularly trapezoidal canals is largely done by trial and error by arbitrarily choosing the bed width depth ratio (B:D). This resulted many times uneconomical section. Also their operational performance have not been as was expected in designed lined canals. These canals have also shown considerable sign of damage to lining, seepage, silting, weed growth, and low discharging capacity.

Best hydraulic sections are most economical as long as the cost of earth work (excavation and embankment) is less than the cost of lining per unit length of channel and this is the most usual case. These sections, besides having least area and perimeter also have minimum top width. Therefore land width required is also minimum, in comparison to other wide sections.

Selection of the ratio of base width to water depth (B:D) so far depends largely upon individual judgment. Different design practices has resulted in values of width, depth and slope significantly different. Velocity are also appreciably at variance.

Such an elementary comparison serves to focus attention to the end results and accordingly to promote further research into the practical aspects of the subject, with a view to more economical and efficient design practices.

This study attempts a comparative analysis of some lined Major Canals in India with the parameter of B/D ratio, velocity, bed slope and discharge.

Significance of comparison :

(i) Comparison aims at understanding the quality of a subject and it's systematizing

(ii) Comparison enables us to see resemblance and difference, and to point out universality and individuality

(iii) Comparison helps validation and explains variation in existing theories and practices

(iv) Lastly comparison helps in development/modification of existing theories, practical limitation and use.

This study discusses the hydraulic comparative analysis of Indira Gandhi Main Canal with approaches of Manning's formula, Kennedy's formula, Lindley's formula. Lastly Lacey's formula and tractive force theory are also discussed, a reach from Km 384 to Km 410 lined canal.

It is found that parameter of B:D ratio, inner side slope and then bed slope are very important in influencing velocity, and economical section.

CONTENTS

Page No.

LIST OF TABLES

list continued

LIST OF FIGURES

list continued

table continued

table continued

NOTATIONS

SYMBOLS	MEANING	METRIC UNITS
A	Major axis of the sediment	-
A	Area of cross section; Projected area of a particle; dimensionless number = a/D	m^2
A_E	Dimensionless height	-
B or b	Bed width of channel	m
B_T or T	Top width of channel	m
C	Coefficient in sediment transport function ; Silt factor in the kennedy equation sediment charge or sediment transport divided by water discharge in the iglis equation, or bed load charge in hundred thousand the by weight ; a factor of flow resistance ($C = 1.49/n\ R^{1/6}$)	ppm
$\overline{C}$	Total load concentration in percent by weight sediment transport	%
C_a	Suspended sediment concentration at a distance a above the bed; coefficient	ppm
C_F	Coefficient C_F is unity for laboratory data and 1.268 for field data, $C_F = 1.268$	-
C_0	Maximum sediment concentration in fraction by volume	ppm
C_m	Sediment discharge concentration in weight per unit volume	ppm
$\overline{C}_T$	Average sediment concentration as defined at appropriate places	ppm
C_s	Sediment concentration by volume or by weight	ppm
C_{2d}	Suspended sediment concentration at a distance 2d	ppm
D or h,y	Depth of water flow	m
D	Size of sediment	mm
D_{gr}	Dimensionless particle size	-

d_{50} or d_{si}	Medium size of sediment	mm
d_x	Equivalent diameter for a non uniform granulate	mm
d_m	Effective diameter of the sediment	mm
d_{si}	Mean size of I th size friction of sediment	mm
d_s	The representative size of bed sediment and is usually taken as the mediam size, d_{50}	mm
d_*	Dimensionless diameter of grain	-
d_{65}	Size of bed sediment for which 65 % of the sediment by weight is finer	mm
d_{35}	Size of sediment for which 35 % of the sediment by weight is finer	mm
E	Specific energy	m
eB	The bed load transport efficiency	-
eS	The suspension efficiency	-
F	Force acting on a particle	Kg/m^2
F_1	Factor in Rubey equation for fall velocity of particle	mm
F_b	Bed factor	-
F_s	Side factor	-
F_r	Froud number	-
F_{gr}	Sediment mobility number	-
F'_{gr}	Value of F_{gr} at nominal initial motion	-
F	The friction factor of the bed; silt factor	-
f_b	Darcy-weisbach bed friction factor for the sand grain roughness	-
G	Specific gravity of sediment	Ton/s
G	Gravitational gravity	m/s
G_s	Solid discharge, as total load, by mass	Kg/s
G_{gr}	Dimensionless sediment transport rate	-
G_{sb}	Total bed sediment discharge of the stream in width per unit time	M^3/s
H_t	Height of water flow	m

h_e	Head velocity	m
h_f	Loss of energy	m
h_n	Normal flow depth	m
I_1,I_2	Placticity index for colesive sediment (Intergral by Einstein)	-
K	Coefficient of Karman constant, K = 0.4; A factor reducing the limiting stress, on the bank particle	-
k	Representative size (mm) of roughness of rippled bed	mm
k_s	Effective grain diameter, ($k_s = d_{35}$) equivalent sand grain roughness of the boundary	mm
L	Length of the dune	m
ΔL	Length of the prism	m
M	Kramer's uniformity coefficient in the transport relations	-
m	Critical velocity ratio	-
N	Transition exponent depending on sediment size	-
n	Manning roughness coefficient	-
n'	Manning's coefficient for plane bed	-
Na	Coefficient of rugosity depend only on grain size of the boundary materil of the channel	-
nv, z_i	Dimensionless exponent in rouse equation based on sediment size d_{si}	-
P	Exponent; percent by weight corresponding to give size ; wetted perimeter	-
Pe	Transport parameter	-
P_i	Fraction by weight of that fraction of the bed sediment with mean size	-
Q	Discharge if sides were frictionless	m^3/s
q	Water discharge, in cubic meter persecond per unit width	$m^3/s/m$
Q_s or Q_T	Total solid discharge, as total load by volume	m^3/s
Q_{sb}	Solid discharge, as bed load, by volume	m^3/s
Q_{ss}	Solid discharge, as suspended load, by volume	m^3/s

q_{ss}	Rate of suspended load transport in weight per unit width	Kg/s/m
q_{sb}	Rate of bed load transport in weight per unit width of the channel	Kg/s/m
q_{si}	Discharge of bed sediment of mean size d_{si}	$m^3/s/m$
q_{ci}	Critical value of q for initiating motion of sediment of mean size, d_{si}	$m^3/s/m$
q_{sbi}	Discharge of bed load of mean size of sediment, d_{si}	M^3/s
q_T	Rate of total load transport in weight per unit width	$m^3/s/m$
R	Hydraulic radius	m
R'	Hydraulic mean radius of the channel if the bed were unrippled; Hydraulic radius corresponding to grain resistance	m
R_*	Bed Reynolds number	-
S_0	Bed slope	m
S_f	Friction slope (energy line)	m
S_w	Water surface	m
$\overline{S}_f$	Average value of S_f	m
S^1	Portion of channel slope due to grain roughness	mm
T	Temperature of water	^0C
V	Mean velocity	m/s
V_*	Shear velocity or total bed shear velocity	m/s
V_d	Characteristic velocity	m/s
V_n	Non displacement velocity	m/s
V_p	Detachment velocity	m/s
V_0	Critical velocity	-
V_{cr}	Critical velocity ; average velocity for incipient motion condition	m/s
V_*'	Grain – roughness shear velocity	m/s
W	Weight of the water prism	kg/m^2
X	Variable length of distance, correction term for logarithmic velocity distribution	m

x	Dimensionless factor in Einstein velocity relation; exponent	-
Δx	Length of short channel reach	m
x_1	Distance of the section under consideration from cross regulator	Km
Y	A function of d_{65}/s	-
y	Coordinate; usually normal to water surface (vertical); exponent; distance from the boundary	m
Z	Actual exponent in suspended sediment distribution equation	-
Z_i	Dimensionless exponent in rouse equation	-
Z_t	Total energy in the flow of the section with reperence to a datum line is the sum of the elevation	m
$\Delta\rho$	Difference between the density of mixture and of the water, $\Delta\rho = (\rho_m - \rho)$	Kg/m^3
ρ	Density of fluid (annexure-1)	Kg/m^3
ρ_m	Density of the fluid mixture	Kg/m^3
ρ_s	Density of solid particle	Ton/m^3
ρ_f	Mass density of fluid	Ton/m^3
ρ_s	Specific weight of sediment	Ton/m^3
γ_s	Specific weight of sediment	Kg/m^3
γ_f	Specific weight of fluid, q_f=pg	Kg/m^3
$\Delta\gamma_s$	Difference is specific weight of sediment and fluid	Kg/m^3
τ_0	Average shear stress	Kg/m^2
τ'_c	Critical shear stress for particles of size d_{si}	Kg/m^2
τ_b	Effective stress	Kg/m^2
τ_0	Laursen's bed shear stress due to grain resistance; average shear stress corresponding to grain	Kg/m^2
τ_{*c}	Dimensionless critical shear stress	-
τ_{ci}	Critical value of τ_0 for sediment of size d_{si}	kg/m^2
τ_s	The shear stress responsible for suspended load transport	kg/m^2
τ_{oc}	Critical shear stress from shields curve	kg/m^2

τ_*	Dimensionless shear stress	Kg/m^2
τ'_*	Dimensionless shear stress to grains	Kg/m^2
τ_L	Limiting stress	Kg/m^2
α	Energy coefficient, ($\alpha = 1$ for small bed slope)	-
α_t	Coefficient in rough-turbulent equation	-
ν	Kinematic viscosity of fluid	m^2/s
ϕ	Angle of repose for the bed material	^{0}
ϕ_T	Total load parameter	-
Φ	Intensity (dimensionless) of solid discharge	-
ϕ_*, Ψ_*	Bed-load function	-
Φ_* or Φ	Intensity of transport	-
Ψ_*	Intensity of shear	-
ω_o	Fall velocity of particle under ideal condition	m/s
ω_i	Fall velocity of a grain of bed sediment of size d_{si}	m/s
ξ	A fraction indicating the part of the total area of the particle exposed to flow	-
ξ	A function of d_{si}/x	-
$\tan\theta$	Friction coefficient	-
ψ	intensity (dimensionless) of shear stress	-
ψ_D	Coefficient with dimension of cubic feet per pound per second	$ft^3/lbs/s$
Σ_i	Denotes summation for all sets at values of p_i, d_{si} and g_{ci}	-
η	Elevation of bed	m
δ	Thickness of boundary layer; factor in Wilson equation for velocity	m
τ_g	Geometric standard diviation of size	-
Δ	Apparent roughness diameter	m
z	Rouse exponent, $z = V_{ss}/k\ V_*$ and V_{ss}, settling velocity	-

CHAPTER 1

INTRODUCTION

1.1 GENERAL

Irrigation system consists of head work, intake structure, canal, drainage and appurtenance structures. Canal is one of the essential parts, which are designed on the basis of irrigation power water requirement. The function of the canal is to convey the water to feed the irrigation area, or a Power House.

Design of canal depend on discharge, topography, inner side slopes, with and without lining. Further more design of canal should ensure :

(i) Timely delivery of the required amount of water to the user in conformity with the designed water use schedules and water distribution pattern adopted

(ii) Insignificant water losses or minimum losses

(iii) Minimal land for canals

(iv) Reliability of operation, **non silting, non scouring and non weed** growing conditions

(v) Efficient operation with minimum cost

(vi) Low cost construction/economy.

A large number of theories and practices have been developed U.S. Reclamation Service (1915) Practice, Indian Practice Chow (1964), USBR Practice (1952), Bhakra Canal Manual Guidelines (1954), Central Water Commission (CWC)/Central Board of Irrigation and Power (CBIP) Guidelines, (1968) and (1984) and Indian Standard Codes. Even these have been changed from time to time.

Many variable factors are involved in the design of a lined canal. There are – (i) Bed width (B) (ii) Depth (D) ; or (i) and (ii) can be combined in B/D ratio ; (iii) inner side slope (Z) ; (iv) Bed slope (S) ; (v) Discharge and its variability according to requirement or availability ; (vi) Rugosity coefficient ; Sediment concentration and its variability ; (vii) Velocity and variability according to changing parameters.

The design of lined canal, particularly trapezoidal canals is largely done by trial and error by arbitrarily choosing the bed width dept ratio (B:D). This may result many times in uneconomical sections. Also their operational performance may not be as was expected in the designs. Lined canals have also shown considerable sign of damage to lining, seepage, silting, weed growth, and low discharging capacity. Indira Gandhi canal is one such example.

Best hydraulic sections are most economical as long as the cost of earth work (excavation and embankment) is less than the cost of lining per unit length of channel and this is the most usual case. These, sections, besides having least area and parameter also have minimum top width. Therefore land width required is also minimum, in comparison to other wide sections. Thus economic suitability of good section should also fulfill important criteria as :

(i) Minimum cost and practical economical section
(ii) Minimum seepage loss section
(iii) Minimum silting and minimum abrasion (for lined)
(iv) Minimum weed growth section
(v) Maximum permissible velocity section
(vi) Minimum and maximum permissible bed slope section

Manning's formula is widely used. Kennedy's formula (1895) and the rational empirical Lacey regime theory (1939) (with appropriate modifications by Inglis and others), although originating in India, are also used in many countries. The tractive force theory though developed for sediment transport , is now widely recommended by searchers and used as a check over the parameters calculated by one or other empirical formula.

Here effort is made to draw lessons from experiences of the canals constructed in the past. There are several important aspect as internal hydraulic sections of the canal with respect to discharge, topography, stability of side slope, type of lining, operation and maintenance problems and their solutions. Here attempt is made only for B/D ratio.

No lessons can be drawn without the experience in the past. In fact, much of engineering, rather all sciences have developed in bits and pieces from the experiences of the past. The engineers by and large have never become wiser in one day. They have become wiser slowly and slowly with their own experiences and from experiences of past engineering works. Wisdom lies to gain wisdom from experiences/draw lessons from the marvellous engineering works constructed so far. It helps us in a direction for further development of technological skills.

2.1 COMPARATIVE STUDY OF LINED CANAL SECTIONS IN INDIA

Design of economic and suitable channel section (operationally efficient) had been the concern of every engineer, from the day canal irrigation came to practice. Slowly and slowly many engineers developed theories and guidelines for such design, initially for unlined canals and subsequently for lined canals. With more stress on lining, the emphasis also shifted to the least perimeter for a given area. Chow (1964) has given experience curves of bed width and depth versus discharge. . This is only upto 3000 – 4000 cusecs (about 100 m^3/s).

Internal cross sections at head of some canals in India are given Table 1.1. Their bed width and depth versus discharge are plotted in Figure 1.1. This shows a big variation, and leads to more research and analysis on B/D ratio etc.

2.2 SCOP OF STUDY

The objective of this study is very much limited to Indira Gandhi canal to a scope as under :

(1) Analysis of sediment transporting capacity of Indira Gandhi canal.

(2) Analysis of flow characteristics of Indira Gandhi canal

(3) Analysis and review of hydraulic section of Indira Gandhi canal

(4) Draw lessons for future design or attempt on development of design criteria for future.

Table 1.1
Details of Lined Canal Sections of some Projects , India

S. NO	Name of Project	Head Discharge Q (m^3/s)	Channel Bed Slope S	Value of N	Velocity V (m/s)	Side Slopes (V : H)	Bed Width B (m)	Depth D (m)	B/D Ratio X	Wetted Perimeter P (m)	Area A (m^2)	Hydraulic Radius R=A/P (m)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
1	Narmada Main Canal, Gujarat	1132.83	1/12500 (0.00008)	NA	1.689	1:2.0	73.10	7.60	9.62	107.00	671.08	6.27
2	Rajasthan Feeder Canal, Punjab	523.90	NA	NA	1.425	1:1.5	79.25	4.40	18.01	97.63	389.10	4.40
3	Western Jamuna canal, Haryana	454.00	1/6250 (0.00016)	NA	NA	1:1.5	50.00	1.91	26.18	56.88	100.97	1.78
4	Nangal Hydel Channel, Punjab	354.00	1/10000 (0.0001)	NA	2.190	1:1.5	24.38	6.28	3.88	42.80	196.06	4.58
5	Sundernagar Hydle Channel Beas Project	254.85	1/6666 (0.00015)	NA	1.890	1:1.5	9.44	6.13	1.54	31.51	118.71	3.77
6	Gandak Canal	241.44	NA	NA	1.500	1:1.5	41.403	3.80	10.90	55.06	179.98	3.27
7	Western Kosi Canal, Bihar	236.70	1/8000 (0.000125)	NA	NA	1:1.5	5.06	3.66	9.58	50.34	156.28	3.10

Table 1.1 Continued

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
8	Satluj Yamuna Link Canal, Haryana	212.00	1/10000 (0.0001)	NA	NA	1:1.5	14.02	5.48	2.56	33.78	126.09	3.73
9	Mahi Right Bank Canal, Gujarat	198.10	1/7000 (0.00014)	NA	1.520	1:1.5	16.46	6.23	3.34	32.24	111.52	3.46
10	Bhakra Canals	192.00	1/8000 (0.000125)	NA	NA	1:1.5	12.30	4.57	2.69	28.78	87.56	3.04
11	Jawaharlal Nehru Left Canal, Haryana	90.08	1/5000 (0.0002)	NA	NA	1:1.5	20.25	2.62	7.73	29.71	63.44	2.14
12	Augmentation Canal, Haryana	87.60	1/5000 (0.0002)	NA	NA	1:1.5	13.40	3.35	4.00	25.48	65.16	2.56
13	Dhansiri Main Canal	56.60	1/1500 (0.000667)	NA	NA	1:15	14.44	1.61	8.97	20.24	27.14	1.34
14	Loharu Lift Canal, Haryana	39.65	1/7407 (0.000135)	NA	NA	1:1.5	21.17	1.72	12.31	27.38	40.94	1.50
15	Ukai Left Bank Canal, Gujarat	35.00	1/8000 (0.000125)	NA	NA	1:1.5	6.50	3.00	2.17	17.32	33.00	1.91
16	Damanganga Right Bank Canal , Gujarat	34.78	1/1000 (0.001)	NA	NA	1:1.5	4.50	2.60	1.70	15.87	21.87	1.38
17	Bardikarai main canal	30.01	NA	NA	NA	1:1	15.70	1.38	11.38	20.00	25.07	1.254

Table 1.1 continued

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
18	Karjan Reservoir Left Bank Canal, Gujarat	28.30	1/3000 0.000333	NA	NA	1:1.5	3.89	2.70	1.44	13.62	21.44	1.57
19	Jui Lift Canal, Haryana	21.14	1/6060 0.000165	NA	NA	1:1.5	10.20	1.77	5.76	16.57	22.74	1.37
20	Salauli Irr. Project, Goa Dam & Diu	14.00	1/5000 0.0002	NA	NA	1:1.33	5.95	1.65	3.61	11.44	13.44	1.17

Notes : Incomplete due to unavailability of data, NA = not available
For 454 m^3/s, and A = 100.97 m^2, V = 4.496 m/s which is very high, and data may be wrong

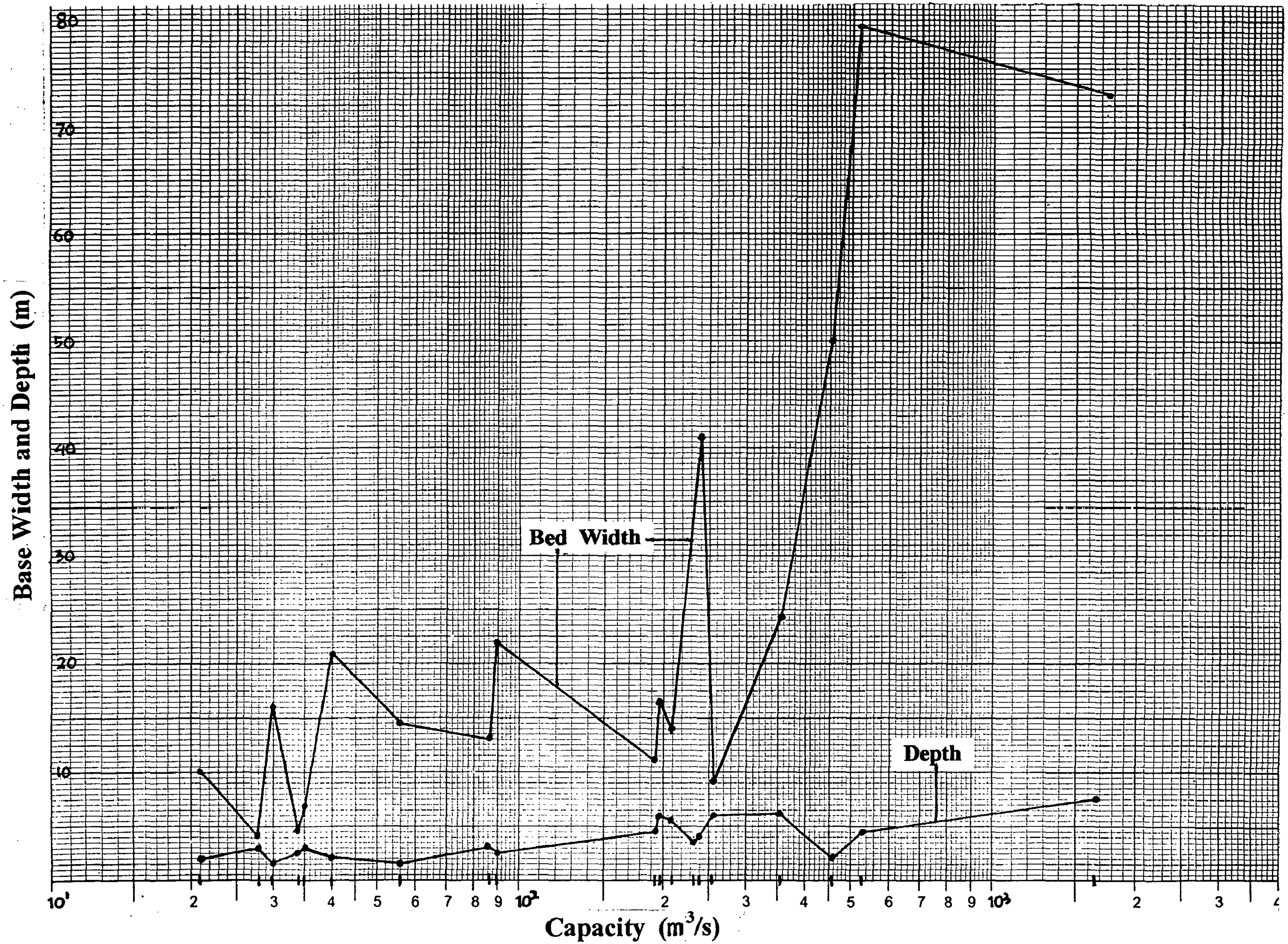
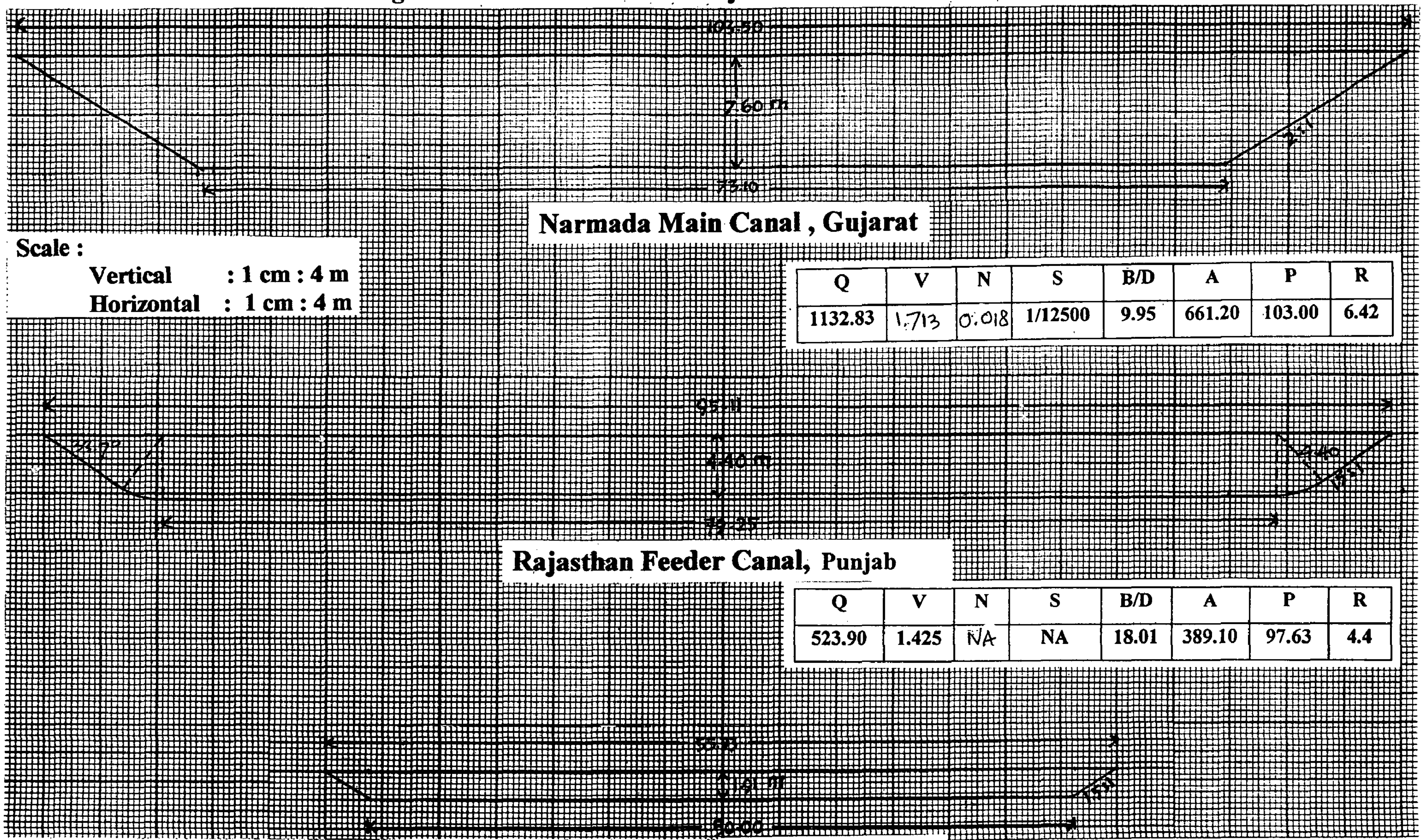


Fig. 1.1 Basse Width and Water Depth, Lined Canal in India

Fig. 1.2 Cross Section of Some Project Lined Canal in India



Scale :
Vertical : 1 cm : 4 m
Horizontal : 1 cm : 4 m

Narmada Main Canal , Gujarat

Q	V	N	S	B/D	A	P	R
1132.83	1.713	0.018	1/12500	9.95	661.20	103.00	6.42

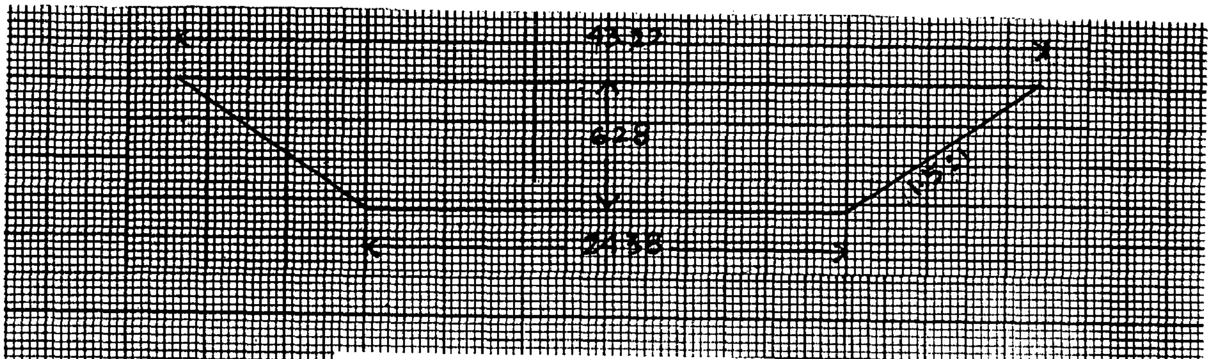
Rajasthan Feeder Canal, Punjab

Q	V	N	S	B/D	A	P	R
523.90	1.425	NA	NA	18.01	389.10	97.63	4.4

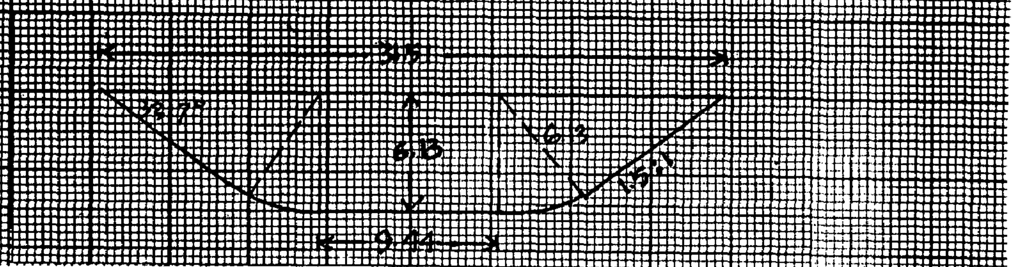
Western Jamuna Canal, Haryana

Q	V	N	S	B/D	A	P	R
454.00	4.50	NA	1/6250	26.18	100.97	56.88	1.78

All data are in metric units
Source : CBIP, Major Canals in India

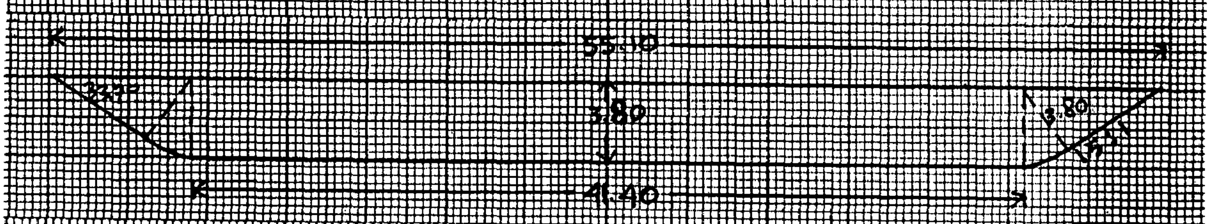


Q	V	N	S	B/D	A	P	R
354.00	2.190	0.013	1/10000	3.88	196.06	42.80	4.58

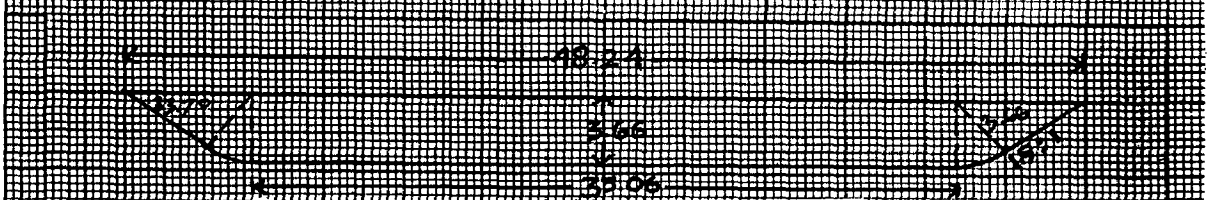


Sundernagar Hydle Channel Beas Project, Himachal Pradesh

Q	V	N	S	B/D	A	P	R
254.85	1.890	0.016	1/6666	1.54	118.71	31.51	3.77

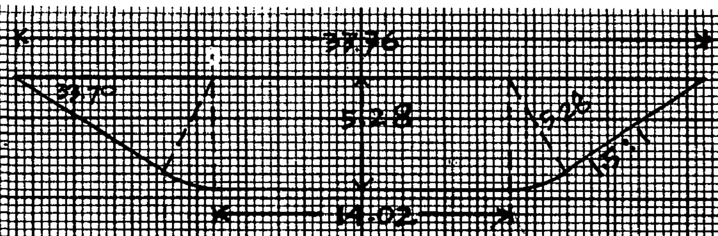


Q	V	N	S	B/D	A	P	R
241.44	1.500	0.013	NA	10.90	179.98	55.06	3.27



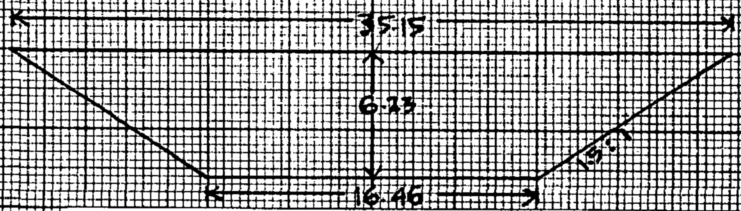
Western Kosi Canal, Bihar

Q	V	N	S	B/D	A	P	R
236.70	1.515	0.017	1/8000	9.58	156.28	50.34	3.10



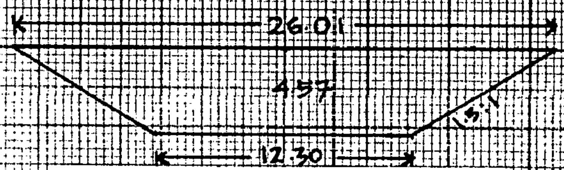
Satluj Yamuna Link Canal, Haryana

Q	V	N	S	B/D	A	P	R
212.00	1.681	0.014	1/10000	2.56	126.09	33.78	3.73



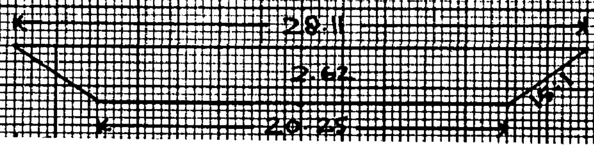
Mahi Right Bank Canal, Gujarat

Q	V	N	S	B/D	A	P	R
198.10	1.520	0.018	1/7000	3.34	111.52	32.24	3.46



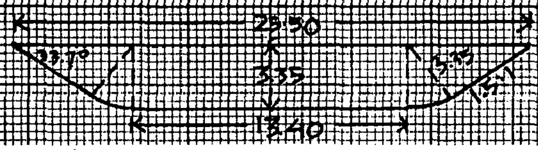
Bhakra Canals, Haryana

Q	V	N	S	B/D	A	P	R
192.00	2.193	0.011	1/8000	2.69	87.56	28.78	3.04



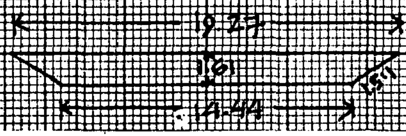
Jawaharlal Nehru Left Canal, Haryana

Q	V	n	S	B/D	A	P	R
90.08	1.420	0.018	1/5000	7.73	63.44	29.71	2.41



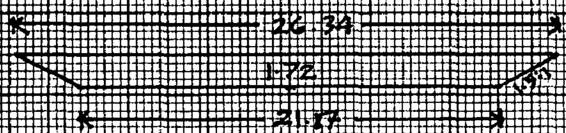
Augmentation Canal, Haryana

Q	V	n	S	B/D	A	P	R
87.60	1.344	0.020	1/5000	4.00	65.16	25.48	2.56



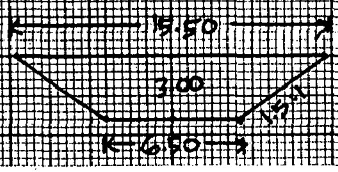
Dhansiri Main Canal, Assam

Q	V	n	S	B/D	A	P	R
56.60	2.085	0.015	1/1500	8.97	27.14	20.24	1.34



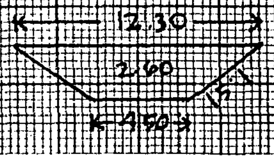
Lohuru Lift Canal, Haryana

Q	V	N	S	B/D	A	P	R
39.65	0.968	0.016	1/7407	12.31	40.94	27.38	1.50



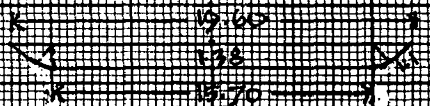
Ukai Left Bank Canal, Gujarat

Q	V	n	S	B/D	A	P	R
35.00	1.061	0.016	1/8000	2.17	33.00	17.32	1.91



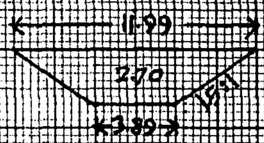
Damanganga Right Bank Canal, Gujarat

Q	V	n	S	B/D	A	P	R
34.78	1.590	0.025	1/1000	1.70	21.87	15.87	1.38



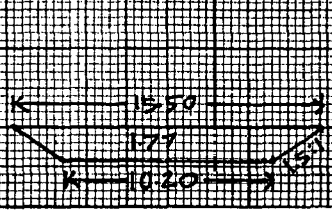
Bardikarai Main Canal, Assam

Q	V	n	S	B/D	A	P	R
30.01	1.197	NA	NA	11.38	25.07	20.00	1.25



Karjan Reservoir Left Bank Canal, Gujarat

Q	V	n	S	B/D	A	P	R
28.30	1.320	0.019	1/3000	1.44	21.44	13.62	1.57



Jui Left Canal, Haryana

Q	V	N	S	B/D	A	P	R
21.14	0.930	0.017	1/6060	5.76	22.74	16.57	1.37

CHAPTER 2

CANAL DESIGN THEORIES AND SEDIMENT TRANSPORT

2.1 GENERAL

All canals, whether lined or unlined carry some silt. The flow of silt may be more in case of direct flow from weirs/barrages or low dams or diversion schemes, particularly when situated in hills or hill toes. The silt inflow is less in canals taking off from reservoirs. In reservoirs much of the silt is deposited in it and clean water passes down in the canal. However, some silt may also pass in the canal depending upon the inflow release pattern and time, location of canal out let and sill level, water level in the reservoir during operation. (for example in moon soon season)

Silt inflow in a canal varies according to the silt inflow in the river, and the flow diversion. Rivers receive huge quantity of sediment along with water due to erosion of drainage basin. Silt may also enter into a canal from the topography, through which the canal is passing, such as wind blown sand and rain cuts on the cut slopes or the storm water inflow in the canal at inlets. All canals of IGNP, Rajasthan, India, are subject to wind blown sand/silt into the canal. Typical example of storm water inflow into the canal are the inlets of Upper Ganga Canal, U.P., India. In some canals, failures of inner slopes of banks have also caused silts and debris. This is also a seasonal and occasional phenomenon and is not uniform over time.

Silt is very harmful in power canals. It damages and erodes the blades of the turbines. In irrigation canals silt is useful when transported to the fields. It has high agricultural productivity and is beneficial to the crops. But this silt becomes harmful in irrigation canal also, when deposited in it. It blocks/reduces water way, carrying capacity/discharges and may encourage weed growth on sides. When at high velocity it has an abrasion effect on lining or damages it. In earthen channels the bed and banks are eroded. The phenomena varies from project to project, velocity and nature of banks. Low velocity helps in deposition of silt strengthening the banks.

An analysis of sediment inflow in river and canals is very essential for proper design and regulation. The objectives of a canal design, operation and maintenance are :

(i) Exclude entry of silt, debris (or sediment) in the canal, as much as possible, i.e. provide Silt Excluders at the source.

(ii) Whatever has entered may be ejected from a canal to the extent possible, so provide Silt Ejectors, at appropriate locations. Pass out some portion of water according to silt inflow.

(iii) Also entrap the maximum silt or as much as possible in Silt traps/Silt tanks or desalting chambers and flush out or eject at suiTable locations. It may be called as a modified Silt Ejector.

(iv) And lastly channel design should be such that it is non-silting and non-scouring i.e. carries the silt and passes out to the distribution system and to fields and does not settle or erode bed. But it can be permitted only in Irrigation canals and not power canals. Since the sediment concentration change with time in a year and available water or discharge change. Therefore such design may not be feasible. A balance design can be attempted.

2.2 SEDIMENT IN RIVERS

Observation done in various rivers show that sediment load in river (streams) from which canals are fed seldom exceeds 5,000 ppm with an annual average of few hundreds of ppm. Concentrations of sediment coarser than 0.075 mm diameter for various locations along the Chenab river in West Pakistan are shown in Figure 2.1. The individual lines are visually placed average of hundreds of readings which show wide Scatter in the extremes. The effect of curvature in the river is demonstrated by comparing values for the Trimmu 1961 left bank with those for the Trimmu 1961 rigth bank. A gradual change in curvature may account for the different between the Trimmu left bank values for 1939 and 1961.

The behavior of sediment depend on the specific gravity, size, and shape of the particles and the size distribution. A typical sample of sediment consists of a mixture of particles of various densities, sizes, and shapes. The variation of density is generally small, and for practical purpose the mean density may be used for all the sediment. The density is in most cases so near the density of quartz (specific gravity 2.65) that this value may be used without significant error in the formulas for sediment transport.

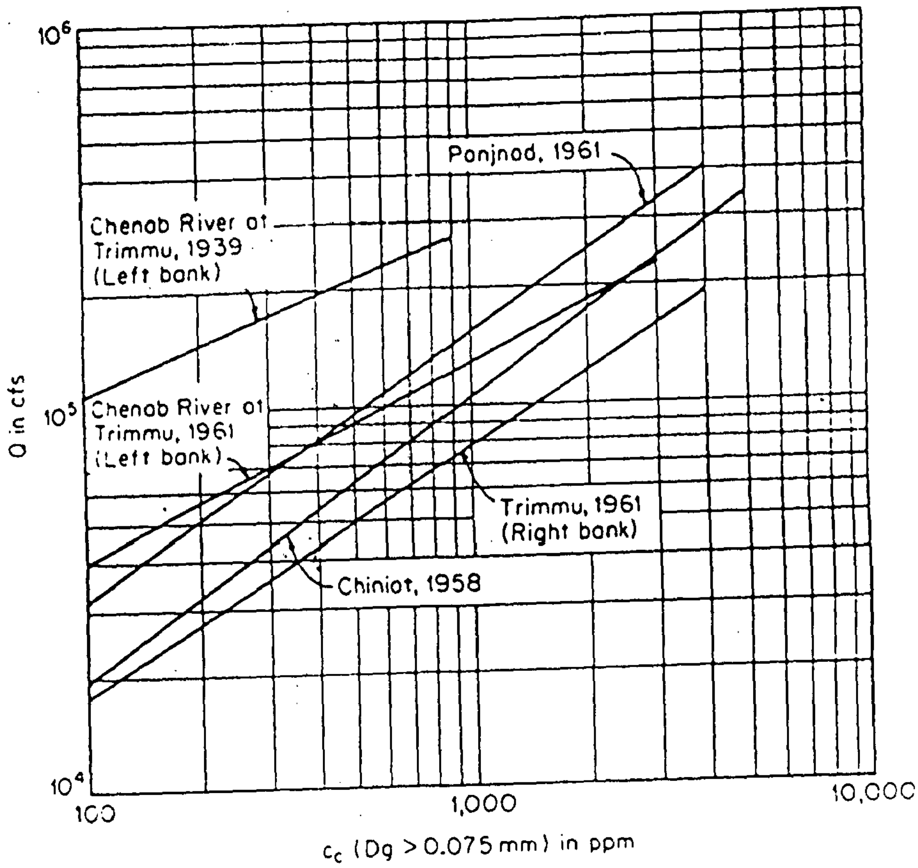


Fig. 2.1 Sediment Concentration vs Discharge, Chenab River, West Pakistan (Davis, 1952)

The size distribution of a sediment mixture can be represented by soil classification and grain size distribution curve or also some times known as frequency diagram as in Figure 2.2, which shows characteristic grain sizes for typical suspended sediment and bed materials. The mean or effective diameter of a mixed sediment is often described by its median or 50 percent size.

Typical grain sizes of suspended and bed load, sampled in eight canals in West Pakistan, are shown in Figure 2.2. The effect of alluvial sorting is indicated by the narrow range of grain sizes of the bed material. In most canals the sizes of the bed material and suspended sediments usually show a marked seasonal variation.

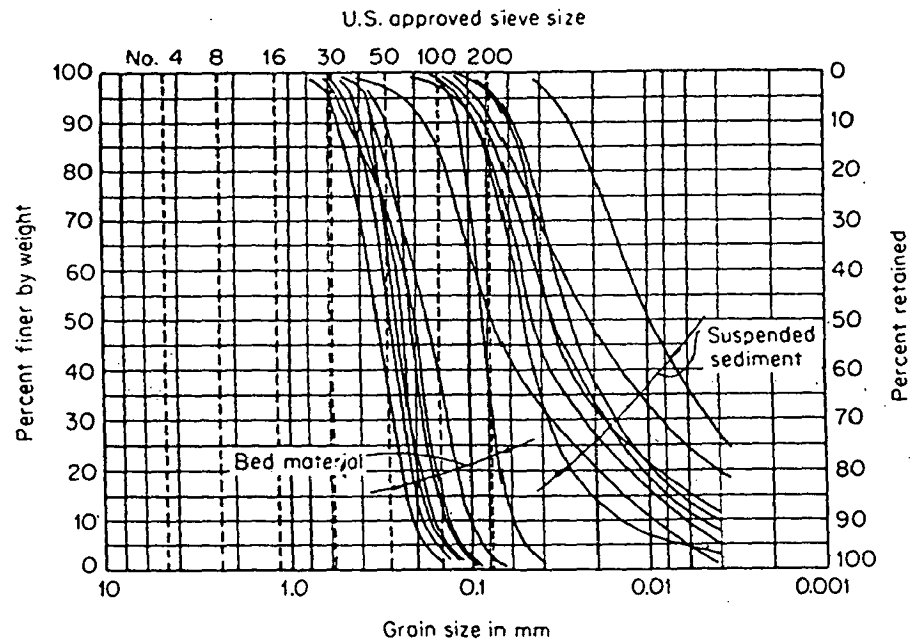


Fig. 2.2. Grain Size Distribution of Suspended and Bed Sediments in Canals (Davis, 1952)

Finer suspended sediments (less than about 0.06 mm) have measurable effect on the performance of a canal, the impairment of canal performance usually results from an excess of coarser material (greater than about 0.06 mm). Accordingly, it is concentration of the coarse size and the variation in such concentration that are significant in canal operation. A typical annual variation, as measured near the head of the Upper Chenab Canal, West Pakistan, is show in Figure 2.3.

The transport capability of a channel is a function of capacity, measured as the quantity of sediment which will be moved, and competence, measured by the maximum size of bed particles which will be moved. Capacity increases with decrease in particle size, and transport capability in any given size range can be for greater than the volume of sediment available.

The size of transport table grain in indicated by the tractive force. In a channel the bed material is usually composed of sediment which deposited during periods of decreased competency or sufficient transport capacity.

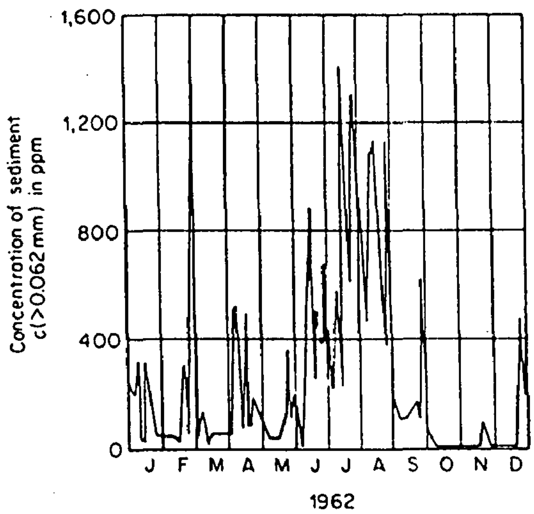


Fig. 2.3. Annual Variation of Sediment Concentration -Upper Chenab Canal, West Pakistan (Davis, 1952)

2.3 CLASSIFICATION OF SEDIMENT LADEN WATER

The gravitational flow a water-sediment mixture or sediment laden water or fluid or simply often called water, can be is distinguished in three types of movement as under :

(i) Non Newtonian mixture

The mixture behaves Non-Newtonian, if the volumic concentration becomes of importance, C_s > 8% (80,000 ppm). The difference between the density of the mixture and of the water is also very large, $\rho > 130$ Kg/m^3.

The flow of a Non-Newtonian fluid modifies all concepts of Newtonian hydraulic, such as the resistance to the flow, as well as the distribution of velocity and of concentration, the settling velocity is also influenced and the solid particles stay longer in suspension. The transport of sediments as hyper concentrated suspension and the debris flow, as well as hyper concentrated turbidity currents fall into this category:

- The transport of sediments as a hyper concentrated suspension is encountered in rivulets (nalah). Usually enormous quantities of sediments being of small size enter the channel due to surface erosion caused by extensive rainfalls in the catchment basin. The soil particle stay usually for long time periods in suspensions, as wash load.

- Torrential flow of debris may establish themselves at rather steep slopes, $S_0 > 15^0$. All kinds of particles, from the finest (having cohesion) to the largest (blocks of 1 m^3) participate in the movement, which is rather rare in occurrence and at short duration, and is usually caused severe rainfall.

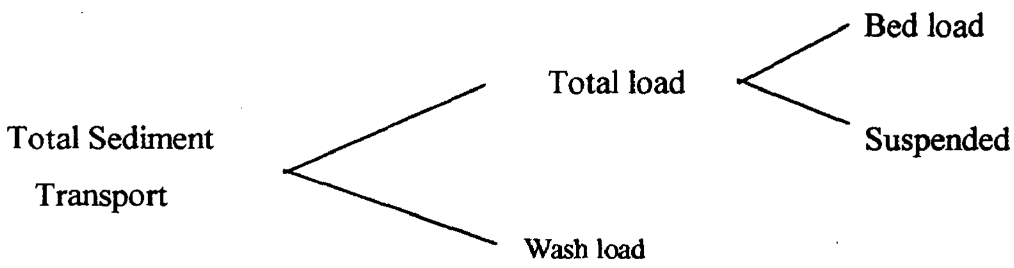
(ii) Quasi - Newtonian mixture.

The mixture behaves quasi-Newtonian, if he volumic concentration remains small, $C_s < 8\%$. He difference between the density of the mixture and of the water becomes important, $\Delta\rho < 130$ Kg/m^3.

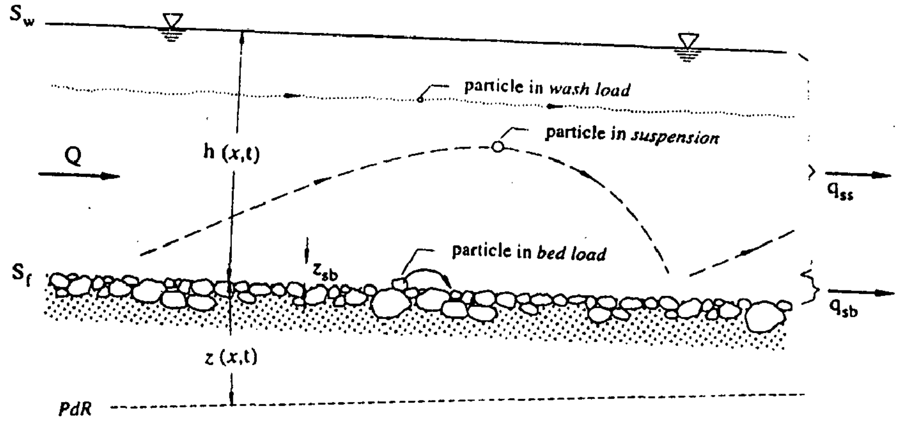
The transport of sediments as concentrated suspension notably close to the bed, as well as the turbidity currents fall into this category.

(iii) Newtonian mixture

The mixture may be considered Newtonian, if the volumic concentration of the particles is very small, $C_s << 1$ % (10,000 pm). The difference between the density of the mixture and of the water, $\Delta\rho_3 = (\rho_m - \rho) = (\rho_s - \rho)\, C_s$ remains also small, $\Delta\rho <<16$ Kg/m^3. The transport of sediment (see Figure 2.4). As bed load and a suspended load, fall into this category. This type of transport of solid particles, which is most often encountered in rivers at foot mills.



(a) Sediment Transport



(b) Scheme of the Modes of Transport
Figure 2.4 Sediment Transport (Graf,1984)

2.4 SEDIMENT LOAD

It is the total of the sediments that move either in suspension or in contact with the bed. It is the sum of suspended load and bed load. Alternatively, it is the total of bed material load and the wash load as follow :

(i) Bed load is the sediment in almost continuous contact with the bed while carried by rolling, sliding or hopping along the bed of the stream.

Bed load is also divided into contact load and saltation load.

a) Contact load is the sediment that is rolling or sliding along the bed of the stream in substantially continuous contact with the bed.

b) Saltation load is the sediment bouncing and hopping along the bed of the stream or moved directly or indirectly by the impact of the bouncing particles.

(ii) Bed material is bed material, the particle sizes of which are found in appreciable quantities in the shifting portions of the bed.

(iii) Bed material load is the coarse part of the sediment load which consists of particle sizes represented in the bed (that is bed material) which is limited in its rate of movement by the transporting capacity of the channel.

(iv) Suspended load is part of the sediment load of a stream which remains in suspension in the flowing water considerable periods of time without contact with the stream bed, being kept up by the upward component of the turbulence or by colloidal suspension and which moves practically with the same velocity as that of flowing water.

(v) Wash load is part of the suspended load which is composed of particle sizes smaller than those found in appreciable quantities in the shifting portions of the stream bed. It is in near permanent suspension and is transported entirely through the stream without deposition. The discharge of the wash load through a reach depends only on the rate with these particles become available in the catchment and not on the transport capacity of flow.

(vi) Sediment concentration is the ratio of dry weight of sediment in water sediment mixture to the total weight of a suspension. It is generally expressed in grams per liter or parts per million (by weight)

Contact load, saltation load, and suspended load may occur simultaneously and the border lines between these are not well defined. This difficulty is avoided in practice by dividing the total load into suspended load and bed load. The bed load moves at a lower velocity than the layer of water through which it is traveling, the traction on it being exercised through the fluid drag. The total load may also be divided into bed material load and wash load the former constituting the coarser part of the sediment load moved by the transporting capacity of the channel which may settle and the latter the fine suspended material which does not settle in the existing conditions of flow.

2.5 GENERAL RELATIONSHIPS.

Table 2.4, first presented by Kennedy and Brooks (1963), lists the variables involved in determining the behavior of alluvial channels and classifies them into several sets of independent and dependent groups. Each of the dependent variables can be determined as a function of the independent variables. In some cases the functions are known and dependent variables can be determined easily. Perhaps the simplest such function is the continuity equation stating that discharge Q is equal to the product of stream width b, depth d, and mean velocity V or $Q = bd\,V$. In the first line of Table 2.4 for the case of flumes, the independent variables are fluid properties of kinematic viscosity, ν; mass density, ρ; sediment properties of density, ρ_s; geometric mean size, d_g; geometric standard deviation of sizes, σ_g; fall velocity, ω; the acceleration of gravity, g; and flow system characteristics Q, b, and d. the dependent variables are sediment discharge, Q_s; mean velocity, V; hydraulic radius of cross section, r; energy gradient, S; and Darcy-Weisbach friction factor, f.

The sediment discharge, Q_s, can be expressed as

$$Q_s = f(Q, d, b, \nu, \rho, \rho_s, d_g, \sigma_g, \omega, g) \quad \text{.....(i)}$$

In a particular flume of a given width the fluid and sediment properties can be kept constant and b and g are constant and the discharge can be replaced by V by means of the continuity equation. When the depth and sediment and fluid properties are kept constant the relation reduces to

$$Q_s = f(V) \quad \text{.....(ii)}$$

Such a relation from experiments by Vanoni and Brooks (1957) is shown in Fig. 2.5. Other data from this set experiments are shown in Fig. 2.6 in which slope, bed shear velocity, V_{sb}, and bed friction factor, f_b, are plotted against V.

Table 2.1
Choices of Independent and dependent Variables for Flow and Sediment in Alluvial Streams (Adopted from Kennedy and Brooks, 1963)

	Independent Variables[a]		
System	Properties of fluid, sediment, gravity, etc.	Characteristics of flow systems (not all combinations listed)	Dependent[a] variables (not all combinations listed)
(1)	(2)	(3)	(4)
Flumes	$\nu, \rho, \rho_s, d_g\ \sigma_g,$	Q, d, b	Q_s, V, r, S, f
	ω, g	Q, Q_s, b	D, r, U, S, f
		V, d, b	Q, Q_s, r, S, f
		D, s, b	Q, Q_s, r, V, f
		R, S, b	Q, Q_s, d, V, f
		Q, S, b	G_s, d, r, V, f
Natural streams	$\nu, \rho, \rho_s, d_g\ \sigma_g,$	Q, d	G_s, b, r, V, S, f
Short term	ω, g	d, S	Q, Q_s, b, r, V,S, f
		r, S	Q, Q_s , b, d, V, f
		Q, S	Q_s, b, d, r, V, f
Long term	ν, ρ, ρ_s, g	Q, Q_s	b, d, r, V, S, f
(graded stream)			$d_g\ \sigma_g$, ω
Very long	ν, ρ, ρ_s, g	Climate, man-	Q, Q_s, b, d, r, V,
Term	Geology	Made works	S, f, d_g, σ_g, ω

[a]Note that plan-form geometry and wash-load concentration are not considered.

Source : ASCE (1975)

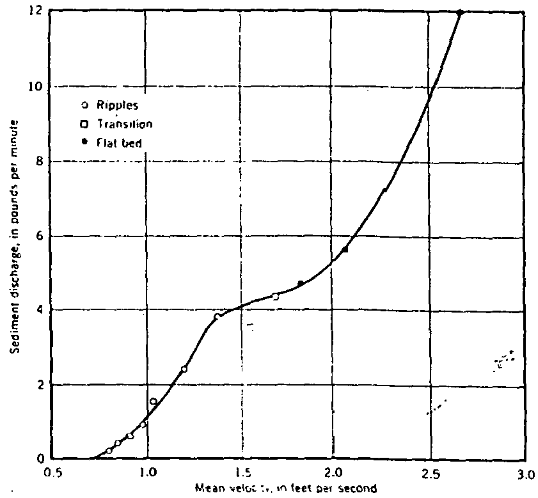


Fig.2.5 Sediment Discharge as Function Of Mean Velocity for Flow 0.241 ft Deep in Bed of Fines and (flum Width = 10.5 in., Bed Sediment Size D_{50} = 0.152 mm, $\sigma_g = 1.76$) (Kennedy and Brooks,1963)

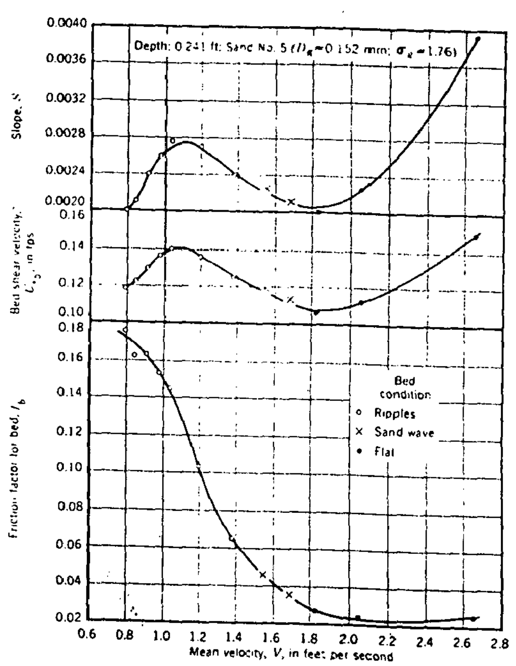


Fig.2.6 Variation of Slope, Bed Shear Velocity and Bed Friction Factor in Constant Depth Flume Experiments (flume width = 10.5 in) Vanoni and Brooks (1957)

2.6 TRACTIVE FORCE THEORY

When water flows in a channel, a force is developed that acts in the direction of flow on the channel bed. This force which is simply the pull of water on the area of wetted perimeters (i.e. perimeter x length) is known as the tractive force.

This approach is based on the consideration of equilibrium of a sediment particle resting on the bed under the action

(i) Drag

(ii) Lift force

(iii) Tractive force caused by the following fluid and the submarged weight of the particle.

Considering steady uniform flow in a rectangular channel and consider equilibrium of a water prism abcd under various forces acting on it Fig.2.7 Since there is no acceleration of the fluid, the summation of all the forces acting in the direction of flow must be zero.

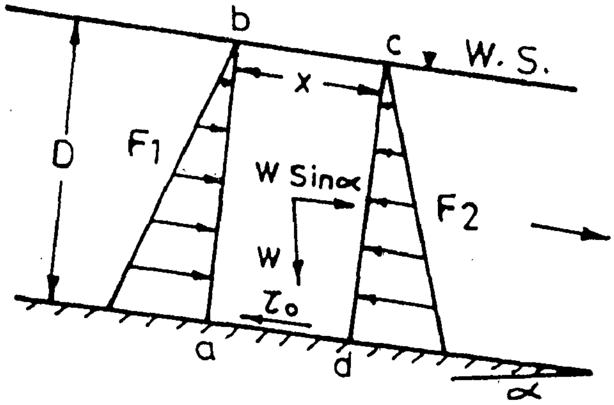


Fig. 2.7. Forces Acting on a Water Prism (Garde, 2000)

Hence $\sum F = F_1 + w \sin \alpha - F_2 - \tau = 0$

Where

F_1, F_2 = hydrostatic forces

W = weight of the water prism per unit area of wetted perimeter

τ_o = average shear stress of the boundary since the depth of flow is the same at section ab and cd

$$F_1 = F_2 \text{, and } \tau_o = \frac{w \sin \theta}{\text{area of wetted perimeter}}$$

Where :

Area pf wetted perimeter = $P.\Delta L$

$W = A\Delta L\gamma_f$; γ_f = specific weight of fluid $\simeq 1$

Therefore,

$$\tau_o = \frac{A\Delta L\gamma_f \sin \theta}{P.\Delta L}$$

$\tau_o = \gamma_f R \sin\theta$; For small θ , $\sin\theta = \tan\theta = S_0$ (channel slope).

$\tau_o = \gamma_f R.S_o$

τ_o is not dimensionally similar so use consistent units on both sides.

Where :

R = hydraulic radius

ΔL = length of the prism

A = area of prism

P = area of wetted perimeter

The force exerted by water on the channel bed will have the same magnitude but will act in the direction of flow. This shear stress can be directly related to the velocity distribution near the boundary and the viscosity of the fluid.

Direction of shear stress needs to be known in the assessment of channel stability. Hence essentially the design reduces to the determination or the following :

(i) Distribution of shear stress along the periphery

(ii) Limiting tractive stress for various material

(iii) Effect of side slopes of limiting tractive forces

(iv) Relation between roughness coefficient and sediment size.

If coarse sediment inters a channel at a rate which exceeds, its carrying capacity, part of sediment is dropped in the channel bed. This carrying capacity of the canal can be changed by :

(i) Changing the discharge of the canal

(ii) Its slopes

(iii) Its shape

(iv) Changing of the particle size of the sediment

The problem of design of the stable channel carrying sediment. Therefore, involves the determination of the hydraulic factors for a canal which will have a transporting capacity sufficient to carry it.

Tractive for on side and bed of various channel section have been prepared for channel design and show in Figure 2.8. The relationship of side slope, angle of repose and critical tractive force, for such section the distribution of applied stress as work out by Lane et.al at U.S.B.R in 1952, on the resistance side the material is divided into three part :

(i) Coarse non-cohesive

The hydraulic roughness depend on the grain size lining the canal after some of the fines have been washed out and is important. For coarse non cohesive more than 5 mm size, the critical tractive force (limiting stress, τ_L) is given as,

$$\tau_L = 0.736\ d_{75}$$

Where τ_L is in kg/m^2 and d_{75} in mm. Due to available armoring effect the representative size used is d_{75} in place of d_{50}.

The particle resting on the banks is acted upon by gravity in addition to water drag. This is to be accounted for by reducing the limiting stress, on the bank particle by a factor

$$K = \cos\alpha\sqrt{1-\frac{\tan^2\alpha}{\tan^2\phi}}$$

Where α is the angle of the side slope or bank, with the horizontal and ϕ is the angle of internal friction of t he bank soil.

(ii) Fine non-cohesive material

The size materials range 0.1 to 5.0 mm for these materials, besides the armoring effect, the influence of adhesion imparted by fine clay and silt sizes suspended in the water is considered important. The roll down effect on bank particle is applicable to this case also. For both of the above materials the channel is to be so designed that the applied maximum stress on bed and banks does not exceed the resistance of either. Thus limiting stress are given in the Table 2.2

(iii) Cohesive material

For this material the inter-particle forces are much more important than gravity forces, and hence the roll down effect may be neglected. For any important project that the most important characteristic determining erosion resistance of clay soils are plasticity index and void ratio are given in Table 2.3 and Figure 2.9

It may be found more economical to provide local protection at bends rather than design the channels for these reduced values of tractive force.

Table 2.2

Limiting Stresses for Fine Non-cohesive Material (less than 5 mm)

Median size of material (d_{50}) mm	Limiting tractive force K/m^2		
	Clear water	Light load of fine sediment	Heavy load of fine sediment
0.1	0.122	0.241	0.369
0.2	0.125	0.250	0.375
0.5	0.145	0.265	0.400
1.0	0.193	0.290	0.435
2.0	0.290	0.386	0.530
5.0	0.675	0.795	0.890

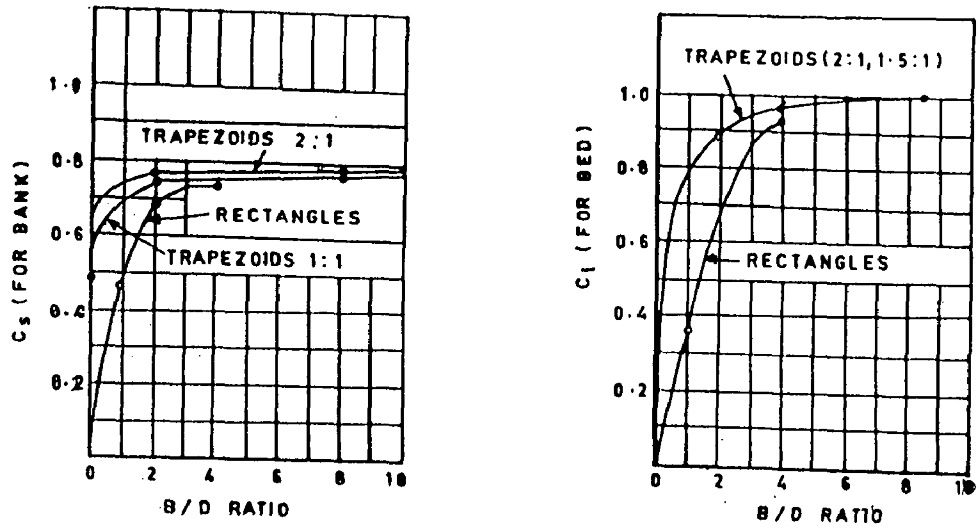
Source : Bharat Singh (1982)

Table 2.3

Typical Limiting Stresses in Cohesive Material

Type of soil	Etchevery	Scobey	
		Clear water	Silty water
	Permissible stress kg/m^2		
Sandy loam	3.32 to 3.97	1.74 to 2.28	3.55 to 5.20
Average loam, alluvial soil	Silty loam 3.97 to 4.78	-	-
Firm loam, clay loam	Alluvial silts 4.78 to 7.43	2.28 3.55	7.11 7.11
Stiff clay soil	13.15 to20.60	12.26	21.78

Source : Bharat Singh (1982)



(a) – on Bank **(b) – on Bed**

Fig.2.8 Maximum Stresses on Bed and Banks (Show, 1959)

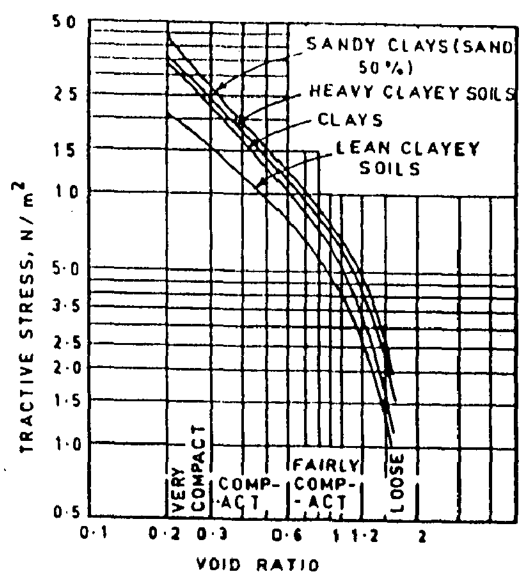


Fig.2.9 Permissible Tractive Stresses for Cohesive Soils (Chow 1959)

2.7 EMPIRICAL EQUATION FOR SEDIMENT DISCHARGE

Many formulaes have been given by various investigators, after Du Boys (1879) presented his tractive force. The results of different formulaes differ drastically. So far it has not been possible to determine positively which method gives most realistic or reliable result. Therefore selection is no straight forward.

Some of the important formulaes, which are used by many Engineer is presented in Table 2.4 to 2.6. These formulaes are presented in a systematic way as under,

(i) Table 2.4 bed load transport equation

(ii) Table 2.5 suspended load transport equation

(iii) Table 2.6 total load transport equation.

2.7.1 Bed Load Transport

The particle move in different modes depending on the flow conditions, the ratio of densities of the fluid and the sediment, and the size of the sediment. Movement of sediment particles is by rolling or sliding along the bed in substantially continuous, when the particles stay in close contact with the bed, sediment transported in this way is known as bed load. For determining the rate of bed load transport various equation have been given different investigators in the Table 2.4.

2.7.2 Suspended Load Transport

The bed material of a alluvial channel moves as contact load or saltation load the stream will have only clear-water at low values of average shear stress on the bed. Further increase in the shear stress, some of the bed particles are carried into the main flow and thus lose contact with the bed. These particles travel with a velocity almost equal to the flow velocity and the constitute the suspended load.

Suspended load transport is a advanced stage of the bed load transport. Thus in the case of uniform sediment, one would expect only bed load transport at low shear stresses, while at high shear stress both the bed transport and suspended load transport would occur. In the case of non-uniform sediment, the finer sizes of the bed material may move predominantly in suspension, while the coarse fraction of the bed material may move mostly (or totally) as bed load, if they move at all. For determining the rate of suspended load transport various equation have been given by different investigators in the Table 2.5

2.7.3 Total Load Transport

The methods of computation of the total sediment transport rate can be broadly classified in two categories namely :

(i) Microscopic Method. It subdivides the total sediment load either into suspended load and bed load

(ii) Macroscopic Method process of suspension based on dimensional analysis, intuition or complete empiricism.

For determining the rate of total load transport various equation have been given by different investigators in the Table 2.6

Table 2.4
Bed Load Sediment Transport
Empirical Equations for Sediment Discharge

Investigator and His Formula (1)	Units (2)	Remark (3)
2.4.1 Du Boys (Brown, 1950) $q_{sb} = \psi_D \tau_o (\tau_o - \tau_c)$ $\tau_0 = \gamma_\phi R.S_0$	FPS	Value of ψ_D and τ_c obtained by Straub (1935) and reported in Brown (1950), given as functions of median size of the bed sediment (d_{50}) in Fig 2.10. Includes data of Gilbert (1914) and Johnson (1943) flume experiments
2.4.2 Meyer Peter (Meyer Peter and Muller, 1948) $q_{sb}^{2/3} = 39.25 q^{2/3} S_0 - 9.95 d_{50}$ $q_{sb}^{2/3} = 250 q^{2/3} S_0 - 42.5 d_{50}$	FPS Metric	It is valid only for beds of relatively coarse sediments for which the flow resistance due to bed forms is a small part of the total resistance-sediment size from 3.1 mm to 28.6 mm. Includes flume data of Gilbert (1914).
2.4.3 Schoklitsch (Shulits, 1935) $q_{sb} = \sum_i P_i \frac{25.3}{\sqrt{d_{si}}} S_0^{3/2} (q - q_{ci})$ $q_{ci} = 0.638 \frac{d_{si}}{S_0^{4/3}}$	FPS	q_{ci} is the critical value of q for initiating motion of sediment of mean size, d_{si}. For values of P_i and d_{si} a mechanical analysis of a representative sample of the bed sediment is made and a size distribution curve prepared. P_i values can be determined form the size distribution curve for corresponding to d_{si} .The formula is based on Gilbert data (1914), for graded sediment with median size from 0.3 mm to 5 mm
2.4.4 Shield (Shield, 1936) $q_s = 10 q S_0 \frac{(\tau_0 - \tau_c)}{\left(\frac{\gamma_s}{\gamma_f} - 1\right)^2 d_{50}}$	Dimensionally homogeneous, FPS or metric, use consistent units both sides	τ_c is the critical bed shear stress for sediment of size d_{50} given by Shields Graph, fig. 2.11. Median sediment size range from 1.7 mm to 2.5mm.

table 2.4 continued

Investigator and His Formula (1)	Units (2)	Remark (3)
2.4.5. Meyer Peter and Muller Formula (1948) $$q_s^{2/3} = \frac{\left[\left(\frac{kr}{k'r}\right)^{3/2} \gamma_f R.S - 0.097(\gamma_s - \gamma_f)\, dm\right]}{0.25\left(\frac{\gamma_f}{g}\right)^{1/3}\left(\frac{\gamma_s - \gamma_f}{\gamma_s}\right)^{2/3}}$$	Dimensionally homogeneous, FPS or metric, use consistent units both sides	The advantage of this formula over the other Meyer Peter formula see item 2.4.2 is that it can be used for graded sediments under flow conditions that give rise to dunes and other bed forms. f'b is Darcy -Weisbach bed fraction factor for the sand grain roughness , show in fig 2.12 (a). The mean diameter of sediment range from 0.4 mm to 30 mm. S^1 is portion of channel bed slope due to grain roughness, k_r and k'_r are determined from some value of V.
$$\frac{kr}{k'r} = \sqrt{\frac{f'b}{8}}\frac{V}{\sqrt{g\,RS_o}}$$	-do-	
$V = kr\,R^{2/3}S_o^{1/2}$; $V = k'r\,R2/3\ {S^1}^{1/2}$	Metric	
$$V = \sqrt{\frac{8}{f'b}}\sqrt{gRS^1}$$	Dimensionally homogeneous	
$k'_r = \frac{26}{(d_{90})^{1/6}}$; $d_m = \sum_i P_i\, d_{si}$	Metric	
2.4.6. Modification of Einstein (1942) by Rose, Boyer and Laursen (Rouse 1950) $$q_{sb} = \phi\gamma_s F_1\sqrt{g\left(\frac{\gamma_s}{\gamma_f} - 1\right)d_s^3}$$ $$\varphi = \left(\frac{1}{\Psi}\right)$$	FPS	$f\left(\frac{1}{\psi_*}\right)$, function is obtained from figure 2.13 through ϕ. F_1 is related to fall velocity , $$\omega_0 = F_1\sqrt{\left(\frac{\gamma_s}{\gamma_f} - 1\right) g\, d_{50}}$$

table 2.4 continued

Investigator and His Formula (1)	Units (2)	Remark (3)
$F_1 = \sqrt{\frac{2}{3}+\frac{36v^2}{gd_s^3\left(\frac{\gamma_s}{\gamma_f}-1\right)}} - \sqrt{\frac{36v^2}{gd_s^3\left(\frac{\gamma_s}{\gamma_f}-1\right)}}$		for $\frac{1}{\Psi}$ in excess of 0.09 and $\tau_o = \gamma_f R.So$, the formula is based on Gilbert data (1994) for grated sediment with median sizes from 0.3 mm to 7 mm.
2.4.7 Einstein Bed Load Function (Einstein, 1950) $q_{sb} = \sum_i q_{si}$ $G_{sb} = bq_{sb}$ $q_{si} = q_{sbi}[P_r \; I_1(\eta_{oi}, z_i) + I_2(\eta_{oi}, z_i) + 1\;]$ $I_1(\eta_{oi} z_i) = 0.216 \frac{\eta_{oi}^{zi-1}}{(1-\eta_{oi})^{zi}} \int_{\eta_{oi}}^{1} \left(\frac{1-\eta}{\eta}\right)^{zi} d_\eta$ $I_2(\eta_{oi} z_i) = 0.216 \frac{\eta_{oi}^{zi-1}}{(1-\eta_{oi})^{zi}} \int_{\eta_{oi}}^{1} \left(\frac{1-\eta}{\eta}\right)^{zi} I_{n\eta} d_\eta$ $P_r = 2.3 \log \frac{30.2\,v\,R}{d_{65}}$ $Z_i = \frac{\omega_i}{0.4V_*^1}$ $\quad V_*^1 = \sqrt{gRS_0}$ $\varphi_{*i} = \frac{q_{sbi}}{P_i \gamma_s} \sqrt{\left(\frac{\gamma}{\gamma_s - \gamma_f}\right) \frac{1}{gd_{si}^3}}$	FPS	q_{sbi} is the bed load of mean size d_{si} discharge of in weight per unit width. Value of φ_* is obtained on the fig 2.14. φ_* as a function of Ψ_*. ξ is a function of d_{si} / x given is fig. 2.15 and Y is a function of given in fig. 2.16. value of ηoi = 2dsi/R , η =y /R and functions, I_1 and I_2 are given in figure 2.17 and v is a dimensionless quantity in the logarithm velocity distribution low shown in figure 2.18.value of v from annexure-1 and ω_i is given in figure 2.19 The formula were obtained in flume experiments with two well - sorted sediments of mean sizes 28.65 mm and 0.785 mm, respectively (for graded fine sands).

table 2.4 continued

Investigator and His Formula (1)		Remark (3)
$\psi_{*i} = \xi_i Y \left(\frac{\log 10.6}{\log \frac{10.64X}{d_{65}}} \right)^2 \frac{(\gamma_s - \gamma_f) d_{si}}{\gamma_f R S_0}$ $X = 0.77 \frac{d_{65}}{x}$ when $\frac{d_{65}}{x\delta} > 1.80$ $X = 1.398\ \delta$ when $\frac{d_{65}}{x\delta} < 1.80$; $\delta = 11.6 \frac{v}{V'_*}$		
2.4.8 Laursen (1958) $q_s = C_m q,$ $G_m = 0.01 \gamma_f \sum_i P_i \left(\frac{d_{si}}{D} \right)^{7/6} \left(\frac{\tau_0}{\tau_{ci}} - 1 \right) f\left(\frac{V_*}{\omega_i} \right)$ $\tau'_0 = \frac{\rho V^2}{58} \left(\frac{d_{50}}{D} \right)^{1/3}$ $\tau_{ci} = \tau_{*c} (\gamma_s - \gamma_f) d_{si}$	FPS	τ_o is Laursen's bed shear stress due to grain resistance an $f\left(\frac{V_*}{\omega_i}\right)$ is the function shown in fig. 2.20. value of τ_{*c} in figure 2.20 density of fluid p from annexure 1 and τ_{*c} = 0.39 for sediment median size from 0.088 mm to 4.05 mm
2.4.9 Blench (1966) $\frac{\left(1 + 0.12 x 10^5 \frac{C_m}{\gamma_f}\right)^{11/12}}{1 + \frac{1}{233} x 10^5 \left(\frac{C_m}{\gamma_f}\right)} = \frac{3.63 g B^{1/4} q^{1/12} S_0}{k_m v^{1/4} \left[1.9 \sqrt{(d_{50})}\right]^{11/12}}$	FPS	k_m the a meander coefficient with values of 1.25 for straight reaches, 2.0 for streams with well-developed meanders and 2.75 for very sinuous streams. The formula and the constants in it were derived from regime relations and by correlating the relations mainly with data of Gilbert (Gilbert,1914 and Johson, 1943) obtained in small flumes with well-sorted sands ranging in size from 0.33 to 7 mm

table 2.4 continued

Investigator and His Formula (1)	Units (2)	Remark (3)
2.4.10 Colby Relations (Colby, 1964) $q_s = k_1 k_2 q_{s1}$	FPS	q_{s1} is the sediment discharge, obtained in with median size ranging from 0.2 mm to 0.3 mm in water at 60^0F. k_1 and k_2 is correction factor shown in Fig 2.21
2.4.11 Engelund Hansen (Engelund , 1966 and Engelund and Hansen , 1967) $q_{sb} = 0.05\gamma_s V^2 \sqrt{\frac{d_{50}}{g\left(\frac{\gamma_s}{\gamma_f} - 1\right)}} \left[\frac{\tau_0}{(\gamma_s - \gamma_f) d_{50}}\right]^{3/2}$	FPS	The formula is based on Guy data (1966) for graded sediment with median sizes 0.19 mm , 0.27mm , 0.45 mm and 0.93 mm.
2.4.12. Inglis -Lacey (Inglis, 1968) $q_{sb} = 0.562 \frac{(v_g)^{1/3}}{\omega_0} \frac{V^2}{gd} \frac{\gamma_f V^3}{g}$	FPS	ω_o is the fall velocity of characteristic sediment particle that is arranged to be the particle having the median size of the bed material in fig 2.19 and kinematics viscosity of fluid, ν from annexure-1 .
2.4.13 Taffaleti, (Taffaleti, 1968,`969) $q_{sbi} = M_i (2d_{si})^{1+n_v-0.758z_i}$ $M_i = 43.2 P_i C_{Li} (1+n_v) VR^{0.758z_i - n_v}$ $(C_i)_y = 2d_{si} = C_i \left(\frac{2d_{si}}{R}\right)^{-0.736z_i}$ $n_v = 0.1198 + 0.00048T$ $Z_i = \frac{\omega_i V}{C_z R S_0}$ $C_z = 260.67 - 0.667T$	FPS	In which $M_i = 43.2 P_i C_{Li} (1+nv) VR^{0.758zi-nv}$. zi , n_v is exponent and if the concentration is in excess of 100 pcf, the concentration , C_{Li} in all equation is reduced so that gives a value of concentration equal to 100 pcf, fall velocity, ωi from figure 2.19 and T is water temperature in degrees Fahrenheit. The formula is based on waterways Experiment Station of the Unified States Corps of Engineer, the bed sediment had median grain sizes ranging from 0.33mm to 0.93mm.

Table 2.5
Suspended Sediment Transport
Empirical Equations for Sediment Discharge

Investigator and His Formula (1)	Units (2)	Remark (3)
2.5.1 Einstein (1950) $q_{ss} = \frac{\gamma_s}{100} 11.6 V'_* C_{2d}\, 2d \left\{ 2.3 \log \left(\frac{30.2 D X}{d_{65}} \right) I_1 + I_2 \right\}$ $C_{2d} = 1.12 \times 10^{-7} \left(\frac{V_*}{\omega_0} \frac{V_* d}{\nu} \right)^{5.25}$ $A = \frac{a}{D}$ $Z = \frac{\omega_0}{V_* K}$	Metric	Two integral I_1 and I_2 numerically obtained by Einstein are shown graphically as at A and Z in figure 2.17. Fall velocity (ω_0) is obtained from figure 2.19, ν from annexure-1 and Einstein has suggested $a = 2d$, $k = 0.4$ (Karman constant)
2.5.2 Lane and Kalinske (Garde, 2000) $q_{ss} = q\, C_a\, P \exp \left\{ 15 \left(\frac{\omega_0}{V_*} \right) A \right\}$, $\frac{C}{C_a} = \exp \left\{ \frac{-\omega_0}{V_*} \left(\frac{y}{D} - \frac{a}{D} \right) 15 \right\}$	Metric	P is a function of ω_o/V_* and V_*/KV using Manning's equation for V, V_*/KV can be expressed in term of n/D1/6, see Fig 2.22 and fall velocity (ω_o) is obtained from figure 2.19
2.5.3 Engelund (Garde, 2000) $q_s = 5.10 \times 10^{-5} \left(\frac{V_*}{\omega_0} \right)^4 \gamma_f q$	Metric	Fall velocity (ω_o) is obtained from figure 2.19

Table 2.5 continued

Investigator and His Formula (1)	Units (2)	Remark (3)
2.5. 4 Samaga et.al. Method (Garde, 2000) $q_{ss} = \left[\frac{\varphi_s \gamma_s}{\sqrt{\frac{\gamma_f}{\Delta\gamma_s} \times \frac{1}{gd}}} \right] / d$; $\varphi_s = 28.0\, \tau_*^6$; $\tau_* = \frac{\tau_0}{(\Delta\gamma_s d)}$	Metric	The method at calculation by considering the individual fractions in a mixture and φ_s, τ_* are parameter. The variation of $K_s L_s \xi_s$ with $\tau_o/(\Delta\gamma s\, d_i)$ is shown in Figure 2.23. The empirical Coefficients K_s and L_s are functions of τ_o/τ_∞ and M respectively as shown in Tables 2.7 and 2.8 respectively. M is Khemer's uniformity coefficient.
2.5. 5. Holtorfft's Method (Garde, 2000) $q_{ss} = \left[\tau_0 V \times 0.055 \sum_{ib} \left(\frac{\tau_s}{\tau_0}\right)_i \left(\frac{V}{\omega_0}\right)_i \right] / \Delta_s$	Metric	$(\tau_s/\tau_o)_i$ and $(V/\omega_o)_i$ are the values corresponding to a particular size, the former being read from figure 2.24
2.5.6. Van Rijn's Method (Garde, 2000) $q_{ss} = \gamma_s FVDC_a$; $C_a = 0.15 \frac{d_{50} T^{1.5}}{a d_*^{0.3}}$; $T = \frac{V_*'^2 - V_{*c}^2}{V_{*c}^2}$; $d_* = d_{50} \left(\frac{\Delta\gamma_s}{\rho_f \nu^2} \right)^{1/3}$; $\frac{V}{V_*} = 5.75 \log \frac{R}{k_s} + 6.25$	Metric	C_a is the reference concentration at y = a, the reference level 'a' is taken to be equal to k_s or D/100. The correction factor F is related to a/D an Z' as shown in figure 2.25.

Table 2.6
Total Sediment Transport
Empirical Equations for Sediment discharge

Investigator and His Formula (1)	Units (2)	Remark (3)
2.6.1 Einstein Method (Garde, 2000) $G_s = Q_s + \rho_s$; $Q_s = Q_{sb} + Q_{ss}$ $Q_{sb} = q_{sb} \times b$; $Q_{ss} = q_{ss} \times b$ $q_T = q_{se} + q_{ss}$; $q_{sb} = \varphi\sqrt{(\gamma_s - 1) g d_{35}^2}$ $\psi_* = (\gamma_s - 1)\frac{d_{35}}{R S_o}$; $\varphi = f(\psi_*)$ $q_{ss} = q_{sb}(P_e I_1 + I_2)$; $P_e = 2.303\ \log(30.2)$ $\Delta = d_{65}/x$; $k_s/\delta = d_{65}/\delta$ $\delta = 11.5\nu/V_*$; $A_E = 2\ d_{35}/h$ $z = V_{ss}/KV_*$; $V_* = \sqrt{gRS_o}$	Metric	Intensity of transport, Φ see fig. 2.26, correction term for logarithmic velocity distribution,x and apparent roughness diameter see fig. 2.27 and two integral I_1 and I_2 numerical obtained by Einstein are show in fig.2.17 and settling velocity from fig.2.28
2.6.2 Swamee and Ojha's Method (Garde, 2000) $q_{sb} = \phi_{Bs}\ \lambda_s\left(\frac{\rho_s}{\rho_f} - 1\right)^{1/2} q^{1/2}\ d_s^{3/2}$ $q_{ss} = \varphi_{ss}\ \lambda_s\left(\frac{\rho_s}{\rho_f} - 1\right)^{1/2} q^{1/2}\ d_s^{3/2}$: $\tau_{*s}^{1} = \frac{\gamma_f R' S_o}{\Delta\gamma_{ss}}$ $\varphi_{Bs} = \left\{\left[\left(\frac{0.8}{M}\right)^{4.75} + M^{0.0004}\right]^{1.75}\left(\frac{0.0871}{\tau_{*s}^{1\ M^{0.25}}}\right) + \left[\left(\frac{0.01}{M}\right)^{0.06} + M^{3.25}\right]^{1/2}\left[\frac{0.339}{\tau_{*s}^{1\ 9M/(8M^2+1)}}\right]^{1.6}\right\}^{-1}$ $\varphi_{ss} = \left\{\left[\left(\frac{0.073}{M}\right)^{4} + M^{3.8}\right]\left(\frac{0.567}{\tau_{*s}^{1\ M^{0.25}}}\right)^{6} + \left[\left(\frac{0.177}{M}\right)^{2} + M^{1.455}\right]^{2}\left[\frac{0.538}{\tau_{*s}^{1\ 8M/(7M^{1.425}+1)}}\right]\right\}^{-1}$	Metric	M is kramer's uniformity coefficient, $\varphi_{Bs}, \varphi_{ss}$, and τ_{*s}^{1} are parameters and d_s see figure 2.29

table 2.6. continued

Investigator and His Formula (1)	Units (2)	Remark (3)
2.6.3 Laursen's Method (Garde, 2000) $\dfrac{\bar{c}}{\left(\frac{d}{D}\right)^{7/6}\left[\left(\tau_0^1/\tau_{oc}\right)-1\right]} = f\left(\dfrac{V_*}{\omega_o}\right)$	Metric	$\bar{C}$ is total load concentration in percent by weight sediment transport. Laursen considered the following parameter to be important in the study of total sediment transport: V_*/ω_o, d/D, the total load concentration $\bar{C}$ in per cent by weight and the ratio of grain shear stress τ_0^1 to the critical shear stress τ_{0c} for the given sediment size see figure 2.30
2.6.4. Garde and Dattari (Garde, 2000) $q_T = 10.39\tau_*^{2.02}(\gamma_s V_* d)$	Metric	The relationship is evidently a simplification of a more complex relationship, even for uniform sediment.
2.6.5. Bagnold' Equation (Garde, 2000) $q_T = \dfrac{\tau_0 V}{\left(1-\frac{\rho_f}{\rho_s}\right)}\left\{\dfrac{eB}{\tan\alpha} + 0.01\dfrac{V}{\omega_0}\right\},$	Metric	es(1-eB) can be taken as 0.01 for all practical purpose and es is a constant value of 0.015. The value of tan α varies from 0.315 to 0.75 and is a function of τ_* and d as show in table 2.9 and fall velocity obtained from fig. 2.19
2.6.6. Bislcop, Simons and Richardson's Method $q_T = \varphi_T \rho_s g^{3/2} d^{3/2}\left(\dfrac{\rho_s}{\rho_f}-1\right)^{1/2}$ $\psi^1 = \dfrac{\rho_s-\rho_f}{\rho_f} \times \dfrac{d_{35}}{R'S_o}$ $\dfrac{V}{V_*^1} = 5.75\log\left(\dfrac{12.27R'x}{d_{65}}\right)$	Metric	The parameter ϕ and Ψ^1 as show in figure 2.31 and d is median size of particle

table 2.6 continued

Investigator and His Formula (1)	Units (2)	Remark (3)
2.6.7. Engelud and Hansen's Method (Garde, 2000) $q_T = [\ k(\tau_*) - 0.06\sqrt{\tau_*}\sqrt{(\gamma_s - \gamma_f)\, d^3 \rho_f}\ ] / \frac{f}{\gamma_s}\left(\frac{h}{Lf}\right)$, $\tau_*^1 - 0.06 = 0.4\tau_*^2$	Metric	$\frac{h}{L f}$ is constant for particular value of τ_*^1 obtained from figure 2.32 and f is the friction factor of the bed.
2.6.8 Ranga Raju $\tau_{*_t} = \tau_*^1(\tau_0^1/\tau_0)^{-m}$ $\varphi_T = 60\tau_*^{1\,3}(\tau_0^1/\tau_0)^{-m}$	Metric	Total transport was analyzed using a fresh approach by vital that a unique relation between the parameter φ_T and τ_* should exist for a plane bed was justified by potting all available plan bed data figure 2.33 . Subsequently, the effective shear stress for total load transport in case of an undulated bed τ_t was defined as the shear stress which would be required on a plane bed to cause a transport rate equal to that observed in case of the undulated bed. Obviously the effective shear stress so defined could be determined from fig.2.33 for the known q_T value. The variation of ϕ_T with $\tau_*^1(\tau_0^1/\tau_0)^{-m}$ for a large amount of flume and field data based on m value is shown in figure 2.34, m is a function of V_*/ω_o , $m = 0$ for $V_*/\omega_o \leq 0.5$, $m = 0.2\,V_*/\omega_o - 0.1$ for $V_*/\omega_o > 0.5$ and ϕ_T value in the large of $0.05 \leq \tau_*^1(\tau_0^1/\tau_0)^{-m} < 1.0$.

table 2.6 continued

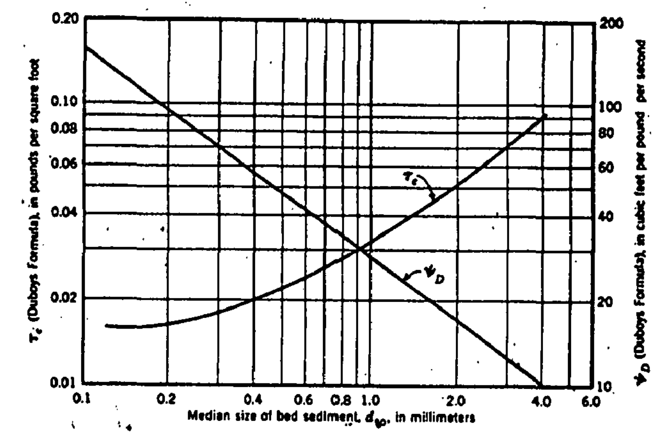
Investigator and His Formula (1)	Units (2)	Remark (3)
2.6.9. Ackers and white (1973) $Qs = Cs\,Q$ ... (i) $C_s = G_{gr} \frac{d_{35}}{h}\left(\frac{V}{V_*}\right)^n$ ... (ii) $G_{gr} = C\left[\frac{F_{gr}}{A} - 1\right]^m$ ... (iii) $V_* = \sqrt{g\,h\,S_0}$ ... (iv) $F_{gr} = \frac{V_*^n}{\sqrt{g\,d_{35}(\gamma_s - 1)}}\left[\frac{V}{\sqrt{32}\,\log\left(\frac{10h}{d_{35}}\right)}\right]^{1-n}$ ...(v) $N = 1.00 - 0.56 \log D_{gr}$ ... (vi) $A = \frac{0.23}{\sqrt{D_{gr}}} + 0.14$ ... (vii) $m = \frac{9.66}{D_{gr}} + 1.39$ ... (viii) $\log C = 2.86\ D_{gr}$ -$(\log D_{gr})2$ -353 ... (ix) $D_{gr} = d_{35}\left[\frac{g(\gamma_s - 1)}{v^2}\right]^{1/3}$ ... (x) $G_s = Qs \times \rho_s$ ... (xi)	Metric	For a direct determination of the total load transport, q_s, purposed the use of same sediment logical parameter; D_{gr}, F_{gr}, C_s. Employed were hydraulic considerations and dimensional analysis. The confinement; N ,A, m and C in the above relations were determined by regression analysis, using close to 1000 experiments in the laboratory and close to 250 experiment the feild Ackers and White (1973, 1980), presented a new total load sediment transport theory incorporating the advantages of dimensional analysis and physical arguments. The data against which these theories were checked consisted at 840 flume experiments with sand, 180 flume experiment with height rivers. The method is more reliable as compared to others method generally considered . Kinematics viscosity of fluid, v show in annexure-1.

table 2.6 continued

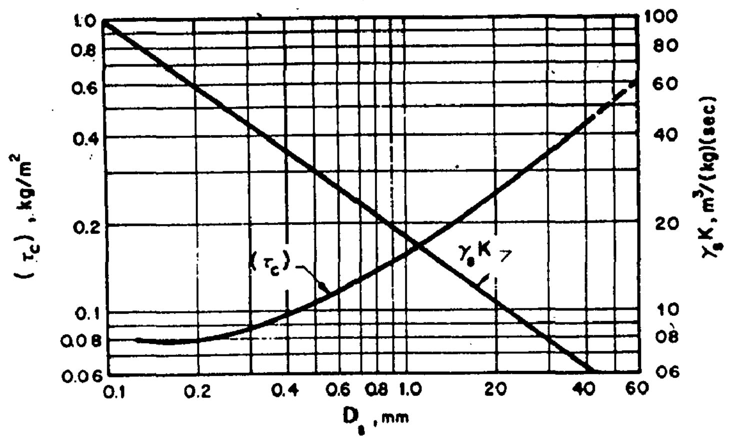
Investigator and His Formula (1)	Units (2)	Remark (3)
2.6.10. Yong's Equation (Garde, 2000) $Q_s = \overline{C}_T Q$, $\log \overline{C}_T = 5.435 - 0.286 \log \frac{\omega_0 d}{v} - 0.457 \log \frac{V_*}{\omega_o} +$ $(1.799 - 0.409 \log \frac{\omega_0 d}{v} - 0.314 \log \frac{V_*}{\omega_0}) \times \log \left(\frac{VS_o}{\omega_0} - \frac{V_{cr} S_o}{\omega_0} \right)$	Metric	The rate of sediment transport in an alluvial channel is primarily by the rate of expenditure of potential energy per unit weight of water i.e the unit stream power. Fall velocity, ω_o ,show in fig. 2.19
2.6.11. Shen and Hung's Equation (Garde, 2000) $Q_s = \overline{C}_T Q$ $\log \overline{C}_T = -10704.5 + 324217 Y_* - 326309.6 Y_*^2 + 109503.9 Y_*^3$ $Y_* = \left[\frac{VS_o^{0.57}}{\omega_0^{0.32}} \right]^{0.0075}$	Metric	By performing regression analysis at laboratory data, obtained dimensional expression for the total load concentration, $\bar{c}_T$, in ppm by weight. Fall velocity, ω_o ,show in fig. 2.19
2.6.12. Brownlie's Equation (Garde, 2000) $Q_s = \overline{C}_T Q$ $\overline{C}_T = 7115 \quad C_F \left[\frac{V}{\sqrt{\frac{\Delta\gamma_s}{\rho_f} d}} - \frac{V_{cr}}{\sqrt{\frac{\Delta\gamma_s}{\rho_f} d}} \right]^{1.978} S_o^{0.06601} \left(\frac{R}{d} \right)^{-0.3301}$	Metric	Based on a study merits and demerits of the parameters used by earlier investigator and by performing regression analysis of laboratory and field data, obtained equation for the concentration $\bar{c}_T$ in ppm by weight. The coefficients C_F is unity for laboratory data and 1.268 for field data.

table 2.6 continued

Investigator and His Formula (1)	Units (2)	Remark (3)
2.6.13 **Karim and Kennedy's Equation (Garde, 2000)** $\log \frac{q_T}{\gamma_s \sqrt{\left(\frac{\rho_s}{\rho_f}-1\right) g d^3}} = -2.2786 + 2.719 V_1 + 0.2989 V_3 V_2 + 1.06 V_1 V_3$ $V_1 = \log \frac{V}{\sqrt{\left(\frac{\Delta\gamma_s}{\rho_f}\right) d}}$ V2 = log (D/d) $V_3 = \log \left[\frac{V_* - V_{*c}}{\sqrt{\left(\frac{\Delta\gamma_s}{\rho_f}\right) d}} \right]$	Metric	Carried out a regression analysis of the sediment data from laboratory flumes and natural streams and gave a highly in value of equation for the sediment concentration



a – FPS



b – Metric

Fig. 2.10 Sediment Coefficient (Ψ_D) and Critical Shear Stress (τ_c) For DuBoys of Median Size of Bed Sediment (Straub,1935)

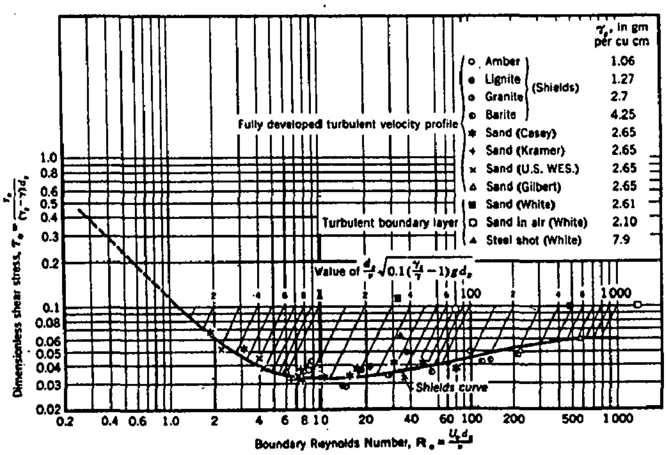


Fig. 2.11 Shield Diagram with White Data Added (Rouse, 1939)

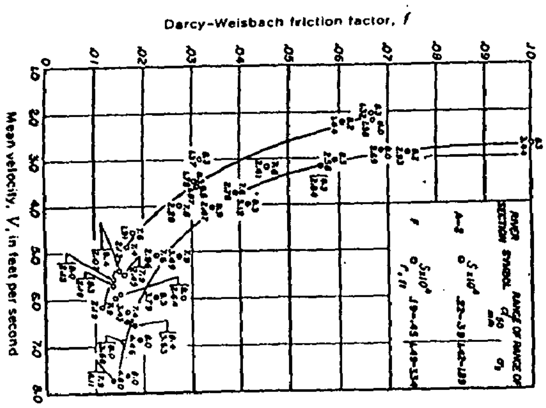


Fig.2.12 Variation of Darcy-Weisbach Friction Factor with Velocity for Two Sections of Rio Grande in New Mexico [(data obtained by Nordin (1964), Figure Presented by Alam and Kennedy (1969)]

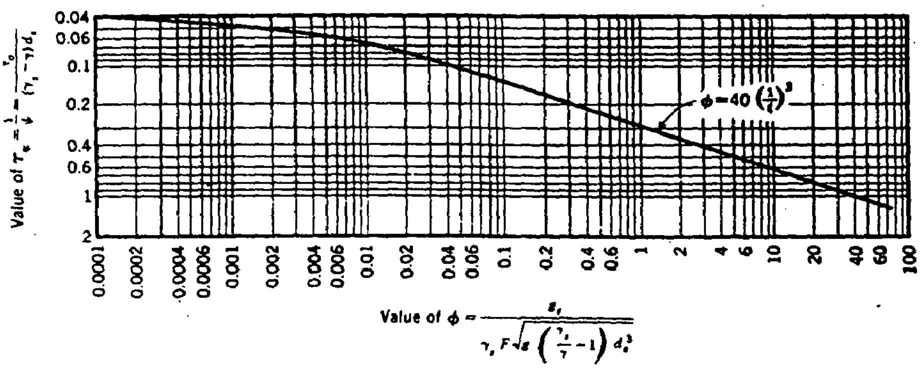


Fig. 2.13 Function $\phi = f\left(i/\Psi\right)$ for Einstein Brown (Einstein, 1942)

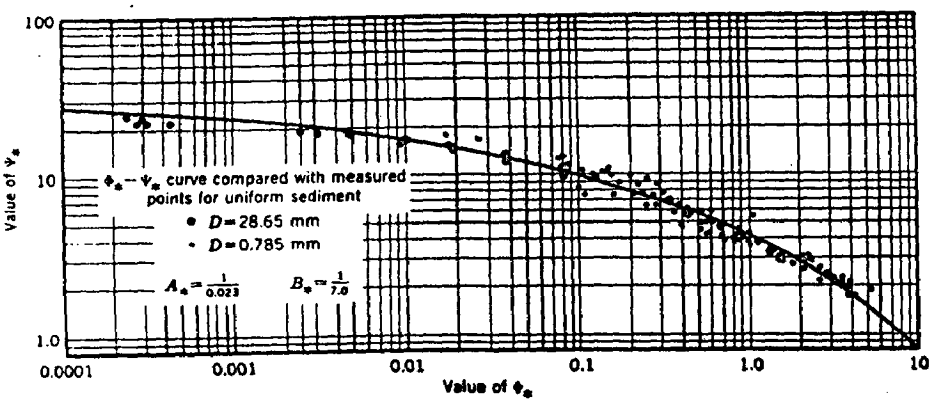


Fig.2.14 Einstein's Φ_* Bed Load Functio (Einstein, 1950)

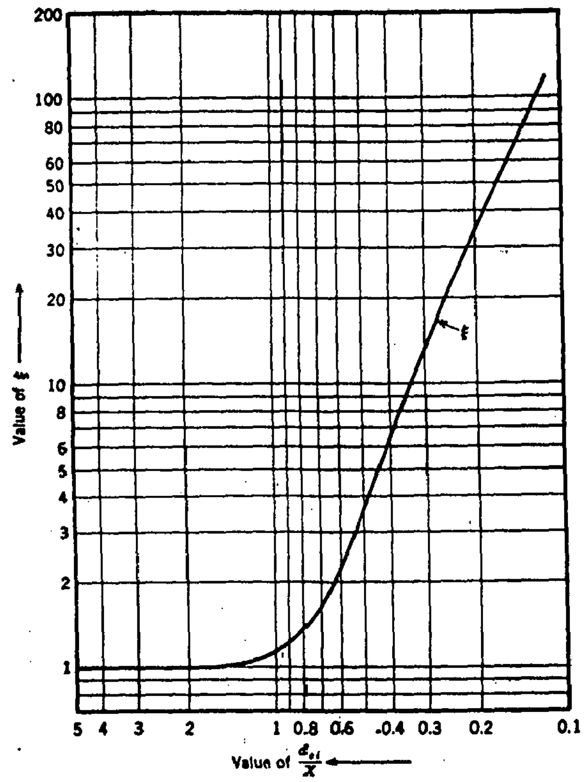


Fig.2.15 Factor ξ in Einstein's Bed Load Function (Einstein,1950) in Terms of d_{si} / X

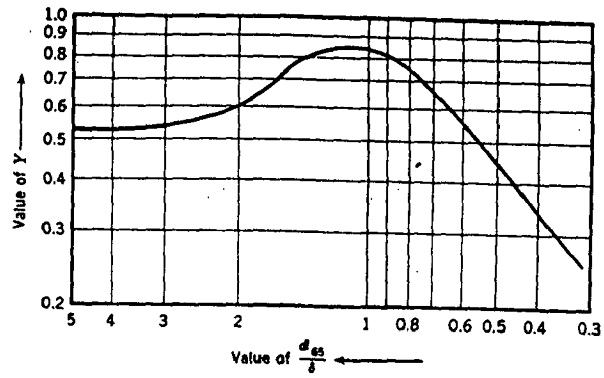


Fig.2.16 Factor Y in Einstein's Bed Load Function (Einstein,1950) in Terms of d_{65i} / δ

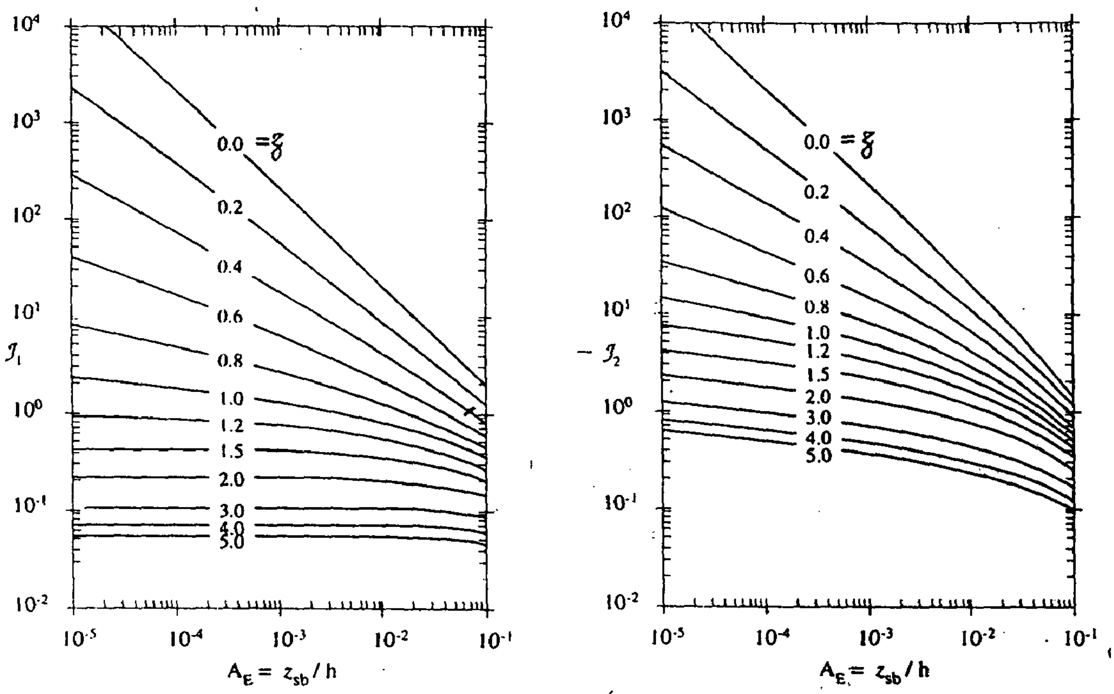


Fig.2.17 The Integral, I_1 (A_E , z) and I_2 (A_E , z), used in the method Of Einstein (1950)

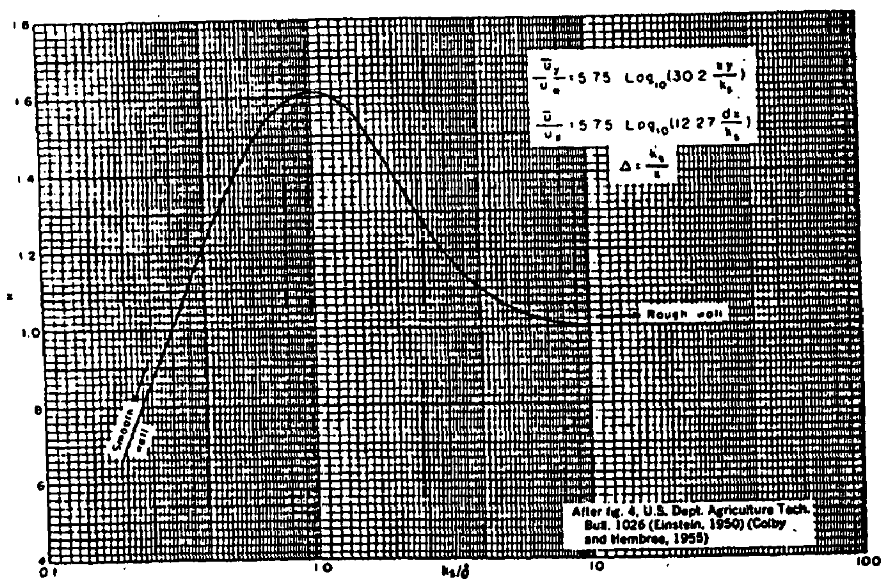


Fig.2.18 Factor x in Velocity Distribution Equation (Einstein, 1950)

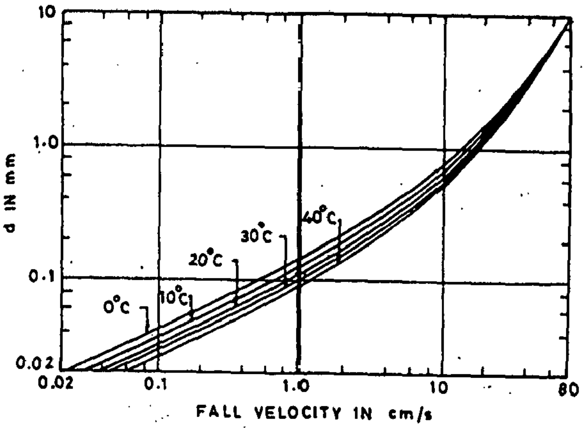


Fig.2.19 Fall Velocity of Spherical Particles (Relative Density = 2.65) in Water (Albertson)

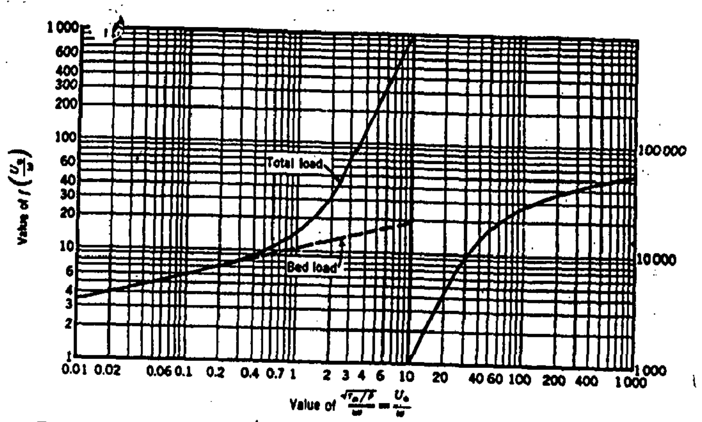


Fig.2.20 Function f (V*/ ω) for Laursen Formula (Laursen, 1958)

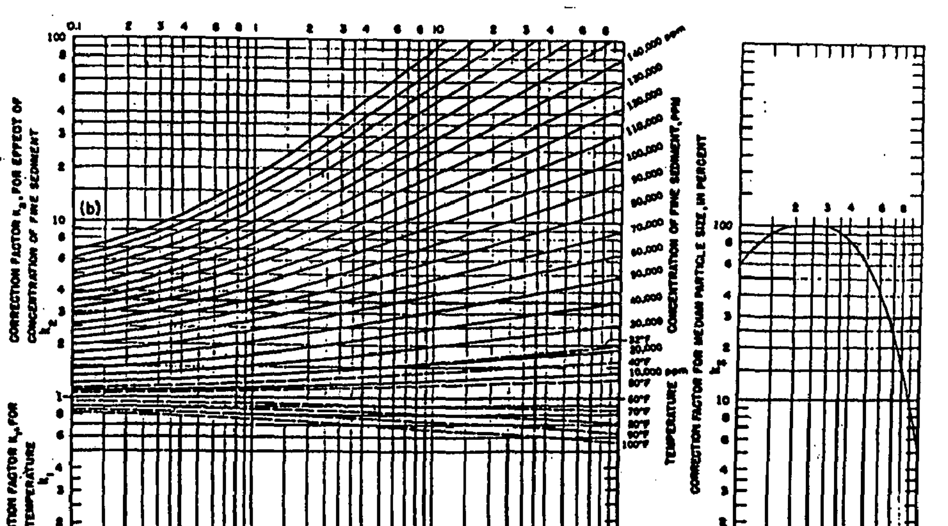


Fig.2.21 Colby's (1964b) Correction Factors for Effect of Water Temperature, Concentration of fine Sediment, and Sediment Size to be Applied to Uncorrected Discharge of Sand (Gilbert, 1914)

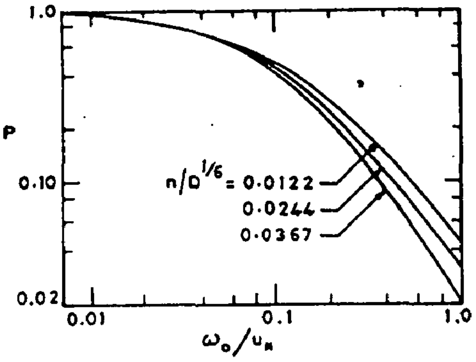


Fig.2.22 Variation of P with ω_0 / V_* and $n/D^{1/6}$ (Lane and Kalinske)

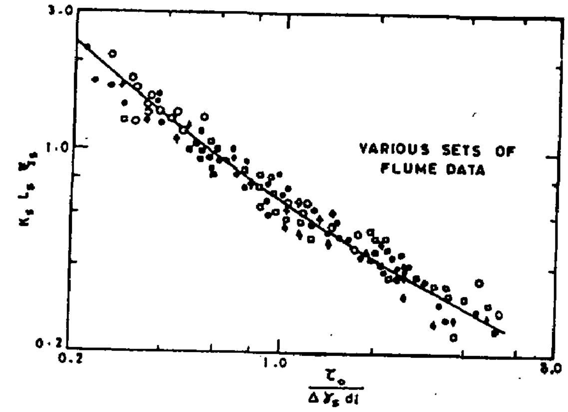


Fig.2.23 Variation of $K_S L_S \xi_S$ with $2D/k_s$ and K (Karman)

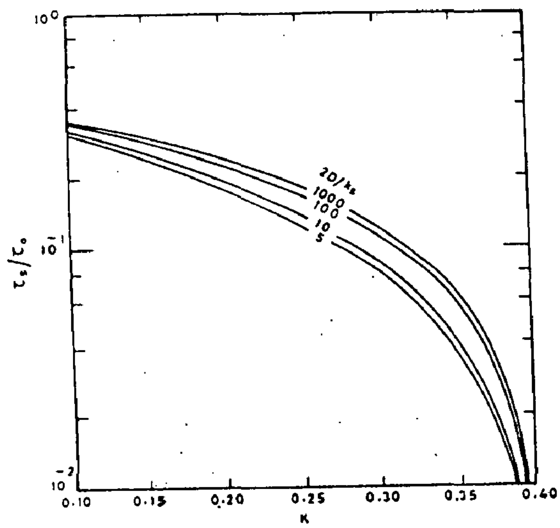


Fig.2.24 Variation of τ_S / τ_O with $\tau_O / \Delta\gamma_S d_i$ (Samaga)

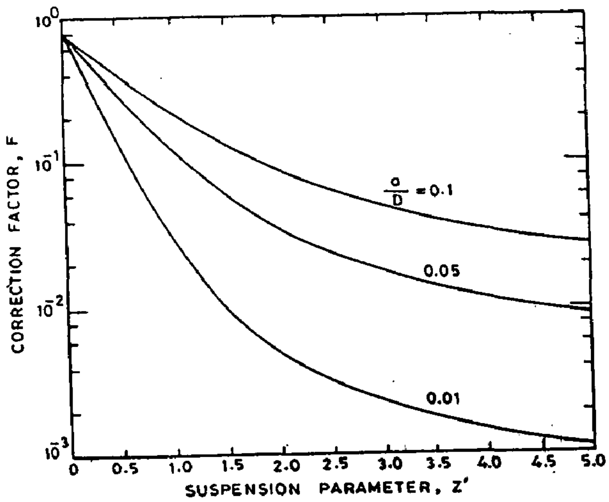


Fig.2.25 Variation of F with Z' and a/D (Van Rijn)

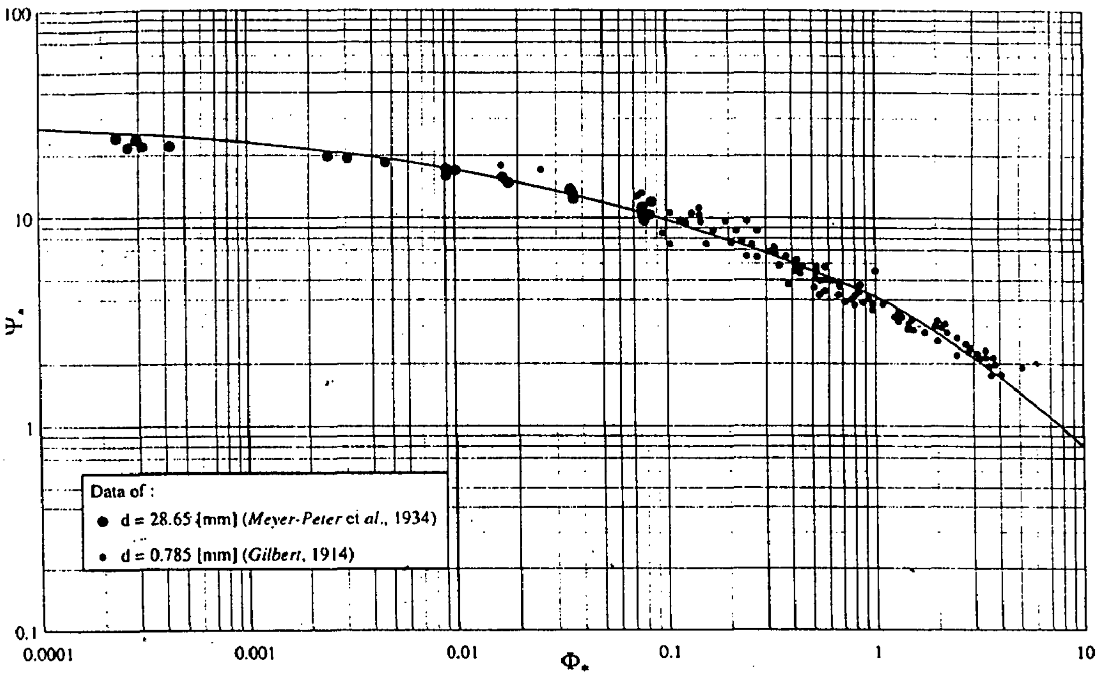


Fig.2.26 Equation of Bed Load, $\Phi_* = f(\Psi_*)$, of Einstein (Mayer Peter and Gilbert)

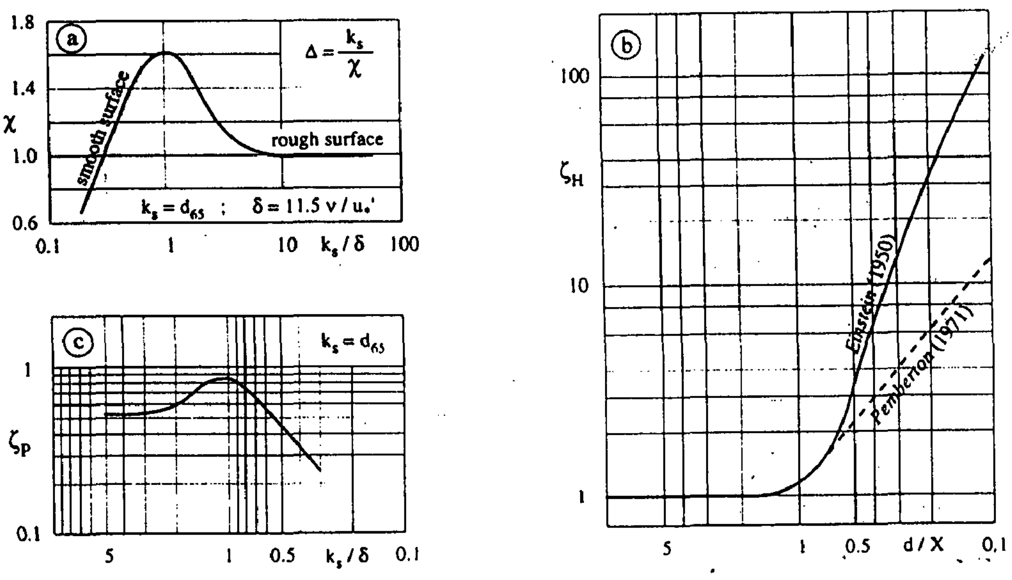


Fig.2.27 Correction Coefficient : (a) of Velocity distribution, (b) of Hiding and (c) of Lift Force (Einstein,1950)

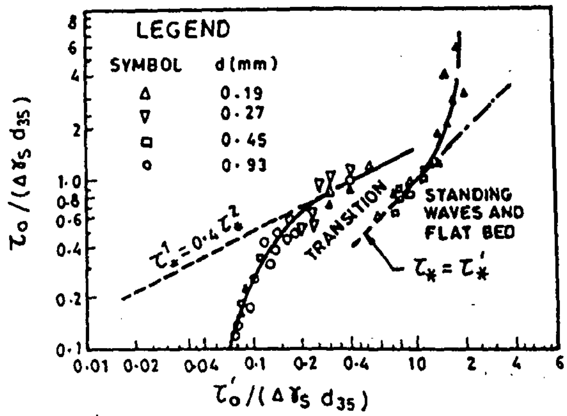


Fig.2.32 Engelund's Resistance Relation Based on Flume Data (Engelund's)

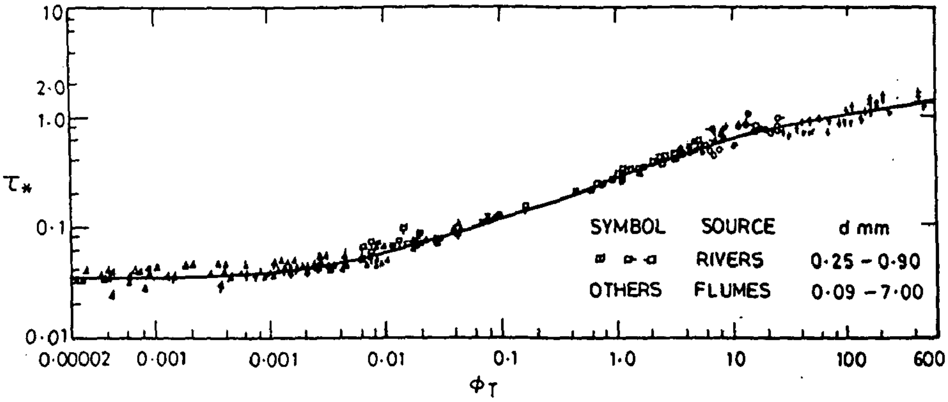


Fig.2.33 Variation of ϕ_T with τ_* for Plane Bed Data (Vittal)

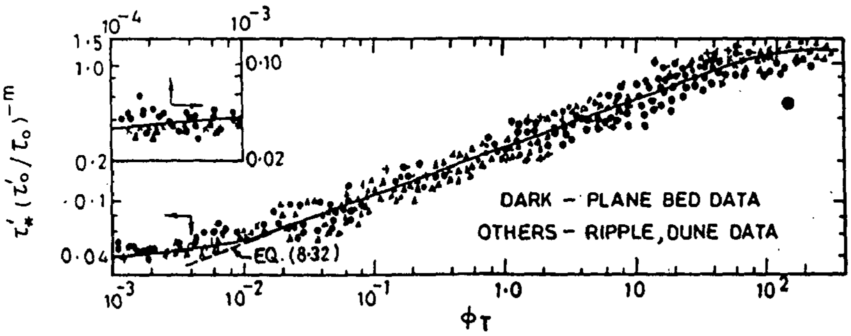


Fig.2.34 Sediment Transport Law for Plane, Ripple, and Dune Beds (Vittal and Ranga Raju)

Table 2.7.
Variation of L_s with M

M	< 0.20	0.25	0.30	0.40	> 0.50
L_s	0.80	0.86	0.90	0.97	1.00

Suorce : Garde and Other (2000)

Table 2.8.
Variation of K_s with τ_o/τ_{oc}

τ_o/τ_{oc}	< 2.0	3.0	4.0	5.0	7.0	9.0	10.0	11.0	14.0	>17.0
K_s	2.2	2.1	1.9	1.8	1.65	1.5	1.35	1.25	1.1	1.0

Suorce : Garde and Other (2000)

Table 2.9.
Variation of tan Kin Bagnold's Equation

τ.	D						
	0.30 mm	0.40 mm	0.50 mm	0.70 mm	1.00 mm	1.50 mm	2.00 mm and larger
0.30	Region of Non applicability					0.42	0.375
0.40	Of Bagnold's Equation				0.52	0.40	0.375
0.60		0.75	0.71	0.55	0.47	0.38	0.375
1.00	0.75	0.73	0.67	0.48	0.42	0.375	0.375
2.00	0.73	0.68	0.58	0.45	0.38	0.375	0.375

Suorce : Garde and Other (2000)

2.8 APPLICATIONS OF RELATIONS

Different formulae for the determination of the solid transport are given in Table 2.10, Graf (1998) has commented as under none of these relations can pretend to translate the intrinsic complexity of the transport of sediments.

Most of these formulae should not be used beyond the conditions within which they were established. Table 2.10 contains a summary of the range of the parameters, d (size) and S_f (energy slope) investigated for the establishment of each formula by their author (s) ; other author(s) may have extended this range. Also the recommendation by the author(s) for the choice of the equivalent diameter, d_x, if the granulometry is quasi or non-uniform, is given in the Table 2.10.

The formulae for the transport of sediments are often established, using laboratory data and less often using field data.

A verification of these formulae in natural channels is a very delicate task, since it is difficult to measure correctly the solid discharge in the field. Furthermore, it is often a rather subjective evaluation, since the zones of he modes of transport cannot easily be separated.

Numerous studies have been reported, comparing measurements in rivers with the different existing formulae.

For a better appreciation of the validity of the above presented formulae, it will now be of interest to compare the computed results with the direct measurements of the solid discharge in the field.

Table 2.10
Parameters Used for Establishing the Different Formulae.

Formula	D(mm)	S_f(-)	D_x(mm), equivalent diameter for a non-uniform granulate
Schoklitsch	0.3 –7.0 (44.0)	1/333 – 1/10	D_{40}
Meyer-Peter	3.1 – 28.6	1/2500 – 1/50	D_m (d_{50})
Einstein (1050)	0.8 – 28.6	-	D_{35}
Graf et Acaroglu (1968)	0.3 – 1.7 (23.5)	-	D_{50}
Ackers et White (1973)	0.04 – 4.0	Fr < 0.8	D_{35}

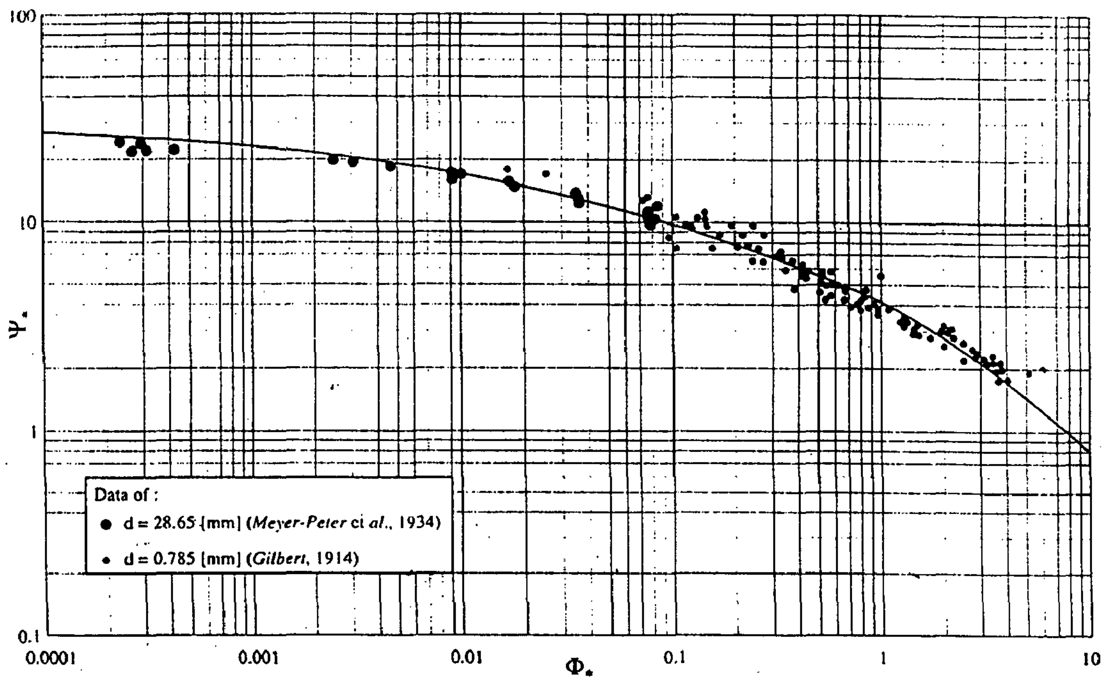


Fig.2.26 Equation of Bed Load, $\Phi_* = f(\Psi_*)$, of Einstein
(Mayer Peter and Gilbert)

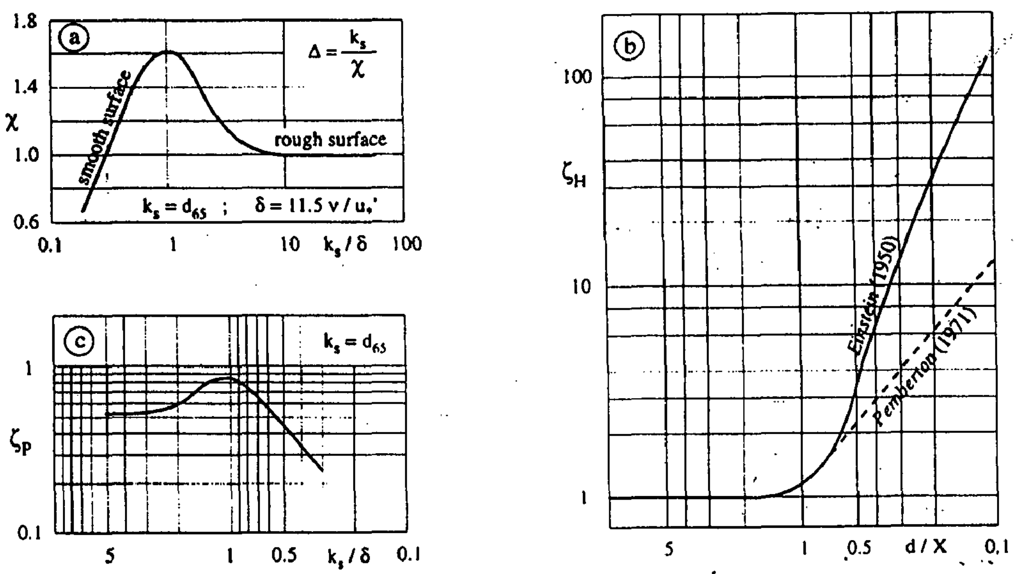


Fig.2.27 Correction Coefficient : (a) of Velocity distribution, (b) of Hiding and (c) of Lift Force (Einstein,1950)

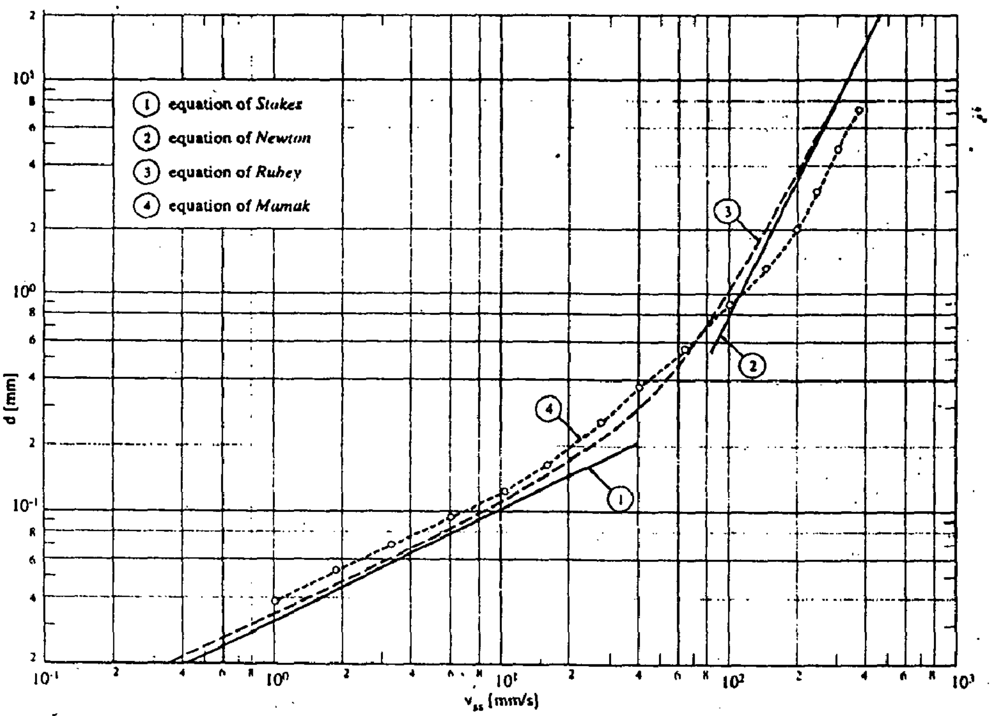
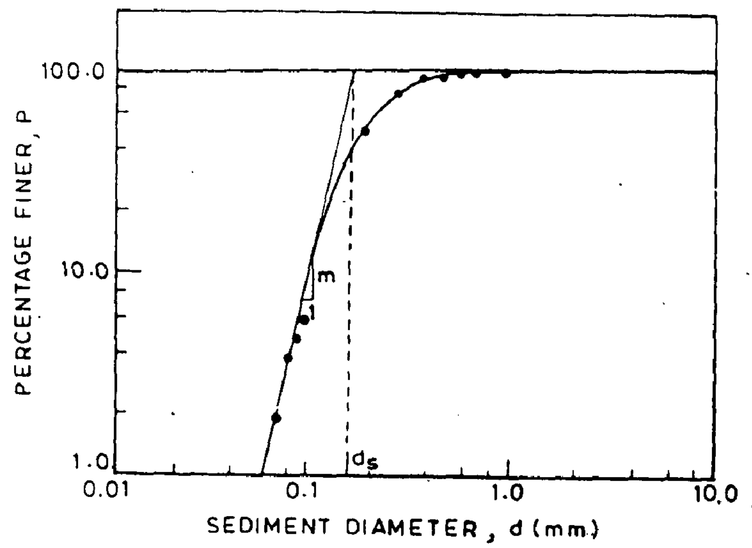


Fig.2.28 Settling Velocity, v_{ss} , as Function of Particle Diameter, d (Rouse, 1938)



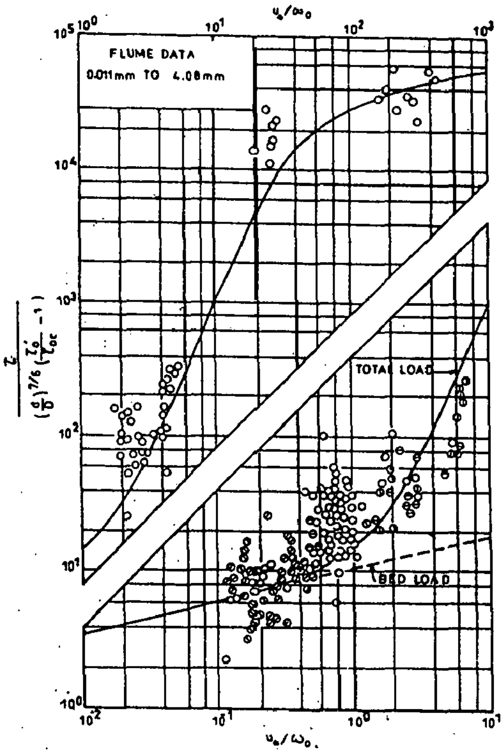


Fig.2.30 Laursen's Total Load Relation (Laursen)

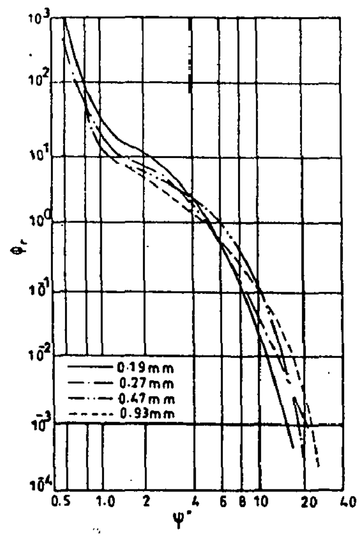


Fig.2.31 ϕ_T VS Ψ' Curve of Bishhop (Bishop, Simons and Richardson)

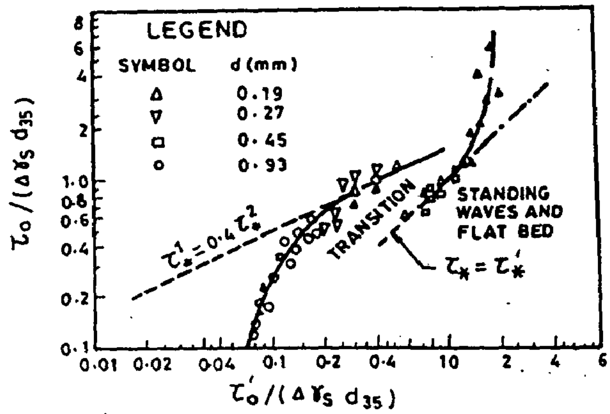


Fig.2.32 Engelund's Resistance Relation Based on Flume Data
(Engelund's)

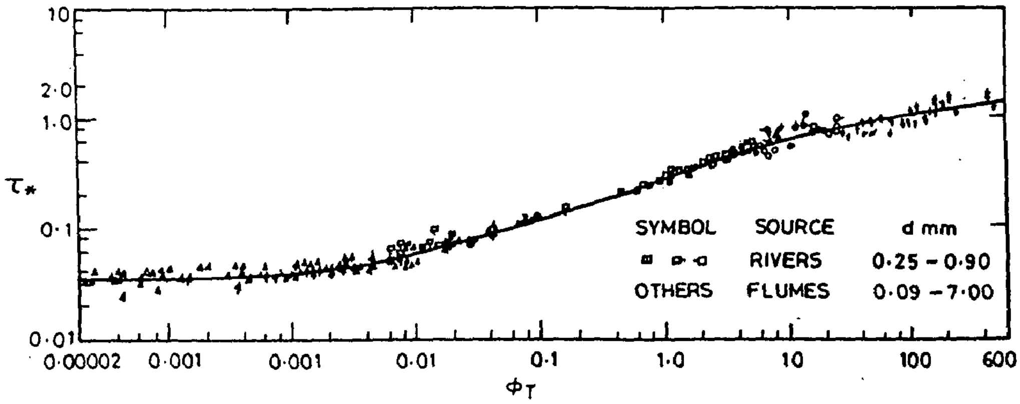


Fig.2.33 Variation of ϕ_T with τ_* for Plane Bed Data
(Vittal)

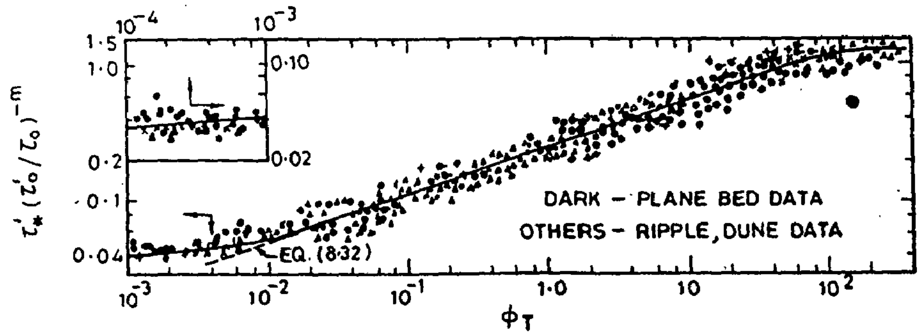


Fig.2.34 Sediment Transport Law for Plane, Ripple, and Dune Beds
(Vittal and Ranga Raju)

Table 2.7.
Variation of L_s with M

M	< 0.20	0.25	0.30	0.40	> 0.50
L_s	0.80	0.86	0.90	0.97	1.00

Suorce : Garde and Other (2000)

Table 2.8.
Variation of K_s with τ_o/τ_{oc}

τ_o/τ_{oc}	< 2.0	3.0	4.0	5.0	7.0	9.0	10.0	11.0	14.0	>17.0
K_s	2.2	2.1	1.9	1.8	1.65	1.5	1.35	1.25	1.1	1.0

Suorce : Garde and Other (2000)

Table 2.9.
Variation of tan Kin Bagnold's Equation

τ.	D						
	0.30 mm	0.40 mm	0.50 mm	0.70 mm	1.00 mm	1.50 mm	2.00 mm and larger
0.30	Region of Non applicability					0.42	0.375
0.40	Of Bagnold's Equation				0.52	0.40	0.375
0.60		0.75	0.71	0.55	0.47	0.38	0.375
1.00	0.75	0.73	0.67	0.48	0.42	0.375	0.375
2.00	0.73	0.68	0.58	0.45	0.38	0.375	0.375

Suorce : Garde and Other (2000)

2.8 APPLICATIONS OF RELATIONS

Different formulae for the determination of the solid transport are given in Table 2.10, Graf (1998) has commented as under none of these relations can pretend to translate the intrinsic complexity of the transport of sediments.

Most of these formulae should not be used beyond the conditions within which they were established. Table 2.10 contains a summary of the range of the parameters, d (size) and S_f (energy slope) investigated for the establishment of each formula by their author (s) ; other author(s) may have extended this range. Also the recommendation by the author(s) for the choice of the equivalent diameter, d_x, if the granulometry is quasi or non-uniform, is given in the Table 2.10.

The formulae for the transport of sediments are often established, using laboratory data and less often using field data.

A verification of these formulae in natural channels is a very delicate task, since it is difficult to measure correctly the solid discharge in the field. Furthermore, it is often a rather subjective evaluation, since the zones of he modes of transport cannot easily be separated.

Numerous studies have been reported, comparing measurements in rivers with the different existing formulae.

For a better appreciation of the validity of the above presented formulae, it will now be of interest to compare the computed results with the direct measurements of the solid discharge in the field.

Table 2.10
Parameters Used for Establishing the Different Formulae.

Formula	D(mm)	S_f(-)	D_x(mm), equivalent diameter for a non-uniform granulate
Schoklitsch	0.3 –7.0 (44.0)	1/333 – 1/10	D_{40}
Meyer-Peter	3.1 – 28.6	1/2500 – 1/50	D_m (d_{50})
Einstein (1050)	0.8 – 28.6	-	D_{35}
Graf et Acaroglu (1968)	0.3 – 1.7 (23.5)	-	D_{50}
Ackers et White (1973)	0.04 – 4.0	Fr < 0.8	D_{35}

Many (nineteen) of the existing formulae for the calculation of the total transport have been studied by White et al. (1973) and compared with experimental results. They evaluated almost 1000 laboratory experiments with uniform and non-uniform sediments of $0.04 < d_{50}$ (mm) < 4.9, at flow depth of $h < 0.4$ (m), and almost 270 experiments in watercourses with sediments of $0.1 < d$ (mm) < 68.0 and a width/depth ratio of $9 < B/h < 160$.

Each formula was applied to all the data of the solid-discharge measurements. Subsequently was established a ratio of the values calculated, C_{calc}, and the values observed, C_{obs}, where $C = C_s$ is the total-load transport, expressed in concentration. Some results of this investigation are given in Fig. 2.35, where one may see the success a prediction (in percentage) for different ranges of the ratio, C_{calc}/C_{obs}.considering only the range of $½ < C_{calc}/C_{obs} < 2$, it can be seen that the percentage for the formula of

Einstein (1950), eq. 6.60 : 44% of success

Graf et Acaroglu (1968), eq. 6.63 : 40% of success

Ackers et White (1973), eq. 6.66 : 64% of success

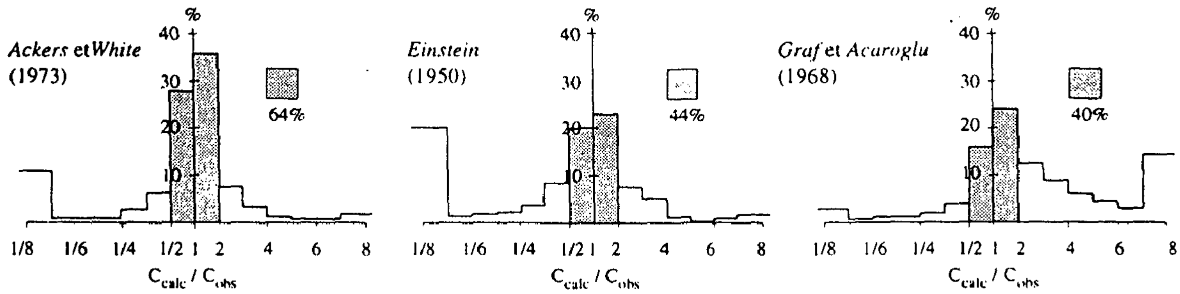


Fig.2.35 Comparison, with Respect to C_{calc}/C_{obs} , of the Success of Prediction for the Presented Formulae (Graf, 1984)

This implies that with the formula of Ackers et White, 64% of the experimental data can be predicted in the above-mentioned range. This is usually considered as a good (or a not-so-bad) result; more than half of the studied (nineteen) formulae give results which are less good, namely < 40%. Also noticed is that with the formula of Einstein there s a slight under-estimation of the solid discharge; while the one of Graf et Acaroglu gives a slight over-estimation. The comparative study of White et al. (1973) is reasonably objective, but certainly not conclusive. Other studies exist (see Raudkivi, (1976), p. 227) which show clearly that an objective validation is nearly impossible.

Amongst the different existing formulae for the determination of the total-load transport, but equally for the ones of the bed-load and suspended-load transport, each one will give an answer, but none will be very precise nor very true.

Finally, it must be said, that the results obtained with these formulae give only valuable guide-lines for the engineer. For practical purposes, it is advised to consult more than one formula ; the obtained result may however render different values (see Graf, 1971, p. 156).

2.9 REGIME THEORIES

The regime formulaes of Kennedy, Lacey and others apply to channels with erodible boundaries in alluvial soils carrying small sediment loads. Analyzed data from irrigation channels mainly in India and Pakistan, but also some in Egypt, Europe and America. They were obtained by the correlation of dimensions and slopes of apparently stable channels with discharges and sizes of bed material. And express three primary relationships namely velocity to dept, velocity to slope, and width to discharge by the velocity is non-silting and non-scouring are show in Figure 2.36. to Figure 2.38

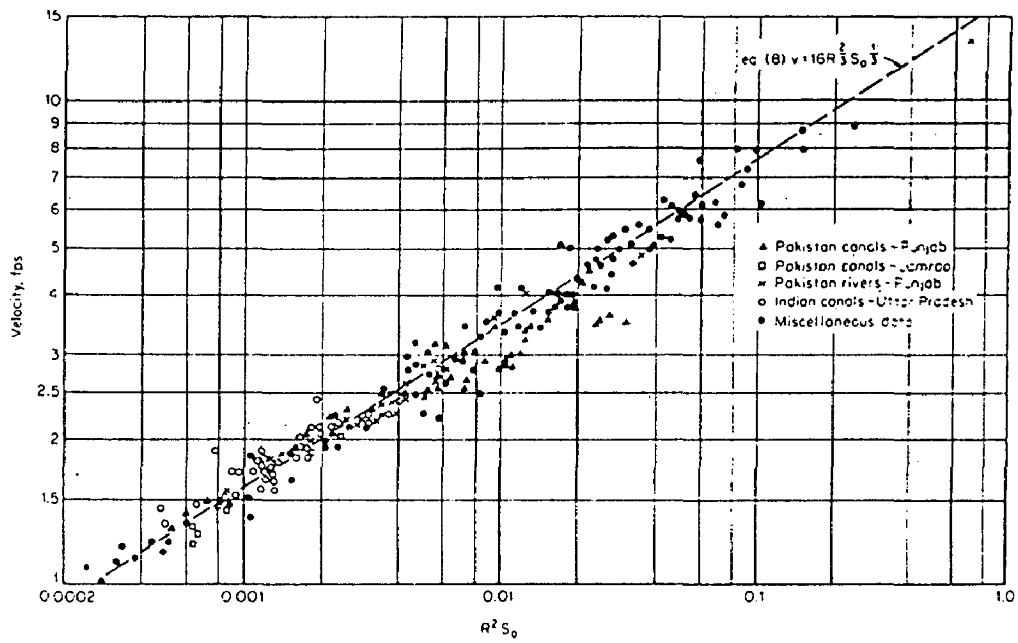


Fig.2.36 Regime Channel - Velocity vs Hydraulic Radius and Slope (Lacey, Proceedings ASCE Vol.86,paper 2484, 1960)

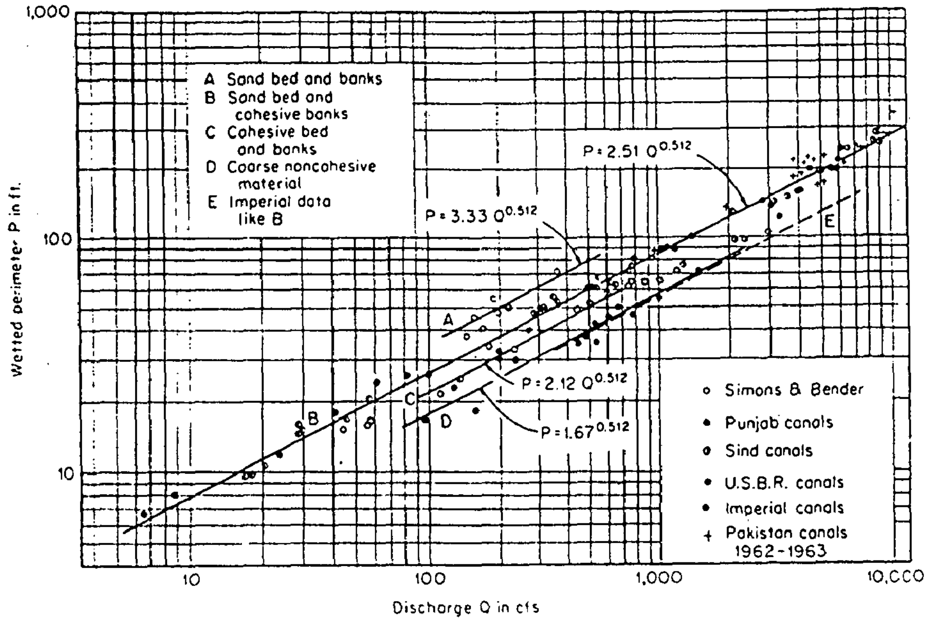


Fig.2.37 Variation of Wetted Perimeter with Discharge and Type of Channel (Lacey, Proceedings ASCE Vol.86,paper 2484, 1960)

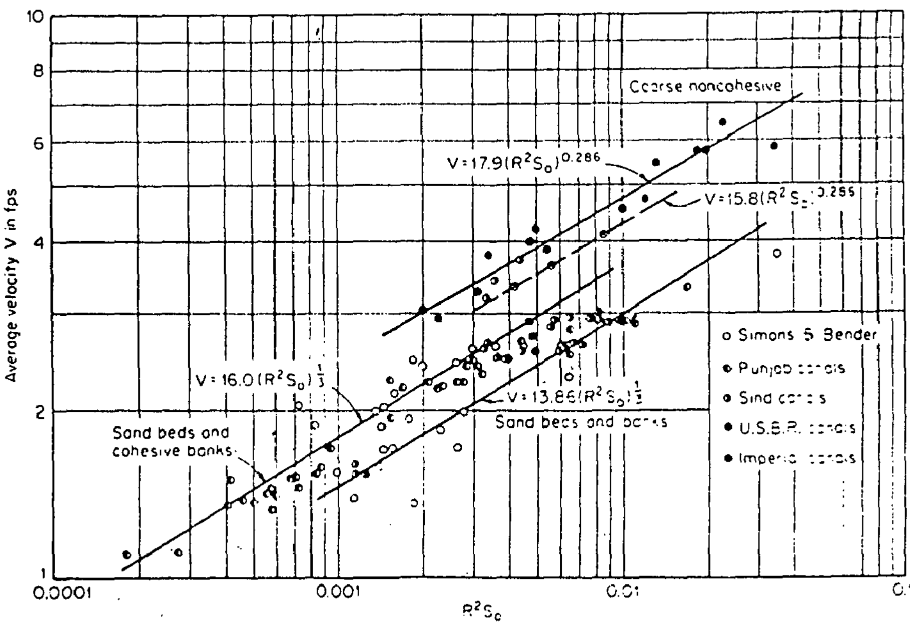


Fig.2.38 Variation of Average Velocity with Hydraulic Radius, Slope and Type of Channel (Lacey, Proceedings ASCE Vol.86,paper 2484, 1960)

2.10 EMPIRICAL EQUATIONS FOR REGIME CHANNELS

In 1875, Kennedy produced his depth velocity relations based on observations on Upper Bari Doab Canal in 1904 he gave a rough rule for the relation of width to depth in non-silting canals. A second edition of Hydraulic Diagrams' was issued in 1907 in which he represented the original paper and added an extended discussion to clarify some of the obcure points and to give the result of his experience since the first paper was printed.

Kennedy's work soon became extensively used through out India. Observations were made on the canals of other irrigation system and a number of other equations of the same type as those of Kennedy's were developed suitable to the various local condition. One of these was for the Godavavi and Kistna Western Deltas in Madras. In 1913, a set of hydraulic diagrams for design of channels was presented by A. Garret, deals with non-silting channels and which is used extensively in the United Provinces (Uttar Pradesh)

In 1917, F.W. Woods proposed the use of the definite rations of depth to width based on the analysis of data from the Lower Chenab Canal System. In 1919, the result of an extensive analysis of canal dimensions of the Lower Chenab Canal by E.S. Lindley (1919) were published. He found a relation of surface with to depth and between velocity and depth and velocity and surface width. No attempt was made by Lindley no correlate rugosity and silt grade, nor he did correlate the width and depth the discharge.

W.T. Bottomly (1928) advanced the idea that irrigation channel would be non-silting and non-scouring if the slope of the canal was of the some order as that of the parent river regardless of the relation of width to depth and the slope of the canal.

In 1930, an excellent paper on this subject was presented by G. Lacey in which he advanced the proposition that the wetted perimeter of stable channel was a simple function of the square root of the discharge and that the shape of the section depend upon the fineness of the silt carried, coarse silt giving rise to wide, shallow section and fine silt to narrow, deep ones.

In his paper of 1935 on stable channels in erodible material, Lane made a comprehensive study of stable channel shaped and stressed that the quantity of solids in motion is an important factor in the shape of stable channels in alluvium.

In 1936, Bose and the staff of the Punjab Irrigation Research, after several years of painstaking collection and statical analysis of data. The former is comparable width the discharge perimeter formula evolved by Lacey, the coefficient being five percent higher, while the letter is closely related to Lacey's slope formula.

In 1941, Blench and King wrote a paper entitled Effect of Dynamic shape on Lacey's relationship. This was followed by Practical Design formulae for stable irrigation channels by C. King.

Many formulaes have been given by various investigator, all available existing regime theories. At the end some conclusions have been mentioned and a suitable design method is recommended.

Some of the important formulaes, which are used by many investigator is presented in Table 2.11.

Table 2.11
Regime Theories Empirical Relationships

Investigator	Formula		Remark
	FPS	Metric	
2.9.1 Kennedy (1895)			
	$V_o = 0.84\, D^{0.64}$	$V_o = 0.546\, D^{0.64}$	- Fundamental equation
	$V = 0.84\, m\, D^{0.64}$	$V = 0.546\, m\, D^{0.64}$	- Fundamental equation ,where $m = V/V_o$ = CVR - Critical velocity was affected by the grade of the silt and in order to account for the effect of the silt grade and introduced in his equation a factor m called Critical Velocity Ratio (C.V.R), Where Value of m : For finer silt m < 1, (m = 0.80 – 0.90) For coarser silt m > 1, (m = 1.10 – 1.20) (see table 2.12)
2.9.2 Lindley (1919)			
	$V = 0.95\, D^{0.57}$	$V = 0.567\, D^{0.57}$	- Fundamental equation
	$V = 0.59\ B^{0.35}$	$V = 0.265\ B^{0.35}$	- Fundamental equation
	$B = 9.8\, D^{1.16}$	$B = 7.86\, D^{1.16}$	- Fundamental equation - Bed width of a channel as a regime variable in additional to depth of flow, and thus made a valuable advance in regime theory

Investigator	Formula		Remark
	FPS	Metric	
			Lindley Stated "when an artificial channel is used to carry silty water, both bed and sides scour or silt, changing depth , gradient and width until a state of balance is attained at which the channel is said to be in regime".
2.9.3 Lacey (1939)	(i) $V = 16 R^{2/3} S^{1/3}$	$V = 10.8 R^{2/3} S^{1/3}$	- Fundamental equation
	(ii) $V = 1.15 f^{1/2} R^{1/2}$	$V = \sqrt{\frac{2}{5} f.R}$	- Fundamental equation
	(iii) $Af^{1/3} = 1.26 Q^{5/6}$	$Af^2 = 140 V^5$	- Fundamental equation
	(iv) $V = \frac{1.346}{N_a} R^{3/4} S^{1/2}$	$V = \frac{1}{N_a} R^{3/4} S^{1/2}$	- Fundamental equation, where $N_a = 0.0225 f^{0.25}$
	(v)	$S = \frac{f^{2/3}}{4980 R^{1/2}}$	- Derived by raising the power of eq.(i) & (ii) by (iii) and then eliminating V^3
	(vi) $P = 2.67 Q^{1/2}$	$P = 4.75\sqrt{Q}$	- Derived by raising the power of eq.(ii) by (iv) and then eliminating f^2 between this and eq.(iii). Then multiplying both sides by A and replacing V,A by Q, and A/R by P
	(vii)	$S = 0.000178 \frac{f^{5/3}}{q^{1/3}}$	- Derived from eq.(ii) with approximation of P =B and q=Q/p
	(viii) $V = 0.79 f^{1/3} Q^{1/6}$	$V = \left(\frac{Qf^2}{140}\right)^{1/6}$	- Derived from eq.(iii) - Derived from eq.(ii) and substituting (iii) by (vi)

Investigator	Formula		Remark
	FPS	Metric	
	(ix) $S = \frac{f^{5/3}}{1859Q^{1/6}}$	$S = \frac{f^{5/3}}{3340Q^{1/6}}$	- Derived from eq.(vi) and (v)
			- Derived from eq.(i) and (ii)
	(x)	$R = 0.47\left(\frac{Q}{f}\right)^{1/3}$	- Holds good for wide and approx rect. channels only. Derived from eq.(viii) and (ii). This is to be used when looseness factor * is more then one (Ref. : I.S 6966)
	(xi) $R = 0.91\left(\frac{q^2}{f}\right)^{1/3}$	$R = 1.35\left(\frac{q^2}{f}\right)^{1/3}$	- Derived from eq.(ii), (viii) and (ix). This is to be used when looseness factor is less than one.
			- Derived from (i), (ii) and substituting q = Q/P, treating P = B for very wide channels and low depth.
			- * looseness factor, the ratio of the overall length of weir/barrage provided to be theoretically computed minimum stable width of river, at the design flood obtained by using Lacey's equation
			- silt factor f, this can be calculated by flowing the average particle size m_r in mm of the soil using the relationship $f = 1.76\sqrt{m_r}$. [f may be checked from Lacey's relationship $f = 290 (R^{1/2} S)^{2/3}$]. Value of f depend on different materials in Table 2.13

Investigator	Formula		Remark
	FPS	Metric	
2.9.4 Bose (1936)			
	$V = 1.12 R^{1/2}$		- Fundamental equation
	$P = 2.68 Q^{1/2}$		- Fundamental equation
	$S = \frac{0.00209 d^{0.86}}{Q^{1/2}}$		- Fundamental equation
	$\frac{R}{P} = \frac{S^{1/4}}{6.25d}$		- Fundamental equation - Both the silt factor f of Lacey and the weighted mean diameter d of Bose define the size of the sediment transported in a channel, but not sediment charge or rate at which the sediment is transported. Where d is the weighted mean diameter of sediment
2.9.5 Malhotra(1939)			
	$V = 18.18 R^{0.63} S^{0.34}$		- Fundamental equation
2.9.6 White (1939)			
	$V = \frac{0.7 \omega^{1/4} g^{2/5} R^{1/2}}{Q^{1/20}}$		- Fundamental equation
2.9.7 Inglis (1941)			
	$V = \frac{\alpha_3 g^{7/18} Q^{1/6} (C \omega d)^{1/12}}{\nu^{1/36}}$		- Fundamental equation

Investigator	Formula		Remark
	FPS	Metric	
	$B = \frac{\alpha_1 Q^{1/2} (C\omega)^{1/4}}{g^{1/3} \nu^{1/12} d^{1/4}}$		- Fundamental equation
	$A = \frac{\alpha_2 \nu^{1/36} Q^{5/6}}{g^{7/18} (C\omega d)^{1/12}}$		- Fundamental equation
	$S = \frac{\alpha_5 (C\omega d)^{5/12}}{\nu^{5/36} g^{1/18} Q^{1/6}}$		- Fundamental equation
	$D = \frac{\alpha_4 \nu^{1/9} Q^{1/3} d^{1/6}}{g^{1/18} (C\omega)^{1/3}}$		- Fundamental equation
	$\frac{B}{D} = \frac{\alpha_5 Q^{1/6} (C\omega)^{7/12}}{g^{5/18} \nu^{7/36} d^{5/12}}$		- Fundamental equation - Sediment charge has a small effect on the area of a channel, relatively great effect on slope and shape and considerable effect on channel width - $\alpha_1, \alpha_2, \alpha_3, \alpha_4$ and α_5 is Coefficients, the values is about 1/400 for uniform sand originally used in the experiment and probably about 1/233 for natural river bed sand.
	$V = (Fb\, Fs\, Q)^{1/6}$		- Fundamental equation
	$S = \frac{Fb^{5/6} Fs^{1/12} \nu^{1/4}}{3.63 (1 + aC) g Q^{1/6}}$		- Fundamental equation

Investigator	Formula		Remark
	FPS	Metric	
	$B = \left(\frac{Fb\,Q}{Fs}\right)^{1/2}$		- Fundamental equation
	$D = \left(\frac{FsQ}{Fb^2}\right)^{1/3}$		- Fundamental equation
	$Fb = \frac{V^2}{D}$		- Fb defining bed sediment factor or bed factor, is related to erosive action on bed - Fs defining side factor, is related to erosive action on sides. Values for side factor in design are as follow: Fs = 0.1 for bank materials of slight cohesiveness Fs = 0.2 for bank materials of medium cohesiveness Fs = 0.3 for bank materials of height cohesiveness $Fs = Fb^2/8$ for rounded gravel
	$Fs = \frac{V^3}{B}$		
2.9.9 Manning (1889 ,1936)	$V = C\,R^{2/3}\,S^{1/2}$		- Fundamental equation
	$V = \frac{1.49}{n} R^{2/3} S^{1/2}$	$V = \frac{1}{n} R^{2/3} S^{1/2}$	- In which n is roughness factor shown in Table 2.14 - This formula was developed from seven different formula viz. Chezy, Ganguillet and Kutter, Bazin, Powell and others. Based on Bazin's experimental data, and further verified by 170 observation. Owing to its simplicity of form and to the satisfactory results it lends to practical applications. The Manning formula has become the most widely used of all uniform flow formulas for open channel computation.

Table 2.12
Values of C.V.R for different type of soil

Type of silt	Values of m
Light sandy silt in the rivers of Northern India	1.00
Somewhat coarse silt and debris of hard soils	1.10
Sandy, loamy silt	1.20
Rather coerce silt or debris of hard soils	1.30
Silt of river Indus in Sindh (Pakistan)	0.7

Source : Varshney, R.S (2000)

Table 2.13
Silt Factor for Different Material

Type of Soil	Value of f
Fine silt	0.5 to 0.7
Medium silt	0.85
Standard silt	1.0
Medium sand	1.25
Coarse sand	1.50

Source : Modi, P.N (1995)

2.11 MANNING'S FORMULA

In 1889, an Irish Engineer Robert Manning (1895), in his effort to co-relate and systematize existing data of flow through natural and artificial channels, developed an equation from seven different formulae viz. Chezy (1796), Ganguillet and Kutter (1869), Bazin (1897), and others. This was later modified to its present well known form. This has been verified by many others and later come to be well known as Mannning's Equation.

$$V = \frac{1}{n} R^{2/3} S^{1/2} \qquad \text{(metric units)} \quad \text{..........Eq. (1)}$$

Or its back conversion gives

$$V = \frac{1.486}{n} R^{2/3} S^{1/2} \qquad \text{(English Units)}$$

Where :

V = mean velocity of flow in m/sec in metric system or feet/sec in FPS system

R = hydraulic radius in m or ft

= A/P , A = water area i.e area of flow in sq. m or sq. ft

P = wetted perimeter in m or ft.

S = slope of the energy line (when flow is uniform energy slope gradient may become parallel to the water surface slope and bed of the channel). This is a very common assumption in the design of canals. Where flow is assumed as uniform. But in actual practice uniform flow rarely exists. Therefore, after the design of channel, actual water surface profile should be computed, energy grade line plotted, and flow phenomena again checked for different discharge likely to occur say, quarter, half, three fourth and full discharge. This will give actual water surface profile, which will be useful in locating level of outlet, operation and maintenance.

n = coefficient of roughness, (also know as rugosity coefficient) specifically known as Manning's n (see Table 2.14)

Owing to its simplicity and satisfactory results, it is most widely used for all open channel flow computation and design.

Table 2.14
Rugosity Coefficient (n) and Limiting Velocities
For Different Types of Linings

S. No.	Surface Characteristic	Surface Characteristic		Maximum Permissible/ Limiting Velocity (m/s)	
		New Surface	Old Surface	(IS 4515-1993)	(IS 10430-2000)
1.	**Concrete Lined with Surface as Indicated below**				27
	(a) formed, no finish	0.013 – 0.017	0.018 – 0.02 *	27	
	(b) Trowel, finish	0.012 – 0.014	0.015 - 0 018*		
	(c) Float finish	0.013 –0.015			
	(d) Float finish, some gravel on bottom	0.015 – 0.017			
	(e) Gunitc, good section	0.016-0.019			
	(f) Gunitc wavy section	0.018-0.022	0.018 - 0.022*		
	(g) P.C.C. Tites/Stabs		0.018 - 0.020*		
	(h) U.C.R. random rubble masonry with pointing	0.024-0.026			
2.	**Concrete Bottom Float Finished Sides as Indicated below**				
	(a) dressed stone masonry		0.019 - 0.021		
	(b) course rubble masonry	0.015-0.017			
	(c) random stone in mortar	0.017-0.020	0.020 - 0.025		
	(random rubble masonry)	0.016-0.020			
	(d) random rubble masonry		0.015 - 0.017*		
	cement plastered	0.020-0.030			
	(e) dry rubble (rip rap) (stone pitching)		0.020 - 0.033*		
3.	**Boulder ling as per IS 4515 – 1993**		0.022 - 0.027*	1.5	
4.	**Gravel Bottom Sides as Indicated Below**				
	(a) Formed concrete	0.017-0.020			
	(b) Random stone in mortar	0.020-0.023			
	(c) dry rubble (rip rap)	0.023-0.033			
5.	**Brick**				
	Burnt clay brick / tile	0.014-0.017			
	(IS 3872-1966)	0.018-0.20	0.018 - 0.020*		18
6.	**Asphalt**				
	(a) Smooth	0.013	0.013 - 0.015*		
	(b) Rough	0.016	0.016 - 0.018*		
7.	**Wood Planned Clean**	0.011-0.013			

* Values recommended by IS code 10430 – (2000): 4745- (1964); 4515 – (1993);3872 (1996)

Note : For canal is curves (other than straight) a small increase in value of n be made to allowed for additional loss of energy

Several research workers have developed many formulae for the flow through canals and have remained in vogue in different times in different parts of the world. But now Manning's formula is widely used through out the world.

For design of channels another following continuity equation (2) is used.

$Q = A_1 V_1 = A_2 V_2$ ....Eq. (2)

Thus equation (1) and (2) are used together and a section is determined by trial and error.

To avoid any trial and error, and to find a unique solution, Suryanvanshi (1973) has developed a master equation

2.11.1 Some Useful Derivation of Manning's Formula for Best Section

It has been stated in most text books of hydraulics that, for the most efficient section of any shape of more then 2 sides, the hydraulic radius (R) will be one-half of the flow depth but the section must be circumscribing a circle.

The most efficient polygonal section of any specified number of sides can be found to be one which can be circumscribing a semicircle.

The most optimum trapezoidal section has side slope of 1 : 0.5777 (V:H), B = 1.155 d, $A = 1.732\ d^2$, P = 3.464 d, R = 0.5 d.

When these values are put in the Manning's formula, the relationship is a unique relationship as under :

$$\mathbf{V = (d/2)^{2/3} S^{1/2} / n}$$

$$= (1/2)^{2/3} S^{1/2} / n$$

$$\mathbf{= 0.63\ d^{2/3} S^{1/2} / n} \quad \text{.........Eq. (1a)}$$

or

$$S = (V n / 0.63\ d^{2/3})^2$$

$$= V^2 n^2 / 0.3969\ d^{4/3}$$

This may be called as the Master Equation (1) of Manning' formula for the best or optimum section (or most hydraulic efficient section).

For any type of the best section R = d/2 only and always only, irrespective of any shape, side slope and size. The above equation shows a direct and unique relationship between V, d, and S, treating n as constant. For a given velocity or treating V as constant, there is unique depth slope only, and can not be altered. This perimeter is minimum, and this area is also minimum, and B/D ratio is also fixed, and can not be altered, thus no trial and error is required.

These equation can also be written in the form

$V = Q / A = 0.63\ d^{2/3}\ S^{1/2} / n$

Putting optimum value of A for trapezoidal section of optimum side slope

Z = 1:0.577 (V:H)

$Q = \sqrt{3}\ d^2\ (0.63\ d^{2/3})\ S^{1/2} / n$

$= 1.732\ (0.63)\ d^{8/3}\ S^{1/2} / n$

$= 1.091\ d^{8/3}\ S^{1/2} / n$

$$\text{or } d = \left[\frac{Qn}{1.091 S^{1/2}}\right]^{3/8} \qquad \text{...........Eq. (1b)}$$

Treating Q as given or Constant, there is a unique relationship between dept and slope and can not be altered.

For a given discharge and depth there is fixed depth and can not be changed. Similarly for a given discharge and depth there is fixed slope or minimum slope, and can not be designed on any slope flatter then this.

2.12 COMPARISON OF MANNING'S, KENNEDY'S AND LINDLEY'S FORMULA

2.12.1 Kennedy's Formula

Canal velocities and their relations to erosion and silting were studied by Kennedy (1895). He obtained a formula for the velocity that can be maintained through erodible materials without causing silting and scouring in Bari Doab Canal. His formula often called Kennedy's Critical Velocity is :

$V_o = C\ D^{0.64}$ (Metric units)

Where V_o = critical velocity, D = depth of flow and C = Coefficient. He found value of C = 0.546 (coefficient of Kennedy) for Bari Doab Canal. From this equation, velocity for different depth of flow as under:

Table 2.15

Relation of Depth to Allowable Velocity by Kennedy's Formula

Depth, m	1	2	3	4	5	6	7	8	9	10
Velocity, (m/s)	0.546	0.851	1.103	1.326	1.529	1.719	1.897	2.066	2.228	2.383

There are plotted in Figure 2.39

2.12.2 Lindley' Formula

Similar to Kennedy's Equation, Lindley (1919) also gave velocity depth relation as:

$V = 0.567\ D^{0.57}$ (Metric units)

From this equation, velocities basis for different depth of flow are as under :

Table 2.15

Relation of Depth to Allowable Velocity byLindley's Formula

Depth, m	1	2	3	4	5	6	7	8	9	10
Velocity, (m/s)	0.567	0.842	1.061	1.250	1.419	1.574	1.719	1.855	1.984	2.107

There are plotted in Figure 2.39

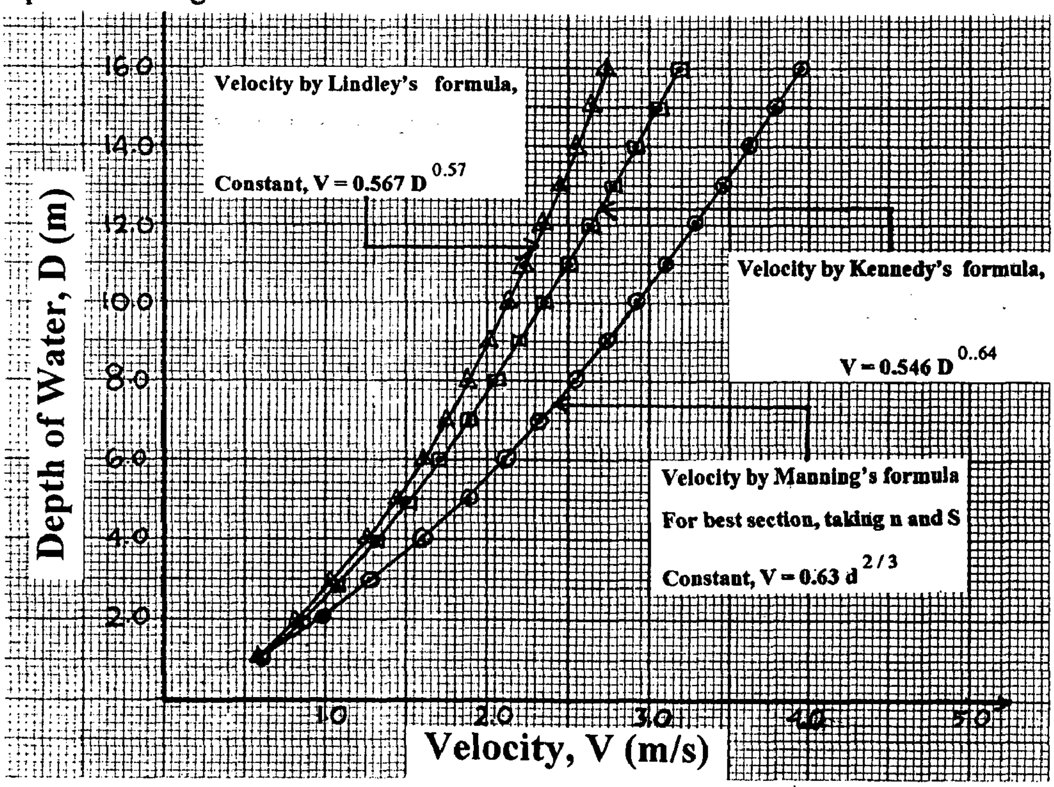


Fig.2.39 Relation of Depth to Allowable Velocity

2.12.3 Comparison of Manning's Formula with Kennedy's and Lindley formula

The equationscan be easily compared with the eq. (1a) of the Manning's formula i.e

$V = 0.63\ d^{2/3}\ S^{1/2} / n = 0.63\ d^{0.667}\ (S^{1/2}/n)$

By treating slope (S) and rugosity coefficient n as constants, generally specified in any canal design. The equation reduces to the simple form of

$V = 0.63\ d^{0.66}$ .... (0.63 for optimum side slope and B/D ratio)

This is very similar to Kennedy's Equation or in other words Kennedy's Experiment all results, (empirical relationship) is very near to the Manning's Equation of most efficient trapezoidal section.

For a bed slope of 1/10000 and n = 0.01 $S^{1/2}/n = \frac{1}{0.01}\left[\frac{1}{10000}\right]^{1/2} = \frac{0.01}{0.01} = 1$, for n = 0.02, and to attain same velocity, slope be raised to 1/2500, then

$$S^{1/2}/n = \frac{1}{0.02}\left[\frac{1}{2500}\right]^{1/2} = 1$$

Similarly Lindley's Equation, a velocity depth relation is also very near to Kennedy's and Manning's formula.

These three relationships are plotted in Figure2.39 for depth (8.00 m normally maximum so far adopted).

CHAPTER III

STUDY OF INDIRA GANDHI NAHAR PROJECT (IGNP) FORMERLY KNOWN AS RAJASTHAN CANAL PROJECT (RCP) INDIA

3.1 GENERAL

This is a gigantic canal project to carry 524 cumecs (18500 cusecs) water from Harika Barrage in a 204 Km long feeder canal in Punjab, to the vast great Indian desert known as Thar desert, in western Rajasthan. The canal network is spread in an area of about 60 Km wide by 1000 km long belt. It consists of 204 Km of feeder, 450 Km main canal, 8000 Km of distribution networks and several thousand km of lined water courses, to spread over a gross command area of 2.5 Mha and provide irrigation to a culturable command of 1.55 Mha.

The project was conceived by the great Indian Engineer Kanwar Sen Jain, around the year 1940 and construction was started in the year 1958. Since then the project has gone under considerable modifications/changes/revision after revision, several times. It is still (year 2002) under construction near tail areas.

Planning, design and construction of canal system is managed by a high powered Canal Board with many advisory, technical committees consisting of eminent engineers/consultants.

3.1.1 Main Canal

It is a contour canal with distribution network and irrigation on the right side only. Few lift schemes are provided on the left.

The main canal though initially thought as a unlined canal was subsequently designed and constructed as a lined canal.

The main canal passes through sandy desert soils. High cuttings of about 20 m above bed level to heavy bed filling of more than 4 m (from bed level i.e. 12 m to top bank level) are encountered in the alignment of the canal.

No materials of construction except the desert sand are available all along the canal, or even within 100 Km. There are no rivers or stone hillocks nearby.

However, there are some stone hillocks near Ratangarh at a distance of 200 to 300 Km from main canal and in tail areas after 450 Km of canal (near Mahangarh) at a distance of about 50 Km from tail. Even the coarse sand (locally known as Bajri) required for cement mortar is available at a distance of 200 to 300 km away from canal in deep quarries of Shivbari (Bikaner) and Bap (Phalodi). However, clay soil for manufacture of tiles/bricks is available in small pockets in depression in between sand dunes, at distances varying from 5 to 100 Km. The section of lined canal and type of lining was decided after detailed deliberation and discussions in a symposium.

3.1.2 Internal Section

The internal hydraulic section of lined feeder (canal) and details of few more sections are given in Table 3.1 and Figure 3.1. The depth of the canal has been limited to 6.5 m (21 ft.), in head reaches for stability of sandy soils, operational problems and easiness in construction. It gradually decreases in tail. Internal side slopes of 1:2 (V:H) were considered safe for sandy soils and provided for depth from 6.5 m to 5 m, through out the entire length of 450 Km of main canal.

Bed slope has also been restricted to 1 in 12000, because of long length of canal, practically 650 km, and to have sufficient command. Even with this flat slope, the drop in water level is 54 m from head to tail. It is uniform from head to tail in 450 km length. It is uniform from head to tail in 450 Km length. Thus velocities are also very much limited. There are only two ways to increase the velocity, one is to increase the slope and second is to increase R (as per Manning's equation). By limiting depth and energy slope and internal side slope, velocity is very much limited in the entire length from 1.5 m/s to 1.2 m/s. Bed width varies, practically from 11 times the depth at head to 2 times the depth at tail.

3.1.3 Lining

Single title lining in bed and double tile lining on sides has been adopted. Burnt clay tile lining up to 365 Km and thereafter P.C.C. block lining is adopted from 365 Km to 450 Km (tail). Details of tile lining are as under:

Burnt clay tile lining in bed and sides

(a) Bed - Single clay tile lining

(i) Base coat 9 mm (3/8") thick, 1:5 cement mortar plaster over compacted and dressed sub grade.

(ii) 50 mm thick 305 x 152 x 50 mm (12" x 6" x 2") burnt clay tiles laid in 6 mm thick 1:5 cement mortar on base coat

(iii) 19 mm (3/4") thick 1:3 (Cement Mortar) plaster over tiles.

Total thickness of lining = 9 + 6 + 50 + 9 = 74 mm (3")

(b) Side slopes - double tile lining.

(i) Base coat 9 mm (3/8") thick, 1:5 cement mortar plaster over compacted and dressed sub grade.

(ii) 50 mm thick 305 x 152 x 50 mm (12" x 6" x 2") burnt clay tiles laid in 6 mm thick, 1:5 cement mortars on base coat.

(iii) 15 mm (5/8") thick 1:3 (Cement Mortar) plaster over tiles, called as sandwich plaster

(iv) 2nd layer of tiles laid in 1:3 (Cement Mortar) 6 mm thick over sandwich plaster, total thickness of lining = 127 mm (5¼ inches)

Thus it can been that lining of bed is cheaper than sides. The ratio of cost may be around 40:60, between bed and sides.

3.1.4 Lining of Branches, Distributaries and Minors:

Initially all branches, their distributaries and minors were planned, designed and constructed as unlined channels. The area through which these are passing varied from sandy dunes of desert to hard clay, soils of all sorts and many times entered kankar, gypsum from soft to very hard (locally known as dhandla, a weak stone).

These channels were designed by Kennedy's formula, with the help of Garrat's Diagram, and the following stipulation about Manning's N. (RCP 1962)

	Discharge	Manning's N	Corresponding value of silt factor (f)
(i)	up to 5 cusecs (0.11 cumecs)	.03	3.16 (gravel)
(ii)	5 to 50 cusecs (1.35 cumecs)	.025	1.52 (coarse sand)
(iii)	51 to 500 cusecs (1.35 to 1.4 cumecs)	.0225	1.00 (standard silt)
(iv)	above 500 cusecs (1.4 cumecs)	.02	0.62 (fine silt)

(Lacey, related his silt factor 'f' to Manning's N, the coefficient of rugosity by the expression, $N = .0225 f^{1/4}$).

Also a bed width depth ratio (B:D) for unlined channels was adopted as per CWPC (1960) practice. According to it, B:D ratio varies from 2.9 to 7.9 for discharge range from 10 cusecs (0.25 cumecs) to 2000 cusecs (60 cumecs). That means most of the channels had width more than 5 to 9 times the depth. Also critical velocity ratio (V / V_o) was kept near unity and velocity was restricted to 2.5 ft/sec.

Thus, the channels are very wide enough even for discharges less than 3000 cusecs, with very flat slopes.

Later on, around the year 1970, it was decided to line these earthen channels and designs, construct future canals as lined channels.

Different practices of internal section for lining the earthen channels were adopted. Most of new channels were designed and constructed as Mehboob section and lined with single or double tile lining. But near tail areas, the design was further changed to suit to different type of lining in bed and sides, and use LDPE/PVC film.

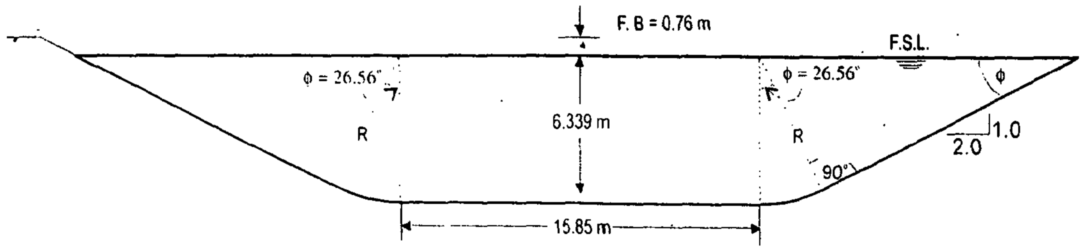
Table 3.1

Details of Indira Gandhi Canal Sections, India. Double Tile Lined (DTL),

Bed Slope 1 in 12000, n = 0.017, Trapezoidal Section with Curved Ends, Side Slopes 2 : 1 (H : V)

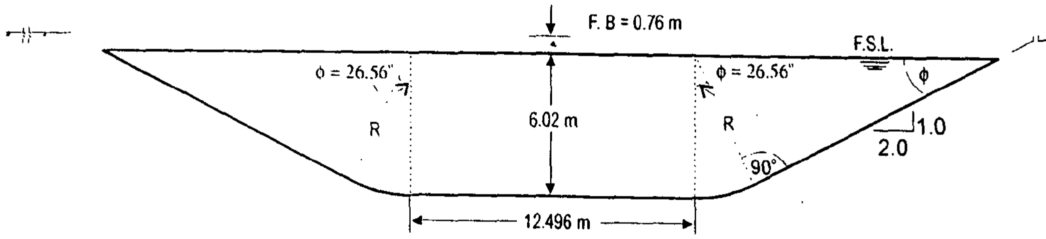
S. No	Location of Canal	Discharge Q (m^3/s)	Velocity V (m/s)	Bed Width B (m)	Depth D (m)	B/D ratio X	Wetted Perimeter P (m)	Area A (sq.m)	Hydraulic Radius R=A/P (m)
1	2	3	4	5	6	7 (5/6)	8	9	10
	km 30 (RD 100)	447	(4.64) 1.529	33.83	6.40	5.29	70.72	317.56	4.49
2.	Km 122 (RD 400)	358	(4.58) 1.469	22.25	6.40	3.48	59.14	243.43	4.12
3.	Km 134 (RD 440)	353	(4.45) 1.466	21.95	6.40	3.43	58.82	241.38	4.10
4.	Km 146 (RD 475)	317	(4.46) 1.43	18.60	6.40	2.91	55.09	216.77	3.93
5.	Km 170 (RD 560)	308	(4.45) 1.43	18.29	6.34	2.88	54.78	214.77	3.92
6.	Km 189 (RD 620)	280	(4.35) 1.41	15.85	6.34	2.50	52.37	199.49	3.81
7.	Km 216 (RD 710)	267	(4.35) 1.395	15.24	6.25	2.49	51.25	191.52	3.74
8.	Km 250 (RD 820)	237	(4.35) 1.360	12.50	6.25	2.00	48.50	174.34	3.59
9..	Km 293 (RD 962)	178	(4.06) 1.263	12.19	5.42	2.25	47.04	163.55	3.48
10.	Km 341 (RD 1120)	166	(4.07) 1.24	12.19	5.33	2.29	42.91	135.03	3.15
11.	Km 384 (RD 1260)	151	(3.98) 1.217	10.67 v	5.27	2.02	41.00	124.51	3.04
12.	Km 445 (RD 1400)	136	(3.88) 1.18	10.36 v	5.06	2.05	39.52	115.55	2.92

Source: IGNB-Revised Project estimate, (1990) 1RD = 1000 ft.



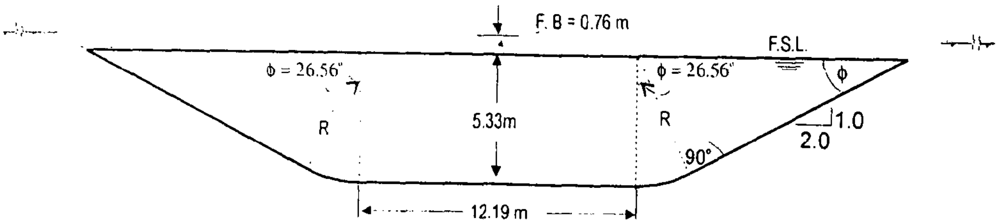
A = 199.49 m²
P = 52.37 m
R = 3.81 m
V = 1.41 m / s
Q = 280.00 m³ / s

(i) Section of IGMN From K.m 189 to 216 (RD - 620 to RD - 710)



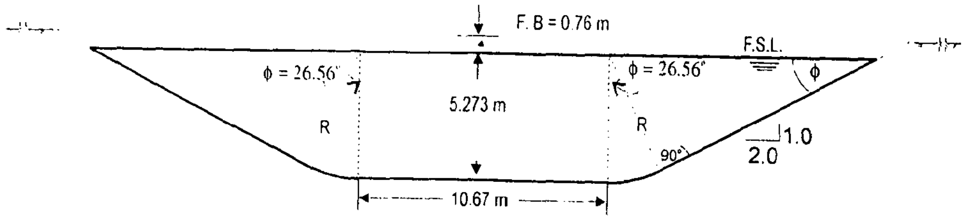
A = 174.34 m²
P = 48.50 m
R = 3.59 m
V = 1.36 m / s
Q = 237.00 m³ / s

(ii) Section of IGMN from km. 250 to km. 293 (RD - 820 to RD - 962)



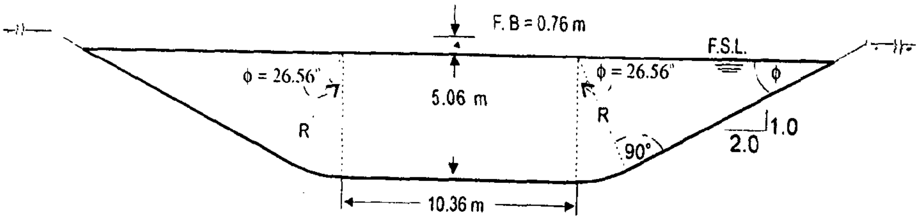
A = 163.55 m²
P = 47.04 m
R = 3.48 m
V = 1.263m / s
Q = 178.00 m³ / s

(iii) Section of I.G.M.N. R.D.-962



A = 124.51 m²
P = 41.00 m
R = 3.04 m
V = 1.217m / s
Q = 155.55 m³ / s

(iv) Section of I.G.M.N. R.D.-1260



A = 155.55 m²
P = 39..52 m
R = 2.92 m
V = 1.18 m / s
Q = 136.00 m³ / s

R.D.-1405

(v) Lined Canal Section of I.G.M.N.

Figure - 3.1 Canal Sections of Indira Gandhi Main Canal (IGMN) Between km. 189 to 445.

Table : 3.2
Details of Off Taking Channels of Indira Gandhi Main Canal
(a) Stage I from Km 0 to Km189.

	MAIN CANAL					OFF TAKING CHANNELS		
S. No.	Km	H/R of Off Taking Channels and Cross Regulator's	Discharge, Q (m^3/s)	Full Supply Level, FSL (m)	Bel level (m)	Discharge, Q (m^3/s)	Full Supply Level, FSL (m)	Bel level (m)
1	2	3	4	5	6	7	8	9
1	31	Birsalur disty	447	190.057	183.657	N.A	N.A	N.A
2	34	Khodan disty	447	189.84	183.44	N.A	N.A	N.A
3	52	Anupgash Branch	447	188.32	181.92	N.A	N.A	N.A
4	52	Bikaner Loonkaran Sar (Kanwar Sen Lift Canal)	447	188.32	181.92	N.A	N.A	N.A
5	52	Cross-Regulator	447/447	188.32	181.92	N.A	N.A	N.A
6	73	Cross-Regulator	447/447	186.57	180.17	N.A	N.A	N.A
7	125	Cross Regulator	447/358	182.30	175.90	N.A	N.A	N.A
8	138.8	H/R Hiroe sherpura & Lalwali disty.	358	181.09	174.69	N.A	N.A	N.A
9	143	H/R Laklesar disty	318	180.79	174.33	N.A	N.A	N.A
10	154	Cross Regulator	318/317	179.49	173.12	N.A	N.A	N.A
11	189	Cross Regulator	308/281	176.96	170.62	N.A	N.A	N.A

* Incomplete due to unavailability of data. N.A = not available.

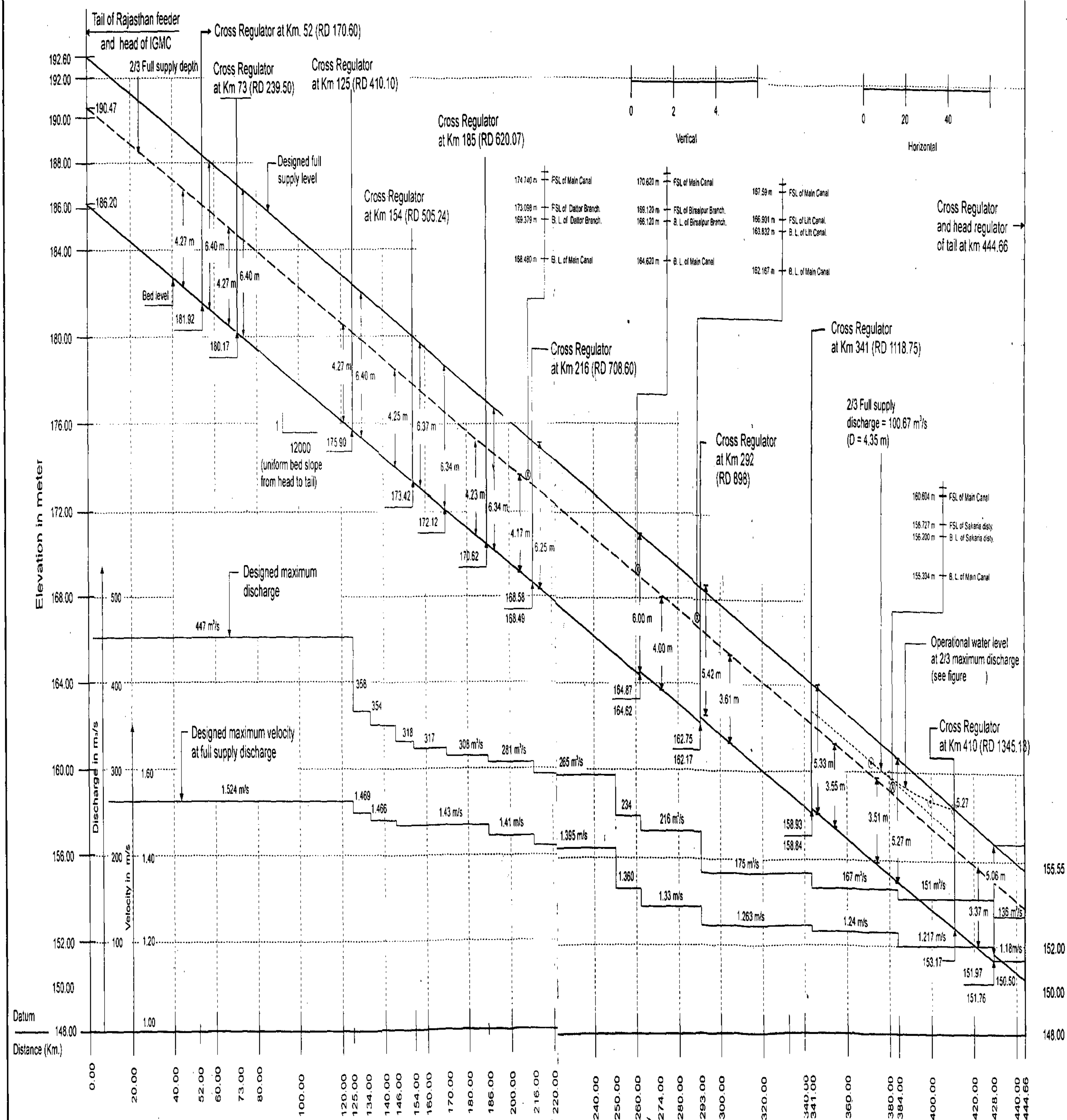
Table : 3.2

Details of Off Taking Channels from Indira Gandhi Main Canal

(b) Stage I From Km 189 to Km 444 (Tail).

			MAIN CANAL			OFF TAKING CHANNELS		
S. No.	Km	H/R of Off Taking Channels and Cross Regulator's	Discharge, Q (m^3/s)	Full Supply Level, FSL (m)	Bel level (m)	Discharge Q (m^3/s)	Full Supply Level, FSL (m)	Bel level (m)
1	2	3	4	5	6	7	8	9
1	216	Cross regulator	281/265	174.740	168.480			
2	216.43	H/R of Dattor branch, Cross Regulator	281	174.634	168.294	15.83	173.098	169.379
3	227.42	H/R of Gajner lift	265	173.644	167.395	12.664	173.644	170.73
4	249.70	H/R of Botta Nala Disty	252.506	171.861	165.609	17.021	170.154	167.814
5	258	Direct outlets	234.927	171.17	164.917	0.485	170.35	169.55
6	261.89	H/R of Bilsalpur Branch	235	170.852	164.604	15.115	169.120	166.120
7	270.66	H/R of Bangsar lift disty	219.885	170.360	164.335	2.166	169.120	167.620
8	275.85	Minor	217.719	169.58	163.58	0.113	167.78	166.28
9	280.42	Minor	217.606	169.17	163.17	0.154	166.67	166.47
10	284.38	Minor	217.452	168.84	162.84	0.127	167.34	166.14
11	286.52	Minor	217.325	168.66	162.66	0.112	166.96	165.96
12	290	Direct out lets	217.213	168.37	162.37	1.719	168.33	167.33

			MAIN CANAL			OFF TAKING CHANNELS		
S. No.	Km	H/R of Off Taking Channels and Cross Regulator's	Discharge, Q (m^3/s)	Full Supply Level, FSL (m)	Bel level (m)	Discharge, Q (m^3/s)	Full Supply Level, FSL (m)	Bel level (m)
13	291.93	H/R Kolayat lift canal	215.00	167.59	162.170	18.95	166.901	163.832
14	291.93	Cross Regulator	215.00/175	167.59	162.170			
15	293	Charanwala Branch	175	167.34	161.92	15.251	166.560	164.060
16	301	Direct Outlet	159.749	166.67	161.25	0.830	162.478	161.478
20	352.57	Nachina Minor	158.919	162.37	156.95	0.190	158.049	157.449
21	368	Awai distributory	158.729	161.08	155.66	1.265	158.498	157.449
22	384.05	H/R of Sarkria disty	157.464	160.604	155.334	4.615	158.727	156.200
23	397.73	H/R of Mitharia disty.	152.849	159.553	154.279	1.789	157.553	156.653
24	410	Cross regulator	151	158.44	153.17			
25	416.04	H/R of Rorund disty	149.849	158.029	152.755	8.549	154.414	153.621
26	428.25	Mohan garh disty.	141.300	157.012	151.738	5.048	155.216	53.216
27		H/R of Digha branch , Tail IGMC	136.252	159.450	150.570	40.006	155.450	153.450
28	444	H/R of Lilva branch,	136.252	159.450	150.570	99.246	155.450	152.570



Note: All off taking channels not shown. For details see table 3.2
Source IGNB - Revised Project Estimates (1996)

Figure 3.2 L - Section of Indira Gandhi Main Canal (IGMC) Showing designed behaviour and operational behaviour near Km 410 (see fig. 3.7 for details)

Post-III.CDR

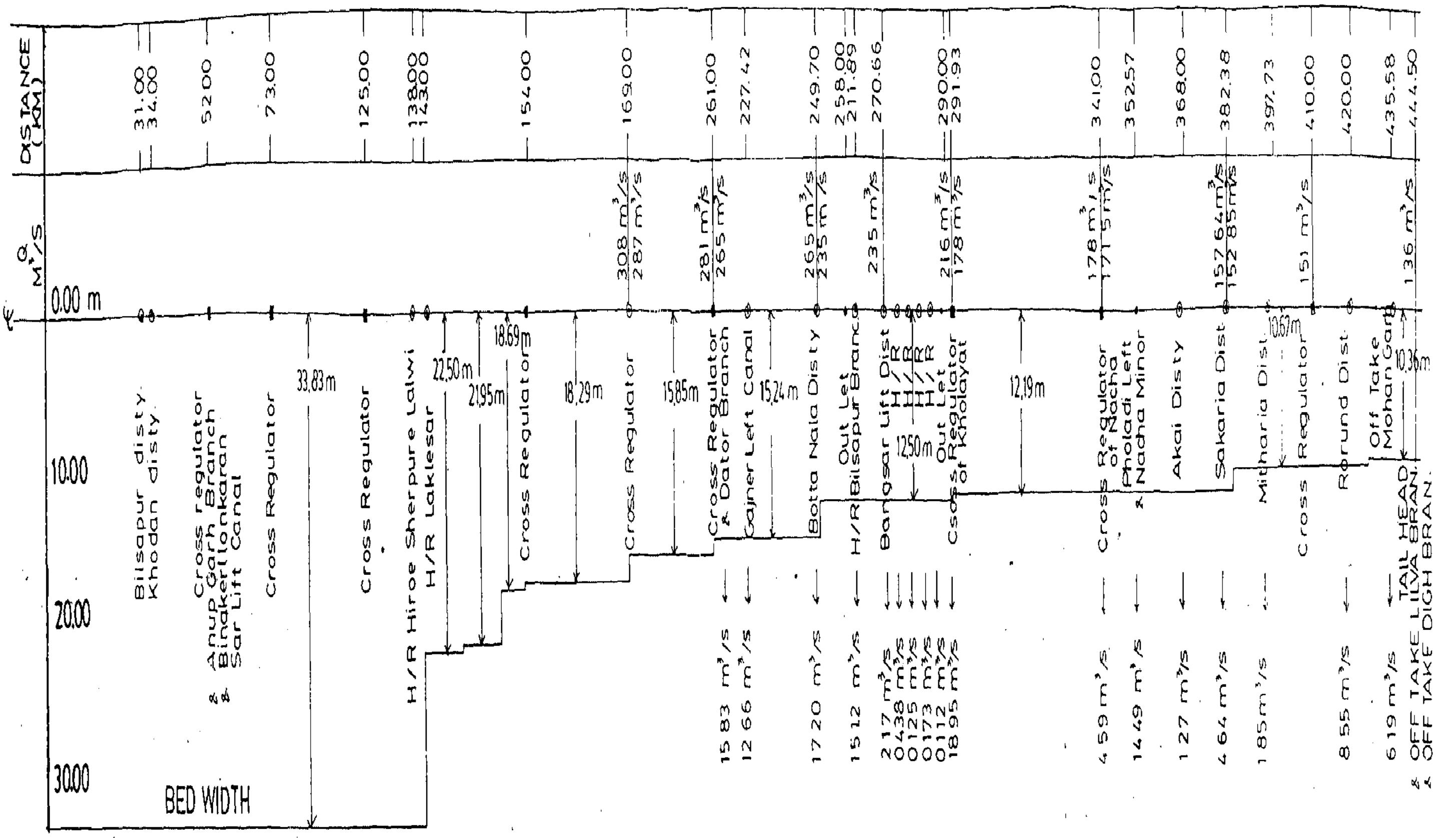


FIG.3.3 - PLAN OF INDIRA GANDHI MAIN CANAL SHOWING OFF TAKING CHANNEL AND BED WIDTH

Sourse: IGMN Revised Project Estimates

Scale:
Vertical 0 5 10 1cm 55m
Horizontal 0 20 40 1cm 20km

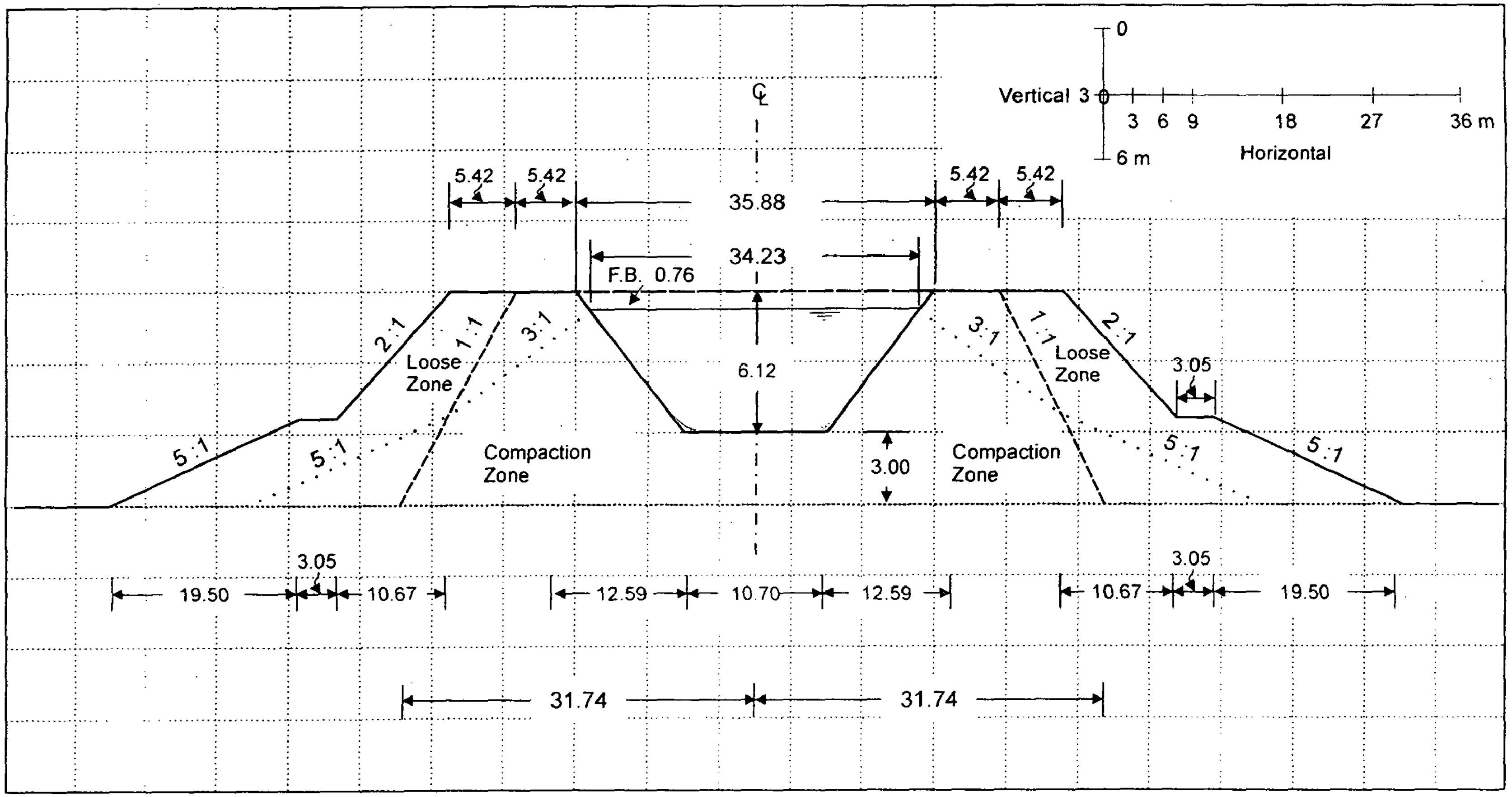


Figure 3.4 **General Typical Cross Section of Heavy Bank Filling up to 9.00 m of Indira Gandhi Main Canal between Km. 380 to 410 (RD 1260 - 1345)**

NEW_ROOR CDR

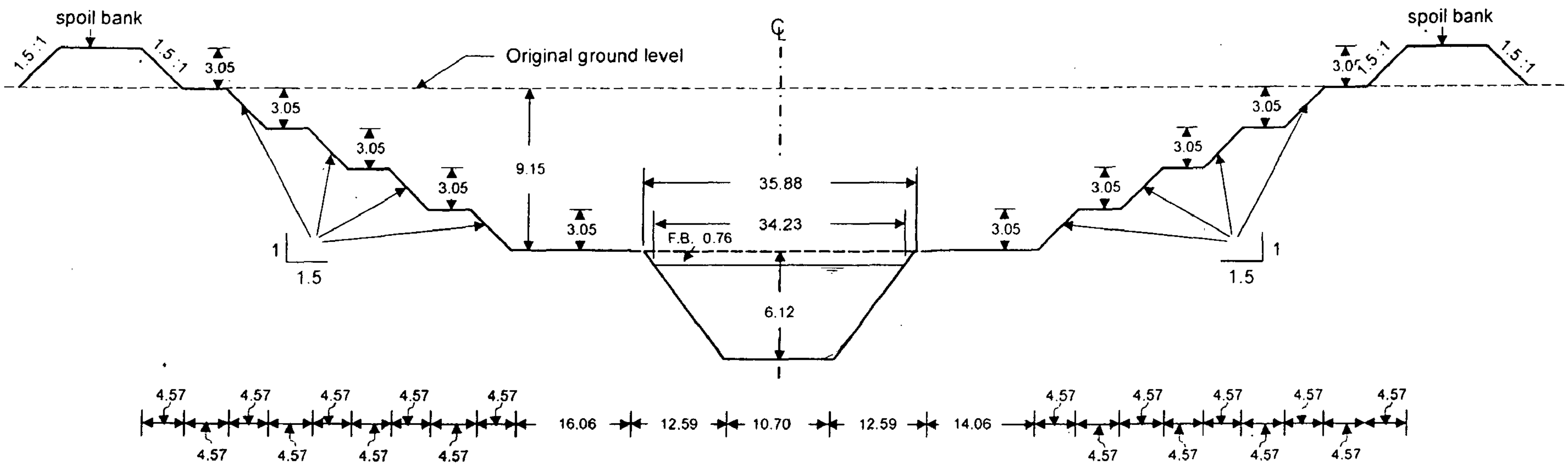


Figure 3.5 General Typical Cross Section of Heavy Cutting Above Bed up to 15.25 m of Indira Gandhi Main Canal, Internal Sections between Km. 384 to 410 (RD 1260 - 1345)

NEW_ROOR.CDR

3.2 STUDY OF FLOW CHARACTERISTIC OF THE MAIN CANAL

The main canal is designed for a maximum discharge of 524 m^3/s at head i.e at km 0. A large of number of branches, distributaries and minors off take from the main canal at different location all along the canal. These are shown in the L- section, Figure 3.2 and a statement off taking canals is given Table 3.2. Many distributaries such as lift canals have been planned subsequently. Also discharges of few off taking channels have been changed. A clear and updated data are not available. So data available are indicated in the Table and plotted in the L - section. A half bottom plan of the canal is also shown in Figure 3.3.

The L-section shows following important features

(i) The bed slope is very mild, 1 in 12000 and uniform through out the entire length.

(ii) Canal has a maximum depth of 6.4 m at head to 5.06 m at tail (Km 444). Decrease in depth is done by a up step in the canal.

(iii) Designed discharge varies from 524 m^3/s at head to 151 m^3/s in tail.

(iv) B/D = X ratio varies from 11 near head to 2 in tail.

(v) Maximum velocity of flow at designed discharge is 1.524 m/s at head and gradually decreases to 1.18 m/s at the tail.

(vi) There are 9 cross-regulators in the main canal and flow is regulated through them.

The maximum discharge is to pass only for a short duration in year. For the rest of period discharge varies according to capacity factor. The capacity factor in lean month is around 0.55 and generally around 0.6 to 0.7, except few months when it may 0.8 or 0.9. It is 1 only in peak demand of 1 month.

With decrease in discharge velocity further decreases. At the cross-regulators the velocity is not uniform. It varies gradually according to discharge.

The canal alignment passes through heavy sand dunes. Two typical cross-sections of heavy cutting and filling are shown in Figure 3.4 and 3.5.

It is reported that the canal get silted due to heavy dust storms. There is no silt ejector in the canal.

In this study an attempt is made to know the flow characteristics of main canal, sediment transporting capacity and comparison with the designed practices.

3.3 EFFECT OF CROSS REGULATOR.

From consideration of regime of channels it will be better if there are no structures across any channel but to carry on regulation without cross regulators is very difficult. The absence of cross regulators envisages constant supply to all off taking channels. But the demand on any channel is not constant with respect to time. Also a particular channel, may have to be closed due to some repairs or on account of breach in it. The supply in the parent channel is not constant. If the supply is reduced the level will fall and other off taking channels will not be able to draw full supply discharge. Thus cross regulators are a necessary evil. Still these are unavoidable and have fallowing advantages.

Advantages of cross regulators:

(i) When there is no head regulator on the main canal the cross regulators to a certain extent minimize the disadvantages for want of head regulator.

(ii) When the water level in the main canal is low, they help in raising the water level and feed the off taking channel.

(iii) They help in raising water level above the designed and thus give full supply to lands slightly above the command level of the canal.

(iv) They enable parent channel to be divided into sections for easy regulation.

(v) They help in absorbing fluctuations in various sections of the canal and thus reduce possibility of flooding tail reach and causing breaches there. The excess discharge is therefore retained all along the canal.

(vi) They help in closing of breaches in lower sections.

(vii) They facilitate working of the whole system by rotation in days of low supply and thus reduce silting in the branch canals.

It is a well-known principle in irrigation engineering that a canal should either run full or dry. When designed for high level and if it runs at low level, considerable silting occurs as the ratio of mean to critical velocity falls.

(viii) They help in increasing revenue by ensuring full supply discharge to most of the tracts.

(ix) They facilitate construction of road bridges with little additional cost.

(x) If the cross regulators were not there all the branches could not be designed with a high full supply level so that much of the area will be thrown out of command.

Disadvantages of cross regulators:

(i) While cross regulators on the parent channel prevent silting of off taking channels by making system of rotation possible, but they by heading up water, cause the parent channel to silt. Part of this silt may be washed away when the cross regulator is opened. Still it must be admitted that too frequent heading up in case of a canal provided with many cross regulators will affect its working.

A relieving factor however is that in the lower reaches where the cross regulators are more frequent, water is usually clearer.

(ii) Cross regulators put in an undue large power in the hands of low paid establishment, who for personal gain are apt to misregulate. A strict watch, control by rotation Tables and surprise visits, though are the remedies for this, but cost much.

On the whole advantages and convenience from regulators outweigh their few disadvantages.

3.4. METHODOLOGY OF COMPUTATION OF FLOW PROFILE UPSTREAM OF A CROSS REGULATOR

By heading up water during low supplies to maintain FSL, a backwater curve is formed for some distance upstream of the cross regulator. The computation of flow profiles involves basically the solution of the dynamic equation of gradually varied flow. Broadly there are three methods of computation; namely.

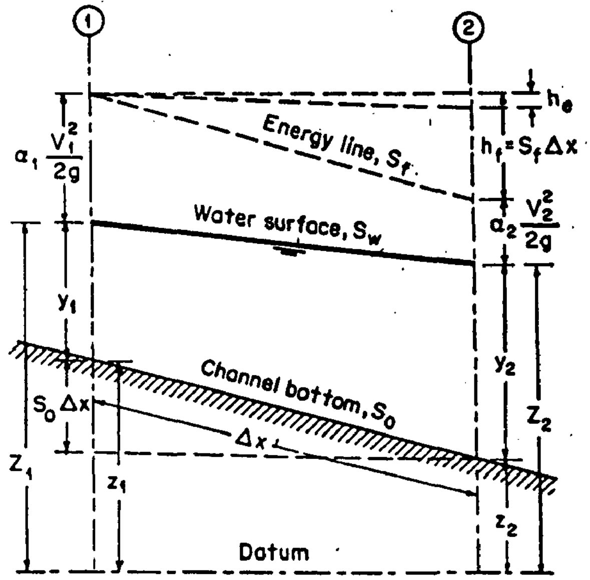
(i) The graphical integration method.

(ii) The direct integration method

(iii) The direct step method

The direct step method is a simple step method applicable to prismatic channels. It is characterized by dividing the channel into short reaches, and computing step by step form one end of the reach to the other.

According to the principle of conservation of energy, the total energy head at the upstream section (i) should be equal to the total energy head of the downstream section (ii) plus the loss of energy between the two section, see figure below.



A Channel Reach for The Derivation of Step Methods (Chow, 1973)

$$S_0 \ \Delta x + y_1 + \alpha_1 \frac{V_1^2}{2g} \quad = \quad S_f \ \Delta x + y_2 + \alpha_1 \frac{V_1^2}{2g} + h_f$$

Solving for Δx , we get

$$\Delta x = \frac{E_2 - E_1}{S_0 - S_f} = \frac{\Delta E}{S_0 - S_f}$$

and for low velocity uniform flow $\alpha_1 = \alpha_2 = 1$ and for short length, h_f may be taken equal to 0.

$S_0 \Delta x + E_1 = S_f \Delta x + E_2$

Also, $y = V^2 / 2g = E$, where E is specific energy

That is specific energy is equal to the sum of the depth of water and the velocity head measured with respect to the channel bottom the total energy in the flow of the section with reference to a datum line is the sum of the elevation z, the piezometric height y, and the velocity head $V^2/ 2g$. Where y the depth of flow, V the mean velocity, α is the energy coefficient = 1, S_0 = the bottom slope, S_f = the friction slope. $\overline{S}_f$ = the average value of S_f. In Manning 's formula , the friction slope is expressed by

$$S_f = \frac{n^2V^2}{R^{4/3}}$$

where ,

n = roughness ; V= mean velocity ; R = hydraulic radius

This requires a discharges rating curve of the canal. Computations of this are done in para 3.5. and Table 3.3. It is shown in Figure 3.6. The computation of backwater are done with the with the help of this Figure and by the above step by step procedure as explained in Table 3.4. The explanatory notes for simplicity are also given below the Table.

The back water curve is plotted in Figure 3.7. This curve is computed for 2/3 full supply discharge, as the same is considered most predominant.

3.5 DEVELOPMENT OF FLOW RATING CURVE OF INDIRA GANDHI MAIN CANAL BETWEEN KM 384 TO KM 410 (RD 1260 TO RD 1345) FOR UNIFORM FLOW

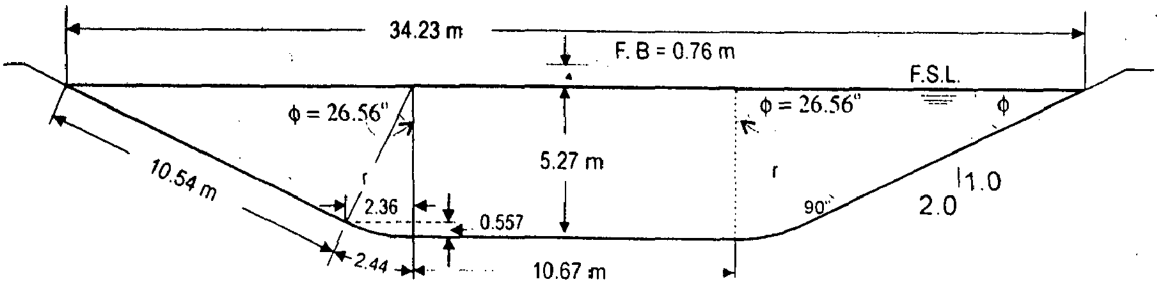
Project Data :

Channel Parameters ,

Discharge, Q = 151,375 m^3/s ; Bed width , B = 10.67 m ; Depth , h = 5.27 m

Bed slope ,S_0 = 1/12000 ; Roughness, n = 0.017 ; Side slope, H:V = 2 :1

Cross section of canal, trapezoidal section with curved ends as below :



Cross Section Of I.G.M.C between Km 380 to Km 410 (RD1260 to RD 1345)

Computation of discharge versus gauge

(i) *Up to ht 0.557 m curved position*

$A = B \times h + 2\ (\pi h^2\phi/360 - ½\ 4.713 \times 2.357)$

$= 10.67 \times 0.557 + 2[\pi \times (5.27)^2 \times 26.57/360 - ½\ 4.713 \times 2.357]$

$= 5.943 + 1.771 = 7.714\ m^2$

$P = B + 2\ (2\pi h \times \phi/\ 360)$

$= 10.67 + 2(\ 2\pi \times 5.27 \times 26.57/360\ ) = 15.558\ m$

$R = A/P = 7.714/15.558 = 0.496\ m$

$V = R^{2/3} \times s_0^{1/2}\ /\ n$

$= (0.496)^{2/3} \times (1/12000)^{1/2}\ /\ 0.017 = 0.336\ m/sec$

$Q = A \times V = 7.714 \times 0.336 = 2.596$ cumecs

(ii) *Up to height 1.00 m*

$A = 7.714 + [(15.384 + 17.156)\ /\ 2] \times 0.443 = 14.905\ m^2$

$P = 15.558 + 2 \times \sqrt{0.443^2+0.886^2} = 17.539\ m$

$R = 14.905/17.539 = 0.850\ m$

$V = (0.850)^{2/3} \times (1/12000)^{1/2}\ /0.017 = 0.482\ m/sec$

$Q = 14.905 \times 0.482 = 7.182$ cumecs

(iii) *Up to height 1.50 m*

$A = 14.905 + [(17.156 + 19.156)\ /2] \times 0.5 = 23.983\ m^2$

$P = 17.539 + 2\sqrt{0.5^2+1^2} = 19.775\ m$

$R = 23.983/19.775 = 1.213\ m$

$V = (1.213)^{2/3} \times (1/12000)^{1/2}\ /\ 0.017 = 0.611\ m/sec$

$Q = 23.983 \times 0.611 = 14.648$ cumecs

(iv) *Up to height 2.00 m*

$A = 23.983 + [(19.156 + 21.156)\ /2] \times 0.5 = 34.061\ m^2$

$P = 19\text{-}77.5 + 2\sqrt{(0.5)^2+1^2} = 22.011\ m$

$R = 34.061/22.011 = 1.547\ m$

$V = (1.547)^{2/3} \times (1/12000)^{1/2}\ /0.017 = 0.718\ m/sec$

$Q = 34.061 \times 0.718 = 24.470$ cumecs

(v) *Up to height 2.50 m*

$A = 34.061 + [(21.156 + 23.156) / 2] \times 0.5 = 45.139\ m^2$

$P = 22.011 + 2\sqrt{(0.5)^2 + 1^2} = 24.247\ m$

$R = 45.139/24.247 = 1.862\ m$

$V = (1.862)^{2/3} \times (1/12000)^{1/2} /0.017 = 0.813\ m/sec$

$Q = 45.139 \times 0.813 = 36.686$ cumecs

(vi) *Up to height 3.00 m*

$A = 45.139 + [(23.156 + 25.156) /2] \times 0.5 = 57.217\ m^2$

$P = 24.247 + 2\sqrt{(0.5)^2 + 1^2} = 26.4832\ m$

$R = 57.217/26.2483 = 2.161\ m$

$V = (2.161)^{2/3} \times (1/12000)^{1/2} /0.017 = 0.897\ m/ sec$

$Q = 57.217 \times 0.87 = 51.348$ cumecs

(vii) *Up to height 3.50 m*

$A = 57.217 + [(25.156 + 27.156) /2] \times 0.5 = 70.295\ m^2$

$P = 26.483 + 2\sqrt{(0.5)^2 + 1^2} = 28.719\ m$

$R = 70.295/28.719 = 2.448\ m$

$V = (2.448)^{2/3} \times (1/12000)^{1/2} /0.017 = 0.975\ m/sec$

$Q = 70.295 \times 0.975 = 68.557$ cumecs

(viii) *Up to height 4.00 m*

$A = 70.295 + [(27.156 + 29.156) /2] \times 0.5 = 84.373\ m^2$

$P = 28.719 + 2\sqrt{(0.5)^2 + 1^2} = 30.955\ m$

$R = 84.373/30.955 = 2.726\ m$

$V = (2.726)^{2/3} \times (1/12000)^{1/2} /0.017 = 1.048\ m/sec$

$Q = 84.373 \times 1.048 = 88.405$ cumecs

(ix) *Up to height 4.35 m*

$A = 84.373 + [(29.156 + 30.556)/2] \times 0.35 = 94.823\ m^2$

$P = 30.955 + 2\sqrt{(0.35)^2 + (0.7)^2} = 32.520\ m$

$R = 94.823/32.520 = 2.916\ m$

$V = (2.916)^{2/3} \times (1/12000)^{1/2} / 0.017 = 1.096\ m / sec.$

$Q = 94.823 \times 1.096 = 103.924$ cumecs

(x) *Up height 4.5 0 m*

$A = 84.373 + [(29.156 + 31.156)] / 2 \times 0.5 = 99.451 \text{ m}^2$

$P = 30.955 + 2\sqrt{(0.5)^2 + 1^2} = 33.191\text{m}$

$R = 99.451/33.191 = 2.996 \text{ m}$

$V = (2.996)^{2/3} \times (1/12000)^{1/2} / 0.017 = 1.116 \text{ m/sec}$

$Q = 99.451 \times 1.116 = 110.993$ cumecs

(xi) *Up to height 4.60 m*

$A = 99.451 + [(31.156 + 31.556)/2] \times 0.1 = 102.587 \text{ m}^2$

$P = 33.191 + 2\sqrt{(0.2)^2 + (0.1)^2} = 33.638 \text{ m}$

$R = 102.587/33.638 = 3.050 \text{ m}$

$V = (3.050)^{2/3} \times (1/12000)^{1/2} / 0.017 = 1.129\text{m / sec.}$

$Q = 102.587 \times 1.129 = 115.850$ cumecs

(xii) *Up to height 4.80m*

$A = 99.451 + [(31.156 + 31.756) / 2] \times 0.3 = 108.888 \text{ m}^2$

$P = 33.191 + 2\sqrt{(0.3)^2 + (0.6)^2} = 34.533 \text{ m}$

$R = 108.888/34.533 = 3.153 \text{ m}$

$V = (3.153)^{2/3} \times (1/12000)^{1/2} / 0.017 = 1.155 \text{ m/sec}$

$Q = 108.888 \times 1.155 = 125.730$ cumecs

(xiii) *Up to height 5.00 m*

$A = 99.451 + [(31.156 + 33.156) / 2] \times 0.5 = 115.529 \text{ m}^2$

$P = 33.191 + 2\sqrt{(0.5)^2 + 0.1^2} = 35.427 \text{ m}$

$R = 115.529 / 35.427 = 3.261 \text{ m}$

$V = (3.261)^{2/3} \times (1/12000)^{1/2} / 0.017 = 11.81 \text{ m/sec}$

$Q = 115.529 \times 1.181 = 136.424$ cumecs

xiv) *Up to height 5.27 m*

$A = 115.529 + [(33.156 + 34.236) / 2] \times 0.27 = 124.627 \text{ m}^2$

$P = 35.427 + 2\sqrt{(0.27)^2 + 0.54^2} = 36.634 \text{ m}$

$R = 124.627 / 36.634 = 3.402 \text{ m}$

$V = (3.402)^{2/3} \times (1/12000)^{1/2} / 0.017 = 1.215$ m/sec

$Q = 124.627 \times 1.215 = 151.37$ cumecs

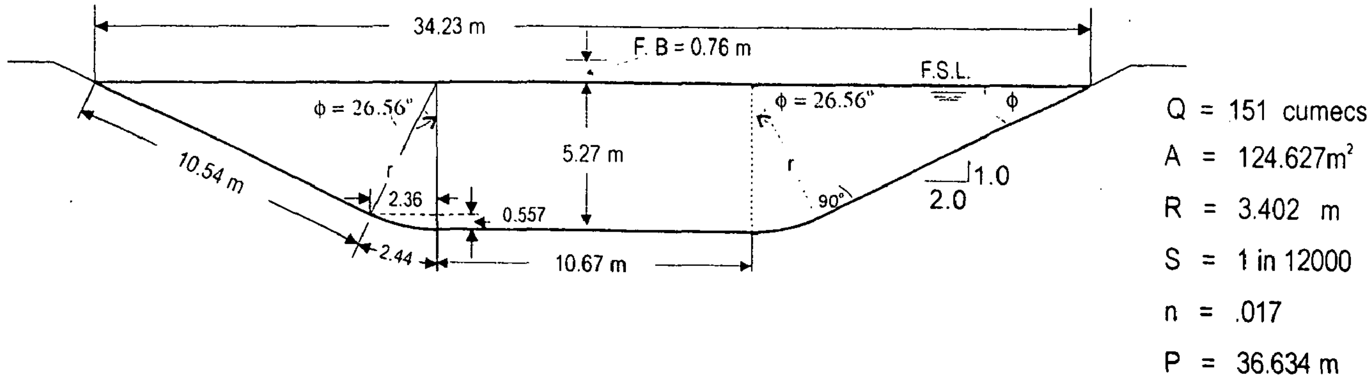
All these data are tabulated in Table 3.3 and plotted in Figure 3.6 which gives discharge rating curves. From this table, discharge can be read for any gauge (stage / depth of flow) or vice versa. This table is further used in development of backwater curve

Table : 3.3.

Flow Characteristics of Indira Gandhi Main Canal from Km 384 to 410 Km (RD 1260 to1345) for Uniform Flow, Project Design

S.No.	Depth of Flow Above Bed Y (m)	Top Width T (m)	Cross-Sectional Area of Flow A (m^2)	Wetted Perimeter P (m)	Hydraulic Radius R = A/P (m)	Velocity V (m/sec)	Discharge Q = A.V (cumecs)	Froude Number $F_r = V/\sqrt{gA/T}$ No.
1	2	3	4	5	6	7	8	9
1.	0.557	15.384	7.714	15.558	0.496	0.336	2.596	0.1515
2.	1.000	17.156	14.905	17.539	0.850	0.482	7.182	0.1651
3.	1.500	19.156	23.983	19.775	1.213	0.611	14.698	0.1743
4.	2.000	21.156	34.061	22.011	1.547	0.718	24.470	0.1807
5.	2.500	23.156	45.139	24.247	1.862	0.813	36.686	0.1860
6.	3.000	25.156	57.217	26.483	2.161	0.897	51.348	0.1899
7.	3.500	27.156	70.295	28.719	2.448	0.975	68.557	0.1935
8.	4.000	29.156	84.373	30.955	2.726	1.048	88.405	0.1967
9.	4.500	31.156	99.451	33.191	2.996	1.116	110.993	0.1994
10.	5.000	33.156	115.529	35.427	3.261	1.181	136.429	0.2020
11.	5.270 (Full supply depth)	39.230	124.627	36.634	3.402	1.215	151.375 (Full supply discharge)	0.2033

Note : For detailed calculation see para 3.5, development of flow rating curve of Indira Gandi Main Canal at Km 410.



Cross Section of I.G.M.N. between Km. 384 to Km.410 (R.D. 1260 - 1345), Project Design

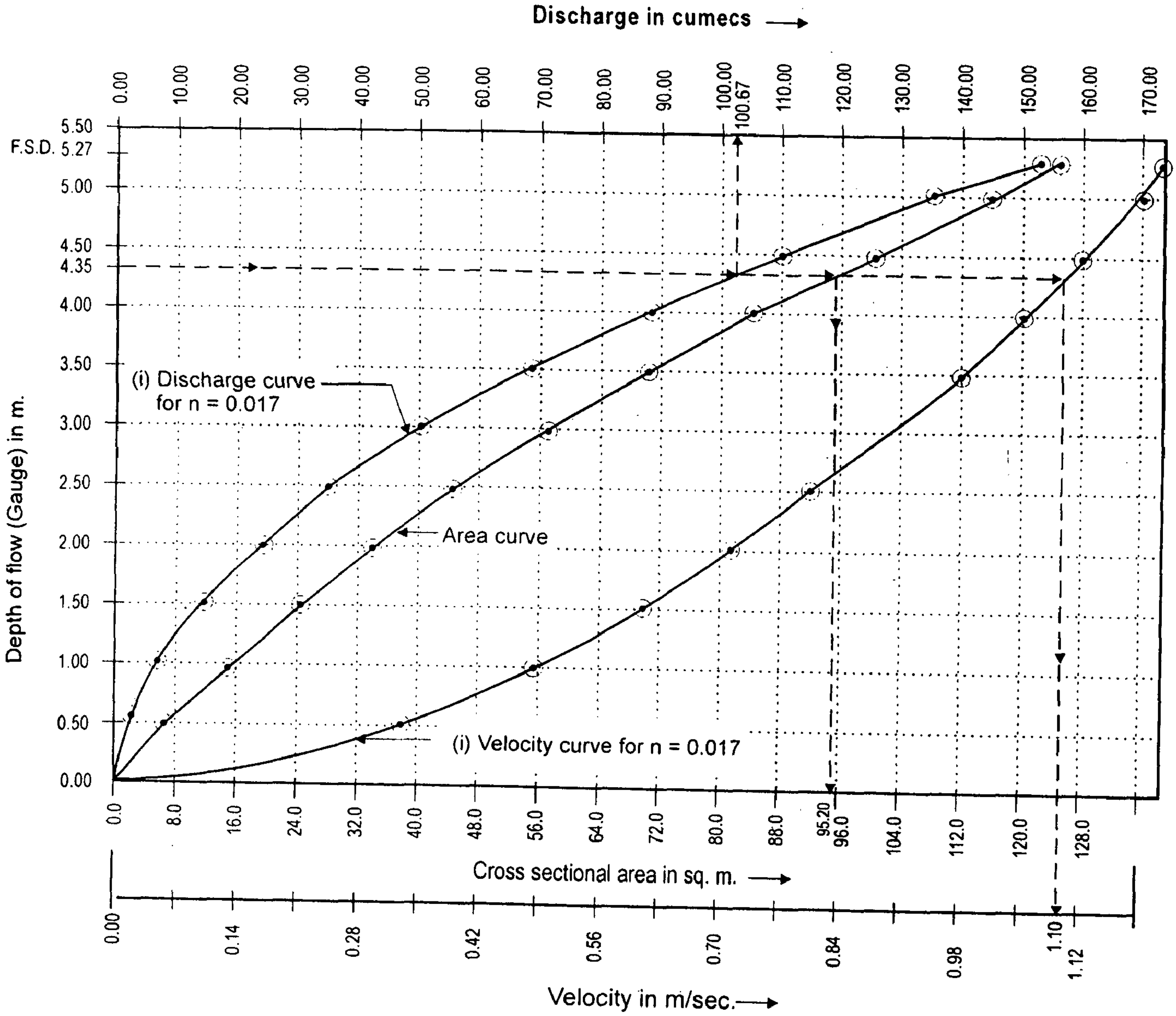


Figure - 3.6 Canal Rating Curve or Flow Characteristics in Trapezoidal Channel With Curved ends, Indra Gandhi Main Canal between Km. 384 to Km. 410 (RD 1260 to RD 1345), Project Design

Notes : To know discharge for a given gauge move arrow horizontally to meet discharge curve and then vertically upwards and read discharge. Similarly for area move upto area curve and down wards to read area scale. Similarly for velocity , move upto velocity curve and than down wards to read velocity scale.

Table 3.4

Indira Gandhi Main Canal, Computation of the Flow Profile Upstream of Cross Regulator at Km 410 (RD 1345), [Back Water Profile] for Gradually Varied Flow, for Q = 2/3 Full Supply Discharge = 100.67 cumec , n = 0.017, S_0 = 1/12000, α = 1,00 Project Design

S. No.	Depth Above Bed, h	Cross-Sectional Area of Flow, A	Hydraulic Radius, R=A/P	$R^{4/3}$	Velocity V=Q/A (100.63) / Col. 3	$\alpha v^2/2g$	Specific Energy, E	ΔE	S_f $\left(\frac{n^2V^2}{R^{4/3}}\right)$	$\overline{S}_f$	$S_0 - \overline{S}_f$	Δx, up Stream of Regulator	X, up Stream of Regulator (Distance)
	(m)	(m^2)	(m)	($m^{4/3}$)	(m /s)	(m)	(m)	(m)				(m)	(m)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
1.	5.27	124.627	3.402	5.117	0.808	0.033	5.303	-	0.0000369	-	-	-	-
2.	5.00	115.529	3.261	4.836	0.871	0.039	5.039	0.264	0.0000453	0.0000411	0.0000422	6,256	6,256
3.	4.80	108.888	3.153	4.623	0.925	0.044	4.844	0.195	0.0000535	0.0000494	0.0000339	5,752	12,008
4.	4.60	102.587	3.050	4.423	0.981	0.049	4.649	0.195	0.0000629	0.0000582	0.0000251	7,769	19,777
5.	4.35	94.823	2.916	4.166	1.062	0.061	4.407	0.242	0.0000782	0.0000706	0.0000127	19,055	38,832

Explainatory Note:

1. Δx, h and V are plotted in figure 3.7
2. Explainatory notes are given on next page

Explainatory notes to Table 3.4 :

Col.2 Depth of flow above bed is m, arbitrarily assigned from 5.27 o 4.35 m.

Col.3 Water area in m^2 corresponding to the depth y in col.2, where A=94.667 taken from canal rating at discharge curve, Figure 3.6.

Col.4 Hydraulic radius from Table 3.3 in m corresponding to y in col.2.

Col.5 Four – thirds power of the hydraulic radius (R), i.e $R^{4/3}$; row (1), $R^{4/3} = (3.402)^{4/3} = 5.117\ m^{4/3}$

Col.6 Mean velocity in m/s obtained by dividing 100.67 cumecs by the water area in col.3 = Q/A ; for row (1), V = 100.67/124.627 = 0.808 m/s

Col.7 Velocity head in m, $\alpha V^2/2g$; for row (1), $\alpha\ V^2 / 2g = (1 \times 0.808^2)/(2 \times 9.81) = 0.033$ m

Col.8 Specific energy in m obtained by adding the velocity head in col.7 to the depth of flow in col.2 i.e col. 2 + col. 7; row (1), E = 5.27 + 0.033 = 5.303 m

Col.9 Change of specific energy in m, equal to the difference between the E value in col.8 and that at the previous step. i.e.row (2)-(1) and son, row (2) : $\Delta E = 5.303 – 5.039 = 0.264$ m

Col.10 Friction slope computed by $n^2V^2 / R^{4/3}$ with n = 0.017 and with V as given in col.5 and $R^{4/3}$ in col.5. row (1) ; $S_f = (0.017^2 \times 0.808^2) / 5.117 = 0.0000369$

Col.11 Average friction slope between the steps, equal to the arithmetic mean of the friction slope just computed in col.10. and that of the previous step i.e row (1) ; row (2) : $S_f = (0.0000369 + 0.0000453) / 2 = 0.0000411$

Col.12 Difference between the bottom slope 0.00008 and the average friction slope ; row (2) : $S_0 – S_f = 0.0000833 – 0.0000411 = 0.0000422$

Col.13 Length of the reach in m between the consecutive steps, by dividing the value of ΔE in col.9. by the value in col.12 ; row (2) : $\Delta X = 0.264 / 0.0000422 = 6.256$ m

Col.14 Distance from the section under consideration to the gate site (upstream from regulator) ; row (2) : X = 6,256 + 0 = 6.256 = 6,256 m

Inference :

With 2/3 discharge i.e. 151x2/3 = 100.67m^3/s. the depth for uniform flow is 4.35 m. In order to maintain FSL at the cross regulator, it is partially closed. The water level at upstream of the regulator is maintained at FSL (corresponding to 151 m^3/s) and there is a gradually varied flow in a length of 38.8 Km (known as back water curve). The velocity at uniform flow for 100.67m^3/s is 1.09 m/s and it goes on reducing in the zone of back water till the regulator to 0.808 m/s. See Figure 3.7 therefore this zone shows a higher rate of silting, see para 3.6.1 and 3.6.2.

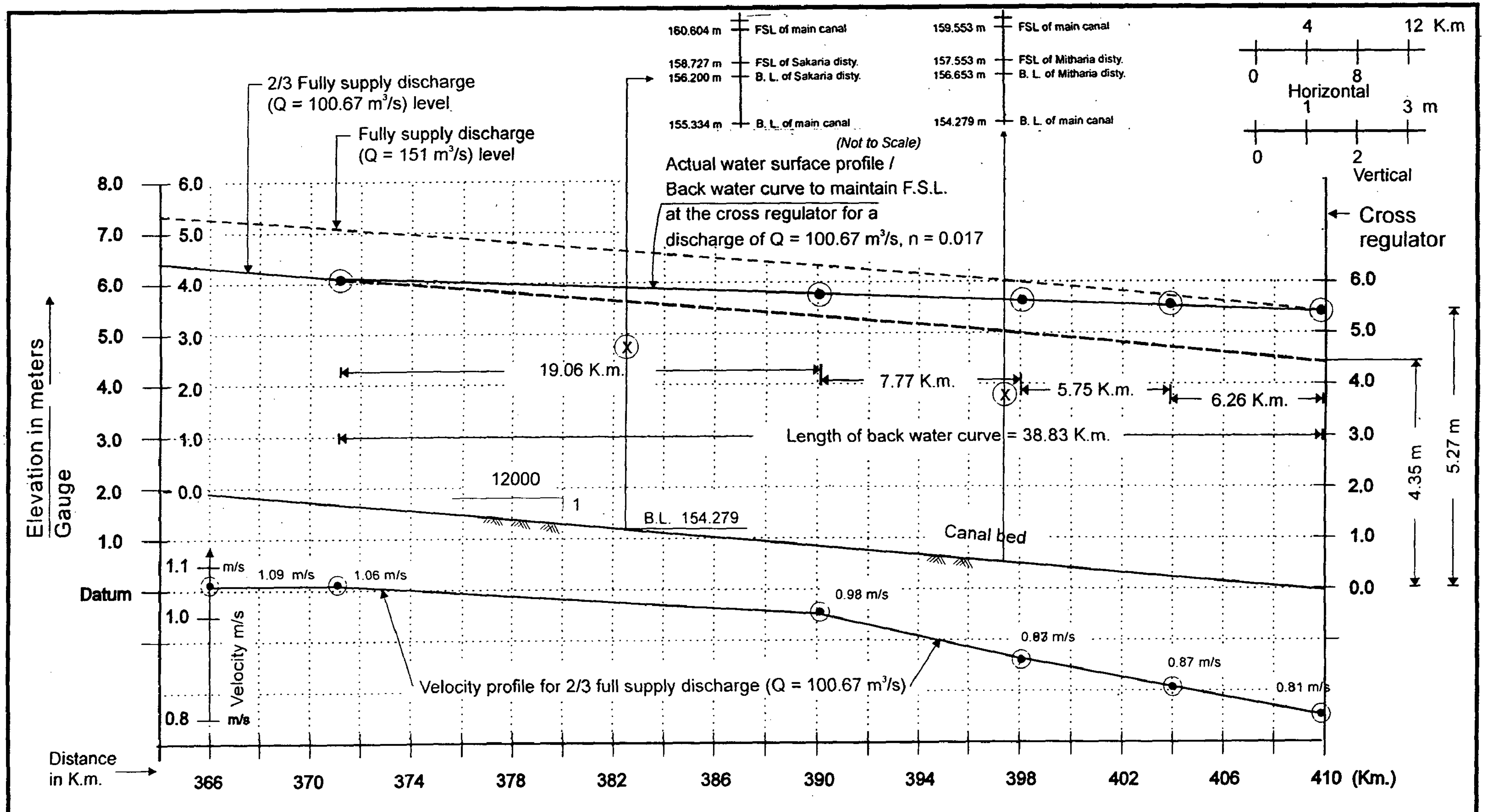


Figure 3.7 Velocity and Back Water Surface Profile at Cross Regulator at Km. 410 (RD 1345.13) To Maintain FSL For 2/3 Discharge in Indra Gandhi Main Canal (IGMC) For Existing / Project Design

Roorkee.cdr

3.6 SEDIMENT IN FLOW AND ITS TRANSPORT

Lot of sediment (fine sand for classification, see Figure 3.8) enters in the entire length of the canal, through dust storms a peculiar phenomena of the desert area. General typical cross - section shows, Figure 3.4 and Figure 3.5, shows that the canal passes through heavy cutting and filing. The entire alignment is in alternate heavy cutting and heavy filling, again a peculiar phenomena of desert dune areas. More sediment enters in heavy cutting and adjoining filling reaches.

Though efforts of plantations are being made in the area, yet the magnitude of reduction in sediment is not certain. Detailed observation of sediment are not available. The aspect of sediment transport is not available in the revised project report (Revised Project Estimate Volume I, 1993). Here an effort is made to know the sediment transporting capacity of channel.

Temperature

Indira Gandhi canal suffer extreme of temperature. Winter is quite cold and at places the mean minimum temperature is normally recorded in the month of January and varies from 4.7 to 7.9^{0}C. The hottest months are from April to September with the peak temperature being mostly in the month of May when the mean maximum temperatures vary from 41.5 to 42.0^{0}C.

Wind

The general wind direction in the region is southwest. The wind speed remains highest in Bikaner and Phalodi and Jaisalmer through out the year and gradually decreases as one moves towards North-East. Dust storm are very common during summer when hot winds prevail. Maximum number of dust storms occur in April to June. Due to poor rainfall, humidity, in this tract is extremely low, sand storms are of frequent occurrence during major part of the year. There is very little vegetation, which could stabilize the sandy soil.

3.6.1 Total Sediment Transport by Einstein Method, for Uniform Flow between Km 384 to Km 410 of IGMC At 20⁰C of Water, Medium Season - March, July to November, for Project Designed Section.

Project Data :

(i) Channel parameters:

Discharge Q = 151.375 m/s ; Bed width, B = 10.67 m; Top width , B_T = 34.23 m

Full supply depth, D = 5.27 m : Bed slope, S_0 = 1/12000

Average bed width, $B_a = \frac{B + B_T}{2} = \frac{10.67 + 34.23}{2} = 22.45$ m

Flow characteristic of uniform flow are given in Table 3.3 and Figure 3.6

(ii) Water properties

Temperature of water, T = 20^0 C ; Density at 20^0 C, ρ = 998.2 Kg/m^3

(see Annexure -1)

Specific weight of fluid, γ_f = 1Ton/m^3 ; Viscosity at 20^0 C, ν = 1.003 x 10^{-6} m/s

(see Annexure –1)

(iii) Sediment properties:

Diameter of sediment, d_{35} = 0.145 mm, d_{50} = 0.15 mm, d_{65} = 0.16 mm

(from Figure 3.8)

Density of solid particle, ρ_s = 2650 Kg/m^3 ; Specific weight of sediment,

γ_s = 2.65 Ton/m^3 ; Settling velocity, v_{ss} (d_{35}) = 0.017 m/s (from Figure 2.28)

(iv) Computations of sediment transport

Sediment transport is calculated by Einstein's method given at 2.6.1 in Table 2.6, for different depths of flow in the main canal. The depth of flow will change with the change in discharge. The computations are done in a tabular manner as given in Table 3.5. Explanatory notes for calculations are also given below the Table. The total sediment transport (discharge) is plotted in Figure 3.9 versus gauge and also shown against corresponding discharge.

Table : 3.5

Indira Gandhi Main Canal, Computation of Stage – Solid – Discharge Curve at 20 ^{0}C, by Einstein Method.

S. No	Flow Depth h (m)	Hydraulic Radius R (m)	Shear Velocity V_* (m/s)	Thickness of Viscous sub layer δ (m)	Relative Rough ness k_s / δ	Correcti on Term X	Apparent Roughness Diameter Δ (m)	Transport Parameter Pe	Intensity of Shear ψ_*	Intensity of Transpo rt Φ_*	Bed Load Transport: Volume by unit width q_{sb} ($m^3/s/m$)	Bed Load Transport: Volume, Q_{sb} (m^3/s)	Bed Load Transport: Volume Q_{sb} (m^3/day
1	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
1	0.557	0.496	0.020	$5.767x10^{-4}$	0.277	0.600	$2.667x10^{-4}$	11.054	6.029	0.40	$2.810x10^{-6}$	$6.308x10^{-5}$	5.450
2	1.000	0.850	0.026	$4.436x10^{-4}$	0.361	0.673	$2.377x10^{-4}$	11.754	3.518	1.50	$1.054x10^{-5}$	$2.366x10^{-4}$	20.442
3	1.500	1.213	0.031	$3.721x10^{-4}$	0.430	0.764	$2.094x10^{-4}$	12.287	2.465	2.50	$1.756x10^{-5}$	$3.942x10^{-4}$	34.059
4	2.000	1.547	0.035	$3.296x10^{-4}$	0.485	0.891	$1.796x10^{-4}$	12.728	1.933	3.50	$2.459x10^{-5}$	$5.521x10^{-4}$	47.701
5	2.500	1.862	0.038	$3.035x10^{-4}$	0.527	0.964	$1.660x10^{-4}$	13.030	1.606	4.71	$3.309x10^{-5}$	$7.429x10^{-4}$	64.187
6	3.000	2.161	0.041	$2.813x10^{-4}$	0.569	1.018	$1.572x10^{-4}$	13.267	1.384	6.00	$4.215x10^{-5}$	$9.463x10^{-4}$	81.760
7	3.500	2.448	0.044	$2.621x10^{-4}$	0.610	1.055	$1.517x10^{-4}$	13.457	1.222	6.50	$4.566x10^{-5}$	$1.025x10^{-3}$	88.560
8	4.000	2.726	0.046	$2.508x10^{-4}$	0.638	1.127	$1.420x10^{-4}$	13.656	1.097	7.50	$5.269x10^{-5}$	$1.183x10^{-3}$	102.211
9	4.500	2.996	0.048	$2.403x10^{-4}$	0.666	1.182	$1.354 x10^{-4}$	13.822	0.998	7.75	$5.444x10^{-5}$	$1.222x10^{-3}$	105.581
10	5.000	3.261	0.051	$2.262x10^{-4}$	0.707	1.236	$1.295x10^{-4}$	13.972	0.917	8.75	$6.147x10^{-5}$	$1.380x10^{-3}$	119.232
11	5.270	3.402	0.052	$2.218x10^{-4}$	0.721	1.255	$1.275x10^{-4}$	14.040	0.879	9.00	$6.322x10^{-5}$	$1.419x10^{-3}$	122.631

Table 3.5 Continued

S. No.	Dimension less Height A_E	Rouse Exponent	Integral I_1	Integral I_2	Suspended Load Transport: Volume by Unit Weight q_{ss} ($m^3/s/m$)	Suspended Load Transport: Volume Q_{ss} (m^3/s)	Suspended Load Transport: Volume Q_{ss} (m^3/day)	Total Sediment Transport: Volume Q_s (m^3/s)	Total Sediment Transport: Volume Q_s (m^3/day)	Total Sediment Transport: By Mass G_s (kg/s)	Total Sediment Transport: By Mass G_s (ton/day)
(15	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)	(26)
1	$5.207x10^{-4}$	2.125	0.182	-1.667	$9.700x10^{-6}$	$2.175x10^{-5}$	1.879	$8.483x10^{-5}$	7.329	0	0
2	$2.900x10^{-4}$	1.635	0.380	-3.000	$1.546x10^{-5}$	$3.471x10^{-4}$	29.989	$5.837x10^{-4}$	50.432	2	173
3	$1.933x10^{-4}$	1.371	0.600	-3.500	$6.800x10^{-5}$	$1.527x10^{-3}$	131.933	$1.921x10^{-3}$	165.974	5	432
4	$1.450x10^{-4}$	1.214	0.900	-5.000	$1.587x10^{-4}$	$3.563x10^{-3}$	307.843	$4.115x10^{-3}$	355.536	11	950
5	$1.160x10^{-4}$	1.118	1.333	-7.250	$3.348x10^{-4}$	$7.516x10^{-3}$	649.382	$8.258x10^{-3}$	713.491	22	1901
6	$9.667\ x10^{-5}$	1.037	1.750	-9.000	$5.993\ x10^{-4}$	$1.345x10^{-2}$	1162.080	$1.440x10^{-2}$	1244.160	38	3283
7	$8.286x10^{-5}$	0.966	2.500	-12.000	$9.882\ x10^{-4}$	$2.219x10^{-2}$	1917.216	$2.322x10^{-2}$	2006.208	62	5357
8	$7.250x10^{-5}$	0.924	3.250	-15.000	$1.548x10^{-3}$	$3.475x10^{-2}$	3002.400	$3.593x10^{-2}$	3104.352	95	8208
9	$6.444x10^{-5}$	0.885	4.150	-16.250	$2.238x10^{-3}$	$5.025x10^{-2}$	4341.600	$5.147x10^{-2}$	4447.008	136	11,783
10	$5.800x10^{-5}$	0.833	5.100	-20.000	$3.151x10^{-3}$	$7.074x10^{-2}$	6111.936	$7.212x10^{-2}$	6231.168	191	16,513
11	$5.503x10^{-5}$	0.817	5.500	-22.000	$3.491x10^{-3}$	$7.837x10^{-2}$	6771.168	$7.979x10^{-2}$	6893.856	211	18,269

Conclusion: Maximum Sediment discharge capacity at full supply discharge of 151.375 m^3/s = **6893.856 m^3/ day**
Explanatory notes are given on next page

Explainatory notes to Table 3.5

Col.2.	h	Flow depth (from Table 3.3)
Col.3.	R	Hydraulic radius (from Table 3.3)
Col.4.	V_*	Friction velocity, $V_* = \sqrt{gRS_0}$
Col.5.	δ	Thickness of viscous sub layer, $\delta = 11.5\ \nu/V_*$; row (1) ; $\delta = 11.5\ (1.003\text{x}10^{-6}/0.02)$ $= 5.767\text{x}10^{-4}$ m
Col.6.	k_s/δ	Relative roughness, $k_s/\delta = d_{65}/\delta$ row (1) ; $k_s/\delta = 1.45\text{x}10^{-4} / 5.767\text{x}10^{-4} = 0.277$
Col.7.	X	Correction term for logaritmatic velocity distribution (from Fig 2.27)
Col.8.	Δ	Apparent roughness diameter, $\Delta = d_{65}/X$, row (1) ; $\Delta = 1.6\text{x}10^{-4}/0.6 = 2.667\text{x}10^{-4}$ m
Col.9.	Pe	Transport parameter, $P_e = 2.203 \log (30.2\ h/\Delta)$, row (1) ; $P_e = 2.203 \log (30.2\text{x}\ 0.557/2.667\ \text{x}\ 10^{-4}) = 11.054$
Col.10.	ψ_*	Intensity of shear, $\psi_* = (\gamma_s - 1)d_{35}/RS_0$, row (1); $[2.65-1)\text{x}\ 1.45\text{x}\ 10^{-4}]/(0.496\text{x}0.00008) = 6.029$
Col.11.	ϕ_*	Intensity of transport, $\phi_* = f(\psi_*)$, (from Figure 2.26)
Col.12	q_{sb}	Solid discharge,as bed load, by volume and per unit width, $q_{sb} = \phi_* \sqrt{(\gamma_s - 1)\ g d_{35}^3}$, row (1) $= 0.4\text{x}\sqrt{(2.65-1)\ \text{x}\ 9.81\ \text{x}\ (1.45\ \text{x}\ 10^{-4})^3} = 2.810\text{x}10^{-6}\ m^3/s/m$
Col.13.	Q_{sb}	Solid discharge, as bed load, by volume, row (1); $Q_{sb} = 2.810\ \text{x}\ 10^{-6}\ \text{x}\ 22.45 = 6.308\ \text{x}\ 10^{-5}\ m^3/s$
Col.14.	Q_{sb}	Q_{sb} x 12 x 3600 x 24 in tons/day, row (1) ; $Q_{sb} = 6.308\ \text{x}\ 3.600\ \text{x}\ 24 = 5.450\ m^3/day$
Col.15.		S.No
Col.16.	A_E	Dimensionless height, $A_E = 2d_{35}/h$, row (1) ; $A_E = (2\ \text{x}\ 1.45\ \text{x}\ 10^{-4})/0.557 = 5.207\ \text{x}\ 10^{-4}$

Col.17. z Rouse exponent, $z = V_{ss} / KV_*$,

V_{ss} Settling velocity (from Figure 2.28), row (1) ;

$z = 0.017/(0.4 \times 0.02) = 2.125$

Col.18. I_1 Einstein's first integral, (from Figure 2.17)

Col.19. I_2 Einsten's second integral, (from Figure 2.17)

Col.20. q_{ss} Solid discharge, as suspended load, by volume and by unit width,

$q_{ss} = q_{sb} (P_e I_{2+} I_2)$, row (1) ;

$q_{ss} = 2.810 \times 10^{-6} (11.054 \times 0.182) - 1.667 = 9.7 \times 10^{-6}\ m^3/s/m$

Col.21. Q_{ss} Solid discharge, as suspended load, by volume, $Q_{ss} = q_{ss} \times B_m$, row (1) ;

$Q_{ss} = 9.7 \times 10^{-6} \times 22.45 = 2.175 \times 10^{-5}\ m^3/s$

Col.22. Q_{ss} Solid discharge, as total load, by volume, row (1);

$Q_{ss} = 2.175 \times 10^{-5} \times 3600 \times 24 = 1.879\ m^3/day$

Col.23. Q_{ss} Solid discharge, as total load, by volume, $Q_s = Q_{sb} + Q_{ss}$, row (1) ;

$Q_s = 6.308 \times 10^{-5} + 2.175 \times 10^{-5} = 8.483 \times 10^{-5}\ m^3/s$

Col.24. Q_s Solid discharge, as total load, by volume, col. 23 x 3600 x 24, row (1) ;

$Q_s = 8.483 \times 3600 \times 24 = 7.329\ m^3/day$

Col.25. G_s Solid discharge, as total load, by mass, $G_s = Q_s \times \rho_s$, row (1) ;

$G_s = 8.483 \times 10^{-5} \times 2650 = 0.00008 = 0$ Kg/s

Col.26. G_s G_s x 3600 x24 in tons/day, row (1) ;

$G_s = 0 \times 3600 \times 24 \times 10^{-3} = 0$ tons/day

3.6.2 Total Sediment Transport by Ackers Method, for Uniform Flow between Km 384 to Km 410 of IGMC At 32^0 C of Water, Very Hot Season, April, May and June, for Project Designed Section

Project Data :

(i) Channel parameters:

Discharge Q = 151.375 m/s ; Bed width, B = 10.67 m ; Top width , B_T = 34.23m

Full supply depth , D = 5.27 m ; Bed slope, S_0 = 1/12000

$$\text{Average bed width } B_a = \frac{B + B_T}{2} = \frac{10.67 + 34.23}{2} = 22.45 \text{ m}$$

Flow characteristic of uniform flow are given in Table 3.3 and Figure 3.6

(ii) Water properties

Temperature of water, $T = 32^0$ C ; Density at 32^0 C, $\rho = 994.923$ Kg/m^3 (see Annexure -1)

Specific weight of fluid, $\gamma_f = 1$ Ton/m^3; Viscosity at 32^0 C, $\nu = 0.768 \times 10^{-6}$ m^2/s (see Annexure –1)

(iii) Sediment properties:

Diameter of sediment, $d_{35} = 0.145$ mm , $d_{50} = 0.15$ mm , $d_{65} = 0.16$ mm (from Figure 3.8); Density of solid particle, $\rho_s = 2650$ Kg/m^3 ; Specific weight of sediment, $\gamma_s = 2.65$ Ton/m^3 ; Settling velocity , $V_{ss}(d_{35}) = 0.017$ m/s (Figure 2.28)

(iv) Computations : (from at 2.6.9 in Table 2.6 Ackers and White)

Dimensionless diameter of grain D_{gr} ,Eq. (x) $= d_{35}\left[\dfrac{g(\gamma_s - 1)}{\nu^2}\right]^{1/3}$

$$= 1.45 \times 10^{-4}\left[\frac{9.81(2.65-1)}{(0.768\text{x}10^{-6})^2}\right]^{1/3}$$

$$= 4.3737$$

Parameter A, Eq. (vii) $= \dfrac{0.23}{\sqrt{D_{gr}}} + 0.14 = \dfrac{0.23}{\sqrt{4.3737}} + 0.14 = 0.2500$

Parameter N, Eq (vi) $= 1.00 - 0.56 \log D_{gr} = 1.00 - 0.56 \log 4.3737 = 0.6411$

Parameter m, Eq. (viii) $= 9.66/D_{gr} + 1.34 = 9.66/4.3737 + 1.34 = 3.5487$

Parameter C, Eq. (ix) $\log C = 2.86 \log D_{gr} - (\log D_{gr})^2 - 3.53$

$$\text{Log } C = 2.86 \log 4.3737 - (\log 4.3737)^2 - 3.53 = -2.1079$$

$$\text{or } C = 0.0078$$

Sediment transport computation for different discharges, according to flow rating curve given in Figure are done in a tabular manner given in Table 3.6

Table : 3.6

Indira Gandhi Main Canal, Computation of Stage – Sediment Discharge Curve at 32^0C for Uniform Flow between Km 384 to Km 410 for Project Section by ACKERS METHOD

S. No.	Depth Above Bed h (m)	Hydraulic Radius R (m)	Shear Velocity V_* (m/s)	Velocity V (m/s)	Discharge Q (m^3/s)	Parameter of Mobility F_{gr}	Transport Parameter G_{gr}	Concentration by Volume C_s	Total Sediment Transport: Concentration by Volume, C_s x 10^6 (ppm)	Total Sediment Transport: Volume Q_s (m^3/s)	Total Sediment Transport: Volume Q_s (m^3/day)	Total Sediment Transport: By Mass. Gs (kg/s)	Total Sediment Transport: By Mass G_s Tons /day
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
1.	0.557	0.456	0.020	0.336	2.596	0.353	3.353 x (10^{-4})	5.300 x (10^{-7})	0.530	1.38 x (10^{-6})	0.119	0	0
2.	1.000	0.850	0.026	0.482	7.182	0.467	4.720 x (10^{-3})	4.450 x (10^{-6})	4.450	3.196 x (10^{-5})	2.761	0	0
3.	1.500	1.213	0.031	0.611	14.648	0.561	1.693 x (10^{-2})	1.107 x (10^{-5})	11.070	1.622 x (10^{-4})	14.014	0	0
4.	2.000	1.547	0.035	0.718	24.470	0.637	3.677 x (10^{-2})	1.849 x (10^{-5})	18.490	4.525 x (10^{-4})	39.096	1	10.4
5.	2.500	1.862	0.038	0.813	36.686	0.698	6.182 x (10^{-2})	2.555 x (10^{-5})	25.550	9.373 x (10^{-4})	80.983	2	215
6.	3.000	2.261	0.041	0.897	51.348	0.755	9.456 x (10^{-2})	3.304 x (10^{-5})	33.040	1.697 x (10^{-3})	146.621	4	389
7.	3.500	2.448	0.044	0.975	68.557	0.810	1.365 x (10^{-1})	4.122 x (10^{-5})	41.220	2.826 x (10^{-3})	244.166	7	647
8.	4.000	2.726	0.046	1.048	88.405	0.852	1.764 x (10^{-1})	4.744 x (10^{-5})	47.440	4.194 x (10^{-3})	362.362	11	960
9.	4.500	2.996	0.048	1.116	110.995	0.893	2.229 x (10^{-1})	5.399 x (10^{-5})	53.990	5.993 x (10^{-3})	517.795	16	1372
10.	5.000	3.261	0.051	1.181	136.424	0.945	2.937 x (10^{-1})	6.386 x (10^{-5})	63.860	8.712 x (10^{-3})	752.717	23	1995
11.	5.270	3.402	0.052	1.215	151.375	0.965	3.248 x (10^{-1})	6.739 x (10^{-5})	67.390	1.020 x (10^{-3})	881.280	27	2335

Conclusion: Sediment Discharge capacity at full supply discharge at 151.375 m^3/s at F.S.L. = 67.39 ppm = **881.28 m^3 / day**

Explainatory notes are given on next page

Explainatory notes to Table 3.6 :

Col.2 h Flow depth. (from Table 3.3)

Col.3 R Hydraulic radius (from Table 3.3)

Col.4 V_* Total shear velocity, $V_* = \sqrt{g\ R\ S_0}$, row (1);

$$V_* = \sqrt{9.81 \times 0.496 \times 0.00008} = 0.02 \text{m/s}$$

Col.5 V Average velocity (from Table 3.3)

Col.6 Q water flow (Liquid) discharge (from Table 3.3)

Col.7 F_{qr} Parameter of mobility,

$$F_{gr} = \frac{V_*}{\sqrt{(\gamma_s - 1)\ g\ d_{35}}} \left[\frac{V}{\sqrt{32}\ \log(10h/d_{35})}\right]^{(1-n)} \text{row (1)} ; \qquad F_{qr} =$$

$$\frac{(0.02)^{0.6411}}{\sqrt{(2.65-1) \times 9.81 \times 1.45 \times 10^{-4}}} \left[\frac{0.336}{\sqrt{32} \times \log(10 \times 0.557) / 1.45 \times 10^{-4}}\right]^{(1-0.6411)} = 0.353$$

Col.8 G_{gr} Transport parameters, $G_{gr} = C \left[\frac{Fgr}{A} - 1\right]^m$, row (1) ; G_{gr} = 0.0078

$$\left[\frac{0.353}{0.25} - 1\right]^{3.5487} = \qquad 3.353 \times 10^{-4}$$

Col. 9 G_s Concentration by Volume,

$$C_s = Ggr \frac{d_{35}}{h}\left(\frac{V}{V_*}\right)^n \text{, row (1); } C_s = 3.353 \times 10^{-4} \times \frac{1.45 \text{x} 10^{-4}}{0.557} \left[\frac{0.336}{0.02}\right]^{0.6411}$$

$$= 5.30 \times 10^{[-7}$$

Col.10 $C_s \times 10^6$, row (1) ; $C_s = 5.30 \times 10^{-7} \times 10^{-6} = 0.530$ ppm

Col.11 Q_s Solid discharge, as total load, by volume, $Q_s = C_s \times Q$, row (1) ; $Q_s = 5.30 \times 10^{-7} \times 2.596 = 1.38 \times 10^{-6}\ m^3/s$

Col.12 $Q_s \times 60 \times 60 \times 24$, row (1) ; $Q_s = 1.38 \times 10^{-6} \times 3600 \times 24 = 0.119\ m^3/day$

Col.13 G_s Solid discharge, is total load, by mass, $G_s = Q_s\ \rho_s$, row (1) ;
$G_s = 1.38 \times 10^{-6} \times 2650 = 0.0037 \approx 0$ Kg/s

3.6.3 Total Sediment Transport by Ackers Method, for Uniform Flow between Km 384 to Km 410 of IGMC at 20^0C of Water Medium Season, March, and July to November.

Project Data :

(i) Channel parameter :

Discharge Q = 151.375 m/s ; Bed width, B = 10.67m ; Top width , B_T = 34.23 m
Full supply depth, D = 5.27 m : Bed slope, S_0 = 1/12000

Average bed width, $B_a = \frac{B + B_T}{2} = \frac{10.67 + 34.23}{2} = 22.45$ m

Flow characteristic of uniform flow are given in Table 3.3 and Figure 3.6

(ii) Water properties:

Temperature of water, T = 20^0C; Density at 20^0 C, ρ = 998.2 Kg/m^3 (Annexure-1)

Specific weight of fluid, γ_f = 1.0 Ton/m^3 ; Viscosity at 20^0 C, ν = 1.003 x 10^{-6} m/s (see annexure –1)

(iii) Sediment properties:

Diameter of sediment, d_{35} = 0.145 mm, d_{50} = 0.15 mm, d_{65} = 0.16 mm (Figure 3.8)

Density of solid particle, ρ_s= 2650 Kg/m^3; Specific weight of sediment, γ_s= 2.65 Ton/m^3 ; Settling velocity, V_{ss} (d_{35}) = 0.017 m/s (from Figure 2.28)

(iv) Computations : (from at 2.6.9 in Table 2.6 Ackers and White)

Dimensionless diameter of grain D_{gr} ,Eq. (x) = $d_{35} \left[\frac{g(\gamma_s - 1)}{\nu^2}\right]^{1/3}$

$$= 1.45 \times 10^{-4} \left[\frac{9.81(2.65 - 1)}{(1.003x10^{-6})^2}\right]^{1/3} = 3.6606$$

Parameter A, Eq. (vii) $= \frac{0.23}{\sqrt{D_{gr}}} + 0.14 = \frac{0.23}{\sqrt{3.6606}} + 0.14 = 0.2602$

Parameter N, Eq (vi) = 1.00-0.56 log D_{gr} = 1.00 - 0.56 log 3.6606 = 0.6844

Parameter m, Eq. (viii) = 9.66/D_{gr}+1.34 = 9.66/3.6606 + 1.34 = 3.9789

Parameter C, Eq. (ix) log C = 2.86 log D_{gr} – (log D_{gr})^2-3.53

Log C = 2.86 log 3.6606 – (log 3.6606)^2 - 3.53 = - 2.2358

or C = 0.0058 .

Sediment transport computation for different discharges, according to flow rating curve given in Figure are done in a tabular manner given in Table 3.7

Table : 3.7

Indira Gandhi Main Canal, Computation of Stage – Sediment Discharge Curve at 20 °C for Uniform Flow between Km 384 to Km 410 for Project Section by ACKERS METHOD

S. No.	Depth Above Bed h (m)	Hydraulic Radius R (m)	Shear Velocity V_* (m/s)	Velocity V (m/s)	Discharge Q (m^3/s)	Parameter of Mobility F_{gr}	Transport Parameter G_{gr}	Concentration by Volume C_s	Total Sediment Transport: Concentration by Volume, $C_s \times 10^6$ (ppm)	Total Sediment Transport: Volume Q_s (m^3/s)	Total Sediment Transport: Volume Q_s (m^3/day)	Total Sediment Transport: By Mass Gs (kg/s)	Total Sediment Transport: By Mass G_s Tons /day
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
1.	0.557	0.496	0.020	0.336	2.596	0.369	$1.806 \times (10^{-4})$	$3.200 \times (10^{-7})$	0.324	$8.300 \times (10^{-7})$	0.072	0	0
2.	1.000	0.850	0.026	0.482	7.182	0.494	$3.789 \times (10^{-3})$	$4.050 \times (10^{-6})$	4.050	$2.909 \times (10^{-5})$	2.513	0	0
3.	1.500	1.213	0.031	0.611	14.648	0.601	$1.697 \times (10^{-2})$	$1.262 \times (10^{-5})$	12.620	$1.849 \times (10^{-4})$	15.975	0	0
4.	2.000	1.547	0.035	0.718	24.470	0.687	$4.155 \times (10^{-2})$	$2.382 \times (10^{-5})$	23.820	$5.829 \times (10^{-4})$	50.363	2	173
5.	2.500	1.862	0.038	0.813	36.686	0.756	$7.543 \times (10^{-2})$	$3.560 \times (10^{-5})$	35.800	$1.306 \times (10^{-3})$	112.838	3	259
6.	3.000	2.161	0.041	0.897	51.348	0.821	$1.231 \times (10^{-1})$	$4.916 \times (10^{-5})$	49.160	$2.524 \times (10^{-3})$	218.074	7	605
7.	3.500	2.448	0.044	0.975	68.557	0.885	$1.893 \times (10^{-1})$	$6.537 \times (10^{-5})$	65.370	$4.482 \times (10^{-3})$	387.245	12	1037
8.	4.000	2.726	0.046	1.048	88.405	0.933	$2.541 \times (10^{-1})$	$7.824 \times (10^{-5})$	78.240	$6.917 \times (10^{-3})$	597.629	18	1555
9.	4.500	2.996	0.048	1.116	110.995	0.980	$3.324 \times (10^{-1})$	$9.226 \times (10^{-5})$	92.260	$1.024 \times (10^{-2})$	884.736	27	2333
10.	5.000	3.261	0.051	1.181	136.424	1.040	$4.572 \times (10^{-1})$	$1.139 \times (10^{-4})$	113.900	$1.554 \times (10^{-2})$	1342.656	41	3542
11.	5.270	3.402	0.052	1.215	151.375	1.063	$5.132 \times (10^{-1})$	$1.220 \times (10^{-4})$	122.000	$1.847 \times (10^{-2})$	1595.614	49	4234

Conclusion: Sediment Discharge capacity at full supply discharge at 151.375 m^3/s at F.S.L. = **122.0 ppm = 1,595.6 m^3 / day**.
All additional sand due to dust storms will be deposited in the canal

Explanatory Notes: Same as in Table 3.6

Table : 3.8

Indira Gandhi Main Canal, Sediment Flow at 20^0C for Gradually Varied Flow between Km 384 to Km 410 for Project Section (existing) by ACKERS METHOD , for 2/3 Full Supply Discharge Q = 100.67 m^3/s , with FSL at Cross Regulator

Km Of Canal	Depth Above Bed	Hydraulic Radius	Shear Velocity	Velocity	Discharge	Parameter of Mobility	Parameter Transport	Concentration by Volume,	Concentration by Volume,	Total Sediment Transport			
										Volume	Volume	Mass	Mass
	h	R	V_*	V	Q	F_{gr}	C_{gr}	C_s	$C_s x10^6$	Q_s	Q_s	C_s	C_s
	(m)	(m)	(m/s)	(m/s)	(m^3/s)				(ppm)	(m^3/s)	(m^3/day)	(kg/s)	(t/day)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
390.22	4.60	2.916	0.0489	0.981	100.67	0.879	$1.822x10^{-1}$	$4.472x10^{-5}$	44.72	$4.502x10^{-3}$	388.873	12	1037
398.00	4.80	3.153	0.0497	0.925	100.67	0.871	$1.730 x10^{-1}$	$3.866x10^{-5}$	38.66	$3.892x10^{-3}$	336.260	10	864
403.74	5.00	3.262	0.0506	0.871	100.67	0.865	$1.663 x10^{-1}$	$3.382x10^{-5}$	33.82	$3.405x10^{-3}$	294.163	9	780
410	5.27	3.502	0.0520	0.808	100.67	0.859	$1.598 x10^{-1}$	$2.874x10^{-5}$	28.74	$2.894x10^{-3}$	250.004	8	663
(X-Regula tor													

Conslusion : In the zone of gradually varied flow upstream of cross –regulator, sediment transport is gradually decreasing from 389 m^3/day at Km 390.2 to 250 m^3/day at cross-regulator at Km 410. (in 20 Km). This reach of canal will silt at an additional rate of 139 m^3/day. During heavy dust storms when ever there is more sediment in the canal, only above rate will be transported and all balance will be deposited.

Explainatory notes : same as in Table 3.6

GRAIN SIZE DISTRIBUTION (sieve analysis)	
diameter (mm)	% finer
710	100
500	100
300	99
212	95
150	46
075	1
063	0

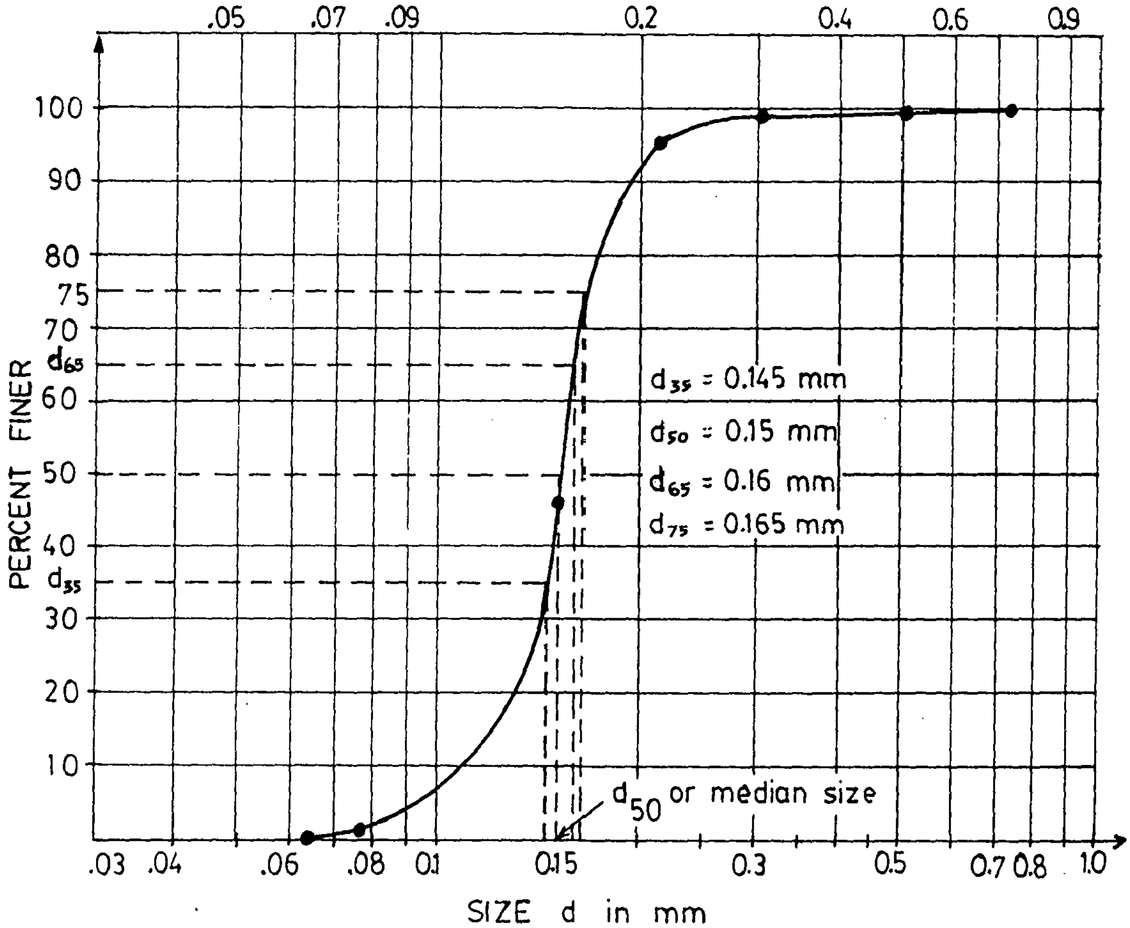


FIG. 3.8- CUMULATIVE SOIL CLASSIFICATION CURVE

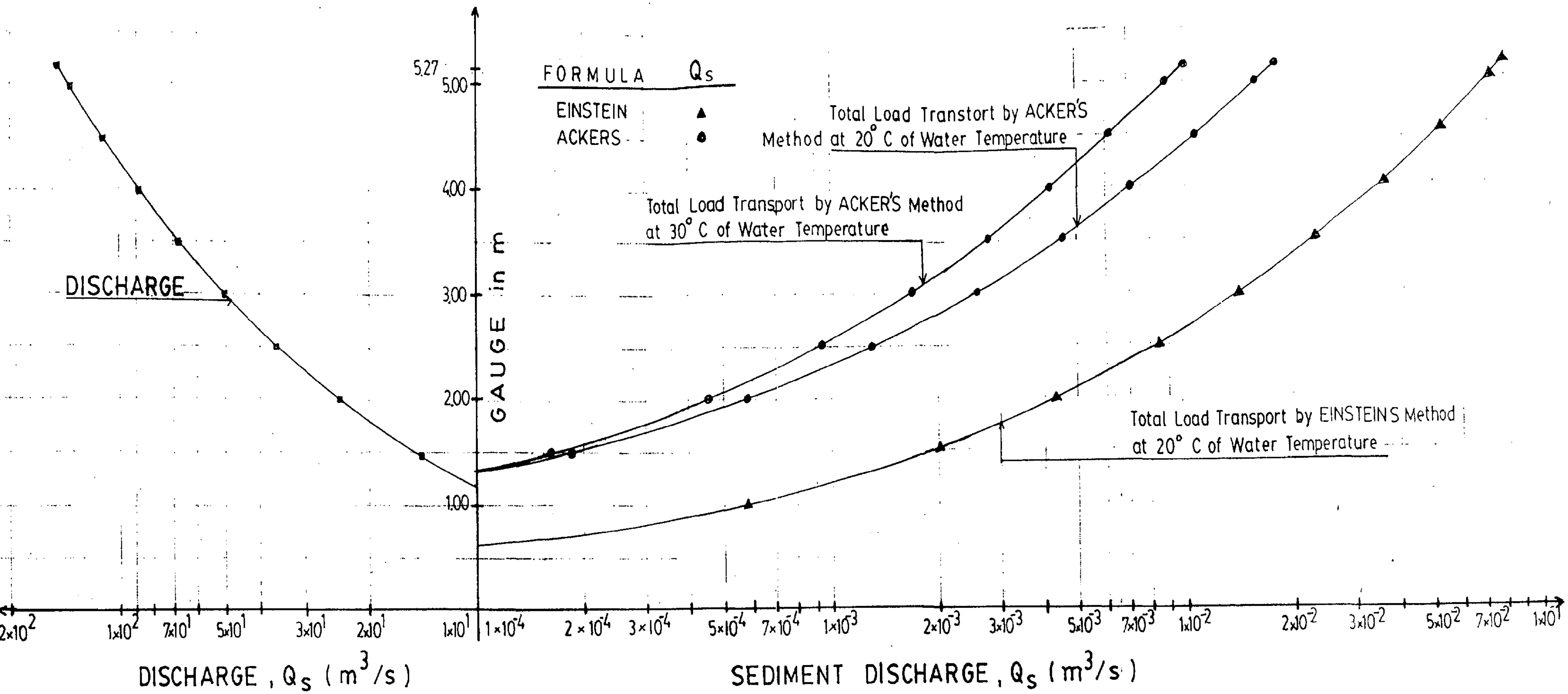


FIG. 3.9 — GAUGE - DISCHARGE AND SEDIMENT DISCHARGE CURVE
For Indira Gandhi Main Canal between Km 384 to Km 410
(RD 1260 to RD 1345)

" Difference in sediment transport at ful l suppy discharge between Einstein's Method and Ackers Method = 6,893.86 - 1,595.64
= 5298 22 m^3/day

$$\text{Ratio} = \frac{5298\ 22}{1595\ 64} \times 100\% = 332\% \text{ "}$$

3.6.4 Comparison of Sediment Transport :

The sediment transporting capacity of the channel according to discharge for uniform flow is plotted in Figure 3.9 against gauge. Since the day and night temperature and also month to month temperature varies considerably (see para 3.6) sediment transporting capacity should be worked out for a range of different temperatures. Here it is worked out for two temperatures of water at 20^0 C and 32^0 C by Acker's method. These are plotted in Figure 3.9. With increase in temperature, sediment transport decreases. This is due to decrease in viscosity of water with rise in temperature.

Also sediment transport by Einstein Method is shown in the graph. This method gives, **more sediment transport by 332.04 %** at peak discharge than by Acker's Method.

Sediment transport and deposits upstream of cross regulator:

Sediment transporting capacity of the channel in the zone of gradually varied flow computed by Acker's Method is shown in Table 3.8 for 2/3 discharge. 2/3 full supply discharge is considered more prominent, next to the full supply discharge. For any other discharge it can be worked by above procedure.

This Table shows additional silt deposits in the up stream of cross-regulator as under:

Volume of channel between back water length of curve = 20.0 Km (approx). Area of cross section at 390 Km, corresponding to 2/3 full supply,

discharge i.e. at gauge 4.35 m = 94.0 m^2

Area of cross section at 410 Km, corresponding to FSL i.e. at gauge 5.27 m = 124.6 m^2

Average area = 109.3 m^2

Volume of canal between back water curve = 109.3 x 20,000 = 2,186,000 m^3

(i) % loss of volume by Acker's Method for water at 20^0 C for gradually varied flow = [(388,87 – 250)/2186,000] x 100 = 0.00064% per day

(ii) If same % is considered through out the year then 0.0064 x 365 = 2.34% additional capacity more than normal silt deposit is lost or that much sediment will have to be cleared every year. Otherwise this will reduce discharging capacity.

3.7 REVIEW OF PROJECT DESIGN (FOR CANAL PERFORMANCE)

It has been reported that canal get silted by wind blown sand more on sides. Also weed grow in the canal. Then canal starts behaving as unlined canal. Therefore same is reviewed by taking ***n = 0.02 , 0.025 and 0.03*** computations are done for the reach.

3.7.1 For Uniform Flow, n = 0.02

For cross section of canal, trapezoidal section with curved ends and computation of A, P, and R see para – 3.5

Computation of discharge versus gauge

i) *Up to Height 0.557 m curved position*

$A = 7.714\ m^2$; $P = 15.558$ m ; $R = 0.496$ m

$V = R^{2/3} \times S_0^{1/2} / n = (0.496)^{2/3} \times (13/12000)^{1/2} / 0.02 = 0.286$ m/sec

$Q = A \times V = 7.714 \times 0.286 = 2.206$ cumecs

(ii) *Up to height 1.00 m*

$A = 14.905\ m^2$; $P = 17.539$ m ; $R = 0.850$ m

$V = (0.850)^{2/3}\ (1/12000)^{1/2} /0.02 = 0.410$ m/sec

$Q = 14.905 \times 0.410 = 6.105$ cumecs

(iii) *Up to height 1.50 m*

$A = 23.983\ m^2$; $P = 19.775$ m ; $R = 1.213$ m

$V = (1.213)^{2/3}\ (1/12000)^{1/2} \times 0.02 = 0.519$ m/sec

$Q = 23.983 \times 0.519 = 12.451$ cumecs

(iv) *Up to height 2.00 m*

$A = 34.061 m^2$; $P = 22.011$m ; $R = 1.547$ m

$V = (1.547)^{2/3}\ (1/12000)^{1/2} /0.02 = 0.611$ m/sec

$Q = 34.061 \times 0.611 = 20.795$ cumecs

(v) *Up to height 2.50 m*

$A = 45.139\ m^2$; $P = 24.247$ m ; $R = 1.862$ m

$V = (1.862)^{2/3}\ (1/12000)^{1/2} /0.02 = 0.691$ m/sec

$Q = 45.139 \times 0.691 = 31.183$ cumecs

(vi) *Up to height 3.00 m*

$A = 57.217\ m^2$; $P = 26.483\ m$; $R = 2.161m$

$V = (2.161)^{2/3}\ (1/12000)^{1/2}\ /0.02 = 0.763$ m/ sec

$Q = 57.217 \times 0.763 = 43.652$ cumecs

(vii) *Up to height 3.50 m.*

$A = 70.295\ m^2$; $P = 28.719\ m$; $R = 2.448\ m$

$V = (2.448)^{2/3}\ (1/12000)^{1/2}\ /0.02 = 0.829$ m/sec

$Q = 70.295 \times 0.829 = 58.279$ cumecs

(viii) *Up to height 4.00 m*

$A = 84.373\ m^2$; $P = 30.955\ m$; $R = 2.726\ m$

$V = (2.726)^{2/3}\ (1/12000)^{1/2}\ /0.02 = 0.891$ m/sec

$Q = 84.373 \times 0.891 = 75.151$ cumecs

(ix) *Up height 4.50 m*

$A = 99.451\ m^2$; $P = 33.191m$; $R = 2.996\ m$

$V = (2.996)^{2/3}\ (1/12000)^{1/2}\ /0.02 = 0.949$ m/sec

$Q = 99.451 \times 10.949 = 94.337$ cumecs

(x) *Up to height 5.00 m*

$A = 115.529\ m^2$; $P = 35.427\ m$; $R = 3.261\ m$

$V = (3.261\ )^{2/3}\ (1/12000)^{1/2}\ /0.02 = 1.004$ m/sec

$Q = 115.529 \times 1.044 = 115.969$ cumecs

xi) *Up to height 5.27 m*

$A = 124.627\ m^2$; $P = 36.634\ m$; $R = 3.402\ m$

$V = (3.402\ )^{2/3}\ (1/12000)^{1/2}\ /0.02 = 1.032$ m/sec

$Q = 124.\ 627 \times 1.032 = 128.671$ cumecs

3.7.2 For Uniform Flow, $n = 0.025$

Computation of discharge versus gauge

(i) *Up to Height 0.557 m curved position*

$V = R^{2/3} \times S_0^{1/2} / n = (0.496)^{2/3} \times (1/12000)^{1/2} / 0.025 = 0.229$ m/sec

$Q = A \times V = 7.714 \times 0.229 = 1.765$ cumecs

(ii) *Up to height 1.00 m*

$V = (0.850)^{2/3} (1/12000)^{1/2} /0.025 = 0.328$ m/sec

$Q = 14.905 \times 0.328 = 4.884$ cumecs

(iii) *Up to height 1.50 m*

$V = (1.213)^{2/3} (1/12000)^{1/2} / 0.025 = 0.415$ m/sec

$Q = 23.983 \times 0.415 = 9.960$ cumecs

(iv) *Up to height 2.00 m*

$A = 34.061 m^2$; $P = 22.011 m$; $R = 1.547$ m

$V = (1.547)^{2/3} (1/12000)^{1/2} /0.025 = 0.488$ m/sec

$Q = 34.061 \times 0.448 = 16.636$ cumecs

(v) *Up to height 2.50 m*

$A = 45.139 m^2$; $P = 24.247$ m ; $R = 1.862$ m

$V = (1.862)^{2/3} (1/12000)^{1/2} /0.025 = 0.553$ m/sec

$Q = 45.139 \times 0.553 = 24.946$ cumecs

(vi) *Up to height 3.00 m*

$V = (2.161)^{2/3} (1/12000)^{1/2} /0.025 = 0.610$ m/ sec

$Q = 57.217 \times 0.610 = 34.922$ cumecs

(vii) *Up to height 3.50 m*

$V = (2.448)^{2/3} (1/12000)^{1/2} /0.025 = 0.663$ m/sec

$Q = 70.295 \times 0.663 = 46.623$ cumecs

(viii) *Up to height 4.00 m*

$V = (2.726)^{2/3} (1/12000)^{1/2} /0.025 = 0.713$ m/sec

$Q = 84.373 \times 0.713 = 60.121$ cumecs

(ix) *Up to height 4.35 m*

$V = (2.916)^{2/3} (1/12000)^{1/2} / 0.025 = 0.745$ m / sec.

$Q = 94.823 \times 0.745 = 70.671$ cumecs

(x) Up height 4.50m

$V = (2.996)^{2/3} (1/12000)^{1/2} /0.025 = 0.759$ m/sec

$Q = 99.451 \times 0.759 = 75.470$ cumecs

(xi) *Up to height 4.60 m*

$V = (3.050)^{2/3} (1/12000)^{1/2} / 0.025 = 0.768$ m / sec.

$Q = 102.587 x 0.768 = 78.782$ cumecs

(xii) *Up to height 4.80 m*

$V = (3.153)^{2/3} (1/12000)^{1/2} /0.025 = 0.785$ m/sec

$Q = 108.888 x 0.785 = 85.493$ cumecs

(xiii) *Up to height 5.00 m*

$V = (3.261)^{2/3} (1/12000)^{1/2} /0.025 = 0.803$ m/sec

$Q = 115.529 x 0.803 = 92.767$ cumecs

xiv) *Up to height 5.27 m*

$V = (3.402)^{2/3} (1/12000)^{1/2} /0.025 = 0.826$ m/sec

$Q = 124.627 x 0.826 = 102.937$ cumecs.

3.7.3 For Uniform Flow, *n = 0.03*

Computation of discharge versus gauge

(i) *Up to Height 0.557m curved position*

$V = R^{2/3} x S_0^{1/2} / n = (0.496)^{2/3} x (1/12000)^{1/2} / 0.03 = 0.191$ m/sec

$Q = A x V = 7.714 x 0.191 = 1.471$ cumec

(ii) *Up to height 1.00 m*

$V = (0.850)^{2/3} (1/12000)^{1/2} /0.03 = 0.273$ m/sec

$Q = 14.905 x 0.273 = 4.070$ cumecs

(iii) *Up to height 1.50 m*

$V = (1.213)^{2/3} (1/12000)^{1/2} x 0.03 = 0.346$ m/sec

$Q = 23.983 x 0.346 = 8.300$ cumecs

(iv) *Up to height 2.00 m*

$V = (1.547)^{2/3} (1/12000)^{1/2} /0.03 = 0.407$ m/sec

$Q = 34.061 x 0.407 = 13.863$ cumecs

(v) *Up to height 2.50 m*

$V = (1.862)^{2/3} (1/12000)^{1/2} /0.03 = 0.461$ m/sec

$Q = 45.139 x 0.461 = 20.789$ cumecs

(vi) *Up to height 3.00 m*

$V = (2.161)^{2/3} (1/12000)^{1/2} /0.03 = 0.509$ m/ sec

$Q = 57.217 \times 0.509 = 29.102$ cumecs

(vii) *Up to height 3.50m*

$V = (2.448)^{2/3} (1/12000)^{1/2} /0.03 = 0.553$ m/sec

$Q = 70.295 \times 0.553 = 38.853$ cumecs

(viii) *Up to height 4.00 m*

$V = (2.726)^{2/3} (1/12000)^{1/2} /0.03 = 0.594$ m/sec

$Q = 84.373 \times 0.594 = 50.101$ cumecs

(xi) *Up height 4.50 m*

$V = (2.996)^{2/3} (1/12000)^{1/2} /0.03 = 0.632$ m/sec

$Q = 99.451 \times 0.632 = 62.891$ cumecs

(x) *Up to height 5.00 m*

$V = (3.261)^{2/3} (1/12000)^{1/2} /0.03 = 0.669$ m/sec

$Q = 115.529 \times 0.669 = 77.306$ cumecs

xi) *Up to height 5.27 m*

$V = (3.402)^{2/3} (1/12000)^{1/2} /0.03 = 0.688$ m/sec

$Q = 124.627 \times 0.688 = 85.781$ cumecs

All these data are tabulated in Table 3.9 and plotted in Figure 3.10 which gives discharge rating curves. From this Table, discharge can be read for any gauge (stage / depth of flow) or vice versa. This Table is further used in development of backwater curve for n = 0.025 in Figure 3.11

Inference :

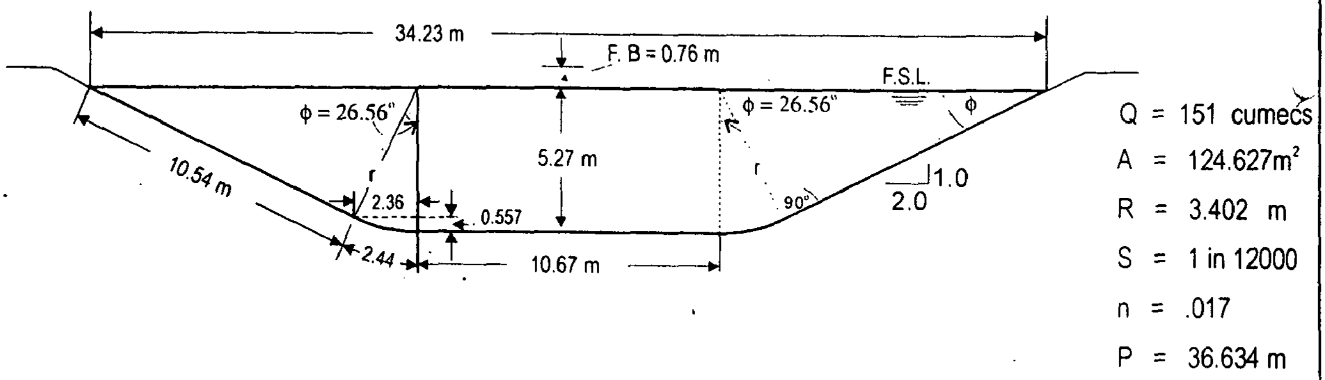
This Figure shows that for n = 0.02 discharge reduces from 151.375 m^3/s to 128.00 m^3/s and further increase in n to 0.025, discharge reduces to 102 m^3/s i.e 67% of original discharge. These computations do not take into account the reduction in cross-sectional area due to silting, as no data could be collected. The discharge will further reduce. With reduction in discharge velocity reduces and so more silting.

Table : 3.9

Comparison of flow Characteristics of Indira Gandhi Main Canal for n = 0.02, n = 0.025 and n = 0.03, from Km 389 to 410 Km (RD 1260 to1345) for Uniform Flow, Project Design Section

		n = 0.02			*n = 0.025*			*n = 0.03*		
S. No.	Depth of flow above bed Y (m)	Velocity V (m/sec)	Discharge Q = A.V (cumecs)	Froude Number $F_r = V/\sqrt{gA/T}$ No.	Velocity V (m/sec)	Discharge Q = A.V (cumecs)	Froude Number $F_r = V/\sqrt{gA/T}$ No.	Velocity V (m/sec)	Discharge Q = A.V (cumecs)	Froude Number $F_r = V/\sqrt{gA/T}$ No.
1	2	3	4	5	6	7	8	9	10	11
1.	0.557	0.286	2.206	0.1290	0.229	1.765	0.1033	0.191	1.471	0.0861
2.	1.000	0.410	6.105	0.1404	0.328	4.884	0.1124	0.273	4.070	0.0935
3.	1.500	0.519	12.451	0.1481	0.415	9.960	0.1184	0.346	8.300	0.0987
4.	2.000	0.611	20.795	0.1537	0.488	16.636	0.1228	0.407	13.863	0.1024
5.	2.500	0.691	31.183	0.1580	0.553	24.946	0.1265	0.461	20.789	0.1054
6.	3.000	0.763	43.652	0.1615	0.610	34.922	0.1291	0.509	29.102	0.1078
7.	3.500	0.829	58.279	0.1645	0.663	46.623	0.1316	0.553	38.853	0.1097
8.	4.000	0.891	75.151	0.1672	0.713	60.121	0.1338	0.594	50.101	0.1115
9.	4.500	0.949	94.337	0.1696	0.759	75.470	0.1356	0.632	62.891	0.1129
10.	5.000	1.004	115.969	0.1717	0.803	92.767	0.1373	0.669	77.306	0.1144
11.	5.270 (Full supply depth)	1.032	128.671 (Full supply discharge)	0.1727	0.826	102.937 (Full supply discharge)	0.1382	0.688	85.781 (Full supply discharge)	0.1151

Note : For detailed calculation see para 3.5, development of flow ratig curve of Indira Gandi Main Canal at Km 410.



Cross Section of I.G.M.N. between Km. 384 to Km.410 (R.D. 1260 - 1345), Project Design

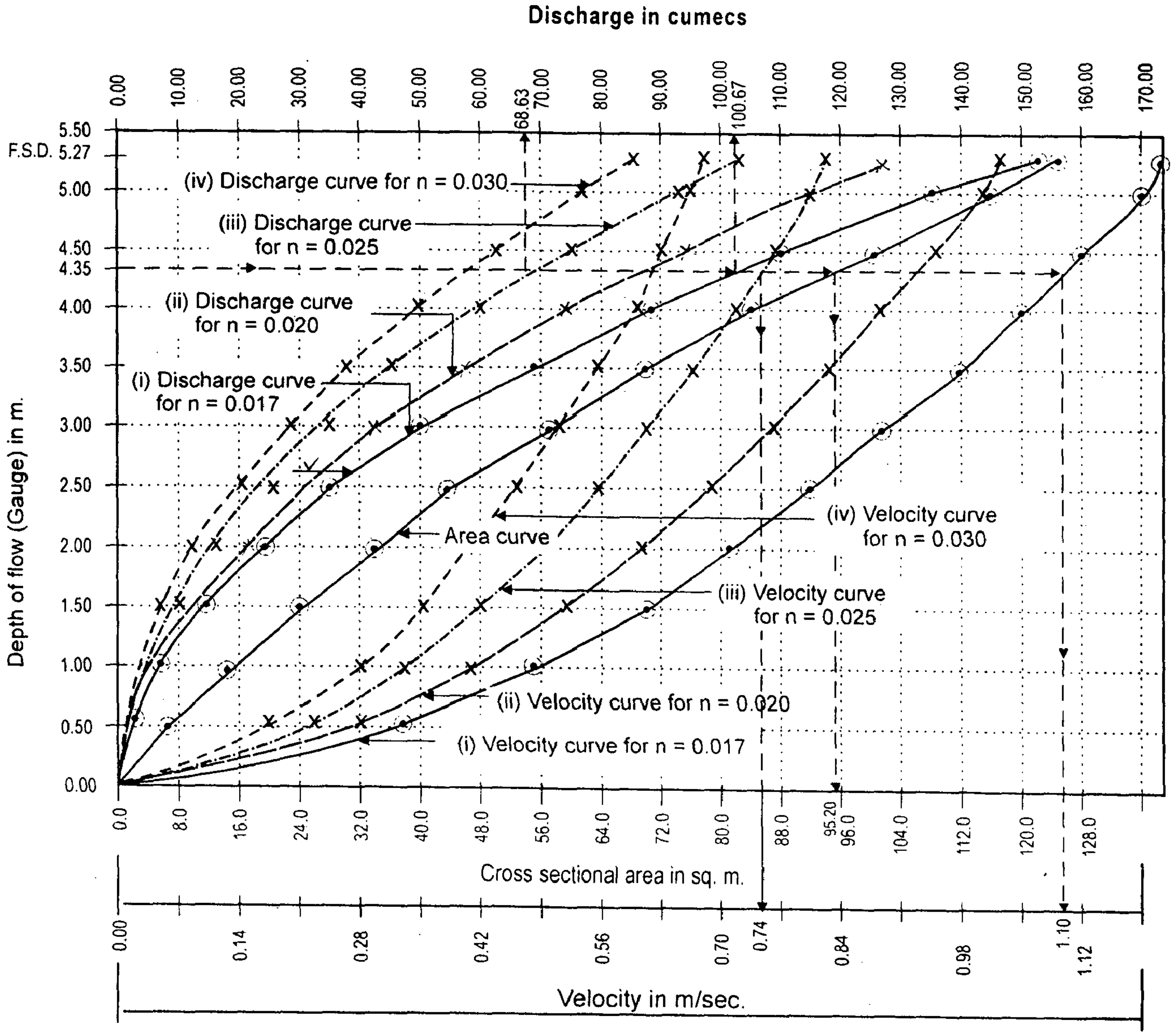


Figure - 3.10 Canal Rating Curve or Flow Characteristics in Trapezoidal Channel With Curved ends, Indra Gandhi Main Canal between Km. 384 to Km. 410 (RD 1260 to RD 1345), Project Design

Notes : To know discharge for a given gauge move arrow horizontally to meet discharge curve and then vertically upwards and read discharge. Similarly for area move upto area curve and down wards to read area scale. Similarly for velocity , move upto velocity curve and than down wards to read velocity scale.

Table 3.10

Indira Gandhi Main Canal, Computation of the Flow Profile Upstream of Cross Regulator at Km 410 (RD 1345), [Back Water Profile] for Gradually Varied Flow, for Q = 2/3 Full Supply Discharge = 100.67 cumecs , n = 0.017 , S_0 = 1/12000, α = 1,00 Project Design

S. No.	Depth Above Bed H	Cross-Sectional Area of Flow A	Hydraulic Radius R=A/P	$R^{4/3}$	Velocity V=Q/A (100.63) / Col. 3	$\alpha v^2/2g$	Specific Energy E	ΔE	S_f $\left(\frac{n^2V^2}{R^{4/3}}\right)$	$\overline{S}_f$	$S_0 - \overline{S}_f$	Δx, Up stream of Regulator	X, Up stream of Regulator (Distance)
	(m)	(m^2)	(m)		(m/s)		(m)	(m)				(m)	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
1.	5.27	124.627	3.402	5.117	0.551	0.015	5.285	-	0.0000371	-	-	-	-
2.	5.00	115.529	3.261	4.836	0.594	0.018	5.018	0.267	0.0000456	0.0000414	0.0000419	6, 372	6, 372
3.	4.80	108.888	3.153	4.623	0.630	0.020	4.820	0.198	0.0000537	0.0000497	0.0000336	5, 893	12, 265
4.	4.60	102.587	3.050	4.423	0.669	0.023	4.623	0.197	0.0000632	0.0000585	0.0000248	7, 944	20, 209
5.	4.35	94.823	2.916	4.166	0.724	0.027	4.377	0.246	0.0000786	0.0000709	0.0000124	19,838	40, 047

Explainatory Notes : same as in Table 3.4

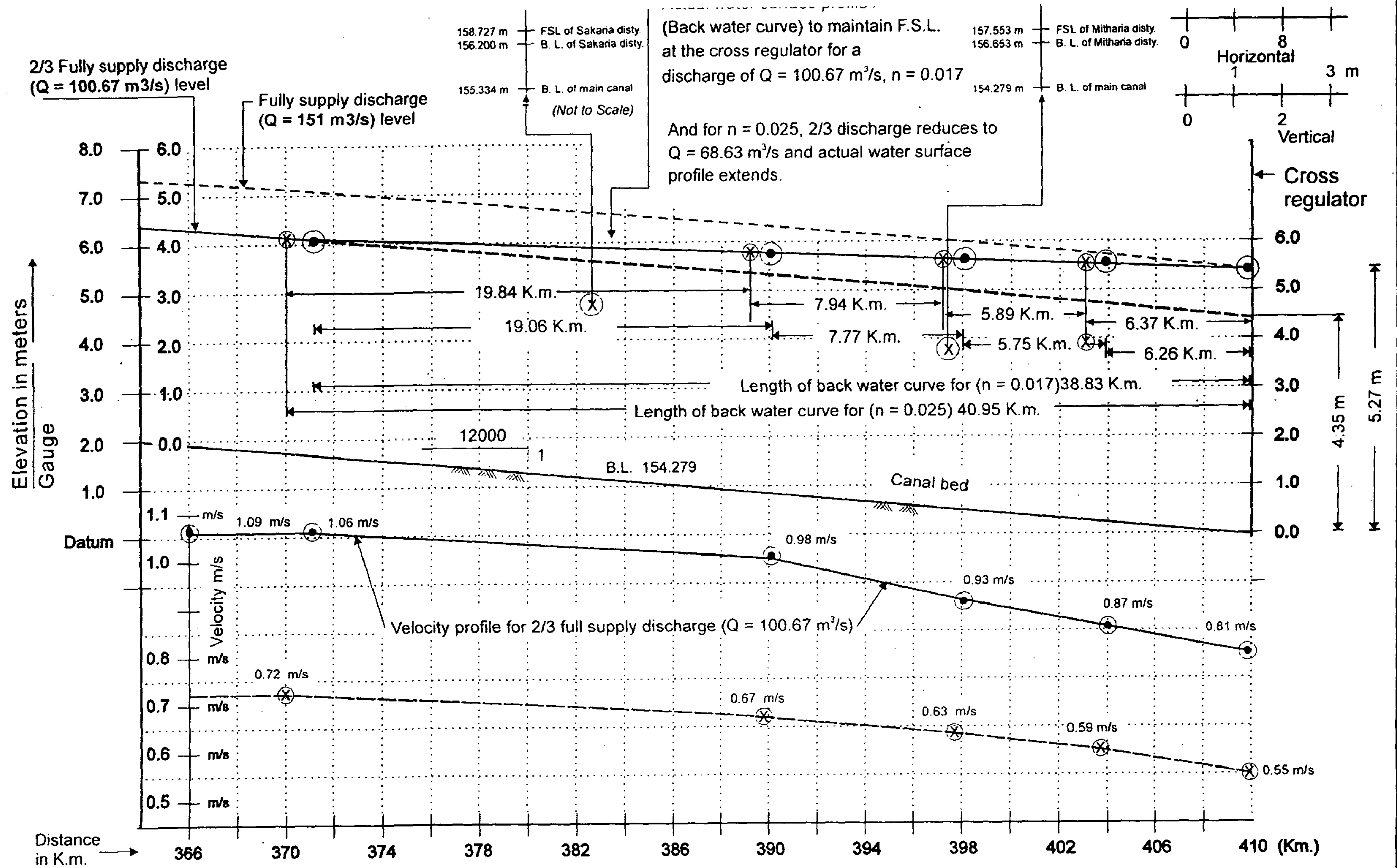


Figure 3.11 Comparative analysis of Velocity and Back Water Surface Profile **for n = 0.017 and 0.025** at Cross Regulator at Km. 410 (RD 1345.13) To Maintain FSL For 2/3 Discharge in Indra Gandhi Main Canal (IGMC) For Existing / Project Design

Roorkee.cdr

3.8. ALTERNATIVE HYDRAULIC DESIGN

Indira Gandhi Main Canal, between km 384 to km 410 (RD 1260 to 1345)

3.8.1 For Various B/D Ratio

With view to understand and explain the mechanics of flow for different B/D ratio, a revised alternate design is made. The purpose is not to reconstruct the canal, but only to explain the effect of B/D ratio in canal design. Also it is compared with the critical velocity ratio given by Kennedy.

A comparison of canals section for the same bed slope of 1 in 12000 and side slope Z = 2:1 (H:V) (as per project design) for various B/D (bed width : depth) ratios along with Kennedy's critical velocity is made in Table 3.11 and show in Figure 3.13. From this Table it is observed as under :

1) Kennedy's velocity of 1.808 m/s at X = 0.5 decreases to 0.935 m/s at X = 30.0

2) Kennedy's critical velocity ratio (m) increases very slowly from 0.685 at X = 0.5 to 0.950 at X = 30 where the velocity are considerably reduced.

3) Manning's velocity decreases from 1.239 m/s at X = 0.5 to 0.888 m/s at X = 30.0

4) Froud number is gradually increasing (very little) from 0.2022 at X = 0.5 to 0.2033 at X = 1.7 and then gradually decreases to 0.1931.

5) Thus the concept of critical velocity ratio of $m \geq 1$ gives very large bed width and uneconomical section, for the very flat slope and given side slope.

6) For the project design, critical velocity ratio (CVR or m) is 0.7625 < 0.8. much less than 1

7) At X = 1.6 or say 1.7 the depth of flow is 5.519 m, and 5.458 m, and velocities are 1.223 m/s and 1.222 m/s and F_r is also near to 0.20.

8) In fact there appears little justification for increasing X greater 1.5 or say 1.6.There after neither depth decreases rapidly, nor critical velocity ratio increases rapidly.

9) Comparison with Indian Standard Codes :

IS 10430 - 1982 lays down as under in para 6.8, "Critical velocity ratio is not applicable in lined canals but the possibility of silting can not be neglected. Hence the critical velocity ratio should be aimed at higher than unity or by any other method , it should be ensured that silting would not take place in the lined canal".

IS 10430 - 2000 lays down as under in para 8.8.4, "The critical velocity ratio should be aimed at higher than unity or by any other method, it should be ensured that silting will not take place in the lined canal".

10) But it is not practical to achieve above criteria of IS Codes in very flat slope. Also it may not be desirable. For this either bed slope or side slope should be increased. A comparison of Kennedy's velocity and Manning's velocity for optimum best section is given in para 3.8.6 and figure 3.17

11) To obtain more velocity of canal sections for the same bed slope of 1 in 12000 (as per project design) and side slope of Z = 1.5:1 (H:V), for various B/D ratio along with Kennedy's critical velocity is made in Table 3.14 and show in Figure 3.17, discussed in para 3.8.6

Table : 3.11

Indira Gandhi Main Canal, from Km 384 to Km 410 (RD 1260 to RD1345) for Uniform Flow, Computations of Sections for Q = 151, 375 m^3/s, S_o = 1/12000 n=0.017, Trapezoidal Sections with Curved Ends. Z = 2:1 (H:V) for Different B:D Ratios and Comparison with Kennedy's Critical Velocity

S.No.	X=B/D	Depth of Flow D (m)	Bed Width B (m)	Top Width $T(B_T)$ (m)	Area of flow A (m^2)	Wetted Perimeter P (m)	Hydraulic Radius R (m)	Manning's Velocity V (m/s)	Kennedy's Critical Velocity V_0 (m/s)	Critical Velocity Ratio $m=V/V_0$	Discharge Q=AV (m^3/s)	Fraud Number Fr
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
1	0.5	6.421	3.211	31.913	122.168	34.841	3.506	1.239	1.808	0.685	151.397	0.2022
2	0.6	6.318	3.791	32.034	122.281	34.915	3.502	1.239	1.790	0.692	151.487	0.2025
3	0.7	6.220	4.354	32.159	122.384	34.996	3.497	1.237	1.772	0.698	151.375	0.2025
4	0.8	6.127	4.901	32.287	122.477	35.081	3.491	1.235	1.755	0.704	151.286	0.2025
5	0.9	6.039	5.435	32.428	122.639	35.182	3.486	1.234	1.738	0.710	151.346	0.2026
6	1.0	5.955	5.955	32.571	122.787	35.287	3.480	1.233	1.723	0.716	151.415	0.2028
7	1.1	5.874	6.462	32.721	122.956	35.399	3.473	1.231	1.708	0.721	151.336	0.2028
8	1.2	5.796	6.956	32.866	123.074	35.509	3.466	1.230	1.694	0.726	151.418	0.2029
9	1.3	5.724	7.441	33.026	123.282	35.636	3.459	1.229	1.680	0.731	151.462	0.2031
10	1.4	5.651	7.912	33.173	123.372	35.750	3.451	1.227	1.666	0.736	151.340	0.2031
11	1.5	5.584	8.376	33.338	123.579	35.884	3.444	1.225	1.654	0.741	151.437	0.2031
12	**1.6**	**5.519**	**8.830**	**33.498**	**123.767**	**36.014**	**3.436**	**1.223**	**1.641**	**0.745**	**151.269**	**0.2031**
13	1.7	5.458	9.278	33.674	124.000	36.163	3.429	1.222	1.629	0.750	151.506	0.2033

table 3.11 continued

S.No.	X=B/D	Depth of Flow D (m)	Bed Width B (m)	Width $T(B_T)$ (m)	Area of Flow A (m^2)	Wetted Perimeter P (m)	Hydraulic Radius R (m)	Manning's Velocity, V (m/s)	Kennedy's Critical Velocity V_0 (m/s)	Critical Velocity Ratio $m=V/V_0$	Discharge Q=AV (m^3/s)	Fraud Number Fr
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
14	1.8	5.397	9.714	33.837	124.153	36.298	3.420	1.219	1.618	0.754	151.380	0.2032
15	2.0	5.284	10.568	34.187	124.609	36.597	3.405	1.215	1.596	0.761	151.439	0.2032
16	2.5	5.031	12.578	35.067	125.622	37.361	3.362	1.205	1.547	0.779	151.324	0.2033
17	3.0	4.816	14.448	35.975	126.701	38.171	3.319	1.194	1.504	0.794	151.34	0.2031
18	3.5	4.629	16.201	36.891	127.761	39.002	3.276	1.184	1.466	0.808	151.299	0.2031
19	4.0	4.466	17.865	37.828	128.915	39.865	3.234	1.174	1.433	0.819	151.322	0.2030
20	6.0	3.969	23.812	41.552	133.294	43.362	3.074	1.136	1.329	0.855	151.434	0.2025
21	8.0	3.626	29.008	45.216	137.565	46.869	2.935	1.100	1.254	0.877	151.340	0.2014
22	10.0	3.369	33.690	48.750	141.459	50.286	2.813	1.070	1.197	0.895	151.425	0.2006
23	12.0	3.168	38.018	52.180	145.173	53.625	2.707	1.043	1.150	0.906	151.348	0.1996
24	15.0	2.933	43.999	57.111	150.256	58.449	2.571	1.008	1.095	0.920	151.452	0.1984
25	20.0	2.650	53.006	64.853	157.784	66.062	2.388	0.960	1.026	0.935	151.397	0.1964
26	25.0	2.446	61.162	72.098	164.373	73.213	2.245	0.921	0.975	0.945	151.448	0.1948
27	30.0	2.291	68.748	78.957	170.330	80.002	2.129	0.888	0.935	0.950	151.310	0.1931

Note : Explanatory notes are given on next page

Explanatory notes to table 3.11:

Col.2 B/D ratio, where B is bed width and D is depth of flow

Col.3 Depth of flow in m, computed by equation, $D = \left[\frac{Q.n}{e.S^{1/2}}\right]^{3/8}$

Col.4 Bed width in m corresponding to the depth of flow in col. 3, B = XD

Col.5 Top width in m corresponding to D in col. 3, $B_T = D(X + 4.47)$

Col.6 Water area in m^2 corresponding to D in col. 3, $A = D^2(X + 2.463)$

Col.7 Wetted parameter in m corresponding to D in col. 3, P = D(X + 4.926)

Col.8 Hydraulic radius in m corresponding to D in col. 3, R = A/P

Col.9 Manning's velocity in m/s, $V = 1/n\ R^{2/3}\ S^{1/2}$

Col.10 Kennedy's critical velocity in m/s, $V_o = 0.55\ D^{0.64}$

Col.11 Critical velocity ratio obtained by dividing col. 9 by col. 10.

Col.12 Discharge in m^3/s, Q = A x V

Col.13 Froud number, $F_r = V/\sqrt{g\ A/B_T}$

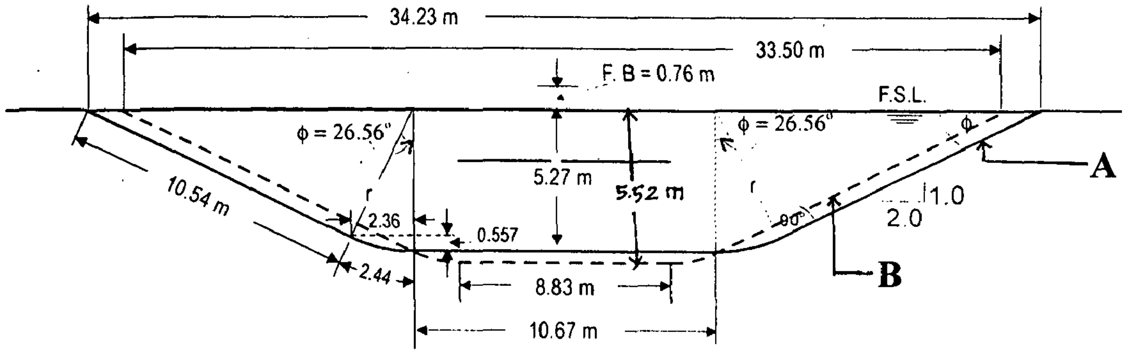


Fig. 3.12. Comparison of cross sections IGMC for B/D = 1.6 and 2.0, Project Design between Km 384 to Km 410 (RD 1260 to RD 1345)

Notes : A = Project design for B/d = 2.0

B = project design for B/D = 1.6

Table : 3.12

Comparison of Section for B/D = 1.6 and 2.0

Parameter	B/D Ratio	
	B/D = 2.0	**B/D = 1.6**
Q	151 m^3/s	151 m^3/s
D	**5.27 m**	**5.52 m**
B	10.67 m	8.83 m
A	124.627 m^2	123.76 m^2
R	3.402 m	3.436 m
n	1 in 12000	1 in 12000
S	0.017	0.017
P	36.634 m	36.014 m
V	**1.215 m/s**	**1.223 m/s**
T	**34.23 m**	**33.50 m**

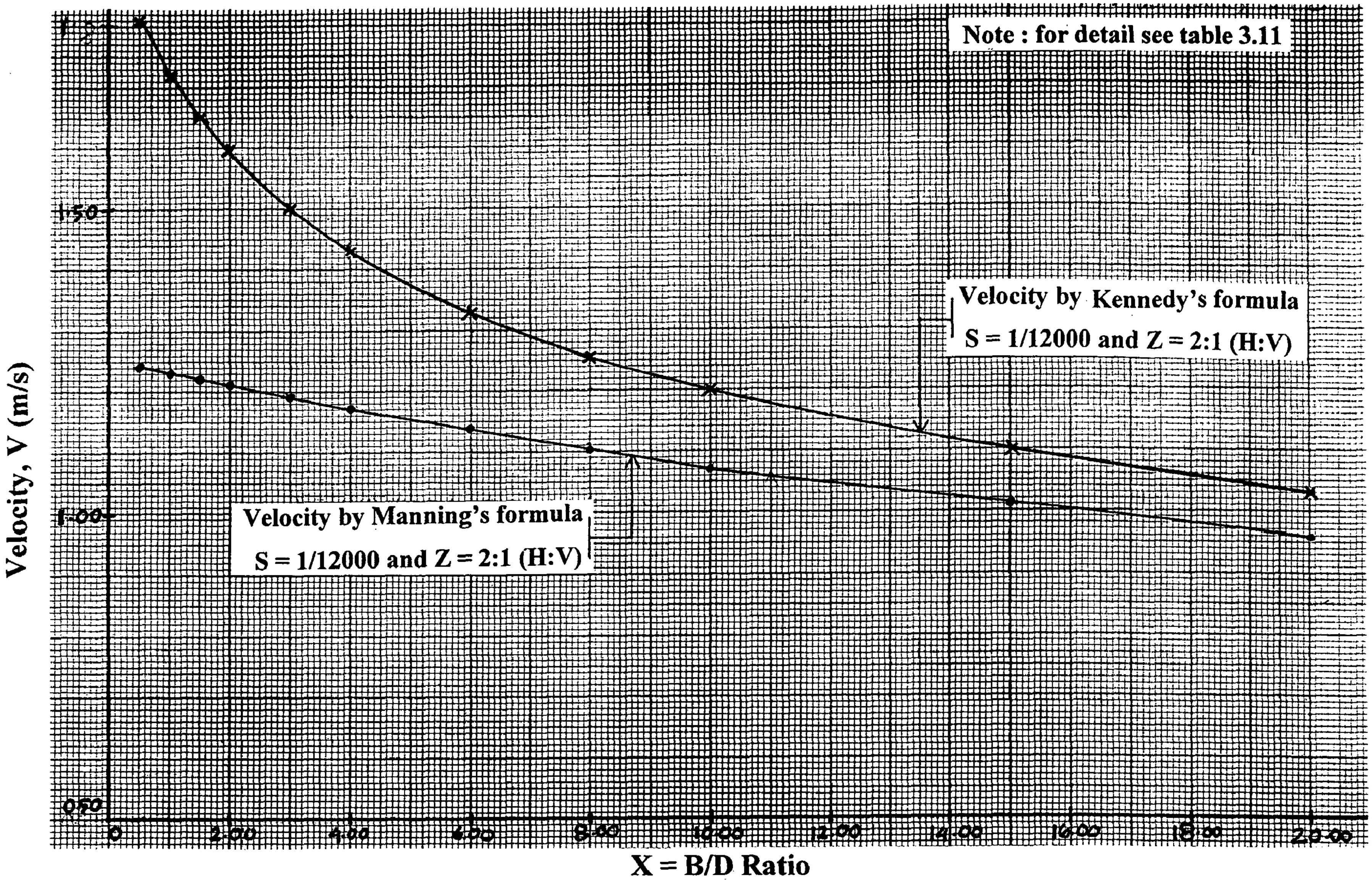


Fig. 3.13. Indira Gandhi Main Canal, from Km 384 to Km 410 (RD 1260to RD 1345) For Uniform Flow, Computations of Sections for Q = 151.375 m^3/s, S_o =1/12000,n = 0.017 and Z = 2:1 (H:V) for different B/D Ratios and Comparison with Kennedy's Critical Velocity

3.8.2 Design by Kennedy Method.

Channel parameters through Manning's formula:

Depth of flow, h = 5.27 m; Bed slope , S_0 = 1/12000 ; Discharge , Q = 151.375 m^3/s

Velocity, V = 1.215 m/s; Trapezoidal section with curved ends side slope Z = 2:1 (H:V).

Velocity by Kennedy's formula for the adopted depth,

$V_0 = 0.55\ h^{0.64} = 0.55\ (5.27)^{0.64} = 1.5934$ m/s

Critical velocity ratio,

$m = V/V_0 = 1.215/1.5934 = 0.7625 < 0.8$

Much less than 1,designed section do not satisfy m=1,for that V should be1.593 m/s

Corresponding to this velocity, required **area of flow,**

$A = Q/A = 151.375/1.5934 = 95.001\ m^2$

Depth of flow By Manning's formula for best trapezoidal section (most optimum) of

Z = 1: 0.577 (V:H), for the given slope of 1/12000,

$D = [\ (Q \times n)/\ (1.091\ S^{1/2})\ ]^{3/8}$

$= [\ 151.375 \times 0.017/1.091 \times \sqrt{(1/12000)}\ ]^{3/8} = 8.028$ m

Maximum Velocity ,

$V = (D/2)^{2/3} \times \delta^{1/2} \times 1/n$

$= (8.028/2)^{2/3} \times \sqrt{(1/12000)} \times 1/0.017 = 1.356$ m/s.

This is still less than 1.593 m/s.

But V_o by Kennedy's formula for **D = 8.028 m**

$V_o = 0.55\ (8.028)^{0.64} = 2.086$ m/s

Which is furthermore. As depth goes on increasing, V_o also increases and Manning's velocity will be lower than V_o for the given slope of 1/12000.

Alternative may be to compute slope by Manning's formula for the adopted Kennedy's Velocity.

Slope by Manning's formula for Kennedy velocity:

a). For V = 1.593 m/s

$1.593 = (5.27/2)^{2/3} \times S^{1/2} \times 1/0.017$ (R = D/2 for best section)

$1.593 = 112.22\ S^{1/2}$

$S = 0.0002 = 1/0.0002 = 5000$ m , (1: 5000) say 1 in 5000

b). For $V = 2.086$ m/s

$2.086 = (8.028/2)^{2/3} \times S^{1/2} \times 1/0.017 = 148.572\ S^{1/2}$

$S = 0.000197 = 1/0.000197 = 5073$ m, (1 : 5073) say 1 in 5000.

(slightly steeper and not flatter)

3.8.3 Design by Lindley Method

Velocity by Lindley's formula for the adopted depth of h = 5.27 m,

$V = 0.567\ D^{0.57} = 0.567\ (5.27)^{0.57} = 1.462$ m/s

Corresponding to this velocity, required **area of flow**,

$A = Q/A = 151.375/1.462 = 81.428\ m^2$

Depth of flow by Manning's formula for best trapezoidal section (most optimum), for the given slope of 1/12000, Z = 2:1 (H:V) is 8.028 m and **maximum velocity** of 1.356 m/s Corresponding to this depth and velocity, required bed width of 5.873 m and top width of 41.762 m and area of flow is 205.933 m^2.

But V by Lindley's formula for **D = 8.028 m**

$V = 0.567\ (8.028)^{0.57} = 1.859$ m/s

Which is furthermore. As depth goes on increasing, V also increases and Manning's velocity will be lower than V of Lindley for the given slope of 1/12000.

Alternative may be to compute slope by Manning's formulae for the adopted Lindley's Velocity.

Slope by Manning's formula for Lindley velocity:

a). For $V = 1.462$ m/s

$1.462 = (5.27/2)^{2/3} \times S^{1/2} \times 1/0.017 = 112.22\ S^{1/2}$

$S = 0.00017 = 1/0.00017 = 5891$ m , (1: 5800) say 1 in 5800

b). $V = 1.859$ m/s

$1.859 = (8.028/2)^{2/3} \times S^{1/2} \times 1/0.017 = 148.572\ S^{1/2}$

S = 0.000157 = 1/0.000157 = 6387 m, (1 : 6387) say 1 in 6300

(slightly steeper and not flatter)

3.8.4 Comparison of Cross Sections for Various B/D Ratios and Slopes

3.8.4.1 Computation of comparison of canal sections for the bed slope of 1 in 5000 for Kennedy's velocity and side slope Z = 2:1 (H:V) **for various B/D (bed width : depth) ratio** along - with Kennedy's critical velocity is made in Table 3.12 and shown in Figure 3.15. From this Table it is observed as under :

1) Kennedy's critical velocity (m) is slightly greater than 1, even at X = 0.50

2) Kennedy's critical velocity ratio (m) increases very slowly from 1.057 at X = 0.5 to 1.442 at X = 20 where the velocity are considerably reduced.

3) Kennedy's velocity decreases slowly from 1.628 m/s at X = 0.5 to 0.924 m/s at X = 20.0

3) Manning's velocity decreases from 1.721 m/s at X = 0.5 to 1.332 m/s at X = 20.0

5) Froud number is gradually increasing (very little) from 0.3049 at X = 0.5 to 0.3064 at X = 1.75 and then decreases to 0.2691.

6) At X = 0.5 the depth of flow is 5.45 m and velocity is 1.721 m/s and F_r is also near to 0.30.

In fact there appears little justification for increasing X greater 0.5. There after neither depth decreases rapidly, nor critical velocity ratio increases rapidly. But may not be practical to achieve the bed slope of 1 in 5000. But this canal has more silt transporting capacity.

3.8.4.2 A comparison of canal sections for the bed slope of 1 in 6300 for Lindley's velocity and side slope Z = 2:1 (H:V) for various B/D (bed width : depth) ratio along - with Lindley's critical velocity is made in Table 3.13 and show in Figure 3.15. From this Table it is observed as under :

1) Lindley's velocity decreases from 1.528 m/s at X = 0.5 to 0.923 m/s at X = 20.0

2) Manning's velocity decreases from 1.239 m/s at X = 0.5 to 0.888 m/s at X = 20.0

3) Froud number is gradually increasing (very little) from 0.2735 at X = 0.5 to 0.2749 at X = 1.75 and then decreases to 0.2656.

4) At X = 0.5 the depth of flow is 5.691 m and velocity is 1.578 m/s and and F_r is also near to 0.27

Again there appears little justification for increasing X greater 0.5. There after depth decreases slowly and velocity also decrease. Again it may not be practical to achieve flat slope.

3.8.4.3 Therefore a third alternative is considered to increase the side slope. A comparison of canal sections for the same bed slope of 1 in 12000 (as per project design) and side slope Z = 1.5:1 (H:V) for various B/D (bed width : depth) ratio along - with Kennedy's critical velocity is made in Table 3.14 and show in Figure 3.15. From this table it is observed as under :

1) Kennedy's velocity decreases from 1.864 m/s at X = 2.0 to 1.028 m/s at X = 20.0

2) Kennedy's critical velocity ratio (m) increases very slowly from 0.692 at X = 0.5 to 0.944 at X = 20 where the velocity are considerably reduced.

3) Kennedy's velocity for side slope, Z = 1.5:1 (H:V) greater than for side slope, Z = 2:1 (H:V)

2) 4) Manning's velocity decreases from 1.291 m/s at X = 2.0 to 0.971 m/s at X = 20.0

5) Froud number is gradually increasing (very little) from 0.1999 at X = 0.5 to 0.2026 at X = 2.75 and then decreases to 0.1965.

6) At $X = 2.0$ the depth of flow is 5.42 m and velocity is 1.260 m/s and F_r is also near to 0.20.

In fact there appears little justification for increasing X greater 2.0 there after neither depth decreases rapidly, nor critical velocity ratio increases rapidly. But it is not practical to achieve above criteria. Also it may not be desirable.

Thus for flat slope of 1 in 12000, a side slope of 1:1.5 in more desirable. It gives $D = 5.423$ m say 5.43 m, $X = 2.0$, $B = 10.85$ m, Manning's $V = 1.26$ m/s and Kennedy's $V = 1.623$ m/s, and Kennedy's critical velocity ratio of 0.776, though still less than 1.

In general it may be concluded that Kennedy's formula $V = 0.546\ m\ D^{0.64}$ gives velocity little higher velocity than that by Manning's formula for best section $V = 0.63 d^{2/3} S^{1/2} / n$ and are more suitable to carry the carry or (transport) the sediment entering into canal. Kennedy's formula also gives slopes with the help of Manning's formula.

A comparison of all the section is shown in figure 3.15. For the project slope of 1 in 12000, a side slope of 1:1.5 (V:H) is much better, otherwise change in slope is desirable.

Table : 3.12

Comparison of Sections for Q = 151.375 m^3/s, S_o = 1/5000, n = 0.017, Trapezoidal Section With Curve Ends, Z = 2:1 (H:V), for Various B/D Ratio

S.No.	X=B/D	Depth of Flow D (m)	Bed Width B (m)	Top Width $T(B_T)$ (m)	Area of Flow A (m^2)	Wetted Perimeter P (m)	Hydraulic Radius R (m)	Manning's Velocity V (m/s)	Kennedy's Critical Velocity V_0 (m/s)	Critical Velocity Ratio $m = V/V_0$	Discharge Q=AV (m^3/s)	Froud Number Fr
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
1	**0.50**	**5.449**	**2.724**	**27.081**	**87.972**	**29.566**	**2.975**	**1.721**	**1.628**	**1.057**	**151.397**	**0.3049**
2	0.75	5.239	3.929	27.345	88.174	29.734	2.965	1.717	1.587	1.081	151.354	0.3052
3	1.00	5.053	5.053	27.640	88.418	29.944	2.953	1.712	1.551	1.104	151.415	0.3056
4	1.25	4.888	6.110	27.959	88.711	30.188	2.939	1.706	1.518	1.124	151.355	0.3058
5	1.50	4.739	7.108	28.290	88.988	30.451	2.922	1.702	1.489	1.143	151.437	0.3063
6	1.75	4.605	8.059	28.645	89.356	30.746	2.906	1.695	1.462	1.160	151.461	0.3064
7	2.00	4.484	8.968	29.011	89.730	31.055	2.889	1.688	1.437	1.175	151.439	0.3064
8	2.50	4.269	10.673	29.757	90.460	31.704	2.853	1.673	1.392	1.201	151.324	0.3063
9	3.00	4.087	12.260	30.527	91.237	32.391	2.817	1.659	1.354	1.225	151.344	0.3064
10	3.50	3.928	13.748	31.305	91.999	33.096	2.780	1.645	1.320	1.246	151.299	0.3063
11	4.00	3.790	15.160	32.101	92.831	33.829	2.744	1.630	1.290	1.263	151.322	0.3061
12	6.00	3.368	20.206	35.260	95.983	36.796	2.609	1.578	1.196	1.319	151.434	0.3053
13	8.00	3.077	24.616	38.370	99.060	39.773	2.491	1.528	1.129	1.353	151.340	0.3036
14	10.00	2.859	28.589	41.368	101.864	42.672	2.387	1.487	1.077	1.380	151.425	0.3025
15	15.00	2.489	37.337	48.464	108.196	49.599	2.181	1.400	0.986	1.420	151.452	0.2991
16	20.00	2.249	44.980	55.033	113.619	56.059	2.027	1.332	0.924	1.442	151.397	0.2961

Explanatory notes: Same as in Table 3.11

Table :3.`13

Comparison of Section for Q = 151.375 m^3/s, S_o = 1/6300, n = 0.017 Trapezoidal Section With Curve Ends, Z = 2:1 (H:V) for Various B/D Ratio

S.No.	X=B/D	Depth of Flow D (m)	Bed Width B (m)	Top Width $T(B_T)$ (m)	Area of Flow A (m^2)	Wetted Perimeter P (m)	Hydraulic Radius R (m)	Manning's Velocity V (m/s)	Lindley's Velocity V (m/s)	Velocit y Ratio V_m/V_e	Discharge Q=AV (m^3/s)	Froud Number Fr
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
1	**0.50**	**5.691**	**2.845**	**28.283**	**95.956**	**30.878**	**3.108**	**1.578**	**1.528**	**1.033**	**151.397**	**0.2735**
2	0.75	5.471	4.103	28.559	96.176	31.054	3.097	1.574	1.494	1.054	151.354	0.2738
3	1.00	5.277	5.217	28.867	96.442	31.273	3.084	1.570	1.463	1.073	151.415	0.2742
4	1.25	5.105	6.381	29.200	96762	31.528	3.069	1.564	1.436	1.089	151.355	0.2743
5	1.50	4.949	7.424	29.546	97.065	31.802	3.052	1.560	1.411	1.106	151.437	0.2748
6	1.75	4.810	8.417	29.917	97.466	32.110	3.035	1.554	1.388	1.120	151.461	0.2749
7	2.00	4.683	9.366	30.299	97.874	32.434	3.018	1.547	1.367	1.132	151.439	0.2749
8	2.50	4.459	11.147	31.078	98.670	33.111	2.980	1.534	1.329	1.154	151.324	0.2748
9	3.00	4.268	12.804	31.883	99.517	33.829	2.942	1.521	1.300	1.173	151.344	0.2748
10	3.50	4.102	14.358	32.695	100.349	34.566	2.903	1.508	1.268	1.189	151.299	0.2748
11	4.00	3.958	15.833	33.526	101.256	35.331	2.866	1.494	1.242	1.203	151.322	0.2746
12	6.00	3.517	21.103	36.825	104.695	38.429	2.724	1.446	1.161	1.246	151.434	0.2740
13	8.00	3.214	25.708	40.073	108.050	41.538	2.601	1.401	1.103	1.270	151.340	0.2723
14	10.00	2.986	29.858	43.205	111.109	44.566	2.493	1.363	1.058	1.288	151.425	0.2713
15	15.00	2.600	38.995	50.615	118.019	51.801	2.278	1.283	0.977	1.313	151.452	0.2683
16	20.00	2.349	46.977	57.476	123.931	58.547	2.117	1.222	0.923	1.324	151.397	0.2656

Explanatory Notes :

Col . 2 Col. 2 to col . 9 same as in Table 3.13

Col .10 Lindley's velocity in m/s , $V = 0.567\ D^{0.57}$

Col .11 Col. 11 to col. 13 same as in Table 3.13

Table : 3.14

Comparison of Section for Q = 151.375 m^3/s, S_o = 1/12000, n = 0.017, Trapezoidal Section With Curve Ends, Z = 1.5:1 (H:V)

S.No.	X=B/D	Depth of Flow D (m)	Bed Width B (m)	Top Width $T(B_T)$ (m)	Area of Flow A (m^2)	Wetted Perimeter P (m)	Hydraulic Radius R (m)	Manning's Velocity V (m/s)	Kennedy's Critical Velocity V_0 (m/s)	Critical Velocity Ratio $m = V/V_0$	Discharge Q=AV (m^3/s)	Froud Number Fr
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
1	0.50	6.733	3.366	27.605	117.318	31.483	3.726	1.291	1.864	0.692	151.402	0.1999
2	0.75	6.438	4.829	28.006	117.639	31.715	3.709	1.286	1.811	0.710	151.290	0.2003
3	1.00	6.185	6.185	28.451	118.127	32.013	3.690	1.281	1.765	0.726	151.327	0.2007
4	1.25	5.960	7.450	28.906	118.575	32.339	3.667	1.276	1.724	0.740	151.336	0.2012
5	1.50	5.762	8.643	29.385	119.114	32.704	3.642	1.270	1.687	0.753	151.304	0.2014
6	1.75	5.585	9.774	29.879	119.713	33.096	3.617	1.264	1.654	0.765	151.366	0.2017
7	**2.00**	**5.423**	**10.846**	**30.370**	**120.233**	**33.494**	**3.590**	**1.260**	**1.623**	**0.776**	**151.467**	**0.2021**
8	2.25	5.278	11.876	30.877	120.847	33.917	3.563	1.252	1.595	0.785	151.281	0.2020
9	2.50	5.146	12.864	31.388	121.479	34.352	3.536	1.247	1.569	0.795	151.462	0.2024
10	2.75	5.023	13.814	31.897	122.072	34.790	3.509	1.241	1.545	0.803	151.508	0.2026
11	3.00	4.911	14.732	32.411	122.698	35.239	3.482	1.233	1.523	0.810	151.336	0.2024
12	3.50	4.710	16.485	33.442	123.971	36.155	3.429	1.221	1.483	0.823	151.331	0.2024
13	4.00	4.536	18.145	34.476	125.277	37.089	3.378	1.208	1.448	0.835	151.352	0.2023
14	6.00	4.011	24.066	38.505	130.117	40.815	3.188	1.163	1.338	0.869	151.342	0.2020
15	8.00	3.654	29.229	42.382	134.663	44.486	3.027	1.124	1.260	0.891	151.300	0.2012
16	10.00	3.390	33.897	46.100	138.894	48.053	2.890	1.089	1.201	0.907	151.307	0.004
17	15.00	2.945	44.171	54.772	148.178	56.468	2.624	1.022	1.098	0.931	151.380	0.1983
18	20.00	2.658	53.151	62.718	155.999	64.249	2.428	0.971	1.028	0.944	151.403	0.1965

Explanatory notes :

Col. 2 Col. 2 to col. 4 same as in Table 3.13

Col. 5 Top width in m corresponding to D in col. 3, B_T = D (x+3.60)

Col. 6 water area in m^2 corresponding to D in col. 3, A = D^2 (x +2.088)

Col. 7 watted parameter in the corresponding to D in col. 3, P = D (x+4.176)

Col. 8 Col. 8 to col. 13 same as in Table 3.13

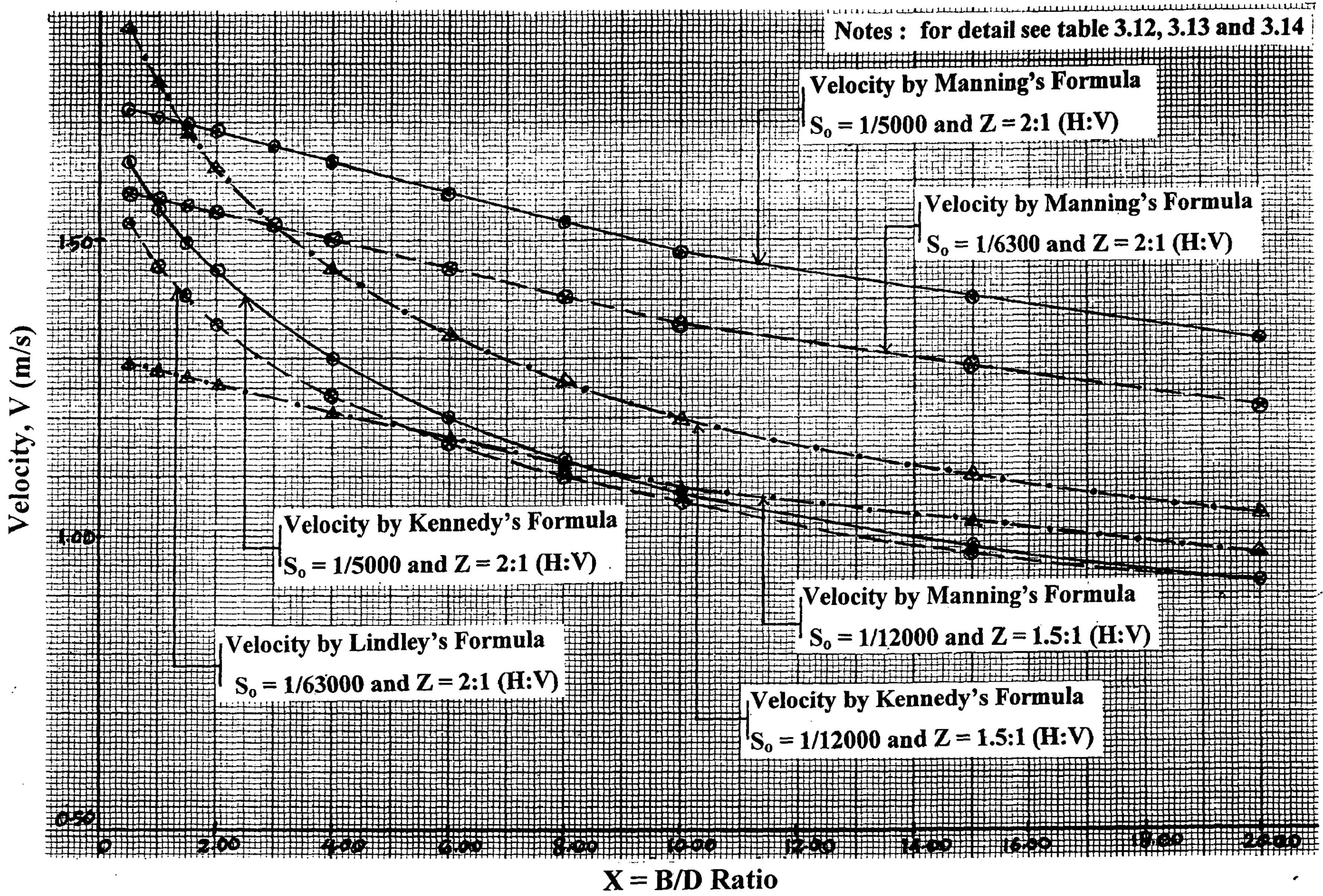


Fig. 3.14 Comparison of Cross Sections for Q = 151.375 m^3/s, n = 0.017, Project Design For Uniform Flow and Different B/D Ratios with Kennedy's Critical Velocity and Lindley Velocity

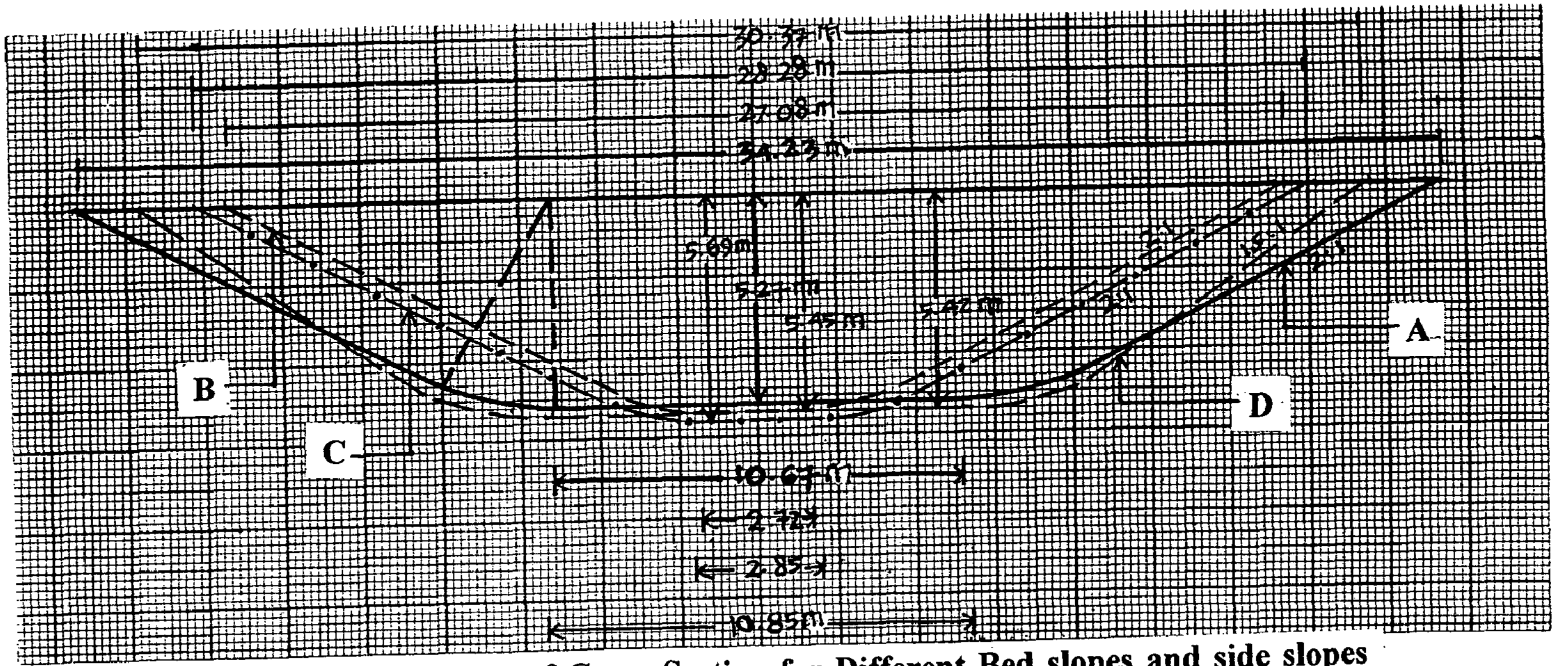


Fig.3.15 Comparison of Cross Section for Different Bed slopes and side slopes Project Design between Km 384 to Km 410 (RD 1260 to RD 1345)

Table : 3.16

Comparison of Cross Sections of Project Design with Various Bed Slopes and Side Slopes

Parameter	Z = 2:1 (H:V)			Z = 1.5:1 (H:V)
	S_o = 1/12000 A	S_o = 1/5000 B	S_o = 1/6300 C	S_o = 1/12000 D
Q	151 m^3/s	151 m^3/s	151 m^3/s	151 m^3/s
D	5.27 m	5.45 m	5.69 m	5.42 m
B	10.67 m	2.72 m	2.85 m	10.85 m
B/D	2.0	0.5	0.5	2.0
A	124.627 m^2	87.97 m^2	95.96 m^2	120.23 m^2
R	3.402 m	2.98 m	3.11 m	3.59 m
n	0.017	0.017	0.017	0.017
P	36.634 m	29.57 m	30.88	33.49 m
V	1.215 m/s	1.721 m/s	1.578 m/s	1.260 m/s
T	34.23 m	27.08 m	28.28 m	30.37 m

Note : A = project design for Z = 2:1 (H:V), S_o = 1/12000 and X = B/D = 2.0

B = project design for Z = 2:1 (H:V), S_o = 1/5000 and X = B/D = 2.0

C = project design for Z = 2:1 (H:V), S_o = 1/6300 and X = B/D = 2.0

D = project design for Z = 1.5:1 (H:V), S_o = 1/12000 and X = B/D = 2.0

3.8.5 Design by Lacey Method

*Sediment properties: M*ean particle size, $m_r = d_{50} = 0.15$ mm.

Silt factor,

$$f = 1.76\sqrt{m_r} = 1.76\sqrt{0.15} = 0.6816 = 0.7$$

Velocity of flow,

$$V = [Qf^2/140]^{1/6} = [\,151.375\,(0.7)^2/140]^{1/6} = 0.8995 \text{ m/s}$$

Area of channel,

$$A = Q/V = 151.375/0.8995 = 168.2879 \text{ m}^2$$

Computation of **bed width**, B :

$$P = 4.75\sqrt{Q} = 4.75 \times \sqrt{151.375} = 58.441 \text{ m}$$

$$A = B \times D + 2\,(\,\pi D^2 \times 26.57/360 + 0.5\,D \times 2D\,) = 168.288 \quad \ldots\ldots\ldots \text{ eq. (1)}$$

$$BD + 2.464\,D^2 = 168.288$$

$$P = B + 2\,(\,\pi D \times 26.57/180 + 2D\,) = 58.441$$

$$B + 2\,D(\,\pi \times 26.57/360 + 2\,) = 58.441$$

$$B + 4.927\,D = 58.441$$

$$B = 58.441 - 4.927D \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad \text{eq. (2)}$$

Substituting eq. (2) to eq. (1) :

$$BD + 2.464\,D^2 = 168.288$$

$$(58.441 - 4.927D)D + 2.464\,D^2 = 168.288$$

$$58.441D - 4.927\,D^2 + 2.464D^2 = 168.288$$

$$-2.464\,D^2 + 58.441D - 168.288 = 0$$

Using, $D = \dfrac{-b \pm \sqrt{b^2 - 4ac}}{2a}$

$$= \frac{-58.441 + \sqrt{58.441^2 - 4x(-2.464x - 168.288)}}{2x(-2.464)} = 3.354\,m$$

$B \times 3.354 + 2.464 \times 3.354^2 = 168.288$

$B = 41.911$ m

Hydraulic radius,

$$V = \sqrt{\frac{2}{5} f.R} \quad \text{or} \quad R = 5/2 \times V^2/f = 5/2 \times (0.8995)^2/0.7 = 2.890 \text{ m}$$

also, $R = A/P = \dfrac{BD + 2\,(\pi D^2\ 26.57/360 + D^2)}{B + 2\,(\pi D\ 26.57/180 + 2D)}$

$$= \frac{(41.911 \times 3.354) + 2\,(\pi \times 3.354^2 \times 26.57/360 + 3.354^2)}{41.911 + 2\,(\pi \times 3.354 \times 26.57/180 + 2 \times 3.354)}$$

= 2.880 m (Hence checked)

Bed slope,

$$S = \frac{f^{5/3}}{3340 \times Q^{1/6}} = \frac{(0.7)^{5/3}}{3340 \times (151.375)^{1/6}} = 0.000072\ (1 : 13889)$$ say 1 in 13800.

Discharge,

$Q = A \times V = 168.2879 \times 0.8995 = 151.375\ m^3/s$

Thus Lacey's theory gives $S = 1/13800$, $V = 0.90$ m/s, $B = 41.91$ m, $D = 3.35$ m

3.8.5. Design by Tractive Force Method.

Water properties : Specific weight of fluid, $\gamma_f = 1$ ton/m^3.

Sediment properties : Diameters of sediment, $\mathbf{d_{75}} = 0.165$ mm (the diameter of 75% (by weight) is finer sediment particles from figure 3.5.

Permissible shear stress,

$$\tau_L \text{ (from table 2.2 by interpolaion)} = \frac{(0.15 - 0.1)}{(0.2 - 0.1)} x\,(0.250 - 0.241) + 0.241$$

$$= 0.247 \text{ kg/m}^2$$

Limiting Tractive force,

$\tau_L = \gamma_f D.S$ or $D = \tau_L / \gamma_f S$

$= (0.247 \times 12000) / 1000 = 2.964$ m , (say 3.00 m)

Using **Manning equation** and continuity equation,

$$Q = \frac{1}{n} A R^{2/3} S^{1/2}$$

$$= \frac{1}{0.017} A R^{2/3} (1/12000)^{1/2} \quad \text{..............(i)}$$

Area of cross-section,

$$A = B \times D + 2 (\pi D^2 \phi / 360 + 0.5 D \times 2D)$$

$$= 2.964 B + 2 (\pi \times 2.964^2 \times 26.57/360 + 2.964^2) = 2.964 B + 21.645$$

Wetted parameter of cross section,

$$P = B + 2 (\pi D \phi / 180 + 2D)$$

$$= B + 2 (\pi \times 2.964 \times 26.57 / 180 + 2 \times 2.964) = B + 14.605$$

Hydraulic radius,

$$R = \frac{A}{P} = \frac{2.964 B + 21.645}{B + 14.605}$$

Substituting in equation (i)

$$151.375 = \frac{1}{0.017} (2.964 B + 21.645) \times [(2.964 B + 21.645) / (B = 14.605)]^{2/3} \times (1/ 12000)^{1/2}$$

B is the only unknown, solving **by trial** B = 43.14 m

$A = 2.964 B + 21.645 = 2.964 \times 43.14 + 21.645 = 149.512$ m^2

$P = B + 14.605 = 43.14 + 14.605 = 57.745$

$R = \frac{A}{P} = 149.512 / 57.745 = 2.589$ m

$$V = \frac{1}{0.017} (2.589)^{2/3} (1/12000)^{1/2} = 1.013 \text{ m/s}$$

$Q = A \times V = 149.512 \times 1.013 = 151.381 \text{ m}^3/\text{s}$

Tractive force theory gives, for S = 1/12000, B = 43.14 m and D = 3.0 m

A comparison of canal sections of Tractive Force Method with Lacey's Method is as under :

1) Tractive force's velocity is 1.013 m/s greater than Lacey's velocity i.e 0.899 m/s

2) Tractive force's area of cross section of 149.512 m^2 less than Lacey's area of cross section i.e 168.288 m^2

4) Cross section of tractive force by A =149.512 m^2, B = 43.14 m, B_T = 56.93 m, D = 2.96 m, B/D = 14.57 and Cross section of Lacey by A =168.288 m^2, B = 41.63 m, B_T = 56.00 m, D = 3.35 m, B/D = 12.51 is straighly different.

Tractive force theory can be composed with the Lacey's theory and designs very near.

These theories may be good to the extent that no soil particales of the canal will move forward. But raises a very a good question that how much of silt/sediment coming into the canal will settle or transported.

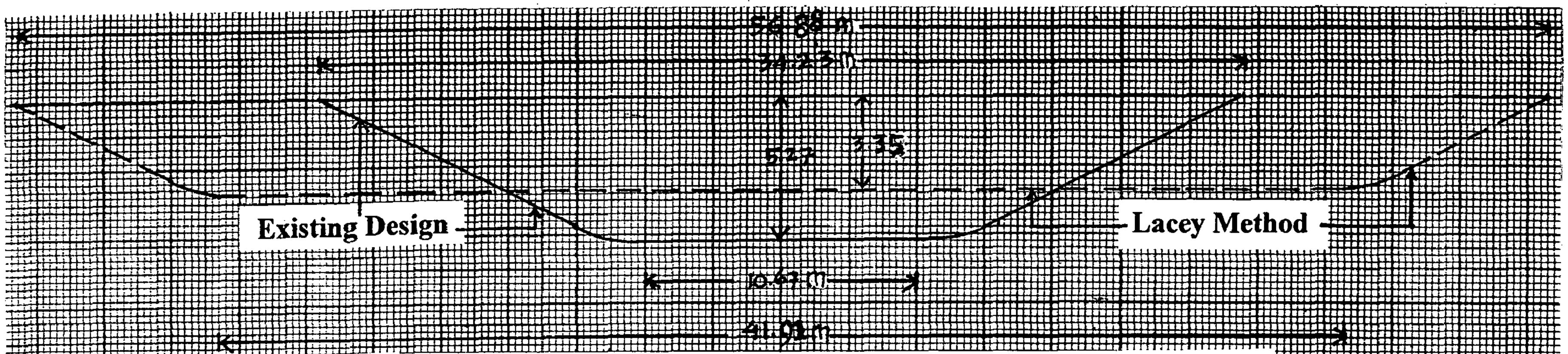


Fig. 3.16 Comparison of Cross Sections of IGMC for Existing Design and Lacey Method Between Km 384 to Km 410 (RD 1260 to RD 1345)

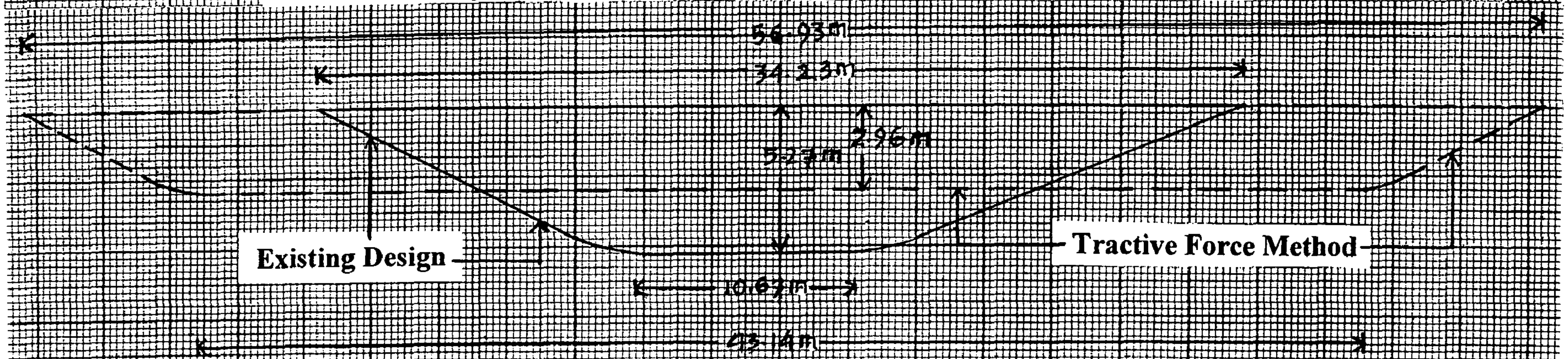


Fig. 3.17 Comparison of Cross Sections of IGMC for Existing Design and Tractive Force Method Between Km 384 to Km 410 (RD 1260 to RD 1345)

Table 3.17

Comparison of Sections of Project Design with Lacey Method and Tractive Force Method

Parameter	Design		
	Existing	Lacey Method	Tractiv Force Method
Q	151 m^3/s	151 m^3/s	151 m^3/s
D	**5.27 m**	**3.37 m**	**2.964 m**
B	10.67 m	41.63 m	43.14 m
A	124.627 m^2	168.288 m^2	149.512 m^2
R	3.402 m	2.890 m	2.589 m
n	1 in 12000	1/13800	1/12000
S	0.017	0.017	0.017
P	36.634 m	58.441 m	57.745 m
V	1.215 m/s	0.899 m/s	1.013 m/s
T	34.23 m	56.70 m	56.93 m

CHAPTER 4

SUMMARY AND COCLUSIONS

(1) A comparative study of bed width and depth of 20 lined canals in India is made in chapter 1. Their hydraulic and section particulars are given in Table 1.1. Bed width versus discharge and depth versus discharge is plotted in Figure 1.1, that shows very wide variations. Also a comparison of these sections is shown in Figure 1.2, where in all section are plotted on the same scale. There is a wide variations in adopted depth and B/D ratio, even for identical slopes and value of rugosity coefficient n.

This leads to search out the criterias for adoption of B/D ratio.

(2) Sediment concentration at various points in a river, and its transport in a river/canal are discussed in chapter 2.

(3) Modes of sediment transport are also discussed in this chapter. All the empirical equations of sediment transport are given in Table 2.4 to 2.6 as under:

(i) Bed load sediment transport - Table 2.4

(ii) Suspended sediment transport – Table 2.5

(iii) Total sediment transport – Table 2.6

(4) Graff (1998) has shown a wide variation in the results of above formulas from less than 50% to 200% i.e. 4 time variations. It is very difficult to make a judicious choice of suitable method.

(5) (a) Various regime formulas given by Kennedy (1895), Lacey (1934) and many others are listed in Table 2.11.

(b) Tractive force theory is also discussed in this chapter in para 2.6.

(6) Manni ng's Equation most widely used now, and a very useful derivation of it for the most hydraulic efficient section i.e. when R = D/2, velocity becomes, $V = 0.63\, d^{2/3} S^{1/2} / n$ is given at the end of regime theories.

(7) A comparison of Manning's, Kennedy's and Lindley's formula is made in para 2.12. It can be seen from figure 2.39 that velocity by Kennedy's formula is very near to the velocity by Manning's formula for a constant $S^{1/2}$ and n ratio (say 1). The Kennedy's formula gives velocity between Lindley's and Manning's formula.

It is well recognised that velocity changes with slope and n. But Manning has not examined sediment transport or regime velocity. Therefore the two equation can be very conveniently used.

(8) Lastly an attempt is made to examine the applicability of all theories to Indira Gandhi Main Canal. This canal and applicability to different flow conditions are discussed in chapter 3. Results of study are also simultaneously analised in that chapter and a summary is given below.

(i) Indira Gandhi Canal is a major/canal 450 Km long to carry 524 m^3/s at head to 136 m^3/s at tail (Figure 3.2). It passes through sandy desert, alternate heavy fillings and cuttings (Figure 3.4 and 3.5). Wind blown sand (silt) enters through out the entire length of the canal more in cutting reaches. Thus silting is a major problem. It is reported in revised estimates that more silt and weeds are a common phenomenon in the entire canal.

It is a lined canal with inner side slopes of 2:1, through out the length. Velocity varies from 1.52 m/s at head to 1.18 m/s at tail (Figure 3.2).

A reach between Km 348 to Km 410, (upstream of a cross-regulator at Km 410) is analised in this dissertation.

(ii) Impact of cross-regulator, development of flow rating curve, backwater surface profiles, and velocities are discussed from para 3.3 to 3.5. Results are plotted in figure 3.6 and 3.7.

To run the canal at 2/3 discharge (most often) and maintain FSL at cross-regulator, the backwater curve is 38.83 Km long, and velocity reduces from 1.06 m/s to 0.8 m/s. Practically very low velocity.

(iii) Sediment transport in the canal are computed by Einstein's and Ackers method for temperature at 20^0 C and 32^0 C (para 3.6). Wind blown sand classification is shown in Figure 3.8. d_{50} soil is 0.15 mm size.

(iv) A gauge- discharge and sediment discharge curve for uniform flow is prepared and shown in Figure 3.9. Einstein's method (Table 3.5) gives a sediment transport of 332% more than that by Ackers method, (Table 3.6 and 3.7) for full supply discharge.

Sediment transport for gradually varied flow is also shown in Table 3.7. Because of the gradually varied flow velocity reduces and sediment transport is also decreasing from 389 m^3/day to 250. 389 m^3/day.

(v) Because of silt and weed growth, canal performance is also examined for n = 0.02, 0.025, and 0.03 (Table 3.9). Results shown in Figure 3.10 indicate a loss of canal capacity by from n = 0.017 to n = 0.03.

Its effects on backwater curve and velocity are also shown in Figure 3.11 for n = 0.025. Thus the supply downstream is greatly effected.

(vi) Now alternative hydraulic design and comparative analysis is made for Kennedy's, Lindley's, Lacey's formula and Tractive force theory for various B/D (bed widh : depth) ratio as below :

(A) Comparison of canal sections for the same bed slope of 1 in 12000 and side slope Z = 2:1 (H:V) (as per project) for various B/D (bed width : depth) ratio with Kennedy's critical velocity is made in Table 3.11 and show in Figure 3.13.From this Table it is observed as under :

1) Kennedy's velocity decreases from 1.808 m/s at X = 0.5 to 0.935 m/s at X = 30 .

2) Kennedy's critical velocity ratio (m) increases very slowly from 0.685 at X = 0.5 to 0.950 at X = 30 where the velocity are considerably reduced.

3) Manning's velocity decreases from 1.239 m/s X = 0.5 to 0.888 m/s at X = 30. **Thus B/D ratio (X) has a very Crucial role in design.**

4) Froud number is gradually (very little) increasing from 0.2022 at X = 0.5 to 0.2033 at X = 1.7 and then gradually decreases to 0.1931.

5) Thus the concept of critical velocity ratio of m ≥ 1 for the given bed slope gives very large bed width and uneconomical section, and also does not increase the sediment transport capacity.

6) For the project design, critical velocity ratio (CVR or m) is 0.7625 < 0.8. much less than 1

7) At X = 1.6 or say 1.7 the depth of flow is 5.519 m, and 5.458 m, and velocities are 1.223 m/s and 1.222 m/s and F_r is also near to 0.20.

There appears little justification for increasing X greater then 1.5 or say 1.6. There after, neither depth decreases rapidly, nor critical velocity ratio increases rapidly. Therefore it is not desirable in this case to achieve criteria of IS 10430- 2000.

8) A comparison of two sections is shown in Figure 3.12 and Table 3.15. **Section B appears better then A.**

(B) Comparison of canal sections for the bed slope of 1 in 5000 and side slope Z = 2:1 (H:V) for various B/D (bed width : depth) ratio with Kennedy's critical velocity is made in Table 3.12 and shown in Figure 3.14. From this Table it is observed as under :

1) Kennedy's critical velocity ratio (m) is slightly greater than 1, even at X = 0.5.

More detailed analysis is given in para 3.8.4 and therefore not repeated here.

A gain there appears little justification for increasing X greater 0.5. There after neither depth decreases rapidly, nor critical velocity ratio increases rapidly.

(C) Comparison of canal sections for the bed slope of 1 in 6300 and side slope Z = 2:1 (H:V) for various B/D (bed width : depth) ratio with Lindley's velocity, see Table 3.14 and Figure 3.14. From this Table it is observed as under :

1) Lindley's velocity decreases from 1.528 m/s at X = 0.5 to 0.923 m/s at X = 20.

More detailed analytical comments are given in para 3.8.4.2 and there not repeated here.

Again there appears little justification for increasing X greater than 0.5.

(D) Comparison of canal sections for the same bed slope of 1 in 12000 (as per project design) and side slope Z = 1.5:1 (H:V) for various B/D (bed width : depth) ratio with Kennedy's critical velocity is made in Table 3.14 and Figure 3.14. From this table it is observed as under :

1) Kennedy's velocity decreases from 1.864 m/s at X = 0.5 to 1.028 m/s at X = 20.

More detailed analytical comments are given in para 3.8.4.3 and hence not repeated here.

Main Conclusion :

Thus for flat slope of 1 in 12000, a side slope of 1:1.5 in more desirable.It gives D = 5.423 m say 5.43 m, X = 2.0, B = 10.85 m, Manning's V = 1.26 m/s and Kennedy's V = 1.623 m/s, and Kennedy's critical velocity ratio of 0.776, though still less than 1.

In general it may be concluded that Kennedy's formula $V = 0.546\ m\ D^{0.64}$ gives little higher velocity than that by Manning's formula for best section $V = 0.63 d^{2/3} S^{1/2} / n$ (treating $S^{1/2}/n \approx 1$, see para 2. 11.1) and are more suitable to carry or transport the sediment entering into canal. Velocity by Kennedy's formula can be need to compute slopes with the help of Manning's formula. For this slope minimum B/D ratio and steepest possible side slopes needs to be adopted.

A comparison of all the sections is shown in figure 3.15. For the project slope of 1 in 12000, a side slope of 1:1.5 (V:H) is much better, otherwise change in slope is desirable.

(E) Comparison of canal sections by Tractive Force Theory with Lacey's Method :

1) Tractive force velocity is 1.013 m/s, greater than Lacey's velocity i.e 0.899 m/s

2) Tractive force's area of cross section is 149.512 m^2, less than Lacey's area of cross section i.e 168.288 m^2

3) Cross section by tractive force theory is B = 43.14 m, T = 56.93 m, D = 2.96 m, B/D = 14.57 and Cross section by Lacey theory is B = 41.91 m, T = 56.88 m, D = 3.35 m, B/D = 12.51. Difference in two is small. But the difference with (a) to (d) is very large.

(9) (i) The purpose of alternative is not to reconstruct the canal, but only to explain the effect of B/D ratio in canal design. Also it is comparative with the critical velocity ratio given by Kennedy , Lindley method , and Lacey method.

(ii) For side slope from 1.5:1 (H:V)to 2:1 (H:V) top width increases by 15% and perimeter by 11%. Velocity also reduce by 4.2% and decrease in depth is normally 5%, This thesis explains the role of B : D ratio and side slopes. Both should not arbitrarily choosen but should be adopted very judiniously.

(iii) In contrast above for B = D and with the side of 2:1 (H:V), the decrease in depth by 1.4%. Though increase in bed width by 8.5%. Yet the top width increase by 0.4% and perimeter by 0.3%. Velocity remains same. Therefore it is economical to increase bed width the side slope.

(iv) For flatter side slope there is a good different in all dimension i.e. depth, perimeter, top width. It reduce velocity also.

(v) With increase in bed width beyond B = D, there is drastic change in depth, top width, and perimeter. The difference in velocity is nominal. There appears little justification to increase bed width beyond B = D.

(Vi) For a depth of 5.27 m, a bed slope should be avoid. A bed slope of Kennedy's formula i.e. 1/5000, Lindley's formula i.e. 1/6300, Lacey's i.e. 1/13800.

(10) Such elementary comparison serves to focus attention to the end results and accordingly promote further research into practical aspect of the subject, with a view to economical and efficient design. Such more studies are required in view of large inter basin link canals.

(11) The aspects that could not be attempted in this thesis are the concept of non silting and non scouring which can be devided into two parts:

(i) Clear water enters the canal, no sediment enters at any point there after, then velocity should not erode banks and bed, i.e.non scouring. There is no question of silt deposit. Such situation is rare.

(ii) Sediment is entering into canal – (a) from source (b) from drainage inlets or winds (c) inner bank failures or rain water erosion in cuttings above FSL.

This sediment concentration may be varying, Also discharge may vary according to availability of water. Then equilibrium concept over a year i.e. some sediment deposit during higher concentration time, erosion during low concentration time and a balance over a year may be come important. Typical example is Upper Ganga Canal off taking from Hardwar. More studies in this respect are need in actual canal flow condition.

REFERENCES

1. ASCE M&R No.54 (1975), Sedimentation Engineering, Report of America Society of Civil Engineers Task Committee for the Manual on Sedimentation of The Sedimentation Committee , New York.
2. CBIP (1966), Sediment Control in Rivers and Canals, Report of Central Board of Irrigation and Power, Govt. of India, New Delhi.
3. CBIP (1967), Symposium on Canal Lining, Report of Central Board of Irrigation and Power, Govt. of India, New Delhi.
4. CBIP (1986), Major Canal in India , Report of Central Board of Irrigation and Power, Govt. of India, New Delhi.
5. Chow,V.T (1973), Open – Channel Hydraulic, Mc. Graw – Hill Book Company Ltd.
6. Cheng (1985), Fluids Mechanics with Engineering Applications, Mc. Graw – Hill Book Company Ltd.
7. Chow, V.T. (1964), Handbook of Applied Hydrology, Mc. Graw-Hill Book Company Ltd.
8. Davis, C.V. (1952), Handbook of Applied Hydraulics, Mc. Graw-Hill Book Company Ltd.
9. Framji, K.K. (1972), Design Practices of Irrigation Canals in The World, Report of International Commission Des Irrigation EC DU Drainage, New Delhi.
10. Graf, W.H. (1971,1984), Hydraulics of Sediment Transport, McGraw-Hill, Publishing Company Ltd.
11. Garde. R.J. and Ranga Raju, K.G. (2000), Mechanics of Sediment Transportation and Alluvial Stream Problems, K.K. Gupta, Publishing Company Ltd.
12. Modi, P.N (2000), Irrigation Water Resource and Power Engineering, Rajinder Kumar Jain, Publishing Company Ltd.
13. Mohapatra. S (1972), Design of Stable Channel in Alluvium, M.E. dissertation, WRDTC, University of Roorkee.

14. Mirajgaoker, A.G. (1957), Design of Stable Channels In Alluvial Soils, M.E. dissertation, WRDTC, University of Roorkee.
15. Prasad, V (1975), A Review of Canal Linings in India,M.E. dissertation, WRDTC, University of Roorkee.
16. Sharda, C.B. (1968), Lining of Canals With Special Reference of Rajasthan Canal M.E. dissertation, WRDTC, University of Roorkee.
17. Safi, A.G.(1975), A Review on Design of Stable Channel by Empirical Formula M.E. dissertation, WRDTC, University of Roorkee.
18. Sharma, K.R., Irrigation Engineering, D.V. Sharma, Publishing Company Ltd.
19. Singh, B. (1982), Fundamental of Irrigation Engineering , N.C. Jani, Publishing Company Ltd.
20. Varshney (2000), Theory and Design of Irrigation Structures, Nem Chand & Bros Civil Lines, Publishing Company Ltd.

ANNEXURE – 1

Physical Properties of Water in SI Units

Temperature, ^0C	Specific Weight γ, kN/m^3	Density ρ, kg/m^3	Viscosity $\mu \times 10^3$, $N.s/m^2$	Kinematic. Viscosity $v \times 10^6$, m^2/s	Surface Tension σ, N/m	Vapor Pressure P_v, kN/m^2,abs	Vapor Pressure Head P_v/γ, m	Bulk Modulus of Elasticity $E_v \times 10^{-6}$, kN/m^2
0	9.805	999.8	1.781	1.785	0.0756	0.61	0.06	2.02
5	9.807	1000.0	1.518	1.519	0.0749	0.87	0.09	2.06
10	9.804	999.7	1.307	1.306	0.0742	1.23	0.12	2.10
15	9.798	999.1	1.139	1.139	0.0735	1.70	0.17	2.14
20	9.789	998.2	1.002	1.003	0.0728	2.34	0.25	2.18
25	9.777	997.0	0.890	0.893	0.0720	3.17	0.33	2.22
30	9.764	995.7	0.798	0.800	0.0712	4.24	0.44	2.25
40	9.730	992.2	0.653	0.658	0.0696	7.38	0.76	2.28
50	9.689	988.0	0.547	0.553	0.0679	12.33	1.26	2.29
60	9.642	983.2	0.466	0.474	0.0662	19.92	2.03	2.28
70	9.589	977.8	0.404	0.413	0.0644	31.16	3.20	2.25
80	9.530	971.8	0.354	0.364	0.0626	47.34	4.96	2.20
90	9.466	965.3	0.315	0.326	0.0608	70.10	7.18	2.14
100	9.399	958.4	0.282	0.294	0.0589	101.33	10.33	2.07

Source: Change (1985). Fluid Mechanics with Engineering Applications, 8th ed., McGraw Hill, New York.