

Government of India

CENTRAL WATERPOWER IRRIGATION & NAVIGATION
RESEARCH STATION, POONA.



RESEARCH PUBLICATION No. 13

**“THE BEHAVIOUR AND CONTROL
OF RIVERS AND CANALS
(WITH THE AID OF MODELS).”**

Part I

BY

Sir Claude Cavendish Inglis, C.I.E., B.A., B.A.I., M.I.C.E.,
M. Am.Soc.C.E., M.I.E. (I).

(In charge of Hydrodynamic Research at Poona from 1920-45 ; Director of the Indian Waterways Experiment Station from 1936-45 ; appointed Director, Hydraulic Research Organisation of the Department of Scientific and Industrial Research, London, England, March 1947).



POONA

PRINTED AT THE YERAVDA PRISON PRESS, 1949.

D26,8:4:p.2
H9.1
057936

Price: Rs. 16-8-0 or 25 sh. 6 d.
(The price is inclusive of price of Part II.)

IWE. 3-1
2,000

Government of India

CENTRAL WATERPOWER IRRIGATION & NAVIGATION
RESEARCH STATION, POONA.



RESEARCH PUBLICATION No. 13

**“THE BEHAVIOUR AND CONTROL
OF RIVERS AND CANALS
(WITH THE AID OF MODELS).”**

Part I

BY

Sir Claude Cavendish Inglis, C.I.E., B.A., B.A.I., M.I.C.E.,
M. Am.Soc.C.E., M.I.E. (I).

(In charge of Hydrodynamic Research at Poona from 1920-45; Director of the Indian Waterways Experiment Station from 1936-45; appointed Director, Hydraulics Research Organisation of the Department of Scientific and Industrial Research, London, England, March 1947).

POONA

PRINTED AT THE YERAVDA PRISON PRESS, 1949.

**List of Research Publications of Central Waterpower Irrigation and Navigation
Research Station, (formerly Indian Waterways Experiment Station, Poona)
available for sale with the Manager of Publications, Delhi, and Agents**

Symbol No.	Research Publication No.	Subject.	Price.	Year of Publication.
I.E.R. 1-38 ...	1	Annual Report of work done during 1937-38 by C. C. Inglis, A. R. Thomas, Rao Sahib D. V. Joglekar and staff at Khadakwasla.	Rs. a p. 7 8 0	1939
I.E.R. 1-39 ...	2	Annual Report of work done during 1938-39 by same authors as in 1.	7 8 0	1940
I.E.R. 2 ...	3	Note on "The theory, design and construction of Gibb Modules which, without moving parts, give a constant discharge within working limits irrespective of variations in upstream and downstream water levels" by C. C. Inglis and Rao Sahib D. V. Joglekar.	8 8 0	1941
I.E.R. 1-40 ...	4	Annual Report of work done during 1939-40 by C. C. Inglis, A. R. Thomas, Rao Sahib D. V. Joglekar and staff at Khadakwasla.	7 8 0	1941
I.E.R. 1-41 ...	5	Annual Report (Technical) of work done during the year 1940-41 and index for 1937-41 by C. C. Inglis, A. R. Thomas, Rao Sahib D. V. Joglekar and staff at Khadakwasla.	7 8 0	1942
I.E.R. 1-42 ...	6	Annual Report (Technical) of work done during the year 1941-42 by C. C. Inglis, J. S. Reid, Rao Bahadur D. V. Joglekar and staff at Khadakwasla.	7 8 0	1944
I.E.R. 1-43 ...	7	Annual Report (Technical) of work done during the year 1942-43 by C. C. Inglis, J. S. Reid, Rao Bahadur D. V. Joglekar and staff at Khadakwasla.	7 8 0	1945
I.E.R. 1-43 ...	8	Annual Report (Technical) of work done during the year 1943 by C. C. Inglis, Rao Bahadur D. V. Joglekar and staff at Khadakwasla.	7 8 0	1945
I.E.R. 1-44 ...	9	Annual Report (Technical) of work done during the year 1944 by C. C. Inglis, Rao Bahadur D. V. Joglekar and staff at Khadakwasla.	7 8 0	1945
I.W.E. 1-45 ...	10	Annual Report (Technical) of work done during the year 1945 by Rao Bahadur D. V. Joglekar and staff at Khadakwasla.	25 0 0	1948
I.W.E. 1-46 ...	11	Annual Report (Technical) of work done during the year 1946 by K. K. Framji, Rao Bahadur D. V. Joglekar and staff at Khadakwasla.	7 8 0	1947
I.W.E. 1-47 ...	12	Annual Report (Technical) of work done during the year 1947 by K. K. Framji, Rao Bahadur D. V. Joglekar and staff at Khadakwasla.	25 0 0	1948

**THE BEHAVIOUR AND CONTROL OF RIVERS AND CANALS
(WITH THE AID OF MODELS)**

CONTENTS



	Page.
SYMBOLS AND DEFINITIONS.	... 1
CHAPTER 1.—Introduction	... 35
CHAPTER 2.—Review of factors controlling flow in alluvial channels	... 85
CHAPTER 3.—Historical note on regime flow in channels in alluvium and divergence from regime.	95
CHAPTER 4.—Meandering of rivers.	... 143
CHAPTER 5.—Behaviour of rivers—as exemplified by the Indus in Sind	... 167
CHAPTER 6.—Exclusion of excess bed material from canals and distributaries	... 217
CHAPTER 7.—Factors affecting flow in alluvial channels—as exemplified by the dimensions of canals in Sind.	233
CHAPTER 8.—Maximum depth of scour at heads of guide banks and groynes, pier noses, and downstream of bridges.	327
CHAPTER 9.—The design of falling aprons	... 355
CHAPTER 10.—Control of rivers by guide banks and groynes	... 379
CHAPTER 11.—Fluming	... 387
CHAPTER 12.—Points worth considering when designing and remodelling canal and river works.	407
CHAPTER 13.—Methods followed in fixing model scale ratios and overcoming model limitations, based on experience gained at the Indian Waterways Experiment Station, Poona.	427
APPENDIX 1.—List of Bombay P. W. D. Technical Papers and Annual Reports of the I. W. E. S., Poona.	453
APPENDIX 2.—Bed ripples and bed dunes	... 459
APPENDIX 3.—Factors affecting exclusion of bed sand from canals taking off from alluvial rivers.	471
APPENDIX 4.—Equivalent size and specific gravity of stones and blocks in a model.	485
INDEX	

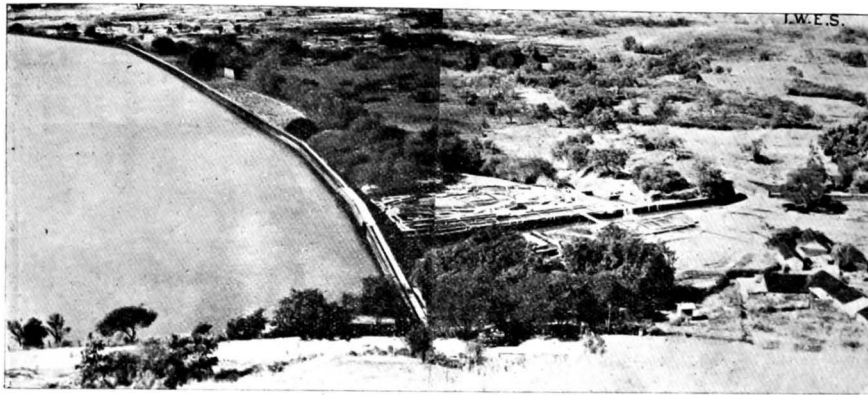


Photo 1690. A composite photo taken from the top of the hill above Khadakwasla Dam (Lake Fife) showing a portion of the upper area of the Research Station just downstream.

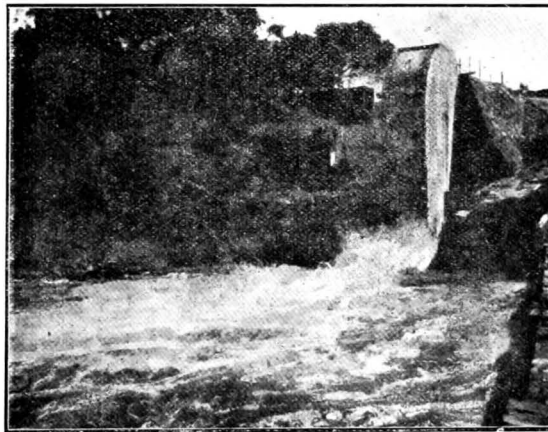


Photo 328. Full scale siphon spillway model—5 ft. x 4 ft.
—discharging 520 cusecs, for comparison
with 1/8th scale model.

[Photo 1690 from sheet 1, A. R. (T.) 1937-38.]

[Photo 328 from sheet 3, A. R. (T.) 1937-38.]

THE BEHAVIOUR AND CONTROL OF RIVERS AND CANALS

SYMBOLS AND DEFINITIONS



Much of the confusion regarding the complex problems relating to river flow is due to lack of precision in terminology, with consequent confusion of thought.

Take for example the expression "silt" and "silting". These have come to be applied to any material whether silt, sand, pebbles or boulders; yet the process of accretion of each of these materials is widely different.

There are three distinct ranges of behaviour of material:

Where V_s (the terminal velocity of fall of particles in water)—		In this Report called.
$\propto m^2$...	silt
$\propto m$...	sand
$\propto m^{\frac{1}{2}}$...	coarse sand, gravel, pebbles or boulders,

m being the effective mean diameter of particles in millimetres—unless specified otherwise.

The term "silting" should, therefore, only be used where accretion results from deposition of silt; and we would be wise to take the plunge and say "silting", "sanding", "bouldering"—for clarity. Erosion should only be used for reference to bank erosion; and scour, in connection with bed scour—because bank erosion and bed scour are essentially different processes, about which there must be no slipshod thinking.

Another example of confused terminology arises over the use of the terms "regime" and "stable".*

Originally the term "stable" was applied to any channel in alluvium which did not change appreciably from year to year; while a channel in good condition was said to be "in good regime". Lacey then used "regime" to define "the ideal dimensions to which channels flowing with constant discharge and natural silt charge tended to develop" which was assumed to conform with "statistical mean values based on average conditions". These, Lacey held—I think rightly—were fundamental relationships.

Provided the original meanings were adhered to, there would be no confusion and little disagreement; but when the meaning of "regime" was altered to include what we had hitherto called "stable"—on the grounds that a rigid channel is stable, but is not "in regime"—"regime" came to be applied in a foggy sort of way, being sometimes applied to "perfect regime"—i.e. "equilibrium"—the ideal dimensions to which a channel flowing with constant discharge and natural silt charge tends to develop, and at other times to all channels which lay, undefined between "in equilibrium" and "stable".

Now Lacey has gone one step further: †

"A regime channel is a stable channel transporting a regime silt charge."

"A regime silt charge is the minimum transported load consistent with a fully active bed. Full activity is such that any reduction would lead to effective partial rigidity, and at the limit to complete rigidity and immobility of the bed."

* "Divergence from regime in stable channels in alluvium". By C. C. Inglis. C. I. and H. R. S. A. R. (T) 1941-42.

† "A General Theory of flow in alluvium" Paper No. 5518, Procs. Inst. C. E. November 1946.

THE BEHAVIOUR AND CONTROL OF RIVERS AND CANALS

This is a knife-edge condition, which rarely, if ever, occurs in nature; so we must retrace our steps, using equilibrium for the ideal condition towards which a channel is ever tending to develop, stability being applied to a channel which does not change appreciably from year to year.

- Q (i) Discharge in cusecs.
 q (ia) Discharge per ft. width.
 A (ii) Area of cross section.
 V (iii) Mean velocity = Q/A .
 W (iv) A length characterising the effective width of an alluvial river—unless otherwise stated.
 D (v) A length characterising the effective depth of an alluvial channel—unless otherwise stated.
 P (vi) Wetted perimeter.
 R (vii) Hydraulic mean depth = A/P .
 w (viii) Surface width; or weight of a cu. ft. of water in lbs.
 d (ix) Average depth = A/w .
 B (x) Bed width.
 D_B (xi) Mean depth of bed portion of a channel.
 S (xii) Slope = energy gradient = $\frac{1}{L}(H_1 - H_2) + \frac{V_1^2 - V_2^2}{50}$
 where H_1 and H_2 are the heights of water above a common datum and V_1 and V_2 —the mean velocities at the upper and lower station. Where V_1 and V_2 are nearly equal, the difference of kinetic energy may be neglected, but where they are considerably different, it must be taken into account, and experience shows that where mean velocities are being used, 2g in the denominator is too high a value, 50 being a good approximation.
 S* (xiii) Slope per thousand.
 Rn (xiv) Reynold's number = $(VD)/\nu$.
 " (xv) Kinematic Viscosity = μ/ρ .
 μ (xvi) Viscosity.
 ρ (xvii) Density of water.
 g (xviii) Gravitational constant.
 Fr (xix) Froude number = V^2/gD in terms of depth.
 = V^2/gW in terms of width.
 f (xx) Lacey number = $1.155 V^2/R \propto g$.
 f_{SV} (xxi) = $48 \sqrt{SV} \propto (g\nu)^{1/6}$.
 N (xxii) Coefficient of rugosity—usually Manning's

$$N = \frac{1.4958}{V} R^{2/3} S^{1/2}$$

 Na (xxiii) Lacey's coefficient of rugosity

$$Na = \frac{1.346}{V} R^{2/3} S^{1/2}$$

 Qs (xxiv) "Load"—in lbs. per second—the weight of material in movement in a channel in lbs. per sec. or any other specified time.
 X (xxv) "Charge" = $Qs(\sigma - 1)/(Q \times w)$ is the weight in water of the load relative to the weight of water flowing.
 m (xxvi) Equivalent weighted mean diameter of particles exposed on the bed of a channel in millimetres unless specified otherwise.

INTRODUCTION

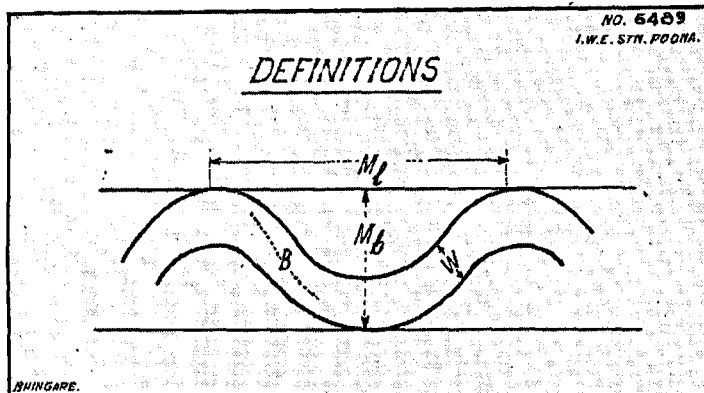
- Vs (xxvii) "Terminal velocity" is the rate of fall in ft./sec. of particles falling through still water. It is usually applied to the weighted mean value of a sample of particles of different sizes. Vs covers shape, size, specific gravity of the material and temperature of the water. If the Vs value of a sample is determined in a laboratory at a different temperature from the river temperature, the Vs value must be corrected to river temperature. Though Vs eliminates the individual effects of temperature, shape, and specific gravity, it does not cover the effect of size relative to velocity gradient near the bed—that is to say a large particle of very low specific gravity with the same Vs value as a small particle of high specific gravity will behave quite differently; because the large light particle will project into a region of higher velocity. In the great majority of rivers, however, the specific gravity is 2.65—that of quartz, so this difficulty is generally avoided. The relation between Vs and m varies widely for different grades of material. At normal temperatures—
- Vs \propto m² for fine material < .08 mm.
 Vs \propto m (Approx.) for m .15 mm. to .50 mm.
 Vs \propto m^{1/2} for m > .6 mm.
- X.Vs (xxviii) "Weighted mean charge". More correctly this is the "Deposition Momentum"—i.e. the weighted mean terminal momentum of the charge which determines the rate of deposition of the particles and hence the slope of the channel. Vs is a parameter which covers shape, size, and temperature of the water; so it is easier to visualise (X.Vs) as the "weighted mean charge".
- (xxix) "Equilibrium" is the ideal condition towards which a channel is ever tending to develop and a channel is in equilibrium when the energy available due to the discharge and slope is just sufficient to carry the charge without any tendency for the channel to change its shape or slope; this entails every part of the cross section being in equilibrium. In nature, this is never fully attained.
- (xxx) "Stability"—a channel is said to be stable when, though it may change slightly at different times of year, as a result of conditions varying, there is no appreciable change from year to year—that is to say, accretion balances scour.
- (xxxi) "Dominant discharge" is the discharge at which equilibrium is most closely approached. It is associated with a dominant charge and slope.
- (xxxii) "Specific discharge Gauge" is the expression used in Sind for the Gauge reading corresponding to a particular discharge—such as 200,000, 300,000, 400,000 cusecs in the Indus; and is arrived at by drawing a smoothed—most probable—curve through the Gauge readings observed with discharges approximating to the specific discharges during (R) the rising flood season and (F) the falling flood season.
- (xxxiii) "Valley"—low land lying between hills; a low extended plain, usually watered by a river.

THE BEHAVIOUR AND CONTROL OF RIVERS AND CANALS

- (xxv) "River"—a large running stream of water.
- (xxv) "Channel"—the bed of a stream of water, the deeper part of a strait, bay or harbour.
- (xxvi) "Reach"—a straight portion of a stream between bends.

Notes.—These last four definitions are from Chamber's 20th Century Dictionary. "Thalweg"—so often used instead of "channel"—is incorrect as a definition and undesirable as a term. It means literally "valley way" which is not what we wish to describe: whereas "channel" means exactly what we want to convey, and is, therefore, used throughout this Report.

A "reach" differs from a "cross-over" in that whereas a "reach" refers to a long straight stretch of river, a "cross-over" refers to the relatively short length in which curvature of flow is reversed.



- (xxvii) A "meander" consists of two consecutive loops, one flowing clockwise, the other anti-clockwise. A meandering river follows a sinuous path due to natural physical causes not imposed by external restraint, and occurs where varying discharges and silt charges lead to curved flow and erosion of banks.
 - M_L (xxviii) "Meander length"—the tangential distance between corresponding points at extreme limits of successive fully-developed meanders.
 - M_b (xxix) "Meander belt"—the distance between lines drawn tangential to the extreme limits of successive fully developed meanders.
 - (xl) "Meander ratio"—the ratio of meander belt to meander length.
 - (xli) "Tortuosity"—the ratio of the length of the river channel to the axial length of the river.
 - (xlii) "Cross-over"—or "Crossing" (at B in Figure)—the portion of a river between bends in one of which flow is clockwise and in the other anti-clockwise.
 - (xliii) A "cut-off" channel is a chord channel which develops by short-cutting across the neck of a bend.
- "Cut-off" is also applied in Sind to the difference of water level upstream and downstream of a regulator, and is equal to the loss of head.

INTRODUCTION

- (xlv) "Cut-off ratio" is the ratio of the lengths of arc to chord of a meander at which a cut-off normally takes place. This holds for a developed meander, the arc and chord lengths of which are both $\propto Q^{\frac{1}{2}}$.
- (xlv) "Retgression of levels" generally refers to a drop of specific discharge Gauge reading—i.e. to a lowering of water level for the same discharge; but retrogression is also applied to retrogression of bed-levels.
- (xlv) "Afflux" is the upstream rise of water levels caused by an obstruction, such as a bridge or weir, or by regulation.
- (xlvii) "Heading up" is the rise of water level above a regulator due to regulation—as when gates are closed at a Barrage.
- (xlviii) "Ponding" is the condition when water is headed up temporarily above a regulator.
- (xlix) "Draw-down" occurs when, after water has been ponded, the regulator gates are opened and the downstream water level falls below what is normal, so that the slope of the water surface changes from concave to convex.
 - (l) "Accretion" is applied to any process of accumulation whether of silt, sand, pebbles, etc.
 - (lv) "Siltng" should be applied only to accretion of silt.
 - (lvi) "Sanding" should be applied only to accretion of sand.
 - (lvii) "Erosion" should be applied to bank-erosion.
 - (lviii) "Scour" should normally be applied to bed-scour.
 - (lv) "Blanketing" occurs where a layer of one type of material is overlaid by a layer of another type of material.
 - (lvi) "Vertical exaggeration" is the term in general use to describe the condition when the depth relative to length is greater in a model than it would be if the model were geometrically similar. This term assumes—unjustifiably—that the lateral and length scales are the same.
 - (lvii) "Slope exaggeration" is the exaggeration in a model slope relative to the prototype slope.
 - (lviii) "Tilt" is the degree of difference between slope exaggeration and vertical exaggeration; and is positive or negative according as the slope exaggeration is greater or less than the vertical exaggeration.
 - (lix) "Width contraction ratio"—a more precise and accurate term than "vertical exaggeration"—is the ratio of contraction of width to depth in a model.
 - (lx) "Length contraction ratio" is the ratio of length to depth contraction in a model.

The width contraction ratio may differ from the length contraction ratio; and either or both may differ—sometimes considerably—from slope exaggeration.

In Lacey regime, all are assumed equal. Divergence from regime leads to the width contraction ratio and slope exaggeration varying inversely.

CONTENTS

	PAGE.
CHAPTER I—INTRODUCTION	
Photo sheets 1-1 to 1-24	9
1.00. Object and scope of the Report	35
1.01. Experience on which Report is based	35
1.02. Facilities available for model research	36
1.03. Work done between June 1916 and 1929, by the Special Irrigation Division and the Irrigation Development and Research Circle	36
1.04. Modules, standing wave flumes, flume meter falls and the fluming of masonry structures	40
1.05. Problems investigated in connection with Sukkur Barrage and canals construction	40
1.06. The design of Falls—Tando Mastikhan Fall, Rohri Canal, Sind	40
1.07. Modification of downstream pavement of Sukkur Barrage	45
1.08. Design of piers	45
1.09. Enforcement of a standing wave downstream of the undersluices at Sukkur.	46
1.10. The best design of divide wall to exclude excess bed material from Canals	46
1.11. Long, curved, submerged, vane to prevent accretion in Right Bank Canals at Sukkur	46
1.12. Right Bank Approach Channel	51
1.13. Demonstration models	51
1.14. Effect of a cut-off demonstrated in Malaprabha demonstration model.	51
1.15. Bed-sand exclusion from distributaries taking off the Jamrao Canal at Mile 17	51
1.16. Sand channel experiments	52
1.17. Scale effect	52
1.18. Wave velocities as affected by channel shape	53
1.19. Small 'pilot' model and large model of the Ganges above and below the Hardinge Bridge	53
1.20. Model of the Sarda River at Banbassa	54
1.21. Model of the Watrak River near the town of Kaira, Bombay Presidency	54
1.22. Proposed bridge over the Brahmaputra at Amingaon	54
1.23. Model of the Gandak River at Bagaha	54
1.24. Experiments with a model of the Mahanadi and Katjuri Group of Rivers near Cuttack, Orissa	64
1.25. Models of the Hooghly above Calcutta	68
1.26. Rupnarain model investigation	73
1.27. Report on "Causes of river breaches in Sind and steps necessary to minimise the danger of a recurrence."	75
1.28. Report on "Causes of accretion in the bed of the Indus at Sukkur."	75
1.29. Design of Lower Sind Barrage	75
1.30. Other investigations dealt with in Annual Reports of the Poona Station.	75
1.31. The importance of model investigations after field inspection and analysis of data.	76
1.32. Acknowledgments.	77

CONTENTS

	PAGE.
CHAPTER I—INTRODUCTION	
Photo sheets 1-1 to 1-24	9
1.00. Object and scope of the Report	35
1.01. Experience on which Report is based	35
1.02. Facilities available for model research	36
1.03. Work done between June 1916 and 1929, by the Special Irrigation Division and the Irrigation Development and Research Circle ...	36
1.04. Modules, standing wave flumes, flume meter falls and the fluming of masonry structures	40
1.05. Problems investigated in connection with Sukkur Barrage and canals construction	40
1.06. The design of Falls—Tando Mastikhan Fall, Rohri Canal, Sind ...	40
1.07. Modification of downstream pavement of Sukkur Barrage ...	45
1.08. Design of piers	45
1.09. Enforcement of a standing wave downstream of the undersluices at Sukkur.	46
1.10. The best design of divide wall to exclude excess bed material from Canals	46
1.11. Long, curved, submerged, vane to prevent accretion in Right Bank Canals at Sukkur	46
1.12. Right Bank Approach Channel	51
1.13. Demonstration models	51
1.14. Effect of a cut-off demonstrated in Malaprabha demonstration model.	51
1.15. Bed-sand exclusion from distributaries taking off the Jamrao Canal at Mile 17	51
1.16. Sand channel experiments	52
1.17. Scale effect	52
1.18. Wave velocities as affected by channel shape	53
1.19. Small 'pilot' model and large model of the Ganges above and below the Hardinge Bridge	53
1.20. Model of the Sarda River at Banbassa	54
1.21. Model of the Watrak River near the town of Kaira, Bombay Presidency	54
1.22. Proposed bridge over the Brahmaputra at Amingaon	54
1.23. Model of the Gandak River at Bagaha	54
1.24. Experiments with a model of the Mahanadi and Katjuri Group of Rivers near Cuttack, Orissa	64
1.25. Models of the Hooghly above Calcutta	68
1.26. Rupnarain model investigation	73
1.27. Report on "Causes of river breaches in Sind and steps necessary to minimise the danger of a recurrence."	75
1.28. Report on "Causes of accretion in the bed of the Indus at Sukkur."...	75
1.29. Design of Lower Sind Barrage	75
1.30. Other investigations dealt with in Annual Reports of the Poona Station.	75
1.31. The importance of model investigations after field inspection and analysis of data.	76
1.32. Acknowledgments.	77

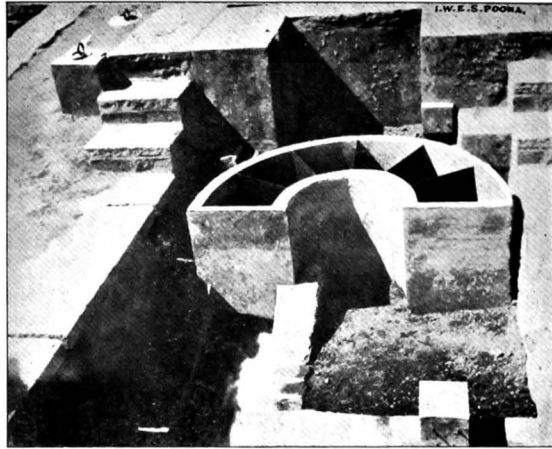


Photo 356. 3 cusecs Gibb module with no moving part—standard design—built in reinforced concrete. Gives constant discharge within working range of 1.4 ft.

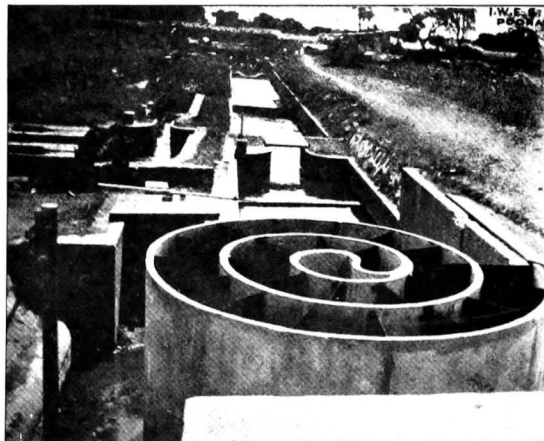


Photo 929. 1 cusec spiral module for dissipating extra head—built in "Nofrango."

(Photos from C. I. & H. R. S. Res. Publ. 3 "Note on the theory, design and construction of Gibb modules, which, without moving parts, give a constant discharge within working limits, irrespective of variations in upstream and downstream water levels.")

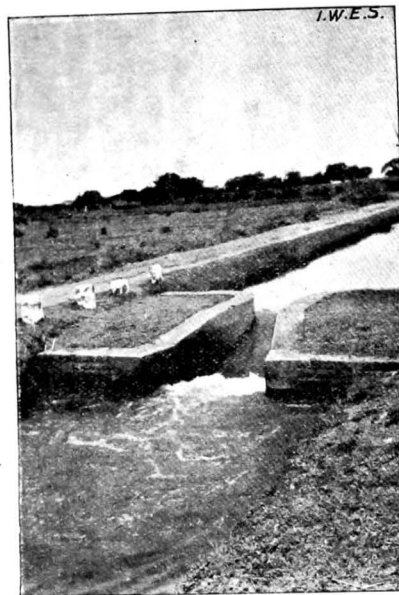


Photo 98. Flume type standing wave meter discharging 20 cusecs, looking upstream, Distributary 8, Nira Left Bank Canal.

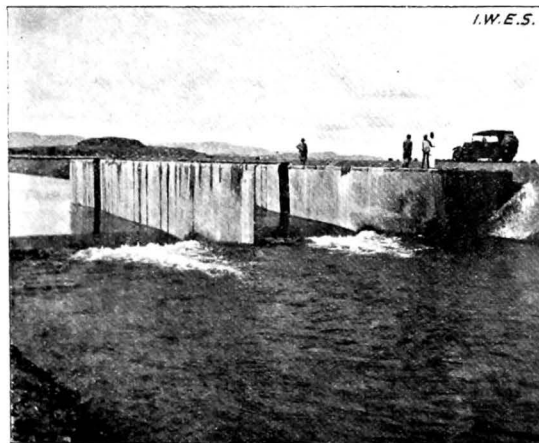


Photo 159. 1,750 cusecs standing wave flume meter at head of Nira Right Bank Canal. Designed for minimum loss of head—less than 10 per cent.

(Photos from Bombay P. W. D. T. P. No. 15)



Photo 662. 1/24th scale model of Tando Mastikhan Fall with 1 in 10 diverging sides. F. S. Q \cong 10,590 cusecs. Fall \cong 8.7 ft. showing downstream scour, which would have caused failure of pavement.

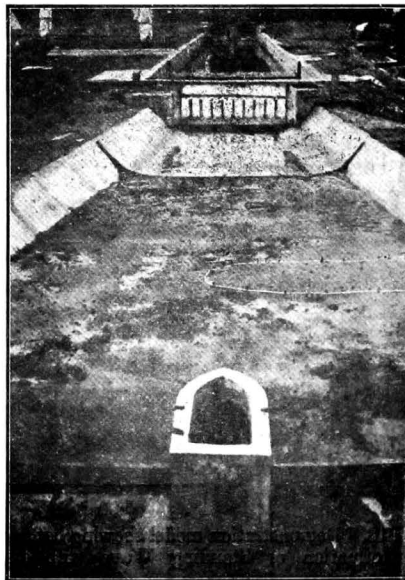


Photo 826. Scour eliminated by adding a baffle downstream of toe of fall and a deflector at end of pavement.
(Photos from Bombay P. W. D. T. P. No. 44.)

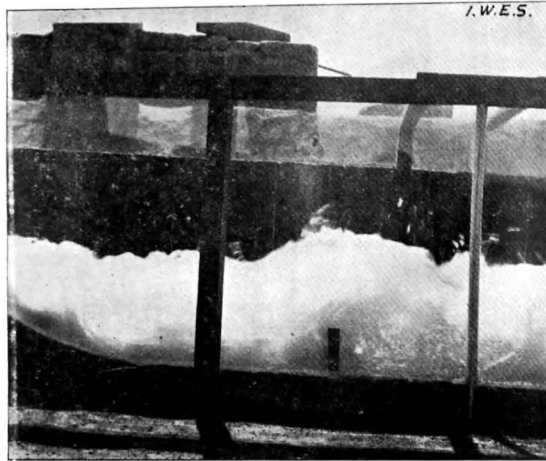


Photo 819. Demonstration model—showing baffle at $5\frac{1}{4}$ times the height of baffle from toe of fall acting effectively with optimum depth downstream.

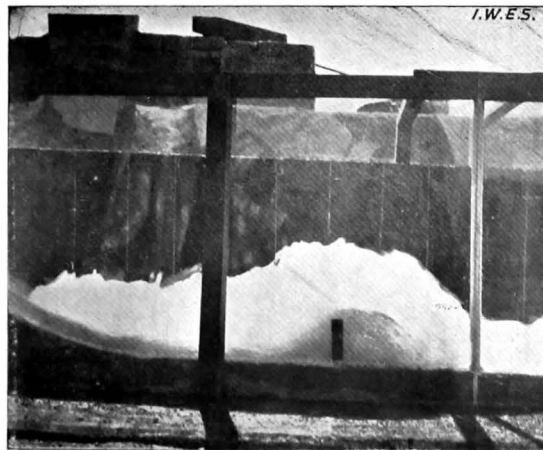


Photo 818. Demonstration model showing baffle causing formation of Standing Wave with free flow downstream.

(Photos from Bombay P. W. D. T. P. No. 44)

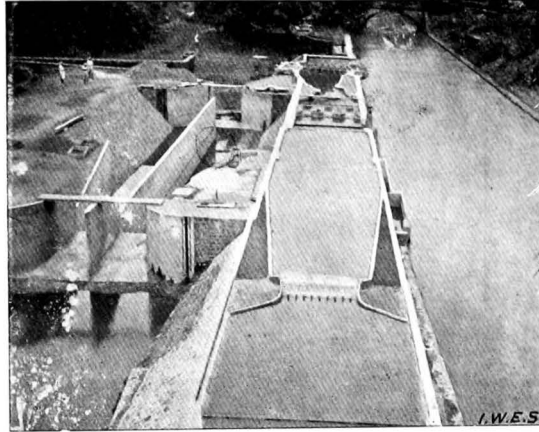


Photo 909. Photo taken from top of Khadakwasla Dam showing—

- (i) Station design of standing wave meter fall in foreground showing highly satisfactory flow conditions downstream.
- (ii) Plain overfall weir constructed downstream for comparison.
- (iii) First model of Indus from above the Gorge to below Barrage in distance.

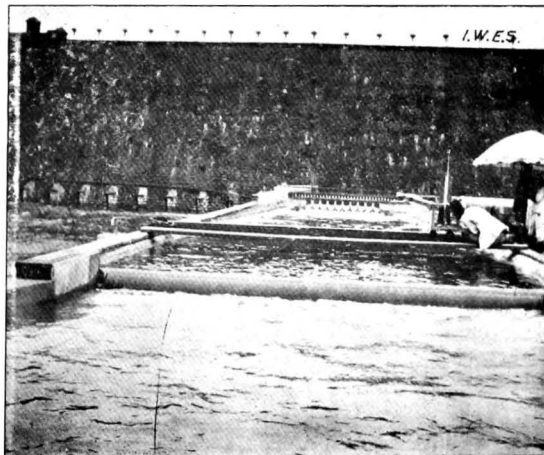


Photo 901. Photo taken from downstream of Overfall Weir (looking towards Dam) showing unsatisfactory conditions downstream of Weir where flow was disturbed and wave-wash considerable.

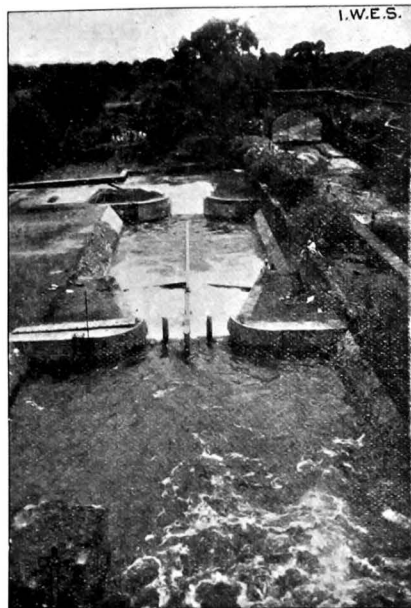


Photo 275. 300 cusec model of a Standing Wave Flume Meter Fall (6.5 ft.) with bed baffles and floating booms downstream. Divide wall added, so as to compare two alternative designs. Note aeration and high degree of stilling downstream of booms. Where water level kept constant fixed booms satisfactory but floating booms unsuitable in practice.

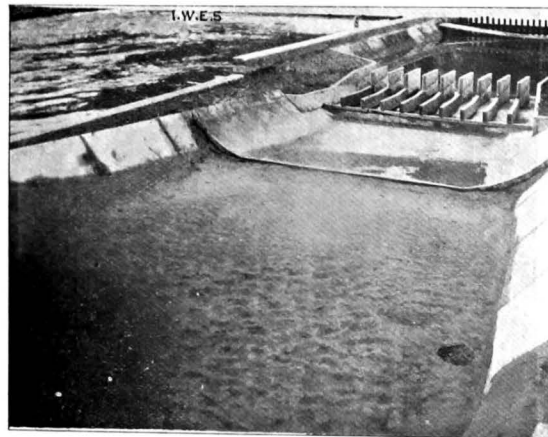


Photo 925. showing 1/16th scale model of Tando Mastikhan Fall (Rohri Canal) which was constructed with 1 in 5 side divergences. The model showed it would have worked highly unsatisfactorily, so it was remodelled by adding.—

- (i) Vanes—in continuation of the piers—to fan out flow;
- (ii) A baffle—downstream of the toe of the fall—to enforce a standing wave.
- (iii) A cistern—bowed—to develop natural flow.
- (iv) A deflector—to prevent bed scour and develop a normal distribution of isotachs.



Photo 134. Two dimensional model of sill of Sukkur Barrage (as designed with pavement sloping at 1 in 70). $Q \cong 420,000$ cusecs. Sill at R. L. 176 and downstream water level at R. L. 184.2. Standing wave at downstream end of pavement which would have failed.

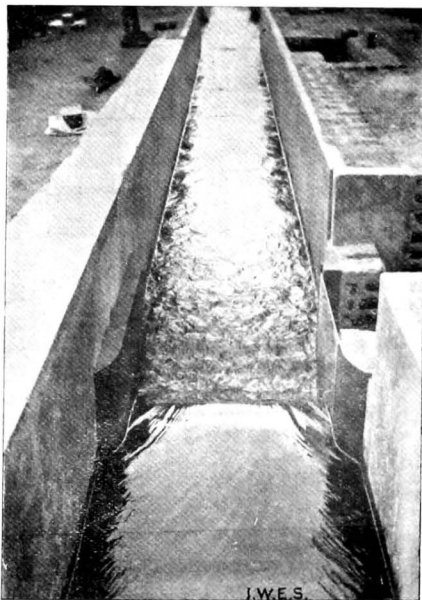


Photo 305. Two dimensional model of modified sill at R. L. 177.0; pavement sloping at 1 in 5 downstream. $Q \cong 479,000$ cusecs. Downstream water level at 184.4. Standing wave at toe of glacis.

(Photos from Bombay P. W. D. T. P. No. 29.)

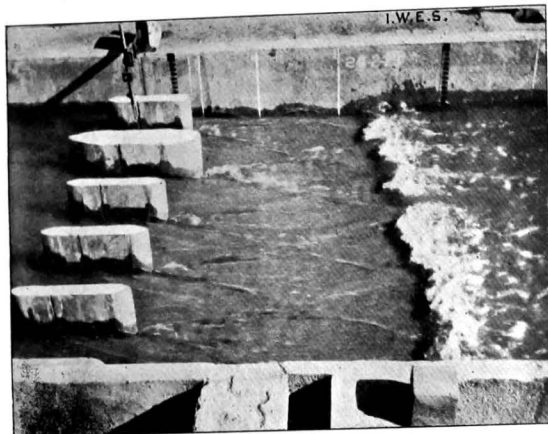


Photo 440. Model of floor of Sukkur Barrage as originally designed with 1 in 30 pavement slope upstream and 1 in 70 slope downstream; Barrage sill at R. L. 176.

Standing wave unstable and forming downstream of concrete block \approx 263 feet to 275 feet downstream of Barrage Gates, causing failure.



Photo 401. Same water levels as in Photo 440, with pavement lowered and slopes altered to 1 in 5 upstream and 1 in 10 downstream, sill raised to R. L. 177.

Standing wave brought up to end of piers.

(Photos from Bombay P. W. D. T. P. No. 29.)

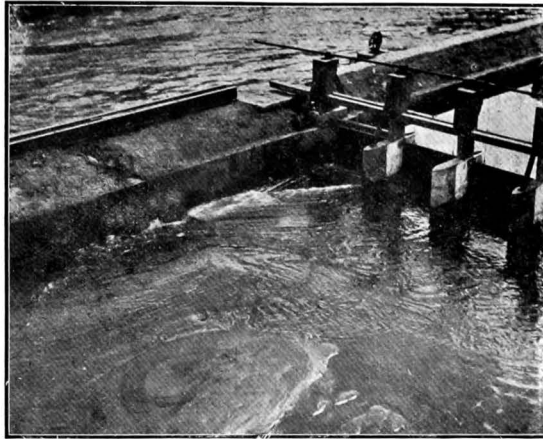


Photo 1256. Model of Right Bank end of Sakkur Barrage — looking upstream — showing jetting which caused heavy scour (water level lowered slightly when photograph taken to accentuate action).

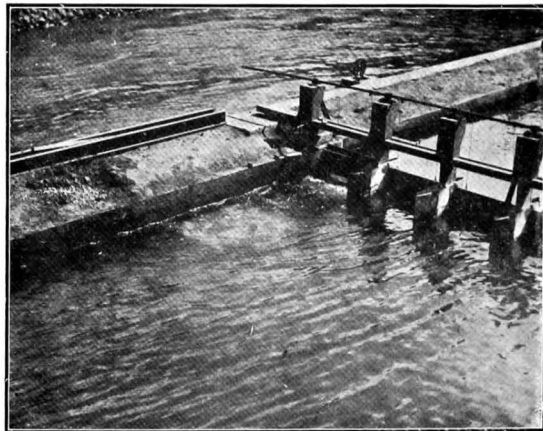


Photo 1263. With baffle added downstream of gates and deflector at end of pavement. Standing wave formed close downstream of gate and downstream flow was sweet.

(Photos from C. I. and H. R. S. A R. (T.) 1937-38.)

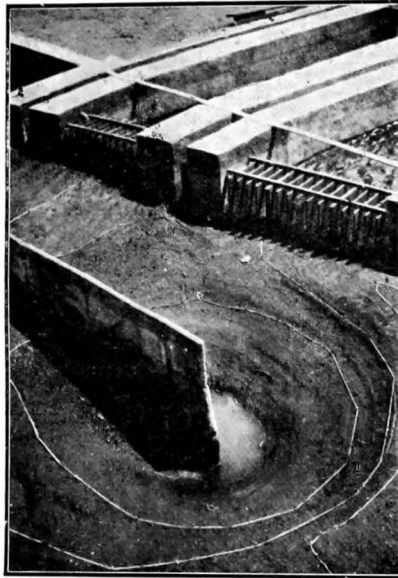


Photo 521. Part-width model of Sukkur Barrage showing the deep scour-hole which formed at the nose of the Divide wall due to curvature of flow. Looking downstream, showing Canal Heads in background.

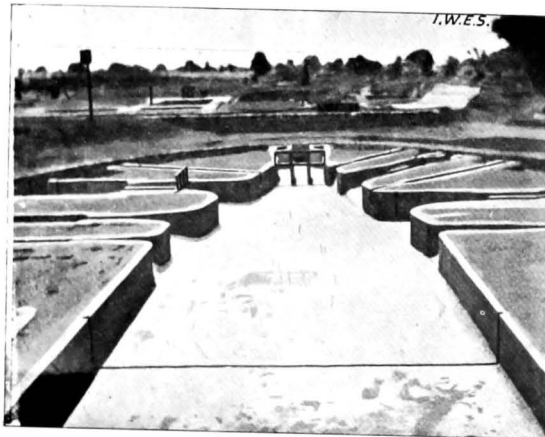


Photo 263. Model of a group of 8 regulators at Mile 169 of the Rohri Canal, Sind, designed to determine the distribution of bed material between the various off-taking channels—to ascertain whether any modifications required — Bombay P. W. D. T. P. 26.

(Photo 521 from Bombay P. W. D. T. P. No. 52, Sheet 4.
Photo 263 from Bombay P. W. D. T. P. No. 26.)

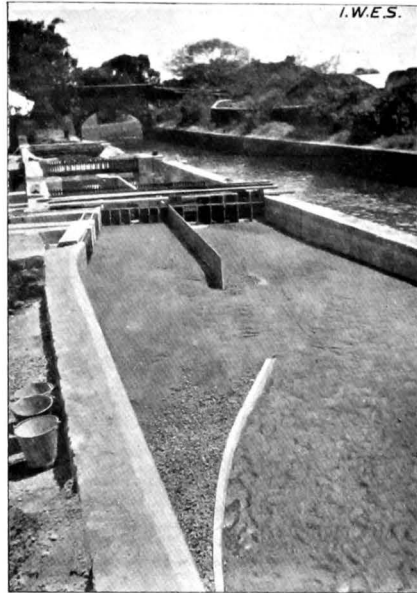


Photo 778. Part-width (reversed) model of the Sukkur Barrage with a large radius curved vane curving out from Outfall Section to exclude sand from Canal Heads.

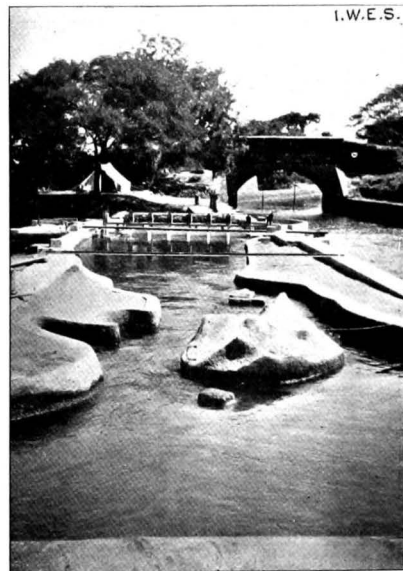


Photo 842. Model of the Indus looking downstream from above the Gorge (Bukkur Island in foreground) with Barrage (white piers) in middle distance. Used to study lines of flow and movement of bed material. Bukkur Gauge at G.

(Photo 842 from C. I. and H. R. S. A. R. (T.) from 1937-38.)

Experiments with Models of the River Indus in connection with
the design of Sukkur Barrage

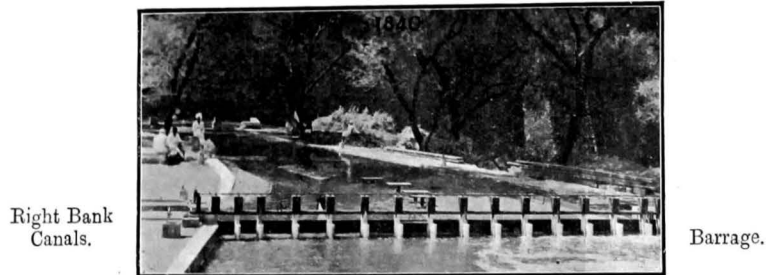


Photo 1840. General view of 1/150 scale Sukkur Barrage model, looking upstream. (Depth scale ratio = 1/30 and discharge scale ratio = 1/20,700). Maximum discharge 36 cusecs.

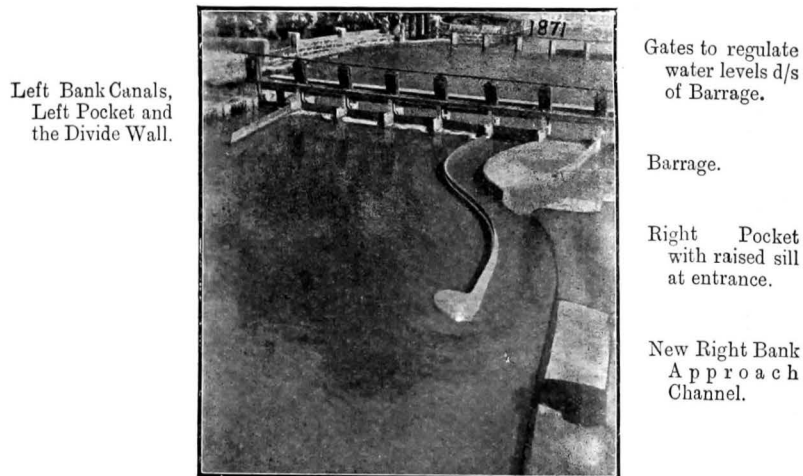


Photo 1871. 1/300 scale Sukkur Barrage model (Depth scale = 1/40 and discharge scale = 1/63,500) showing the new Right Bank Channel. Flow conditions equivalent to 400,000 cusecs in the River and 40,000 cusecs in the Right Bank Approach Channel, looking downstream. Maximum discharge = 12 cusecs.

(Photos from C. I. and H. R. S. A. R. (T.) 1938-39.)

1/40 SCALE MODEL OF APPROACH CHANNEL AT SUKKUR

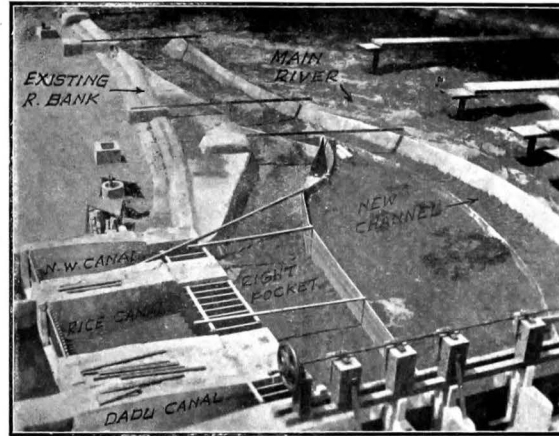


Photo 1967. Large scale part-width model of Sukkur Barrage, showing the Right Bank Approach Channel, Tail Channel, Right Pocket, with raised sill at entry and Right Bank Canals, looking upstream. The island between the tail channel and the Pocket formed naturally, by siltation.



Photo 2061. Looking downstream. Series I Experiment 1 with the original design of lay-out, viz. length of sill 700 ft. with its top at R. L. 183.5. The banks upstream and downstream of the raised sill were vertical as originally proposed.

Equivalent discharges were:

- Q River = 400,000 cusecs.
- Q of Approach Channel = 36,000 cusecs.
- Q of Right Bank Canals = 18,000 cusecs.
- Q of Tail channel = 18,000 cusecs.

Note.—The stool-like supports in the river portion are for supporting planks for observation and are not part of the model.

(Photo 1967 from C. I. and H. R. S. A. R. (T.) 1938-39)

(Photo 2061 from C. I. and H. R. S. A. R. (T.) 1939-40)

Photo 2107. Nose consisting of loose stones sloping at 2:1. showing damage after \approx 900,000 cusecs (Q of Approach Channel \approx 30,000 cusecs.)

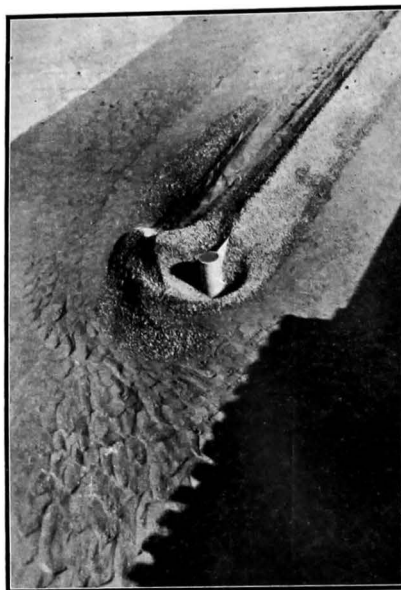


Photo 2110. Nose protected by crates (\approx 8' \times 5' \times 3') at 2½:1 slope. Top 25 ft. radius. Showing damage after \approx 900,000 cusecs. (Q of Approach Channel \approx 30,000 cusecs).

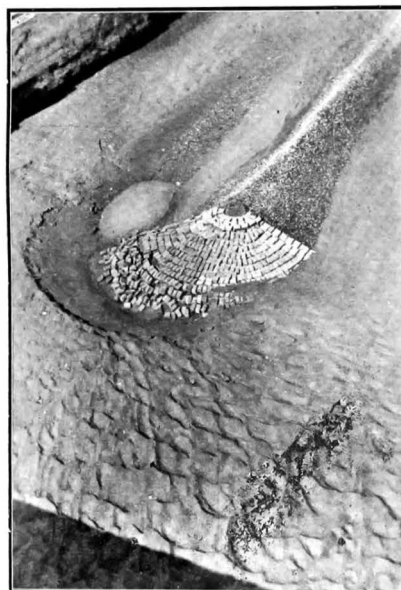


Photo 2125. Nose with elliptical sector and loose stone protection laid at 3:1 slope. Showing reduced damage after \approx 900,000 cusecs. (Q of Approach Channel \approx 30,000 cusecs.)



(Photos from C. I. and H. R. S. A. R. (T.) 1939-40)

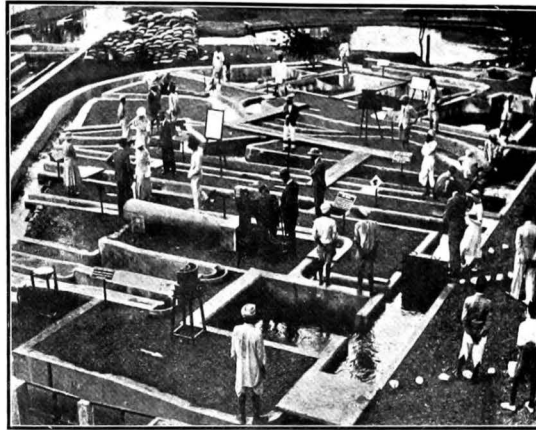


Photo 1203. His Excellency the Governor of Bombay (Lord Brabourne) being shown the Demonstration Models—October 4th, 1935.



Photo H. 1454. Demonstration model of the Malaprabha River—to show effect of downstream cut-off on flow which threatened to undermine left bank abutment. Three alternative solutions demonstrated—see A. R. (T) 1942-43 pp. 10, 11

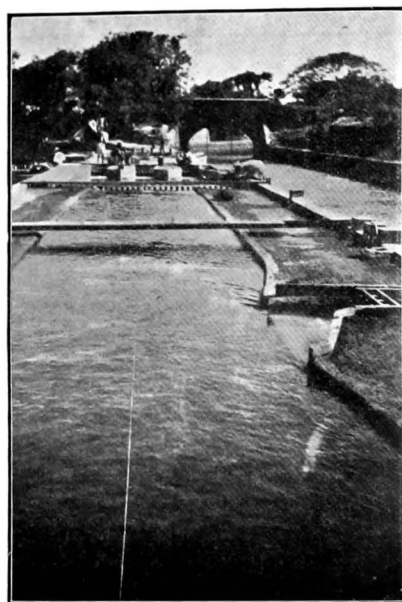
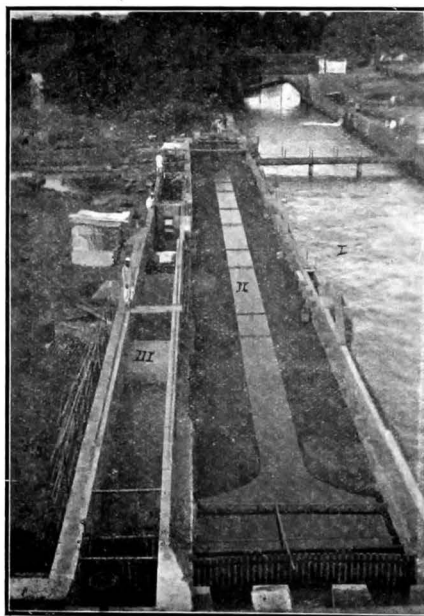


Photo 841. Model of Jamrao Canal (reversed) showing long sand-excluding sill with bank warped from 1 to 1 to vertical to exclude sand from the Head Dim Minor at Mile 17.



I Mutha Right Bank Canal.
II Sand channel.
III 5 ft. experimental channel.

Photo 1896. 22 ft. wide channel, 190 ft. long—later increased to 240 ft.—for sand channel experiments. In the experiment shown, the discharge was 1.5 cusecs and the mean diameter of sand \approx 1.4 mm.

(Photo 841—Sheet 4, C. I. and H. R. S. A. R. (T.) 1937-38.)
(Photo 1896—Sheet 10, C. I. and H. R. S. A. R. (T.) 1938-39.)



Photo 2255. Separation of boundary layer at crest of high coefficient weir, indicated by lifting of short silk threads, with formation of slight return roller near glass side wall due to side wall drag. Deflection of threads inwards denotes flow towards middle of sill where negative pressure unrelieved.

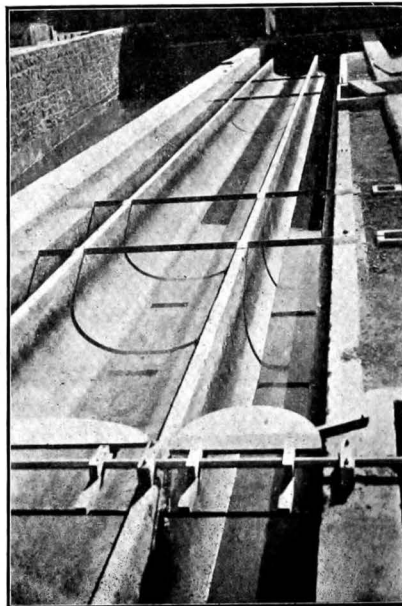


Photo 2329. Three semi-elliptical channels for observing wave velocities as affected by channel shape.

(Photos from C. I. and H. R. S. A. R. (T.) 1940-41)

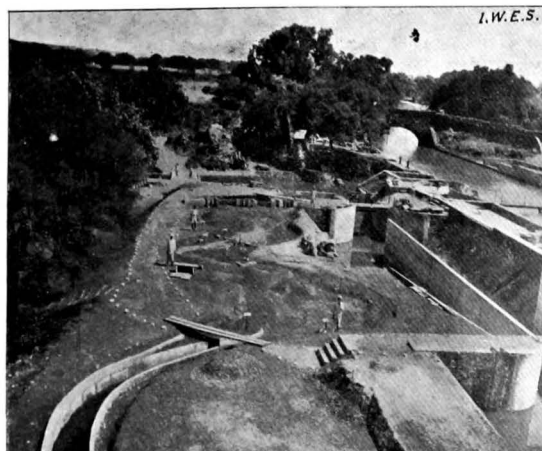


Photo 1176. Small scale Pilot model of the Ganges near Hardinge Bridge. Model scale $1/1320:1/120$. Discharge = 1 cusec.

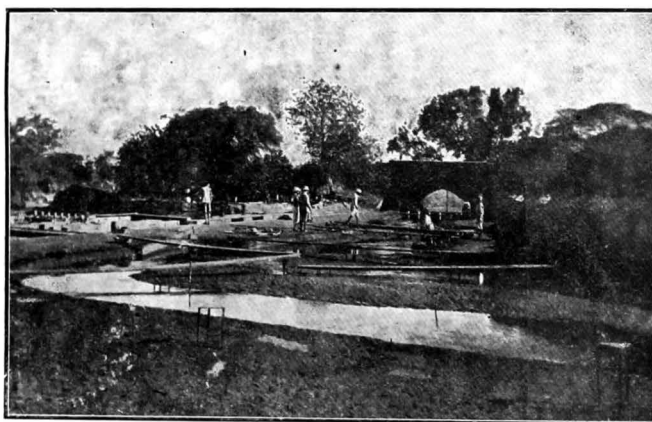


Photo 943. Large Model of the Ganges at Hardinge Bridge. Scale $1/500:1/63$ ($\approx 175,000$ cusecs). Discharge = 7 cusecs.

(Photo 943 from C. I. and H. R. S. A. R. (T) 1937-38. Also see Bombay P. W. D. T. P No. 55)

INTRODUCTION

Sheet 1—19

MODEL OF THE SARDA RIVER AT BANBASSA

Discharge scale ratio = 1/70,000.
Horizontal scale ratio = 1/150.
Depth scale ratio = 1/50.

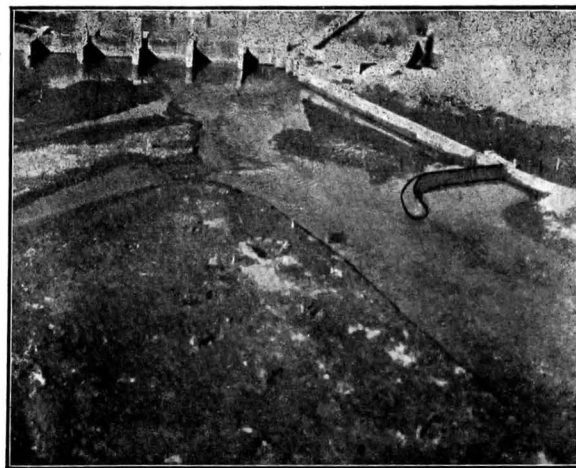


Barrage.

Right Afflux Bund.

Old low level boulder
Bund.

Photo 1869. Series VII, Experiment 2, without spur on Right Afflux Bund (i.e. 1938 conditions) looking downstream. Model run with small discharge to show channel which was deep along the Right Afflux Bund; which would have been undermined and breached.



Approved
short spur.

Photo 1891. Series VII, Experiment 3, with approved short spur added at corner of the Right Afflux Bund, looking downstream, which deflected the river away from Right Afflux Bund, thus preventing failure and the risk of an avulsion into the low ground beyond.
(Photos from C. I. and H. R. S. A. R. (T.) 1938-39)

EXPERIMENTS WITH A MODEL OF WATRAK RIVER
NEAR KAIRA (BOMBAY)

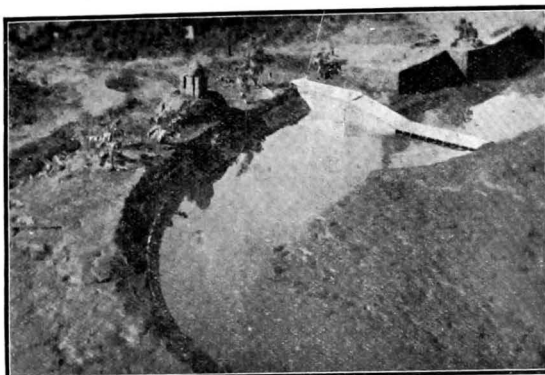
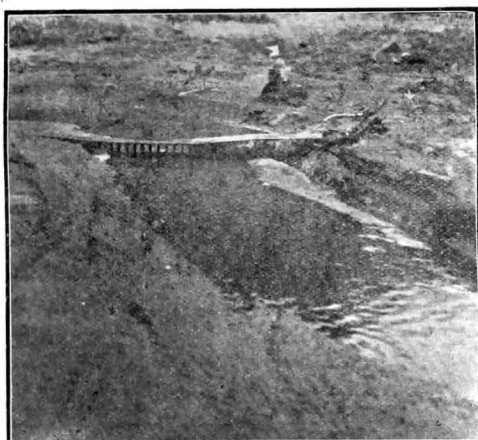


Photo 1863. Showing vertically-exaggerated model with causeway upstream of mouth of Vehela. Silting downstream which protected town face may be seen (Series III, Expt. 6) looking downstream.



Town wall.

Scale ratios:

1/125 horizontal.

1/25 vertical.

Photo 1862. Showing vertically-exaggerated model with submersible bridge at town wall corner directed 30° upstream recommended position—silt bank may be seen along the town wall (Series V, Expt. 9), looking upstream.

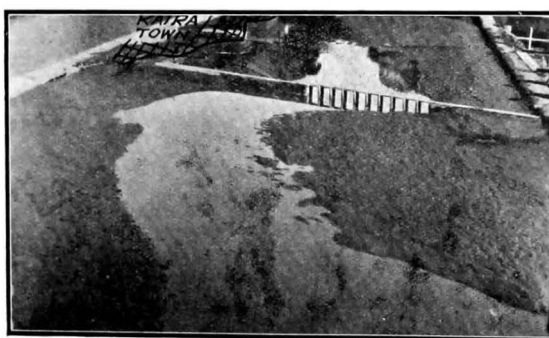
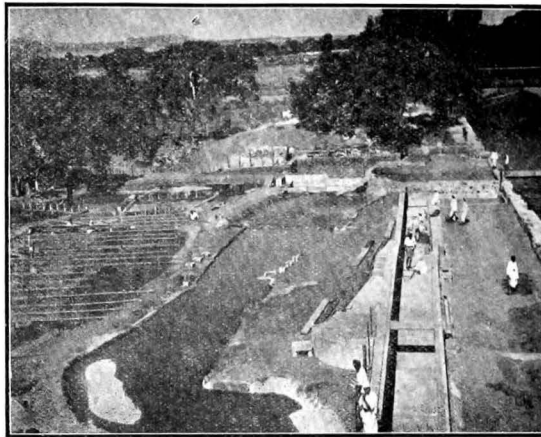


Photo 1873. Showing the 1/40 scale geometrically-similar model with high level bridge at corner of town. (Series VI, Expt. 4) looking downstream. Advice was ignored and bridge built downstream of the town, the Approach Embankment being breached and the bridge outflanked.

(Photos from C. I. and H. R. S. A. R. (T.) 1938-39)

Photo 1647. Model of the proposed Railway Bridge over the Brahmaputra-River at Amingaon. Looking downstream. Longitudinal and latitudinal scale ratio ... 1/300 Depth scale ratio 1/50



Model of the Sarda River U. P.

Sand channel.

Photo 1680. Model of the proposed bridge over the Brahmaputra at Amingaon looking downstream showing conditions with a discharge of 935,000 cusecs. Maximum estimated flood exceeds 3,200,000 cusecs.

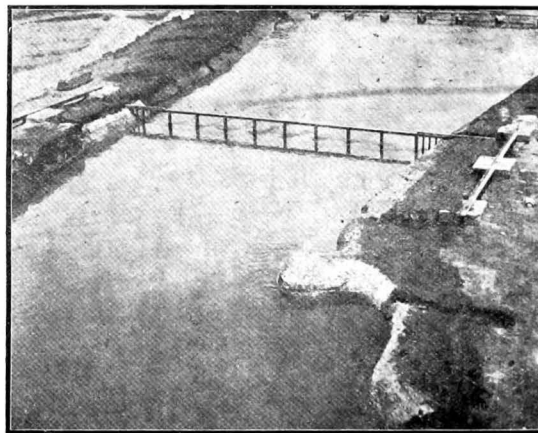


Photo 2978. 1/200: 1/30 model of the Gundak river (looking upstream). A cut-off chord channel (A) formed in 1943 and became the main channel. Prior to 1943, the arc channel (B) was the main channel. (C) became the main channel in 1944, see C. I. and H. R. S. A. R. (T) 1944.

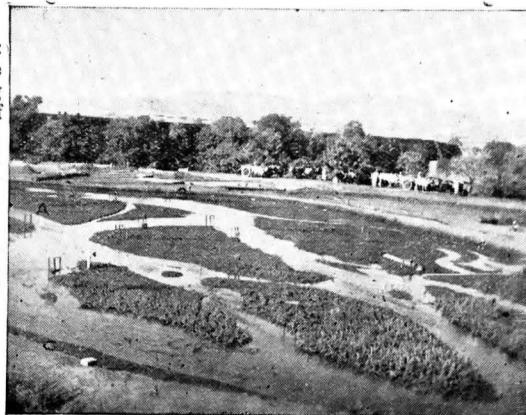
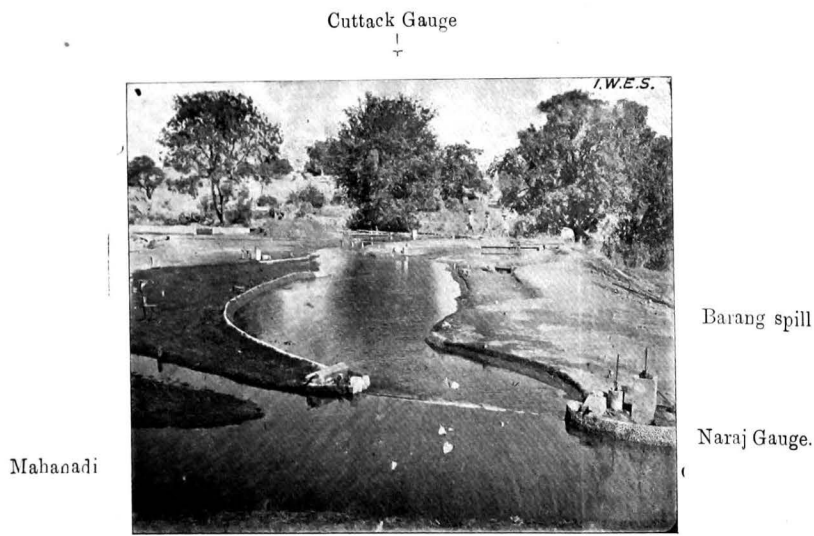


Photo 1647 from C. I. and H. R. S. A. R. (T). 1938-39
 " 1680 " " " " 1937-38
 " 2978 " " " " 1944
 29



Photo H. 1441. Photo showing part of the Mahanadi Model, with Naraj Gorge in foreground and intake of Katjuri over Naraj.



Naraj Weir, Katjuri downstream.

Photo 3021. Photo looking down the Katjuri showing an embankment along the left bank to close off the Biransi gap and the Barang spill on the right bank. See I. W. E. S. A. R. (T) 1943. (Photo 3021 from I. W. E. S. A. R. T. for 1945)



Photo H. 1467. Long tidal model of the Hooghly, showing tidal bore passing Palta, 12 miles north of Calcutta. Originally the model was >1,000 ft. long. Later reduced to 500 ft. with tidal generation at both ends.



Photo H. 1458. Large-scale, short tidal model of Hooghly from Amaalpara to Ichapur, 15 miles north of Calcutta, where a "knuckle" has formed. Tides generated at both ends.



Photo No. 4000. Rupnarain River tidal model.
Horizontal scale 1/650.
Vertical scale 1/90.

Photo showing ebb flow conditions before groyne was constructed. Note the concentration of flow along the left bank with return flow along the right bank; also concentration of flow along the right bank in the Approach Channel to the Railway Bridge.

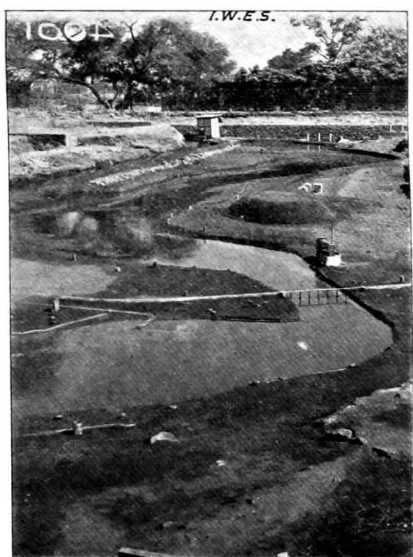


Photo No. 4001. Rupnarain River tidal model
with mobile bed. Model scale 1/650:
1/90.

Photo looking downstream showing the Railway Bridge and the groyne on the left bank. The small shed at the further end was the tide generating station at the downstream end of the model.

(Photos from I. W. E. S. A. R. (T.) 1944.)

FIG.1-1

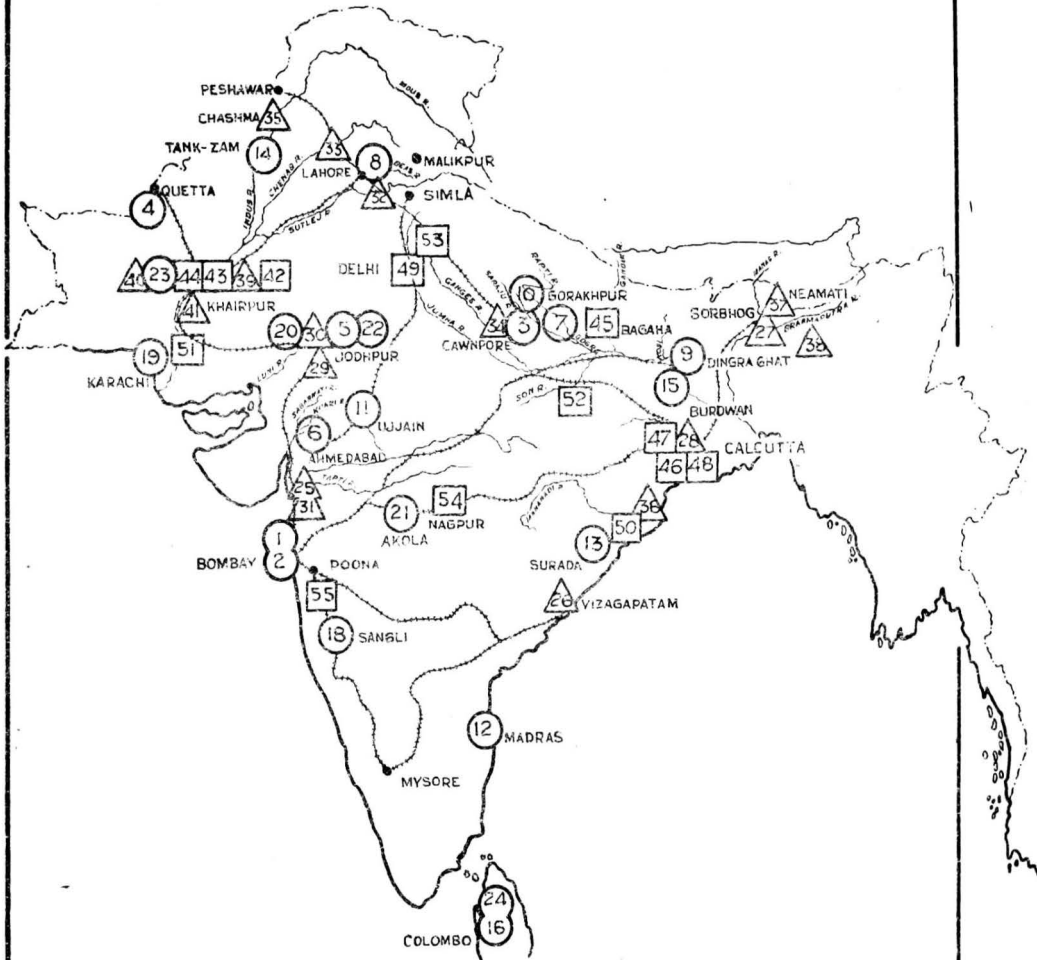
NO.5057 A.
INDIAN W. EXPT. STN. POONA.

(FIG.1.FROM A.R.T. 1944 OF I.W.E. STN.)

MAP OF INDIA

1944

SCALE - 1" = 520 MILES.



REFERENCES :-

- PLACES WHERE ADVICE WAS GIVEN WITHOUT INSPECTION OR EXPT.
- △ PLACES WHERE ADVICE WAS GIVEN AFTER INSPECTION WITHOUT EXPT.
- PLACES WHERE ADVICE WAS GIVEN WITH MODEL EXPT.

SHOWING FIFTY FOUR PLACES IN INDIA AND CEYLON
FROM WHICH PROBLEMS WERE RECEIVED FOR ADVICE
DURING 1944.

INTRODUCTION

BEHAVIOUR AND CONTROL OF RIVERS AND CANALS

CHAPTER 1.—INTRODUCTION.

1.00. OBJECT AND SCOPE OF THE REPORT

The main object of this Report is to outline and call attention to the advisory and research work done by the Author and his staff at Poona between 1916 and 1945 in connection with the control of canals and rivers.

Though Papers on many field studies and model investigations were published in Bombay P. W. D. Technical Papers * prior to 1938, and since then in Annual Reports (Technical) of the Central Irrigation and Hydrodynamic Research Station, Poona, (renamed the Indian Waterways Experiment Station, Poona, in 1943) many of the former are now out of print or are not easily available, so the time has come to bring the subjects dealt with up to date in the form of a general survey in the light of our present knowledge, especially regarding the effect of fluctuating discharges on the behaviour of canals and rivers flowing in alluvium.

This Report is not intended, however, to set forth the subject in a comprehensive way; but rather to deal with those aspects of the subject which are not contained in text books nor easily traced elsewhere. References are given to Reports in which further details can be found.

1.01. EXPERIENCE ON WHICH REPORT IS BASED

The experience on which the Report is based extends back to 1906 when the Author, on appointment to the Public Works Department, was posted to the Jamrao Canal in Sind, completed a few years earlier. After two years in charge of Sub-divisions, and two and a half years acting as Inspecting Assistant Engineer, for the three North Sub-divisions, he was transferred to the Godavari Canals, Bombay, to introduce Northern India methods of irrigation. Two years later he was made Executive Engineer, Poona Irrigation Division, for the same purpose; but soon found that introducing new methods on old canals presented major difficulties, in that 40 per cent of the best land commanded had already been rendered uncultivable by water-logging and salt efflorescence; so he urged the necessity of opening a Research Division to deal with this and other allied problems of a type which Executive Engineers had neither the time, the specialised knowledge, nor the inclination to study. As a result, the Special Irrigation Division was opened in 1916, the Author being put in charge.

The work included:—

- (i) Preparing contour plans of the areas commanded by the canals;
- (ii) preparing surveys and classifying soils and subsoils;
- (iii) selecting areas suitable for various types of crops for irrigation;
- (iv) designing the drainage required to reclaim damaged lands;
- (v) reclamation of waste lands and damaged areas by cultural methods, and "base exchange";
- (vi) planning for development by constructing new roads, classifying areas suitable for different types of irrigation, indicating suitable centres for factories, etc.;
- (vii) the utilization of Poona Sewage Effluent for irrigation;
- (viii) investigation of rainfall and run-off of rivers in Bombay Presidency; and
- (ix) investigation of flow problems.

* See Appendix 1.

THE BEHAVIOUR AND CONTROL OF RIVERS AND CANALS

It is with the last of these that this Report is mainly concerned. The quantity of work now being dealt with at the Station will be seen from Fig. 1-1 which shows the 54 places from which problems were received for advice during 1944.

1.02. FACILITIES AVAILABLE FOR MODEL RESEARCH

In 1919 a Hydrodynamic Research Station was initiated at Distributary No. 5 of the Mutha Right Bank Canal, six miles east of Poona, where the discharge normally available was 6 cusecs.

In 1933 a full scale 5 ft. x 4 ft. siphon spillway—capable of discharging 520 cusecs—was constructed downstream of Khadakwasla Dam (Lake Fife—see Photo 328, Frontispiece, and Fig. 1-2) to observe velocities and pressures at all points of flow, (especially as limiting conditions—due to low pressure causing vaporisation—were approached) for comparison with those observed in a 1/8th scale model.

Photo 328
Fig. 1-2.

By 1934 most of the hydrodynamic work of the Research Station had been moved from Distributary 5 to Khadakwasla, and from the end of 1936 this Station was financed by the Government of India. In 1938 it became the Central Irrigation and Hydrodynamic Research Station, Poona. It was renamed—Indian Waterways Experiment Station—early in 1943.

The move to Khadakwasla was made not merely because the old Station was much too small, but also because Khadakwasla possessed facilities unequalled at any other Station—a large supply of clear water being available, free of silt charge, under ample head, from nine entirely independent sources of supply, through sluices at canal bed level, so that an absolutely constant discharge is available—which is never the case where the supply is obtained from a canal—and the water is at constant temperature, being drawn from the bottom of a large lake.

1.03. WORK DONE BETWEEN JUNE 1916 AND 1929 BY THE SPECIAL IRRIGATION DIVISION AND THE IRRIGATION DEVELOPMENT AND RESEARCH CIRCLE

Work done at the original Hydrodynamic Research Station at Distributary 5 is outlined in Bombay P. W. D. T. P. 28 "Note on Research Work done by the Special Irrigation Division since its formation in June 1916." This included:—

Photo Sheets
1-1 and 1-2.

- (1) Duty of irrigation water in relation to the capacity of Deccan Canals (Paper 30, Bombay Engineering Congress—1918).
- (2) Modules and semi-modules for irrigation—(Vol. X, Bombay Engineering Congress, 1921—see photo sheets 1-1 and 1-2).
- (3) Important factors affecting the design and working of Deccan Canals.
 - (i) Aquatic weed growth.
 - (ii) Percolation and absorption.
(Paper 48, Vol. XI, Bombay Engineering Congress, 1922, also C. I. & H. R. S. A. R. (T) 1937-38, pp. 7-13).
- (4) The block system of irrigation in the Deccan Canal Tracts. (Bombay P. W. D. T. P. 16).
- (5) The design of distributaries and their outlets in the Bombay Deccan. (Bombay P. W. D. T. P. 19).
- (6) Moduling Distributary 8 of the Nira Left Bank Canal—explaining the factors determining design. (Bombay P. W. D., T. P. 25).
- (7) Methods suitable for reducing losses from Deccan Canals. (Paper of 1928, Bombay Engineering Congress).

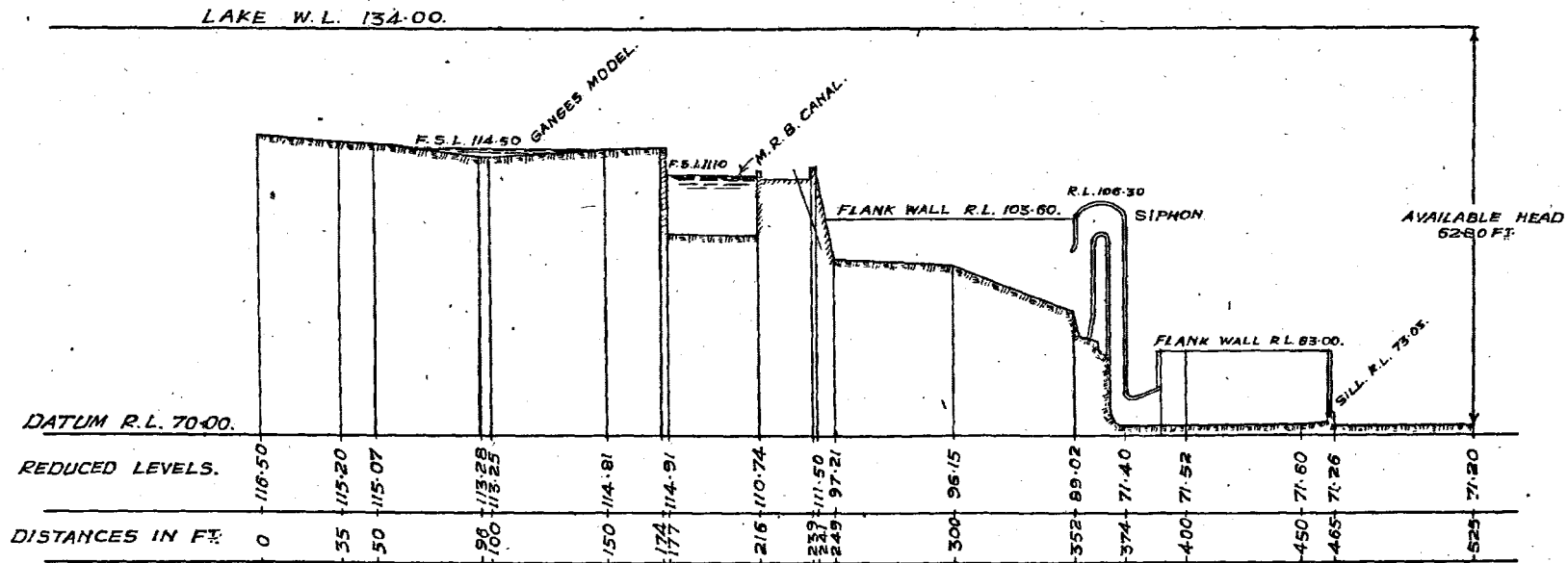
CROSS SECTION THROUGH THE SIPHON SPILLWAY
AT KHADAKWASLA.

FIG. 1-2

SCALE. $\left\{ \begin{array}{l} \text{HCR: } 1'' = 60' \\ \text{VER: } 1'' = 20' \end{array} \right.$

(FIG. 3. FROM A.R.T. 1937-38 OF C.I. & H.R. STN)

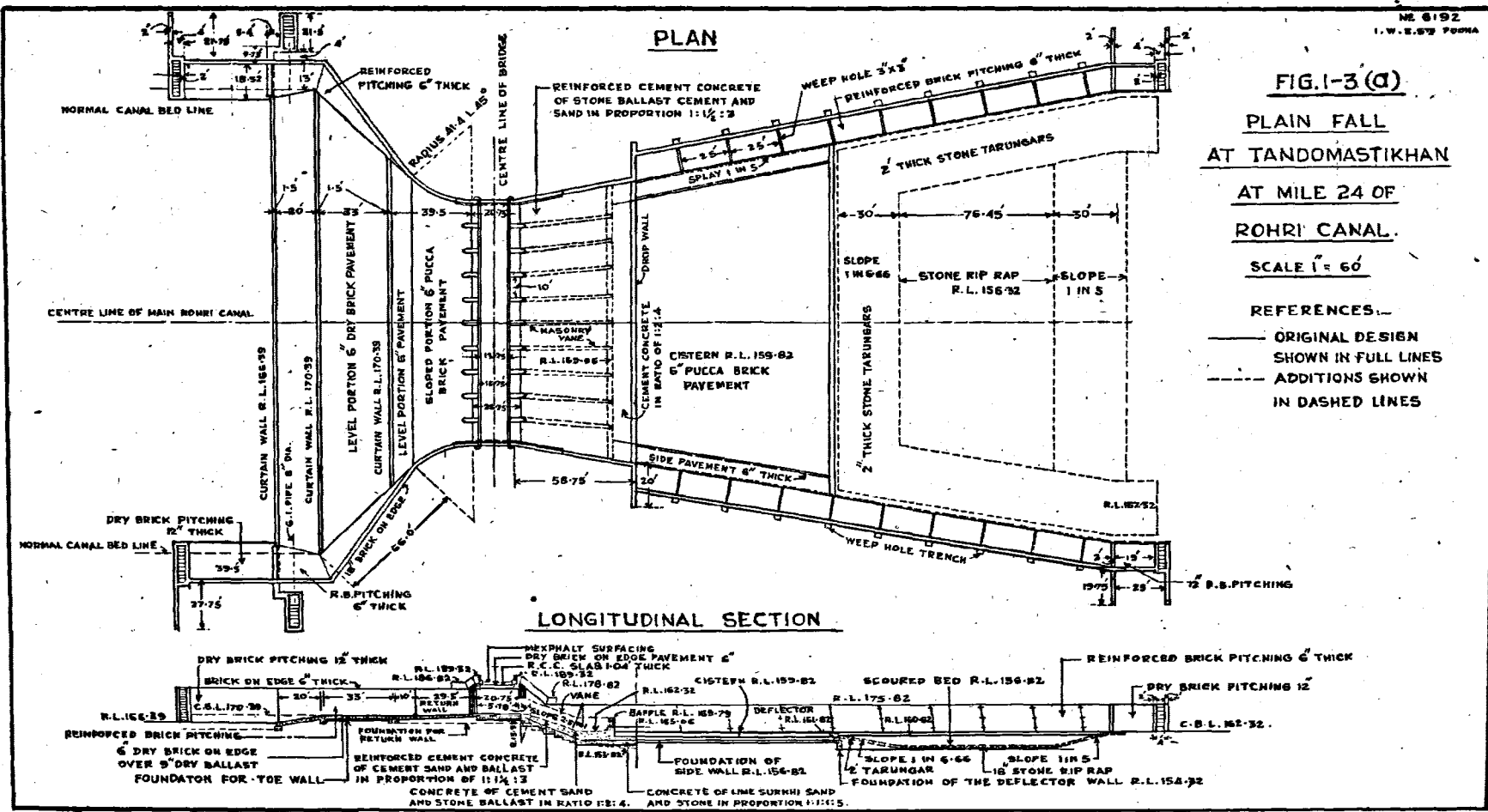
78



CROSS SECTION ON A. A.

INTRODUCTION

- (8) A critical study of run-off and floods of catchments of the Bombay Presidency compared with other parts of India and other countries, and formulæ deduced. (Bombay P. W. D. T. P. 30).
- (9) Isohyet map of Bombay Presidency showing lines of equal rainfall from 300" per year down to 15". (Irrigation Development and Research Division Drawing 65).
- (10) Distribution of Poona Sewage Effluent for irrigation—mixture supplied at rate varying from Rs. 215 (£16) per acre per year, down to Rs. 100 (£ 7-10-0) per acre, according to rates ruling for manures. (Bombay P. W. D. T. P. 57, Vols. I and II).
- (11) Relation between maximum floods and size of catchments. (Bombay P. W. D. T. P. 30).
- (12) Relation between annual rainfall and run-off in Bombay Presidency. (Bombay P. W. D. T. P. 30).
- (13) Absorptive effect of a lake, and cumulative run-off.
- (14) Cost of canals, distributaries, dams, etc. of the Bombay Deccan.
- (15) Duties (areas irrigated per cusec) at Distributary 5, Mutha Right Bank Canal.
- (16) Soil and sub-soil surveys of the Deccan Canal areas (Bombay P. W. D. T. P. 13).
- (17) Improvement of damaged land in the canal irrigated tracts of the Bombay Deccan (Bombay P. W. D. T. P. 14).
- (18) Land drainage experiments at Baramati. (Bombay P. W. D. T. P. 24).
- (19) Lindley type Standing Wave Flume. (Bombay P. W. D. T. P. 22).
- (20) Standing wave pipe outlet. (Bombay P. W. D. T. P. 23).
- (21) Crump type Standing Wave Flume. (Bombay P. W. D. T. P. 15).
- (22) Standing Wave Flume fall. (Bombay P. W. D. T. P. 15).
see Photo Sheet 1-3.
- (23) Effect of partial lining of Malegaon Cut in miles 38 and 39 of the Nira Left Bank Canal. (Bombay P. W. D. T. P. 20). Photo Sheet 1-3.
- (24) An adaptation of the Standing Wave principle to hydraulic testing. (Paper 74, Vol. XII, 1923, Bombay Engineering Congress).
- (25) "Fluming" masonry works to reduce cost. (Bombay P. W. D. T. P. 42).
- (26) Pier cut-water and ease-water experiments to ascertain the best design for the Sukkur Barrage. Bombay P. W. D. T. P. 29. (*see* Photo Sheet 1-7). Photo Sheet 1-7.
- (27) Experiments to ascertain the effect of scour on the downstream protection of the Barrage. (Bombay P. W. D. T. P. 29. Photo Sheet 1-7 and 1-8). Photo Sheets 1-7 and 1-8.
- (28) Effect of design of Barrage sill on coefficient of discharge. (Bombay P. W. D. T. P. 29).
- (29) Experiments (with models) to ascertain the best design for Approach Channels and the best method of working the gates at Sukkur Barrage to prevent dangerous scour and the deposition of silt. (Bombay P. W. D. T. P. 52. Photo Sheet 1-9). Photo Sheet 1-9.
- (30) Tests made on a group of regulators at mile 169, Rohri Canal, Sind. (Bombay P. W. D. T. P. 26. Photo Sheet 1-10). Photo Sheet 1-10.
- (31) Experiments with a V notch for field conditions.



43

INTRODUCTION

other designers in India and elsewhere. This type of design is illustrated in Photos 819 and 818, Sheet 1-4. The latest design is dealt with in great detail on pp. 20-55 of I. W. E. S. A. R. (T) 1944.

Photo
Sheet
1-4.

Photo 909, Sheet 1-5 shows the effective stilling in a model of a fall designed by the Station; and Photo 901 the relatively unsatisfactory conditions downstream of an Overfall Weir under similar conditions.

Photo
Sheet
1-5.

The best results were obtained when floating booms were used in addition to a baffle and deflector—Photo 275, Sheet 1-6—but their use was impracticable in large canals.

Photo
Sheet
1-6.

Photo 925, Sheet 1-6, and Fig. 1-3 show the Tando Mastikhan Fall after remodelling. This was constructed with 1 in 5 diverging sides instead of 1 in 10 required for natural expansion, and the model showed that the Fall would have failed in a few months. The design was, therefore, modified, at small cost, as a result of model investigations, by adding—

Photo
Sheet
1-6
Fig. 1-3.

- (i) vanes—in continuation of the piers—to fan out flow;
- (ii) a baffle at a point $5\frac{1}{2}$ times the height of the baffle downstream of the toe of the Fall—to enforce a standing wave under a wide range of downstream water levels;
- (iii) a cistern—“bowed”—to develop natural flow; and
- (iv) a deflector—to prevent bed scour and develop a normal distribution of isotachs.

This modified Fall has worked extremely satisfactorily.

1.07. MODIFICATION OF DOWNSTREAM PAVEMENT OF SUKKUR BARRAGE

Originally the Barrage was designed with the sill of the Barrage at R. L. 176 and the downstream pavement sloping at 1 in 70; but model experiments, carried out at Poona in 1928, showed that with this design the standing wave would form downstream of the pavement under certain conditions, causing it to fail—see Photo 440, Sheet 1-8 and Photo 134, Sheet 1-7.

Photo
Sheets
1-7
and
1-8.

As a result of the experiments, the sill of the regulator was raised to R. L. 177, the glacis slope altered to 1 in 10, and the pavement downstream of the glacis was built at a lower level—thus insuring the formation of a standing wave on the glacis, close to the Barrage, under any conditions that might arise during the flood season—see Photo 401, Sheet 1-8 and Photo 305, Sheet 1-7 and Bombay P. W. D. T. P. 29.

Photo
Sheets
1-7
and
1-8.

1.08. DESIGN OF PIERS

Photos 440 and 401, Sheet 1-8, show the design of cut-waters and ease-waters recommended for the piers. This was the well-tried design of equilateral arcs of circles. It was cheaper to build and caused less loss of head than the original design—which was similar in type, but twice as long.

Many other types of cut-waters and ease-waters were tested,* including sides diverging at 1 in 5 upstream and 1 in 10 downstream.

The conclusions arrived at were:—

Loss of head caused by piers may be said to be composed of:—

- (i) Impact losses (form drag) and
- (ii) friction losses (skin friction).

*Bombay P. W. D. T. P. 29. C. I. & H. E. S. A. R. (T) 1940-41 and 1941-42.

THE BEHAVIOUR AND CONTROL OF RIVERS AND CANALS

These are affected not merely by the shape and degree of smoothness of the cut and ease-water and the velocity of flow, but also by the slope of the pavement, if any, the length of piers and the ratio of depth of piers submerged to width of piers and to the span; but the major factor is the shape of cut and ease-waters.

It may be said that where flow is axial, cut and ease-waters, consisting of equilateral arcs of circles, which may end in a nose subtending an angle of 90° , are the best for practical purposes; but semi-circular cut and ease-waters are nearly as good hydrodynamically and as they catch less brush-wood and are more satisfactory than other designs with non-axial flow, they are the most suitable for such conditions.

45° triangular cut and ease-waters (i. e. subtending an angle of 90° at the nose) gave very bad results, both as regards afflux and downstream scour, and should be avoided.

1.09. ENFORCEMENT OF A STANDING WAVE DOWNSTREAM OF THE UNDERSLUICES AT SUKKUR

Photo
Sheet
1-9.

Although the pavement of the main part of the Barrage was constructed according to the revised design, the undersluice spans had already been constructed to the original design, and it was found that jetting would occur under certain working conditions, see Photo 1256, Sheet 1-9. The model showed that this could be largely overcome by adding a submerged wall in continuation of the pier and constructing a baffle and deflector at suitable points on the pavement—Photo 1263, Sheet 1-9.

1.10. THE BEST DESIGN OF DIVIDE WALL TO EXCLUDE EXCESS BED MATERIAL FROM CANALS—SEE PHOTO 521, SHEET 1-10; BOMBAY P. W. D. T. P. 52, CHAPTER 7 AND APPENDIX 3 OF THIS REPORT

The best design was found to be with the wall ending opposite the middle span of the uppermost canal.

1.11. LONG, CURVED, SUBMERGED, VANE TO PREVENT ACCRETION IN RIGHT BANK CANALS AT SUKKUR

Photo
Sheet
1-11.

Photo 842, Sheet 1-11, shows the first of several full-width models of the Indus from upstream of the Gorge to downstream of the Barrage. Sukkur Island and the twin channels of the Gorge can be seen in the foreground. This model was used to ascertain lines of flow and movements of bed materials and to visualise the problem.

Photo
Sheet
1-12.

It indicated—as had been foreseen—that accretion would occur in the Right Bank Canals, due to their taking off from the inside (convex) bank of the river—see Photo 1840, Sheet 1-12.

Photo
Sheet
1-11.

Photo 778, Sheet 1-11 shows a reversed part model, the vane being on the left instead of the right bank, for convenience. The recommendation to build this vane was not adopted, it being decided to wait and see what happened.

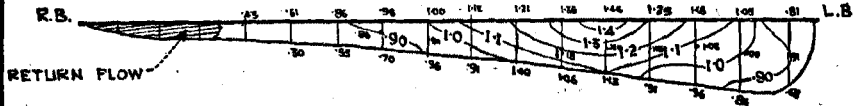
For three years—1932-34—retrogression took place in the river, and there was consequently no accretion in the canals; but thereafter accretion began, and by 1938 the Rice and North Western Canals had silted heavily to a depth of 5 ft. and dragline excavators were used to remove the deposits. The Author was again called in to advise. By this time the original proposal—to build a curved vane—was no longer feasible; because heavy silting had occurred along the right bank of the river upstream of the canals.

FIG-1-5

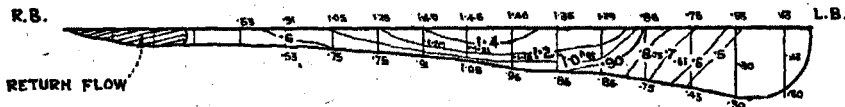
SHOWING VELOCITIES AT CROSS SECTION (X Y)
1/3 OF THE BRIDGE & PARALLEL TO IT.

SCALE 1" = 1.5'

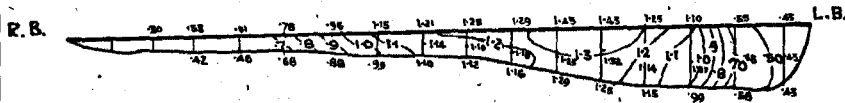
(a) - EXPERIMENT: 1
WITH EXISTING CONDITIONS.



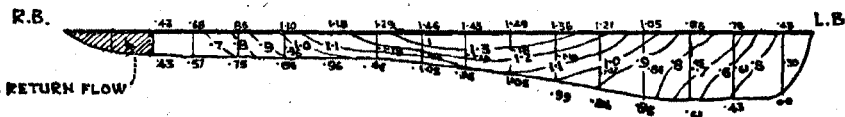
(b) EXPERIMENT 2.
WITH OLD LOOP FLOWING; CUT-OFF CLOSED.



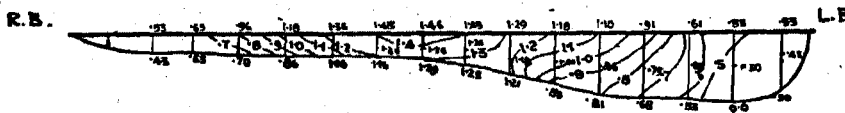
(c) EXPERIMENT 3
WITH GROUYNE ON R.B. 1040' BELOW BRIDGE.



(d) EXPERIMENT 4.
WITH PARTIAL CUT IN R.B. 1700' U/S OF BRIDGE



(e) EXPERIMENT 5.
WITH FULL CUT IN R.B. 1700' U/S OF BRIDGE.



INTRODUCTION

1.12. RIGHT BANK APPROACH CHANNEL

After testing three alternative designs—each of which would have been successful in excluding excess sand from the Right Bank Canals—an Approach Channel, 5,000 ft. long, was adopted—see Photo 1871 on Sheet 1-12, and Sheets 1-13, 1-14 and Chapter 7.* The work had to be completed without interfering with irrigation, and hence without lowering the ponded water level in the river upstream of the Barrage. In spite of this, the work was successfully completed in two working seasons.†

Photo
Sheets
1-12,
1-13,
1-14.

Three models of the Approach Channel and part of the river—with different degrees of vertical exaggeration—were used in this investigation. The largest of these is shown in Photos 1967 and 2061, Sheet 1-13. A model of the nose of the outer bank of the Approach Channel is shown in Photo Sheet 1-14, see also pp. 12-20 of C. I. and H. R. S. A.R. (T) 1939-40.

Photo
Sheets
1-13,
1-14.

Since the Approach Channel has been constructed, excess bed-material has been excluded from the Right Bank Canals;‡ and within three years, the bed of the North Western Canal had scoured down to its original designed level. The Rice Canal, being non-perennial, has taken longer to recover. This question is dealt with in greater detail in Chapters 6 and 7.

1.13. DEMONSTRATION MODELS

Visitors, students, and even engineers, find it difficult to visualise and understand the problems being dealt with in the larger models. This is not surprising, bearing in mind that many of them are several hundred feet long; so a set of 20 small demonstration models were prepared. These are shown on arrival to those who visit the station. Several can be seen in Photo 1203, Sheet 1-15.

Photo Sheet
1-15.

1.14. EFFECT OF A CUT-OFF DEMONSTRATED IN MALAPRABHA DEMONSTRATION MODEL

One of the demonstration models is shown in Photo H 1454, Sheet 1-15. It is used to demonstrate the effect of a "cut-off" which developed downstream of the Railway Bridge at Hol-Alur on the M. & S. M. Railway. Before the "cut-off" opened, flow through the bridge was normal; but after the cut-off developed, heavy erosion occurred upstream of the left bank abutment. Three alternative treatments were tested.

Photo Sheet
1-15.

1. Closing the cut-off channel.
2. Constructing an attracting groyne on the right bank downstream of the bridge.
3. Cutting away portion of the right bank upstream of the bridge.

The third alternative was adopted.§

Figs. 1-4 and 1-5, which are self-explanatory, show lines of flow and isotachs observed in experiments carried out in a much larger mobile model.

Fig. 1-4
1-5.

1.15. BED-SAND EXCLUSION FROM DISTRIBUTARIES TAKING OFF THE JAMRAO CANAL AT MILE 17

Photo 841, Sheet 1-16, shows a model of a sand-excluding sill. This was recommended to exclude sand from Dim Minor, which takes off at

Photo Sheet
1-16.

* C. I. & H. R. S. A. R. (T) 1938-39, pp. 9-10 and 1941-42, p. 116.

† See Discussion by Mr. C. G. Hawes, C. I. E., M. C., on Maritime Paper No. 3. Inst. C. E. 1946.

‡ I. W. E. S. A. R. (T) 1943 pp. 33-35 and 1944 p. 104.

§ Malaprabha River.—Experiments to compare relative merits of various alternative measures for relieving heavy scour at the Railway Bridge abutment. C. I. & H. R. S. A. R. (T) 1942-43, pp. 10, 11.

THE BEHAVIOUR AND CONTROL OF RIVERS AND CANALS

17th mile Jamrao Canal. The effect of the warped side is to cause mid-depth water to dive and so to deflect bed filaments away from the off-take. This principle has been repeatedly utilised by the Station in designs proposed for excluding excess sand—see C. I. & H. R. S. A. R. (T) 1937-38, pp. 21-26, also chapter 6.

1.16. SAND CHANNEL EXPERIMENTS

Photo Sheet 1-16. Photo 1896, Sheet 1-16, shows the 22 ft. wide sand channel in which many hundreds of experiments have been carried out. It was originally 130 ft. long, but was later extended to 190 ft. and later still, to 240 ft.

The difficulties which have to be overcome in getting accurate results are enormous; because up to 200 hours are required to establish equilibrium, and an error in regulation of the downstream water-level, or injection of sand, will waste several days, or even vitiate the final result. Discharges up to 18 cusecs have been used.

1.17. SCALE EFFECT

In models designed to investigate problems of movement of bed material, the depth relative to width and length is exaggerated. This is called "vertical exaggeration". More correctly we should use the terms "width contraction", "length contraction" and "slope exaggeration", which are rarely the same in mobile models, so should not be lumped under the single term. If this practice were followed, less nonsense would be talked about river models reproducing river conditions.

Exaggeration of vertical and slope scales follow a natural law, which Gerald Lacey has shown $\propto Q^{\frac{1}{2}}$ in regime channels.*

Photo Sheet 1-17. Even in the case of rigid models there may be a marked scale effect. Photo 2255, Sheet 1-17, exemplifies one such case—a high coefficient weir.

Where the length of a weir crest in the line of flow is sufficient for flow "parallel to the friction slope" to be attained on the weir, a very close approximation to geometrical similarity can be obtained—provided allowance is made for the relatively greater friction loss in the model; but over a "high coefficient weir" there is a marked divergence from geometrical similarity of pressure and velocity distribution.†

In this particular investigation, three models with different scales were used, the coefficient of discharge for the prototype being obtained by extrapolation. When, in the following year, a channel was constructed which was large enough to test a full-scale model in which the discharge per ft. run was equal to that in the prototype, the experiment was repeated, with the idea that a higher degree of accuracy would be obtained. Though the "depth scale-effect" was eliminated, the width of the model being much less than in the prototype, "width scale-effect" was introduced.

Photo Sheet 1-17. Photo 2255, Sheet 1-17, shows that the short silk threads fixed on the surface of the weir near the glass side of the model did not lie along the surface of the weir, indicating separation of the boundary layer, with the formation of a small return roller, caused by side wall drag. This deflection of the threads inwards denoted flow away from the wall, due to the pressure being greater and velocity less near the wall than that at a distance from its influence; and, as a result of this side-wall scale-effect, the accuracy of the coefficient obtained in this large model was little, if at all, greater than the coefficient determined by extrapolation from the three smaller models.

This example is given to show that even in the case of rigid models, scale effect is often an important factor, which, in some cases, can only be eliminated by the use of very large models.

* Proc. Inst. C. E. (Paper) 229 (1930).

† C. I. & H. R. S. A. R. (T) 1939-40, pp. 45-50.
C. I. & H. R. S. A. R. (T) 1940-41, pp. 42-50.

INTRODUCTION

1.18. WAVE VELOCITIES AS AFFECTED BY CHANNEL SHAPE.*

Photo 2329, Sheet 1-17, shows three semi-elliptical channels used in a series of experiments designed to ascertain the effect of channel shape on wave and surge velocities. This is an important factor in experiments with models like the long tidal model of the Hooghly—especially with regard to questions connected with the tidal bore. Photo Sheet
1-17.

It was found that, other factors being the same, there was very little difference between the velocities of waves and surges. The velocities relative to the fluid, $V+V_w$ were compared with four formulæ of which (c) and (d) take height of wave into account:—

- | | |
|---|----------------------|
| (a) \sqrt{gd} | (Lagrange) |
| (b) \sqrt{gR} | |
| (c) $\sqrt{gd} \left(1 + 75 \frac{h}{d}\right)$ | (St. Venant) |
| (d) $\sqrt{g(d+h)}$ | (Raleigh-Boussinesq) |

Where

- d = mean depth
 R = hydraulic mean depth
 h = average height of wave or surge.

- Formula (a) gave a good approximation to observed wave velocity and would have been accurate had the wave height been negligible;
 „ (b) which was the only formula in terms of R (Hydraulic mean depth) gave widely incorrect results;
 „ (c) slightly over-estimated velocity in most cases; while
 „ (d) was the most accurate of the formulæ tested.

The enquiry showed, as anticipated, that in tidal models, time-scale should be calculated in terms of \sqrt{gd} and not of \sqrt{gR} .

† 1.19. SMALL “PILOT” MODEL AND LARGE MODEL OF THE GANGES ABOVE AND BELOW THE HARDINGE BRIDGE

It was not till 1934 that work began to be received at the Station from the Railways. In December of that year the Author was called in by the Hardinge Bridge Committee, which had been appointed by the Railway Board to advise in connection with damage caused by erosion upstream of the Hardinge Bridge. The bridge, which spans the Ganges at Paksey, on the Bengal-Assam Railway—then known as the Eastern Bengal Railway—was considered to be in danger of being outflanked, and nearly a million pounds had been spent during the previous two years in an endeavour to prevent further damage. After inspection, and detailed model experiments, recommendations were made which were generally accepted by the Committee.

Photo 1176, Sheet 1-18, shows the pilot model, which had a maximum discharge of 1 cusec. Photo 943, Sheet 1-18 shows the large model. The “pilot” model did not give similar results to those obtained in the larger model, nor did the larger model produce the same results *over a period of years* as occurred subsequently in the Ganges; because it was found that erosion took place much more rapidly than in Photo Sheet
1-18.

* C. I. & H. R. S. A. R. (T) 1940-41, pp. 58-59.

† Bombay P. W. D., T. P. 55, “Report on experiments carried out in connection with the protection of the Hardinge Bridge”.

C. I. and H. R. S., A. R. (T) 1937-38 pp. 66-74;
 1938-39 pp. 22-23;

I. W. E. S., A. R. (T) 1944 pp. 74-75.

THE BEHAVIOUR AND CONTROL OF RIVERS AND CANALS

the river, whereas silting took place much more slowly; but the Author was able to predict, with a high degree of accuracy, what would happen in the river from year to year, and he predicted the year by which flow would change from the right bank to mid-stream before moving to the left bank, with an error of only one season. This was after an extended analysis of what had previously occurred in the river, combined with information derived from the model experiments.

The relative merits of alternative treatments were, however, clearly demonstrated in the model experiments.

* 1.20. MODEL OF THE SARDA RIVER AT BANBASSA

Photo Sheet 1-19. Several series of experiments were carried out with this model. The particular investigation shown in Photos 1869 and 1891, Sheet 1-19, was to determine the best method of protecting the Right Afflux Bund, along the face of which scour was active; so that it was feared the Bund might fail—in which case the river would have cut a new course into the low ground behind the bund, causing an avulsion, the Barrage being outflanked. The time available for arriving at a decision was restricted to a few weeks. A short "hockey-stick" (Burma) spur—shown in Photo 1891, Sheet 1-19—was recommended and constructed.

† 1.21. MODEL OF THE WATRAK RIVER NEAR THE TOWN OF KAIRA, BOMBAY PRESIDENCY

Photo Sheet 1-20. In this case, the north-west corner of the town wall was being undermined, and it was feared that the river might change its course to the east, and outflank the town. It was also proposed to build a causeway (or possibly a bridge), and the Station was asked to consider whether the two questions could be solved by a single structure. The three photos on Sheet 1-20 show two of the alternatives tested. The advice given was not acted on. Instead, a bridge was built downstream of the town. The Approach Channel leading to this bridge was cut through, the bridge being outflanked.

Nearly Rs. 3,00,000 had to be spent on repairs, and the flow conditions through the bridge still remain highly unsatisfactory.

‡ 1.22. PROPOSED BRIDGE OVER THE BRAHMAPUTRA AT AMINGAON

Photo Sheet 1-21. Photos 1647 and 1680, Sheet 1-21, show the bridge which it was proposed to build over the Brahmaputra at a point where the river had remained unchanged for centuries. It was considered extremely important not to interfere with the flow in any way.

The chief recommendations were regarding the necessity for very deep pier foundations, and the desirability of having a single span, instead of several short spans, at the two ends of the bridge, and to have piers with deep foundations in the banks instead of abutments, so that the natural flow conditions would not be appreciably altered when adjustments took place.

§ 1.23. MODEL OF THE GANDAK RIVER AT BAGAHA

Fig. 1-6. Fig. 1-6 shows conditions in the river in 1921-22 when it was still flowing normally through the bridge. On this plan, vestiges of earlier

-
- * C. I. and H. R. S., A. R. (T) 1937-38 pp. 34-37;
1938-39 pp. 10-16;
1939-40 pp. 21-23;
1941-42 p. 19.
 - † C. I. and H. R. S., A. R. (T) 1937-38 pp. 77-80,
1938-39 pp. 16-19.
 - ‡ C. I. and H. R. S., A. R. (T) 1937-38 pp. 44-46;
1938-39 pp. 23-28.
 - § I. W. E. S., A. E. (T) 1944 pp. 10-13.

NR 5285

I. W. (COST. STY. ROOM)

TRACED FROM D. M. 1948 (A. R. T. SHEET NO. 63, 64, 65, 66, 67, 68, 69, 70, 71, 72, 73, 74, 75, 76, 77, 78, 79, 80, 81, 82, 83, 84, 85, 86, 87, 88, 89, 90, 91, 92, 93, 94, 95, 96, 97, 98, 99, 100)





(FIG. 7 FROM A. R. T. 1944 OF I. W. E. STY.)

FIG-1-6

GANDAK RIVER COURSE.

SEASON 1921-22.

SCALE 1" = 4 MILES.

REF	RIVER COURSE	
	CANAL	
	EXTREME LIMIT	
	RY LINE	

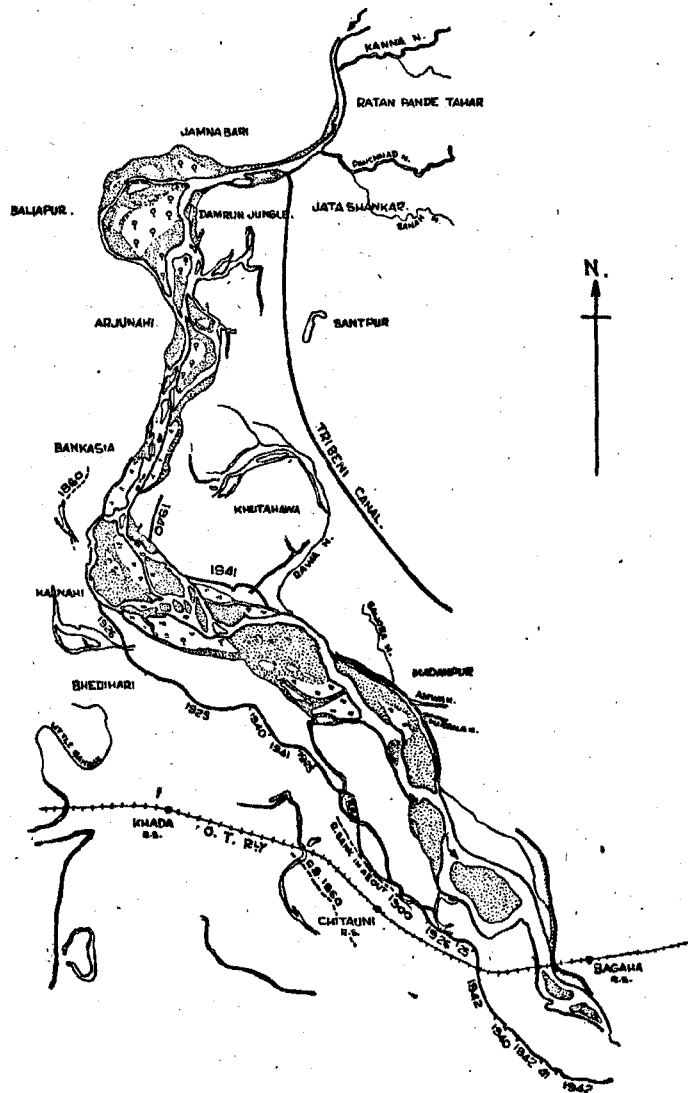


FIG-1-7
SHOWING EXTREME LIMIT OF
GANDAK RIVER AT BAGAHA
BETWEEN 1922 TO 1943

SCALE 1" = 2 MILES.

REFERENCE
BORE HOLES ++

57

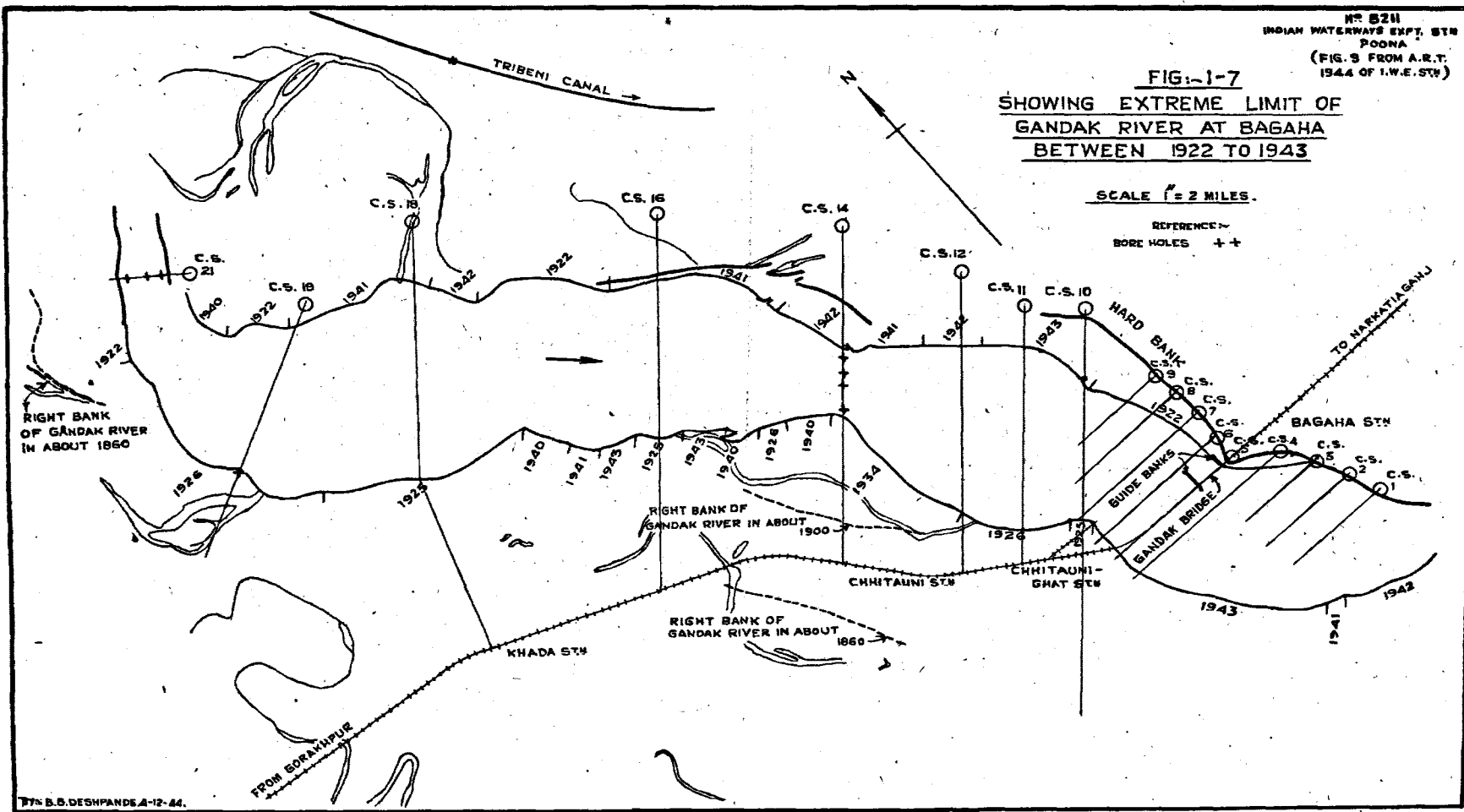
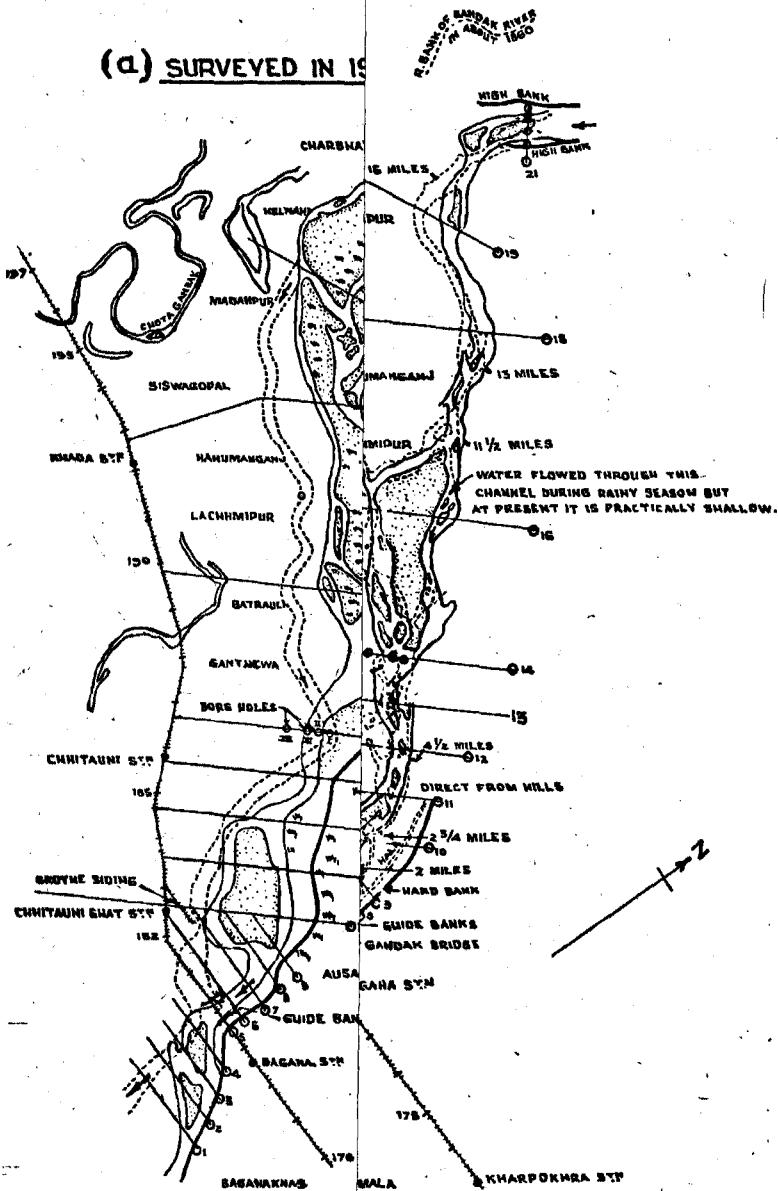


FIG. 1-8

VEYED IN DECEMBER 1942 & 43.

(a) SURVEYED IN 1942 & 43



REFERENCES :-
 1942 COURSE
 1943 COURSE

INTRODUCTION

river channels can also be seen. One of the old river channels lay some 5 miles north of the present course, near mile 8 of the Tribeni Canal; while vestiges of other, still earlier, channels can be seen to the south of the existing river. Since 1860, however, the river has remained within a belt some 2 to 5 miles wide; and since the Railway was constructed the river swing has not exceeded some 3 miles—Fig. 1-7.

Fig. 1-7.

A comparison of conditions in 1886-96, before the bridge was built, and conditions in 1921-22, 10 years after the bridge was constructed, showed that in that period of 25 years little change had taken place, the river continuing to flow along the hard left bank, upstream of the position selected for the bridge. By 1922, the river had bifurcated into two channels, 4 miles upstream of the bridge, joining again about 1½ miles above the bridge to flow axially through it. Fig. 1-8 (a) shows river conditions both in 1922 and also in 1925, for comparison. By 1924, the river upstream of the bridge had begun to embay behind the right embankment; and the flow, after concentrating at the nose of the right guide bank, flowed across, as a concentrated stream, to the left bank, to flow with high velocity and sharp curvature through the three left spans of the bridge. As a result, pier No. 14 failed, and the right guide bank also failed.

Fig. 1-8 (a).

After the flood of 1924, the Chhitauni groyne was constructed, the idea being to deflect water back through the bridge [Fig. 1-8 (a)]. Actually, however, the effect of this groyne was to pull the river still more strongly round its nose, thus acting as an "attracting spur"; and as a result, the river cut through the south-west approach embankment to the bridge. As the river was still pressing towards the western approach embankment in 1925, flooding the country and producing 6' afflux between the east and west sides of the railway embankment at a point 7 miles upstream of the bridge, repairs to the bridge—which had been planned—were abandoned, and the breach in communications was accepted.

Fig. 1-8 (a).

In 1934, the pitching material of the Chhitauni groyne was removed above water level and the groyne abandoned.

In 1938, the river was still unfavourable for diverting it back through the bridge, because a considerable portion of the flow was along the nose of the Chhitauni groyne, which was still holding the river; but in January 1940, flow along the right bank decreased, and a large part of the river was flowing along the outside of the right guide bank, a condition favourable for diverting the flow back through the bridge. After the 1940 flood season, conditions had again become less favourable, and by December 1942 the main flow of the river was again along the right bank near Chhitauni; and in December 1943 the whole river was at its most extreme westerly position, near Chhitauni Ghat Station.

Fig. 1-8 (b).

Fig. 1-8 (c).

It was at this stage that a model was constructed to see whether the river could be deflected back through the bridge? And if so, how this could best be effected?

Material exposed on river bed

The bed of the river consists of medium sand having a mean diameter of 0.38 mm. No stratum of hard material could be found.

Fig. 1-9.

River discharge

The maximum flood discharge in 1924 was estimated by Railway staff to be 294,000 cusecs. This was based on a fall of 1.25 ft. per mile measured on a straight reach downstream of the bridge, using the Mississippi formula*

$$V = \left[(225 R. S^{\frac{1}{2}})^{\frac{1}{4}} - 0.0388 \right]^2$$

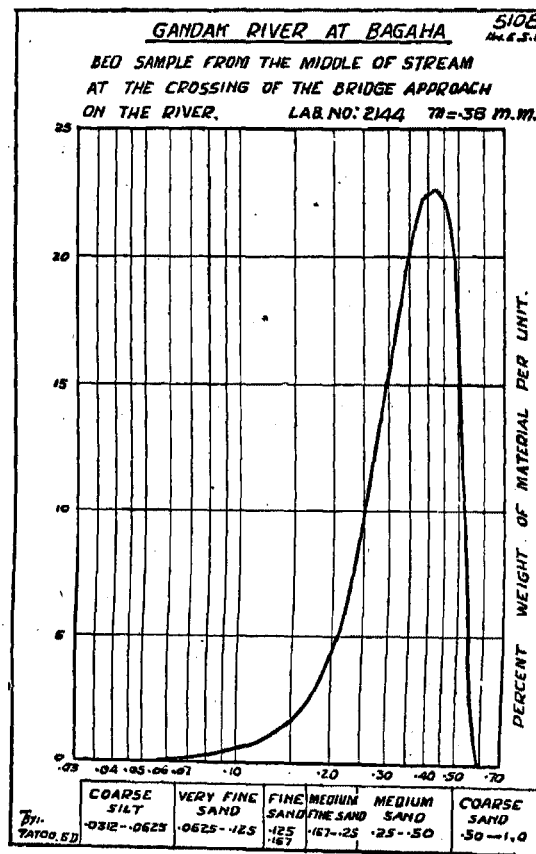
*Plate III of F. J. Spring's Technical Paper 153, "River Training and Control."

Unfortunately, the values of A, R, S, on which the calculation was based are not now traceable.

Fig. 1-9.

FIG. 1-9

(Fig 10 from
A. R. T. 1944
of I.W.E.S.)



It was decided that to reproduce alternative possible river positions in the model, an entry length ≈ 4 miles should be provided in addition to the model length—i.e. equivalent to 10 miles upstream of the bridge to 3 miles downstream. The material selected as most suitable for this experiment was Their sand, passed through 30 mesh sieves, giving $m=0.20$ mm. which would become coarser in the model, because part of the finer material would be washed away.

The longitudinal scale adopted for the model was 1/200, giving a maximum discharge, based on meander length, of 7.5 cusecs $\approx 300,000$ cusecs.

Experiments

Basing the programme of experiments on conditions in the cold weather of 1942-43, a groyne was planned to run out from the high ground along the left bank midway between cross-sections 10 and 11, pointing downstream at an angle of 45°. This would extend about 1,500 ft. into the then existing main channel. The object of this groyne was to attract flow towards the left bank, and so open up the existing channel between cross-sections 9 and 10. Experience showed that as soon as the water reached the high ground on the left bank, this channel would be permanently held, and would develop; while the main stream, which in 1943 flowed across from cross-section 11 to the Chhitauni groyne, near cross-section 6 on the right bank, would simultaneously deteriorate, causing a corresponding further enlargement of the Left Bank Canal, and eventually, the main river would flow through the bridge, as was the case prior to 1924.

INTRODUCTION

Unfortunately, during the flood season of 1943, a cut-off "chord-channel"—marked on the plan in Fig. 1-8 (c)—began to form in the flood season of 1944. This developed rapidly, the Left Bank "arc" channel deteriorating; so that by the end of the flood season, the cut-off channel had become the main channel. The main current then took an abrupt turn at cross section 14, bifurcated one mile downstream of this section, and then passed through the cut-off chord-channel; and by November 1944, it appeared to be taking about 80 per cent of the total discharge. The velocity was then 4 ft./sec. and the depth of water was 22' on the outside of the curve and 8' on the inside of the curve. Fig. 1-8 (c)

Thus, the "arc" channel, which was the main channel in 1943 had, by 1944, become a subsidiary channel carrying only about 20 per cent of the discharge.

This change meant that the scheme contemplated in 1942 was no longer feasible, and that the river would desert the left bank between cross-section 13 and 6; that, in fact, the axis of the stream had permanently changed.

A new cut-off chord-channel may, however, develop after some 10 years, swinging towards the right bank between cross-sections 13 and 11 and back again between cross-sections 11 and 10 towards the left bank channel of 1925, i.e. along the outside of the right guide bank of the bridge. In that case, the pull of the Chhitauni Ghat and the railway embankment on the right bank would rapidly decrease, and it would be possible to divert the river back through the guide banks of the bridge. Should this be achieved, it would then be possible to hold the river permanently along the left bank by building an attracting spur somewhere near cross-section 13. It might also be advisable to build an upsteam island about one-fourth of a meander length above the bridge, just inside the extended line of the right guide bank; but a corresponding island inside the extended line of the left guide bank would not be necessary so long as the high bank on the left bank persists.

Experiments were carried out to see whether the model would give any idea as to what changes are likely to take place. These showed that the "chord" channel would develop further and take nearly the whole flow—see Photo 2978, Sheet 1-21—but due to the fact that further changes will result as much from accretion as scour, and that the time scale for silting is quite different from, and much slower than, the time scale for scour, the model failed to give any indication on this point. Photo Sheet
1-21.

The conclusions arrived at were that—

- (i) it might be possible to divert the river back through the bridge after some ten years; but the betting against this was at least 3 to 1;
- (ii) the depth of foundations of the old bridge are inadequate and would be likely to give trouble;
- (iii) a really good site for a bridge exists 68 miles downstream, where the river flows between hard, and well-defined, banks, which have undergone no appreciable erosion within living memory. The hydraulic gradient flattens immediately below the proposed bridge site, and the highest flood recorded (1903) gave a gentle fall of 9" per mile.

Two borings taken on the proposed bridge site showed sand to a depth of from 30 ft. to 50 ft. below cold weather water level with underlying stiff clay to a depth of 100 ft., which was the limit of depth to which the borings were carried.

This example of an extremely difficult investigation is quoted at length, to show what nonsense it is to hope that a model will tell one what to do in such cases. Nevertheless, in combination with analysis of past behaviour and experience, it gave just the information required, showing that—

- (i) it was very unlikely that the old bridge could be brought into use again;
- (ii) it would always be liable to give trouble;
- (iii) it would be wise to abandon the damaged bridge and build a new one at a highly suitable site 68 miles downstream.

* 1.24 EXPERIMENTS WITH A MODEL OF THE MAHANADI AND KATJURI GROUP OF RIVERS NEAR CUTTACK, ORISSA

Fig. 1-10.

Photo Sheet
1-22.

Fig. 1-10 is a plan showing the Mahanadi and its branches near Cuttack. The Mahanadi flows through a rocky gorge at Gailbank and then expands into a wide stretch, to contract again in another rocky gorge at Naraj, shown in Photo H. 1441, Sheet 1-22, $4\frac{1}{2}$ miles downstream of Gailbank. Here, the Mahanadi bifurcates into two channels—the Mahanadi flowing straight on in a north-easterly direction, and the Katjuri taking off at right angles and following a south-easterly course. Photo H. 1441 shows part of the model extending downstream from Naraj Gorge, showing the Katjuri taking off from the right bank of the Mahanadi just downstream of Naraj Gorge, with the Biranasi Gap between the Katjuri and Mahanadi in the middle distance.

In the first half of the 19th Century, it was observed that the Mahanadi was deteriorating, while the Katjuri was enlarging; so in 1868, Naraj weir (anicut) was constructed across the head of the Katjuri. This has since been reported to have maintained a balance between the two branches. This may be true as regards water levels; but the Katjuri has widened by one mile during the last 100 years.

The Kuakhai takes off from the Katjuri opposite Cuttack, and 3 miles further downstream the Katjuri again bifurcates—where the Surua takes off. Downstream of this off-take, the main Katjuri channel has been silting, while the Surua loop has been eroding its clay banks.

$1\frac{1}{2}$ miles downstream of Naraj, the Mahanadi turns east upstream of Dhabaleswar island; and the Mahanadi anicut was built 7 miles further downstream, at Jobra, in 1869.

The B. N. Railway Bridge was constructed 0.4 mile downstream of the anicut in 1893. The Birupa river takes off from the left bank just upstream of Jobra anicut. It also has an anicut across it, $1\frac{1}{4}$ miles from its off-take from the Mahanadi.

Photo Sheet
1-22.

Below Naraj, the Mahanadi and the Katjuri used to be separated by shallows; but an embankment was constructed in continuation of Naraj Weir for a distance of $2\frac{1}{4}$ miles from the Weir. Downstream of this, some of the Katjuri water spills back into the Mahanadi, during floods, over a length of $1\frac{1}{2}$ miles. This stretch of spill area is called the Biranasi gap. Water from the Katjuri also spills over its right bank during high floods for some distance upstream of the Kuakhai off-take. This spill is called the Barang spill—see Photo 3021, Sheet 1-22.

The distribution of the flood waters between the Mahanadi and Katjuri (observed in 1940 when the discharge was 1,454,500 cusecs) was 40 per cent and 60 per cent respectively. Of the 60 per cent discharge in the Katjuri below Naraj Weir, it is estimated that about $4\frac{1}{2}$ per cent flowed back through the Biranasi gap into the Mahanadi.

No water flows into the Katjuri till the water level reaches 72.2 in the Mahanadi at Naraj. This is the crest R. L. of Naraj weir. As the discharge increases, the proportion flowing into the Katjuri increases rapidly until a maximum proportion is reached. When the flood increases still further, the proportion entering the Katjuri decreases. This reduction in the proportion entering Katjuri at peak discharge is due to the slope in the Mahanadi increasing as the discharge increases—due to the difference between the water levels upstream and downstream of Jobra anicut decreasing.

Flow conditions in the Mahanadi are largely affected by the presence of outcrops of rock along its left bank. Many of these have been noted, some in the bed of the river, but further data in this connection should be collected.

* I. W. E. S. A. R. (T.) 1949 pp. 37-39.

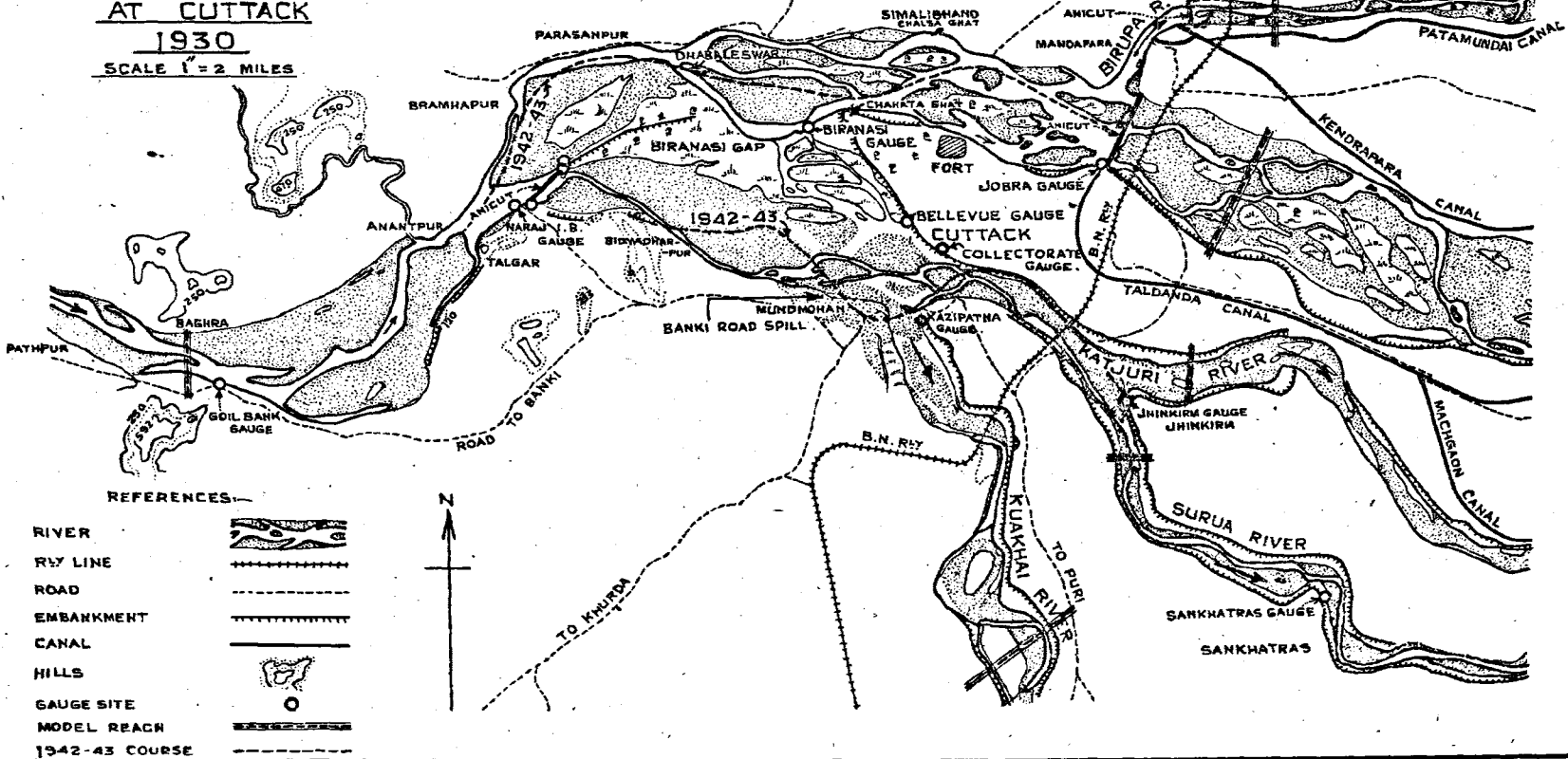
„ „ 1945 Chapter 9, River Control (i) Flood Control.

FIG. 1-10

PLAN SHOWING
MAHANADI & ITS BRANCHES
AT CUTTACK
1930

SCALE 1" = 2 MILES

NO 4795
CENTRAL IRRIGATION AND
HYDRODYNAMIC RESEARCH STN.
POONA.
(FIG. 40. FROM A.R.T.
1943 OF I.W.E.S.P.)



65

INTRODUCTION

Problem

During the past 100 years, the Katjuri has been eroding its right (south) bank towards the Banki Road and has thus widened by a mile and become shallower; and the bed of the old Katjuri channel below the Surua head has risen considerably and flow does not begin till the Mahanadi river discharge upstream of Naraj exceeds 500,000 cusecs, or nearly 1/3rd the estimated maximum flood discharge; so the Surua is gradually developing at the cost of the Katjuri loop by eroding its banks. The Kuakhai now commences to flow when the Bellevue gauge is at 8 ft., which is believed to be several feet lower than in last century. The head of the Kuakhai has widened by one-third, thus indicating that the Kuakhai is now developing. If this development continues, the Barang spill will also increase; and if these two tendencies are not checked, a large increase of flow will occur towards Puri—which the existing channels in that area cannot carry.

Diagnosis

There appeared to be two main causes at work leading to the right bank erosion:—

- (1) The charge of sand entering the Katjuri exceeds what it can carry with the slope available, which we know leads to widening and shoaling.
- (2) So long as Naraj weir is “drowned”, flow tends to be along the left bank; but when a clear standing wave forms downstream of the weir, flow is towards the right bank; so, with varying discharges, the direction of flow tends to vary widely in the Katjuri.

Proposed lines of investigation

To meet these conditions, two possible remedial measures required investigation:—

- (1) How best to reduce the excess sand charge in the Katjuri, and
- (2) whether the Naraj weir can be remodelled so as to prevent the direction of flow from varying with different discharges.

It had also been suggested that flow conditions in the Katjuri would be improved if the Barang spill were closed, and that closing the Biranasi gap might also be desirable; so experiments were carried out:—

- (1) to investigate the immediate effect which closing the Biranasi gap and the Barang spill would have on gauge readings at Cuttack.

Other experiments are to be carried out to determine—

- (2) the effect of improving the Surua to pass an increased discharge;
- (3) the effect of remodelling Naraj Weir:—
 - (a) to reduce sand entry;
 - (b) to control downstream flow;
- (4) how sand distribution and flood discharges in the Kuakhai will be affected if erosion of the Katjuri right bank and Kuakhai head continues;
- (5) whether the cold weather channel of the Katjuri can be brought back to the left bank at Cuttack—which is considered to be very desirable in the interests of the people of Cuttack;
- (6) the effect of constructing a bridge with restricted waterway across the Mahanadi near Cuttack Fort, which would presumably cause considerable silting along the right bank between the bridge and the anicut;
- (7) the possibility of improving the present control at Mahanadi anicut, which has an antiquated system of centre sluices,

INTRODUCTION

Problem

During the past 100 years, the Katjuri has been eroding its right (south) bank towards the Banki Road and has thus widened by a mile and become shallower; and the bed of the old Katjuri channel below the Surua head has risen considerably and flow does not begin till the Mahanadi river discharge upstream of Naraj exceeds 500,000 cusecs, or nearly 1/3rd the estimated maximum flood discharge; so the Surua is gradually developing at the cost of the Katjuri loop by eroding its banks. The Kuakhai now commences to flow when the Bellevue gauge is at 8 ft., which is believed to be several feet lower than in last century. The head of the Kuakhai has widened by one-third, thus indicating that the Kuakhai is now developing. If this development continues, the Barang spill will also increase; and if these two tendencies are not checked, a large increase of flow will occur towards Puri—which the existing channels in that area cannot carry.

Diagnosis

There appeared to be two main causes at work leading to the right bank erosion:—

- (1) The charge of sand entering the Katjuri exceeds what it can carry with the slope available, which we know leads to widening and shoaling.
- (2) So long as Naraj weir is "drowned", flow tends to be along the left bank; but when a clear standing wave forms downstream of the weir, flow is towards the right bank; so, with varying discharges, the direction of flow tends to vary widely in the Katjuri.

Proposed lines of investigation

To meet these conditions, two possible remedial measures required investigation:—

- (1) How best to reduce the excess sand charge in the Katjuri, and
- (2) whether the Naraj weir can be remodelled so as to prevent the direction of flow from varying with different discharges.

It had also been suggested that flow conditions in the Katjuri would be improved if the Barang spill were closed, and that closing the Biranasi gap might also be desirable; so experiments were carried out:—

- (1) to investigate the immediate effect which closing the Biranasi gap and the Barang spill would have on gauge readings at Cuttack.

Other experiments are to be carried out to determine—

- (2) the effect of improving the Surua to pass an increased discharge;
- (3) the effect of remodelling Naraj Weir:—
 - (a) to reduce sand entry;
 - (b) to control downstream flow;
- (4) how sand distribution and flood discharges in the Kuakhai will be affected if erosion of the Katjuri right bank and Kuakhai head continues;
- (5) whether the cold weather channel of the Katjuri can be brought back to the left bank at Cuttack—which is considered to be very desirable in the interests of the people of Cuttack;
- (6) the effect of constructing a bridge with restricted waterway across the Mahanadi near Cuttack Fort, which would presumably cause considerable silting along the right bank between the bridge and the anicut;
- (7) the possibility of improving the present control at Mahanadi anicut, which has an antiquated system of centre sluices,

THE BEHAVIOUR AND CONTROL OF RIVERS AND CANALS

difficult to control and apt to become jammed by debris if manipulated during the flood season;

(8) how to exclude sand from the Kendrapara canal, which takes off above the Birupa anicut.

(7) and (8) would require additional large-scale, part-models.

It was in order to study such questions that the Orissa Flood Advisory Committee proposed that model experiments should be carried out at Poona.

As explained in Chapter 14, the model which reproduces the Mahanadi from Goilbank to downstream of the Railway—Fig. 1-10—presents great difficulties. The effects of these have been reduced so far as practicable, and most important information has been obtained.

In the first series—which alone will be dealt with here—the effect of closing spills was examined:—

In (i), the Banki road spill over the right bank of the Katjuri—see Photo 3021, Sheet 1-22—was closed, keeping the Biranasi gap on the left bank open.

Photo Sheet 1-22. (ii), a curved embankment was constructed along the left bank—see Photo 3021, Sheet 1-22—closing off the Biranasi gap and the Banki spill was left open.

while in (iii), both the Biranasi gap and the Banki spill were kept closed.

Fig. 1-11. Fig. 1-11 shows that $1\frac{1}{2}$ million cusecs in the Mahanadi above Naraj (i) would cause a rise of $3\frac{1}{2}$ ft. at Cuttack, and hence a major disaster,

(ii) would cause a rise of 8 ft.,

and (iii) would cause a rise of 4.7 ft.

This information shows how extremely unwise it would be to close off the Banki spill, a proposal seriously contemplated at one time.

The second alternative is, however, feasible; because the modified Katjuri channel would improve rapidly and would, in a few years, carry its discharge with a lower Gauge at Cuttack than at present.

1.25 MODELS OF THE HOOGHLY ABOVE CALCUTTA*

Due to the success of the Hardinge Bridge experiments, the station became more widely known, and thereafter the number and variety of enquiries increased rapidly. These included several tidal investigations, one of which was in connection with the Hooghly above Calcutta, duplicate models being constructed.

Photo Sheet 1-23. Photo H. 1467, Sheet 1-23, shows a tidal bore rounding Palta point—12 miles north of Calcutta—on its way up the Hooghly in the long model of the Hooghly.

This model was more than 1,000 ft. long to start with, tidal flow being generated only at the downstream end; but it was found that better results could be obtained by shortening the model to 500 ft., producing the ebb flow at the upper end.

Photo Sheet 1-23. The main object of this long model, with a natural vertical exaggeration of 5, was to reproduce lines of flow and tidal conditions, to visualise the general problem and to know the flow conditions to impose in a short, large-scale model with only a slight width reduction, suitable for examining the particular problem in hand—how to minimise the effect of a “knuckle” produced by tidal action. The short model, with low “width reduction”, is illustrated in Photo H. 1458. In this case tides had to be generated at both ends, and the slope exaggeration produced in this way was much greater relatively than the vertical exaggeration in terms of width—i.e. than the width-reduction scale.

* C. I. & H. R. S. A. R. (T) 1940-41, pp. 18, 19.

1941-42, pp. 23, 24.

1942-43, pp. 18, 19.

L. W. E. S. A. R. (T) 1943, pp. 25, 37.

1944, p. 7.

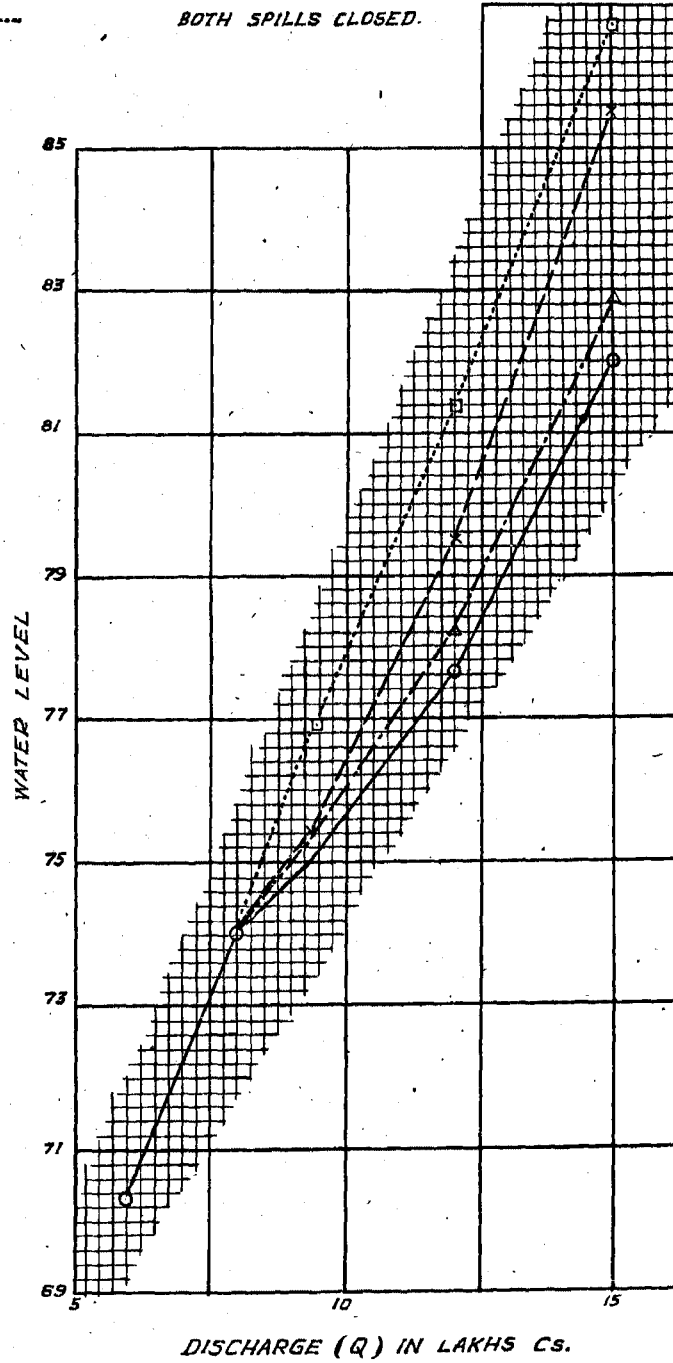
COLLECTORATE GAUGE.

FIG. 1-11

NO. 5970.
I.W. EXPT. STN. POONA.
(FIG. 21. FROM A.R.T.
1945 OF I.W. E. STN.)

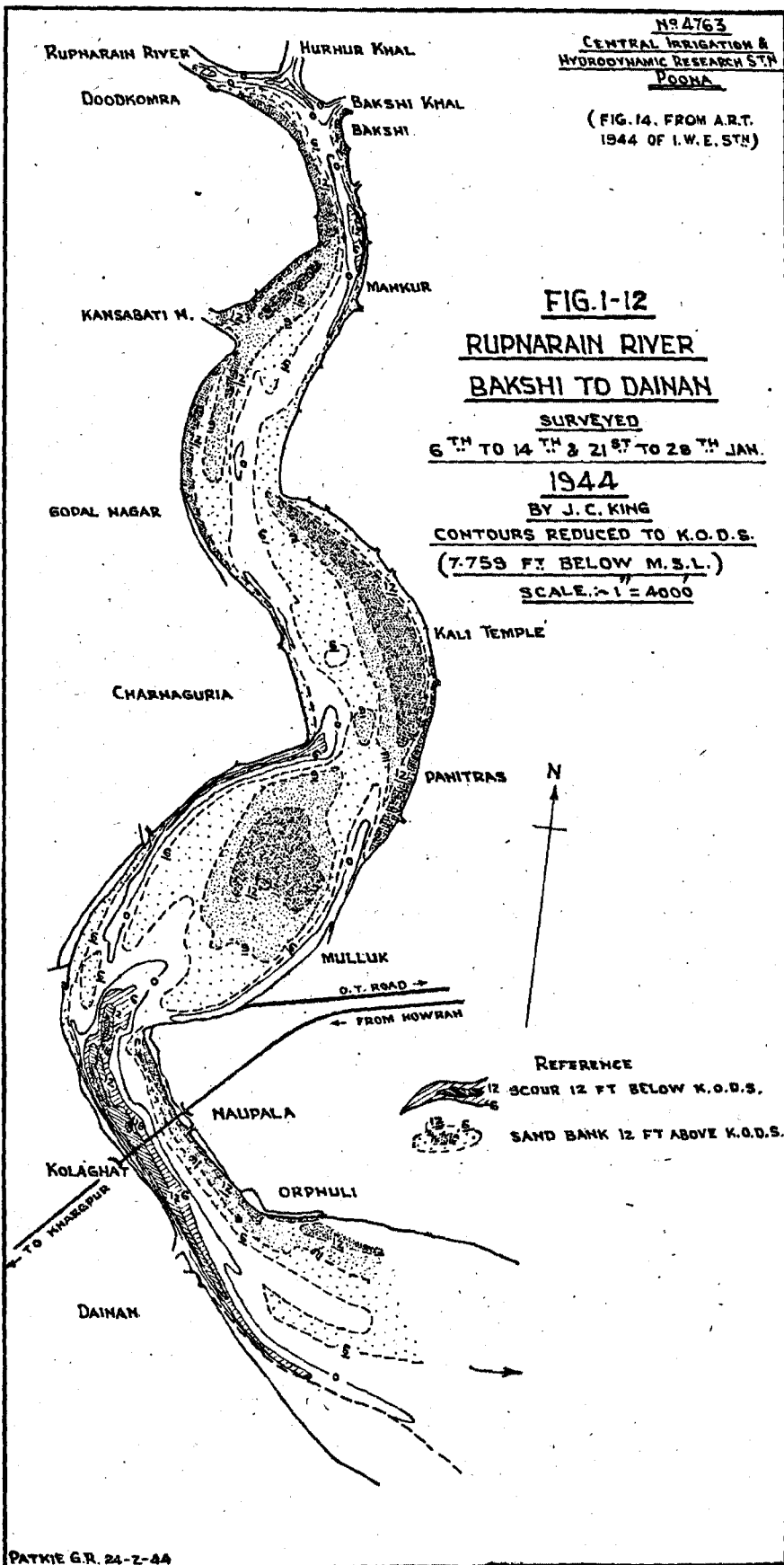
SHOWING W. L. OBSERVED AT
THE COLLECTORATE GAUGE ON SOUTH
SIDE OF CUTTACK.

- ——— EXPT. WITH EXISTING CONDITIONS.
- x ——— " " BANKI ROAD SPILL CLOSED.
- △ ——— " " BIRNASI SPILL CLOSED.
- ——— BOTH SPILLS CLOSED.



By. Parkhi
23/2/46

G. R. Z. P. POONA, 1946



INTRODUCTION

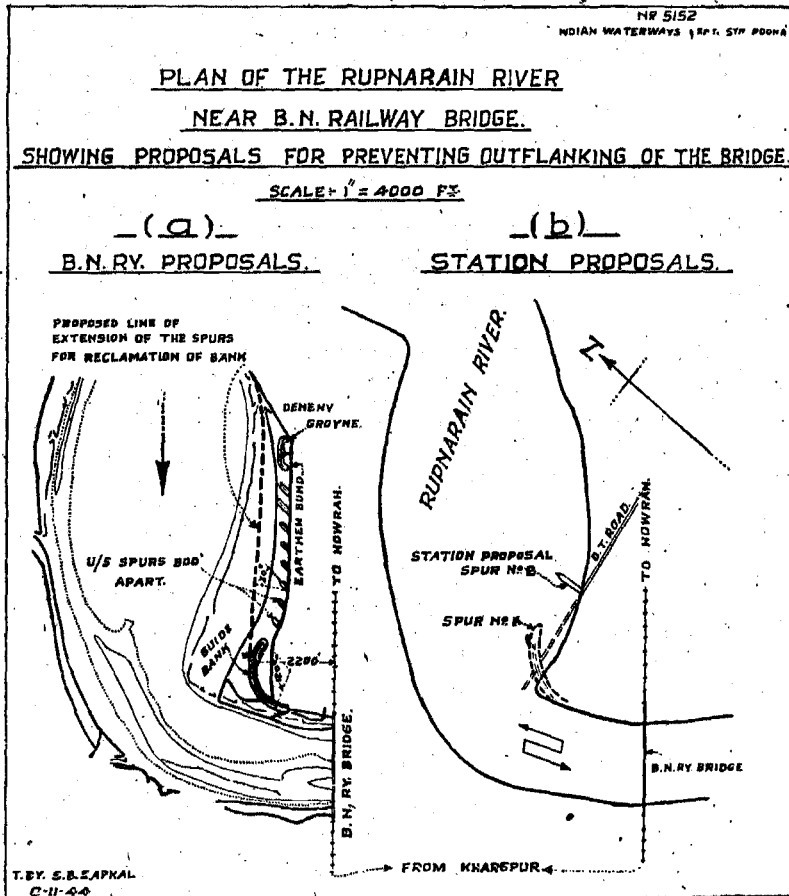
Various alternative treatments were tested; and it was found that the "knuckle" cannot be entirely eliminated; because it is caused by the excessive curvature of the bend—so can only be removed by easing the curve.

1:26 RUPNARAIN MODEL INVESTIGATION

In the case of the Rupnarain River, which flows into the Hooghly near Hooghly Point, opposite the ill-famed "James and Mary" shoal, the river had eroded 1,400 ft. of its left bank since 1938, and had advanced to within 1,500 ft. of the main line of the Bengal-Nagpur Railway connecting Calcutta with Bombay and Madras—see Fig. 1-12—and it was considered that the railway embankment to the east of the river was in grave danger of being breached. Had that occurred, the bridge would have been outflanked.*

Little time was available for deciding what action to take—for it was then near the end of December and all work had to be completed by June—but a small rigid model was fortunately available, which made it possible to decide, without delay, what was the best action to take. Three alternatives were considered—see Figs. 1-13 (a) and (b).

Fig. 1-13



(Fig. 16 from A. R. T. 1944 of I. W. E. Stn.)

* Maritime Paper No. 3 Inst. C. E. "Training Works constructed in the Rupnarain River in Bengal, after model experiments, to prevent further bank erosion endangering the Bengal-Nagpur Railway linking Calcutta with Bombay and Madras" by Sir Claude Inglis, C. I. E.

THE BEHAVIOUR AND CONTROL OF RIVERS AND CANALS

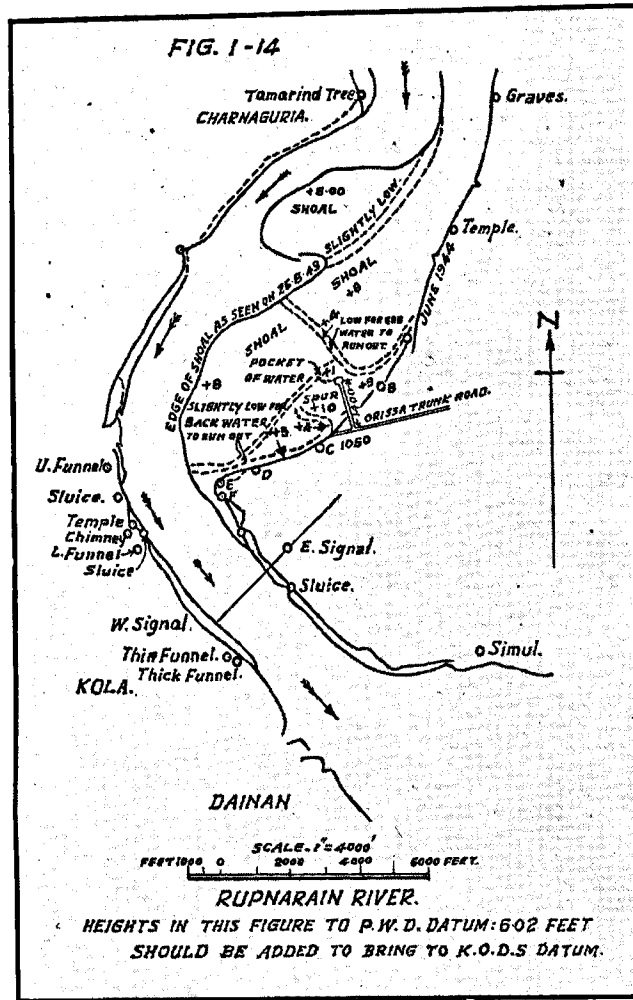
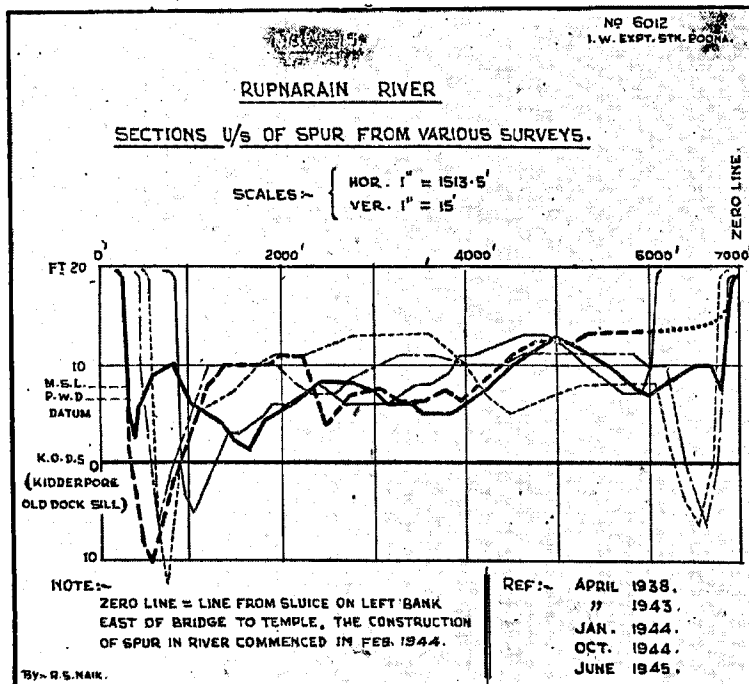


Fig. 1-15



INTRODUCTION

The design selected, which was the only design which could have been completed in time, consisted of a single groyne, 850 ft. long, pointing 60° upstream, at a point some 5,000 ft. above the east abutment of the bridge. Work was completed a month before the break of the monsoon. The groyne not merely stopped further erosion, but led to the main stream of the river moving some 2,000 ft. away from the left bank during the 1945 flood season, to flow naturally through the bridge. Fig. 1-14 shows the conditions in December 1945 and Fig. 1-15 shows cross-sections of the river near the spur observed in April 1938, 1943 and January 1944 before the groyne was built, and in October 1944 and June 1945 after the groyne was completed. These show how the attack was moved away from the eroding left bank as a result of the groyne.

Fig. 1-14.
Fig. 1-15.

1-27 REPORT ON "CAUSES OF RIVER BREACHES IN SIND AND STEPS NECESSARY TO MINIMISE THE DANGER OF A RECURRENCE"*

In 1942, when a major breach occurred in the right embankment of the Indus, 20 miles upstream of Sukkur, 2,600 square miles of country were inundated. The Author was called in to investigate and report on the "causes of breaches in the river embankments in Sind and the steps necessary to minimise the danger of a recurrence." This is dealt with fully in Chapter 5.

1-28 REPORT ON "CAUSES OF ACCRETION IN THE BED OF THE INDUS AT SUKKUR†

In 1944, the Author was asked to report on "the causes of accretion in the bed of the Indus at Sukkur Barrage and how to reduce the rate of rise of flood levels." This is dealt with in Chapter 5.

1-29 DESIGN OF LOWER SIND BARRAGE

In 1944, the Station was asked to do model experiments in connection with the design of the proposed Lower Sind Barrage.‡ As a result of those experiments, the proposal to build the Barrage in the dry, and then to divert the river through it, was condemned; and an alternative site was recommended at a point three miles further downstream where conditions were favourable for excluding sand from the Left Bank Canals.** Though this design would exclude excess bed-material, it was later considered desirable—after consideration of points raised by the Author—to build the Barrage some 22 miles upstream at a point above Kotri, in order to improve command and to reduce unwatering difficulties during construction.

1-30 OTHER INVESTIGATIONS DEALT WITH IN ANNUAL REPORTS OF THE POONA STATION

Many other investigations have been dealt with in Bombay P. W. D. Technical papers and Annual Reports of the Central Irrigation and Hydrodynamic Research Station, Poona; but enough have been described to show the wide scope of the work and the importance of model experiments.

Appendix

* I. W. E. S., A. R. (T) 1943, pp. 10-21, "Summary of Technical Enquiry into the Causes of Breaches in River Embankments in Sind and Steps required to minimise the danger of a recurrence." by C. C. Inglis.

† I. W. E. S. A. R. (T) 1944, pp. 99-104.

‡ I. W. E. S. A. R. (T) 1944 pp. 55-56 and 98.

** I. W. E. S. A. R. (T) 1944, pp. 55-57 and 97 and 99.

1.31. THE IMPORTANCE OF MODEL INVESTIGATIONS AFTER FIELD INSPECTION AND ANALYSIS OF DATA

Though advice has been given in many cases without the use of models, this has rarely been because models were considered unnecessary; but because the time, or facilities available were inadequate. In such cases, advice has been based on field experience combined with the results of earlier model investigations. Similarly when advice was given without inspection, this was not because inspection was considered to be superfluous, but because time could not be spared; and the Author's clear-cut opinion is that in all but very simple cases, river problems should be investigated on the spot, by personal inspection and discussion with local officers and the people; because this alone makes it possible to visualise conditions adequately, to arrive at a comprehensive diagnosis of the case. It is unlikely that any important work—whether it be a new construction or remodelling an old work—will be carried out in future in India without model experiments. The limitation of models must, however, be kept clearly in mind, and in the case of mobile models, success will depend mainly on the ability of the person directing the experiments to visualise and diagnose the conditions which have led up to the present position and, in the light of that knowledge, to predict changes which will take place in years to come. A model cannot possibly give reliable information unless the conditions imposed are correct.

It is difficult, after many years of experience of a subject, to decide what has helped one most to understand conditions and to diagnose causes; but comparisons of model studies with field results are undoubtedly a most valuable aid, in that models enable one to understand what is happening in a river in far greater detail than is possible from mere inspection, while field inspection enables one to understand where the model fails to reproduce river conditions, and why, and how far inherent differences must be taken into account.

Reproducing in a model what has occurred in a river does not prove that the experimenter can predict the future—as is so generally assumed. Indeed reproducing river conditions is apt to degenerate into "fudging" unless differences in the model and the river are admitted, and the reasons for them are carefully thought out.

As an example:

On one occasion, before going on tour, the Author had explained to a new assistant why it was impossible to reproduce in a model of the Jumna river certain changes which had taken place during the preceding flood season. On return from tour, the assistant told him he had had no difficulty in obtaining the correct result. What he had actually done, however, was to develop a main channel which was roughly in the same position as the main channel in the river at the end of the flood season; but whereas the model channel had altered its course only once, the river had changed course four times during the flood season, leaving vestiges of three channels, one of which would re-open the following year—as the main channel—and the distribution of coarse and fine bed material was consequently highly complex in the river instead of simple, as in the model.

As explained in Chapter 14, the true scales for accretion and scour being different in river models, they can only give closely correct results during periods of scour; and where portions of the model are rigid, the rigid parts of the model have to be distorted to give even approximate results. River model cannot, therefore, be used to predict long period changes; but they are most valuable—provided these inherent limitations are allowed for—in helping those investigating river problems to understand conditions and, in the light of this knowledge, to predict future changes.

INTRODUCTION

1.32. ACKNOWLEDGMENTS

Although the Author has tried to give full credit to the work of others, he is aware that his acknowledgments are inadequate; and in many cases the station has arrived at conclusions independently, only to find later that others had found out the same facts, sometimes years earlier.

The person who influenced the Author's work most was the late Henry Fitzgerald Beale, Chief Engineer, Bombay, who retired in 1916. It was he who obtained sanction to start the Special Irrigation Division, which originated the research work carried out in Bombay and Sind and led indirectly to the development of the Central Irrigation and Hydrodynamic Research Station at Poona. His ideas were far in advance of his time, and it was he who planned and prepared the schemes for the Deccan Canals with their associated dams—including the enlarged Bhatghar dam (Lake Whiting)—the biggest mass masonry dam in the world; and Bhandardara (Lake Arthur Hill) which, when constructed, was the highest dam in the world—270 ft. When Mr. Beale had only two years service, his immediate superior, Mr. Whiting, himself a great engineer, wrote in his Confidential Report: "This young officer is fit for any post of authority."

Other engineers to whom the Author has been especially indebted include E. S. Lindley, and the late H. W. Nicholson of the Punjab; S. C. Mould, A. Gordon and C. G. Hawes, Chief Engineers in Sind, each of whom called him in, again and again, to discuss and advise; but especially he must acknowledge his debt to E. S. Crump of the Punjab, the most brilliantly original thinker of all the hydraulicians produced by the P. W. D. in India in recent years; and to Gerald Lacey of the U. P., who first correlated the dimensions of regime channels with the discharge and material exposed on the bed. Since then, Lacey's ideas have formed the background of every advance made in India in connection with flow in alluvial channels.

The Author received much assistance at the Station from his Deputies, Messrs. A. R. Thomas, J. S. Reid and Rao Bahadur D. V. Joglekar. The last, during a period of more than 20 years, has produced an enormous amount of first class work, and the Author's long association with him has been most happy and fruitful.

To the engineering, field, and office staff—too numerous to mention individually—much credit and the Author's grateful thanks are due.

Lastly, he must thank Mr. K. K. Framji, his successor as Director, Indian Waterways Experiment Station, Poona, for having had photographs and figures prepared and for supplying much necessary data. Without his co-operation the Report could not have been completed in its present form.

CHAPTER 2

REVIEW OF FACTORS CONTROLLING FLOW IN ALLUVIAL CHANNELS

	Page.
2.00. Introduction	85
2.01. Equilibrium	85
2.02. Regime as defined by Lacey	86
2.03. The Lacey regime formulæ	87
2.04. Ripple turbulence	87
2.05. Dune movement and rough turbulence... ..	87
2.06. Sheet movement	88
2.07. Smooth bed and smooth turbulence	88
2.08. Dominant discharge	88
2.09. Adjustments... ..	88
2.10. Continuity of progression of material down a river :	88
(a) boulders	89
(b) pebbles	89
(c) gravel	89
(d) sand	89
2.11. River characteristics	90
2.12. River characteristics are mainly determined by rate of fluctuation of conditions.	90
2.13. Model experiments in connection with bed movement	91

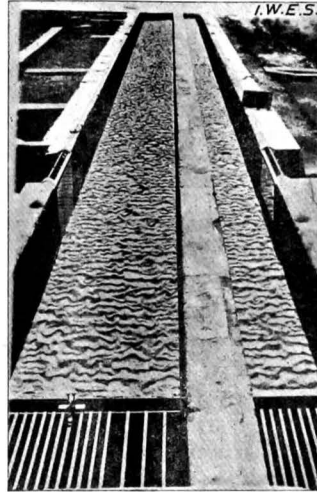


Photo 3096. Ripples formed on the bed in sand load experiments in $4\frac{1}{2}$ ft. and $2\frac{1}{4}$ ft. flumes with $4\frac{1}{2}$ cusecs discharge flowing in each.



Photo 2984. Dunes in $2\frac{1}{4}$ ft. channel carrying 5 cusecs.

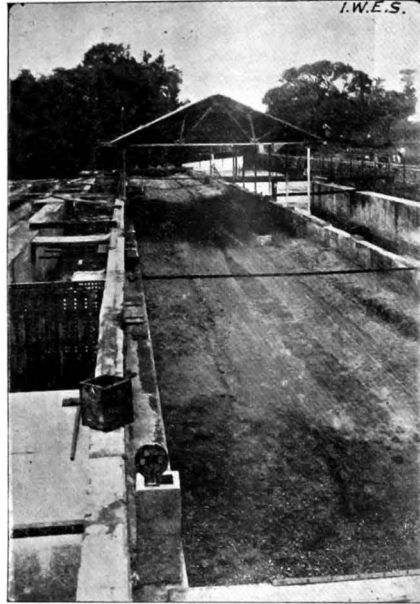


Photo 2977. Sand channel experiment, Series X, Experiment 2. 2 cusecs discharge $m = .43$ m.

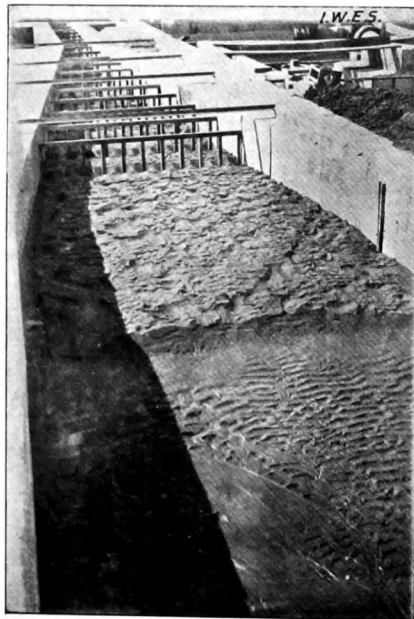


Photo 2760. Elutriator Channel for separating different grades of bed material. Shows a charge of medium sand beginning to blanket the original deposit of silt. Incipient ripple of fine sand forming on top of silt bed.

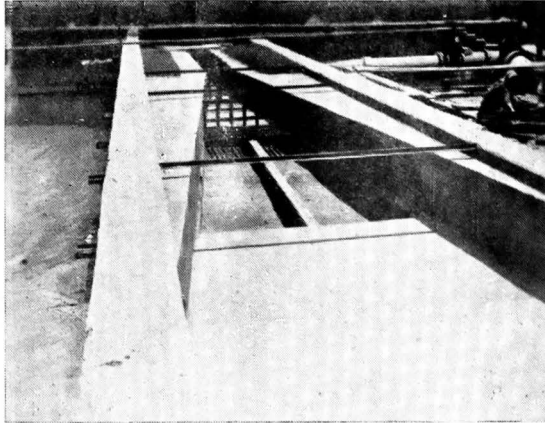


Photo 2932. Photo showing elutriator; note the system of vertical grids to create turbulence and horizontal grids over a pit to collect falling sand without affecting flow. Pit uncovered to show dimensions. (Looking upstream.)

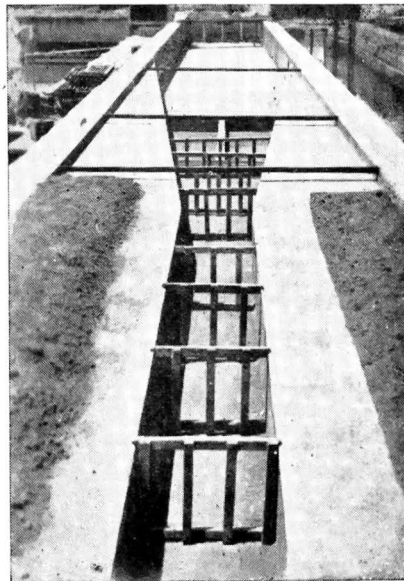


Photo 2933. Same as 2932 but looking downstream.
Photos from I. W. E. S. A. R. (T) 1943,
pp. 64-66.

FACTORS CONTROLLING FLOW

CHAPTER 2

REVIEW OF FACTORS CONTROLLING FLOW IN ALLUVIAL CHANNELS

2.00. INTRODUCTION

In order to make it possible to visualise river changes, the highly complex problem of the relation between the flow of sand and water in alluvial rivers must be simplified and clarified.

2.01. EQUILIBRIUM

The basic fact regarding flow in alluvial channels is that there is a never-ceasing tendency to attain equilibrium—that is to say to attain a condition in which the load is carried with the minimum expenditure of energy for the conditions. This means that conditions at every part of the periphery must be in equilibrium. Though such a condition is never fully attained in nature—except under controlled conditions—it is closely approached where a channel runs with constant discharge, charge, and water temperature for a long period of time.

There is only one equilibrium slope and shape for any particular set of conditions; but where turbulence is low, there may be a range of stable channels.

The slope increases with grade and charge; but the length of the perimeter is little, if at all, affected by grade; but increases with increase of charge.

In a channel with a rigid boundary and no material in movement, flowing with constant discharge, the most efficient shape is a semi-circle; but if a small amount of material is in movement, consisting of a normal population of particles of various sizes, the equilibrium section—except in so far as the angle of repose affects the case—approximates to a semi-ellipse.*

Under such conditions, the various grades of material in movement tend to sort themselves into suspended material and bed material—part of which saltates†, the finer particles tending to deposit at the sides—where the velocity is low—to form the banks of the channels; while the coarser grades move along the bed, with a gradation of mean size from coarser in the middle of the channel to finer towards the banks.

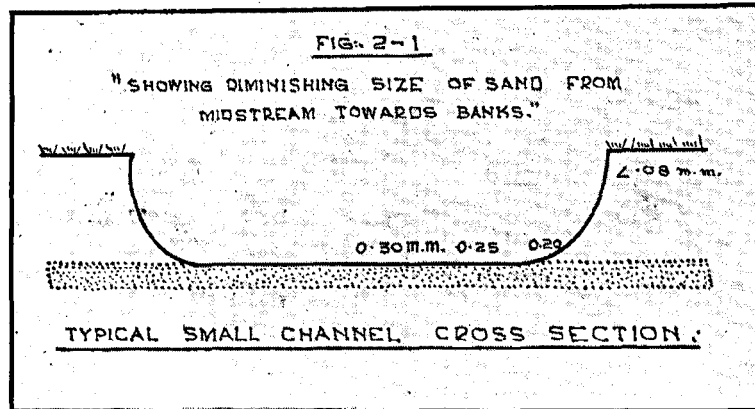
As pointed out by C. G. Hawes in 1934‡, berms consist almost entirely of material finer than 0.08 mm.; while the bed, except when temporarily blanketed by fine material, consists for the most part of material coarser than 0.08 mm.

A channel carrying a heavy charge of bed material tends to spread its load over a considerable width, so that the channel is wide and shallow; but where fine materials consolidate and mature, they become relatively tough and inerodible especially where they are knit together by grass and binding vegetation. The channel then has a relatively flat bed and steep side-slopes, limited only by the angle of repose; so that, in small channels, it is not uncommon to find a considerable amount of overhang.

*Gerald Lacey "Stable Channels in Alluvium" Paper 4736, Inst. C. E.

†Grove, Karl Gilbert "Transportation of Debris by Running Water" U. S. Geol. Survey Prof. Paper 86, Government Printing Office, Washington D. C. U. S. A. (1914).

‡Note on the Result of Silt Analysis carried out in the D. and E. Division, Bombay P. W. D. T. P. 53.



The probable reason for the abrupt change at the toe of a bank—from a flat bed to steep side-slopes—is that there is a change of law governing the relation between m , the weighted mean diameter of material exposed on the bed, and the terminal velocity of the same particles falling in water— V_s . The rate of fall of grains finer than 0.08 mm. follows Stokes' law, V_s being $\propto m^2$, whereas with particles greater than about 0.15 mm. but less than about 5 mm., $V_s \propto m$ (approximately) and for particles greater than 0.6 mm. there is another change of law, V_s then being $\propto m^2$. For grades between .08 and .15 mm. and .5 mm. and .7 mm. there are transitional regions*.

To the first of these three grades of material the term silt will be applied in this Report; to the second, sand; and to the coarser grades >6 mm. coarse sand, gravel, pebbles, etc.

In rivers, the discharge, the charge, and the temperature of the water all vary; and although the discharge may be maintained almost constant at the head of a canal channel, the charge of material and the temperature of the water vary widely at different times of year; so final equilibrium is never fully attained, though each channel is continuously re-adjusting its conditions towards equilibrium.

2.02. REGIME AS DEFINED BY LACEY.

As the term is at present understood in India (1946) regime is a special case of equilibrium, in which the charge in movement is "the regime silt charge", defined by Lacey† as "the minimum transported load consistent with a fully active bed. Full activity is such that any reduction would lead to effective partial rigidity and at the limit to full rigidity of the bed."

* Punjab Irrigation Research Institute Report for Year ending April 1937; also William W. Rubey "Settling Velocities of Gravel, Sand and Silt Particles". American Journal of Science, Vol. XXV, No. 148 of April 1922.

† "A General Theory of Flow in Alluvium" Inst. C. E. Paper 5519 (1946), by Gerald Lacey.

FACTORS CONTROLLING FLOW

2.03. THE LACEY REGIMÉ FORMULÆ.

When re-stated in terms of discharge and the Lacey slope factors—see Chapter 3—

LACEY REGIME FORMULÆ

$$P = 8/3 Q^{1/2} \quad (1)$$

$$R = .4725(Q/f)^{1/3} \quad (2)$$

$$\frac{P}{R} = 5.644 Q^{1/6} f^{1/3} \quad (3)$$

$$A = 1.26 \frac{Q^{5/6}}{f^{1/3}} \quad (4)$$

$$V = .7937 Q^{1/6} f^{1/3} \quad (5)$$

$$S = \frac{.000547 f^{1/6} \cdot f_{RS}^{2/3}}{Q^{1/6}} \quad (6)$$

$$VR = Q/P = q = \frac{2}{3} Q^{1/2} \quad (7)$$

$$SV = \frac{(1 f_{sv})^2}{48} = .000434 f_{sv}^2 \quad (8)$$

(Note.—At regime, f , f_{RS} , f_{SQ} , and f_{SV} are numerically equal, but not dimensionally the same, due to g and ν having been omitted for simplicity.

Formula (2) shows that with the same discharge and charge, the depth varies inversely as the grade of material exposed on the bed—which holds equally in straight and meandering channels; and formula (6) shows that with a constant sand factor, the slope required varies inversely as the one-sixth power of the discharge; and for constant slope, the sand factor tends to vary as the one-tenth power of the discharge.

2.04. RIPPLE TURBULENCE

In theory, the assumption of a regime charge is required to explain divergences from regime; but in nature, no such condition persists; and for practical purposes it is unnecessary to assume a regime charge, but only that the bed charge is very small. This is because, with ripples on the bed, a condition analogous to smooth turbulent flow exists, in which the curve relating the log of the Reynold's number— $\log (VR/\nu)$ —and the log of $(gRS)/V^2$ is parallel to the line for smooth turbulent flow; the 'shift' from the smooth turbulent line being determined by the configuration of the ripples which, in turn, is determined by the grade and charge of the material in movement. The Author holds, therefore, that regime comprises a band of values, not a knife-edge value, as hypothesized by Lacey; but, for the present, we had better accept the Lacey concept.

2.05. 'DUNE MOVEMENT' AND ROUGH TURBULENCE

Where the charge in movement is great, ripple movement is replaced by dune movement, flow changing from ripple turbulent to rough turbulent.

THE BEHAVIOUR AND CONTROL OF RIVERS AND CANALS

2.06. SHEET MOVEMENT

Under extreme conditions the rate of movement of bed material becomes much greater locally, movement then being in the form of sheet movement.

2.07. SMOOTH BED AND SMOOTH TURBULENCE

Finally there is the case of a very heavy charge of fine silt in suspension, which is so great that it cannot be carried with the slope available (either in parts of the river or in an off-taking channel). Part of the charge then falls to the bed and gets mixed with the coarser material forming the bed, so that the weighted mean diameter of the bed-mix decreases and may continue to do so until the bed-mix becomes so fine that ripples disappear. The bed is then immobile.

2.08. DOMINANT DISCHARGE

In para. 2.01 and 2.02 two idealised concepts were explained—

(i) 'equilibrium', in which discharge, charge, and temperature remain constant long enough for the channel to develop optimum slope and shape,

and (ii) 'Lacey regime', in which in addition to constant discharge, charge, and temperature, the charge is the minimum consistent with a fully active bed.

To connect these with what occurs in nature we must link them with a particular discharge—the dominant discharge, at which equilibrium is most nearly approached—which may be looked on as the 'integrated effect of all varying conditions over a long period of time'. At this discharge—with its associated dominant charge, slope, and water temperature—the channel may be looked on as possessing a year to year stability.

This concept of a dominant discharge enables us to apply the Lacey formulæ quantitatively to river analysis provided we make suitable assumptions regarding coefficients.

2.09. ADJUSTMENTS.

Looking on dominant conditions as the norm, all variations in temperature, charge, and discharge lead to readjustments.

Small adjustments are effected, without appreciable alteration of overall slope, by changes in the material exposed on the bed. Such changes occur either by deposition—where there is an excess charge—or by scour, where the charge is in defect. Deposition of finer grades, by reducing the weighted mean grade of material exposed on the bed, reduces 'drag'; whereas during scour, the finer particles are washed out of the bed-mix, thus leaving coarser grades exposed on the bed, thereby increasing the 'drag'.

Where the discharge and charge of materials in movement vary widely, major adjustments are brought about by meandering—see Chapter 4.

2.10. CONTINUITY OF PROGRESSION OF MATERIAL DOWN A RIVER.

A point which may seem puzzling is how there can be continuity of progression when a river consisting of pebbles and coarse sand near the hills changes progressively to fine silt in its lower reaches near the sea. This change is due to several causes.

FACTORS CONTROLLING FLOW

Very slow solution and weathering are taking place continuously throughout the length of the river, and abrasion is occurring where the velocity exceeds about 2 ft. per sec., being especially great near the hills where the velocities of flow are high and the materials comprising the bed are large; but the chief factor causing reduction of grade down a river is "sorting". This is due to the rate of movement varying inversely as the size of particles.

Thus,

(a) *Large boulders* move only under extreme conditions, and tend to accumulate as diagonal ridges, forming natural weirs, which frequently lead to the development of islands. Once boulders have 'packed' they may not move again for centuries; so, on the average their movement is excessively slow—so slow that they never extend far from the hills.

(b) *Pebbles*, rounded by abrasion, also progress intermittently and consequently very slowly.

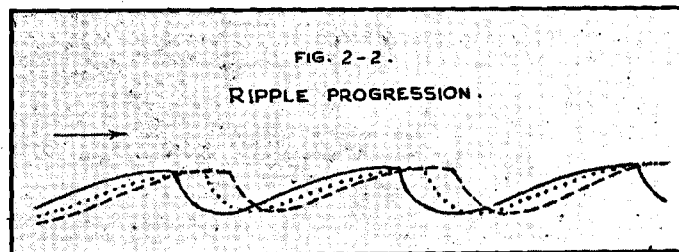
(c) *Gravel* travels much faster—especially where the depth of flow is small—probably because it tends to move before sand is set in motion due to its being larger and hence acted on by high velocity filaments than small particles. This was most markedly demonstrated in the experiments with the Sarda River model. Sand is often missing, or present only in small quantity; because it tends to get trapped in pockets, where it may remain for long periods of time or even permanently—after being covered by sand.

(d) *Sand* movement has been studied in great detail at Poona and has been described in a Paper entitled "Bed Ripples and Bed Dunes"* based on observations carried out by Mr. S. C. Desai, B. E.—see Appendix 2.

Sand movement takes place in ripples, dunes, or by sheet movement.

Where there are ripples, movement is intermittent—because it is only the surface of the ripple which is in movement at any one time, so particles in the body of the ripple remain stationary until they are again exposed at the surface by the downstream progression of the ripple profile.

Fig. 2-2



The rate of movement of particles of various sizes comprising a ripple are not the same. Thus the coarser particles tend to settle at the bottom of each ripple trough and may not move when next exposed, forming a semi-permanent trough-bed horizon. Slightly finer particles roll up the sloping face and drop into the succeeding trough and repeat this as each ripple passes. Still finer particles saltate from ripple to ripple; and yet finer particles are thrown into temporary suspension, returning to the bed some considerable distance downstream. Thus,

* I. W. E. S. A. R. (T) 1943 "Bed Ripples and Bed Waves" pp 53-59.

THE BEHAVIOUR AND CONTROL OF RIVERS AND CANALS

even in ripple movement, there is a marked sorting process, the coarsest particles tending to remain stationary, while there is every gradation of velocity of movement, the finer grades (.08 mm.) being carried for long distances in suspension with approximately the same velocity as the water.

The above description of the various kinds of movement shows why there must be a progressive diminution in particle size down all channels—due to finer particles moving more rapidly than coarser particles—but superimposed on this normal movement and its associated gradient of particle size, there are abnormal movements, due to divergence from the normal dominant conditions.

2.11. RIVER CHARACTERISTICS

Each river has distinctive characteristics, depending on the quantity and quality of material washed into it and its rate of rise and fall during floods.

It must be remembered that *the behaviour of a river throughout its whole length depends primarily on the material washed into it—mostly near the hills or from tributary streams.*

This self-evident fact has been frequently overlooked. Thus, it has been stated that “the only requirement for meandering is bank erosion;”^{*} but the material of the bank and the slope of the river have been determined by the water and material entering the river in the past; and the overall rate of bank erosion and meandering are determined by the material in movement down the river, not the meandering; and the rate of erosion of the bank is essentially a dependant variable not the primary cause (except locally and temporarily).

2.12. RIVER CHARACTERISTICS ARE MAINLY DETERMINED BY RATE OF FLUCTUATION OF CONDITIONS

At dominant discharge in a river, the charge and slope are normally considerably greater than Lacey's regime values; and each river has its own characteristics—determined by the charge (i.e. the load relative to the discharge) and rate of fluctuation of discharge. Thus, where a channel flows with relatively constant conditions—as in a canal—conditions closely approach regime, and this is also true of some rivers; but in flashy rivers, the slope and width are greatly exaggerated and the depth is relatively small. This is because a heavy charge requires a high degree of turbulence to carry it; and the banks cannot resist this high turbulence; so are eroded until the turbulence near the banks is reduced to a non-eroding value.

Let us consider this further:—

For any discharge below the dominant discharge, the slope will be in defect for the dominant charge, so material will deposit, and there will then be a subdominant charge of material in movement; but when the discharge exceeds the dominant discharge, the energy available exceeds that required to carry the dominant charge, so excess energy in the upper reaches—but not always in the lower reaches—is damped out by scouring the bed and eroding the banks, thus increasing the charge.

If the amount of excess energy is small—as in a canal—the whole of it may be consumed in

- (a) scouring the bed—and hence washing out the finer particles, leaving the coarser grades exposed,
- (b) in overcoming the increased roughness,
- and
- (c) in carrying a large charge of coarser material.

^{*} “A Laboratory Study of the Meandering of Alluvial Rivers” by Captain J. F. Friedkin, U. S. Waterways Experiment Station, Vicksburg, Mississippi.

FACTORS CONTROLLING FLOW

Where, however, the fluctuations and hence excess energy are great, it cannot all be damped out in that way for all varying conditions; so meanders and overflow of banks develop—see Chapter 4.

2.13. MODEL EXPERIMENTS IN CONNECTION WITH BED MOVEMENT

Photo 3096 shows ripple beds developed with relatively low charges of bed sand, and Photo 2984 shows dunes which developed when the charge was great. Photo Sheet 2-1.

Experiments carried out in Poona in 1938 in channels laid in coarse sand = 1.4 mm., gave areas and velocities differing little from Lacey's regime values (for discharges down to 4 cusecs) the curve being approximately asymptotic to the Lacey values; but widths were greater, depths less and slopes much less than according to Lacey regime values.

$f_{v_0} = 2 V^3/Q^{1.5}$ gave the most consistent f value.

Ripples persist so long as the bed consists entirely of incoherent sand (greater than .08 mm. and less than 5 mm.) even though all bed movement has ceased; and when no sand is injected the final channel shape depends to a considerable extent on the initial dimensions of the channel, as laid, and the rate at which the discharge was increased; so there was no uniquely determined channel slope or shape—such as occurs when silt and sand are injected.

Photo 2977 shows one of a later series of channel experiments in which sand was injected; and Photo 2670 shows "blanketing" occurring in the elutriator channel at the Indian Waterways Experiment Station, Poona. A heavy charge of material had been injected into the narrow channel of the elutriator, in which great turbulence was developed by baffles. To start with, the coarser grades of material were trapped under grids laid at bed level, fine sand being carried downstream to deposit on the bed where the velocity fell below that required to carry it in suspension; but owing to a misunderstanding, material had been injected until the bed was completely blanketed with sand. Some fine sand also deposited downstream of the blanket on top of the underlying silt, which gave rise to incipient ripples. Photo Sheet 2-2.
Photo Sheet 2-3.

CHAPTER 3

HISTORICAL NOTE ON REGIME FLOW IN CHANNELS
IN ALLUVIUM AND DIVERGENCE FROM
REGIME

	Page
3.00 Kennedy—1895	95
3.01 Lindley's formulæ of 1919—linking bed-width and depth with velocities	95
3.02 Lacey's Papers of 1929 and 1933 on flow in alluvial channels (Inst. of C. E.)	96
3.03 Dr. N. K. Bose's formulæ of 1936 based on statistical analysis	98
3.04 Lacey's early formulæ modified in accordance with latest coefficients	99
3.05 Inglis' discussion on Dr. Bose's Paper—introducing normality = $f_m = \sqrt{f_{v_n} f_{s_n}}$ and divergence = $(f_{v_n}/f_{s_n})^{\frac{1}{2}}$	99
3.06 E. W. Lane's Paper of 1935 on "Stable Channels in Erodible Material"	100
3.07 Lacey's discussion on Lane's Paper	100
3.08 Lacey's reply to the discussion	101
3.09 Lacey's C. B. I. Paper on "Regime Flow in Incoherent Alluvium"—1939	102
3.10 Lacey's "shock theory"	102
3.11 Dr. C. M. White's Paper of 1939 on "The Influence of Transported Solids upon Rivers"	102
3.12 Lacey's discussion on Dr. White's Paper	107
3.13 Significance of (X. Vs)—weighted mean charge	107
3.14 Inglis' Paper of 1941 on "Divergence from Regime in Stable Channels in Alluvium"	107
3.15 (i) m: Q relationship in a canal system	111
3.16 (ii) f_{v_n} : Q; f_{s_n} : Q; f_m : Q relationship in a canal system	119
3.17 (iii) S: Q relationship in a canal system	120
3.18 (iv) V: (R ² S) relationship in a canal system	120
3.19 (v) $V = 1.346 R^{\frac{2}{3}} S^{\frac{1}{2}}/N_a$ —the Lacey rugosity formula	125
3.20 (vi) P/R: Q relationship in a canal system	125
3.21 (vii) Conclusions drawn in Paper of 1941 on "Divergence from regime"	126
3.22 Blench's Institution Paper on a Universal flow formula	130
3.23 Lacey's 1946 Paper entitled "A general theory of flow in alluvium"	132
3.24 Equations introducing charge	136

CHAPTER 3

HISTORICAL NOTE ON REGIME FLOW IN CHANNELS
IN ALLUVIUM AND DIVERGENCE FROM
REGIME

	Page
3.00 Kennedy—1895	95
3.01 Lindley's formulæ of 1919—linking bed-width and depth with velocities	95
3.02 Lacey's Papers of 1929 and 1933 on flow in alluvial channels (Inst. of C. E.)	96
3.03 Dr. N. K. Bose's formulæ of 1936 based on statistical analysis	98
3.04 Lacey's early formulæ modified in accordance with latest coefficients	99
3.05 Inglis' discussion on Dr. Bose's Paper—introducing normality = $f_m = \sqrt{f_{va} f_{as}}$ and divergence = $(f_{va}/f_{as})^{\frac{1}{2}}$	99
3.06 E. W. Lane's Paper of 1935 on "Stable Channels in Erodible Material"	100
3.07 Lacey's discussion on Lane's Paper	100
3.08 Lacey's reply to the discussion	101
3.09 Lacey's C. B. I. Paper on "Regime Flow in Incoherent Alluvium"—1939	102
3.10 Lacey's "shock theory"	102
3.11 Dr. C. M. White's Paper of 1939 on "The Influence of Transported Solids upon Rivers"	102
3.12 Lacey's discussion on Dr. White's Paper	107
3.13 Significance of (X. Vs)—weighted mean charge	107
3.14 Inglis' Paper of 1941 on "Divergence from Regime in Stable Channels in Alluvium"	107
3.15 (i) m: Q relationship in a canal system	111
3.16 (ii) f_{va} : Q; f_{as} : Q; f_m : Q relationship in a canal system	119
3.17 (iii) S: Q relationship in a canal system	120
3.18 (iv) V: (R S) relationship in a canal system	120
3.19 (v) $V = 1.346 R^{\frac{2}{3}} S^{\frac{1}{2}}/N_a$ —the Lacey rugosity formula	125
3.20 (vi) P/R: Q relationship in a canal system	125
3.21 (vii) Conclusions drawn in Paper of 1941 on "Divergence from regime"	126
3.22 Blench's Institution Paper on a Universal flow formula	130
3.23 Lacey's 1946 Paper entitled "A general theory of flow in alluvium"	132
3.24 Equations introducing charge	136

CHAPTER 3

HISTORICAL NOTE ON REGIME FLOW IN CHANNELS
IN ALLUVIUM AND DIVERGENCE FROM REGIME

3.00 KENNEDY—1895*

In 1895, Kennedy produced his classic empirical equation:—

$$V_0 = cD^m = 0.84D^{0.64}$$

based on observations on the Bari Doab Canal in the Punjab, V_0 representing mean velocity, and D = depth of water over the bed portion of channel. Kennedy concluded that channels of various sizes with velocities based on this formula did not either silt or scour their beds. He recognised that the grade of sand played a part in this relationship and regarded the sandy silt of the Bari Doab Canal as standard. His formula can be re-written

$$V = 0.84 CD^{0.64} \quad \dots \quad \dots \quad \dots \quad (1)$$

C being equivalent to V/V_0

This depth formula held for channels of different sizes in this one canal system—each channel having developed a regime section with a particular bed-width to depth relation—but Kennedy did not derive any width formula; so, according to the formula, it was permissible to design a narrow deep channel or a wide shallow channel to carry the same discharge.

Actually, a certain amount of latitude in shape is permissible if the charge is small—especially if the banks are scour-resistant;† but where the charge and discharge are high, the range of natural shape is small; so, in general, Kennedy's formula only gives correct results if the correct shape is chosen. Later, when data were collected for other canals in India, it was found that the exponent, as well as the coefficient, differed under different conditions, the exponent varying from .64 to .52 and the coefficient from .67 to .95.

3.01 LINDLEY'S FORMULAE OF 1919—LINKING BED WIDTH AND DEPTH WITH VELOCITIES

Lindley in 1919‡ stated in his Paper on "Regime Channels"

"When an artificial channel is used to convey silty water, both bed and banks scour or fill, changing depth, gradient and width, until a state of balance is attained at which the channel is said to be in regime. These regime dimensions depend on discharge, quantity and nature of bed and berm-silt, and rugosity of the silted section; rugosity is also affected by velocity, which determined the size of wavelets into which the silted bed is thrown."

At that time he introduced bed-widths as well as depths as regime variables and by correlating data obtained from 786 observations in Branch surveys of the L. C. C. derived the following regime formulae for the Lower Chenab Canal:—

$$V = 0.95 D^{.57} \quad \dots \quad \dots \quad \dots \quad (2)$$

$$V = 0.57 B^{.355} \quad \dots \quad \dots \quad \dots \quad (3)$$

$$B = 3.80 D^{1.61} \quad \dots \quad \dots \quad \dots \quad (4)$$

*Procs. Inst. C. E. Vol. CXIX, 1895.

†Discussion by C. C. Inglis on Paper, Punjab Engineering Congress, by Lala Ishardas.

‡"Regime Channels" by E. S. Lindley. Procs. Punjab Engineering Congress, Vol. 7, 1919.

THE BEHAVIOUR AND CONTROL OF RIVERS AND CANALS

In concluding his reply to the Discussion on his Paper, he stated:—

“The existence of these relations meant that the dimensions, width, depth and gradient, of a channel to carry a given supply, loaded with a given silt-charge were all fixed by nature”—i.e. uniquely determined.

The data on which these formulae were based, included bed-widths, depths, and slopes; but velocities were not observed; so Kutter's equation was used to work out velocities, N being assumed constant at 0.225. In this connection, Lacey wrote*

“Despite this defect, Lindley's empirical equations are consistent with the theory put forward by him—that the dimensions—width, depth and gradient—of a channel to carry a given supply, loaded with a given silt charge, are all fixed by nature.”

No attempt was made by Lindley to correlate rugosity and silt grade, nor did he correlate the width and depth with the discharge. Had he done so he would have found that the following exponential formulae held for discharges of 4267 cusecs and under:

$$w = 2.29 Q^{0.50} \dots \dots \dots (5)$$

$$d = 0.57 Q^{0.33} \dots \dots \dots (5a)$$

$$V = 1.04 d^{0.51} \dots \dots \dots (5b)$$

$$P = 2.95 Q^{0.47} \dots \dots \dots (5c)$$

$$R = 0.44 Q^{0.33} \dots \dots \dots (5d)$$

where

w = surface width

d = A/w = mean depth

P = wetted perimeter

R = hydraulic mean depth

3.02. LACEY'S PAPERS OF 1929 AND 1933 ON FLOW IN ALLUVIAL CHANNELS (INST. OF C. E.)

Lacey accepted the Lindley theorem; but adopted-P and R as his variables.

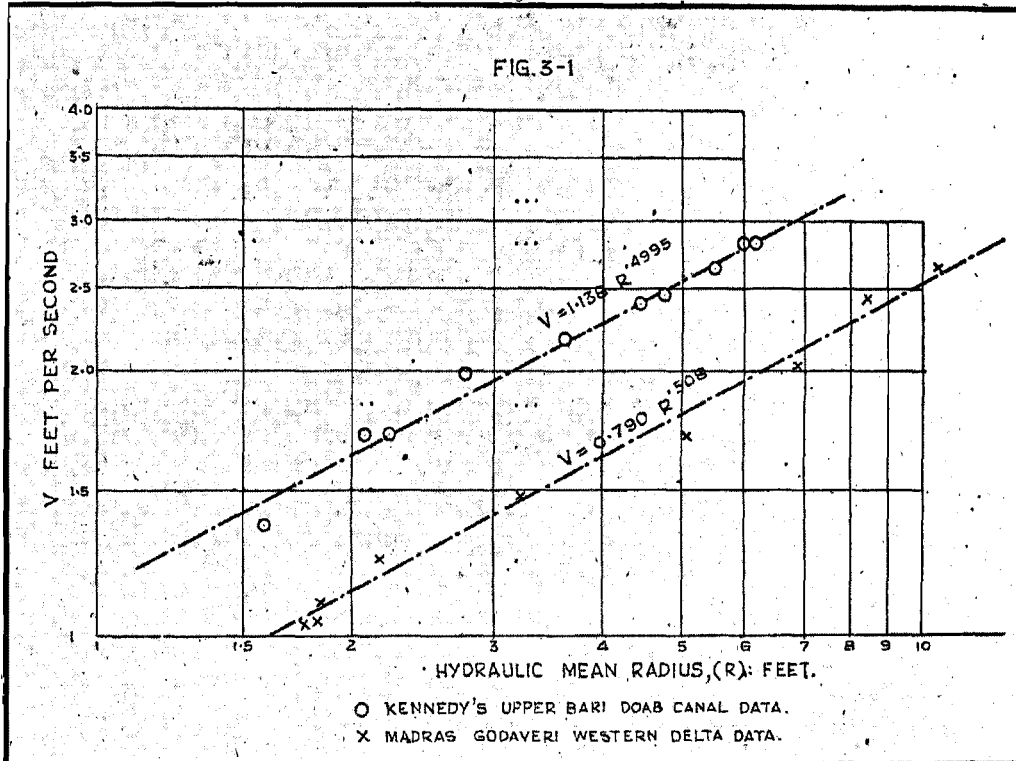
He propounded his early ideas and formulae in two Papers written in 1929 and 1933 for the Institution of Civil Engineers, entitled “Stable Channels in Alluvium”† and “Uniform Flow in Alluvial channels”‡

*Central Board of Irrigation Publication No. 26 “Regime Flow in Incoherent Alluvium” (1939).

†Paper No. 4736 Inst. C. E. Vol. 229 (1929-30) “Stable Channels in Alluvium”.

‡Paper No. 4893 Inst. C. E. Vol. 239 (1933-34).

DIVERGENCE FROM REGIME



RELATION BETWEEN VELOCITY AND HYDRAULIC MEAN RADIUS

(From Lacey's "Regime Flow in Incoherent Alluvium")

THE BEHAVIOUR AND CONTROL OF RIVERS AND CANALS

Lacey's early formulae were :—

$$V = 1.17 \sqrt{fR} \quad \dots \quad \dots \quad \dots \quad (6.0)$$

$$Af^3 = 3.8 V^6 \quad \dots \quad \dots \quad \dots \quad (7.0)$$

$$S = 0.00387 f^{3/2} / Q^{1/2} \quad \dots \quad \dots \quad \dots \quad (8.0)$$

$$P = 2.668 Q^{1/2} \quad \dots \quad \dots \quad \dots \quad (9.0)$$

(Fig. 3-1)

These, he later modified to :—

$$V = 1.155 \sqrt{fR} \quad \dots \quad \dots \quad \dots \quad (6.1)$$

$$Af^3 = 4.0 V^6 \quad \dots \quad \dots \quad \dots \quad (7.1)$$

$$S = 0.00383 f^{3/2} / R^{1/2} \quad \dots \quad \dots \quad \dots \quad (8.1)$$

Restated, in terms of discharge:—

$$P = 2.668 Q^{1/2} \quad \dots \quad \dots \quad \dots \quad (9.0)$$

$$A = 1.26 Q^{5/2} / f^{3/2} \quad \dots \quad \dots \quad \dots \quad (10.2)$$

$$R = 4.725 Q^{1/2} / f^{3/2} \quad \dots \quad \dots \quad \dots \quad (11.2)$$

$$V = 0.794 Q^{1/6} f^{1/2} \quad \dots \quad \dots \quad \dots \quad (12.2)$$

Lacey in his reply to the Discussion on his Paper added four other formulae,

$$S = 0.00547 f^{5/2} / Q^{1/2} \quad \dots \quad \dots \quad \dots \quad (13.0)$$

$$N_s = 0.0225 f^4 \quad \dots \quad \dots \quad \dots \quad (14)$$

$$V = \frac{1.3458 R^{3/2} S^{1/2}}{N_s} \quad \dots \quad \dots \quad \dots \quad (15)$$

$$V = 16.116 R^{3/8} S^{1/8} \quad \dots \quad \dots \quad \dots \quad (16.1)$$

3.03. DR. N. K. BOSE'S FORMULAE OF 1936 BASED ON STATISTICAL ANALYSIS

In 1936* Dr. Bose and the staff of the Punjab Irrigation Research Institute, after several years of painstaking collection, and statistical analysis, of data, derived the formulae,

$$P = 2.8 Q^{1/2} \quad \dots \quad \dots \quad \dots \quad (9B)$$

$$\text{and } S \times 10^3 = 2.09 m^{*6} / Q^{21} \quad \dots \quad \dots \quad \dots \quad (8B)$$

the former is comparable with formula (9) of Lacey, the coefficient being 5 per cent higher; while the latter is closely related to Lacey's slope formula † (13.0).

‡ These were followed by †

$$R = .47 Q^{1/2} \quad \dots \quad \dots \quad \dots \quad (11B)$$

which is almost identical with formula (11.2)—based on Lacey's formulae (9) and (6.1)—but omits grade of bed material.

* Dr. N. K. Bose, Procs. Punjab Eng. Congress, Paper No. 192 (1936) "Silt Movement and Design of Channels" Punjab Irrigation Research Inst. Report for Season ending April 1936, p. 28, P.I.R.I. Report for year ending April 1937, p. 61.

† C. I. and H. E. S. A. R. (T) 1941-42 para. (28) p. 9.

‡ P.I.R.I. A. R. 1937, p. 80 and 1938, p. 57.

DIVERGENCE FROM REGIME

3.04. LACEY'S EARLY FORMULAE MODIFIED IN ACCORDANCE WITH LATEST COEFFICIENTS

Lacey's formulae, as they now stand, are given here to avoid the necessity of repeated small corrections and restatement.

$$V = 1.15473 \sqrt{fR} \quad \dots \quad \dots \quad \dots \quad (6)$$

$$Af^2 = 4.00 V^3 \quad \dots \quad \dots \quad \dots \quad (7)$$

$$S = 0.002759 f^{3/2} / R^{1/2} \quad \dots \quad \dots \quad \dots \quad (8)$$

$$P = 8/3 Q^{1/2} \quad \dots \quad \dots \quad \dots \quad (9)$$

$$A = 1.260 Q^{5/3} / f^{1/3} \quad \dots \quad \dots \quad \dots \quad (10)$$

$$R = 47247 Q^{1/3} / f^{1/3} \quad \dots \quad \dots \quad \dots \quad (11)$$

$$V = 7937 Q^{1/3} / f^{1/3} \quad \dots \quad \dots \quad \dots \quad (12)$$

$$S = 0.005469 f^{3/5} / Q^{1/5} \quad \dots \quad \dots \quad \dots \quad (13)$$

3.05. INGLIS' DISCUSSION ON DR. BOSE'S PAPER—INTRODUCING

$$\text{NORMALITY} = f_m = \sqrt{f_{VR} \cdot f_{RS}} \quad \text{AND DIVERGENCE} = (f_{VR}/f_{RS})^{1/2}$$

Inglis, in his discussion on Dr. Bose's Paper of 1936, pointed out that f in formulae (6) and (8) were different, the former being the f of equation (2) of chapter 2, now called "the Lacey number", while the latter was the f value of formula (6) of chapter 2, connecting slope with hydraulic mean depth. Calling these f_{VR} and f_{RS} respectively, formula (6) may be rewritten:—

$$f_{VR} = 75 \frac{V^3}{R} \quad \dots \quad \dots \quad \dots \quad (6A)$$

and formula (13) may be rewritten,

$$f_{RS} = 192.00 R^{1/3} S^{3/5} \quad \dots \quad \dots \quad \dots \quad (8A)$$

In the same discussion he pointed out that Lacey's "final regime formula."

$$V = 16.00 R^{2/3} S^{1/3} \quad \dots \quad \dots \quad \dots \quad (16)$$

might be rewritten,

$$V = 16.00 R^{2/3} S^{1/3} (f_{VR}/f_{RS})^{1/2} \quad \dots \quad \dots \quad \dots \quad (17)$$

(Note.—The coefficients of equations (14)–(17) are the latest values and hence slightly different from the original figures.)

$$\text{and } C_L = (f_{VR}/f_{RS})^{1/2} \quad \dots \quad \dots \quad \dots \quad (18)$$

which gives the degree of divergence from Lacey regime,

$$\text{while } f_m = \sqrt{f_{VR} \cdot f_{RS}} \quad \dots \quad \dots \quad \dots \quad (19)$$

is a measure of the silt value of a canal system."

"Working on these lines, f_m and C_L represent 'normality' and divergence."

He also pointed out that,

"the data indicated that the weighted mean diameter of material exposed on the bed varies as $Q^{1/5}$ for both the Lower Jhelum and Lower Chenab Canals."

3.06. E. W. LANE'S PAPER OF 1935 ON "STABLE CHANNELS
IN ERODIBLE MATERIAL"

In this Paper*, E. W. Lane made a comprehensive study of stable channel shapes. The Paper contains an excellent summary, and elicited a very interesting discussion, which merits close study. The main point which he stressed was that,

"the quantity of solids in motion is an important factor in the shape of stable channels in alluvium and has not received the attention that its importance warrants."

"...it is a common occurrence in some irrigation systems to have the upper section of the ditch fill during periods of high silt content in the streams from which they draw, and for this fill to scour out later during periods of clear-water flow. In other words, the ditch is unstable, at times being unable to transport all the material brought to it, hence filling up, and at other times transporting more material than brought to it, and, therefore, cutting down its bed. Over long periods the ditches are approximately stable because the two actions counteract one another. Another common example is the change from unstable to stable conditions which result when an effective sand trap is applied to a ditch that is becoming filled. There are also numerous cases in which the channel of a natural stream is stable but begins to scour severely when a dam is built on it and cuts off the supply of solid material which formerly came down to renew that which was moved forward by the flowing water."

3.07. LACEY'S DISCUSSION ON LANE'S PAPER

In his discussion, Lacey observed,

"In the writer's view 'turbulence' is susceptible of definition, and the ratio V^2/R for any grade of silt and irrespective of the charge, epitomizes 'turbulence'. It is thus found that fine silt with a heavy surcharge may well demand the same value of V^2/R as a coarse silt with a smaller charge in another locality."

Though not specifically stated, this gives the impression that if the Lacey value of f_{VR} were increased as the charge increased, all the formulae would still hold. Actually, however, they would not — for, though Lacey's formulae show an increase in slope and velocity, the formula $P=8/3 Q^{\frac{1}{2}}$, which Lacey holds is not affected by grade, is undoubtedly considerably affected by charge.

Lacey also observed in this discussion :—

"The writer has shown how dimensional arguments can be fitted to empirical relationships, thus substituting for the energy gradient in terms of the geometrical slope, density, and the acceleration due to gravity,

$$\frac{gRS}{V^2} = K'' \left(\frac{RV}{\nu} \right)^{-\frac{1}{2}} \left(\frac{V^2}{gR} \right)^{\frac{2}{3}} \quad \dots \quad (20)$$

If the silt factor is treated as $\propto V^2/R$, the above equation may be solved and will prove the basis of all the writer's formulae other than those involving the discharge, thus leaving the silt factor in the ratio form,

$$V \propto g^{\frac{1}{3}} \nu^{-\frac{1}{3}} R^{\frac{2}{3}} S^{\frac{1}{3}} \quad \dots \quad (21)$$

and this relationship is true for any regime channel in incoherent alluvium whether fine sand, coarse sand, shingle or boulders.

* Paper No. 1957 Proc. Am. Soc. C. E. November 1935.

DIVERGENCE FROM REGIME

The very low power of the kinematic viscosity should be noted and the complete disappearance of the "silt factor", which is inherent in the dimensions assumed by the channel.

The formula may be recast as,

$$V \propto g^{\frac{1}{2}} \nu^{-\frac{1}{2}} f^{-\frac{1}{2}} R^{\frac{1}{2}} S^{\frac{1}{2}} \dots \dots \dots (22)$$

The introduction of P gives the solution,

$$\frac{P}{R} \propto \left(\frac{RV}{\nu}\right)^{\frac{1}{2}} \left(\frac{V^2}{gR}\right)^{\frac{1}{2}} \dots \dots \dots (23)$$

from which $P \propto Q^{\frac{1}{2}}$ and $P/R \propto V$.

All the most recent work in India tends to confirm the expression $P \propto Q^{\frac{1}{2}}$ as fundamental."

Formula (23) gives,

$$P/R \propto V/(g\nu)^{\frac{1}{2}} \dots \dots \dots (24)$$

and,

$$P \propto Q^{\frac{1}{2}}/(g\nu)^{\frac{1}{2}} \dots \dots \dots (25)$$

3.08. LANE'S REPLY TO THE DISCUSSION

Lane, in reply to Lacey, observed :—

"The empirical relations derived by Mr. Lacey seem to have been widely accepted in India. The writer believed that they are probably the best quantitative relations yet developed. The striking agreement with the results obtained by Colonel Pettis on an entirely independent basis indicates strongly the general accuracy of the conclusions of both men for average conditions. It is believed, however, that at least one important factor has been omitted, namely, the concentration of the silt load, and when the load is high, this becomes such an important factor that it must be considered."

"The importance of silt quantity as a factor in channel shape and the tendency of heavily silt-laden water to form wide shallow channels seems to have been generally accepted."

"The writer does not believe that the factor V^2/R , empirically derived by Mr. Lacey and also independently by Mr. Tchikoff, has been demonstrated to be a rational measure of turbulence, although it may be a very convenient empirical index of it. Some of the arguments advanced in support of it do not appear to be valid. For example, the fact that the ratio, V^2/R is a valuable criterion in non-uniform flow seems to be pure coincidence and to bear no relation to the phenomena of stable channel shapes. Assuming that all the energy loss due to the friction of the water flowing in a ditch is dissipated by turbulence, the average turbulence of a stream (under conditions of steady, uniform flow), expressed in terms of the energy loss per unit time per unit of volume (occupied by a unit weight), is proportional to the product of the slope and the mean velocity of flow. This follows from the fact that if the water is not accelerating, the energy dissipated per unit of volume per unit of time must be equal to the work represented by the drop of the water in that time, which, in turn, is equal to the mean velocity of the water times the slope. Hence, the criterion of turbulence should be $S \cdot V$."

THE BEHAVIOUR AND CONTROL OF RIVERS AND CANALS

3.09. LACEY'S C. B. I. PAPER ON "REGIME FLOW IN INCOHERENT ALLUVIUM"

At the end of 1938, Lacey was put on special duty "to write up his work in one comprehensive volume." This was submitted on 9th January 1939 and published under the title "Regime Flow in Incoherent Alluvium."*

Fig. 3-2 In that Paper he showed in his Fig. 3-2—reproduced here—that the Lindley data gave the relation

$$R^{\frac{1}{2}} S = .000350 \dots \dots \dots (26)$$

Fig. 3-3 and in his Figs. 3-3 and 3-4 he plotted the relation between V and R³ S for all available data for "canals in approximate regime and rivers and torrents in coarse incoherent material in the active full stage"—to which he applied the term "quasi-regime"; and the resultant formula was

$$V = 16.0 R^{\frac{2}{3}} S^{\frac{1}{3}} \dots \dots \dots (16)$$

$$\propto g^{\frac{5}{3}} / \nu^{\frac{1}{3}} R^{\frac{2}{3}} S^{\frac{1}{3}}$$

3.10. LACEY'S "SHOCK THEORY"

In this same Paper, Lacey introduced his "shock theory"; concerning which he stated:—

"Slope may include shock due to bends or irregularities in the channel, and also to "channel condition" as opposed to "channel material"; but later he used the term "normal shock" for "normal channel condition". He also stated:—

"Provided that the grade of silt transported is perfectly incoherent or of constant coherence the slope should be a function of the bed silt grade.

Shock in such channels is indicated by a departure from the regime slope, and a corresponding departure from normal bed silt grade.

When there is no shock the equation must take the form

$$f_R = km_R^{\frac{1}{2}} \dots \dots \dots (27)$$

This equation is fundamental and the basis of the writer's shock theory. If the writer's equation correlating the silt factor and the silt grade has assigned dimensions, it must take the form

$$f \propto m^{\frac{1}{2}} g^{\frac{3}{2}} (\nu g)^{\frac{1}{2}} \dots \dots \dots (28)''$$

He also derived the formula

$$"f_R = 1.76 m_R^{\frac{1}{2}} \dots \dots \dots (29)''$$

3.11. DR. C. M. WHITE'S PAPER OF 1939 ON "THE INFLUENCE OF TRANSPORTED SOLIDS UPON RIVERS"

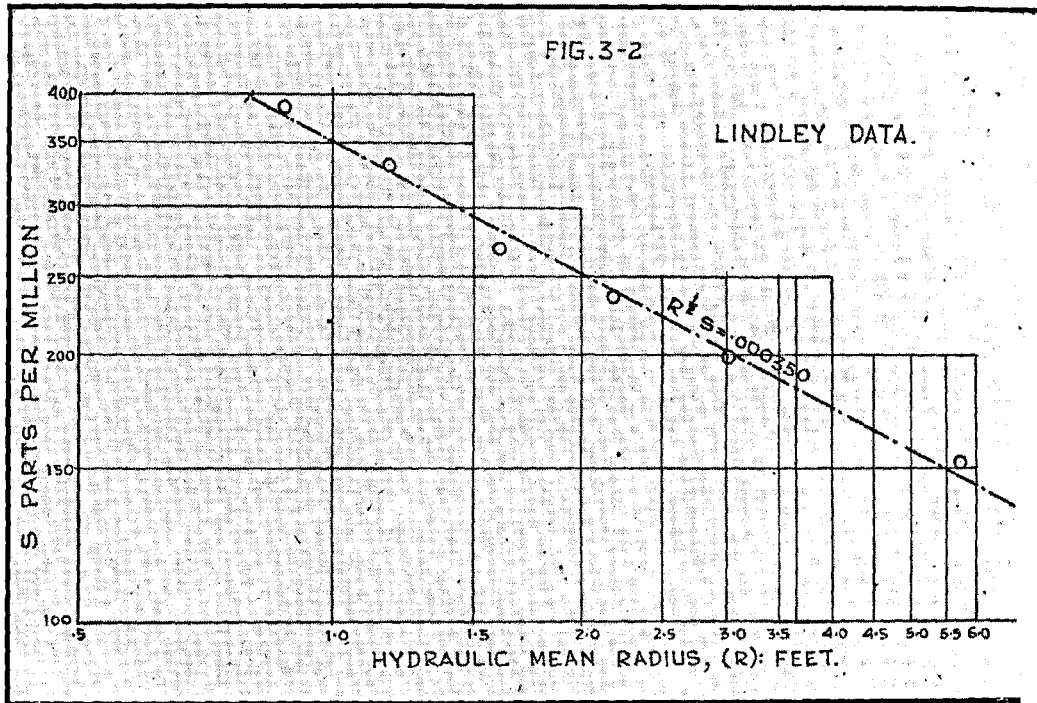
† In 1939, Dr. C. M. White of the City and Guilds, London, wrote a Paper entitled "The Influence of Transported Solids upon Rivers", † in which he derived a formula

$$\frac{V}{(gR)^{\frac{1}{2}}} = \frac{0.7 V_s^{\frac{1}{2}}}{g^{\frac{1}{2}} Q^{\frac{1}{2}}} \text{ or } \frac{V}{R^{\frac{1}{2}}} = f^{\frac{1}{2}} = \frac{0.7 g^{\frac{3}{2}} V_s^{\frac{1}{2}}}{Q^{\frac{1}{2}}} \dots (30)$$

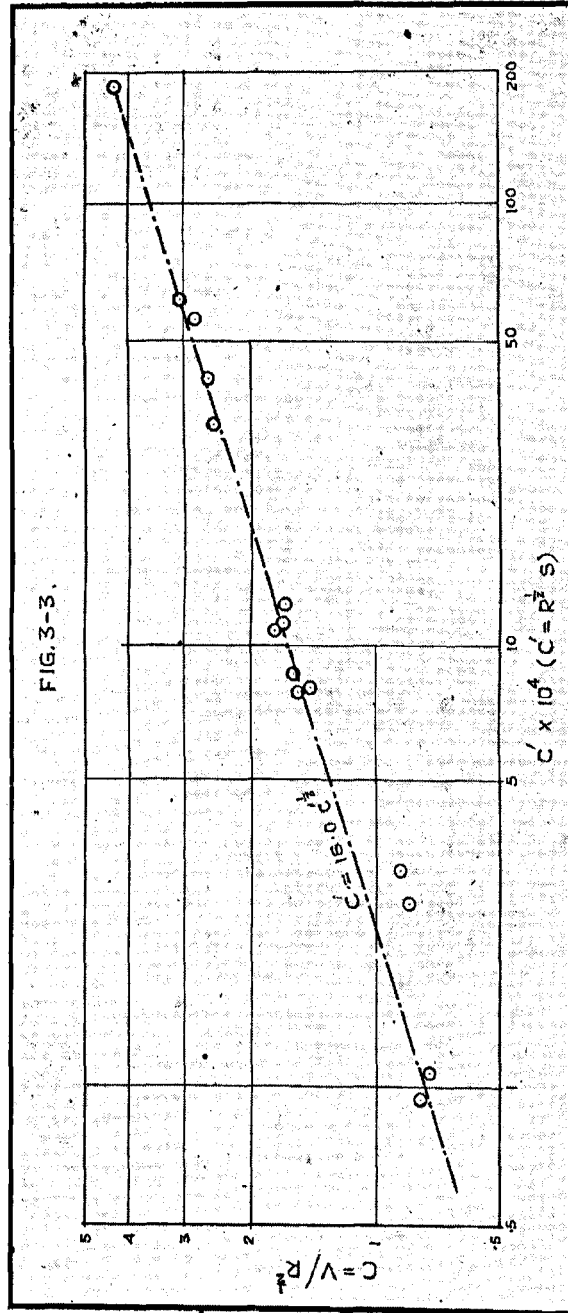
Vs being "terminal mean velocity of particles of sand falling in water."

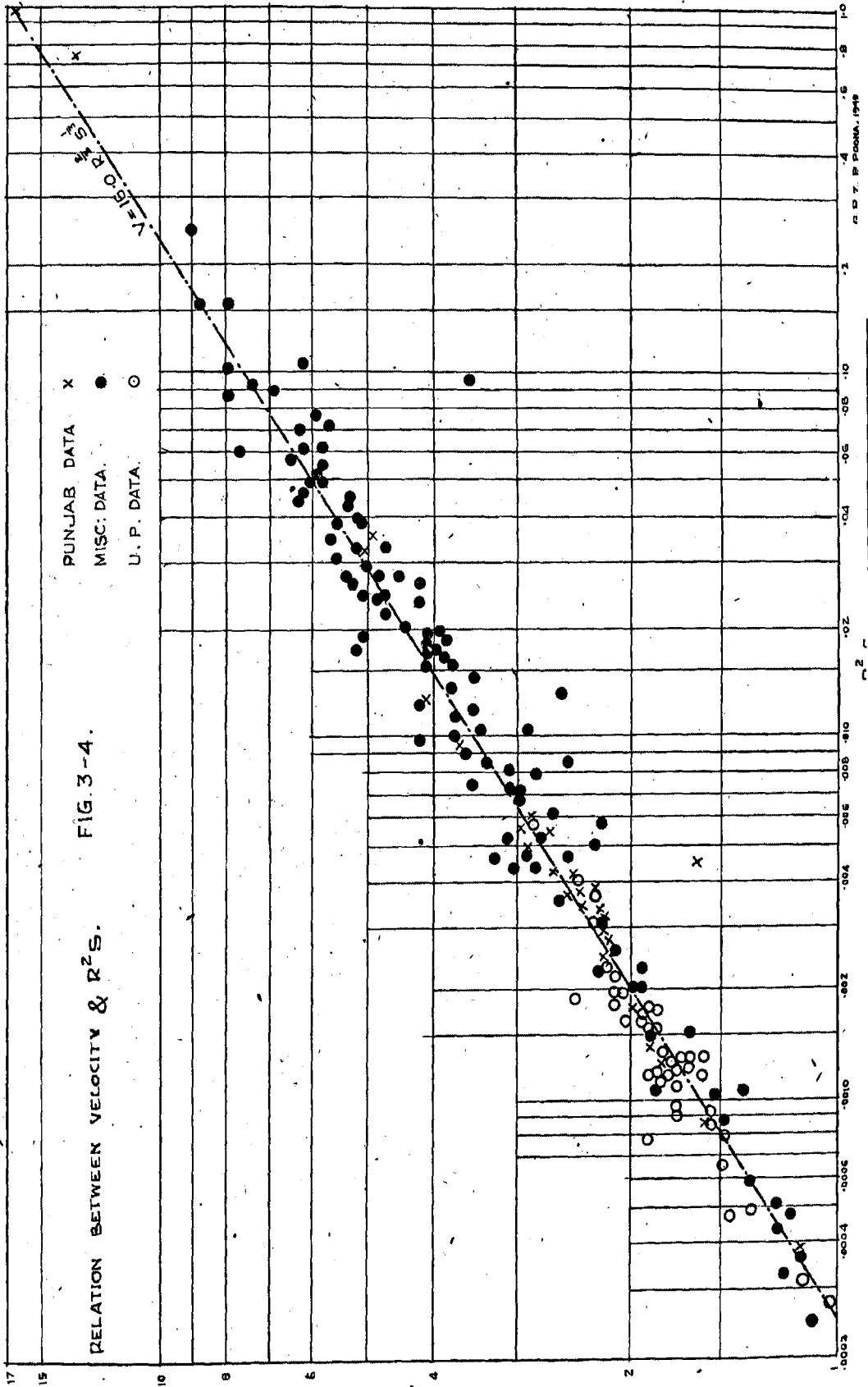
* C. B. I. Publication No. 20 (18th July 1939) by Gerald Lacey.
 † See the Section of Potamology, International Union of Geology and Geophysics, Washington D. C. September 1939.

DIVERGENCE FROM REGIME



RELATION BETWEEN SLOPE AND HYDRAULIC MEAN RADIUS—
LINDLEY'S DATA FOR LOWER CHENAB CANAL





M. D. T. P. POONA. 1949

DIVERGENCE FROM REGIME

This formula was based on observations carried out on the Mississippi, Nile, Elbe, Inn, Isar, Miami, Tirolar Asche and a 0.5 cusec model river; but neither the data nor authorities were quoted by Dr. White. It is obvious, however, that the channels were subject to widely-fluctuating discharges and charges, so the data were not based on regime.

3.12. LACEY'S DISCUSSION ON DR. WHITE'S PAPER

Lacey, in discussing Dr. White's Paper, drew the conclusion that

"a constant value of the product (N. Vs.) for a given degree of agitation could be assigned a physical meaning—N being the silt charge and Vs the terminal velocity. If, for example, two samples, of which the product (N. Vs) was equal for each, were introduced into two vessels identical in all respects, and with equal volumes of water, then, at a given interval of time—after equal agitation—the total amount of silt deposited in each should be the same."

3.13. SIGNIFICANCE OF (X. Vs)—WEIGHTED MEAN CHARGE

This conclusion—that deposition was constant for a constant (N. Vs), now called (X. Vs) to avoid confusion, as N is the usual notation for rugosity—was later confirmed at Poona*—see notes prepared for the Research Officers' Meeting held at Simla in 1940 and 1941 entitled "Rate of Deposition of sand as governed by charge and terminal velocity."

The final formula derived† for material varying from 0.191 mm. to 0.40 mm., based on data shown in Fig. 3-5, was

$$\begin{aligned} \text{Depth of deposition in centimetres per second} \\ = 0.000695 (X. Vs) \quad \dots \quad (31) \end{aligned}$$

†
Fig. 3-

3.14. INGLIS' PAPER OF 1941 ON "DIVERGENCE FROM REGIME IN STABLE CHANNELS IN ALLUVIUM"‡

In his synopsis, the Author wrote:—

"The Director, who has had to travel in all parts of India, of recent years, in connection with river training and control, has found that the subject of widest interest to irrigation engineers, about which there is the least understanding and the greatest divergence of opinion is, undoubtedly, the "Lacey theory and flow formulæ."

(2) Much of this misunderstanding has been due to lack of definitions and misleading terminology; much to the fact that large numbers of stable channels give values which do not approximate to Lacey values based on mean diameter of bed sand; and much to the 'silt factor' values calculated by the different Lacey formulæ dealing with shape and slope, differing from each other and from f_a the value calculated from mean diameter of bed sand.

(3) Lacey introduced his theory of 'shock and coherence in regime flow' to explain these divergences; but here again the terminology is confusing and the subject has become so theoretical that Dr. Malhotra, in a very able survey of the theory, has had to write 30 pages to explain its development in precise language; so that practical engineers have become perplexed and "suspicious".

*C. I. and H. R. S. A. R. (T) 1940-41, page 50.

†C. I. and H. R. S. A. R. (T) 1941-42, pages 33-34 "Effect of residual turbulence on rate of deposition of sand."

‡C. I. and H. R. S. A. R. (T) 1941-42, pages 4-11.

THE BEHAVIOUR AND CONTROL OF RIVERS AND CANALS

“In this Paper it is shown that the main reason for the dimensions of stable channels diverging from those derived from Lacey regime formulæ is his omission of any ‘sand charge’ factor from his formulæ. The causes leading to variation in charge in different channels of the same canal system, and the effects of these variations are also explained; and it is shown that the main divergences can be eliminated by using a single-value factor (f_m) for a whole system, which is the geometric mean of Lacey’s shape and slope factors.”

“Lacey’s shock theory satisfied neither research workers nor practical engineers. Some considered that the formulæ were wrong because sections were assumed to be semi-elliptical; whereas, in their experience, channels under normal conditions did not approximate to a semi-ellipse. This divergence they considered to be basic and not due to channels being excavated to a trapezoidal shape; because even in the case of channels which had developed their sections by accretion, they were not, generally, semi-elliptical. It was also held that the application of Lacey formulæ by assuming a trapezoidal section which gave the same P and R values as the Lacey formulæ, was unsound—especially in the case of small channels—and hence they were in difficulties as to how to apply his formulæ.

Lacey’s assumption that slope $(h_m/L_m) = (h_o/L_p) \times$ vertical exaggeration (in terms of $R_m/P_m: R_p/P_p$) where h_m is the ‘drop’ in length L_m in the model, and subscript ‘p’ applies to similar prototype values, is also not accepted at Poona; because its application would lead to distortion in plan and incorrect ‘meander length’ (see page 108 of Programme of Proceedings, Central Board of Irrigation, 11th Annual Meeting—1940). It is also incorrect because $h_m/d_m =$ head acting mean depth in the model varies widely from h_p/d_p in the prototype, and hence energy relations are wrong. Besides this, experiments carried out recently at Poona have proved that taking R as the effective depth parameter leads to incorrect results in the case of tidal models, as explained in a Note entitled “Wave and surge velocity in open channels” in the 1940-41 Annual Report—see also para. 1.18 and Photo Sheet 1-17—and Mr. Thomas has shown* the necessity for making ‘slope exaggeration’ equal to ‘vertical exaggeration’ in a tidal model to ensure that tides shall be correctly reproduced.

The most widespread doubt arose, however, due to the conviction that ‘shock and coherence due to channel condition’ went only a part of the way towards explaining the wide divergences from regime dimensions found in stable channels such as those of the Lower Chenab.

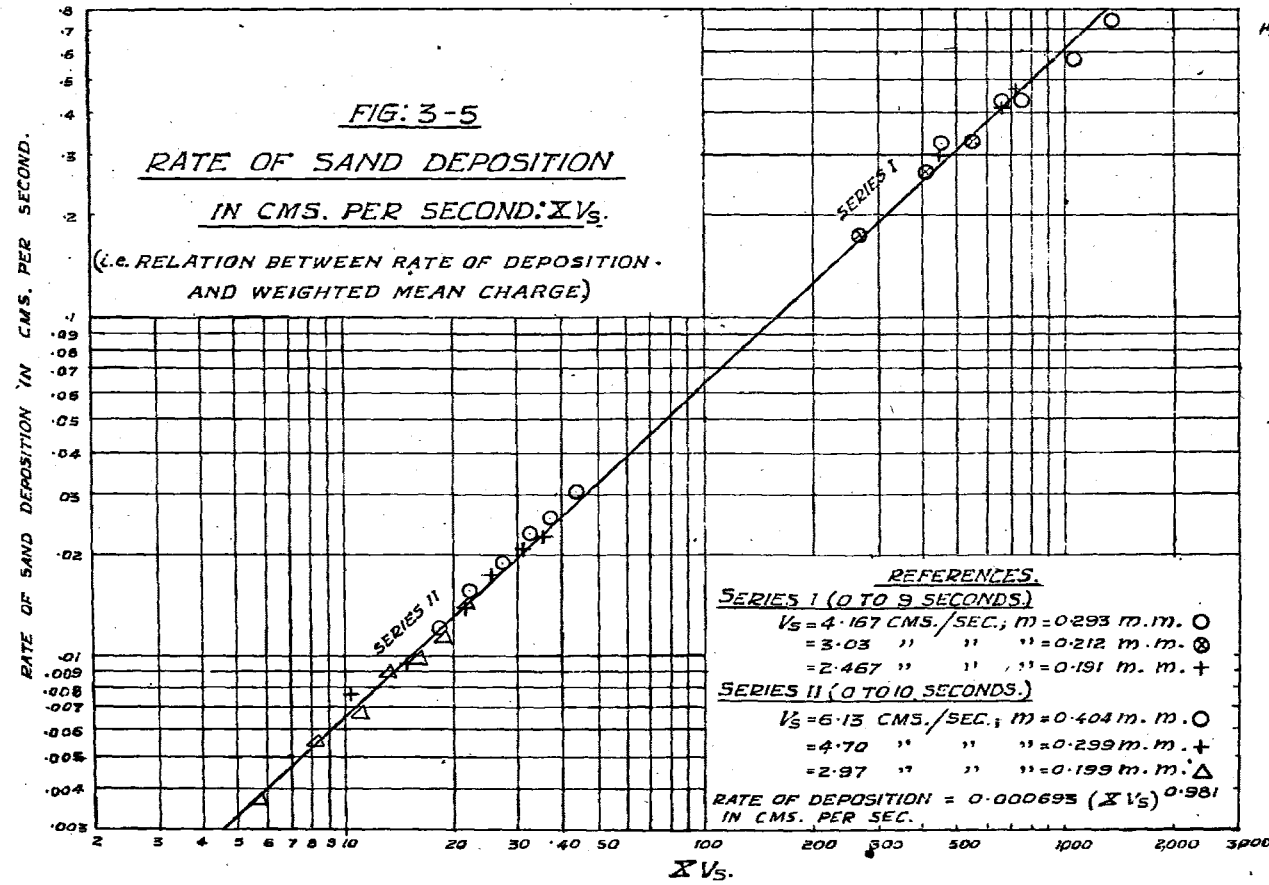
TABLE 3-1

Lower Chenab Canal	Mean value.	Standard deviation. plus/minus.
P_1/P varied from 0.82 to 1.45 ...	0.99	0.178
V_1/V ,, ,, 0.89 to 1.21 ...	1.04	0.095
S_1/S ,, ,, 0.69 to 1.31 ...	1.00	0.177

It was also felt that if ‘shock’ (meaning thereby not merely bank conditions, but also shock due to wrong shape resulting from wrong design) were the explanation of divergence; then it should be possible to vary channel design, and hence shock, to a similar extent, in order to suit local conditions.”

“Before dealing with the causes of divergence from regime in stable channels, it is necessary to state clearly what is meant by ‘regime’.

*C. I. and H. R. S. Basic Notes Nos. 65 and 77 of 30th October 1940 and 1st February 1941.



BY:- PARKHI, P.D. 2-6-42

DIVERGENCE FROM REGIME

The quintessence of the 'regime theory' is that under uniform flow conditions (which imply constant discharge, temperature and silt charge) there is a natural regime slope and section towards which a channel tends to develop and to which it will eventually attain, either by accretion, if the slope be too flat or the section too large, or by scouring, if the channel be too small or the slope too steep—assuming the channel to be flowing in material which can scour freely.

The theory also implies that for any particular discharge and mean diameter of bed sand, the 'charge' is normal; that is to say, the quantity of bed material and its size distribution is normal.

In practice, water temperatures vary throughout the year, the variation being much greater in some Provinces than in others and also greater in small than large channels. Similarly, the silt and sand charge entering the canals, and hence the 'bed-material size distribution', varies widely at different times of year; hence it must be clearly understood that the Lacey formulæ are based on statistical average values for the conditions of his data, which cause variations from regime at different times of year; but the net result of such variations is much less than might be expected."

The Author then analysed the degree of divergence exemplified in the various channel dimensions. These are dealt with in the following paragraphs:—

- 3.15 (i) $m : Q$ relationship in a canal system.
- 3.16 (ii) $f_{vs} : Q$; $f_{as} : Q$; $f_m : Q$ relationships in a canal system.
- 3.17 (iii) $S : Q$ relationship in a canal system.
- 3.18 (iv) $V : (R^2 S)$ relationship in a canal system.
- 3.19 (v) $V = 1.346 R^{\frac{2}{3}} S^{\frac{1}{2}} / N_a$ —the Lacey rugosity formula.
- 3.20 (vi) $P/R : Q$ relationship in a canal system.
- 3.21 (vii) Conclusions drawn in Paper on "Divergence from Regime."

3.15. (i) THE $m : Q$ RELATIONSHIP IN A CANAL SYSTEM

In 1934, the Author stated:—

"There is a balance between velocity, depth and sand across a section, with a gradient in size from coarsest at midstream to less coarse near the sides.

In present day terminology we would say that turbulence decreases from midstream to the sides with a resultant decrease in the coarseness of the material exposed on the bed.

Similarly, the smaller the channel in a canal system, the finer the bed-mix should be to maintain equilibrium; and the finer the bed material for the same discharge, the deeper and narrower the channel will be."

The relation between m and Q , worked out statistically, gave the following results:—

TABLE 3-II

	All Sites.		Excluding unsuitable Sites.	
	No. of Sites.	Statistical relation.	No. of Sites.	Statistical relation.
Jamrao Canal ...	24	$m = 0.06 Q^{0.115}$	12	$m = 0.70 Q^{0.088}$
Lower Chenab Canal.	16	$m = 0.165 Q^{0.105}$	11	$m = 0.174 Q^{0.093}$
Lower Jhelum Canal.	20	$m = 0.139 Q^{0.114}$	19	$m = 0.141 Q^{0.110}$
L. C. C. + L. J. C. ...	36	$m = 0.152 Q^{0.110}$	30	$m = 0.156 Q^{0.102}$

THE BEHAVIOUR AND CONTROL OF RIVERS AND CANALS

The above Table and Figs. 3-6 (a) and (b) show a gradual diminution of mean diameter of bed sand with discharge, the reduction being approximately proportional to $Q^{1/10}$; whereas on plotting "m" of the Punjab data against L = distance from Canal Head, it was found that there was wide scatter; and, on comparing with Fig. 3-7 (b), it was obvious that the m:Q relation gave a considerably better fit than the m:L relation.

It was similarly found that sand exposed on the beds of off-taking channels—resulting from natural selection at the heads of off-takes—tended to vary as $Q^{1/10}$.

TABLE 3-III

L. C. C.	Serial numbers.	Formulae.
1. Whole Canal Group	1 to 16 (2) & (3).	$m = .167 Q^{.103}$
2. 11 regime sites (i. e. excluding sites 2, , 9, 8, 15.)	1 to 16 (2) & (3).	$m = .174 Q^{.093}$
3. Sagar group	(13), 3 and 1	$m = .209 Q^{.075}$
4. Nanuana group	6, 10, 8 & 7	$m = .167 Q^{.119}$
5. Khudayar group	12, (2), 11, 16, 14, 13 and 15.	$m = .170 Q^{.091}$

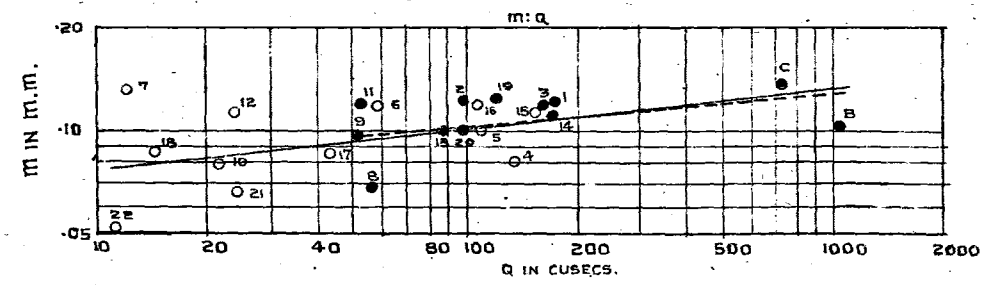
(Remarks: sites shown in brackets are according to old numbering of sites in 1937 P. I. R. I. Report.)

Mean of 3 groups 3, 4 and 5: $m \propto Q^{.095}$

TABLE 3-IV

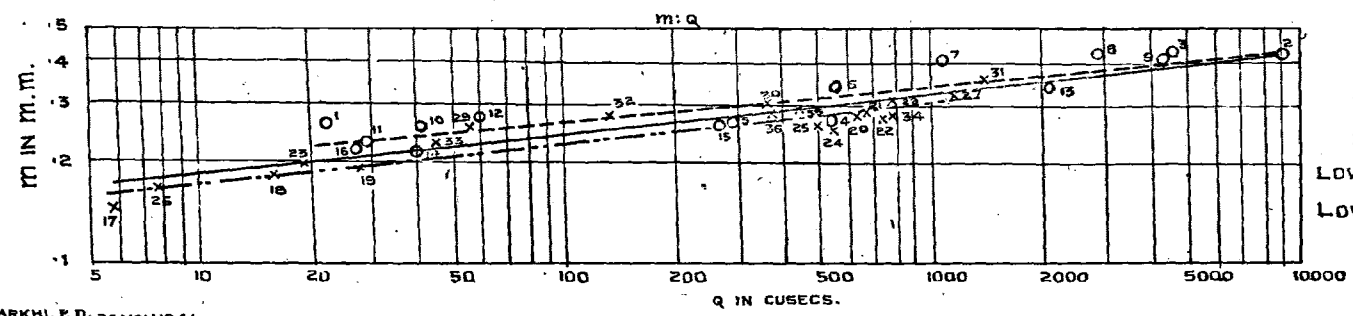
L. J. C.	Serial numbers.	Formulae.
1. Whole Canal	17—36	$m = .139 Q^{.114}$
2. 19 regime sites (i. e. excluding site 31)	...	$m = .141 Q^{.110}$
3. Rurala group	17, 19, 20-23	$m = .132 Q^{.112}$
4. Ludewala group	28, 29, 30	$m = .196 Q^{.068}$
5. Satghara group	31, 32, 33	$m = .153 Q^{.118}$
6. Mean of 3 groups	items 3, 4 and 5	$m \propto Q^{.099}$

FIG: 3-6 (a)
 JAMRAO ALL SITES (OMITTING A)
 1933 DATA OF m IN T. P. 54; Q IN 1932
 (ORIGINAL VALUES OF m)



$m = 0.06 Q^{.115}$ (FOR 24 SITES) ———
 $m = 0.06998 Q^{.0875}$ (FOR 12 SITES) - - -
 REGIME SITES ARE SHOWN THUS

FIG: 3-6 (b)
 PUNJAB DATA



$m = 0.152 Q^{.110}$ FOR ALL 36 SITES ———
 $m = 0.165 Q^{.105}$ FOR 1 TO 16 SITES - - -
 $m = 0.139 Q^{.114}$ FOR 17 TO 36 SITES ———
 LOWER CHENAB CANAL o
 LOWER JHELUM CANAL x

113

FIG-3-7 (a)
PUNJAB DATA
LOWER CHENAB CANAL
 $M:Q$

N^o 2034
CENTRAL IRRIGATION & HYDRODYNAMIC
RESEARCH STATION
POONA

(FIG. 10 & 11. FROM A.R.T.
1941-42 OF C.I. & H.R.S.)

RELATION FOR	SITE NOS	REFERENCES	EQUATION
THE WHOLE CANAL	ALL SITES (700) (6) & (3) OLD	—	$M = 0.167 Q^{0.103}$
MAIN LINE	2, 3, 4, 8, 9 & 13	—	$M = 0.09367 Q^{0.1767}$
SITES ON DISTRIBUTARIES	1, 3 & 15	—	$M = 0.2088 Q^{0.0853}$
	6, 7, 8 & 10	—	$M = 0.1667 Q^{0.1132}$
	11, 12, 13, 14, 15, 16 & 17	—	$M = 0.1704 Q^{0.0906}$

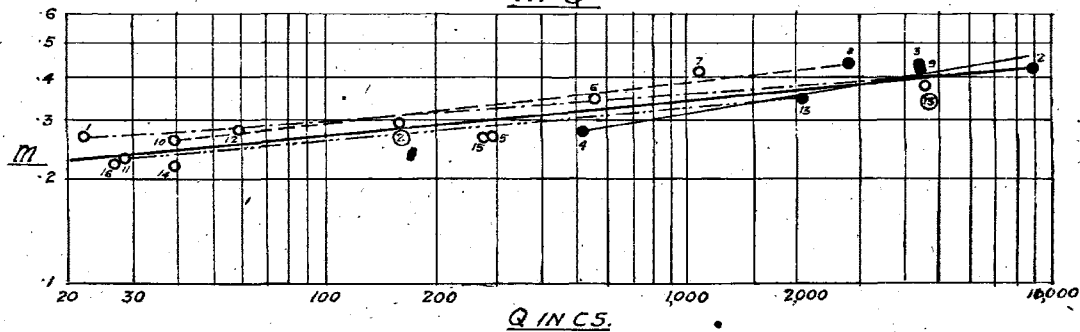
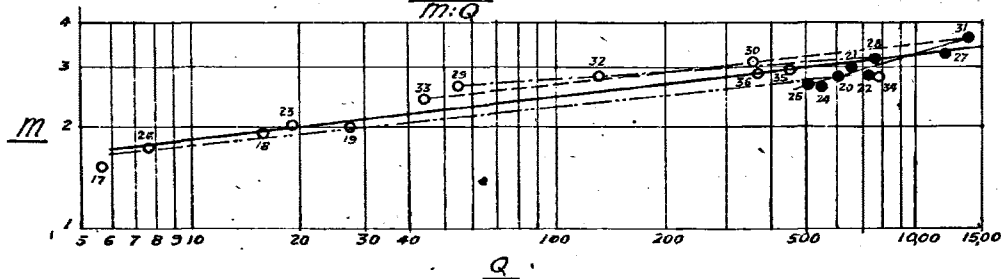


FIG-3-7 (b)
LOWER JHELUM CANAL
SHOWING
 $M:Q$



RELATION FOR	SITE NOS	REFERENCES	EQUATION
WHOLE CANAL	17 TO 36	—	$M = 0.139 Q^{0.114}$
MAIN CANAL	20, 21, 22, 24 25, 27, 28, 31	—	$M = 0.0339 Q^{0.3001}$
SITES ON DISTRIBUTARIES	17, 18, 20, 28	—	$M = 0.1523 Q^{0.117}$
29	28, 29, 30	—	$M = 0.1962 Q^{0.0883}$
30	31, 32, 33	—	$M = 0.1508 Q^{0.1177}$

(b). LOWER CHENAB CANAL.
 $f_{RS} = 0.7884 Q^{-0.0535}$; $f_{VR} = 1.2818 Q^{-0.0416}$; $f_m = 1.001 Q^{0.0069}$

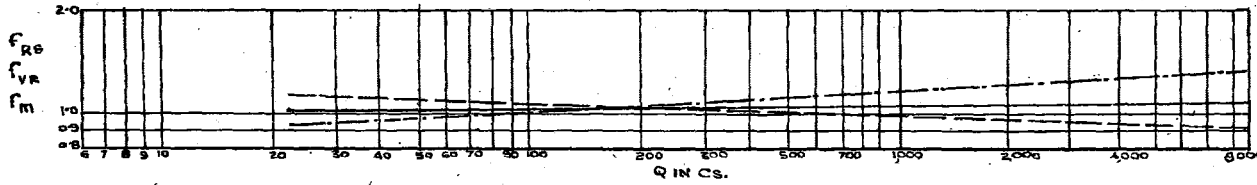
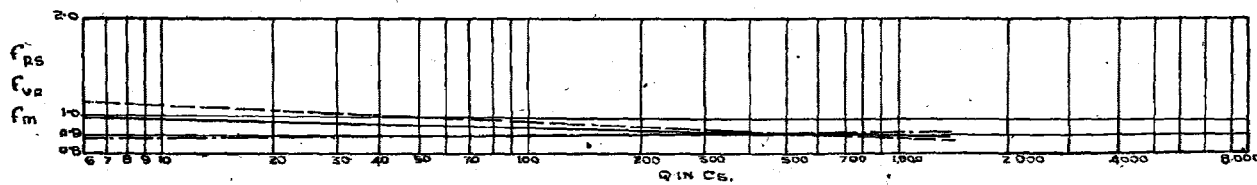


FIG:- 3-8
 GRAPH SHOWING RELATION BETWEEN
 f_{RS}, f_{VR} & $f_m : Q$ OF
 20 SITES OF LOWER JHELUM CANAL
 16 SITES OF LOWER CHENAB CANAL

(c) LOWER JHELUM CANAL.

$f_{RS} = 0.8667 \cdot Q^{-0.00724}$; $f_{VR} = 1.2025 Q^{-0.0453}$; $f_m = 1.021 Q^{-0.0192}$



(FIG. 12 b, c & 13 b. c. FROM
 A.R.T. 1941-42 OF C.I. & H.R. STN)

(b) LOWER CHENAB CANAL (11 SITES)

$f_{VR} = 1.119 Q^{-0.0115}$; $f_{RS} = 0.868 Q^{-0.266}$; $f_m = 0.997 Q^{0.0069}$

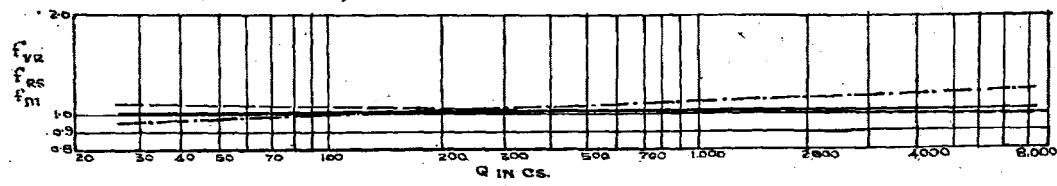
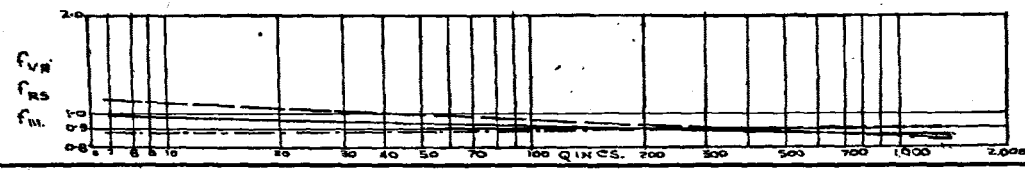


FIG:- 3-9
 PUNJAB DATA.
 $f_{VR}, f_{RS}, f_m : Q$ FOR
 REGIME SITES.

(c) LOWER JHELUM CANAL (19 SITES)

$f_{VR} = 1.201 Q^{-0.0451}$; $f_{RS} = 0.883 Q^{0.0021}$; $f_m = 1.03 Q^{-0.0215}$



117

DIVERGENCE FROM REGIME

Taken as a whole, therefore, the sand-selective relation for the two canals is $m \propto Q^{-0.95}$ and $\propto Q^{-0.88}$ compared with $Q^{-1.05}$ and $\propto Q^{-1.14}$ for the whole of the L. C. C. and L. J. C.; and $\propto Q^{-0.93}$ and $\propto Q^{-1.10}$ for regime sites for L. C. C. and L. J. C. (This data is lacking for the Jamrao.)

3.16 (ii) $f_{vB} : Q$; $f_{BS} : Q$; $f_m : Q$ RELATIONSHIPS IN A CANAL SYSTEM.

TABLE 3-V

All sites.	No.	m =	$f_{vB} =$	$f_{BS} =$	$f_m =$
Jamrao Canal	24	$.060 Q^{-1.15}$	—	—	—
Lower Chenab	16	$.165 Q^{-1.05}$	$1.282 Q^{-0.416}$	$.788 Q^{-0.54}$	$1.001 Q^{-0.069}$
Lower Jhelum	20	$.139 Q^{-1.14}$	$1.202 Q^{-0.453}$	$.867 Q^{-0.007}$	$1.021 Q^{-0.192}$
L. C. C. + L. J. C.	36	$.152 Q^{-1.10}$	$1.244 Q^{-0.440}$	$.827 Q^{-0.030}$	$1.011 Q^{-0.061}$
SELECTED SITES					
Jamrao Canal	12	$.07 Q^{-0.88}$	$0.601 Q^{-0.009}$	$.604 Q^{-0.144}$	$0.593 Q^{-0.094}$
Lower Chenab	11	$.174 Q^{-0.93}$	$1.119 Q^{-0.113}$	$.858 Q^{-0.366}$	$0.997 Q^{-0.069}$
Lower Jhelum	19	$.141 Q^{-1.10}$	$1.202 Q^{-0.451}$	$.883 Q^{-0.021}$	$1.031 Q^{-0.215}$
L. C. C. + L. J. C.	30	$.156 Q^{-1.02}$	$1.153 Q^{-0.283}$	$.870 Q^{-0.194}$	$1.013 Q^{-0.073}$

A statistical analysis—see Table 3-V and Figs. 3-8 and 3-9 (for all sites, and selected sites respectively) showed that whereas f_{BS} decreased with decrease in discharge, f_{vB} increased with decrease in discharge. This inverse relation, Lacey called "the Crump effect."

When, however, f_m , the geometrical mean of f_{BS} and f_{vB} was taken

$$f_m = \sqrt{f_{BS} f_{vB}} \quad \dots \quad (32)$$

this canal system value was practically constant irrespective of the discharge, and despite the fact that m was proportional to $Q^{1.0}$.

THE BEHAVIOUR AND CONTROL OF RIVERS AND CANALS

3-17 (iii). S:Q RELATIONSHIP IN A CANAL SYSTEM

TABLE 3-VI

Canal.	No.	All Sites.	No.	Selected Sites.
Jamrao Canal	24	...	12	$S = .000228/Q^{.147}$
Lower Chenab	16	$S = .00033/Q^{.059}$	11	$S = .000452/Q^{.116}$
Lower Jhelum	20	$S = .000449/Q^{.162}$	19	$S = .000462/Q^{.170}$
L. C. C. + L. J. C.	36	$S = .000385/Q^{.11}$	30	$S = .000456/Q^{.143}$

Lacey values $S = .000547 f_m^5 / Q^{.5}$ $S = .000547 f_{VR}^5 f_{RS}^2 / Q^{.5}$
 or $S = .000547 f_m^3 f_{RS}^4 / Q^{.5}$

Fig. 3-10
(a), (b) &
(c)

Table 3-VI and Figs. 3-10 (a), (b) and (c) show the sensitivity of slope to divergence from regime, which is borne out by the L. C. C. data diverging considerably from the L. J. C. and Jamrao data, showing that L. C. C. slopes had not reached final stability when these data were collected, which is known to have been the case. In effect this means that the slope of a canal cannot be estimated from grade, but must also take 'charge', which is a major factor, into account—see Chapter 7.

If we throw the slope formula into the form

$$S \propto f_m^y / Q^x$$

to obtain a canal system type of formula, we get

L. C. C. + L. J. C. 30 selected sites.	Jamrao 12 selected sites.
$S = .000456 Q^{-1.43}$	$S = .000228 Q^{-1.47}$
$= .000458 Q^{-1.45}$	$= .000225 Q^{-1.46}$
$f_m = 1.0103 Q^{-0.075}$	$f_m = .593 Q^{-0.054}$
$= .98 Q^0$	$= .62 Q^0$

Combining the two sets of data for L. C. C., L. J. C. and Jamrao gives

$$S = .000474 f_m^{1.53} / Q^{1.45} \dots \dots \dots (33)$$

3.18. (iv) V: (R²S) RELATIONSHIP IN A CANAL SYSTEM

TABLE 3-VII

Canal.	Sites.	All Sites.	Sites.	Selected Sites.
Jamrao Canal	24	...	12	$V = 11.44(R^2S)^{.290}$
Lower Chenab	16	$V = 9.215(R^2S)^{.248}$	11	$V = 11.98(R^2S)^{.287}$
Lower Jhelum	20	$V = 11.71(R^2S)^{.281}$	19	$V = 12.09(R^2S)^{.286}$
L. C. C. + L. J. C.	36	$V = 10.39(R^2S)^{.265}$	30	$V = 12.03(R^2S)^{.286}$

FIG. 3-10 (a)
JAMRAO DATA
 FOR 12 SITES IN CANAL &
 HEADS OF DISTRIBUTARIES.
 $S = 0.0002284 Q^{-1.473}$

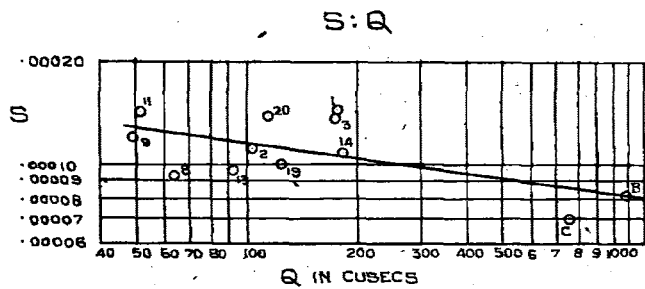


FIG. 3-10 (b)
PUNJAB DATA

SITES 1 TO 16 LOWER CHENAB CANAL
 $S = 0.00033 Q^{-0.0586}$ FOR 16 SITES. ———
 $S = 0.00045 Q^{-1.16}$ FOR 11 SITES. - - - - -
 S: Q

NO 2031.
 CENTRAL IRRIGATION &
 HYDRODYNAMIC RESEARCH
 STN. POONA.
 (FIG. 14 D TO C FROM
 A. R. T. 1941-42 OF
 C. I. & H. R. STN)

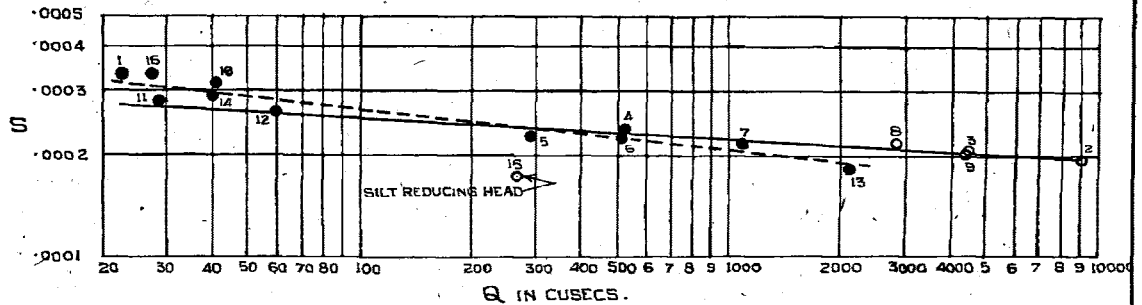


FIG. 3-10 (c)
PUNJAB DATA
 SITES 17 TO 36 LOWER JHELUM CANAL
 $S = 0.000449 Q^{-0.1618}$ FOR 20 SITES. ———
 $S = 0.000462 Q^{-1.17}$ FOR 13 SITES. (SITE 31 OMITTED) - - - - -
 S: Q

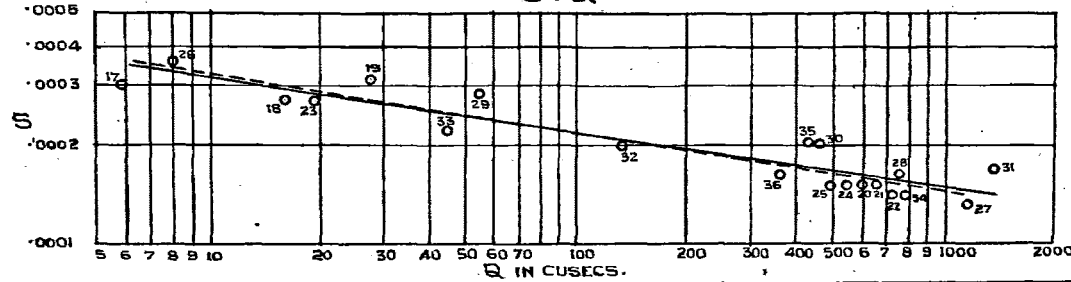


FIG-3-11(a) JAMRAO DATA.

SITES AT HEAD OF DISTRIBUTARIES

$V: R^2 S.$

$V = 11.44 (R^2 S)^{.2895}$ REGIME SITES.-----

$V = 16.05 (R^2 S)^{1/3}$ LACEY.-----

(FIG. 15 & TO C
FROM A. R. P.
1941-42 OF C.I.
& H. R. S. N.)

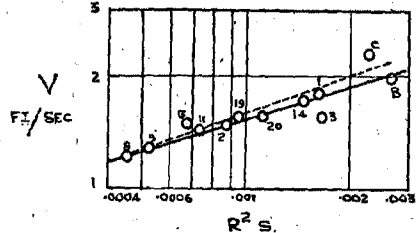


FIG-3-11(b) PUNJAB DATA (L.C.C.)

$V: R^2 S.$

1 TO 16 SITES FOR L.C.C.

$V = 9.21 (R^2 S)^{.248}$ FOR 16 SITES.-----

$V = 11.98 (R^2 S)^{.287}$ FOR 11 SITES.-----

$V = 16.05 (R^2 S)^{1/3}$ LACEY.-----

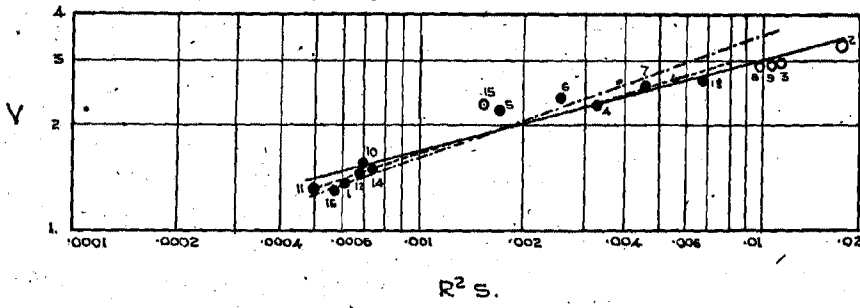


FIG-3-11(c) PUNJAB DATA (L.J.C.)

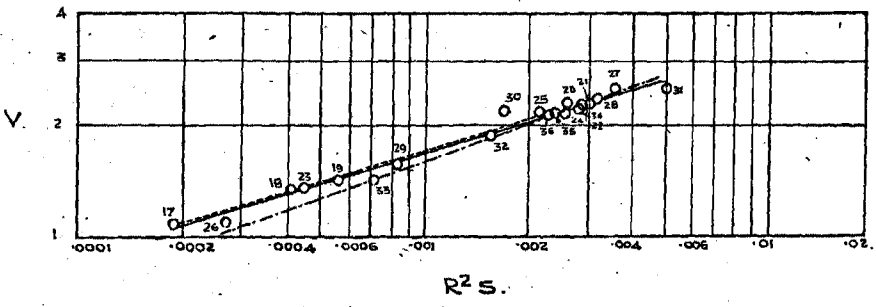
$V: R^2 S.$

17 TO 36 SITES FOR L.J.C.

$V = 11.71 (R^2 S)^{.281}$ FOR 20 SITES.-----

$V = 12.09 (R^2 S)^{.286}$ FOR 19 SITES.-----

$V = 16.05 (R^2 S)^{1/3}$ LACEY.-----



DIVERGENCE FROM REGIME

The data in Table 3-VII and Figs. 3-11 (a), (b) and (c) give
 $V = 12.0 (R S)^{2/7}$ very approximately ... (34)

Fig. 3-11
(a), (b) &
(c)

This may be compared with Lacey's final regime formula

$$V = 16.00 (R^2 S)^{1/3} \dots \dots \dots (16)$$

Thus, formula (34)—based on the canal data—differs appreciably from Lacey's final regime formula (16).

3.19. (v) $V = 1.346 R^{3/4} S^{1/2} / N_a$ —THE LACEY RUGOSITY FORMULA

Lacey's rugosity formula is

$$V = \frac{1.346}{N_a} R^{3/4} S^{1/2} \dots \dots \dots (15)$$

Lacey linked this formula with his final regime equation (16) as follows:

$$V \propto \sqrt{fR} \text{ and } V \propto R^{3/4} S^{1/2} / N_a$$

He then took $N_a \propto f_n^{1/4}$ and $f_n = f_{vR}$; and, by substitution got

$$V \propto R^{3/4} S^{1/2} R^{1/4} / V^{1/2} \text{ or } V \propto R^{3/2} S^{1/2} \dots \dots (35)$$

Actually, however, f_n is not proportional to f_{vR} ;

$$f_n \propto f_{RS}^2 / f_{vR}^2 \dots \dots \dots (36)$$

$$\text{So } N_a \propto f_{RS}^{3/4} / f_{vR}^{1/2} \dots \dots \dots (37)$$

Now f_{RS} varies directly as Q , whereas f_{vR} varies inversely. This explains why there is a big divergence between Lacey's formula and the statistical formula for the L. C. C., L. J. C. and Jamrao canal systems.

3.20. (vi) P/R: Q OR SHAPE IN A CANAL SYSTEM

Lacey's assumption that channels in incoherent alluvium are naturally semi-elliptical in shape leads to a semi-circle in the limit, and to both V and Q having limits: From Lacey's dynamic shape formula $P/R = 7.111 V$, we get limiting conditions for

$$(P/R) = \frac{\pi r}{r/2} = 2\pi = 7.111 V \dots \dots (38)$$

$$\text{So } V = 2\pi / 7.111 = 0.884 \text{ ft./sec.} \dots \dots (39)$$

Similarly the Lacey relation $P/R = 5.65 f^{1/2} Q^{1/6}$ gives

$$Q_{lim} = (44/7 \times 5.65 f^{1/2})^6 \cdot 112 = 1.89 / f^2 \dots \dots (40)$$

Actually, there is no such limit, nor apparently any change of law for steady flow conditions till much lower velocities are reached.

Fig. 3-12 (a) and (b) show curves relating $P/R: Q$ drawn by eye from the data for the Jamrao, L. C. C. and L. J. C. All these show a definite curvature which is less in the Punjab where the channels contain medium sand* and are relatively wide and shallow, than in the Sind channels, which contain fine sand and are relatively deep and narrow, so that the divergence from Lacey's straight line relation is greater. An increasing value of P/RV with lower discharge is borne out by model experiments.

Fig. 3-12
(a) & (b)

*Silt is material less than .0625 mm. in diameter.
 Very fine sand .0625 to .125 mm. in diameter.
 Fine sand .125 to .167 mm. in diameter.
 Medium fine sand .167 to .25 mm. in diameter.
 Medium sand .25 to .5 mm. in diameter.
 Coarse sand .5 to 1.0 mm. in diameter.

3.21. CONCLUSIONS DRAWN IN PAPER ON "DIVERGENCE FROM REGIME" (1941)

The origin of this Note was not the desire of another fool to rush in where angels fear to tread; but the fact that two problems had been submitted for advice, which were outside the range covered by Central Board of Irrigation discussions on "Flow in Open Channels". A study of these two cases left no doubt that 'sand load'—not 'shock' due to bad maintenance or wrong design—was the dominant factor causing divergence from regime.

The first problem was in connection with the Kosi River,* which, after piercing the foothills of the Himalayas, flows into the plain, carrying a very heavy charge of detritus and white sand. The charge of sand is so great that, despite a flood discharge of some 200,000 cusecs, the river has no permanent deep channel; but flows with a small depth over a wide stretch of country, scouring new channels wherever a resistant face creates sufficient resistance to enable it to cut a curved channel, while it is continually filling in its recent channels. This river has been moving remorselessly towards the west in a series of periodic attacks for hundreds of years, at an average rate of a mile a year.

The second example was the Jamrao Canal in Sind, which forms part of the Eastern Nara system of Canals. These derive their supply from the Indus through the Eastern Nara Canal, which is one of the Left Bank Canals of the Sukkur Barrage Group.

Completed in 1896, the head reaches of the Jamrao began to widen and shoal at once; and by 1908 the fall at 7th mile had almost disappeared.

The late Sir Frederick Gebbie, the then Executive Engineer, endeavoured to control its width by building brushwood groynes all along both banks; but these were steadily attacked and had to be continually repaired.

After he left, the groynes were not maintained so well, and Mr. Trench eventually decided, about 1910, to abandon them. The effect of this was surprisingly small, the channel widening by only a few feet, the width then remaining steady.

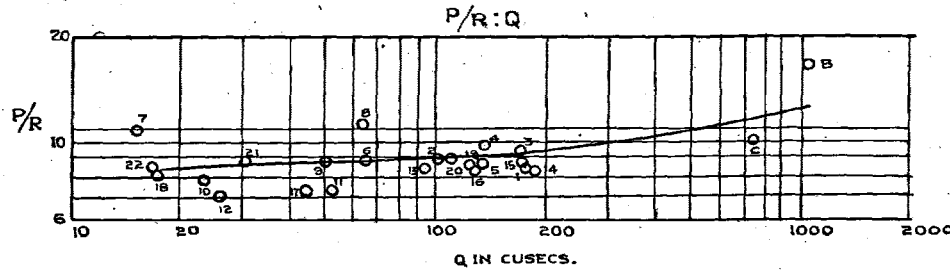
The width of the Jamrao Canal in 1941 was very much the same as in 1910—about 1.3 times the Lacey regime width, i.e. $P = 3.5 Q^{\frac{1}{2}}$. The slope in the head reach is steep, being 1/5,000 against 1/6,800 according to Lacey regime: While N_a is .0289 against .0237 calculated from

$$N_a = 0.0225 f^{\frac{1}{4}} = \frac{1.346 R^{\frac{3}{4}} S^{\frac{1}{2}}}{V} \dots \dots (41)$$

The reverse effect, retrogression, which results from a sub-regime sand-charge, is equally definite; as exemplified in the case of the Mithrao Canal in Sind, where the addition of a sand excluding approach channel increased the discharge from 1,750 cusecs to 2,140 cusecs in 6 months; and, in addition, lowered the water level at its head by 1' 6".

*See pp. 13-15 of C. I. & H. B. S. A. R. (T) 1941-42.

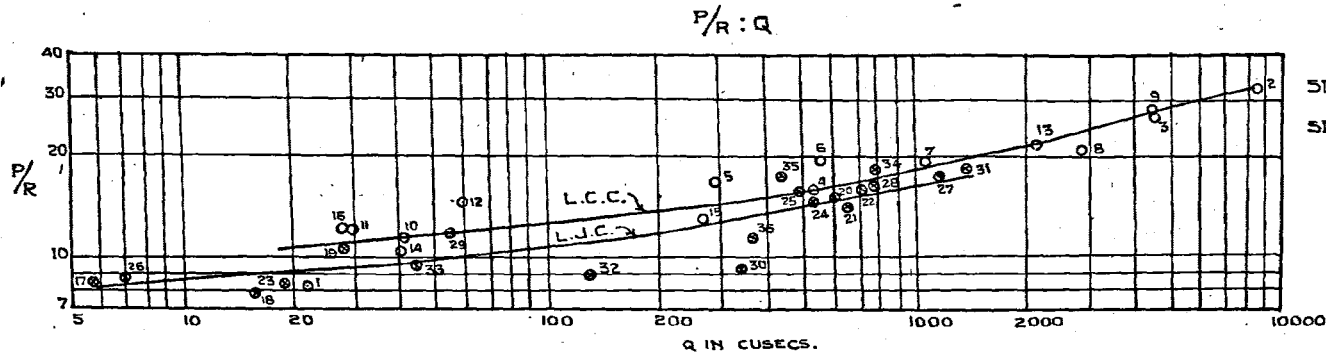
FIG: 3-12 (a)
 JAMRAO CANAL EX NARA RIVER
 1930 DATA FOR SITES 1 TO 22
 1932 DATA FOR SITES B & C



NO: 2035
 CENTRAL IRRIGATION AND
 HYDRODYNAMIC RESEARCH STN POONA

(FIG. 16 a & b FROM A.R.T.
 1941-42 OF C.I. & H. R. STN)

FIG: 3-12 (b)
 PUNJAB DATA



SITES 1 TO 16 LOWER CHENAB ○
 SITES 17 TO 36 LOWER JHELUM ●

DIVERGENCE FROM REGIME

There is nothing new in calling attention to the marked effect of sand-charge on slope and dimensions. Indeed Mr. E. W. Lane*, when replying to the discussion on his Paper No. 1957 of the Am. Soc. of C. E. "Stable Channels in erodible material", said, in 1936,

"The importance of silt quantity as a factor in stable channel shape and the tendency of heavily silt-laden water to form wide shallow channels seems to have been generally accepted."

and Lacey himself wrote in his discussion on that Paper:—

"In the writer's view 'turbulence' is susceptible of definition, and the ratio V^2/R , for any grade of silt and irrespective of the charge, epitomised turbulence. It is thus found that fine silt with a heavy surcharge may well demand the same value of V^2/R as a coarse silt with a smaller charge in another locality."

What matters is that none of the formulæ we have been considering in India have taken load explicitly into account, nor has divergence from regime been hitherto definitely linked with load.

In this Paper, the data has been grouped by canals, and hence produced what are essentially "canal system relationships"; by which is meant formulæ for constant load (X), the grade diminishing with discharge.

"The data shows that in a Canal system, the relation between discharge and grade of bed-mix is

$$m \propto Q^{1/10} \text{ (approx.)} \quad \dots \quad \dots \quad \dots \quad (42)$$

This holds not merely for the main Canal; but also for sand selection at offtakes."

The most important point brought out is that Lacey's assumption that $f_{VR} = f_{RS}$ does not hold in canal systems, in which f_{RS} varies with the discharge, whereas f_{VR} varies inversely as the discharge; so there is only one region where the f_{VR} and f_{RS} lines intersect where $f_{VR} = f_{RS}$ see Fig. 3-8.

Fig. 3-8

$f_m = \sqrt{f_{VR} f_{RS}}$ gives, however, an almost constant value.

Similarly $f_n = (Na)^*$ is not equal to f_{VR} in a canal system but is proportional to f_{RS}/f_{VR} .

"It has been shown—para. 3.13—that after an equal degree of agitation, the rate of deposition of sand is the same for two samples when the product ($X \cdot V_s$) is the same; but Punjab investigations showed the relation $V_s = 16.2 m - 0.58$; so for sands V_s is not far from $\propto m$."

It appears, therefore, that $X \cdot V_s \propto X \cdot m \propto Q^{1/10}$ holds approximately for constant load and varying grade, and for constant grade and varying load, or any variation giving constant ($X \cdot V_s$). So a channel which was not in regime, due to the grade being too coarse for the load, can be brought into regime—even should the bed grade remain the same—by extracting part of the sand load. It has, of course, long been realised that sand-excluders and extractors have this effect; but it has generally been assumed that this was due to extracting the coarser particles. Indeed many engineers have believed that off-takes normally extract a disproportionate share of the coarser particles from the parent channel and have assumed that excluders and extractors are required merely to set this right.

Actually, however, sand extractors cause a reduction both of grade and load; and so, by extracting a small quantity of sand, work effectively.

*"Stable Channels in erodible material" by E. W. Lane. Paper No. 1957 Am. Soc. C. E. November 1935.

$(X \cdot m)/Q^2$ should replace $m:Q^2$ in Bose type formulæ; and for canal system formulæ f_m (const.) should be used as a sand-factor linking up load, grade and discharge. This should be done even before X is evaluated, and even if X cannot be, or is not, directly measured; being only inferred or estimated from experience or from non-regime slope, and shape, data; because we must prevent ourselves slipping back into the careless habit of ignoring 'charge'.

It will be noted that exponents in all the formulæ derived from the canal data vary slightly from those of Lacey, and that the formulæ based on data of selected sites—i.e. excluding points with super-regime bed-sand charge—approach the Lacey formulæ more closely than 'all sites' data. When the formulæ are modified for variable $f \propto Q^{-0.44}$ (derived from the all sites data) the formulæ agree very closely with the Lacey formulæ, except the final regime formula $V = 16.00 R^{2/3} S^{1/3}$ and the slope formulæ

$$S = .000547 f^{5/3} / Q^{1/6} \dots \dots \dots (13)$$

These formulæ should be written

$$V = 16.00 R^{2/3} S^{1/3} (f_{VR}/f_{RS})^{1/2} \dots \dots \dots (17)$$

$$S = .000547 f_{VR}^{1/6} \cdot f_{RS}^{3/2} / Q^{1/6} = .000547 f_m^{1/6} \cdot f_{RS}^{4/3} / Q^{1/6} \dots \dots \dots (43)$$

Lacey's final-regime formula (17) is very sensitive to divergence from regime; because f_{VR} varies inversely as f_{RS} ; so that divergence both in f_{VR} and f_{RS} pulls the curve away from the regime curve in the same direction; and hence the canal system formula

$$V = 12.00 (R^2 S)^{2/7} \dots \dots \dots (44)$$

differs widely from Lacey's regime formula

$$V = 16.00 (R^2 S)^{1/3} \dots \dots \dots (16)$$

In a stable channel, the load equivalent to that which enters the canal, though not of the same grade, must be transported over a period of a year; but temporarily, accretion or scour will occur; the former, where the load exceeds the stable load, so that the channel is building up its slope with the excess load to enable it to carry the increased load; but, in practice, a new stability is rarely attained before a further change occurs. The mechanism of change is, however, clear. If the energy and turbulence are only slightly in excess of normal, some of the finer bed sand will be whipped into suspension causing coarser particles to be exposed on the bed; whereas, when a load considerably in excess of normal enters a channel, 'blanketing' will occur—that is to say the true bed will be covered over by a layer of sand, which is generally considerably finer than the original bed material. If, however, $X \cdot Vs$ is only slightly greater than before, blanketing will not occur, but grade dilution will result, the finer sand mixing with the coarser sand exposed on the bed."

3.22. BLENCH'S INSTITUTION PAPER ON A UNIVERSAL FLOW FORMULA

In 1939, Mr. T. Blench wrote a Paper entitled "A New Theory of Turbulent Flow in Liquids of Small Viscosity,"*

In this he postulated that the apparently different flow formulæ for rough and smooth stage boundary conditions of conventional hydraulics and for the mobile boundary stage of sediment-bearing channels, must be special cases of a "universal flow formula". As the Blasius formula for smooth stage boundary flow was undisputed over a wide range of Reynolds number, he accepted its form as giving a major clue to the exact expression of this formula and concluded that it must have the form

$$V = \text{abs. const. } (R/x)^{1/4} \sqrt{gRS} \dots \dots \dots (45)$$

* Journal, Institution of Civil Engineers, April 1939.

DIVERGENCE FROM REGIME

In the rough stage, x would be a measure of protruberance height; in the smooth stage, it would be a measure of 'laminar film thickness'; and for the mobile stage would represent a dimensionally correct bed-sand factor.

For the 'rough stage boundary flow' this gave a formula of the Manning type, with the exponent $2/3$ replaced by $3/4$ as in the Lea-Lacey formula; for smooth stage boundary flow it reduced to the Blasius formula; and in the third case it reduced to the Lacey V, R, S formula. So, accepting the postulate of universality, it was valid up to Reynolds numbers of the order of 10^6 .

In 1941 Blench and King working in co-operation, wrote a Paper entitled "Effect of Dynamic shape on Lacey Relation;"*. This was followed by "Practical Design Formulæ for Stable Irrigation Channels"† by C. King, and later by "Turbulent Flow Theory from the view-point of an Irrigation Engineer from India"‡ by T. Blench.

Their conclusions were that—

(i) The constants of Lacey formulæ are different for different channel systems.

(ii) The values of f derived from different formulæ applied to the same collection of data are different though functionally inter-related.

(iii) Within appreciable limits, changes of width enforced by engineering action will remain, and be accompanied by corresponding changes in depth and slope.

(iv) The completely different nature of canal beds and sides and the apparently different boundary stage conditions of bed and sides of rivers, require that Lacey's f be replaced by a bed factor b and a side factor s .

(v) Lacey tacitly averaged the relative importance of beds to sides; so his results apply to a special system of regime channels selected from the totality of possible regime channels.

(vi) V^2/D is more representative of bed material than is f based on V^2/R .

(vii) The Lacey function VR/P (or Q/P^2) which can be averaged over an enormous range of Q must have a physical significance but VR/P is approximately equal to $V^3/W \div V^2/D$ and if V^2/D is accepted as being more representative of bed material than V^2/R then, as V^2/D is a proportional to the Froude number, it follows that the numerator must also be physically significant and a measure of the side factor.

(viii) If the stage of flow along the sides is assumed smooth turbulent, then conventional hydraulics shows that $\sqrt{V^3/\nu}$ is a measure of side traction, and $\rho V^3/W$ is a measure of the rate of working per unit volume in the laminar film. So, for the Reynolds Number, sectional shapes, and side material covered by Lacey analysis, it appears that the normal side-flow stage is smooth turbulent.

(ix) Thus, two Lacey formulæ related to section but not to slope, must be replaced by:—

$$V^2/D = b \quad \dots \quad \dots \quad \dots \quad (46)$$

$$\text{and } V^3/W = s \quad \dots \quad \dots \quad \dots \quad (47)$$

* Annual Report (Technical), Central Board of Irrigation (India) 1941, pages 75-84.

† A. R. (T) C. B. I. (India) 1943.

‡ Written for U. S. Bureau of Reclamation Library, Denver, Colorado, also in La Houille Blanche "Theorie de l'Ecoulement Turbulent".

THE BEHAVIOUR AND CONTROL OF RIVERS AND CANALS

(x) For canals, with their cohesive sides, s lies between extremes of about 0.10 to 0.30 with 0.20 a good mean. The engineer can enforce it over a range of about 0.10 for any given material, because, taking it as a measure of tractive force, it can lie anywhere between values depending on the force required to prevent deposition of suspended clay and that required to erode clay that has cohered.

(xi) The Lacey relation $VR/P = \frac{1}{7.11}$ is an arbitrary average of the dynamic shape a/b ; and since his derived equations contain, in some form or other, his constant 7.11, whereas they ought to contain dynamic shape instead, it follows that the f 's of his different equations, when found from actual data, must be related, through dynamic shape, and be different from each other.

(xii) In the conventional Reynolds logarithmic diagram for pipes $2gdS/V^2$ is plotted against Vd/ν . If gDS/V^2 for canals be plotted against VW/ν , the result is practically coincident with the Blasius smooth turbulent line of the pipe diagram. The coincidence seems accidental, but the parallelism is physically significant.

The equation may be written

$$V = \text{abs. const. } (VW/\nu)^{1/8} \sqrt{gDS} \dots \dots (48)$$

whose form is exactly that of the Blasius line for pipes, except that the Reynolds Number appears in terms of W .

The fact that this equation is of the Blasius form makes it fit into the frame-work of the "universal flow formula", which it must do if that formula is valid.

Equation (48) can be rewritten,

$$V = \text{abs. const. } (g^2/\nu)^{1/8} (b^2/s)^{1/8} D^{3/4} S^{1/2} \dots \dots (49)$$

Comparing this with the corresponding Lacey equation,

$$V = \frac{1.3458}{N_a} R^{3/4} S^{1/2}$$

the bed factor appears in the numerator instead of in the denominator.

For practical design (48) is solved with (46) and (47) to give

$$S = b^{-5/6} s^{1/12} / (2,080 Q^{1/6}) \dots \dots (50)$$

(xiii) The Lacey shock theory is not accepted.

3.23. LACEY'S 1946 PAPER ENTITLED "A GENERAL THEORY OF FLOW IN ALLUVIUM"*

To a large extent this Paper is a restatement of Lacey's previous Papers; and he repeats some ideas which have never been accepted in India—such as 'shock'—see para. 3.10—being the explanation of divergence from regime in large channels.

In this Paper he gives a clearer physical significance to his formulæ; but the most important advance is the replacing of his original silt factor

$$f_{VB} = .75 V^2/R \dots \dots \dots (6A)$$

$$\text{by } f_{SV} = 48 \sqrt{S.V} \dots \dots \dots (51)$$

He calls (SV) a new parameter; but this is incorrect, because Mr. E. W. Lane introduced this parameter as long ago as 1935†—see paras. 3.03—3.05.

* Journal, Institution of Civil Engineers, November 1946, Paper No. 5518.

† Am. Soc. C. E. 1935 "Stable Channels in Erodible Material" by E. W. Lane.

DIVERGENCE FROM REGIME

Postulating that particles of silt (sand) of the same grade and density fall in the restricted depths of a model at the same rate as in the greater depths of the prototype, he has produced the relation

$$S \cdot V = K \cdot V_s \dots \dots \dots (52)$$

where V_s is the terminal velocity of particles falling in still water; but this relation could only hold for a particular charge—because $(S \cdot V)$ increases with charge, whereas V_s does not.

We may expect that $(S \cdot V)$ tends to vary as $(X \cdot V_s)$ where X is the charge.

Some corresponding equations in terms of f_{sv} instead of f_{va} are shown below.

Earlier form of equation.	New form of equation.	
$Na = .0225 f^{\frac{1}{2}} \propto f_{RS}^{\frac{1}{2}} / f_{VB}^{\frac{1}{2}}$	$Na = .0257 (S^*V)^{1/8} \propto f_{SV}^{\frac{1}{2}}$	(53)

$m = .3226 f^2 \propto f_{VA}^{\frac{1}{2}} \cdot f_{RS}^{3/2}$	$m = .73 (S^*V) \propto f_{SV}^2$	(54)
---	-----------------------------------	------

$V = 1.155 \sqrt{R} \propto f_{VB}^{\frac{1}{2}}$	$V = 1.42 (S^*V)^{\frac{1}{4}} R^{\frac{1}{2}} \propto \sqrt{f_{SV} R}$	(55)
---	---	------

In this Paper Lacey has produced the equation

$$(S^*V) = K (R^{\frac{1}{2}} S^*)^n \dots \dots \dots (56)$$

One form of this is his formula

$$(S \cdot V) = 16.00 (R^{\frac{1}{2}} S)^{4/3} \dots \dots \dots (57)$$

and Lacey points out that "the Phillips equation for the Nile at Beleida is

$$(S \cdot V) = 2.08 (R S)^{1.8} \dots \dots \dots (58)$$

and the Buckley Beleida equation for constant silt charge was

$$N = K R^{.65} S^{.79}$$

which is almost identical with

$$S \cdot V = K (R^{\frac{1}{2}} S)^{1.97} \dots \dots \dots (59)$$

Actually equation (57) is incomplete, and should be written

$$S \cdot V = 16.00 R^{9/3} S^{4/3} (f_{VB}/f_{RS})^{\frac{1}{2}} \dots \dots \dots (60)$$

Lacey has plotted (S^*V) against $(R^{\frac{1}{2}} S^*)$ in his normal diagram about which he wrote:

"There is no doubt of the general trend of the diagram, and no statistical analysis is necessary in order to obtain a sufficiently accurate constant and power. It must be remembered that the observations vary enormously in weight, and even if they all carried the same weight, a statistical relationship, treating V , R and S as independent variables, would be objectionable on physical grounds. Such a course of procedure would be equivalent to the abandonment of, say, the Reynolds number, in the analysis of data obtained from pipes."

Fig. 3-13.

This diagram is undoubtedly most impressive—showing a clear-cut trend and little scatter—but Lacey's argument, though plausible, is very dangerous. Actually, alternative formulæ which appear to be equally justifiable on physical grounds, give a slightly better fit, and Dr. Malhotra,* in 1939, worked out the best statistical correlation of Lacey's data to be

$$V = 18.178 R^{.6321} S^{.3426} \dots \dots \dots (61)$$

* Central Board of Irrigation, India, Annual Report (Technical) 1939-40.

THE BEHAVIOUR AND CONTROL OF RIVERS AND CANALS

It will help to clarify the question if the various Lacey formulæ are shown to be derivable from a single general formula

$$V = 1.15473 (2633.88)^n f_{VR}^{\frac{1}{2}} \cdot f_{RS}^{-3n/2} \cdot R^{\frac{1+n}{2}} \cdot S^n \quad \dots \quad (62)$$

n	Lacey formula	where	correctly stated
-1	$(S.V) = .000438f^2$	$f_{SV} = f_{VR}^{\frac{1}{2}} \cdot f_{RS}^{\frac{3}{2}}$	$f_{SV} = f_{VR}^{\frac{1}{2}} \cdot f_{RS}^{\frac{3}{2}} = 48\sqrt{SV}$ (63)
0	$V = 1.15473\sqrt{fR}$	$f^{\frac{1}{2}} = f_{RS}^{\frac{1}{2}}$	$f_{VR} = .75V^2/R$ (64)
$\frac{1}{3}$	$V = 16.00 R^{2/3} S^{\frac{1}{3}}$	$\sqrt{\frac{f_{VR}}{f_{RS}}}$ is omitted by Lacey.	$\sqrt{f_{RS} / f_{VR}} = 16.00R^{2/3} S^{\frac{1}{3}}/V$ (65)
$\frac{1}{2}$	$V = \frac{59.558}{f^{\frac{1}{4}}} R^{\frac{3}{4}} S^{\frac{1}{2}}$	$f_n^{\frac{1}{4}} = f_{RS}^{\frac{3}{4}} / f_{VR}^{\frac{1}{2}}$	$N_R = .0226 f_{RS}^{\frac{3}{4}} / f_{VR}^{\frac{1}{2}} \cdot \frac{R^{\frac{3}{4}} S^{\frac{1}{2}}}{V}$ (66)
1	$V = 3041.42 \frac{R.S}{f}$ Inglis canal system formula*	$f_E = f_{RS}^{3/2} / f_{VR}^{\frac{1}{2}}$	$f_E = f_{RS}^{3/2} / f_{VR}^{\frac{1}{2}} = 3041.42 RS/V$ (67)
$-\frac{1}{3}$	$V = \frac{f_m (R/S)^{\frac{1}{3}}}{12}$	$f_m = f_{VR}^{\frac{1}{2}} \cdot f_{RS}^{\frac{1}{2}}$	$f_m = 12.00V (S/R)^{\frac{1}{3}}$ (68)

It may be pointed out that any number of formulæ could be evolved by assigning other values to n, most of them of no value.

The 'general formula'—(62)—consists of two parts

$$V = 1.15473\sqrt{fR} \quad \dots \quad (6)$$

$$\text{and } S^{-n} = 2633.88^n \cdot f_{RS}^{-3n/2} \cdot R^{n/2} \quad \dots \quad (69)$$

which gives the Lacey relation

$$S = .0003759 f_{RS}^{3/2} / R^{\frac{1}{2}} \quad \dots \quad (8)$$

$$\text{and } f_{RS} = 192.00 R^{\frac{1}{3}} S^{2/3} \quad \dots \quad (8A)$$

$$\text{and } f_{SV} = f_m^{\frac{1}{2}} \cdot f_{RS}^{\frac{1}{2}} = f_{VR}^{\frac{1}{2}} \cdot f_{RS}^{\frac{3}{2}} = 48\sqrt{SV} \quad \dots \quad (51)$$

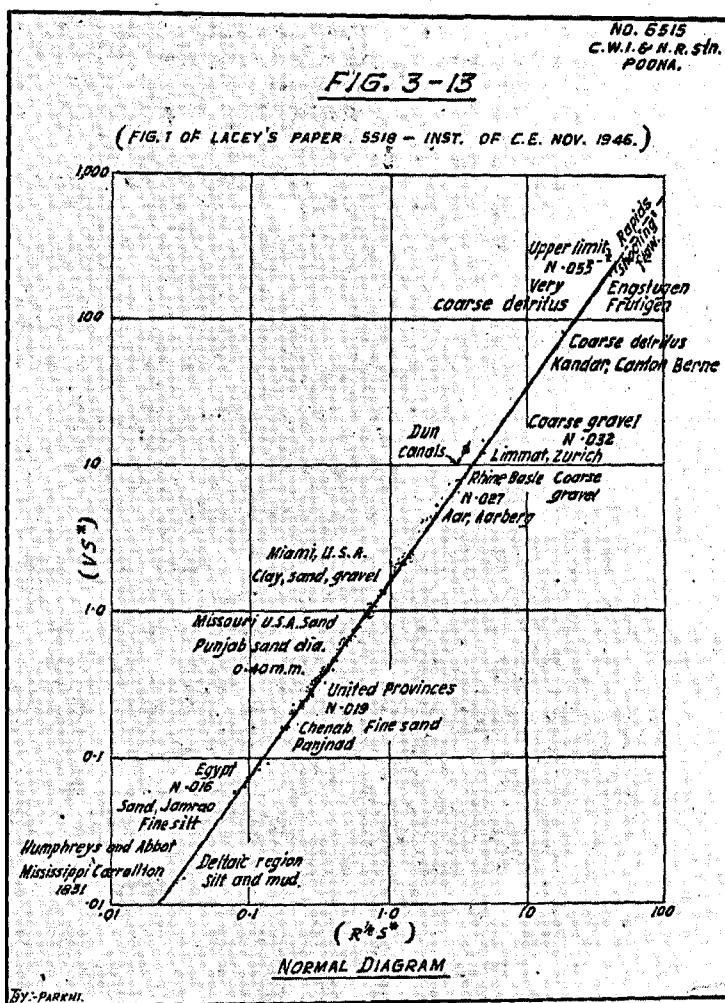
When $P < Q^{\frac{1}{2}}$ is added to the foregoing formulæ, all the Lacey relations follow—provided it is assumed, as Lacey has done, that

$$f_{VR} = f_{RS} \quad f_{SV} = f_m$$

This, effectively, is Lacey's real definition of regime, though never so stated. He has, in fact, given two quite different definitions in this Paper. In his Introduction he wrote

"In two previous Papers, the Author defines the theory of silt transported by 'stable' or 'regime' channels in alluvium. Such channels by definition neither silt nor scour."

* C. I. & H. R. S. A. R. (T) 1941-42 "Divergence from Regime" by C. C. Inglis.



That definition of regime has been generally accepted in India for many years and all the statistical analyses carried out in India have been based on data of such, recognised, regime channels. In these channels the charge of bed material has differed widely. Later in this Paper, Lacey has, however, used the term 'regime' in a markedly different sense, thus confusing this very difficult question still further.

His new definition of regime—restated to avoid unnecessary and undesirable use of the term 'regime silt charge'—as though it had some physical significance and was not merely a yardstick—is

“a regime channel is a stable channel transporting the minimum bed load consistent with a fully active bed. Full activity is such that any reduction would lead to effective partial rigidity, and at the limit to complete rigidity and immobility of the bed.”

As yet, there is no proof that with the minimum silt load consistent with a fully active bed,

$$f_{VR} = f_{RS}$$

This may be true or it may only be a rough approximation; but in any case the fit of Lacey's regime formulæ depends on

$$f_{VR} = f_{RS} = 1.76\sqrt{m}$$

which is rarely found in Nature and then only in restricted lengths even in the case of canals.

3.24. EQUATIONS INTRODUCING CHARGE

Professor C. M. White in his discussion on the Author's Paper* "Meanders and their Bearing on River Training" presented a set of dimensional formulæ of the type

$$\frac{bg^{1/5}}{Q^{3/5}} = \alpha \left(\frac{Q}{m^{5/2} g^{1/2}} \right)^{n_1} \left(\frac{m (c'g)^{1/2}}{g^{1/2}} \right)^{n_2} \left(\frac{G}{Q} \right)^{n_3} \dots \dots (70)$$

where b denotes breadth, m size of solid particles, and G, quantity of solids conveyed.

Working on the assumption that 'charge' and 'rate of fall of particles' were linked—i.e. that (X.Vs) should be grouped together—but that 'charge' was not necessarily linked with 'grade', the Author modified Professor White's formulæ to the form

$$\frac{bg^{1/5}}{Q^{3/5}} = \alpha \left(\frac{Q}{m^{5/2} g^{1/2}} \right)^{n_1} \cdot \left(\frac{XVs}{(vg)^{1/2}} \right)^{n_2} \dots \dots (71)$$

The justification for linking X with Vs and hence (vg) was twofold:—

(i) An investigation carried out at Poona under the Author's direction had shown that the rate of deposition of material in water was constant so long as the product (X.Vs) remained constant; that was, so long as X varied inversely as Vs. † That was the case where the material was quartz sand and m was greater than about 0.15 millimetre and less than about 0.5 millimetre; but for grains smaller than 0.08 millimetre Stoke's law held, and $V_s \propto m^3$ and for grains coarser than about 0.7 millimetre, $V_s \propto m^{1/2}$.

(ii) Experiments initiated by the Author at Poona, when he was Director of the Indian Waterways Experiment Station, had shown that when the 'grade' of material was kept constant and the 'charge' varied, the exponents were similar to those in the new set of formulæ when those exponents were based on Lacey exponents.

It should be pointed out that though according to these formulæ (on the next page) the dimensions would become zero or infinite when the charge became zero—which would also occur if the grade and Vs became infinitely small—this would not occur in practice, because the state of flow would change from rough turbulent to smooth turbulent, causing a change of law.

It is not claimed that these formulæ are more than close approximations; but they bring out qualitatively and possibly even quantitatively, the important differences in the effect of charge and grade of material in regime channels—using that term in its generally accepted significance. Variation of charge has

(a) only a small effect on the area of a channel, and its mean velocity— $(m \cdot XVs)^{1/12}$.

(b) a relatively great effect on slope— $(m \cdot XVs)^{5/12}$.

(c) a relatively great effect on shape— $(XVs)^{7/12}/m^{5/12}$ (where grade has only a small effect).

and

(d) a considerable effect on channel width— $(XVs/m)^{1/2}$.

*Maritime and Waterways Paper No. 7, Institution of Civil Engineers, by Sir Claude Inglis.

†Central Irrigation and Hydrodynamic Research Station, Poona. Annual Reports (Technical) 1940-41 p. 50; 1941-42, p. 38.

DIVERGENCE FROM REGIME

The full set of derived formulæ were.

NEW FORMULÆ

LACEY EQUIVALENTS

$$b = \alpha 1 \cdot \frac{Q^{3/5}}{g^{1/5}} \cdot \left(\frac{Q}{m^{5/3} g^{1/3}} \right)^{1/10} \cdot \left(\frac{XVs}{\nu g^{1/3}} \right)^{1/4}$$

$$= \alpha 1 \cdot \frac{Q^{1/3}}{g^{1/3} \nu^{1/15}} \cdot \left(\frac{XVs}{m} \right)^{1/4} \quad (72) \quad P = 2.67 Q^{1/3}$$

$$A = \alpha 2 \cdot \frac{Q^{1/5}}{g^{1/5}} \cdot \left(\frac{Q}{m^{5/3} g^{1/3}} \right)^{1/20} \cdot \left(\frac{XVs}{\nu g^{1/3}} \right)^{-1/10} \quad f = 1.76 m^{1/3}$$

$$= \alpha 2 \cdot \frac{\nu^{1/20} Q^{5/6}}{g^{7/10} (m \cdot XVs)^{1/10}} \quad (73) \quad A = 1.26 Q^{5/6} / f^{1/3}$$

$$V = \alpha 3 \cdot \frac{g^{7/10}}{\nu^{1/20}} Q^{1/6} (m \cdot XVs)^{1/10} \quad (74) \quad V = 0.7937 Q^{1/6} f^{1/3}$$

$$d = \alpha 4 \cdot \frac{Q^{9/5}}{g^{1/5}} \cdot \left(\frac{Q}{m^{5/3} g^{1/3}} \right)^{-2/10} \cdot \left(\frac{XVs}{\nu g^{1/3}} \right)^{-1/3}$$

$$= \alpha 4 \cdot \frac{\nu^{1/3}}{g^{1/3}} \cdot \frac{Q^{1/3} m^{1/3}}{(XVs)^{1/3}} \quad (75) \quad R = 0.4725 \left(\frac{Q}{f} \right)^{1/3}$$

$$S = \alpha 5 \cdot \left(\frac{Q}{m^{5/3} g^{1/3}} \right)^{-1/6} \cdot \left(\frac{XVs}{\nu g^{1/3}} \right)^{5/12}$$

$$= \alpha 5 \cdot \frac{(m \cdot XVs)^{5/12}}{\nu^{5/12} g^{1/12} Q^{1/6}} \quad (76) \quad S = 0.000547 \frac{f^{5/6}}{Q^{1/6}}$$

$$(S.V.) = \alpha 6 \cdot \frac{g^{1/3}}{\nu^{1/3}} \cdot (m \cdot XVs)^{1/3} \quad (77) \quad SV = 0.000434 f^{1/3}$$

$$\frac{b}{d} = \alpha 7 \cdot \frac{Q^{1/6} \cdot (XVs)^{1/3}}{g^{1/18} \nu^{1/6} \cdot m^{5/18}} \quad (78) \quad \frac{P}{R} = 5.65 Q^{1/3} f^{1/3}$$

$$M_1 = \alpha 8 \cdot \frac{Q^{1/3}}{g^{1/3} \nu^{1/3}} \cdot \left(\frac{XVs^{1/4}}{m} \right) \quad (79) \quad -$$

$$\frac{V^3}{gd} = \alpha 9 \cdot \left(\frac{XVs}{(g\nu)^{1/3}} \right)^{1/3} \quad (80) \quad \frac{V^3}{R} = \frac{4}{3} f$$

(Froude number)

$$\frac{Vd}{\nu} = \alpha 10 \cdot Q^{1/3} \cdot \left(\frac{g^{1/3}}{\nu^{1/12}} \right) \cdot \left(\frac{m}{XVs} \right)^{1/4} \quad (81) \quad \frac{VR}{\nu} = 0.375 Q^{1/3}$$

(Reynolds number)

CHAPTER 4
MEANDERING OF RIVERS

	Page.
4.00. Introduction	143
4.01. Analysis of factors affecting meanders	143
4.02. Field data	144
4.03. Experiments at Poona	149
4.04. Comparison of river and model results	150
4.05. Vicksburg experiments	151
4.06. Conditions giving rise to meanders	152
4.07. Adjustments to meet varying river conditions and their effect on meanders.	153
4.08. Conclusions	157



Photo 2607. Sheared bank resulting from deep scour at Dalarghat on Katjuri just downstream of Surua junction with Katjuri. Looking downstream.



Photo 2606. Deep scour at Dalarghat on Katjuri just downstream of Surua. Looking upstream.

MEANDERING OF RIVERS

CHAPTER 4

MEANDERING OF RIVERS

4.00. INTRODUCTION

In Chapter 2, the factors controlling flow in open channels were reviewed; and it was explained that though equilibrium was never fully attained in nature, there was a year to year dominant condition of discharge, charge, slope, and water temperature at which equilibrium was closely approached.

It was also explained that small variations from the dominant condition could be adjusted by bed scour when the charge was in defect, the finer grades then being washed out of the bed-mix, so that coarser grades were exposed on the bed, whereas when the charge was excessive, deposition took place, and the bed-mix became finer. When, however, variations from dominant discharge conditions become considerable, the amount of excess energy becomes so great that it cannot be dissipated in this way, and meanders develop.

The term "meander" is derived from "MAIANDROS" the Greek name of a winding river in Asia Minor.

Definitions of terms used in connection with meandering are given at the beginning of this Report.

4.01. ANALYSIS OF FACTORS AFFECTING MEANDERS

The Author had long realised the importance of meanders on river training; but it was not until 1937 that he obtained information which enabled him to write a Technical Paper on "The Relationship between Meander Belts, Distance between Meanders on Axis of Stream, Width, and Discharge of Rivers in Flood Plains and Incised Rivers."* This Paper was mainly based on data contained in a Paper written by Professor Mark Jefferson of the State Normal College, Ypsilanti, Michigan, entitled "The Limiting Widths of Meander Belts."†

The main conclusions arrived at were that M_b (Meander belt); M_l (Meander length); and W (river width) all vary as $Q^{\frac{1}{2}}$ — the square root of the discharge — and so are scalar; and the meander length is of the order $25 Q^{\frac{1}{2}}$ to $30 Q^{\frac{1}{2}}$. The Author stated:— ‡

"No formulæ have as yet been established for non-regime conditions, and the factors affecting non-regime flow, including rate of change of discharge of water; and its dependent variables — rate of scour and silt movement — are so complex as to be almost impossible to enunciate; so we are forced to adopt model experiments for solving non-regime problems.

In a channel in which the discharge varies, the channel obviously cannot alter its slope, width, and depth whenever the discharge changes; but it will tend to do so; and each channel is the integrated effect of all the various discharges which have flowed. The banks of the channels being more resistant to scour than the bed, the biggest changes take place in the bed and it is a well-established fact that after low floods, the beds of rivers rise, to be scoured out again in the next high flood. This scour is not due solely to side resistance (flume flow); but even more to the fact that silt charge

* C. B. I. A. R. (T) 1933-39, p. 49.

† National Geographic Magazine, October 1902, pp. 373-384.

‡ C. I. and H. R. S. A. R. (T) 1939-40, p. 9.

MEANDERING OF RIVERS

CHAPTER 4

MEANDERING OF RIVERS

4.00. INTRODUCTION

In Chapter 2, the factors controlling flow in open channels were reviewed; and it was explained that though equilibrium was never fully attained in nature, there was a year to year dominant condition of discharge, charge, slope, and water temperature at which equilibrium was closely approached.

It was also explained that small variations from the dominant condition could be adjusted by bed scour when the charge was in defect, the finer grades then being washed out of the bed-mix, so that coarser grades were exposed on the bed, whereas when the charge was excessive, deposition took place, and the bed-mix became finer. When, however, variations from dominant discharge conditions become considerable, the amount of excess energy becomes so great that it cannot be dissipated in this way, and meanders develop.

The term "meander" is derived from "MAIANDROS" the Greek name of a winding river in Asia Minor.

Definitions of terms used in connection with meandering are given at the beginning of this Report.

4.01. ANALYSIS OF FACTORS AFFECTING MEANDERS

The Author had long realised the importance of meanders on river training; but it was not until 1937 that he obtained information which enabled him to write a Technical Paper on "The Relationship between Meander Belts, Distance between Meanders on Axis of Stream, Width, and Discharge of Rivers in Flood Plains and Incised Rivers."* This Paper was mainly based on data contained in a Paper written by Professor Mark Jefferson of the State Normal College, Ypsilanti, Michigan, entitled "The Limiting Widths of Meander Belts."†

The main conclusions arrived at were that M_b (Meander belt); M_l (Meander length); and W (river width) all vary as $Q^{\frac{1}{2}}$ — the square root of the discharge — and so are scalar; and the meander length is of the order $25 Q^{\frac{1}{2}}$ to $30 Q^{\frac{1}{2}}$. The Author stated:— †

"No formulæ have as yet been established for non-regime conditions, and the factors affecting non-regime flow, including rate of change of discharge of water; and its dependent variables — rate of scour and silt movement — are so complex as to be almost impossible to enunciate; so we are forced to adopt model experiments for solving non-regime problems.

In a channel in which the discharge varies, the channel obviously cannot alter its slope, width, and depth whenever the discharge changes; but it will tend to do so; and each channel is the integrated effect of all the various discharges which have flowed. The banks of the channels being more resistant to scour than the bed, the biggest changes take place in the bed and it is a well-established fact that after low floods, the beds of rivers rise, to be scoured out again in the next high flood. This scour is not due solely to side resistance (flume flow); but even more to the fact that silt charge

* C. B. I. A. R. (T) 1933-39, p. 49.

† National Geographic Magazine, October 1902, pp. 373-334.

‡ C. I. and H. R. S. A. R. (T) 1939-40, p. 9.

THE BEHAVIOUR AND CONTROL OF RIVERS AND CANALS

which can be carried, increases directly as the effective velocity; and hence, as the discharge increases, the silt per cft. of water also increases.

The practical application of this to a delta is that the more the channels are allowed to split up, the less the quantity of sand that can be carried.

In channels with widely fluctuating discharges and silt charges, there is a tendency for silt to deposit at one bank and for the river to move to the other bank. This is the origin of meandering.

While the discharge of a river is increasing, there is a tendency for the meanders to increase in length and hence to erode the concave bends at their downstream ends. This cutting is highly complex in character; but there is a tendency for a river bend to move downstream, though this is often masked by cut-offs, resistant strata, etc."

4.02. FIELD DATA

The data when analysed gave the following relationships (figures in brackets are based on most likely exponent).

Rivers in Flood Plains if exponent=1.	Incised Rivers if exponent=1.
$M_1 = 6.6 W^{0.99}$ (=6.06 W)	$M_1 = 2.06 Q^{1.27}$ (=11.45 W)
$M_b = 18.65 W^{0.99}$ (=17.38 W)	$M_b = 21.4 W^{1.04}$ (=27.3 W)
$M_b = 1.7 M_1^{1.06}$ (=2.86 M_1)	$M_b = 1.19 M_1^{1.07}$ (=2.2 M_1)

(Note.—Ferguson in a Paper published in 1863 on "The Delta of the Ganges" concluded that for river gradients between 6" and 1 ft. per mile, the oscillation is one in about 6 times the width, thus anticipating the Author's conclusion by nearly 80 years !)

Unfortunately little discharge data were available; but what was available gave the following rough relations:—

(a) Rivers on Flood Plains.	(b) Incised Rivers.
$W = 4.88 Q^{\frac{1}{2}}$ (very roughly)	$W = 5.12 Q^{0.43}$ (= 2.5 $Q^{\frac{1}{2}}$)
$M_1 = 6.46 W$ (=29.6 $Q^{\frac{1}{2}}$)	$M_1 = 17.8 Q^{0.53}$ (=25.4 $Q^{\frac{1}{2}}$)
$M_b = 57.8 Q^{0.54}$ (=84.7 $Q^{\frac{1}{2}}$)	$M_b = 29.4 Q^{0.35}$ (=56.4 $Q^{\frac{1}{2}}$)

In June 1939, Dr. Robert A. Bates wrote a Paper* in which he recorded data showing relation between width of stream W and width of

* Geomorphic History of Kickapoo Region, Wisconsin, Bulletin 50, pp. 319-380. Geological Society of America.

FIG. 4-1 GRAPH SHOWING RELATION BETWEEN
WIDTH OF STREAM (W) & WIDTH OF MEANDER BELT (M_b)
INFLOOD PLAINS.

(FIG. 6 & 7 FROM A.R.T. 1939-40 OF C.I. & H.R. STN)

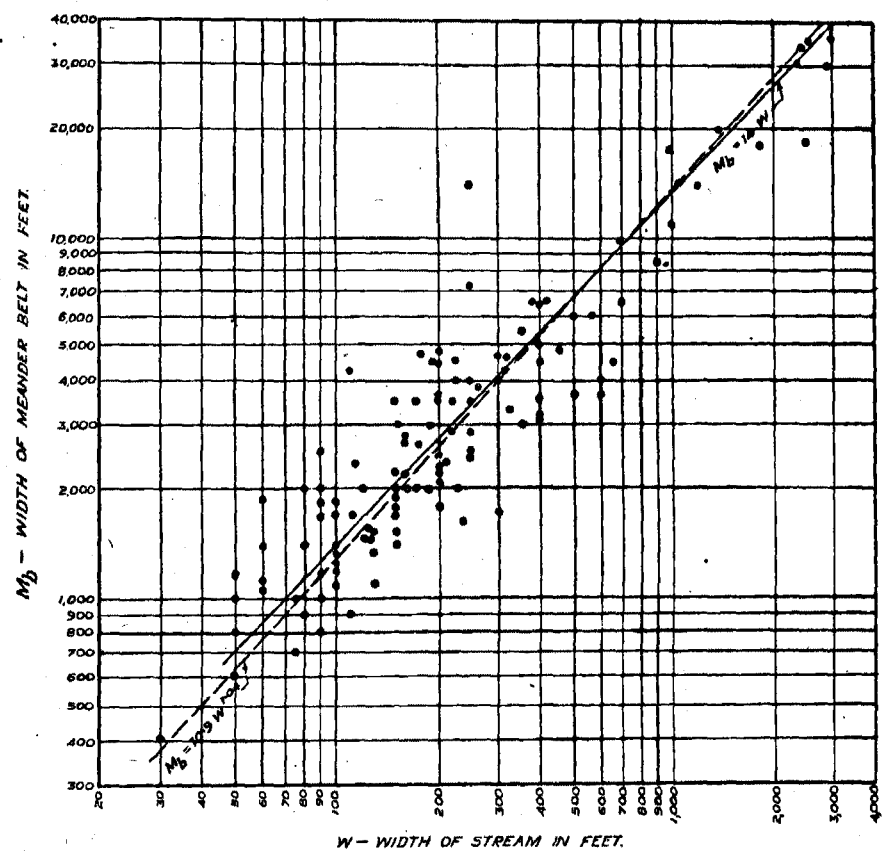
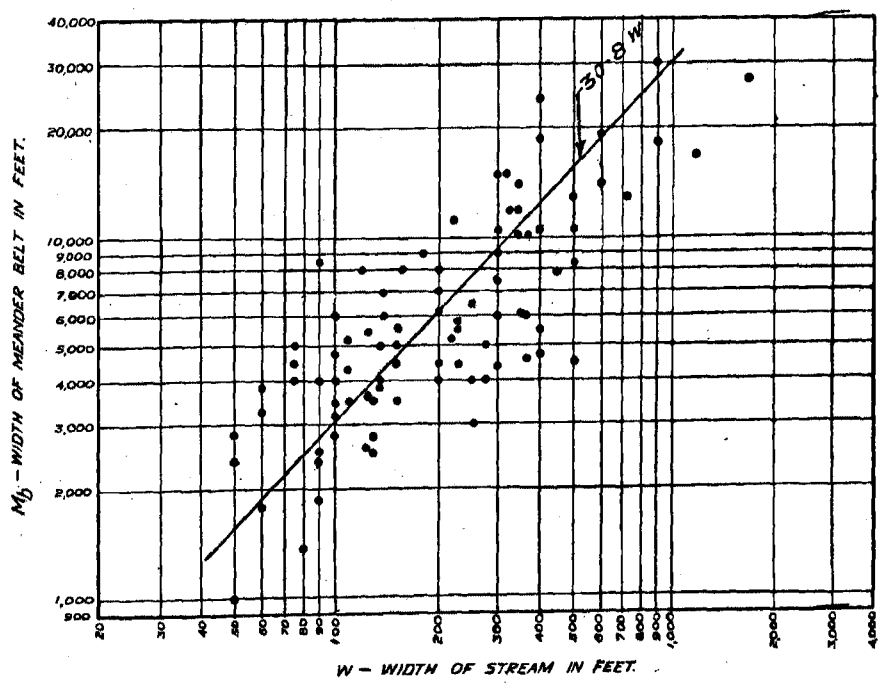


FIG. 4-2 GRAPH SHOWING RELATION BETWEEN
WIDTH OF STREAM (W) & WIDTH OF MEANDER BELT (M_b)
INCISED STREAMS.



MEANDERING OF RIVERS.

meander belt for rivers in flood plains—Fig. 4-1—and incised rivers—Fig. 4-2. This data gave by the method of least squares applied to logarithmic plotting

Rivers in Flood Plains.	Incised Rivers.
$M_b = 10.9 W^{1.04} (=14 W)$	$M_b = 30.8 W$ (Based on assumption that $M \propto W$, very roughly).

Even though the discharge data of this analysis were scanty, the results were suggestive, in that though—as was to be expected—the widths of rivers in alluvial plains were much greater than the widths of incised rivers for the same discharges— $W = 2.5 Q^{\frac{1}{2}}$ —the value for incised rivers approximating closely to Lacey's regime value of $W = 2.67 Q^{\frac{1}{2}}$ —the meander lengths and breadths were of the same order for the same discharges in rivers in flood plains as in incised rivers; but scatter was naturally much greater in incised rivers.

Meander data collected during 1938-39 by Mr. J. Shaw, Executive Engineer, Flood and Drainage Division, Orissa, for 16 Orissa Rivers on flood plains gave,

$$M_l = 27.4 Q^{\frac{1}{2}} + 14.33$$

which was of the same order as the meander length of the earlier American data.

4.03. EXPERIMENTS AT POONA.

Working on the assumption that if the gradient of a straight channel varied from steep at the upper end to flat at the lower end, there must be a point at which the discharge and charge would produce meanders, this assumption was tested out early in 1940, at Poona; and, as anticipated, meanders resulted. See Photos 2769 and 2770, Sheet 4-1. Much work has since been done at Poona in an effort to establish quantitative relationships, but a continuous investigation was ruled out due to the pressure of more urgent work.

In 1942, a Note was written by the Author based on experiments carried out from time to time during the previous two years, entitled "Factors affecting Meanders of Channels."†

In this it was stated:—

"A channel is in equilibrium when the discharge is just sufficient to carry the charge of bed material with the slope available. If the slope is in excess of that required to carry the charge, scour occurs, and this scour increases the charge which then becomes excessive for the slope further downstream and secondary meandering occurs.

It may be asked regarding "secondary meandering" if, as stated, it is due to scour increasing the charge then surely there must be a point where the charge is just what the channel can carry with the slope and discharge available; so why doesn't it remain stable below that point instead of developing a super-charge lower down?

The explanation appears to be that scour due to a sub-equilibrium charge is always accompanied by a rising bed (i.e. depth decreasing downstream) so, though there must be a point where the charge can just be carried, this does not persist, but conditions pass from a sub-equilibrium charge for a deeper channel to a super-charge in a shallower channel.

* C. I. and H. R. S. A. R. (T) 1939-40, p. 11.

† C. I. & H. R. S. A. R. (T) 1942-43 pp. 51-55.

THE BEHAVIOUR AND CONTROL OF RIVERS AND CANALS

Primary meandering, on the other hand, is the direct result of an excess charge being washed into the channel.

“It may be wondered why so much importance is attached to distinguishing between primary and secondary meandering. The reason for this will be apparent later; but it is obvious that at least in the case of model experiments it is of prime importance to distinguish between them; because in one case, the charge being injected is excessive, while in the other case, it is less than the equilibrium charge.”

“COMPARISON OF MEANDER LENGTH AND MEANDER BELT IN RIVERS AND SMALL CHANNELS

TABLE 4-I (Experimental Results)

Model Expt. No.	Q cusecs.	N = approx. charge moving along bed in parts per 1,000 by weight.	C_1 in $M_l = C_1 \sqrt{Q}$	C_b in $M_b = C_b \sqrt{Q}$
1	0.175	Not observed.	34.45	16.6
4	0.175	0.269	30.02	16.55
2	0.175	0.252	36.56	17.0
3	0.175	0.178	34.45	11.47
5	0.40	0.540	37.4	13.9
Mean of experiments 1 to 5			36.4	16.0
6	0.10	51

Where C_1 = coefficient of meander length in formula $M_l = C_1 \sqrt{Q}$
 C_b = coefficient of meander belt in formula $M_b = C_b \sqrt{Q}$

The high meander length coefficient with .1 cusec probably indicated an approximation to limiting conditions.”

4.04. COMPARISON OF RIVER AND MODEL RESULTS

The coefficient of 36 in the model for constant discharge when compared with 28 based on maximum discharge in rivers with varying discharges indicated that the dominant discharge in rivers is of the order of 60 per cent of the maximum discharge.

This is in agreement with field experience, which indicates that the dominant discharge is usually of that order, though it varies in different rivers, sometimes being as low as, or even less than, half. In canal channels, the discharge of which is maintained constant, the dominant discharge is little less than the full supply discharge.

“When the discharge varies, there is a marked tendency for a channel to alter its meander length to fit the discharge; so where the meander is “held” at a point — as at the Railway Bridge at Delhi — the meander length varies considerably during the flood season and from year to year.”

MEANDERING OF RIVERS

4.05. VICKSBURG EXPERIMENTS

Recently, Captain J. F. Friedkin of the U. S. Corps of Engineers has written an extremely valuable Paper for the Mississippi River Commission entitled "A Laboratory Study of the Meandering of Alluvial Rivers."*

Under "Basic Principles of Meandering" he writes:—

"Meandering results from local bank erosion and consequent local overloading and deposition by the river of the heavier sediments which move along the bed."

This corresponds with the Author's secondary meandering; but meandering can also be produced in a straight channel by injecting a charge which is in excess of the natural equilibrium charge for the conditions, and meanders will develop even where the sides are inerodible, as in an incised river and flumes with rigid sides.

The essential difference between primary and secondary meandering is that in the former case the excess charge from upstream crosses to the convex bank; whereas in secondary meandering the excess charge is developed in the meander embayment due to bank erosion increasing the charge until a point is reached at the downstream end of the embayment where the bank-side velocity drops—due to the channel straightening out—so that the excess charge cannot be carried forward, and deposits as a shoal along the same bank.

In Captain Friedkin's Paper the effect of imposing various entry conditions on meanders is dealt with in detail; but the Paper does not set out to indicate which of the conditions imposed is the most normal; but rather to demonstrate the effects of abnormalities found in rivers—particularly in the Mississippi. Captain Friedkin's paper is, therefore, complementary to the work done at Poona, which was designed to determine normality—or rather "modality"—in meandering. This difference is essentially the same as the differences between Lacey's work dealing with regime, and formulæ to allow for divergences from regime.†

This excellent Paper will repay very close study; and the Author is in complete agreement with the experimental results. He does not, however, agree with all the conclusions drawn from the results. Thus, the statement already quoted:—

"Meandering results from local bank erosion and consequent local overloading and deposition by the river of the heavier sediments which move along the bed."

overlooks the fact that bank erosion is itself an effect—i.e. is essentially a dependent, not an independent, variable; and—as already stated—meandering can be produced by injecting a particular charge into a channel even though it has rigid sides.

Again it is stated:—

"The radii of bends increase with increase in discharge or slope for the same alignment of flow into the bends."

Though this statement is quite correct, "alignment of flow has been treated as an independent variable, and enforced in the experiments; whereas in alluvial channels it is a dependent variable; so, though it is correct to say that the radii of meanders increase with the increase of discharge— $M \propto Q^{\frac{1}{2}}$ —"the radii of bends" do not normally increase with increase in slope—except temporarily and locally. Here again, Captain Friedkin has treated the dependent variable—slope—as though it were an independent variable, like discharge.

* Experiments carried out at U. S. Waterways Experiment Station, at Vicksburg (1942-44).

† See Chapter 3.

THE BEHAVIOUR AND CONTROL OF RIVERS AND CANALS

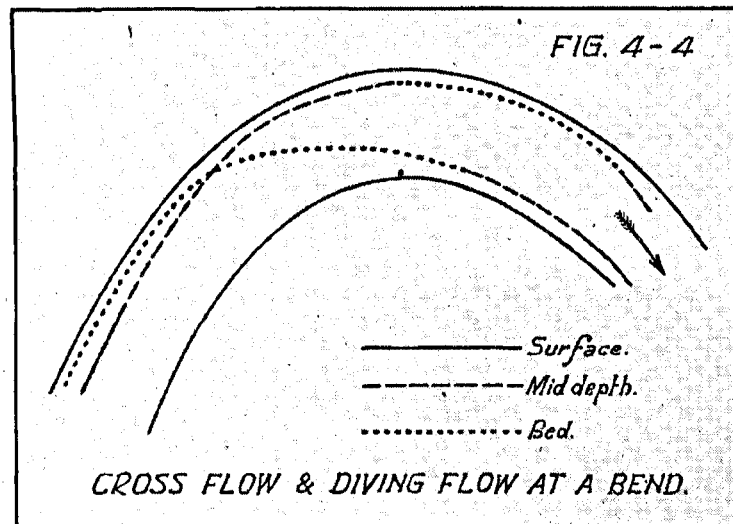
Subject, therefore, to the proviso that the Paper should be read bearing in mind the essential difference between dependent and independent variables, and that divergence from "modal" meanders occurs due to variations from dominant conditions, Captain Friedkin's Paper brings out in a most satisfactory way how meanders are affected by divergences from dominant conditions.

4.06. CONDITIONS GIVING RISE TO MEANDERS

Where there is a heavy load of bed-material in movement, during floods, excess turbulent energy is developed.

Such a condition is unnatural, and any aberration from uniform axial flow across the section tends to cause slightly more flow to concentrate towards one bank or the other. Additional flow is then attracted towards that bank—for reasons analogous to the development of streamlines towards an aerofoil. Under such conditions, outward flow is checked by the bank, part of the kinetic energy due to the velocity of the outward-flowing water being transformed into pressure head, which, in turn, causes bed-water to be deflected towards the inner bank, Fig. 4-4.

Fig. 4-4



This leads to deposition along the inner bank; whilst curved, diving, flow develops along the outer bank, accompanied by forced vortex conditions. High peripheral velocities and surges then occur near the toe of the bank, which leads to undermining, followed by large lumps of the face shearing off. This condition is illustrated in Photo Sheet 4-2. Such slips occur to a marked extent when the discharge—and hence the water level in the river—is falling; because the banks are then saturated. It thus happens that even where the banks are scour-resistant, meander embayments continue to expand, though much more gradually than where the banks are incoherent, until the meander reaches a relatively stable curvature depending primarily on the dominant discharge.

Photo Sheet
4-2

MEANDERING OF RIVERS

4.07. ADJUSTMENTS TO MEET VARYING RIVER CONDITIONS AND THEIR EFFECT ON MEANDERS

Where large-scale adjustments are in progress, enormous quantities of material are eroded from embayments, to be carried downstream to form shoals (see Photos 2769 and 2770, Sheet 4-1) but though movement of material is rapid during bank erosion, so that it is often spectacular, the distance the material is moved is not great, and it may be many years before the same material is brought into motion again; so overall movement is of the same order as in the straighter reaches of the channel.

Photo Sheet
4-1

Bed movement is essentially a progression, resulting from continuous 'trading' throughout the river. This 'trading' is not continuous, the river bed tending to build up in some parts and scour in others; but, over a considerable period of time, movement is general, the finer particles moving more quickly and in greater quantity than the coarser particles; so that there is a progressive diminution in size of particles down a river. Despite this change of grade, the weighted mean change is determined throughout the river by the material entering the river; and bank erosion and meandering are also controlled by this charge and the discharge available to carry it.

It might be thought that a stage would be reached when a year to year stability would occur. To a certain extent this is so—when a reach of river attains its maximum length, the average slope then being a minimum. Conditions may then remain relatively steady for some years; but no final stability is reached—partly because the discharge and charge vary during each year, and from year to year; but chiefly because changes occur in the river upstream and downstream—which upset the temporary balance.

At this point it is necessary to return to the simplifying concept of a 'dominant discharge' at which equilibrium is most closely approached. The discharge will then carry the charge through the 'modal' meanders without appreciable erosion or change. For a discharge slightly in excess of the dominant discharge there will be a tendency for the meanders to lengthen and for flow to become straighter across the convex bends. At dominant discharge—which in many rivers is a little higher than bank-full stage—spill over the banks is considerable, and continues to increase as the discharge increases; so, when this stage is reached, the Lacey number— V^2/gR —decreases in the relatively straight length of the channel at 'the crossings' where accretion takes place; but in the meander embayments, scour may continue.

River adjustments vary widely accordingly as the bed of the river consists of boulders, pebbles, sand, or silt.

Near the hills, where the river bed consists of a mixture of boulders pebbles, and sand, the slope is steep, and excess energy is dissipated by a series of natural diagonal boulder-weirs, which act as rapids when the discharge is low. Rivers of this type are generally split into two or more channels.

The Sarda River upstream of the Barrage at Banbassa, where it forms the boundary between the United Provinces and Nepal, is a good example of this type of river.

Fig. 4-5

In such cases, spill takes place over shoals during small freshets; and, with larger discharges, over the islands. During high floods the shoals get scoured, so are prevented from building up; but new channels, shoals, and islands are continually developing, the older channels tending to fill up, and the older shoals to erode. Under such conditions small local embayments, developed by part of the total discharge, occur; but there is no meandering of the river as a whole.

Downstream of the boulder belt, which ends abruptly, the material exposed on the river bed is sandy, but the slope is still steep, and the river is wide and shallow. Here, also, spill takes place over the shoals, even with relatively low discharges; so, though the shoals tend to build up, they are worn down again during floods; and in extreme cases, no permanent river channel persists, but only a number of interweaving channels, which are continually changing.

An extreme case of this is exemplified by the Kosi in South Nepal and North Bihar,* so aptly described in "Tent Life in Tigerland" by J. Inglis (1892).

"When swollen by the melting of the snows or by the annual rains, the river overflows its banks, and at such times presents the appearance of a broad swiftly flowing sea, for its breadth from bank to bank is often ten and in some places nearly twenty miles across. In the dry season, the waters—always of the same milky hue—are confined to innumerable channels, some so shallow that the stilted plover can wade across, and others running deep and strong with a ceaseless gurgling swish that would sweep the stateliest elephant off its feet; and carry its ponderous bulk far down the stream. These streams seem to run at random over this deltaic plain, diverging here, re-uniting there, forming a wide bend in one place, and cutting direct through the sandy soil in another. The face of the country is split up into an infinitude of islands, and reticulated everywhere by a net-work of dry channels and shifting sand banks."

In the lower portions of rivers—far from the hills—the material of the banks is relatively tough, so meanders develop more slowly and more fully than near the hills. This is because the by-channels do not open up until the meanders have reached an advanced stage of development—for three reasons:—

(a) the depth of spill relative to the depth of water in the river channel is small, i.e. the slope required for the overall spill discharge is high, whereas the slope available round the bend in the river is low.

(b) the material comprising the high ground over which spill occurs is relatively tough, and hence resists scour, and

(c) the charge of silt in the spill water when the meander is developing is considerable.

For these three reasons, by-channels—which develop by cutting backwards from the point where the overflow reaches the lower loop—do not develop into a cut-off until the flow in the by-channel exceeds a critical discharge for the slope available; which, in turn, is affected by the distance between adjoining loops—a distance which gradually decreases as the meander loop develops.

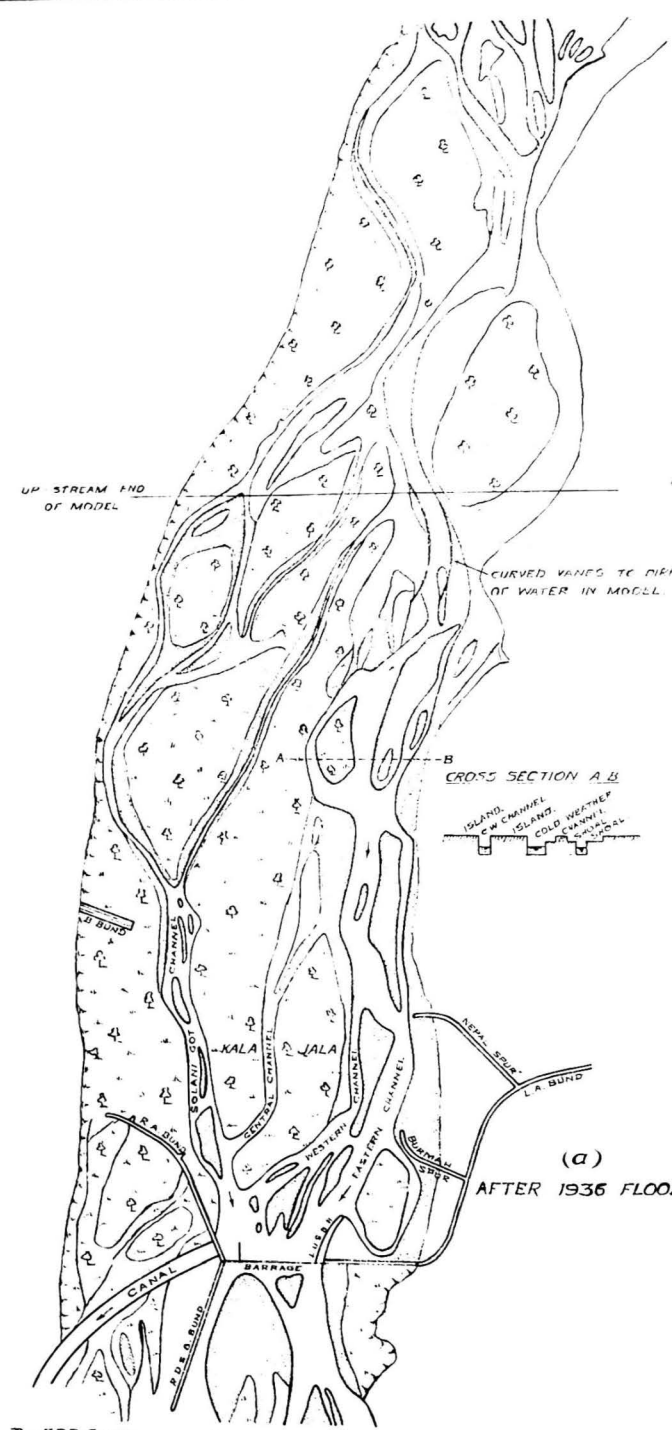
When conditions reach the stage at which development of the cut-off is rapid; a heavy supercharge of material passes out through the downstream end of the developing cut-off, part of which deposits downstream. This affects the angle of flow and tends to cause a change in the direction of meander development downstream of the cut-off; but the effect on conditions at the upper end of the cut-off—where there is draw-down—is much greater than downstream; and the slope in the river increases considerably, setting free excess energy. Conditions are then favourable for the initiation of new meanders or the development of existing meanders. Unless, therefore, steps are taken to utilise or dissipate the energy thus set free, the river will again begin to lengthen its course.

In many cases, full development of the meander is attained before a cut-off occurs; and each river has a critical 'arc to chord' meander ratio which, if exceeded, generally leads to a cut-off occurring.

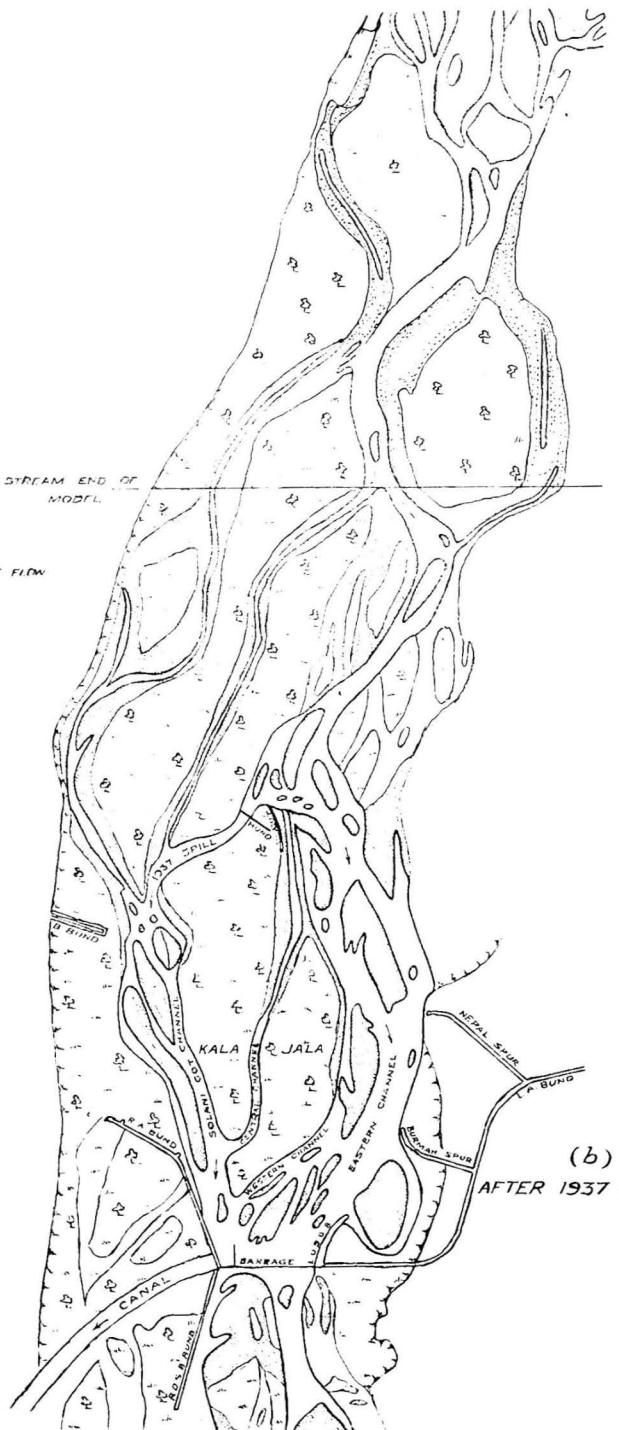
The cut-off ratio has been stated to be 1.7 for Punjab Rivers. It is 2.0 or a little more in the Indus in Sind.

* C. I. and H. E. S. A. R. (T) 1941-42, pp. 13-15.

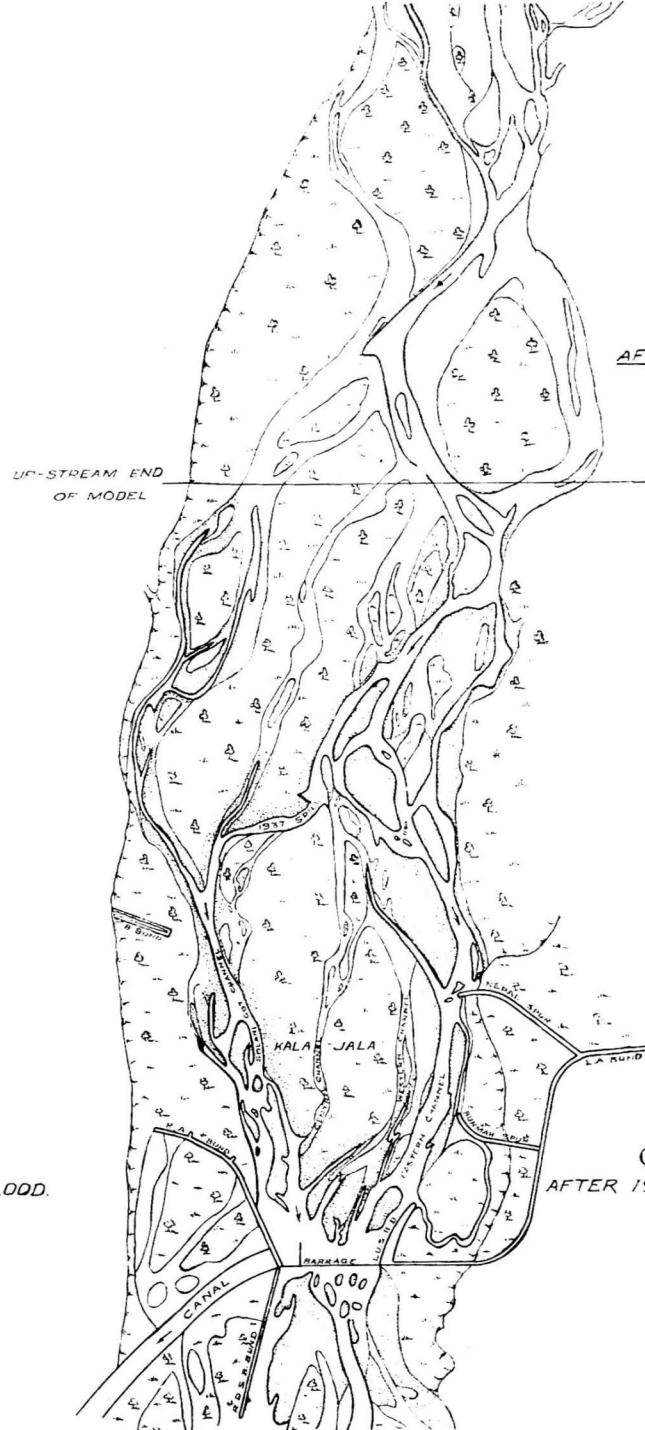
COLD WEATHER SURVEY
 OF
 THE SARDA RIVER
 AT
 BANBASSA (U.P.)
 AFTER FLOODS OF 1936, 1937 & 1938.
 SCALE 1 INCH = 1/2 MILE.



(a)
 AFTER 1936 FLOOD.



(b)
 AFTER 1937 FLOOD.



(c)
 AFTER 1938 FLOOD.

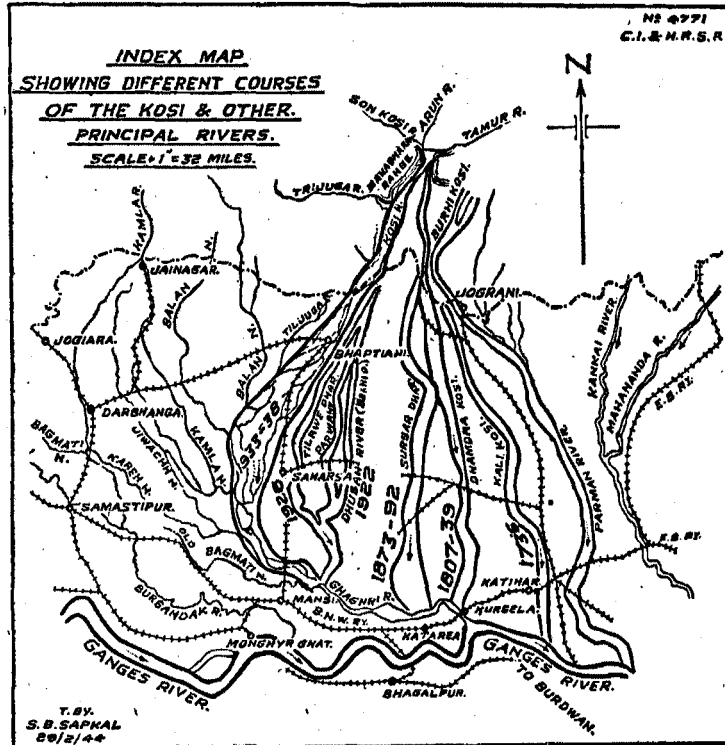


MEANDERING OF RIVERS

In the extreme case of an incised river, there may be no spill. In such a case a cut-off can only occur if adjoining loops cut through the intervening ridge.

Where incised rivers have developed in hard materials by abrasion spread over æons of time, a similar development may occur; but full development is much rarer in such cases than in quickly-erodible material.

Thus we see that every condition occurs in nature, from a river like the Kosi (Fig. 4-6) which has shifted as much as 12 miles in a single year, Fig. 4-6



(FIG. 5 FROM A. R. T. OF I. W. E. STN.)

and 52 miles in 50 years—leaving behind it blankets of silt as thick as 12 feet—to incised rivers with meander belts of maximum width which have developed during enormous periods of time, sometimes with dominant discharges far in excess of those of to-day.

4.08. CONCLUSIONS

In alluvial rivers building up their valleys, 'modal' meander length—i.e. the most usual meander length—depends almost entirely on the dominant discharge and—for the same weighted charge—is little, if at all, affected by the grade of the material in movement. There is, however, a limiting minimum discharge for each material, and the relation is affected when the velocity approaches the limiting velocity. The normal relation is

$$M_1 = 36 \sqrt{Q_{dom}}$$

but the dominant discharge of alluvial rivers in India is rarely known and only very roughly determinable from bank-full stage; so meander

length has generally to be based on maximum discharge. The best fit may be taken as

$$M_1 = 27 \sqrt{Q_{\max}}$$

Much less is known about the factors determining meander belt than those determining meander length, and variations are much greater; but it appears that a charge of fine material tends to increase the meander belt—if not directly, at least indirectly, due to the banks then being tough, so that resistance to downstream progression is greater.

Where the slope of the country exceeds the natural slope for the dominant discharge, the river tends to develop a high degree of tortuosity to absorb excess energy.

Normally, the grade of material exposed in a river varies from coarse near the hills, to very fine near the sea; but owing to variations in the material along the flanks of the valley and to different grades and types of material washed into the main river from tributaries, there may be reaches in which the material is locally coarser than further upstream. In such cases, the finer material tends to be swept through the coarser reach, which may be relatively straight.

Meandering is Nature's way of damping out excess energy during a wide range of varying flow conditions, the pattern depending on the grade of material, the relation between discharge and charge, and the rate of change of discharge and charge.

BEHAVIOUR OF RIVERS

CHAPTER 5

BEHAVIOUR OF RIVERS—AS EXEMPLIFIED BY THE
INDUS IN SIND

	Page.
5.00 Introduction	167
5.01 Historical survey of data collected and steps taken to control the Indus in Sind	167
5.02 The causes of floods in Sind	168
5.03 Factors determining the rate of rise of flood levels in rivers—	169
(a) Deposition of silt brought down by a river	169
(b) Isostatic depression	169
(c) Eustatic rise of sea level	170
5.04 The combined effect of deposition, isostatic depression, and eustatic rise	170
5.05 Effect of seaward extension of deltas, and recession of valleys	171
5.06 Effect of heading up water at a Barrage on water levels upstream	172
5.07 Analysis of data regarding rise of flood levels in the past	172
5.08 Trends in maximum annual Gauge readings at Bukkur	177
5.09 Trends in the relation between specific discharge and corresponding Gauge readings downstream of Sukkur Barrage	178
5.10 Conclusions regarding the future rise of water levels resulting from heading up at Sukkur Barrage	182
5.11 Enquiry into the effect of constructing the Right Bank Approach Channel at Sukkur on the rate of rise of water levels in the Indus	182
5.12 Progressive accretion in the Gorge three miles upstream of the Barrage, between Rohri and Bukkur Island	184
5.13 Progressive change in area of waterway downstream of Sukkur Barrage	184
5.14 Effect of Approach Channel, on rising trend of specific discharge Gauge readings	189
5.15 Short period variations in the relation between specific discharges and Gauge readings	189
5.16 Progressive reduction of silt charge and load—	189
(i) Trends in silt load and charge, related to time	189
(ii) Trends in silt load relative to discharge	189
5.17 Can anything be done to retard deterioration of a river resulting from heading up water at a Barrage?	201
5.18 Effect of reduction of discharge and charge in a river resulting from water and bed material being impounded in a large storage reservoir in the hills	201
5.19 The effect of the proposed Upper Sind Barrage on water levels lower down the river	203
5.20 Effect of ponding at Sukkur Barrage on water levels at a distance upstream...	203
5.21 Cause of reduction of silt in suspension at Sukkur and Kotri	203
5.22 Conclusions	204

FIG: 5-1
 SHOWING EXISTING AND ABANDONED
 BUNDS & MAXIMUM SWING OF DEEP CHANNELS
 OF INDUS.

1911 TO 1942

SCALE: 1" = 16 MILES.

NO: 4862
 CENTRAL INTELLIGENCE & SURVEY
 OFFICE
 (FIG. 8 FROM A.R.T.
 1943 OF I.W.E. ST.)

REFERENCES:

- MAXIMUM LIMIT OF SWING OF FLOW -----
- EXISTING BUNDS _____
- ABANDONED BUNDS - - - - -
- RAILWAY LINE

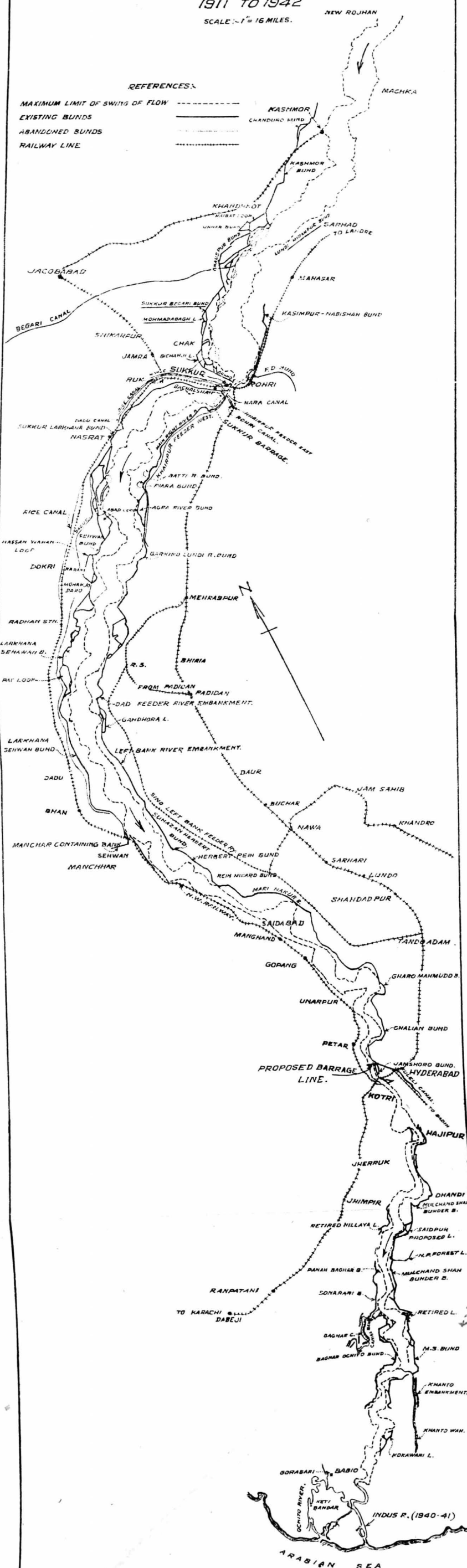
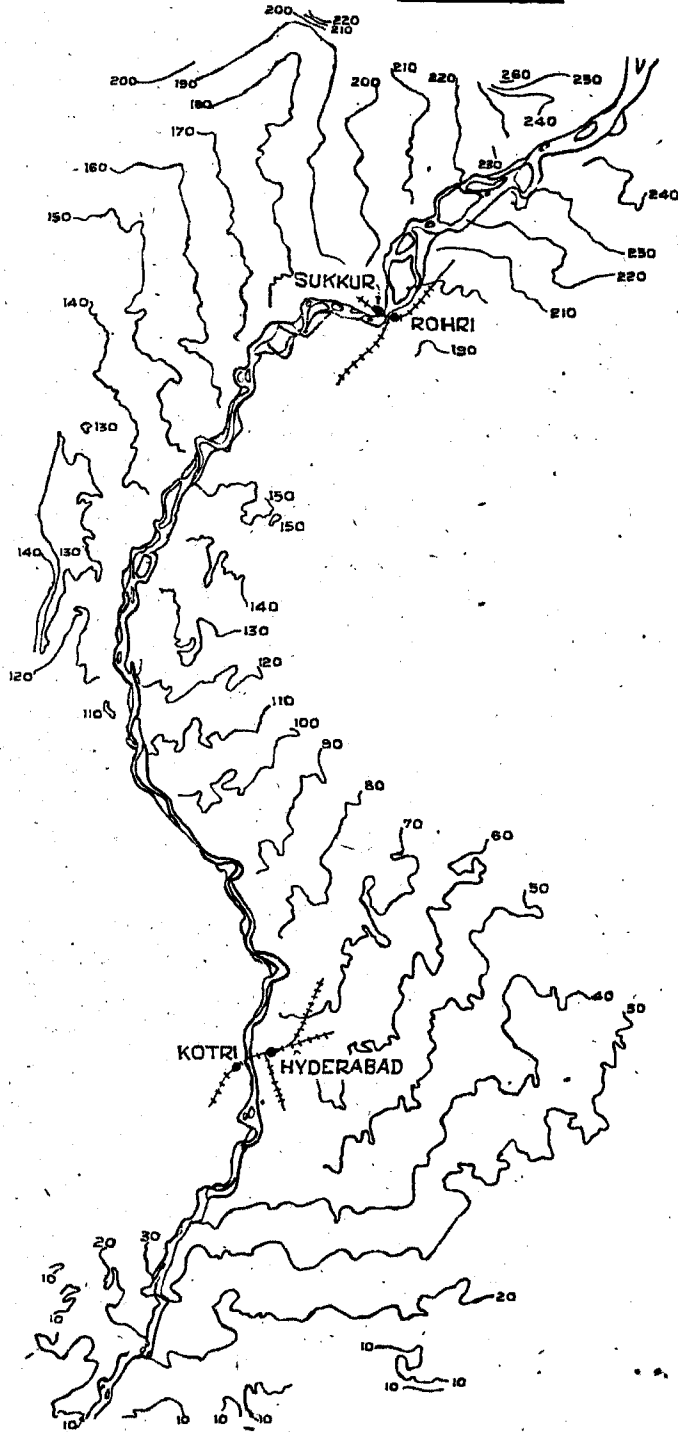


FIG: 5-4
SHOWING CONTOURS OF
THE WHOLE OF SIND
(EXCEPT KHAIRPUR.)

No: 4734
CENTRAL IRRIGATION &
HYDRODYNAMIC RESEARCH STA.
POONA.

(FIG. II. FROM A.R.T.
1943 OF I. W. E. S.)

SCALE: 1:32 MILES.



BY: PARKHI, P.D.
7-2-44

G. P. S. P. POONA, 29.

CHAPTER 5

THE BEHAVIOUR OF RIVERS—AS EXEMPLIFIED
BY THE INDUS IN SIND

5.00. INTRODUCTION

Having outlined the factors affecting river behaviour in the preceding chapters, their practical application to the case of a specific river—the Indus—will now be considered.

The Indus in Sind has been selected because much more information is available about it than any other river in India.

5.01. HISTORICAL SURVEY OF DATA COLLECTED AND STEPS
TAKEN TO CONTROL THE INDUS IN SIND

Fig. 5-1

Fig. 5-1 is an index plan of the Indus in Sind, on which existing and abandoned embankments, and the maximum recorded limits of swing of the river, are shown.

Gauge readings have been observed at Bukkur since 26th January 1848 and at Kotri since 20th April 1863. Thirteen additional Gauge Stations have since been added.

Surveys of portions of the River were started in 1900. Annual surveys of the River have been carried out since 1911, but they were published only for alternate years (in the Indus River Commission Records) until 1930-31; since when annual surveys have been printed.

From 1st September 1887, gauge readings observed in the Punjab have been reported to Sind by wire or letter, and since 1931, discharges at Sarhad, near the Punjab Sind border, have been computed.

Fig. 5-2
Fig. 5-3

Since 1931, registers have been maintained showing water levels at one mile intervals along all bunds, from which maximum recorded water levels have been shown in Figs. 5-2 and 5-3, for the Sukkur-Begari and Kasimpur-Nabishah Bunds.

The charge of suspended silt passing down the river has been observed regularly at Sukkur and Kotri since 1902.

Current-meter discharges began to be observed at intervals from 1899 at Kotri and 1901 at Sukkur. From 1st June 1921 they have been observed regularly at Kotri and Sukkur.

Prior to 1919, the meters were rated in natural ponds from a boat which was moved with constant velocity. From 1919 to 1921 they were sent to Madras for rating, and from 1922, a meter rating station was started at Kotri.

From the early days of bund construction, orders were issued from time to time regarding methods of constructing and maintaining bunds; but, to a large extent, knowledge was passed on by experienced staff from one generation to another.

After the Adurtakio Breach of the Sukkur-Begari Bund in 1930, the Indus River Commissioners recommended the preparation of a comprehensive outline of bund construction and maintenance, known as the Bund Manual. This comprises orders and circulars in force at that time, combined with a selection of recommendation made by officers experienced in Bund work.

No such valuable Manual has been prepared in any other Province in India.

THE BEHAVIOUR AND CONTROL OF RIVERS AND CANALS

So far as is known, the only bunds in existence prior to 1861 were Zamindari bunds. These were improved and new bunds built as shown below:—

Kasmor Bund (Miles 19 to 38)	probably completed in 1875.
(Miles 38 to 50)	„ „ „ 1887.
Kasimpur-Nabishah Bund	„ „ „ 1875.
Sukkur-Begari Bund	„ „ „ 1878.

Other river bunds were subsequently constructed, as shown in Fig. 5-1.

Many bunds were constructed in the nineties of last century, and the remaining gaps were mostly closed during the construction of the Sukkur Barrage Canals.

Both banks are now completely embanked from the northern border of Sind to the sea except where high land renders this unnecessary.

5.02. THE CAUSES OF FLOODS IN SIND

The basic reason for floods in Sind is that the Indus is an alluvial river which is gradually building up its bed on a flood plain of its own making; so the country slopes away from the river on both banks—see Fig. 5-4, showing contours of Sind.

Before the bunds were constructed, spill used to occur annually, flooding large areas. The flood of 1861, prior to construction of the Kashmor and Sukkur-Begari Bunds, was a good example. In that year—to quote Captain Frederick Phillips' Report*—

“Two flood streams burst over the country from the Indus. The first from above Kashmor, which after flooding vast tracts in the Jacobabad district, passed on to the south into the Shikarpur taluka, meeting the second flood stream from a place called “Moromaree” close to the mouth, and north of the Sind Canal. Water five miles in breadth and varying from 5 ft. to 3 ft. then swept across the country in a S. S. E. direction.”

It was in order to prevent such floods that embankments were built. Fig. 5-1 shows, in addition to the bunds, the maximum swing of the deep channel of the Indus between 1911 and 1942.

It will be seen that during the last 30 years the main channel of the river has swung backwards and forwards over a wide belt, the period between successive swings to the same bank being of the order of 35 years. The swing is about 4 miles, where the Indus enters Sind; and increases to 6 miles, some 8 miles upstream of Sarhad. Just upstream of the Barrage the maximum swing is much greater—nearly 12 miles. This is natural above a gorge; because where a river is controlled by a fixed exit, the full meander belt tends to occur on both sides of the axis of the river; and hence tends to be twice that where the exit is not fixed. In this case, however, the swing to the east has been restricted by submerged rock along the left bank.

For 20 miles downstream of the Barrage, the swing of the Indus has varied in width between 4 and 6 miles; but further downstream it gradually narrows, till at Kotri it does not vary appreciably and is restricted to about half a mile. Downstream of that, it remains much narrower than above Kotri.

* Quoted by N. M. Billimoria in the Sind Gazette.

BEHAVIOUR OF RIVERS

The sack-like, wide, portions of the river, shown on the plan near Saidabad and upstream of Hajipur, are due to the river having followed alternative courses near the right and left banks respectively. There are many projecting salients all down the river which have not been attacked in the last 30 year period; but these will disappear in the course of time. There is, however, little reason to anticipate any considerable extension of the maximum swing of the river except for some miles above Sukkur.

At the time the bunds were built, few, if any, doubted they would be highly beneficial; but bunds prevent spill, so where river levels rise while ground levels outside the bunds remain the same, the head against the bunds increases; so it becomes more and more difficult to prevent breaches, or to close them when they occur. Thus, though there is normally less flooding yet, when breaches occur, the damage is greater.

As a result, there is a widespread belief in India that bunds increase the rate of rise of water level in a river.*

5.03. FACTORS DETERMINING THE RATE OF RISE OF FLOOD LEVELS IN RIVERS

Before considering the data, three long-term factors, which are generally ignored, will be considered:

- (a) Delta extension resulting from deposition of silt brought down by a river.
- (b) Isostatic depression.
- (c) Eustatic rise of sea-level.

(a) *Delta extension resulting from deposition of silt brought down by the river*

The quantity of silt carried by the Indus is very great, and were there no counteracting influences at work, the delta would be extending seawards much more rapidly than it actually is; but the effect of deposition is intimately connected with isostatic depression and eustatic rise, so these will be described before their combined effect is considered.

(b) *Isostatic depression*

The effect of isostatic equilibrium has been explained in a Paper by Russell and Fisk—"Isostatic effects of Mississippi River delta sedimentation."[†]

"The daily load of sediment deposited in the lower delta-region of the Mississippi River averages about two million tons. If distributed over such depths as exist in the vicinity of the river's mouth this amount of inorganic material should cause an accretion of about 30 square miles of land each year. In fact the accumulation of locally derived organic materials should accelerate the gain far beyond that rate. A comparison of surveys about a century apart, however, indicates not the addition of 3,000 or more square miles of land surface in southern Louisiana but neither appreciable gain nor loss."

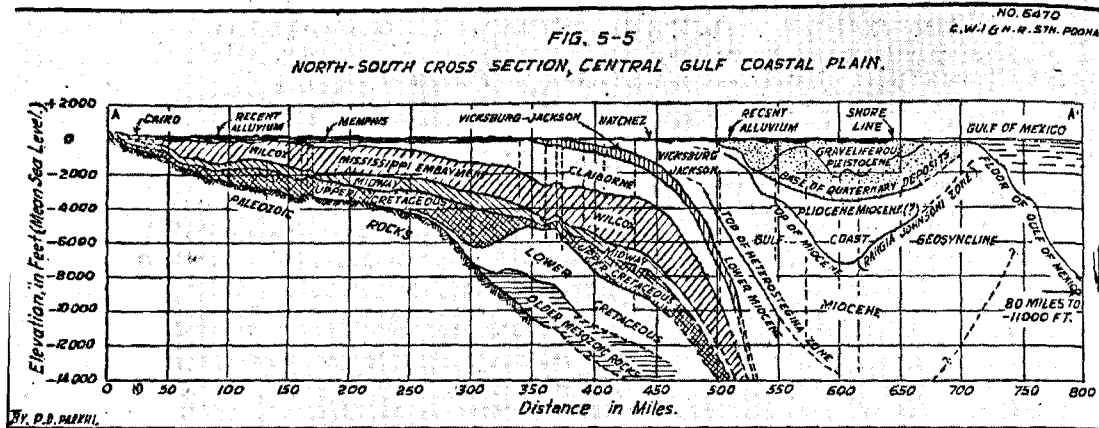
A recent Paper entitled "New Project for Stabilising and deepening Lower Mississippi River"[‡] is also of interest in this connection: Fig. 5-5—derived from that Paper—shows the enormous depressing effect of the superimposed load of sediment.

Fig. 5-5. ‡

* "Selections from the Records of the Bengal Government relating to the Damodar Flood Control Project." Vol. IV, 1926.

† E. J. Russell and H. M. Fisk—presented at the Joint Meeting of the International Association of Geodesy, Oceanography, Meteorology and Hydrology. Washington, D. C., September 12th 1939.

‡ Charles Senior M. A. Soc. C. E. Proc. February 1946.



(c) Eustatic rise of sea level relative to the land

Mr. Day Kimball has kindly contributed a Note on this question, in which he says:

“The Geological evidence seems unambiguous that, 20,000 years ago, the general sea level was substantially lower than it is to-day. The obvious (and for once probably also the true) explanation is the fact that at that time vast quantities of water were still locked up in the last of the great Pleistocene ice sheets. My own estimate of the order of magnitude involved is 100±50 feet below present level.

We have good reason to think that world sea level did not get completely back to its present level until, say 3,000 B. C. + 500 years; although, of course, the rise was much more rapid at first than at the end; and the last 10 or 20 feet may have taken several thousand years. This period of rising sea-level constitutes the last phase of what is usually called the “Flandrian Transgression”.

The result of this relatively rapid change in base-level was to drown the river valleys (cut during the previous low level phases) for a considerable distance upstream from their low level mouths (this “drowning” probably amounted to well over 100 miles in the Thames) and in consequence we have to-day—all over the world—an abnormal number of rivers with abnormally long estuaries (which are, geologically speaking, essentially ephemeral features) and all busily engaged in flooding and aggrading their flood-plains—as indicated, *inter alia*, by well marked natural levees and a cross section (of the flood plain) markedly convex upward and highest at the river margins.

During the course of this transgression, and ever since, most rivers have rapidly built forward estuarine-deltas and some of the bigger ones are now tending to get back to equilibrium again, as far as the position of their mouth goes. But such delta building necessitates raising their flood plains sufficiently to provide the slope required to carry their load from X to M, where X=maximum inland extent of sea during the transgression; and M=present mouth. And this increase in altitude (not of course increase in slope) has to be passed on backwards right away into the hills. The necessary aggradation is now in progress and often far from completed.”

5.04. THE COMBINED EFFECT OF DEPOSITION, ISOSTATIC DEPRESSION, AND EUSTATIC RISE OF SEA LEVEL.

Though there is very little evidence available as to the subtrata in the Indus area, we know that there is a great depth of accumulated sand, which indicates that isostatic depression—similar to that in the Mississippi area—has been occurring in Sind.

BEHAVIOUR OF RIVERS

It may be thought that it is immaterial what caused the 30 ft. rise of water in the Indus valley during the past 5,000 years; but actually it is important, because if the rise took place due to a eustatic rise of sea-level ending 5,000 years ago, its effect must be nearing its end; whereas if it is due to the rate of isostatic fall being less than the rise caused by deposition of silt at the mouth of the Indus, the seaward extension, and rise will continue.

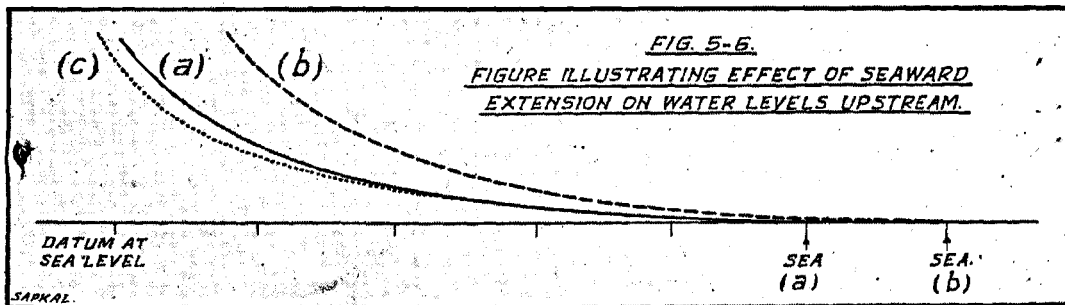
Regarding the Mississippi, it is stated :—

“The filling of the valley to approximate present levels followed final retreat of the glacier consumed from about 20,000 years to 24,000 years, so the present plain is about 6,000 years to 10,000 years old.”..... “The study of ancient meanders had developed another interesting fact—namely, that the elevations of the banks along the river’s various courses and the depths of its previous channels are practically the same as the elevations of its present banks and its present thalweg (i.e. channel) depths at correlative locations. This is convincing evidence that the river is not an aggrading stream in the ordinary sense of that term, and confirms the indication of data procured by surveys, that the Lower Mississippi River is now a “poised” stream—neither aggrading nor degrading.”

Though it cannot be said in the case of the Indus that the delta is not extending; yet the rate of extension, which at one time must have been relatively rapid, is now very gradual, indicating that the rise of water level in the Indus resulting from eustatic rise of sea-level is now nearing its end, so that the rate of rise of water levels up the Indus will hereafter be less than during the past 5,000 years.

5.05 EFFECT OF SEAWARD EXTENSION OF DELTAS AND RECESSION OF VALLEYS

Though the rate of extension of a delta may be slow and the consequent rise of water levels at a point previously at the sea may be very small, it has been argued that the resultant rise at points further upstream, where the slope is steeper, will be greater, and that a seaward movement of the delta will, effectively, cause the longitudinal section of the river throughout its length to move bodily seawards by same distance, the slope at every point being determined by exactly the same factors as before. This assumption is shown, to a very exaggerated scale, in curves (a) and (b) of Fig. 5-6, for a seaward extension of a hundred miles in a 500 mile river. Fig. 5-6



BEHAVIOUR OF RIVERS

Since 1848 we have a record a maximum and minimum levels recorded each year at Bukkur Island, three miles upstream of Sukkur; and since 1919 we have discharge data which enables us to work out the relation between specific discharges and corresponding Gauge readings at Bachalshah Gauge, 3,200 ft. downstream of Sukkur Barrage.

The question of 'rate of accretion' was first dealt with in connection with the "Technical Enquiry into the Causes of Breaches in River Bunds in Sind and Steps Required to Minimise the Danger of a Recurrence"* which was written after a major breach had occurred in the Sukkur-Begari Bund about 20 miles north of Sukkur on the night of 21st July 1942. The discharge flowing through the three breaches which developed was estimated to amount to 90,000 cusecs, which led to 2,600 square miles of country being flooded.

The question of water levels can best be split into two periods:

- (i) 1848 to 1942 based on Bukkur Gauge readings, and
- (ii) 1919 to 1944 based on discharges and Gauge readings at Bachalshah.

5.08. TRENDS IN MAXIMUM ANNUAL GAUGE READINGS AT BUKKUR

Bukkur Gauge is situated on Bukkur Island at the entrance to the larger, left bank, channel of the Gorge, opposite Rohri town. Fig. 5-8

It will be seen from Fig. 5-9 that in 1849 the maximum Gauge reading at Bukkur was 12 ft. and that it was only 12.25 in 1861, when a very high flood occurred, details of which have been given by Mr. N. M. Billimoria in an interesting article published in 1943 in the Daily Gazette. The reason why the Bukkur Gauge was not much higher in the great flood of 1861 than in previous years was because excess flood water spilled over the country. After 1861 the maximum annual Gauge at Bukkur showed a rising trend, but was still only 13 ft. in 1872; but what is important to emphasise here is that the only clear-cut rise in water levels which took place at Bukkur between 1848 and 1924 was some 3 ft. between 1861 and 1882, most of which occurred in the 10 years between 1872 and 1882. This rise synchronised with the period during which old river embankments were strengthened and new embankments constructed on both banks of the river above Sukkur. This, by stopping spill, caused a marked rise in water levels and accretion on sand banks and berms; but this rise was followed by a long period during which new conditions of flow between embankments became established, accompanied by a falling trend of water levels up to 1936. Fig. 5-9

It is important to note—for reasons that will be apparent later—that prior to the construction of embankments the year to year fluctuations in maximum flood levels were small; whereas after the embankments had been built, the year to year fluctuations increased and have continued to increase as a result of continued deposition of silt and sand on islands, sand banks, and berms, so that the river is now contained within its banks with larger discharges than hitherto and, as a consequence, the bed of the main river channel has scoured from the time the bunds were built till the construction of the Barrage, and the annual minimum water level at Bukkur showed a slight, but quite definite fall, up to 1936—and it was not until 1942 that any significant rise took place.

Although the trend relation is somewhat vitiated—because the discharges not being known could not be allowed for—there is little doubt that the falling trend in water levels was greater than could have been caused by a falling trend of discharges.

*Partial Summary contained in I. W. E. S. A. R. (T) 1943, pp. 10-21.

5.09. TRENDS IN THE RELATION BETWEEN SPECIFIC DISCHARGE
AND CORRESPONDING GAUGE READINGS DOWN-
STREAM OF SUKKUR BARRAGE

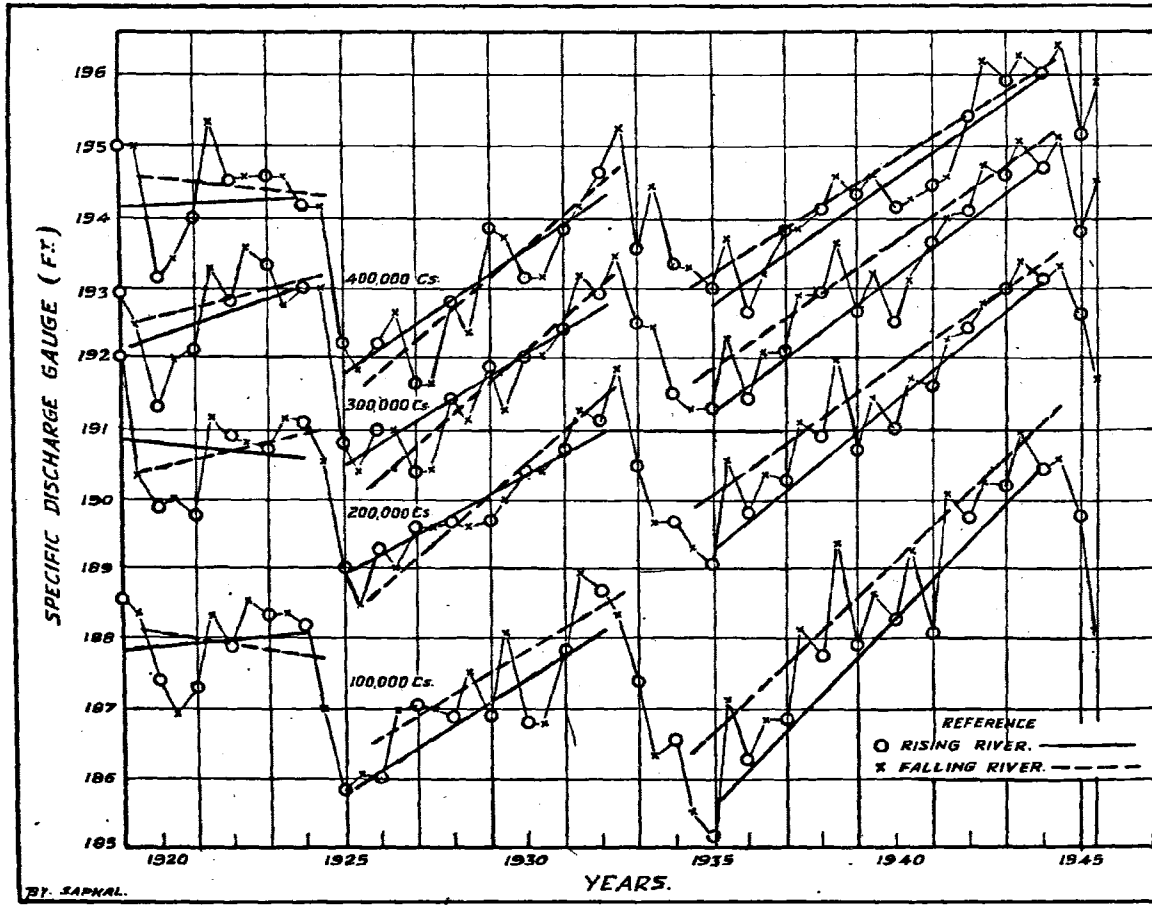
Gauge readings have been observed downstream of the site of the Barrage since 1919; and though Bachalshah Gauge is situated 3,200 ft. downstream of Sukkur Barrage, yet it is the best Gauge for indicating conditions just above the Barrage—because it is not directly affected by ponding above the Barrage, as is the case with the Gauges at Outfall, Bukkur, and the head of the old Sukkur Canal.

Bachalshah is situated where the river is still narrow, and is flowing nearly axially; so variations in Gauge, due to the effect of curvature, are very small for discharges in excess of 100,000 cusecs. Fig. 5-8

It may be wondered why Bachalshah should indicate the effect of the Barrage on upstream conditions. The explanation is that during high floods—when there is little heading-up at the Barrage—silt and sand which have accumulated by deposition upstream during long periods of ponding, are scoured through the Barrage during the relatively short periods when the gates are fully open. Under these conditions, there is a supercharge of material in movement for the slope available downstream, so deposition takes place. Consequently, though this is a measure of the rate of deterioration of the river just downstream of the Barrage; yet this deterioration affects, and hence indicates, conditions upstream of the Barrage, conditions which cannot be assessed from upstream Gauge readings except for discharges exceeding 4 lakhs cusecs—a discharge of rare occurrence for some years prior to 1942, and even, when such a flood occurred, it was of short duration.

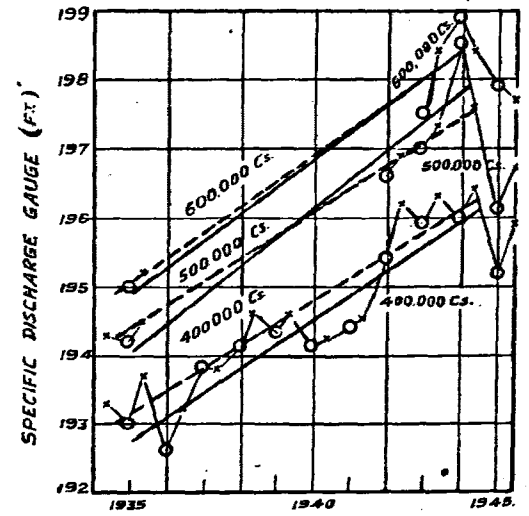
Fig. 5-10 shows that a drop in specific discharge Gauge readings occurred in 1925—which was followed by a steady rise from 1926-32. It has been suggested that this drop in 1925 might have been due to a good river in 1924. This belief that a good river in one year is followed by lower specific discharge Gauges the following year is widespread; but analysis of the Data shows no significant relationship, and the same result was obtained when specific discharges were compared with averages of the 50 maximum discharges of previous years at Sukkur. It was, however, thought that though a single good year might not have this effect, yet a series of good years might bring out such a relationship. This was tested statistically, but no such inverse relation could be found. Hence, the temporary improvement in 1925 must have been due to some other disturbing factor. The improvement in Gauge does not appear to have been due to a cut-off; but to have resulted from the banks between Bunder and the Barrage having been faced with stone pitching. The temporary effect of this would be to cause a deep channel to form along the left, concave, bank—which would persist downstream and cause a flattening of the gradient and hence a lowering of specific discharge Gauges. That a deep channel formed along the bank is shown by river contour plans. Fig. 5-10

This improvement soon disappeared when the stone facing became covered with silt during the period of construction of the Barrage—when large coffer-dams were constructed in order to make it possible to do foundation work in the dry. These obstructed about 1/4 the river width and, as a result, conditions deteriorated rapidly, though not uniformly.



(Same as fig. 110. of 1944 report) No. 6152
 Indian Waterways Expt. Stn.
 (FIG. 13. FROM A.R.T. 1943)

FIG. 5-10
SPECIFIC DISCHARGE GAUGES
AT
BACHALSHAH
1919 - 1945
FITTING OF STATISTICAL TREND LINES.



BEHAVIOUR OF RIVERS

TABLE 5-I

Trend studies of specific discharge Gauges at Bachalshah. Equation type: Specific discharge Gauge = $A + B(t - \bar{t})$ where t stands for any year A. D.

Period under consideration.	Specific discharge (lacs cusecs) R = rising F = falling	\bar{t}	A Mean Gauge over the period.	B Annual "trend" + for rise - for fall.	"Standard deviation" or measure of deviation from trend values.	Significance of B value.
1919 to 1924	1 R	1921-5	187-96	+ .0440	.58	Non-significant
	2 R	"	190-75	- .0426	.94	" "
	3 R	"	192-57	+ .2060	.71	" "
	4 R	"	194-14	+ .0289	.72	" "
	1 F	"	187-95	- .0643	.86	" "
	2 F	"	190-70	+ .1189	.45	" "
	3 F	"	192-86	+ .1423	.67	" "
	4 F	"	194-50	- .0491	.77	" "
1925 to 1932	1 R	1928-5	186-99	+ .3417	.43	†
	2 R	"	189-94	+ .2869	.18	‡
	3 R	"	191-60	+ .3214	.35	‡
	4 R	"	193-01	+ .3607	.52	†
	1 F	"	187-51	+ .3107	.64	•
	2 F	"	190-04	+ .4536	.25	‡
	3 F	"	191-60	+ .4476	.51	†
	4 F	"	193-08	+ .4452	.66	†
1935 to 1944	1 R	1939-5	188-08	+ .5418	.40	‡
	2 R	"	181-19	+ .4321	.27	‡
	3 R	"	192-98	+ .3939	.33	‡
	4 R	"	194-36	+ .3563	.26	‡
	5 R	1941-0	196-575	+ .4180	.63	•
	6 R	1940-6	197-13	+ .3870	.72	
1934 to 1944	1 F	1939	188-845	+ .4918	.56	‡
	2 F	"	191-67	+ .3681	.37	‡
	3 F	"	193-42	+ .3627	.39	‡
	4 F	"	194-62	+ .3263	.50	‡
	5 F	1939-6	196-12	+ .3363	.06	‡
	6 F	1940-6	197-33	+ .3728	.27	

*Denotes significance at 5 per cent level.
 †Denotes significance at 1 per cent level.
 ‡Denotes significance at .1 per cent level.

THE BEHAVIOUR AND CONTROL OF RIVERS AND CANALS

When the Barrage gates were first closed in 1932, ponding was considerable, and as a result the coarser grades of silt and sand came to rest several miles upstream of the Barrage, while the finer grades deposited on the berms and banks, and even on the bed, smoothing out uneven portions. Later, when the Barrage gates were opened to pass the floods, the coarser grades which had deposited near the upper end of the pond began to be scoured; but they moved only a short distance before the flood fell, and did not reach the Barrage—so the charge of bed material which passed through the Barrage was less than previously, and the banks were smoother. As a result, the velocity was high and the depth low, and bed-scour occurred downstream of the Barrage, causing retrogression—with resultant lowering of specific discharge Gauge levels at Bachalshah in 1933 and 1934. But the relief was short-lived; because when all the low portions in the length between the upper end of the pond and the Barrage had filled up, and the bed above the Barrage had begun to rise, and the charge in movement when the gates were opened, increased until it became equal to, and from 1935 greater than, before the Barrage was built; so retrogression was replaced by rapid accretion.

Fig. 5-9

Table 5-1
Fig. 5-10

Table 5-1 and 5-10 show no significant relationship between time and specific discharge Gauge readings for the period 1919-24; a slight significance from 1925-32, during construction of the Barrage; but a high degree of significance between 1934-35 and 1944.

5.10. CONCLUSIONS REGARDING THE FUTURE RISE OF WATER LEVELS RESULTING FROM HEADING UP AT SUKKUR BARRAGE

The conclusions to be drawn are that:—

(1) as the pond gradually extends further and further upstream, the coarser grades of material will deposit further from the Barrage, and less of the material so deposited will be scoured away when the gates are opened—because the effect of draw-down decreases with distance from the Barrage. From this it follows that the quantity of the coarser grades—and also finer grades—washed through the Barrage will hereafter decrease; so the quantity which will deposit downstream will become progressively less.

(2) as material washed through the Barrage builds up the bed further and further downstream, the Gauge readings at Bachalshah will continue to rise for the same specific discharges; so the period during which water will be headed up at the Barrage will become progressively shorter, and the period during which the gates will be open will be correspondingly longer.

(3) for these and other reasons, the rate of rise of water level at Bachalshah will decrease until it becomes less than the rate of rise under pre-Barrage conditions.

5.11. ENQUIRY INTO THE EFFECT OF CONSTRUCTING THE RIGHT BANK APPROACH CHANNEL AT SUKKUR ON THE RATE OF RISE OF WATER LEVELS IN THE INDUS

At the end of 1943 the Author was asked to deal with a Report, dated 7th September 1943 in which the Superintending Engineer in charge of the Barrage had written:—*

“After 1935, accretion started and gradually the river attained 1932 levels in the year 1941. The accretion has continued further and in the last two years there has been further rapid accretion of about 2·2' at 560,000 cusecs discharge.”

* I. W. E. S. A. R. (T) 1944 pp. 99-104 “Causes of accretion at Sukkur Barrage and how to reduce rate of rise.”

BEHAVIOUR OF RIVERS

"The fact that the rate of accretion has rapidly increased after 1940—i.e. after functioning of the new river works (Approach Channel) shows that causes 3 and 4 (reduction of waterway and consequent sudden expansion downstream of the Barrage) play an important part in causing accretion."

Before replying to these specific points it is desirable to explain the origin of the Approach Channel.

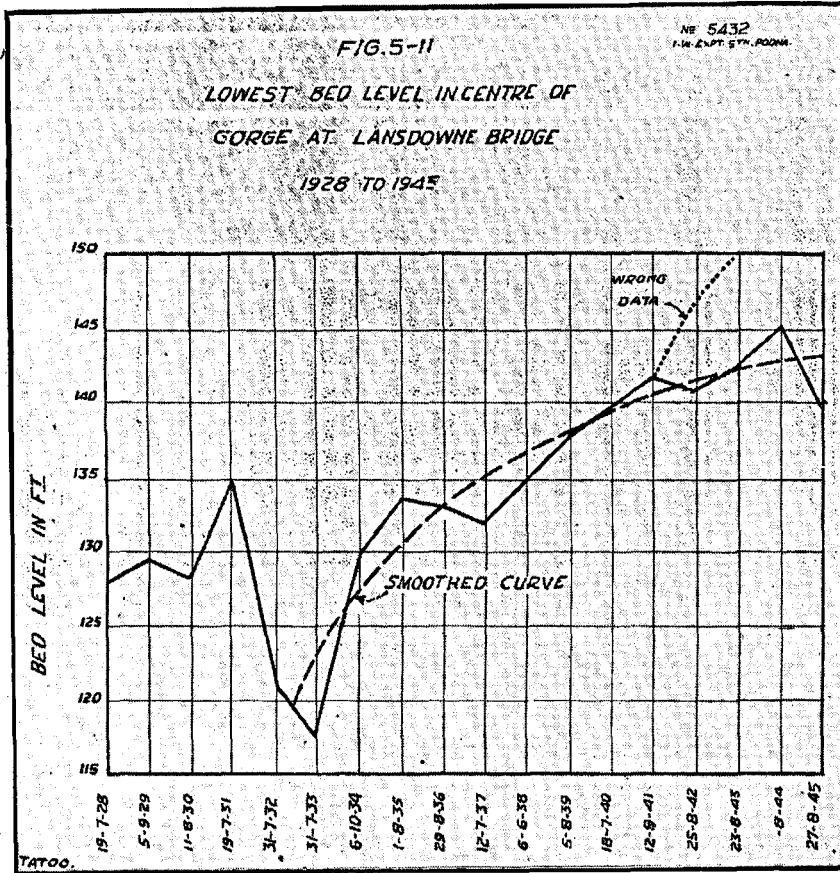
Downstream of the Gorge, the river fans out, curving slightly to the west, and it is due to this unfavourable curvature that excess bed sand was drawn by the Right Banks Canals prior to the construction of the Approach Channel in 1939-40. Fig. 5-8

That this would occur was predicted by the Author, and confirmed by experiments carried out at Poona in 1929;* but it was not until 1938 that the predicted heavy accretion occurred in the Right Bank Canals. The Author was then asked to recommend the best way to exclude the excess sand; and after several alternatives had been tested, Mr. A. Gordon, the then Chief Engineer and Secretary to Government, P. W. D. in Sind, decided to adopt the design shown in Fig. 5-8 in which curvature of flow was enforced by means of a curved Approach Channel 5,000 ft. long. Water to feed the canals was drawn from this channel over a raised sill 2,500 ft. from the head of the Approach Channel; while a larger supply of water, containing most of the coarser grades of sand, was diverted down the tail channel, to pass through spans 15 to 22 of the Barrage. Though this design reduced the river spans from 54 to 36, yet these are ample to pass the maximum discharge; because though the Barrage was originally designed for a maximum flood of 1,500,000 cusecs, the maximum recorded flood (after correcting early data for errors due to meter suspension) was 730,000 cusecs; and the maximum anticipated discharge is 900,000 cusecs, which is only 60 per cent of that assumed in the original design, giving a coefficient of discharge $C=2.7$ in the equation $q=Cd^{1.5}$. This is appreciably less than the standard coefficient adopted by the Punjab. The upstream 1,700 ft. of the outer bank of the Approach Channel was constructed before the flood season of 1940 by Mr. S. B. Hickin—the then Executive Engineer, Barrage Division—and the whole work was completed by him before the flood season of 1941. The Approach Channel has thus been in operation during the flood season of 1941-45 and has proved very effective during that period in excluding bed-sand from the canals.† Fig. 5-8.

In the investigation carried out to determine whether the Approach Channel could be held to have had any deleterious effect on the river, several interesting facts were brought to light.

* Bombay P. W. D. T. P. Nos. 45, 46 and 52.

† C. I. and H. E. S. A. R. (T) 1941-42 pp. 16-19, and see Chapter 8.



(FIG. 113 FROM A. R. T. 1944 OF I. W. E. STN.)

5.12. PROGRESSIVE ACCRETION IN THE GORGE THREE MILES UPSTREAM OF THE BARRAGE BETWEEN ROHRI AND BUKKUR ISLAND

Fig. 5-11

Fig. 5-11 shows that retrogression of minimum bed levels occurred in the main Gorge at Lansdowne Bridge in 1932 and 1933 when it also occurred downstream of the Barrage; but an abrupt rise took place in the Gorge in 1934, whereas the Bachalshah Gauge was not affected till a year later.

Though the minimum bed level continued to rise in the Gorge up to 1944, the rate of rise steadily decreased, and in 1945 there was a fall. The smooth curve drawn on fig. 5-11 seems to indicate that the bed is not likely to rise much further—unless the discharge decreases.

5.13. PROGRESSIVE CHANGE IN AREA OF WATERWAY DOWNSTREAM OF SUKKUR BARRAGE

Fig. 5-12

It was argued that the river had been silting, particularly downstream of the Barrage on the line X. Y., due to the Approach Channel.

Fig. 5-12 shows that not merely was the datum area of the river downstream of spans 23 to 59, 60 per cent greater in 1944 than before the Approach Channel was built; but that the datum area for the whole river width was also greater in 1944 than before the Approach Channel was built; and, when increased depths are taken into account—caused by the rise in water level—the increase in area of waterway was still greater in 1944 than before the Approach Channel was built.

There has thus been an improvement, *not* a deterioration, since the Approach Channel was constructed.

FIGURE. 5-12 (a) DATUM AREA BELOW R.L. 193 ON C.S. X, Y. 1000 FT D/S OF BARRAGE

(a) FOR WHOLE SECTION (b) D/S OF SPANS 24 TO 59 & ALSO AREAS SHOWN IN DASH LINES

ALLOWING FOR:-

- + INCREASE IN AREAS DUE TO YEARLY RISE
- DECREASE " " FALL

OF WATER LEVEL AT BACHALSHAH FOR 300,000 CFS. BASED ON MEAN OF RISING & FALLING GAUGES WITH 1932 LEVELS AS STANDARD

M.O. 5113
L.W. SECT STATION
PROMA

(FIG. 112 a & b FROM
A.R.T. 1944 OF I.W.E. ST^{NS})

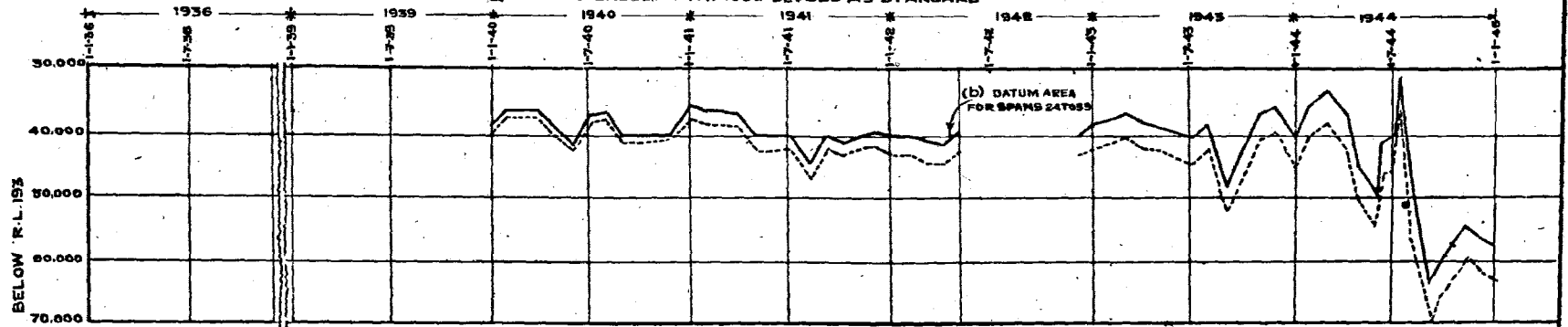
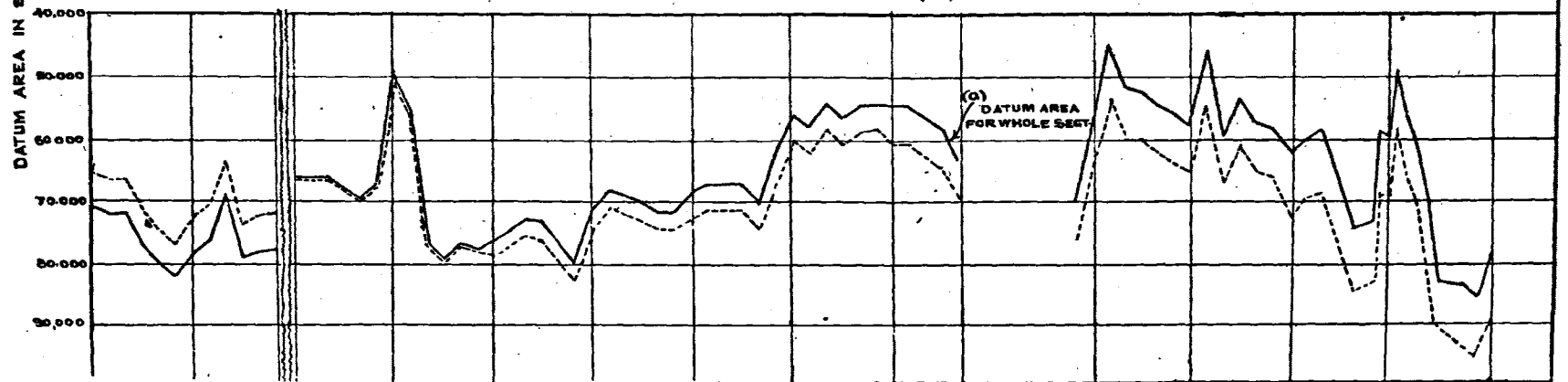


FIGURE. 5-12 (b)



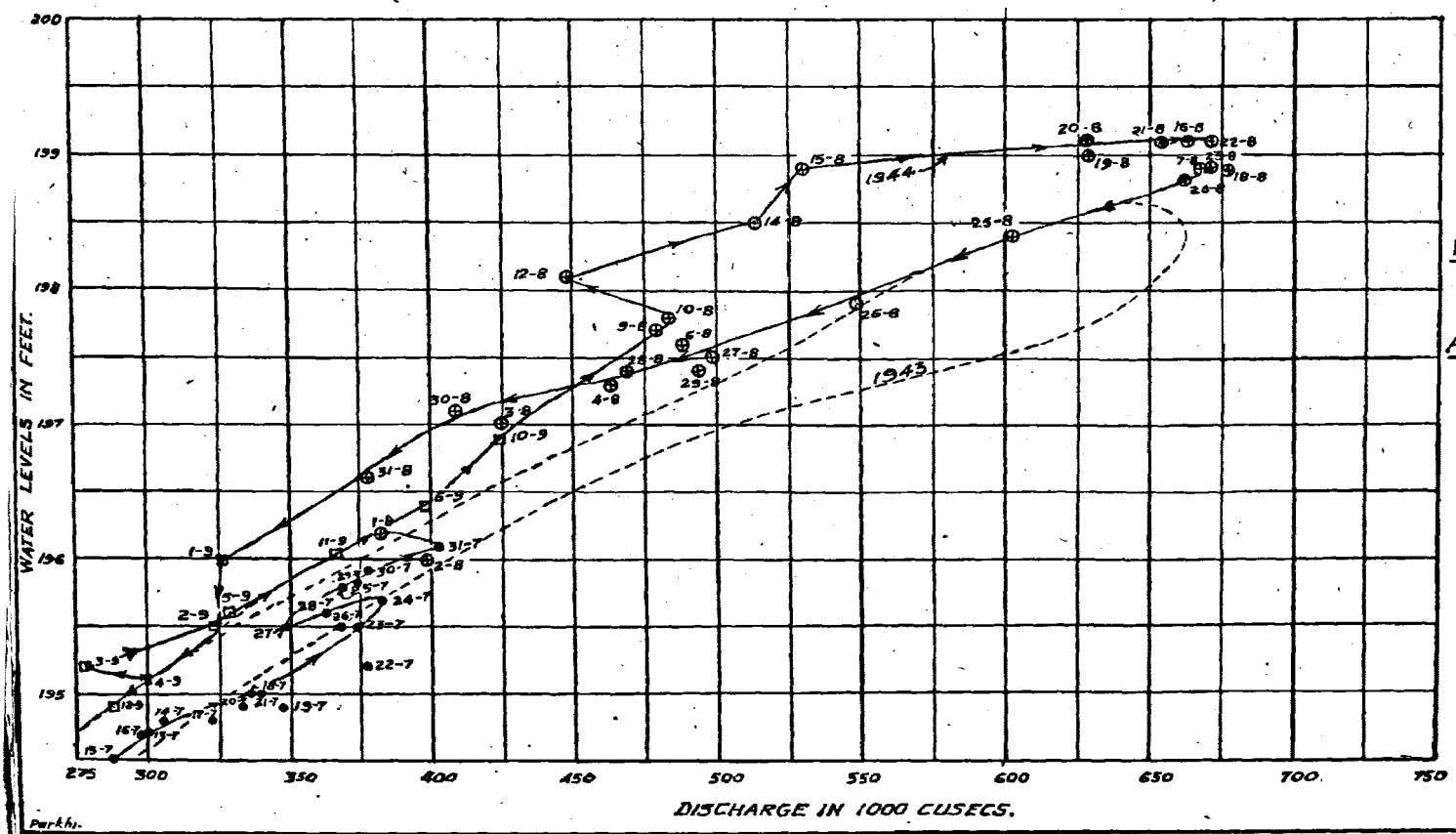
(Same as fig. III. of 1944 Report.)

NO. 5093
INDIAN WATERWAYS EXPT. STR.
POONA.

(FIG. III. FROM A.R.T.
1944 OF I.W.E.S.T.)

FIG. 5-13

BACHALSHAH
WATER LEVEL : DISCHARGE
1943 & 1944
FROM 13-7-44 TO 12-9-44.



REFERENCES:-

- JULY 1944
- AUG. "
- SEPT. "

BEHAVIOUR OF RIVERS

5.14. EFFECT OF APPROACH CHANNEL ON RISING TREND OF SPECIFIC DISCHARGE GAUGE READINGS.

Returning now to Fig. 5-10 and Table 5-I; Despite the high degree of correlation shown in Table 5-I for the whole period 1934-35 and 1943-44, it had been held that the abrupt rise in 1942 had been due to the Approach Channel—work in connection with which was started at the end of 1939 and completed before the flood season of 1941. In 1940 and 1941, during the time the Approach Channel was under construction and for one year after its completion, the specific discharge Gauge readings were abnormally low, and it was not until 1942, when the breach occurred, that an abrupt rise took place. Fig. 5-10
Table 5-1

It is always unwise to jump to conclusions on a single year's results, merely because a change in a river happens to be subsequent to some other change; but even if this fallacious "post hoc ergo propter hoc" arguments were followed, the breach, not the Approach Channel, should have been blamed.

5.15. SHORT PERIOD VARIATIONS IN THE RELATION BETWEEN SPECIFIC DISCHARGES AND GAUGE READINGS

Fig. 5-13 shows that specific discharge Gauge readings may vary by a foot within a period of ten days. This is generally due to the movement of dunes of submerged bed-material; but in 1942—when the rapid rise took place—the dune effect was accentuated by flood-water from the breach re-entering the river 17 miles downstream of the Barrage. Fig. 5-13

Though the afflux effect at the Barrage (calculated by an afflux formula) would appear to be small, the cumulative effect of deposition of sand at the upper end of the pond must have been considerable, as the gates were kept open.

Fig. 5-14 shows the marked improvement which took place in 1945. Had the deterioration in 1942 been due to the Approach Channel, no such subsequent marked improvement would have occurred. Fig. 5-14

5.16. PROGRESSIVE REDUCTION OF SILT CHARGE AND LOAD

Table 5-II shows average discharges, silt loads, and charges (in grains per cubic foot) during "183 day flood-seasons" (from 1st April to the end of September). Table 5-II

(i) TRENDS IN SILT LOAD AND CHARGE, RELATED TO TIME

Fig. 5-15 shows the variations with time of average discharge, charges, and loads—based on a 183 day flood season (Abkalani). All the parameters show a gradual decline prior to ponding at the Barrage in 1932, and also a gradual decline subsequent to ponding in 1932 (the sudden drop in discharge after ponding at the Barrage was due to withdrawals of water for the canals). The only statistically significant trend was for the decrease of silt-charge after ponding at the Barrage. Fig. 5-15

(ii) TRENDS IN SILT LOAD RELATIVE TO DISCHARGE

Fig. 5-16 shows that when average silt load values for 183 days were plotted against average discharge for the same period, high degrees of association were obtained, especially for the period 1913-1932 and for sub-groupings in the period 1933-44. Fig. 5-16

THE BEHAVIOUR AND CONTROL OF RIVERS AND CANALS

Points to be noted are:—

Fig. 5-17

(i) In 1917 the river changed its course upstream of the Gorge, as shown in Fig. 5-17.

(ii) In 1942 the silt load at outfall was relatively low after the Indus breached the Mahomedabad Loop Bund some 20 miles upstream of the Barrage on 21st July 1942 (i.e., just when the peak discharge was reached).

(iii) The silt charges in 1943 and 1944 were relatively more than in 1942; but were still considerably less than before 1942. This was due to the sudden rise of 1942 causing a much larger area of berm to be submerged, thus inviting deposition of silt.

(iv) The data for 1917 and 1942 were abnormal.

(v) The scatter in the pre-Barrage group (1913-32) was less than for the whole period (1913-44).

(vi) During the post-Barrage period when the discharge in the River downstream of the Barrage was reduced considerably by canal withdrawals, the apparent scatter in the silt-load distribution became relatively great. This was due to ponding at the Barrage causing a marked change in the quantity of silt and sand in movement; thus, during the winter months, the water, instead of being silty, was at first bottle-green. The silt load during the flood season 1933, 1934 and 1936 was, however, similar to that during the pre-Barrage period for the same discharges, indicating that the silt load during the flood season—not the sand load—was little affected at first by the Barrage, but the sand charge was markedly less; but between 1937 and 1940 a marked decrease in silt charge occurred, followed by a sudden further decrease in 1942, as already explained.

At first sight, the continued decrease in silt charge with time may seem surprising, for it might be thought that as the water levels upstream of the Barrage rose, silting would become progressively less, and hence the silt carried at outfall would increase; but though accretion has been occurring along the edge of the river—since water was first headed up—the progressive rise in water level up the river commands a greater length and area of low-lying berm behind the silted lips of the berms, so that more and more silt deposits on these low areas; so, until nearly the whole width of the river berm has become silted—which will not be until the rate of rise of water levels caused by afflux at the Barrage has almost ceased—the charge of silt passing down the river will continue to decrease; but eventually it must again increase.

A second confirmation of the close correlation between discharge and silt load was obtained by a method which is not subject to the criticism that the correlation may be spurious—resulting from the mutual dependence of both these variables on time.

The standard method of eliminating the time effect consists first of separately fitting time trend lines to the data for the variates, discharge and silt load in this case; and then correlating the departures of the observed values of these variates from their time trend lines: the resulting correlation, if any, can then be said to be free from the influence of time-effect on the variable.

Fig. 5-18

This method gave an identical degree of correlation for the pre-Barrage years (1913-32) as had been obtained previously from the "absolute values" without eliminating the time trend; while, for the post-Barrage years (1933-44)—during which period the downward trend of silt-load was more marked—a more significant correlation was obtained between the departures than between the absolute values (.889 against .755). Thus, even during the post-Barrage period, the correlation between the discharges and silt loads is confirmed to be real.

FIG. 5-15.

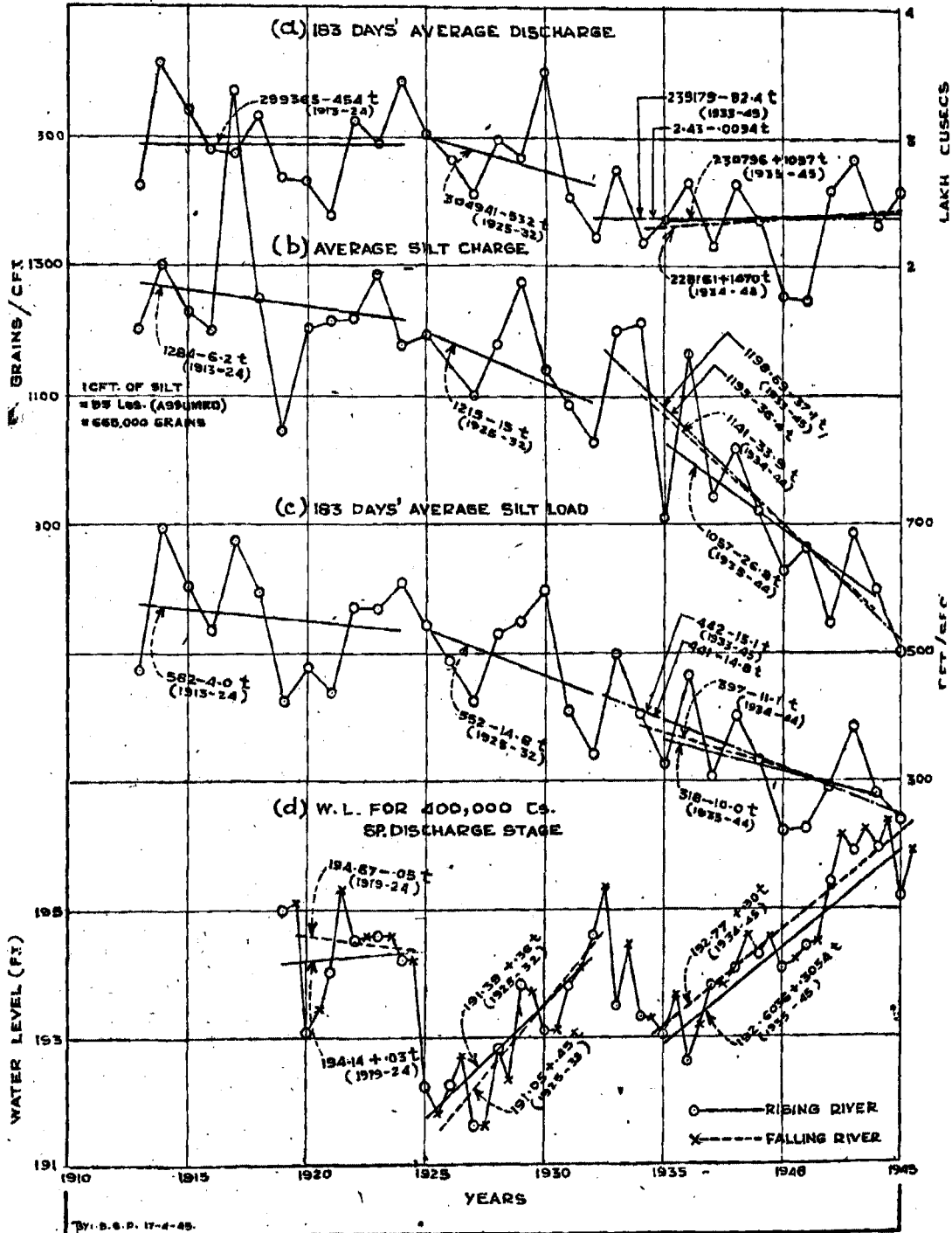
NR 5494
I. W. & S. P. P. 1944

RELATIVE VARIATIONS OF AVERAGE DISCHARGE, SILT CHARGE & LOAD & 400,000 Cs. STAGE SP. DISCHARGE GAUGE AT BACHALSHAH

WITH TIME

N. B.

- (1) CABLE SUSPENSION CORRECTION HAS BEEN APPLIED TO ALL DISCHARGES PRIOR TO 1924.
- (2) DISCHARGES & SILT LOAD FIGURES ARE THE TOTAL VOLUMES TRANSPORTED DURING EACH SEASONAL YEAR (I.E. 1ST OCT. TO FOLLOWING 30TH SEPT.) BUT AVERAGED FOR THE 183 (ABKALANI) DAYS OF RIVER ACTIVITY.
- (3) FIGURES FOR BACHALSHAH HAVE BEEN APPROPRIATELY REDUCED FROM THE FIGURES OBSERVED AT OUTFALL SECTION.

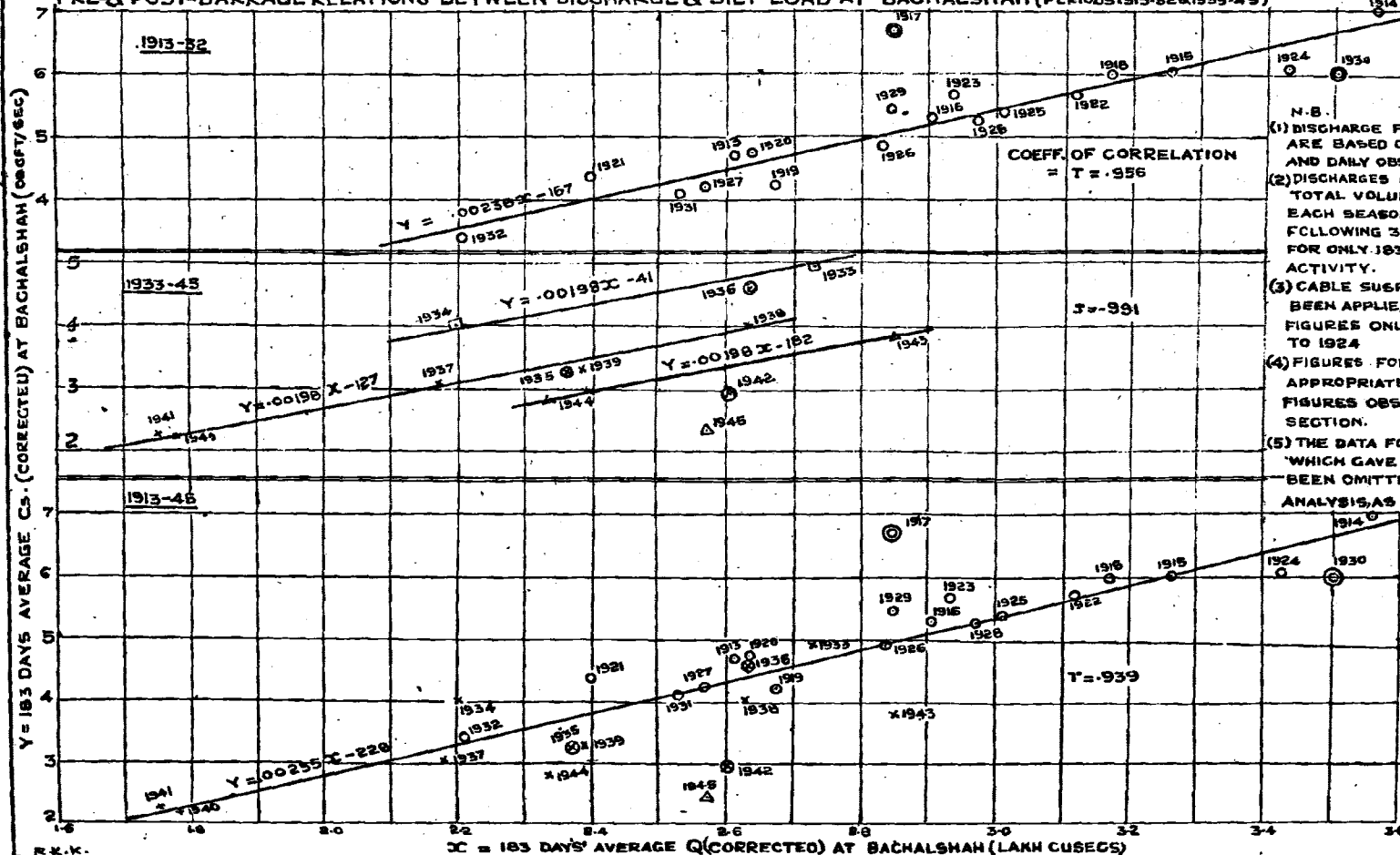


DATA CORRECTED FOR CABLE SUSPENSION ERRORS

PRE- & POST-BARRAGE RELATIONS BETWEEN DISCHARGE & SILT LOAD AT BACHALSHAH (PERIODS 1913-32 & 1933-45)

NO-5567
I.W. EXPT. STATION: POONA
(FIG. 73, FROM
A.R.T. 1945)

FIG. 5-16



- N.B.
- (1) DISCHARGE FIGURES PRIOR TO JUNE 1920 ARE BASED ON BIWEEKLY OBSERVATIONS AND DAILY OBSERVATIONS THEREAFTER.
 - (2) DISCHARGES & SILT LOADS ARE THE TOTAL VOLUMES TRANSPORTED DURING EACH SEASONAL YEAR (i.e. 1st OCT. TO FOLLOWING 30th SEPT) BUT AVERAGED FOR ONLY 183 DAYS (ABKALANI) OF RIVER ACTIVITY.
 - (3) CABLE SUSPENSION CORRECTION HAS BEEN APPLIED TO SUCH AVERAGE FIGURES ONLY FOR ALL DATA PRIOR TO 1924.
 - (4) FIGURES FOR BACHALSHAH HAVE BEEN APPROPRIATELY REDUCED FROM THE FIGURES OBSERVED AT OUTFALL SECTION.
 - (5) THE DATA FOR YEARS 1917, 30, 35, 36 & 48 WHICH GAVE ABNORMAL POINTS, HAS BEEN OMITTED FROM STATISTICAL ANALYSIS, AS BEING INCORRECT.

701

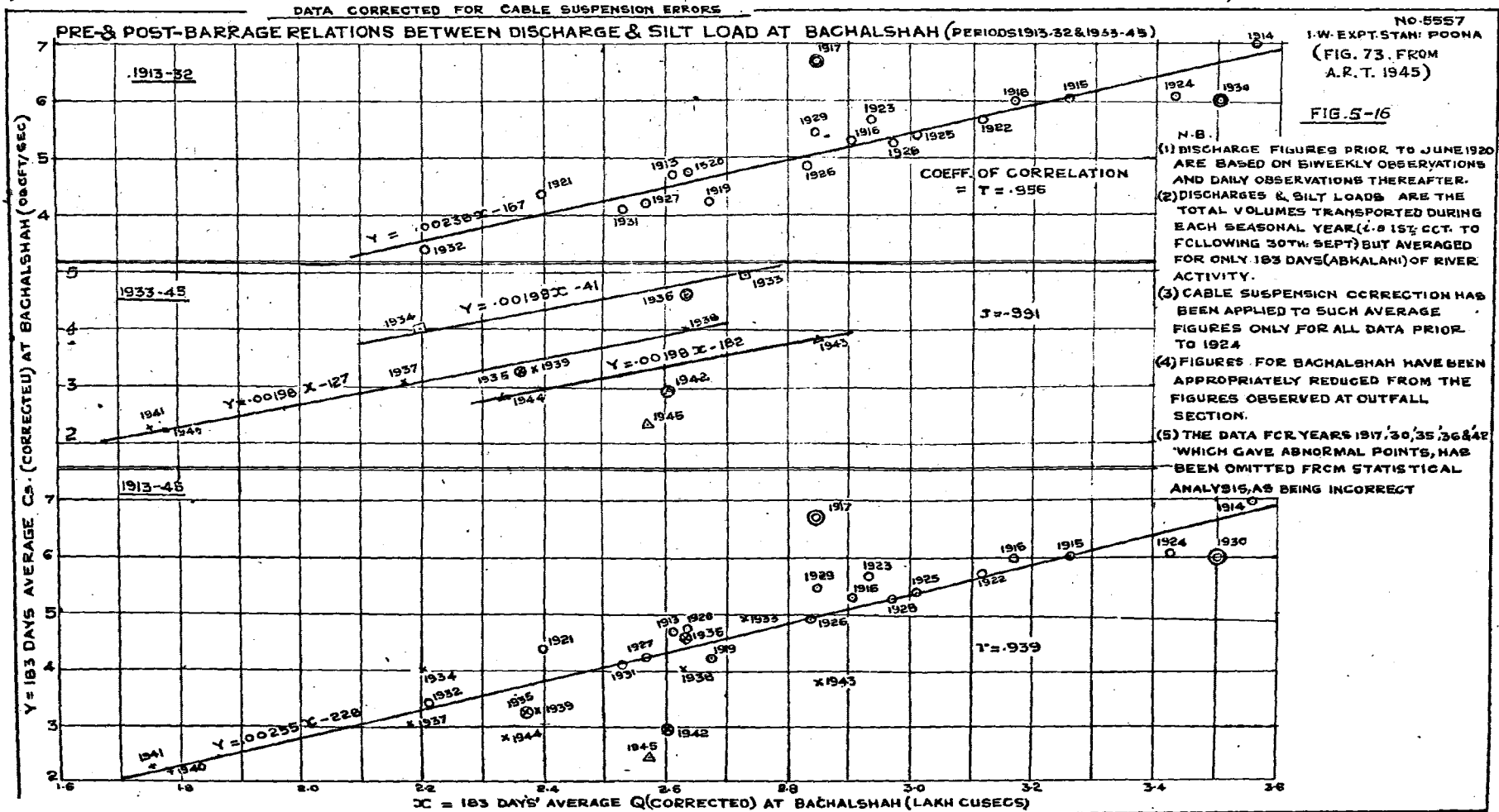
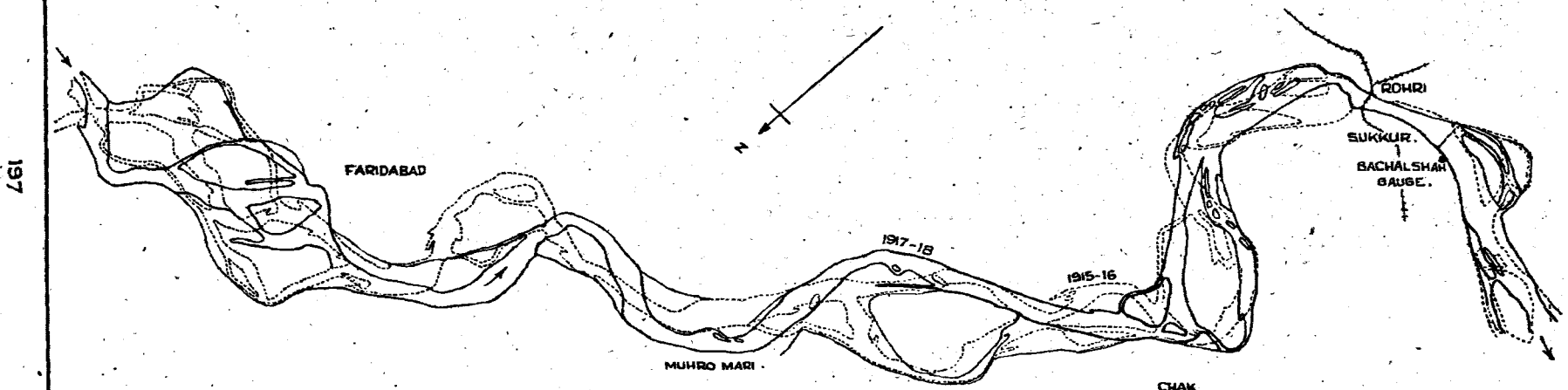




FIG. 5-17

CHANGES IN THE COURSES OF
RIVER INDUS OVER A REACH OF 55 MILES U/S
OF SUKKUR DURING 1916-17

SCALE: 1" = 4 MILES.

NR 5483.
I.W. EXPT. STN. PUNJAB
REDUCED FROM DRAWINGS
FROM INDUS RIVER COMMISSION
RECORD PART II & IV.
(FIG. 69. FROM A.R.T.
1945 OF I.W.E.S.T.)



REF:-
1915-16 
1917-18 

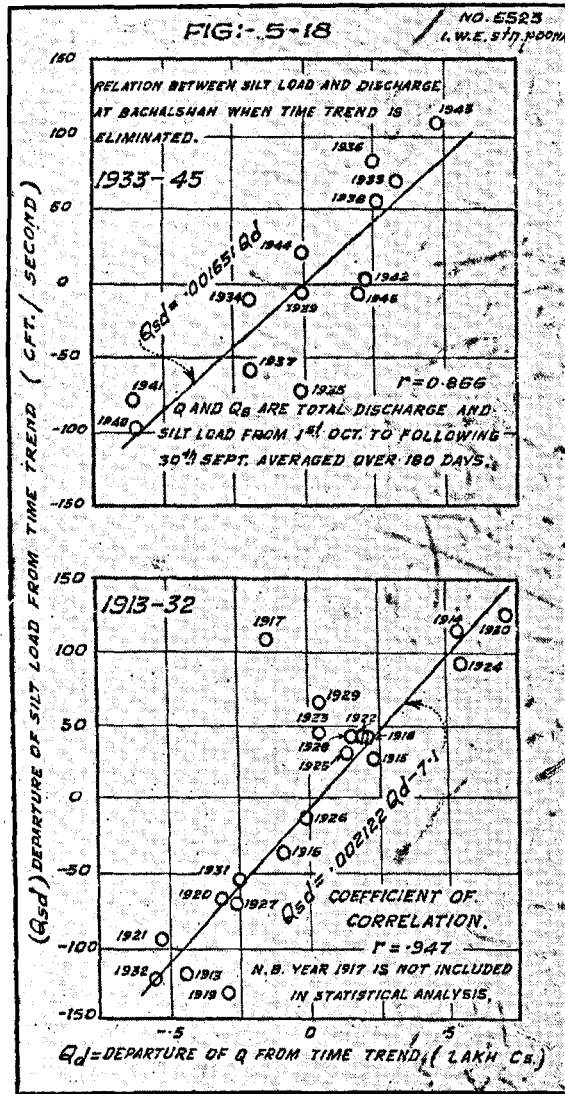
BEHAVIOUR OF RIVERS

TABLE 5-II

183 days averages at Outfall Section and Bachalshah

Years.	Outfall Section.				Bachalshah.			Kotri.
	Not corrected for cable suspension.		Corrected for cable suspension.		Discharge (cusecs).	Silt-load (ft./sec).	Average annual silt-charge in grains per ft.	Average annual silt-charge in grains per ft.
	Discharge (cusecs).	Silt-load (ft./sec).	Discharge (cusecs).	Silt-load (ft./sec).				
1	2	3	4	5	6	7	8	9
1913	289098	524	261134	473	261134	473	1306	1321
1914	409268	802	356137	698	356137	698	1303	1428
1915	370820	688	326246	605	326246	605	1233	1511
1916	325962	598	290827	534	290827	534	1206	1358
1917	318836	752	285151	673	285151	673	1569	1522
1918	359623	679	317400	599	317400	599	1254	1058
1919	297164	467	287700	421	287700	421	1046	1399
1920	292399	530	263825	473	263825	473	1205	1364
1921	263273	482	239847	439	239847	439	1217	1817
1922	352811	646	311991	571	311991	571	1219	1519
1923	329629	639	293736	569	293736	569	1288	1407
1924			342956	608	342956	608	1179	1253
1925	From 1924 onwards current meters have been rigidly suspended on a metal rod. No corrections are, therefore, necessary for these observations.		301137	541	301137	541	1194	1343
1926			283270	489	283270	489	1149	1232
1927			256922	424	256922	424	1098	1045
1928			297635	530	297635	530	1183	1289
1929			285208	548	285208	548	1278	1135
1930			350508	600	350508	600	1138	1210
1931			252767	412	252767	412	1085	1204
1932			249671	386	220723	341	1029	1204
1933	From 1932 the figures for Bachalshah are less than those measured at Outfall due to canal withdrawals		312635	566	273247	495	1203	1855
1934			272240	497	220063	402	1214	1252
1935			289191	395	236891	324	909	880
1936			315422	553	263307	462	1166	1291
1937			268253	380	217636	308	942	1011
1938			317819	498	262913	404	1020	1170
1939			290825	403	238302	330	921	1224
1940			232904	290	177934	222	828	1213
1941			230631	299	175346	227	863	1139
1942			307614	347	260405	294	750	917
1943			332452	445	285306	382	889	...
1944			281586	340	233475	282	802	...
1945			707	1257

THE BEHAVIOUR AND CONTROL OF RIVERS AND CANALS



(FIG. 75 FROM A. R. T., 1945 OF I. W. E. STN.)

5.17. CAN ANYTHING BE DONE TO RETARD DETERIORATION IN A RIVER
RESULTING FROM HEADING UP WATER AT A BARRAGE?

Considering the question in the light of experience gained at Sukkur Barrage:

It has been argued that constructing the Barrage three miles downstream of Rohri Gorge has led to increased accretion; but—as can be clearly seen from Fig. 5-11—this did not prevent retrogression occurring in the Gorge and downstream of the Barrage in 1933 and 1934; and subsequently, when accretion took place, the coarser grades of bed material were trapped and permanently held in the bed of the Gorge. From 1935, accretion occurred in the Gorge; but the rate of rise decreased and has now almost reached its limit; so the quantity of material which is alternately trapped in the Gorge, to be later scoured away, is now less—not more—than it used to be; and almost the whole of the deterioration is now due to alternate heading-up and draw-down resulting from regulation of the Barrage gates. Locating the Barrage where it is, so far from being a mistake, has had the enormous advantage of allowing flow to be easily controlled, so that it is not subject to wide oscillation—as is the case upstream of so many Barrages. Even, therefore, if the position of Sukkur Barrage could be altered, this would be highly undesirable.

Fig. 5-11

Some further deterioration is inevitable, but the rate of deterioration can be retarded, and the total amount of deterioration restricted, by reducing the Full Supply water levels in the Right Bank Canals—as was recommended by the Author in 1942.

It is for serious consideration whether it would not have been better to maintain a minimum difference of 2 ft. between the upstream and downstream water levels at the Barrage; because, though this would have led to quicker initial accretion upstream, it would have reduced the rate of accretion downstream of the Barrage; and over a long period of years would have caused less total alternate accretion and scour, and hence less deterioration.

In conclusion it may be said that all the evidence goes to show that the rate of accretion will decrease as the period of the flood season during which the gates are kept partially closed decreases; and specific discharge Gauge readings are not likely to increase more than about 2½ ft. above the 1942 levels during the next fifty years and thereafter will rise little, if at all, faster than if the Barrage had not been built.

5.18. EFFECT OF REDUCTION OF DISCHARGE AND CHARGE IN A RIVER—
RESULTING FROM WATER AND BED MATERIAL BEING IMPOUNDED
IN A LARGE STORAGE RESERVOIR IN THE HILLS

It is often argued that, in accordance with Lacey's formulæ, a permanent reduction of river discharge requires a steeper slope; and, as a consequence, the river must either build up its bed, or shorten and straighten its course. The problem is highly complex.

We know that—

(a) an increase in grade—charge and discharge remaining the same—requires a corresponding increase in slope; with little increase in width ($P \propto Q^{\frac{1}{2}}$ —Lacey).

(b) an increase of charge—grade and discharge remaining the same—requires an increase of both width and slope; but this is complicated during the period of change—and possibly afterwards—by the fact that an increase of charge leads to primary meandering and may lead to the river tending to split up into two or more channels.

THE BEHAVIOUR AND CONTROL OF RIVERS AND CANALS

(c) an increase of discharge—charge and grade remaining the same; or a decrease of grade—discharge and charge remaining the same—will cause a decrease of slope ($S \propto f^{5/3}/Q^{1/3}$ Lacey).

(d) a decrease of charge—discharge and grade remaining the same—will lead to retrogression and straightening in the length where retrogression is taking place. Further downstream, conditions will differ little from previous conditions, so long as the discharge remains the same.

When water is impounded in a new dam, bed-material is thrown down at the upper end of the lake and much of the material in suspension is deposited where the velocity in the storage drops below that necessary to keep the material in suspension. As a consequence, less material in suspension, and no coarse particles of bed-material from upstream pass through the dam. Part of the energy of flow—hitherto utilised in carrying the charge in the river downstream of the dam—is thus set free, leading to increased residual turbulence. Retrogression then extends rapidly downstream—about 14 miles during the first year in the case of the Boulder dam.* Subsequently, retrogression extends more gradually, until eventually the effect must, in the case of an alluvial river, reach the sea. Each tributary stream is also affected; because lowering the water level in the main stream causes “draw-down” in the tributaries where they join the main stream; so that scour and retrogression occur in each tributary. This works upstream unless prevented by a Barrage, weir, or rock barrier in the tributary.

The late H. W. Nicholson of the Punjab, coined the expression “sympathetic retrogression” for retrogression which develops in this way.

Material exposed by scour, nearly always increases in coarseness with increasing depth; and, even if this were not so, the grades exposed on the bed would become progressively coarser—due to the fine grades being washed out during retrogression of bed levels; so the slope required downstream of a dam may actually exceed the original slope immediately downstream—as was the case downstream of Boulder dam in the Colorado River in U. S. A.* Further downstream, the slope must, however, be flatter than before—due to the weighted charge coming from upstream being less than before the dam was built.

Thus the construction of a dam affects conditions in three ways:—

- (a) it reduces the charge.
- (b) it reduces fluctuations in discharge and charge,
- and (c) it generally reduces the “dominant discharge.”
- (a) tends to straighten the channel and flatten the slope;
- (b) reduces meandering;
- (c) tends to increase the water slope; but, due to the predominant effects of (a) and (b), the overall slope flattens.

The total effect is a straightening and shortening of the channel accompanied by a flattening of water gradient, and retrogression of bed levels. This, however, tends to be limited by the bed-materials becoming coarser with depth, both actually *in situ*, and because the finer grades are progressively washed out until the residue consists almost entirely of the coarser particles.

* Seventh Annual Report on “Retrogression observations—Colorado” of U. S. Dept. of the Interior Bureau of Reclamation River Basin, by J. W. Stanley, July 1st 1942.

BEHAVIOUR OF RIVERS

5.19. THE EFFECT OF THE PROPOSED UPPER SIND BARRAGE ON WATER LEVELS LOWER DOWN THE RIVER

It is the present intention to keep the Upper Sind Barrage open during the months of November to March; so the changes brought about by it, will take very much longer to develop than at Sukkur.

The first effect will be to cause some retrogression downstream; but in two or three years this will be followed by accretion—due to the amount of material washed through the Barrage during periods when the gates are open exceeding what the discharge can carry with the downstream slope; and only part of this will be scoured away when the flow becomes relatively clear. We thus have the apparent anomaly that though the total quantity of material which passes through a Barrage is less than what passed down the river before its construction, accretion occurs. The explanation is simple. The energy available to carry the charge is reduced due to energy being dissipated by heading up at the Barrage.

Thus the initial effect of the Upper Sind Barrage will be to cause a small rise in water levels upstream, and for a short distance downstream—which will progress slowly downstream in subsequent years; but, as the quantity of bed material passing down the river will be slightly less than before, this will tend to make the river slope flatter further downstream and so reduce the rate of rise of water above Sukkur. Constructing the Upper Sind Barrage will therefore tend to reduce the rate of rise of water levels at Sukkur for several years.

5.20. EFFECT OF PONDING AT SUKKUR BARRAGE ON WATER LEVELS AT A DISTANCE UPSTREAM

An analysis of Gauge readings upstream of Sukkur Barrage for the eleven years since water was first ponded, does not show any significant rise even at Rajib mouth 23 miles upstream. In 1944, there was a distinct rise; but further data will be required before it can be decided whether this was due to a normal fluctuation or was the beginning of the rise resulting from ponding at Sukkur Barrage.

5.21. CAUSE OF REDUCTION OF SILT IN SUSPENSION AT SUKKUR AND KOTRI

In spite of water being headed up at the Barrage, the charge of suspended silt was high at Sukkur during 1933 and 1934. Table 5-II
Fig. 5-16.

This was a secondary effect resulting from the coarser grades being checked during the rising flood at the upstream end of the pond; so that when ponding was later reduced during the height of the flood season, excess energy was available downstream of the stretch where bed material had deposited at the upstream end of the pond, with the result that retrogression occurred accompanied by an increased silt charge; but from 1935 the silt charge decreased, and accretion began, indicating that the available silt had been removed, and that coarser bed material was again moving through the Barrage, a fact borne out by Fig. 5-11.

This leads to the question "has this reduction of volume of silt at Outfall (Sukkur) been wholly due to the Barrage, or has it been partly due to less silt coming from the Punjab?"

The only Punjab data available regarding silt charges are from Kalabagh, just downstream of Attock Gorge (Indus); but the bed at that point consists of pebbles, so the data are not comparable with Sind data. They are, in any case, only available for one year.

THE BEHAVIOUR AND CONTROL OF RIVERS AND CANALS

There is reason to believe, however, that there may be a small progressive reduction in the quantity of silt entering Sind, due to deposition caused by constructing barrages in the Punjab, but the marked reduction of silt at Sukkur since the construction of the Barrage must be mainly due to ponding upstream of the Barrage.

There has been a reduction of discharge in Sind, due to the construction of new canals in the Punjab but the relation between discharge and time was not significant either for Sukkur or for Kotri.

Though the average charge per cubic foot was not significant prior to 1930 at Sukkur, it was significant to 1 per cent level, after 1930, and showed a marked drop with time. In the case of Kotri, on the other hand, there was a slight significance between charge and time prior to 1930; but no significance after 1930. The trend-slope after 1930 was similar to that before 1930. This is in accordance with what was to be anticipated; because though the water was clearer at Sukkur than before water was headed up at the Barrage, the river picked up its natural charge before reaching Kotri—indeed it slightly increased its relative charge, due to the river being contained for longer periods of the year inside its banks than before the Barrage Canals were opened.

5.22. CONCLUSIONS

The enquiry into the causes of breaches in Sind—carried out by the Author in 1942-43—led to the following conclusions, which are applicable to a large extent to all alluvial rivers.

(i) It is inevitable that river flood-levels will continue to rise—increasing the danger of breaches and the damage resulting therefrom. Consequently the margin of safety considered adequate for embankments before a Barrage is built, is not sufficient thereafter.

(ii) There is the threat, indeed the certainty, that an avulsion will occur in Sind at no far distant date unless adequate steps are taken to prevent it.

Fig. 5-4.

(iii) Fig. 5-4, which is a contour map of the whole of Sind (except Khairpur State), shows where the danger will be greatest in years to come. Serious breaches can only occur where there is a natural drainage line through which flood waters can develop a deep channel. The greatest threat is along the Sukkur-Begari Bund on the Right Bank, where the breach occurred in 1942; but there are other danger spots:—

(a) near Kashmir on the Right Bank close to the northern limit of Sind, whence a depression runs along the toe of the Bakti Hills, north of the railway from Kashmir to Jacobabad;

(b) along the Left Bank opposite Kashmir down to south of Ghotki;

(c) along the Kasimpur-Nabishah Bund north of Rohri.

and (d) near Larkana on the Right Bank downstream of Sukkur.

(iv) Assuming Barrage regulation and river conditions to be similar to those up to the end of 1941, specific discharge Gauges at Sukkur for a flood of 500,000 cusecs are likely to rise $3\frac{1}{2}$ ft. above pre-Barrage levels by 1955 and 5 ft. by 1990.

(v) It was predicted that the first appreciable rise of water levels due to heading up at Sukkur might extend upstream at a rate of 2 miles in the first year, but the rate would slow down as the effect extended further upstream.

The considerable heading up at Sukkur seems to have affected the Gauge 23 miles upstream after 11 years; but further data is required to determine whether the effect is permanent.

BEHAVIOUR OF RIVERS

(vi) The rate of rise after the pond effect first reaches a point will be affected both by the width of the river at the point, the distance it is upstream of the Barrage, and the rate of rise of specific discharge Gauges at the Barrage.

(vii) The rate of rise downstream of the proposed Upper Sind Barrage will be considerably less than it has been at Sukkur, due to it being kept open from November to the end of March or longer; and the rise will progress very slowly downstream. The rate of rise above Sukkur as a result of this Barrage will at first be reduced; because the bed material passing down from the Upper Barrage will be less than before.

(viii) Additional Barrages in the Punjab will have two effects:

(a) They will cause a limited amount of sand to be thrown down both upstream and downstream; but this accretion will be less than in Sind where slopes are flatter,

and (b) excluding and ejecting coarse material from the canals will increase the quantity of bed sand, and cause increased primary meandering in the river, with resultant further deposition of bed material upstream and downstream of each Barrage. This will have little effect on conditions in Sind for some years.

If dams are constructed in the Punjab,

(ix) Peak flood discharges will be reduced, and the maximum Gauge readings at Sukkur will also be less, due to the flood absorptive effect of the storage.

(x) The dominant discharge value will be slightly reduced, and fluctuation will be considerably reduced; so that meandering will be less, and the channel will tend to straighten and shorten.

(xi) Bed material will be trapped, with the result that there will be retrogression downstream, and the river gradient will tend to flatten.

(xii) The pond level above the Barrage should henceforth be maintained as low as possible, taking into account irrigation requirements. The importance of this was pointed out in 1942. Success depends largely on lowering the Full Supply levels at the heads of the Rice and N. W. Canals. This will be gradually achieved—as a result of sand being excluded from the Right Bank Canals by the Approach Channel—but can be accelerated, and increased, by removing the coarse bed material from the Heads of the Canals.

(xiii) Attack along the Sukkur-Begari Bund will be somewhat reduced if the proposed Upper Sind Barrage is built near the Left Bank of the river; but what is more important is to “centre” the river just upstream of the Barrage. There are several possible methods of achieving this. In 1943 the Author recommended that experiments should be taken in hand at Poona in this connection; but the greater urgency of the Lower Sind Barrage experiments led to the postponement of this important investigation.

(xiv) A long series of low flood seasons prevent staff from getting experience of breaches, and lull people into a false sense of security. This must be guarded against.

(xv) Officers have much less time to devote to detailed bund inspection than they used to have—when marvels of bund protection were achieved. Now-a-days, officers have to spend much more time dealing with canal work, and a generation of officers is growing up, many of whom do not like touring or tramping about the countryside—to see things for themselves. A cheese-paring policy regarding establishment and Bund maintenance has led to a gradual reduction and deterioration in the quality of the staff nucleus; so

THE BEHAVIOUR AND CONTROL OF RIVERS AND CANALS

that trained personnel are not now available in adequate numbers for work in the flood season. Truly, this policy is penny wise, pound foolish.

(xvi) The breach above Sukkur in 1942, which led to 2,600 square miles of country being flooded, was not due to any shortcoming of the Bund Manual of Orders (published in 1932) which was far in advance of any similar Manual in use elsewhere in India; but a new Manual, explaining conditions and giving instructions in even greater detail, should now be prepared on the lines recommended by the Author.

(xvii) A monograph should be prepared dealing with "predicting water levels at canal heads on data supplies by Punjab and Sind Discharge and Gauge Stations."

(xviii) Dams at the heads of rivers will be advantageous to Sind, in that they will reduce bed material in motion and will also reduce fluctuations of discharge, and hence meandering. There seems to be an idea that though dams will reduce normal floods, they will not reduce the peak discharges of abnormally large floods; this will be true where the dam stores only a small proportion of the flood; but in the case of large dams, the peak outflow discharge is markedly less than inflow during periods when other rivers are in high flood; so they increase the river's flood-absorptive capacity, by delaying run-off. As the main cause of meandering and high river-levels is too much sand, rather than too much water, the effect of barrages, dams, levelling and terracing are all advantageous to Sind provided, of course, they do not interfere with water supplies and supply levels during critical periods.

(xix) All river work should be centralised under one officer, a Chief Engineer, and his work should be mainly confined to Canal Headworks, Barrages and River Control in all its forms. Central control will effect economies in staff and material, by facilitating transfer of staff and materials from places where no immediate danger threatens to places where an attack is likely to develop. This officer should make a much deeper study of river movements than has been attempted in the past.

(xx) River gauging should be a central subject. This is especially necessary in the case of the Indus; because its gauging and control is of vital interest to three Provinces and several Indian States.

(xxi) Complex changes are taking place continuously in rivers, and it is frequently extremely difficult, or even impossible, to be sure of the cause of short-period changes. It is, therefore, exceedingly unwise to assume that any one factor, explains any sudden change. Thus the assumption made that because the specific discharge Gauge at Sukkur rose suddenly in 1942 — more than a year after the Approach Channel had been completed — that, therefore, the rise was due to the Approach Channel was a good example of a "post hoc ergo propter hoc" fallacy.

What has subsequently occurred — a decreased rate of rise in 1943 and 1944 and a marked drop in 1945 — shows that other factors were the true explanation. The enquiry has, however, produced valuable information.

(xxii) The annual minimum bed levels in Rohri Gorge under the Lansdowne Bridge — Fig. 5-11 — showed marked retrogression in 1932 and 1933 — the two years following ponding at Sukkur Barrage. This was followed in 1934 by a sharp rise of 15 ft. after which the rise has decreased steadily and appears to have nearly reached its limit.

Fig. 5-11

BEHAVIOUR OF RIVERS

(xxiii) As was to be expected, the total cross sectional datum area of the river downstream of the Barrage on C. S. XY showed a sudden drop when the width of flow was contracted by the Approach Channel—Fig. 5-12; but in 1943 a steady, and rapid improvement set in, and by the middle of 1944 the datum area was greater than at any time since 1936; and the area of waterway—which showed no decrease even at first—showed a marked increase in 1944. While not justified in assuming that this marked improvement is permanent, the data show fairly conclusively that the Approach Channel has not caused any disimprovement; that, on the contrary, a considerable improvement has taken place.

Fig. 5-12

(xxiv) Rapid, and sometimes violent, changes take place in the specific discharge Gauge readings at Bachalshah—Figs. 5-13 and 5-14—indicating movement of heavy charges of bed material in the form of dunes. The sudden rise of water levels in 1942 appears to have been associated with conditions resulting from the breach, and the effect of water from the breach re-entering the river downstream of the Barrage, the gates of which were kept open.

Fig. 5-13

Fig. 5-14

(xxv) Specific discharge Gauge readings will continue to rise until ponding becomes unnecessary during the flood season (when a heavy charge of bed material is in movement).

(xxvi) Lowering the Full Supply Level in the Right Bank Canals would reduce the ponding required during the flood season and so reduce the rate of rise of specific discharge Gauge readings.

EXCLUSION OF EXCESS BED SAND

CHAPTER 6

EXCLUSION OF EXCESS BED MATERIAL FROM CANALS AND
DISTRIBUTARIES

	Page.
6.00. Introduction	217
6.01. Alternative methods of reducing quantity of bed material passing down a canal	219
6.02. Sand excluders and ejectors	219
6.03. Nature's way	219
6.04. Increase in quantity of sand entering canals resulting from construct- ing weirs and barrages	220
6.05. Alternative methods for excluding sand from canals taking off up- stream of a weir or barrage	220
6.06. Reduction of sand entry by means of submerged concave vane— originally proposed for the Right Bank Canals at Sukkur	220
6.07. Reduction of sand entry by means of an Approach Channel—recom- mendations made in 1939 regarding sand exclusion from the Right Bank Canals at Sukkur	231
6.08. Effect of constructing the upper 1,600 ft. of outer bank of the Approach Channel...	233
6.09. Effect of completed Approach Channel	241
6.10. Alternative methods of excluding excess sand from canals	242
6.11. High level sill at canal head	242
6.12. Divide wall	242
6.13. Open-type excluders at heads of branches and distributaries	242
6.14. Exclusion of excess sand from the Mithrao Canal	245
6.15. (a) Exclusion by means of a projection downstream of a canal	245
6.16. (b) Exclusion by means of raised sills projecting into the parent channel with a warped convergence upstream	246
6.17. (c) Exclusion by means of a submerged vane	246
6.18. (d) King's vanes	246
6.19. (e) Divide wall excluder	257
6.20. (f) Approach Channel taking off from outside of a bend	258
6.21. Permanent effect of temporary inflow of sand on regime of canal	266
6.22. Distribution of bed-load between off-takes	266
6.23. Tunnel-type excluders at heads of canals	267
6.24. Tajewala tunnel-type shingle excluders	267
6.25. F. F. Haigh on excluders	267
6.26. Ejectors	268
6.27. Haigh's opinion on advantages and disadvantages of excluders and ejectors	275
6.28. Additional difficulties of exclusion where canals take off from both banks	275
6.29. Where the ratio of discharge of off-take to that of parent channel is high, ejectors may be the only solution	275
6.30. Conclusions	275

EXCLUSION OF EXCESS BED SAND

Sheet 6-1

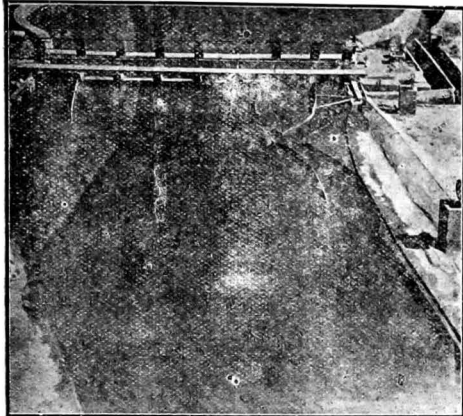


Photo 2342. Model: 1/300 scale model. Looking downstream, showing the new Right Bank Approach Channel and the existing Left Bank pocket with long Divide Wall.

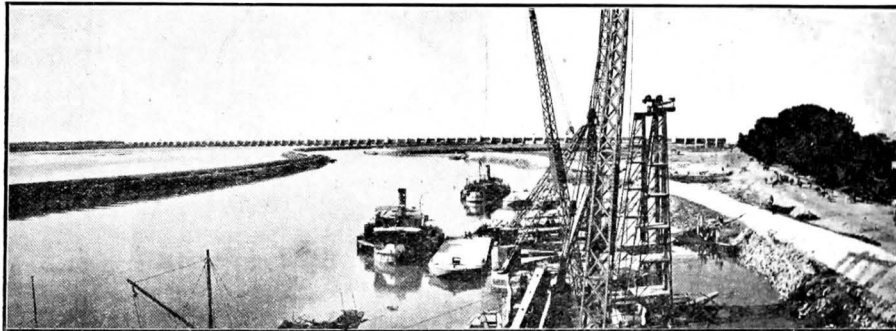


Photo 2641. Sukkur Barrage in distance with Right Bank Approach Channel in foreground, looking downstream from a point 500 ft. from entrance to Approach Channel and 2,000 ft. upstream of the point whence the Approach Channel can be seen to bifurcate into the tail channel (left) and right pocket, leading to R. B. Canals. Water level when photo taken was R. L. 185.

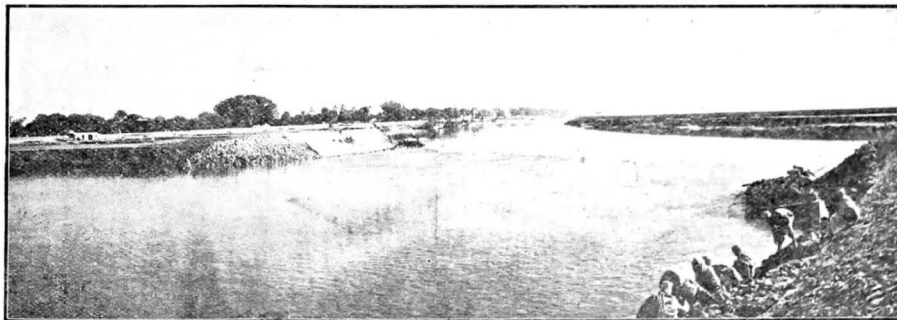


Photo 2640. New R. B. Approach Channel, looking upstream from a point just inside the right pocket when water level was at R. L. 185.0 (normal water level = 194.5). Note imposed curvature of Approach Channel and high sill at entry to pocket at R. L. 181.6 both of which tend to exclude bed sand. The bank on the right, in the middle distance, is the outer bank to the Approach Channel, which continues down to the Barrage. The total length of the Approach Channel is 5,000 ft.

PHOTOS FROM C. I. AND H. R. STN. POONA A. R. (T) 1941-42.

Makhi
Regulator

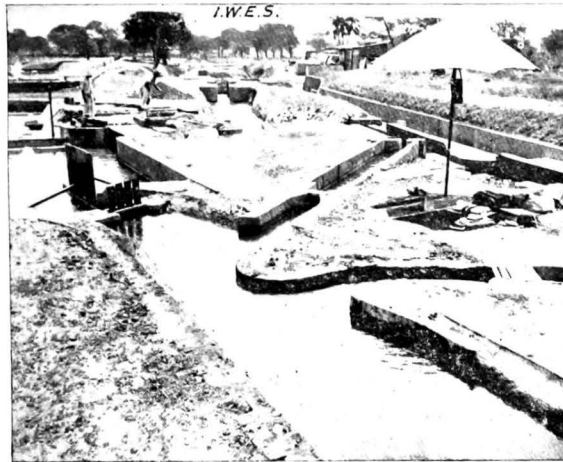


Khipro Canal

Mithrao
Canal

Photo 838. Experiment No. 2 (looking downstream). A spur projecting to midstream was built at the downstream end of the Mithrao to create curved flow to exclude bed sand.

Makhi
Regulator



Khipro
Canal

Mithrao
Canal

Photo 855. Experiment No. 3; 1 in 5 convergence upstream and a spur projecting to midstream, downstream of the Mithrao, (Looking downstream).

Makhi
Regulator



Khipro Canal

Mithrao
Canal

Long raised
sill with 1
in 5 conver-
gence up-
stream

Photo 865. Experiment No. 4: 1 in 5 convergence upstream of the Mithrao and a long raised sill in front of the Mithrao and Khipro Canals. (Looking downstream).

PHOTOS FROM BOMBAY T. P. No. 59.

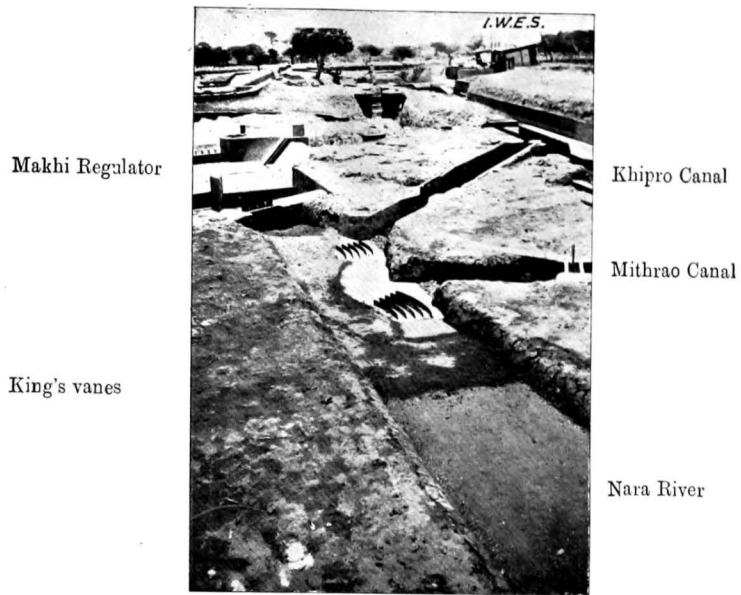


Photo 890. Experiment 7 (a): King's vanes opposite Mithrao and Khipro Canals. (Looking downstream).

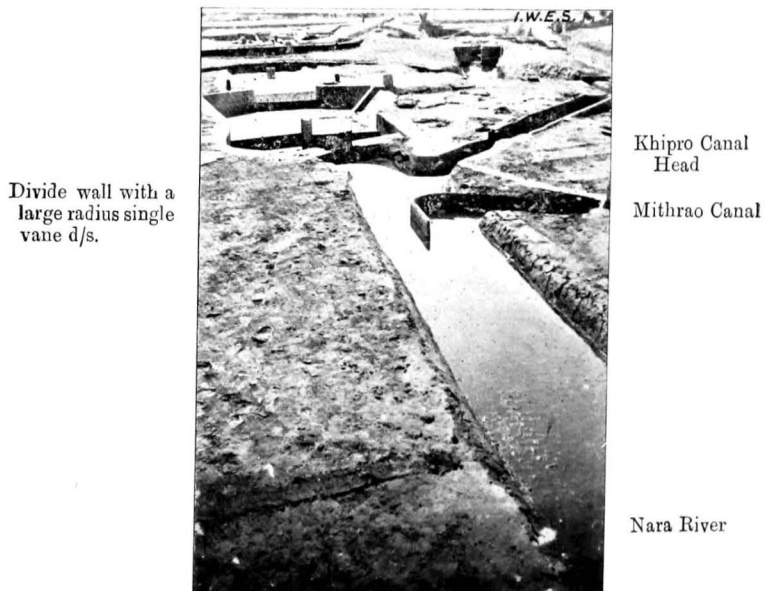


Photo 888. Experiment 8 (a): Divide wall opposite Mithrao and a large radius single vane in continuation of the divide wall at the downstream end to exclude bed sand from Khipro Canal.

PHOTOS FROM BOMBAY T. P. No. 59.



1409. Showing the Eastern Nara on the left and the approach channel to the Mithrao on the right, shortly after construction. (Looking downstream.)

PHOTO FROM BOMBAY T. P. No. 59.

DISTRIBUTARY 5 MODEL OF THE EASTERN NARA AT MAKHI

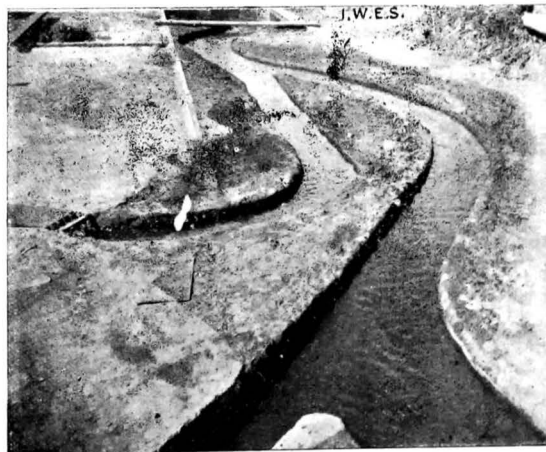
Experiment 10: With part of the old River channel opened to carry water to the Khipro Canal and the Makhi Regulator and the existing straight channel reduced in width to serve as an approach channel to the Mithrao Canal.



Makhi Regulator
Khipro Canal
Mithrao Canal

883. Looking downstream.

Q of Nara above Makhi Regulator = 2.8 cusecs (\approx 7,120 cusecs)
Q of Mithrao = 0.84 cusecs (\approx 2,200 cusecs)
Q of Khipro = 0.56 cusecs (\approx 1,420)



Mithrao
Canal Head

Khipro Canal Head Makhi Regulator
882. Looking upstream.

PHOTOS FROM BOMBAY T. P. No. 59.

CHAPTER 6

EXCLUSION OF EXCESS BED MATERIAL FROM
CANALS AND DISTRIBUTARIES

6.00. INTRODUCTION

The quantity of bed material in movement in a stream depends on the quantity of material washed into the stream. This is a self-evident truth; yet—somewhat surprisingly—it has generally been overlooked in India since Kennedy introduced his formula

$$V_0 = .84 D^{.64}$$

— a formula in which charge was not taken into account.

Later, when Lacey produced his formulae

$$V = 1.155 \sqrt{fR} \quad P = 8/3 \sqrt{Q}$$

which, like Kennedy's formula, did not take charge into account—because it was based on regime conditions in canals—the practice of ignoring charge became almost general. Though Lacey formulae are not directly applicable to alluvial rivers, in which bed movement is greatly in excess of regime charge; the formulae have proved to be very useful for qualitative analysis of river behaviour (see Chapter 2). Indeed Lacey went so far as to state*

“It is thus found that fine silt with a heavy surcharge may well demand the same value of V^2/R as coarse silt with a smaller charge.”

but later work has shown that excess charge has different effects from an increase in grade.

This was not at first obvious; because no direct way of measuring bed charge had then been worked out—nor indeed has yet been worked out—so that the exact causes of scatter are still not clearly known. As a result, people have been apt to overlook the fact that divergences are not entirely due to grade, but are also markedly affected by the charge, especially as regards the depth to width relation, the degree of tortuosity, the distribution and number of channels, and the discharge at which spill occurs—all of which are largely determined by the charge.

As already explained in Chapter 3: where the quantity of material in movement is small, conditions approximate to what has come to be called ‘regime’ in India, the section then being relatively deep and narrow, and the slope small; but where, as in most alluvial rivers, a very heavy charge of bed material is in movement; they are wide and shallow, with a steep gradient, and flow takes place in two or more meandering channels, or in several interlacing streams. Though rivers must carry the greater part of their load to the sea, the remainder spilling over their banks to raise the country level, canal heads should be so designed that the charge they allow to pass into the canals is reduced to the regime charge.

The reason for this is threefold:—

- (a) The regime charge is a function of the dominant discharge; so it is greater in the river than in a canal.
- (b) The charge in the river at dominant discharge exceeds the regime charge.
- (c) Super-dominant discharges in the river carry super-dominant charges.

The combined effect of these three causes necessitates a large proportion of the coarser fractions of the river charge being excluded. This is especially important during floods.

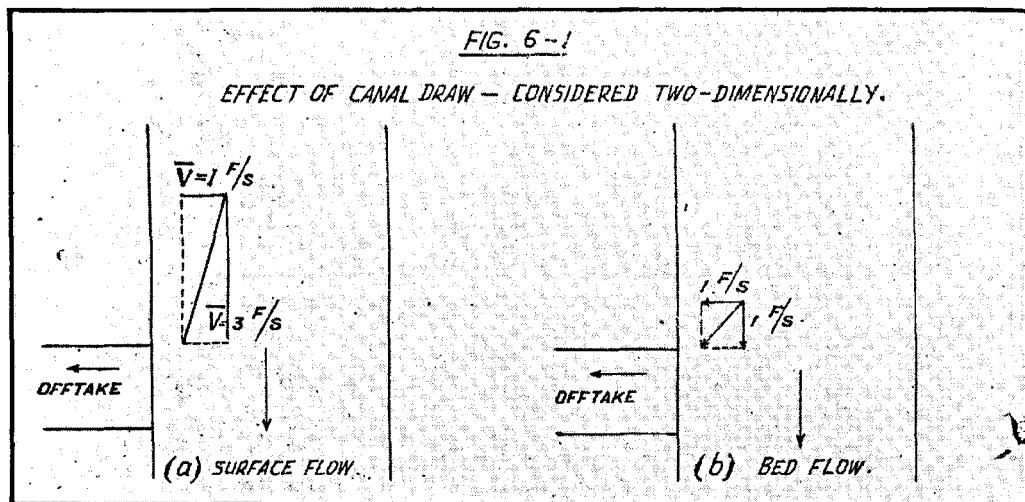
* Discussion by Gerald Lacey on “Stable Channels in Alluvium” by E. W. Lane. Paper No. 1937 Am. Soc. C. E. p. 161.

THE BEHAVIOUR AND CONTROL OF RIVERS AND CANALS

Thirty years ago the charge was considered to be uniform across the bed of a channel*; but we now know that in straight reaches the charge decreases from midstream towards the banks.† From this it follows that where flow in a parent channel is axial, a small channel taking off over a raised sill does not draw an excess charge; but where the off-taking channels draw a considerable proportion of the parent channel discharge, unfavourable curvature of flow develops towards the offtake, and a considerable proportion of bed material is drawn into the canal.‡

It is desirable to visualise the reason for this: For simplicity let us first consider two-dimensional resolution of the velocities in the river and towards the canal

Fig. 6-1
(a) and (b)



(Fig. 2 and 3 from Bombay. T. P. No. 52)

(a) shows that though the high-velocity surface-water is little affected by the 'draw' of the canal, the bed filaments containing the coarser particles are considerably deflected. These figures also make it easy to see that the proportion of bed-material drawn by a canal increases as the proportion of discharge draw-off increases.

When the question is considered as a three-dimensional problem—as it actually is—it is found that, as little top water can enter the upper spans of the Head Regulator, most of the water entering these spans must be from the bed. Thus, the draw-off of the canal induces curved flow, which tends to sweep the bed material towards, and into the canal.

* "Distributary offtakes in relation to silt conditions" by F. V. Elsdon. Procs. Punjab Eng. Congress, 1916.

† C. B. I. Annual Report, 1942.

‡ Bombay P. W. D. T. P. Nos. 45 and 46. "Silt exclusion from canals" Parts I and II.

Bombay P. W. D. T. P. No. 52. "Factors affecting exclusion of bed silt from canals taking off from alluvial rivers, etc."

Bombay P. W. D. T. P. No. 59. "Silt Control at heads of canals and distributaries." by C. G. Inglis and D. V. Joglekar.

EXCLUSION OF EXCESS BED SAND

6.01. ALTERNATIVE METHODS OF REDUCING QUANTITY OF BED MATERIAL PASSING DOWN A CANAL

Three methods can be employed to prevent excess bed material passing down a canal:—

(i) The excess can be trapped near the head of the canal to be later removed by hand or machinery;

(ii) sand can be excluded at the head, i.e. not allowed to enter; or (iii) sand can be ejected after it has entered the canal.

(i) Where the material in movement consists of a very heavy charge of fine material, little of the excess can be excluded; so settling tanks have been provided to trap the excess silt, the deposited material being later removed from one set of settling basins, while the water required for irrigation is passed through another set. This method has not so far been followed anywhere in India; but an analogous practice was followed for many years in Sind, silt deposited in inundation canals during a flood season being removed by hand from the beds of inundated canals before the ensuing flood season. This was done in the belief that it would make it possible to pass appreciably larger discharges down the canals in the early part of the season—when the level of water in the river was low—and that a larger discharge would be available at peak demand. When, however, this practice was discontinued in some channels, it was found that there was little reduction of discharge, and that better supplies could be obtained at much less cost by remodelling the canals.

6.02. SAND EXCLUDERS AND EJECTORS

The object of a sand excluder is to prevent the coarser bed materials in movement in a river during floods, from entering the head of a canal; whereas an ejector is used to eject coarser particles after they have entered a canal. The excluders may be looked upon as a method of prevention of accretion, and ejectors as a cure; and where practicable, prevention is better than cure.

Though the use of the term 'excluder' has been largely restricted to tunnel-type excluders,* there is no justification for this restriction, and it is extremely undesirable that it should be allowed to harden into general usage; because suitably-designed open-type excluders are, in a large majority of cases, superior to tunnel-type excluders.

6.03. NATURE'S WAY

The earliest type of excluder, which is still the best type to use where conditions are suitable, is based on "nature's way"—the off-taking channel being located towards the downstream end of the outer bank of a concave bend, a little downstream of where diving flow, after reaching the bed near the bank, is diverted towards the opposite convex bank, thus sweeping the bed-load away.

Fig. 6.2.

It seems probable that this method of excluding sand from inundation canals has been followed from the dawn of irrigation; and it is only since permanent structures, like weirs and barrages, have been built, that this method of exclusion has begun to be forgotten; but it is still the standard method followed in excluding sand from inundation canals in Sind. There is, however, a tendency for meander embayments to move downstream; so that a canal head which has excluded sand for many years may, in the course of time, begin to draw bed-sand, when the curvature of flow becomes unfavourable. Under such conditions, the usual practice is to open a new Head, which may be some miles upstream, or downstream. The Begari Canal is a good example of a canal which has had many heads.

* C. B. I. Journal July 1945, p. 105.

THE BEHAVIOUR AND CONTROL OF RIVERS AND CANALS

6.04. INCREASE IN QUANTITY OF SAND ENTERING CANALS, RESULTING FROM CONSTRUCTING WEIRS AND BARRAGES

When weirs were built in order to bring higher lands 'under command'; and later, when barrages were built to maintain water at any desired level; these structures had the effect of fixing the position of the heads of the canals, and where barrages have been built at an unsuitable point—generally in order to save on cost of construction, but sometimes through sheer ignorance—sand exclusion has been highly unsatisfactory. In other cases, though exclusion has been satisfactory at first, it has become unsatisfactory later—due to the river changing its course.

Though a weir or barrage may not give rise to unfavourable conditions when canals take off from only one bank, they always create unfavourable conditions on one or other bank.

6.05. ALTERNATIVE METHODS FOR EXCLUDING SAND FROM CANALS TAKING OFF UPSTREAM OF A WEIR OR BARRAGE

The recent Punjab practice has been to construct closed type excluders or ejectors—but a new Head, taking off further upstream where curvature was satisfactory, would often set matters right. This has rarely been done—either because it would have been an admission of a mistake, or because it would entail acquiring land and interfering with existing rights. To avoid such difficulties, the Author recently proposed acquiring land for an alternative Head and channel before the Kalri Canal was constructed upstream of the proposed Lower Sind Barrage at Hajipur—in case it should later be found desirable to change the Head.

Two other methods of excluding excess sand have been recommended by the Poona Station in connection with the Right Bank Canals at Sukkur. Both these methods were based on the same principle—curvature of flow. In this case, the canals take off from inside (convex, bank of a bend. In 1927, five years before the Barrage was completed) it was predicted that unless steps were taken to modify the design the canals would silt heavily, and a submerged concave vane was proposed to prevent this.

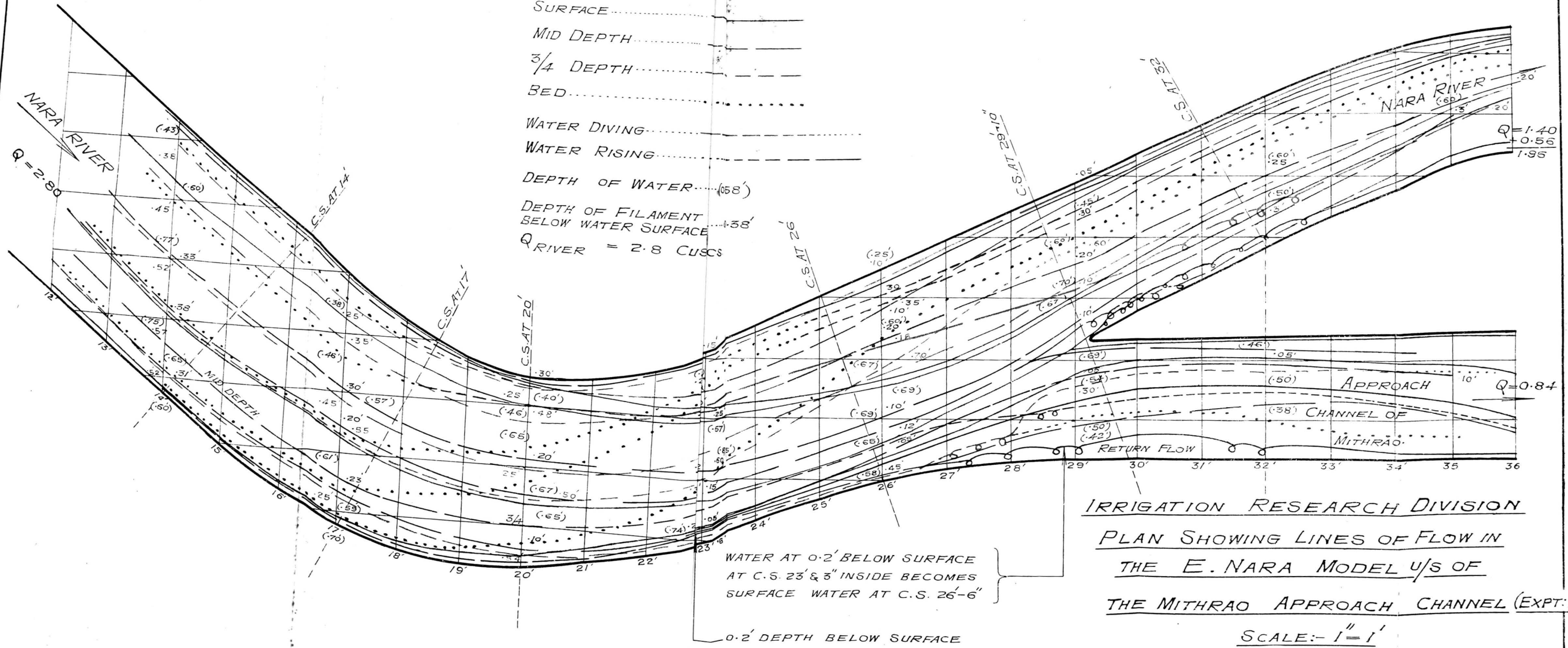
6.06. REDUCTION OF SAND ENTRY BY MEANS OF A SUBMERGED CONCAVE VANE—ORIGINALLY PROPOSED FOR THE RIGHT BANK CANALS AT SUKKUR

Model experiments undertaken at Poona to verify this prediction showed that it was correct, and that accretion could be prevented provided a long, concave, vane were constructed upstream of the Right Bank Canals—to induce favourable curvature of flow. The model showed, however, that this vane would attract too much flow towards the right bank; and that, to neutralise this, a left bank vane should be built—see Fig. 6-3—at the point where the river begins to expand just

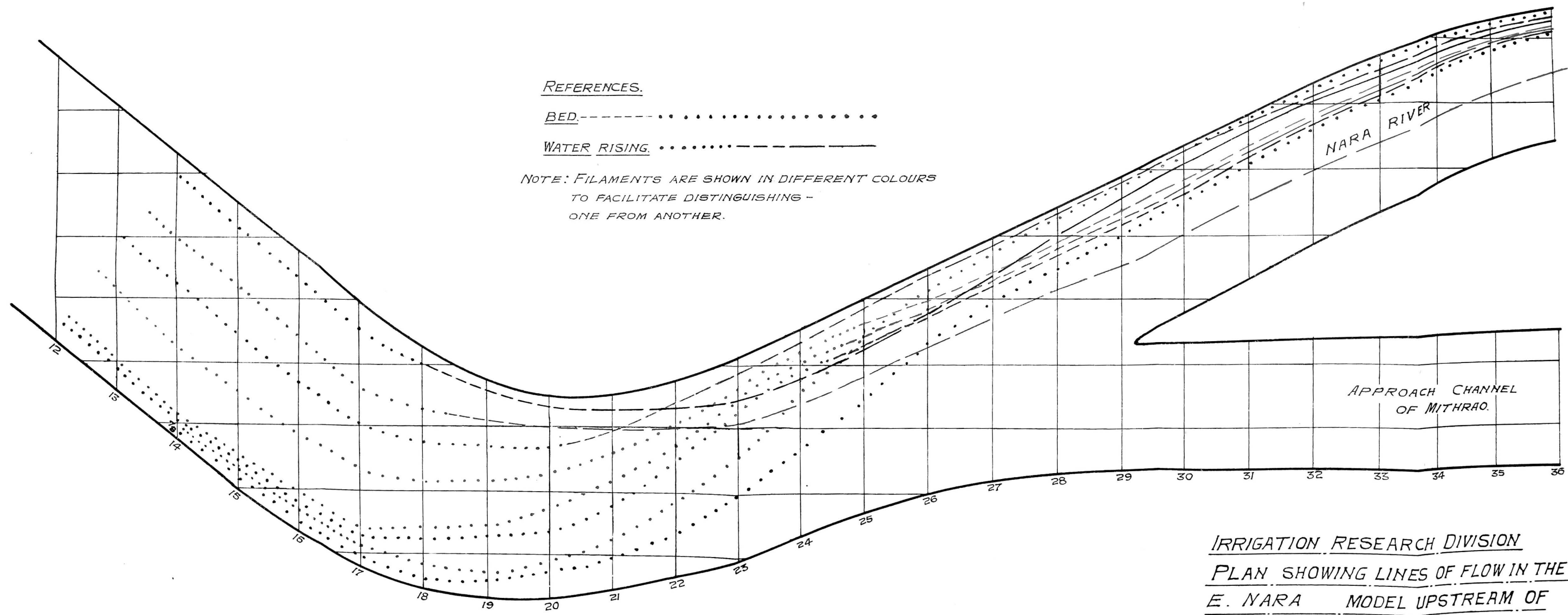
Fig. 6-3

REFERENCE

- SURFACE.....
- MID DEPTH.....
- 3/4 DEPTH.....
- BED.....
- WATER DIVING.....
- WATER RISING.....
- DEPTH OF WATER.....(0.58')
- DEPTH OF FILAMENT BELOW WATER SURFACE.....0.38'
- Q RIVER = 2.8 CUSCS



IRRIGATION RESEARCH DIVISION
 PLAN SHOWING LINES OF FLOW IN
 THE E. NARA MODEL U/S OF
 THE MITHRAO APPROACH CHANNEL (EXPT. 10)
 SCALE: - 1" = 1'



REFERENCES.

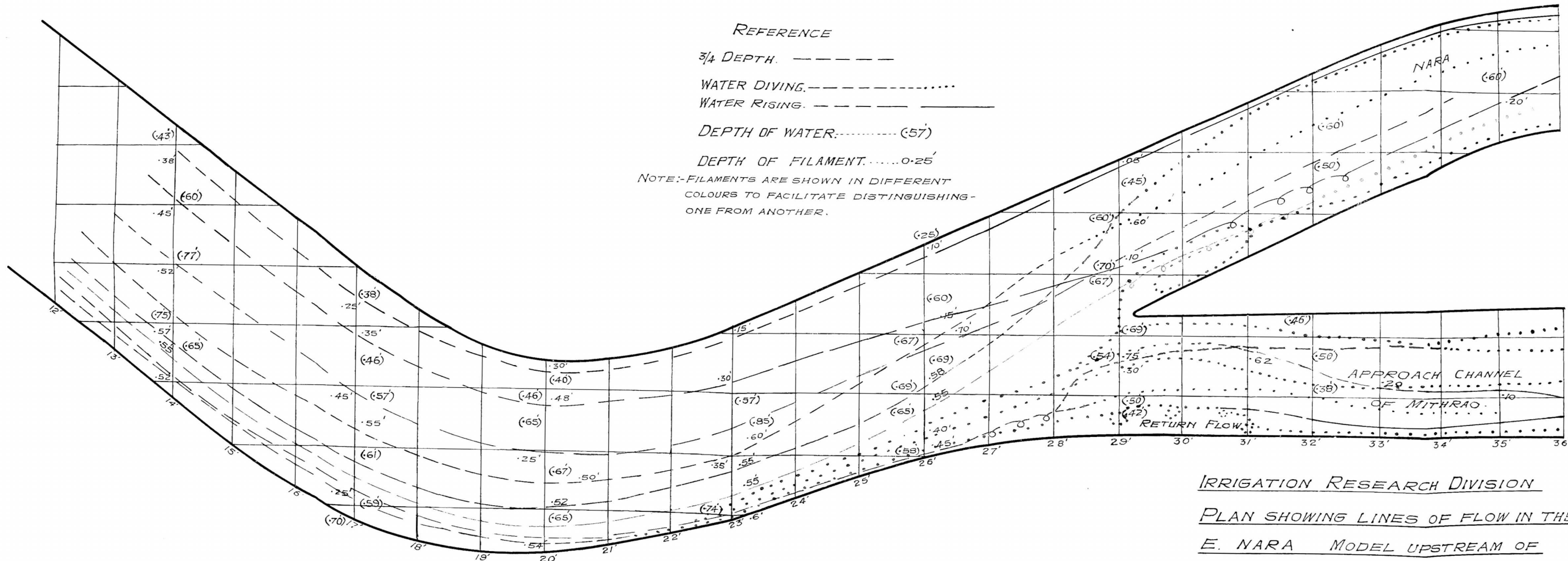
BED.-----

WATER RISING.-----

NOTE: FILAMENTS ARE SHOWN IN DIFFERENT COLOURS
TO FACILITATE DISTINGUISHING -
ONE FROM ANOTHER.

IRRIGATION RESEARCH DIVISION
PLAN SHOWING LINES OF FLOW IN THE
E. NARA MODEL UPSTREAM OF
THE MITHRAO APPROACH CHANNEL. (EXPT. 10)

SCALE. 1 INCH. TO 1 FT.



REFERENCE

3/4 DEPTH. -----

WATER DIVING.

WATER RISING. -----

DEPTH OF WATER. (57)

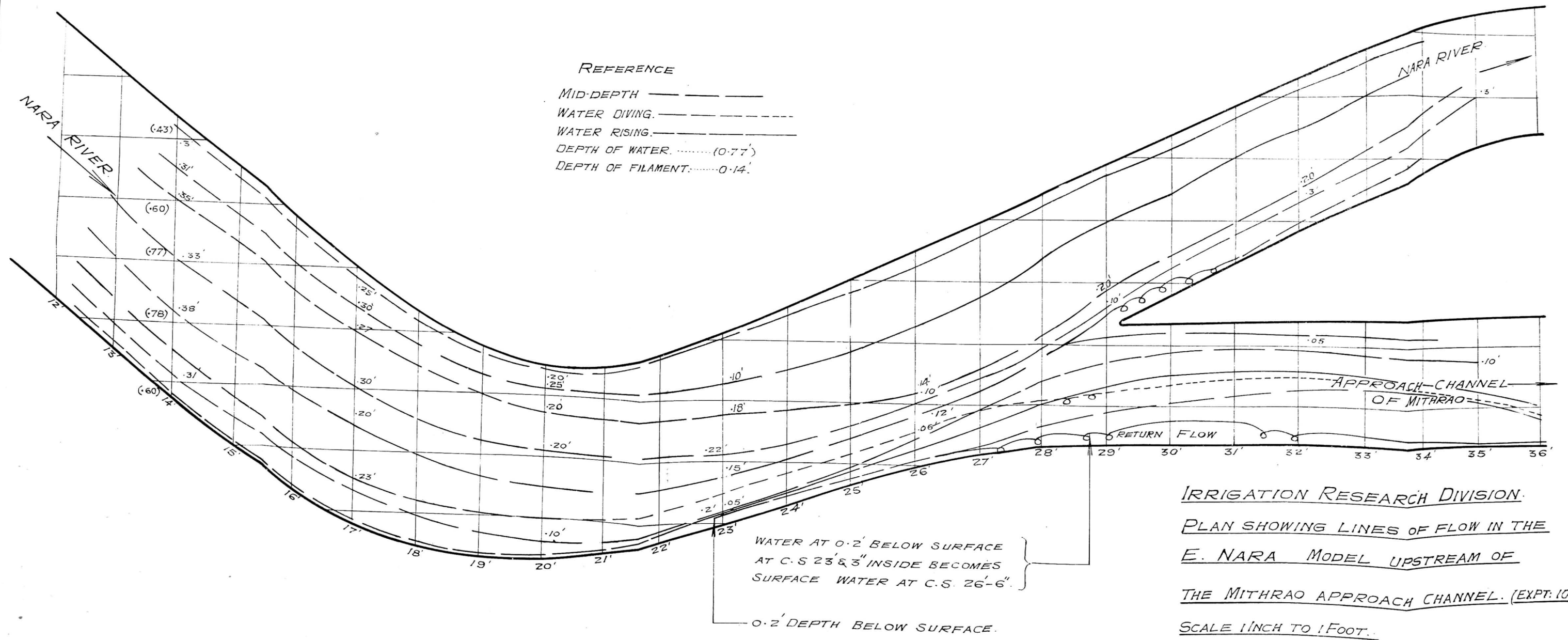
DEPTH OF FILAMENT. 0.25'

NOTE:-FILAMENTS ARE SHOWN IN DIFFERENT COLOURS TO FACILITATE DISTINGUISHING-ONE FROM ANOTHER.

IRRIGATION RESEARCH DIVISION
 PLAN SHOWING LINES OF FLOW IN THE
 E. NARA MODEL UPSTREAM OF
 THE MITHRAO APPROACH CHANNEL. (EXPT.10)
 SCALE 1 INCH TO 1 FOOT.

REFERENCE

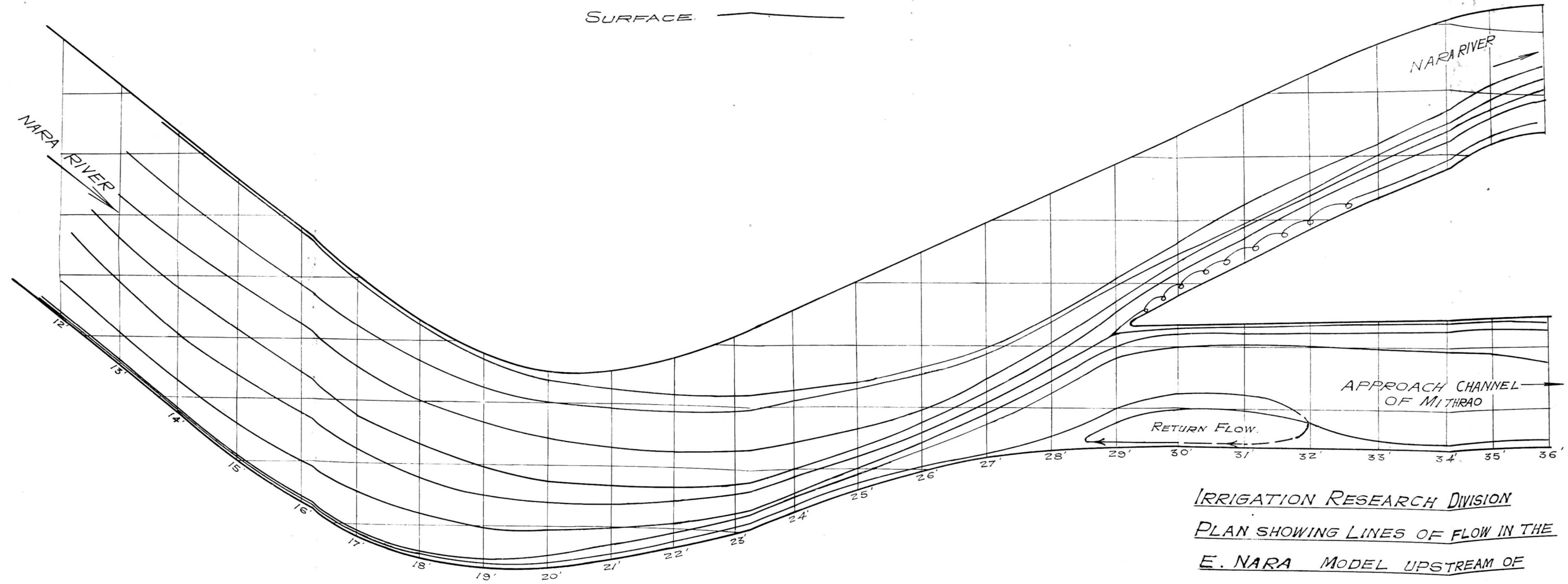
- MID-DEPTH ————
- WATER DIVING. ————
- WATER RISING. ————
- DEPTH OF WATER. (0.77')
- DEPTH OF FILAMENT. 0.14'



IRRIGATION RESEARCH DIVISION.
 PLAN SHOWING LINES OF FLOW IN THE
 E. NARA MODEL UPSTREAM OF
 THE MITHRAO APPROACH CHANNEL. (EXPT. 10)
 SCALE 1 INCH TO 1 FOOT.

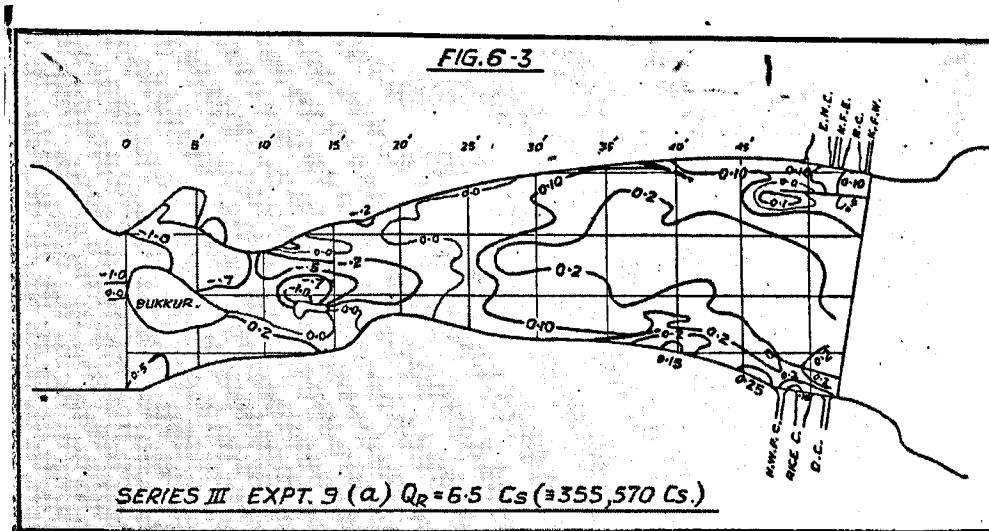
FIG. 6-2 (d)

REFERENCE
SURFACE. ———



IRRIGATION RESEARCH DIVISION
PLAN SHOWING LINES OF FLOW IN THE
E. NARA MODEL UPSTREAM OF
THE MITHRAO APPROACH CHANNEL. (EXPT. 10)
SCALE 1 INCH TO 1 FOOT.

EXCLUSION OF EXCESS BED SAND



upstream of Bunder section. Subsequently it was found that a much shorter vane would have the same effect. This, it was found, would have the effect of maintaining a suitable proportion of flow along the left bank, while at the same time permitting sand control along the right bank.

This method was not adopted by Sind, on the grounds that the conclusions drawn from the model might be wrong; and that it would be wise to "wait and see" if the prediction were correct.

6-07. REDUCTION OF SAND ENTRY BY MEANS OF AN APPROACH
CHANNEL—RECOMMENDATIONS MADE IN 1939 REGARDING
SAND EXCLUSION FROM THE RIGHT BANK CANALS
AT SUKKUR

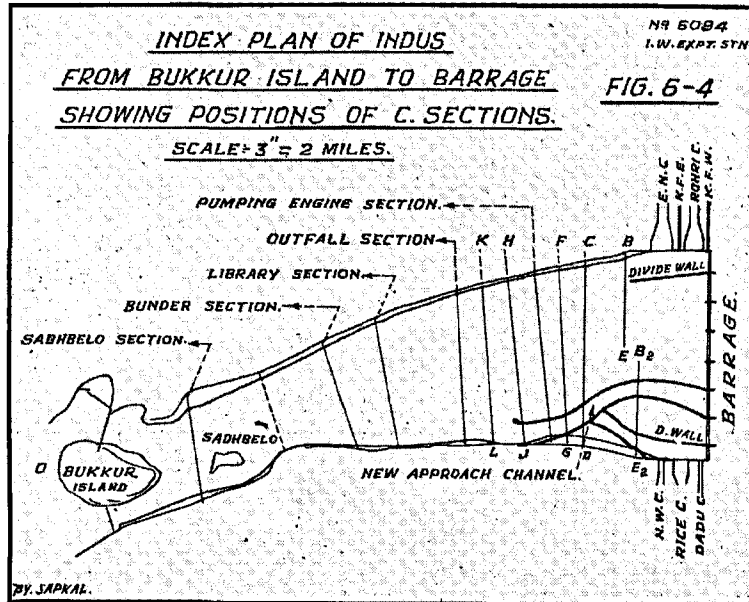
When the Sukkur Barrage was first closed in 1932—to head up water for the canals—retrogression of the downstream bed and water levels occurred as explained in 5.09, and it was not until 1935 that accretion began to take place in the river. The rise was then rapid and during 1938 the Rice and North Western Canals, which take off from the Right Bank above the Barrage, silted heavily—to a depth of more than 5 ft.*

The matter was then referred to the Author for solution. By that time the original proposal—to construct an excluder vane along the Right Bank—could not be carried out; because heavy silting had occurred along the Right Bank where it was originally intended to construct it. Four alternative methods for excluding excess bed material were then tested at Poona; but only the one selected will be described. This is shown in Photo Sheet 6-1 and Fig. 6-4. It consists of an Approach Channel, 5,000 ft. long†, in which favourable curvature of flow is induced in order to prevent excess sand passing over the raised sill provided at the entrance to the Right Bank pocket.

Photo Sheet
6-1.
Fig. 6-4.
†
Fig. 6-5.

* C. I. & H. R. S. A. R. (T) 1943 p. 33.

† C. I. & H. R. S. A. R. (T) 1937-38 pp. 26-27, 1938-39, pp. 9-10.
1939-40 pp. 13-17, 1940-41, pp. 12-16.
1941-42 pp. 16-18.



6.08. EFFECT OF CONSTRUCTING THE UPPER 1,600 FT. OF OUTER BANK OF THE APPROACH CHANNEL

Fig. 6-6(a) and (b) The effect of the upper 1,600 ft. was very great—compare Fig. 6-6(a) and (b) and the Chief Engineer in Sind suggested that the rest of the work should be omitted. To this the Author would not agree, on the grounds that the river had been drawn too much towards the right bank and that the channel along the left bank had almost disappeared—as can be clearly seen in Fig. 6-6(b), so that without the full length of the outer bank of the Approach Channel, control of the river by the Barrage gates, would be small.

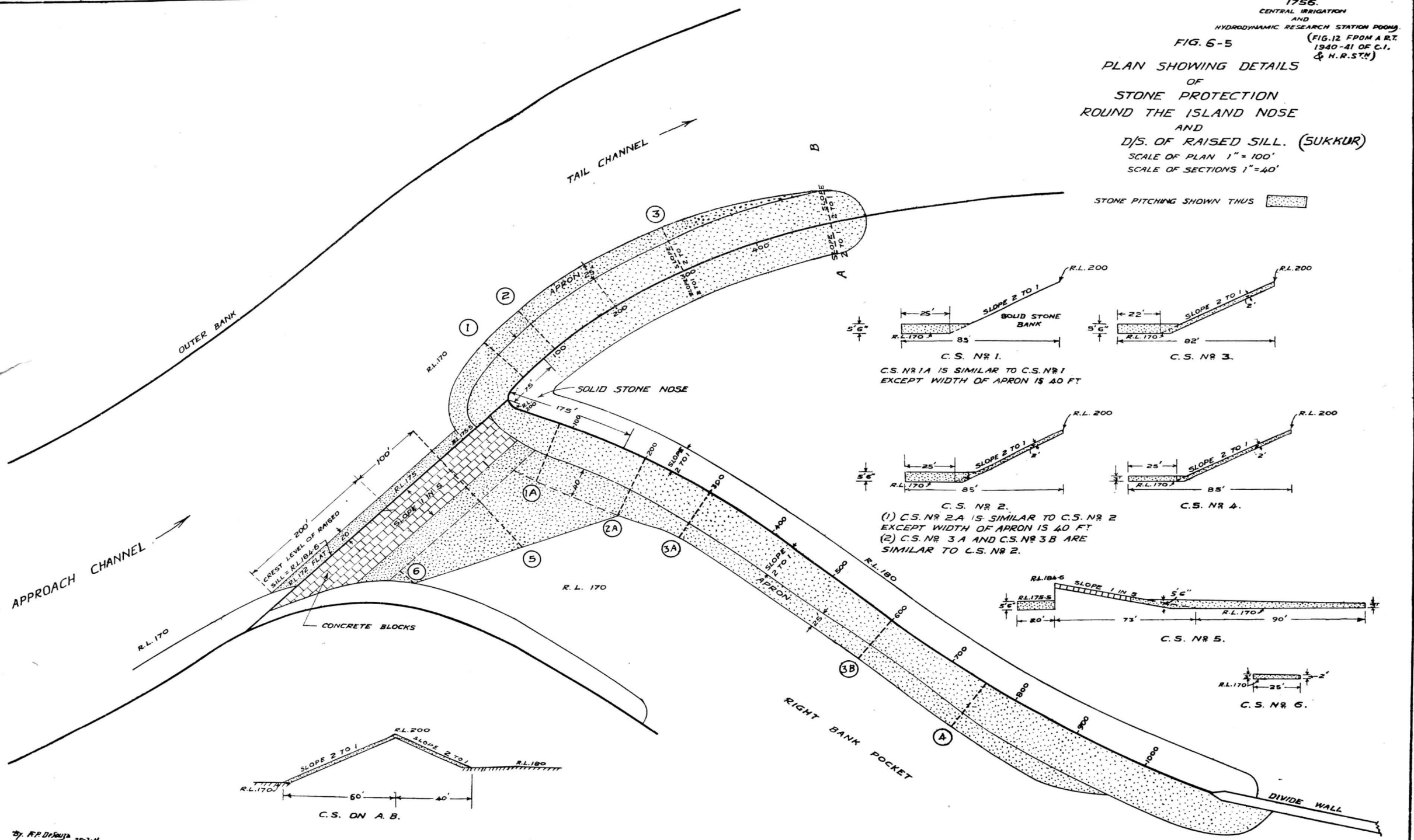
The tremendous draw of the truncated outer bank is, however, of great interest in that it exemplifies the possible danger of constructing such works. If there had been no Left Bank Canals, the 1,600 ft. of bank would have been highly satisfactory, so far as the Right Bank Canals were concerned, but it would have tended to alter the course of the river above the Gorge and downstream of the Barrage. The alterations above the Gorge would have been towards the east and would probably have necessitated the re-alignment of the main line of the North Western Railway, north of Rohri; and it would also probably have led to deep erosion along the Left Bank downstream of the Barrage in Khairpur State; so detailed model experiments would have been required with a 70 mile length of the river to investigate this aspect of the question.

After the whole of the outer bank had been completed flow was brought under control; see Fig. 6-6(c) for June 1944.

FIG. 6-5

(FIG. 12 FROM A.R.T.
1940-41 OF C.I.
& H.R.S.M.)

PLAN SHOWING DETAILS
OF
STONE PROTECTION
ROUND THE ISLAND NOSE
AND
D/S. OF RAISED SILL. (SUKKUR)
SCALE OF PLAN 1" = 100'
SCALE OF SECTIONS 1" = 40'



By: R.P. D... 25-3-M.

FIG. 6-7 (a)

1/80 SCALE PART-WIDTH SUKKUR BARRAGE MODEL.
 PLAN SHOWING CONTOURS & EDGE OF APRON AS LAID
 & AS LAUNCHED WITH $Q = 0.7 \text{ CS.} \equiv 18,000 \text{ CS.}$ IN TAIL CHANNEL AND
 $Q = 0.7 \text{ CS.} \equiv 18,000 \text{ CS.}$ IN R.B. POCKET.
 $Q = 1.4 \text{ CS.} \equiv 36,000 \text{ CS.}$ IN APP. CHANNEL.
 SCALE. $1'' = 4' \equiv 320'$

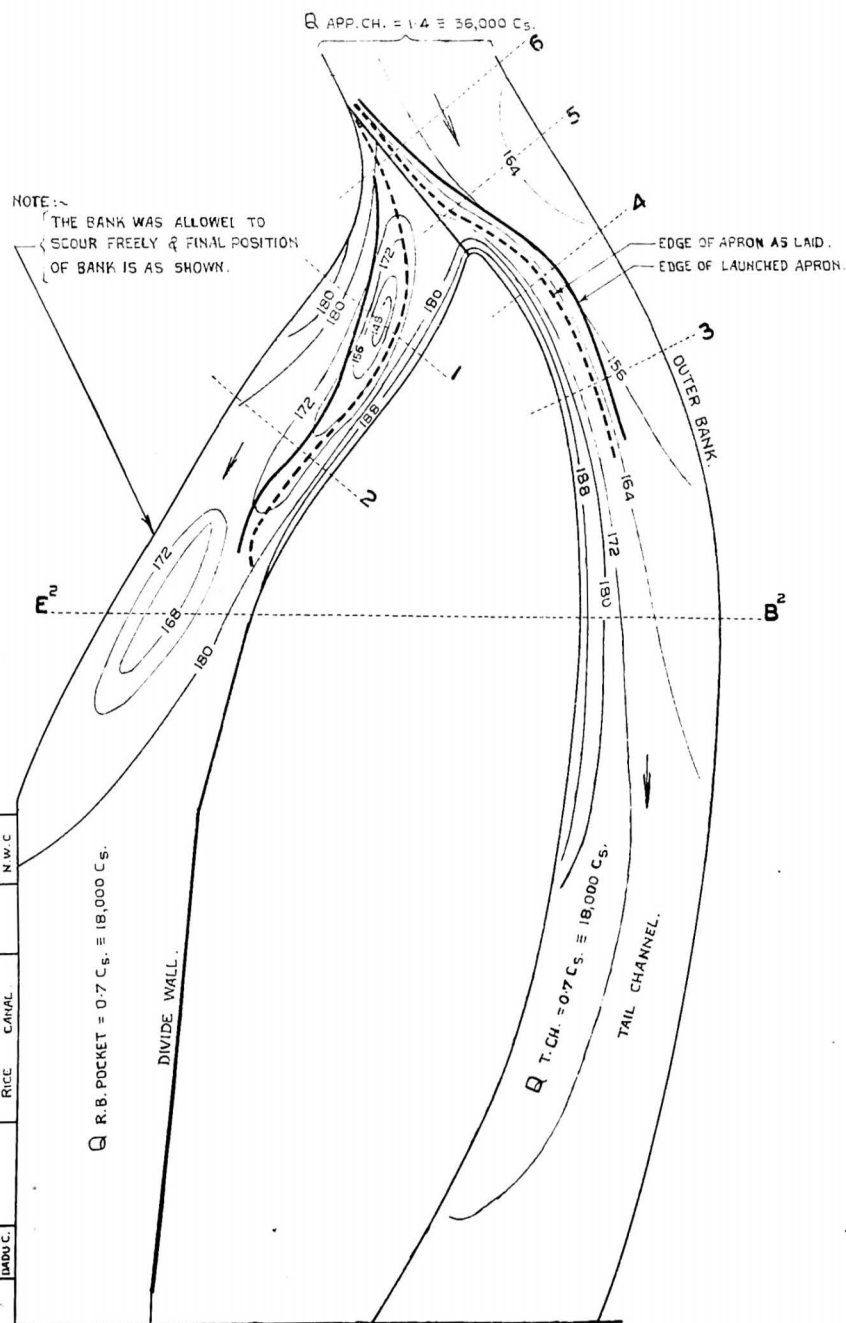
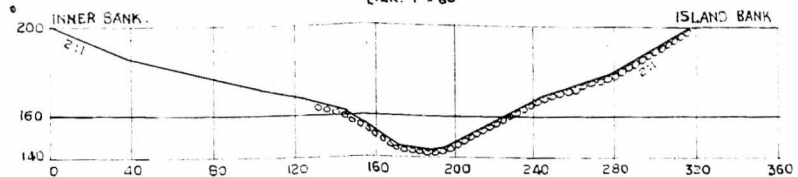


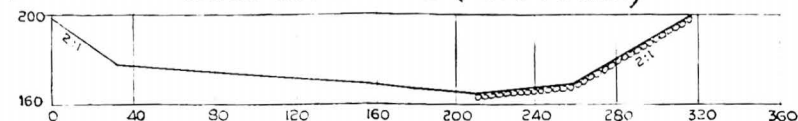
FIG. 6-7 (b)

CROSS SECTION NO. 1. (RIGHT POCKET)

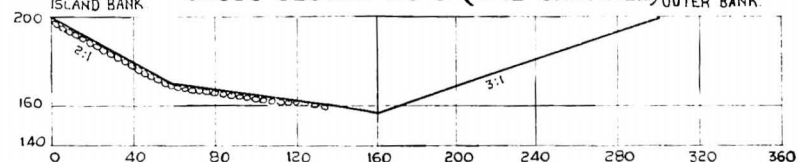
SCALE :-
 HOR. $1'' = 60'$
 VER. $1'' = 60'$



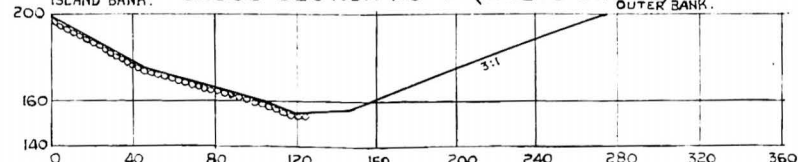
CROSS SECTION NO. 2. (RIGHT POCKET)



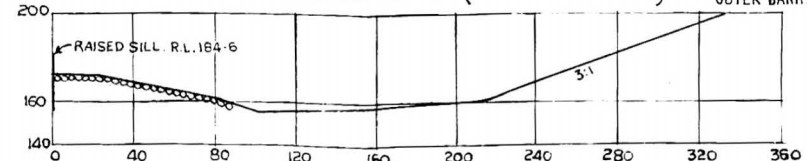
CROSS SECTION NO. 3. (TAIL CHANNEL)



CROSS SECTION NO. 4. (TAIL CHANNEL)



CROSS SECTION NO. 5. (ON RAISED SILL)



CROSS SECTION NO. 6. (ON RAISED SILL)

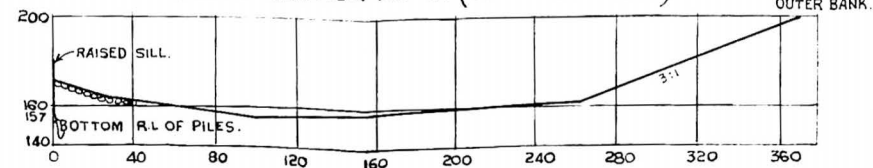
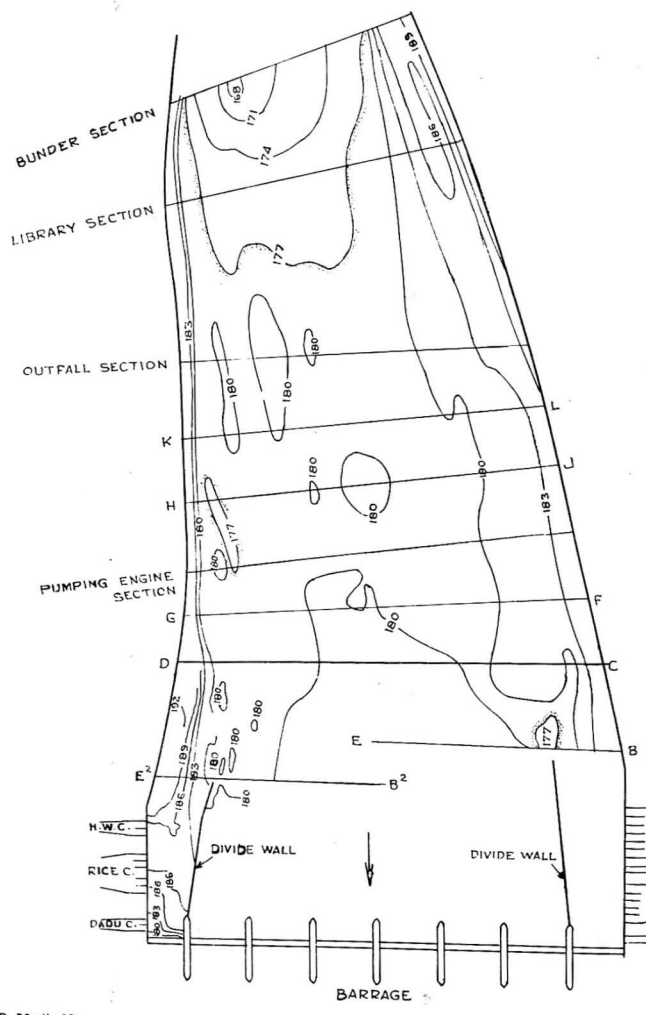
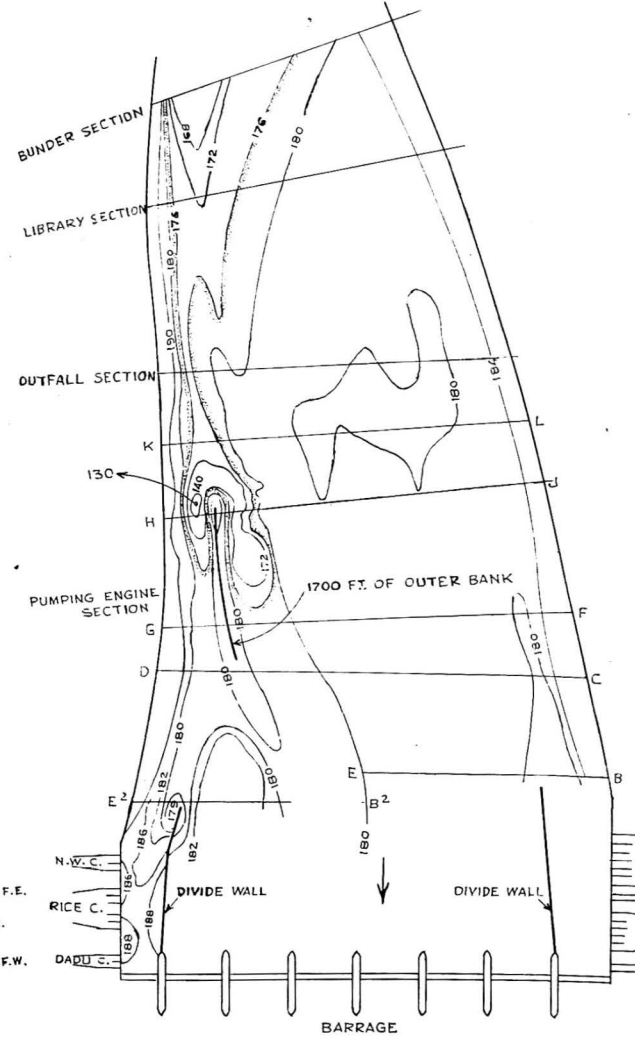


FIG. 6-6 (a, b, c.)
CONTOUR PLANS OF THE INDUS
AT SUKKUR
BARRAGE SILL 177.
SCALE 1"=1200'

(a)
5-7-1939
Q_R = 362,000 Cs.



(b)
19-7-1940
Q_R = 325,000 Cs.
(WITH 1700' OUTER BANK FROM NOSE)



(c)
12-6-1944
Q_R = 171,163 Cs. & Q_{AC} 34,844 Cs.
(WITH APPROACH CHANNEL)

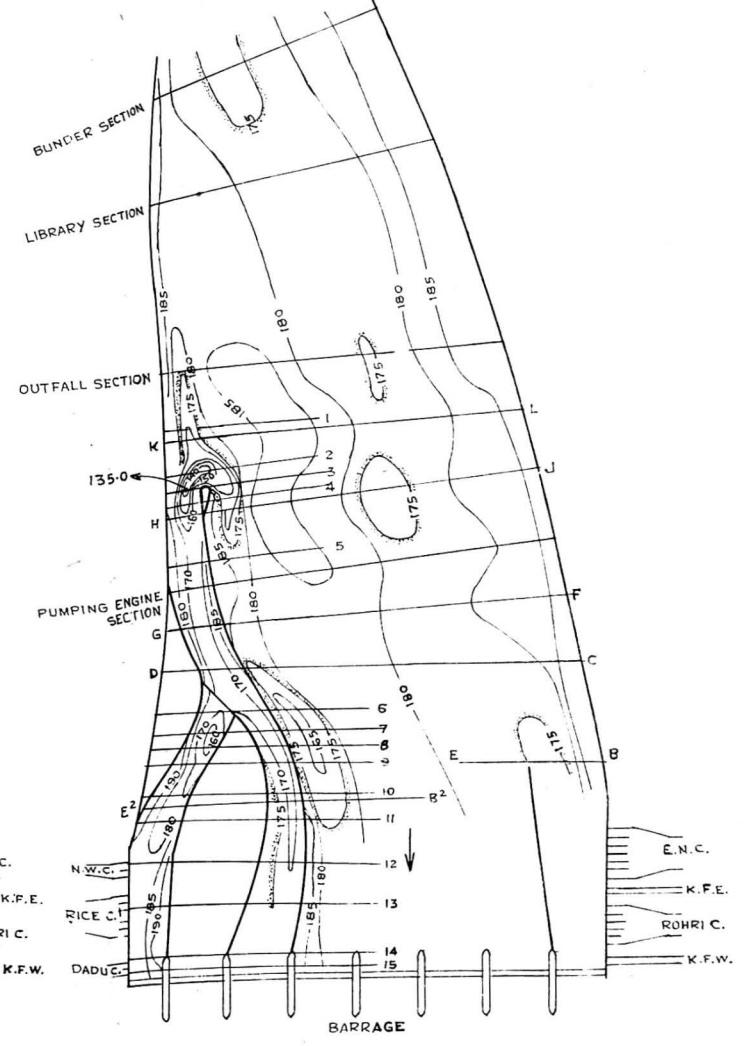
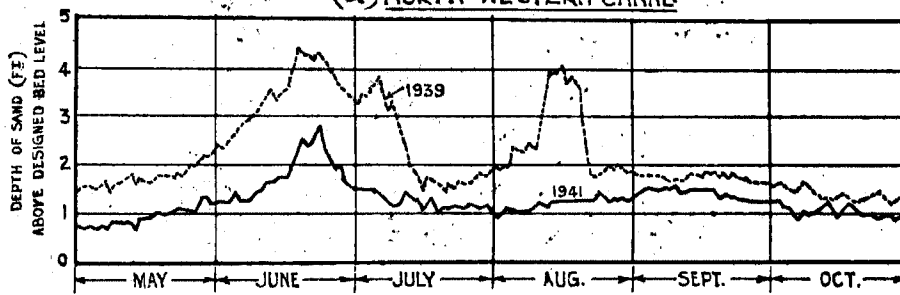
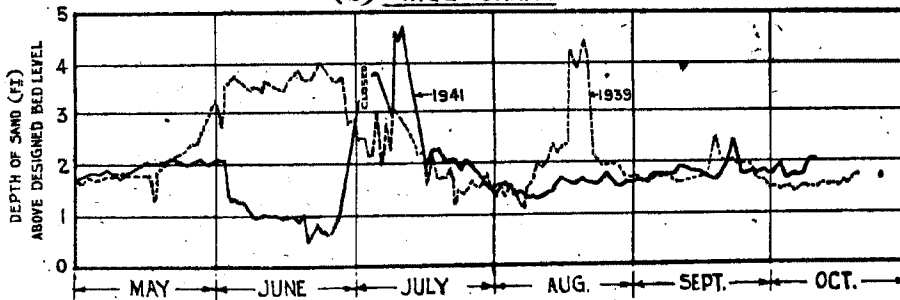


FIG-6-8
SHOWING DEPTH OF SAND ABOVE
DESIGNED BED LEVEL AT DISCHARGE SITE
(R.D. 2000)
OF RIGHT BANK CANALS
ABOVE SUKKUR BARRAGE
1939 & 1941

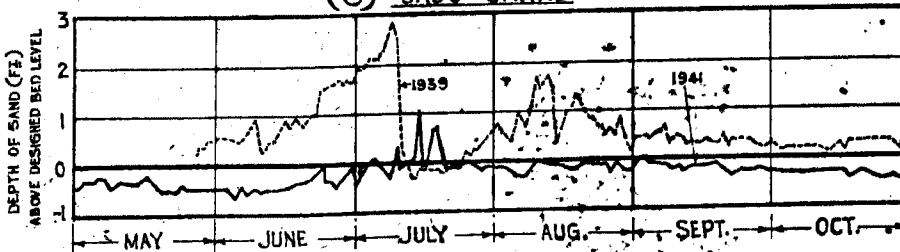
(a) NORTH WESTERN CANAL



(b) RICE CANAL



(c) DADU CANAL



EXCLUSION OF EXCESS BED SAND

6.09. EFFECT OF COMPLETED APPROACH CHANNEL

Photos 2641 and 2640, Sheet 6-1 show the work nearing completion. It was constructed during the fair seasons of 1939-40 and 1940-41 by Mr. S. B. Hickin, M. B. E., Executive Engineer, Barrage Division. Photo Sheet
6-1.

Figs. 6-6(a), (b) and (c) should be compared to see the effects of the first year's work and the completed Approach Channel on the river between Bundar section and the Barrage as shown by surveys done in June 1939, 1940 and 1944—i.e., before the Approach Channel was constructed, after the nose and upper 1,600 ft. of the outer bank of the Approach Channel had been completed, and some years after the whole of the guide bank had been finished. Fig. 6-6(a),
(b) and (c).

The Approach Channel was not intended to exclude sand at its head, except during periods of high floods; its object was to create curvature of flow in the Approach Channel—as illustrated in Fig. 6-2—so that top-water flowing outwards would tend to flow over the sill at the entrance to the pocket [Fig. 6-7(a)] so long as the discharge passing into the pocket was restricted to some 40 per cent to 50 per cent of the total discharge in the Approach Channel. Fig. 6-2
Photo 2760
and
Fig. 6-7(a)

In order to estimate the improvement in the three Canals (N. W. Canal, Rice and Dadu Canals) due to the Approach Channel, bed levels of the three Canals have been plotted for the Abkalani seasons of 1939 and 1941—see Figs. 6-8(a), (b) and (c). Fig. 6-8(a),
(b) and (c).

Fig. 6-8(a) shows that conditions in the N. W. Canal in 1941 were throughout better than conditions in 1939. On not a single day in 1941, was the depth of sand on the bed greater than on the corresponding day in 1939. In 1939, two sand waves entered the canal—one, raising the bed level to 4' on June 20th, and the other, with a maximum sand depth of 4'05" on August 15th; whereas in 1941 only one sand wave entered the canal on June 23rd, and the maximum sand depth was only 2'85 ft.

Fig. 6-8(b) shows that a large sand wave entered the Rice Canal towards the end of May 1939, the maximum depth of sand on June 23rd, being 4 ft. This wave persisted up to the middle of July, the number of days on which the depth of sand exceeded 2 ft. being 58. A second large wave, but of shorter duration, appeared in the middle of August (maximum depth of sand = 4'5 ft. and the depth exceeded 2 ft. on 12 days). Only one wave entered the canal, during the first half of July, in 1941 (maximum depth = 4'8 ft.); and the depth exceeded 2 ft. on a total of only 20 days. Taking the year as a whole, reduction of sand entering the Rice Canal was appreciable. Fig. 6-8(b)

Were it not that the Rice Canal flows only in the Abkalani, so that no recovery occurs in the cold weather, the Rice Canal would scour at much or more than the North Western Canal.

As regards the Dadu Canal [Fig. 6-8(c)]: In the earlier part of July 1939, a large sand wave entered the canal and for most of the Abkalani (from May to October) sand was entering the Canal. In 1941, on the other hand, there was only one small influx in July; and the bed was much lower than in 1939. Fig. 6-8(c)

In 1938 the future of the Right Bank Canals was in serious jeopardy. As a result of constructing the Approach Channel, the menace has been removed.

Sand exclusion by means of an Approach Channel requires little working head, and reduces interference with sand movement to a minimum—because the excess sand is ejected at a point where it can be carried away by the river.

6.10. ALTERNATIVE METHODS OF EXCLUDING SAND FROM CANALS

From time to time various other methods of excluding excess sand from canals have been hailed as brilliantly successful; but further experience has generally shown that the method proposed was only a partial remedy, or a temporary palliative.

6.11. HIGH LEVEL SILL AT CANAL HEAD

The first of these "remedies" was to provide a high-level sill at the canal head, over which water was drawn into the canal. Where upstream flow is axial and only a small part of the total flow is drawn off, this is successful; but it does not neutralise unfavourable curvature of flow in the river upstream, nor does it prevent the development of unfavourable curvature where a considerable part of the total flow is drawn by an offtake.

6.12. DIVIDE WALL

Another "remedy" was the addition of a divide wall running upstream from the Barrage, which was constructed opposite and generally parallel to a canal head—Photo Sheet 1-10. The original idea was to produce a still-pond pocket, in which silt would deposit, to be subsequently scoured through undersluices at the downstream end of the pocket. This type of undersluice scouring is still followed at Sukkur, and is temporarily helpful; but the effect lasts for only a few days; and, as the heads of the canals have to be closed while the pockets are being scoured—and closures interfere seriously with canal irrigation—this method is at best only a palliative, not a solution; and intermittent scouring of the Right Bank Pocket at Sukkur, the Divide Wall of which extended up to cross section E, B, (Fig. 6-4) failed to exclude excess silt in 1938. Where, however, favourable curvature of flow exists, excess sand will be excluded, even if the Pocket is run semi-open flow. This will be the case where

Photo Sheet
1-10.

Fig. 6-4

- (a) the approach flow is axial and the proportion of the flow drawn off into the pocket is small, or
- (b) the approach flow has sufficient favourable curvature to neutralise the "draw" of a large off-take.

Fig. 6-4

(b) is exemplified by the Left Bank Pocket at Sukkur; and as a result, regulation at the heads of the Left Bank Canals is foolproof; so that, despite misplaced efforts to induce more sand to enter the Left Bank Canals, the bed charge entering them has been very small. As a result, the gradient in the canals is also very small—as low as 1-in 18,000 ft. (= 0.56 ft. per thousand).

Favourable curvature of flow can often be created artificially—as was proposed at Dehri on Son.*

6.13. OPEN-TYPE EXCLUDERS AT HEADS OF BRANCHES AND DISTRIBUTARIES

Exclusion of excess sand from distributaries taking off from canals is a relatively simple problem: because

- (a) the charge in the parent canal is much less than in the river;
- (b) the parent channel is generally straight, and is not liable to develop unfavourable curvature;
- (c) small channels drawing water from a large channel tend to draw surface water and hence draw finer material than that moving along the bed of the main canal, especially where they are set back or have a raised sill;
- (d) branches and distributaries generally take off from above regulators, so that the "draw" can be controlled to a considerable degree by gate regulation.

* I. W. E. S. A. R. (T) 1943, pp. 40-44.

FIG: 6-9 MODEL OF THE EASTERN NARA AT MAKHI (SIND)

No 482

EXPT: No 1, EXISTING DESIGN WITH SILTED RIVER

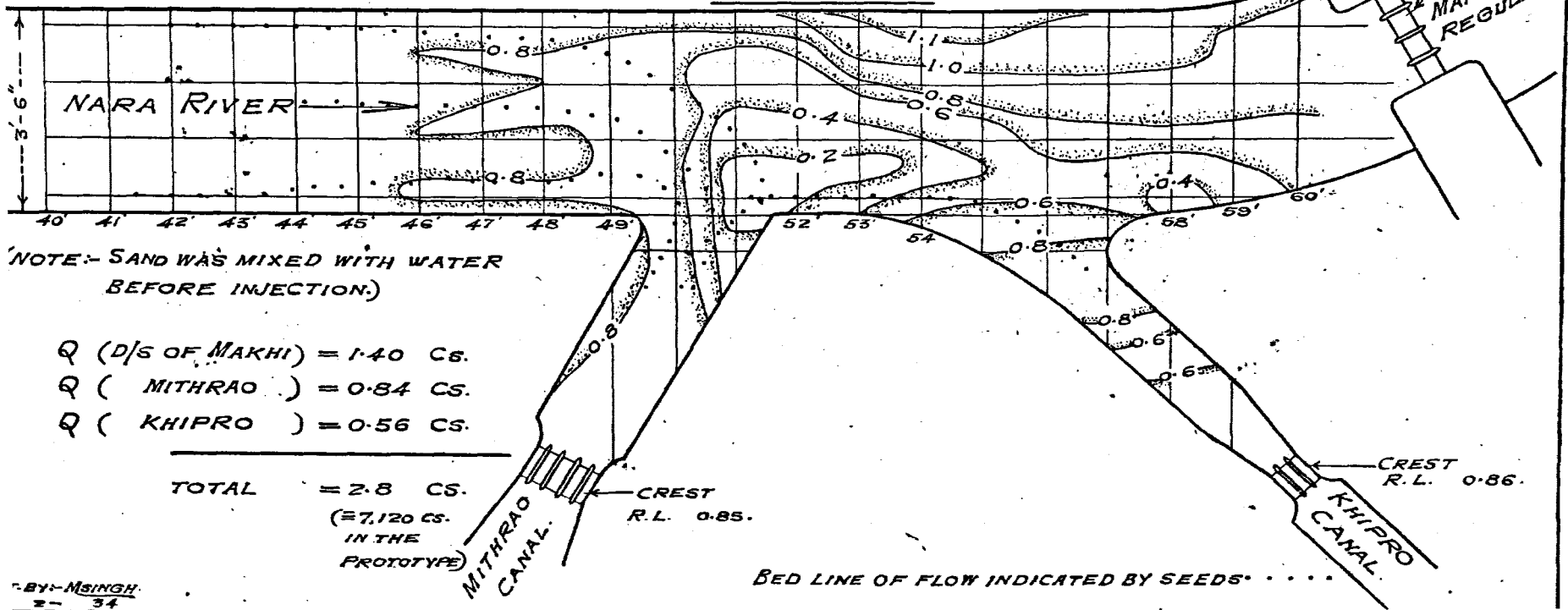
(FIG. 2 FROM B.T.P. No 59)

BED AS IN THE PROTOTYPE IN 1932. BEFORE STARTING THE EXPT.

CONTOURS AFTER SAND

HAD BEEN INJECTED FOR 3 HOURS

SCALE 2' = 1"



NOTE:- SAND WAS MIXED WITH WATER BEFORE INJECTION)

Q (D/S OF MAKHI) = 1.40 CS.

Q (MITHRAO) = 0.84 CS.

Q (KHIPRO) = 0.56 CS.

TOTAL = 2.8 CS.

(= 7,120 CS. IN THE PROTOTYPE)

MITHRAO CANAL. CREST R.L. 0.85.

KHIPRO CANAL. CREST R.L. 0.86.

MAKHI REGULATOR. CREST R.L. 0.69.

BED LINE OF FLOW INDICATED BY SEEDS

EXCLUSION OF EXCESS BED SAND

It is, in fact, only where the off-taking channel draws a considerable proportion of the flow of the parent channel, or the head of the branch or distributary has been built at an unsuitable angle, or at an unsuitable point at the inside of a bend, that such channels are liable to draw an excess charge; but there are many cases of off-taking channels having to feed high ground, so that it is desirable to give them a very flat gradient. This is only practicable if they have a sand-excluding head.

6.14. EXCLUSION OF EXCESS SAND FROM THE MITHRAO CANAL

A stage intermediate between the heads of canals taking off rivers and heads of distributaries is exemplified by the Mithrao Canal, which derives its supply from what is now known as the Eastern Nara Canal (see Fig. 6-9). The Eastern Nara was at one time an old river which derived its supply from the Indus upstream of Bukkur Island; but it is now supplied from a new head taking off from the Left Bank of the river just upstream of Sukkur Barrage. The Jamrao Canal also derives its supply from this canal from a point several miles upstream of Makhi Regulator and the Hiral and other small canals take off downstream of Makhi Regulator. When the Eastern Nara was transferred to the Barrage project and remodelled, the course was considerably straightened and shortened. As a result, the gradient increased and the bed and banks scoured and the material began to move along the bed. This entered the Mithrao and Khipro Canal in 1932 and the "drop" of 2 ft. 6 ins. into the Canal at full supply soon disappeared. That is to say, "command was lost". This is a serious matter; because once command is lost, the head discharge decreases; and as the slope required to carry the same charge increases as the discharge decreases, further deterioration is rapid.

Fig. 6-9

Several alternative methods of excluding the excess charge were tested at Poona*, five of which are here described. The Full Supply discharge in the main channel upstream of the Mithrao off-take was 7,120 cusecs of which

- 2,136 cs. enters the Mithrao Canal;
- 1,424 cs. enters the Khipro Canal;

- 3,560 cs. = combined Mithrao and Khipro discharge.

and the remaining 3,560 cs. flowed down the river, through Makhi Regulator.

Photos 838, 855, 865 (Photo Sheet 6-2), 888 and 890 (Photo Sheet 6-3) show five methods of excluding excess sand. These were tested in models of 1/60 horizontal and 1/142 vertical scale. Prior to remodelling its Head, the Mithrao drew more than twice the charge passing below Makhi Regulator, while the Khipro drew 4/5th the charge passing the Makhi Regulator.

Photo Sheets
6-2, 6-3.

6.15(a). EXCLUSION BY MEANS OF A PROJECTION DOWNSTREAM OF A CANAL

In order to create curved flow, a nose was built out into the Nara downstream of the Mithrao Head Regulator, extending to mid-stream, and the Left Bank was protected so that it did not scour. This reduced the coarse sand entering the Mithrao to about half; but created violent eddy action. When the Left Bank was not "held" so that it scoured, the Khipro Canal drew an excess charge.

Fig. 6-10
Photo Sheet
6-2.

* Bombay P. W. D. T. P. No. 59, "Silt Control at Heads of Canals and Distributaries" by C. G. Inglis and D. V. Joglekar.

THE BEHAVIOUR AND CONTROL OF RIVERS AND CANALS

6.16(b). EXCLUSION BY MEANS OF RAISED SILLS PROJECTING INTO THE PARENT CHANNEL WITH A WARPED CONVERGENCE UPSTREAM

Fig. 6-11
Photo Sheet
6-2.

In this design, curvature of flow was created by the 1 in 5 warped convergence combined with long high sills. This design gave successful results so long as the opposite bank was not allowed to scour. The ratios of Sand in the Mithrao and Khipro to the Sand per cusec in the Nara were reduced to .26 and .25 respectively. If, however, the Left Bank was not made scour-proof, the Khipro Canal received a considerable Sand charge, the ratio rising from .25 to .93 and conditions would have become worse with time. This design was not adopted partly because the cost would have been high, but chiefly because its construction would have necessitated closing the canals for several weeks, which would have seriously interfered with canal irrigation.

6.17(c). EXCLUSION BY MEANS OF A SUBMERGED VANE

Fig. 6-12

In this experiment, a single large-radius vane was constructed upstream of the Mithrao Canal. This vane had a vertical face at a height equal to one-third the depth of water and was protected by downstream pitching laid to a slope of 1 in 3 to prevent scour. The vane was tried in several positions and the best result was obtained with it starting at cross-section 47. The ratios of sand entering the Mithrao and Khipro (per cusec) to that passing down the Nara, were 1.02 and 1.01 respectively—in other words, this device gave proportional distribution, so was not sufficiently sand-reducing for this particular case.

6.18 (d). KING'S VANES

Fig. 6-13
Photo Sheet
6-3.

As can be clearly seen in Photo 890 (Photo Sheet 6-3) and Fig. 6-13, six curved vanes extending from the bed up to about one-third the depth were embedded in a smooth rigid floor opposite Mithrao Head. These vanes were spaced one and a half times their height apart. This reduced the quantity of sand entering the two canals to a ratio of .67 and 1.33 respectively. The sand "draw" into the Khipro Canal was thus excessive. When another set of King's vanes were added upstream of the Khipro (Fig. 6-14) results were satisfactory until the discharge in the Nara fell to less than 40 per cent of the designed Full Supply discharge.

Photo Sheet
6-3.
Fig. 6-14

The conclusion drawn regarding "King's vanes" was that they could exclude practically the whole of the bed charge where a single discharge was run; but their efficiency depended to a marked extent on the vanes being in line with the on-coming current. This cannot be assured if there is a range of discharges or a widely varying proportion of the discharge is drawn by the offtake—when they tend to give unsatisfactory results due to a sand bank forming downstream of the vanes; and where flow is not under complete control, they tend to throw coarse material into suspension and may give highly unsatisfactory results.

Where discharge conditions are relatively steady, they have a place—especially if complete exclusion is required—but under varying flow conditions they should be avoided.

FIG. 6-10. EXPT: N^o. 4, OF EASTERN NARA WITH HEAD REGULATORS.

No 472
(FIG. 5 FROM
B.T.P. No 59)

OF MITHRAO AND KHIPRO CANALS AND MAKHI REGULATOR
in 5 Convergence ups of Mithrao and long high sill
in front of Regulators.

FIG. 6-10

SCALE: 2" = 1 inch

Q of Nara River ups of Regulator	= 2.8 Cs. (= 7120 Cs.)
Q of Mithrao canal	= 0.84 Cs. (= 2200 Cs.)
Q of Khipro "	= 0.56 Cs. (= 1420 Cs.)

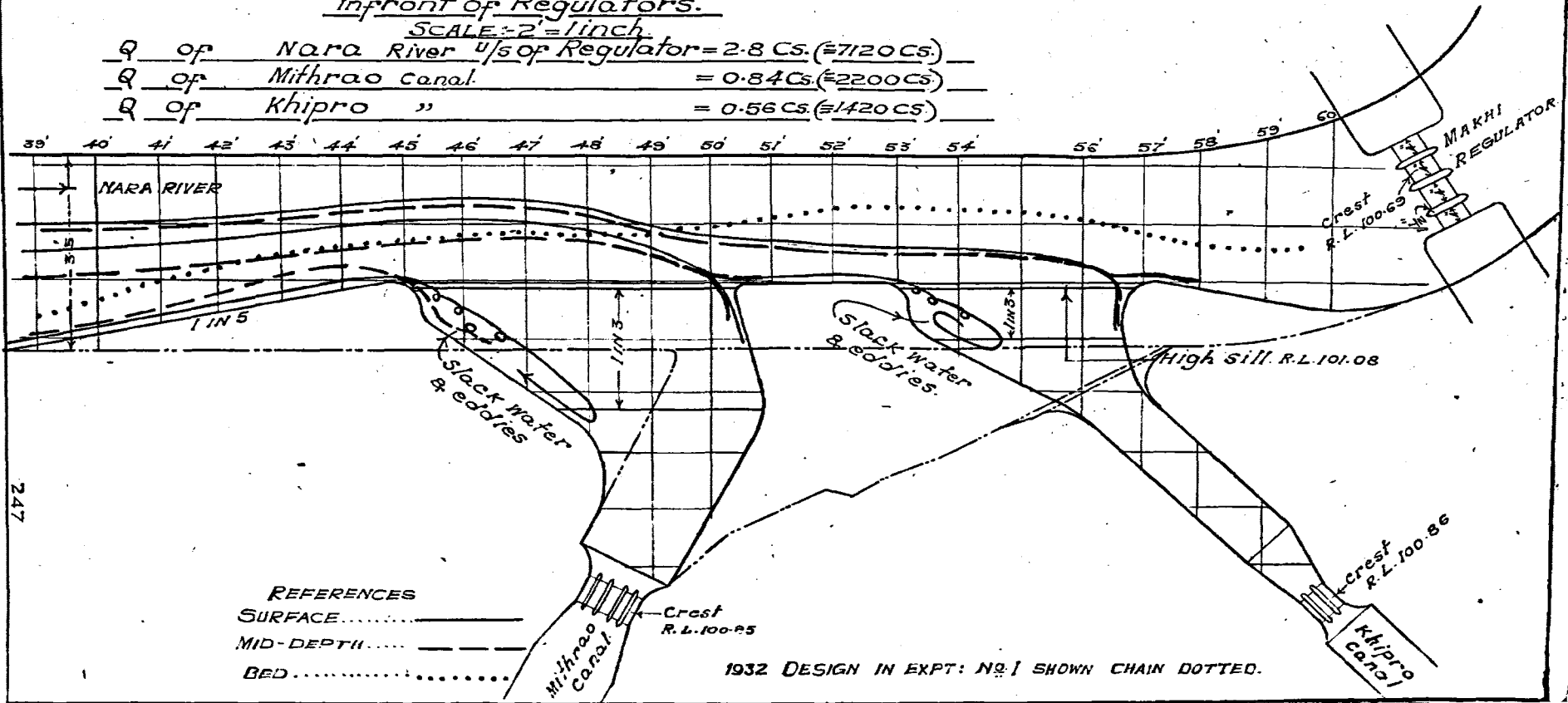


FIG. 6-11 Expt. No. 2, MODIFIED DESIGN FOR SAND EXCLUSION
EASTERN NARA WITH HEAD REGULATORS OF
MITHRAO AND KHIPRO CANALS AND MAKHI REGULATOR

No. 451.
 (FIG. 3 FROM
 B.T. P. No. 59)

SCALE: 2" = 1"

Q OF NARA RIVER	$\frac{4}{5}$ OF REGULATOR	= 2.80 Cs (≡ 7120 Cs)
Q OF MITHRAO CANAL		= 0.84 Cs (≡ 2200 Cs)
Q OF KHIPRO "		= 0.56 Cs (≡ 1420 Cs)

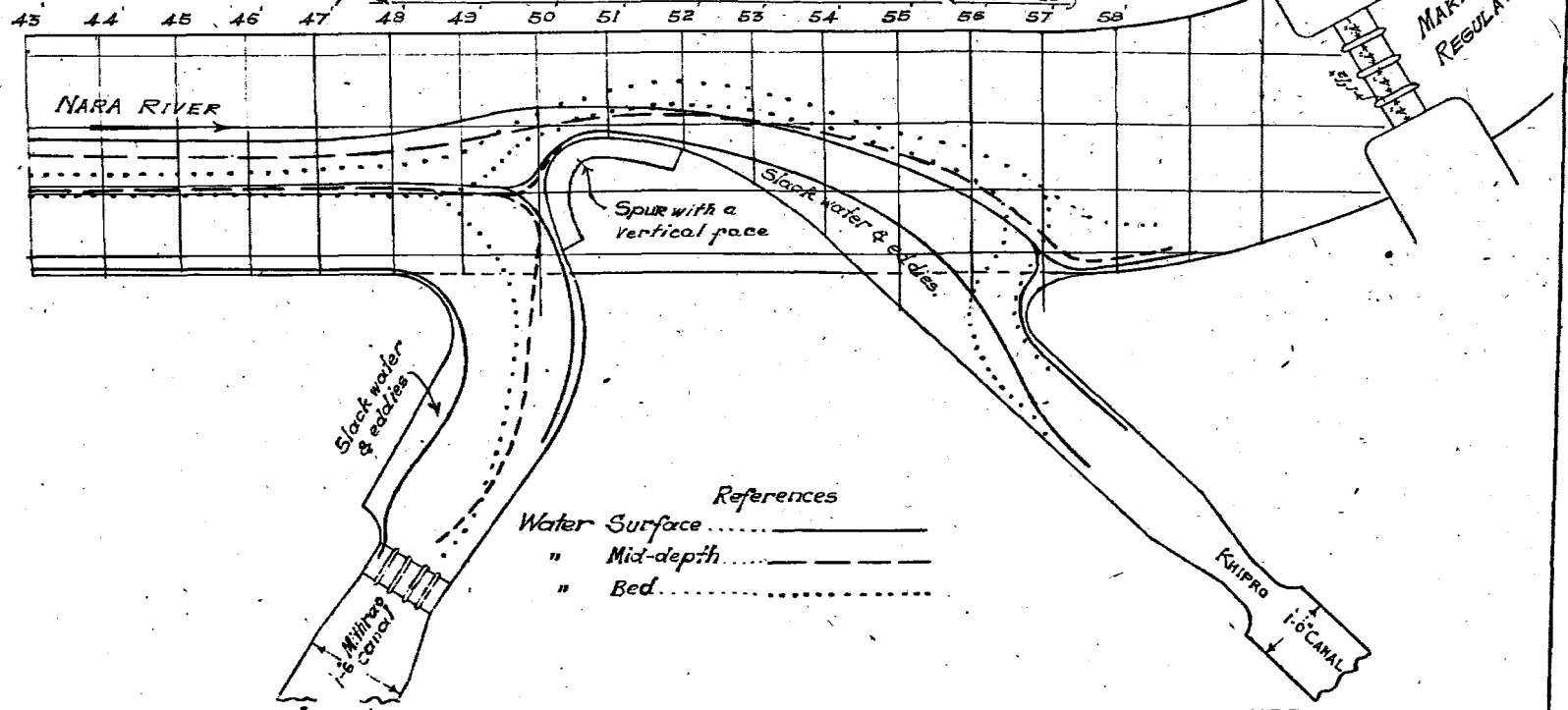


FIG. 6-12 MODEL OF THE EASTERN NARA AT MAKHI REGULATOR

No 487

EXPT. No 5 WITH A SINGLE VANE AND WITH SILTED RIVER

(FIG. 7 FROM B.T.P. No 59)

BED. (1932 IN THE PROTOTYPE)

FIG. 6-12

CONTOURS AFTER SAND HAD BEEN INJECTED FOR 3 HOURS

SCALE:- 2' = 1"

Q (D/S OF MAKHI) = 1.4 CS.

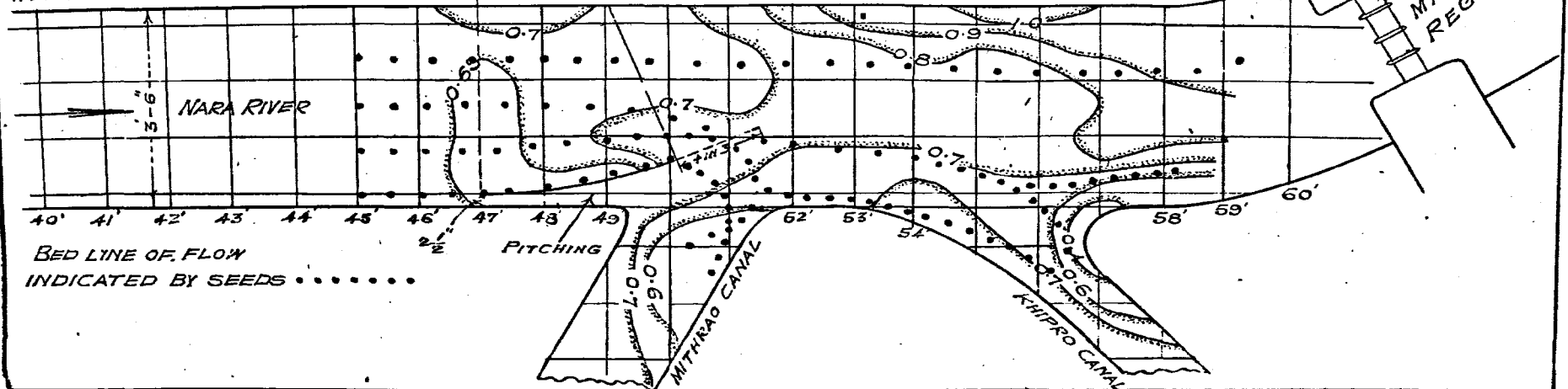
Q (MITHRAO) = 0.84 CS.

Q (KHIPRO) = 0.56 CS.

TOTAL = 2.8 CS (≡ 7120 CS. IN THE PROTOTYPE)

NOTE

THE BED OF THE RIVER
D/S OF VANE WAS PROTECTED
WITH PITCHING.



BED LINE OF FLOW
INDICATED BY SEEDS

CREST
R.L. 0.69
MAKHI
REGULATOR

FIG: 5-13 MODEL OF THE EASTERN NARA AT MAKHI REGULATOR

EXPT. No. 6 WITH KING'S VANES OPPOSITE MITHRAO

No 514
FIG. 6-13
(FIG. 8 FROM
B.T.P. No 59)

(SILTED RIVER BED AS IN THE PROTOTYPE IN 1932 BEFORE STARTING THE EXPT.)

SILT CONTOURS AFTER SAND HAD BEEN INJECTED

FOR 3 HOURS.

SCALE:- 2' = 1"

L. SECTION ALONG 1/3 FACE OF 1ST VANE

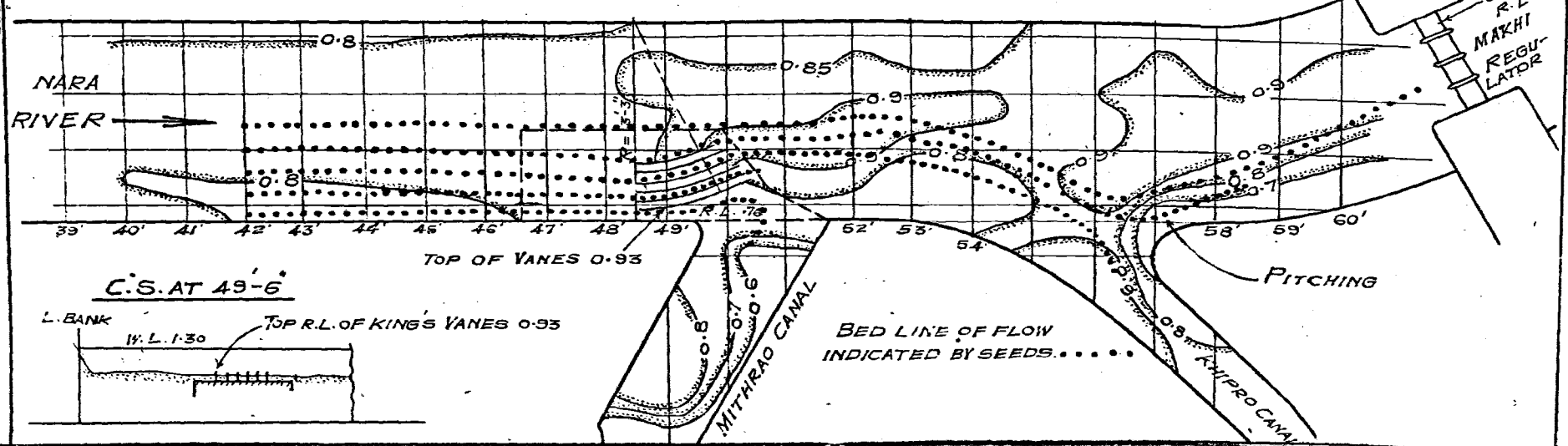
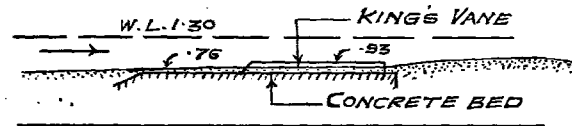


FIG:5-14 MODEL OF THE EASTERN NARA AT MAKHI REGULATOR:
EXPERIMENT No 7 — FALLING GAUGE.

(FIG. 9 FROM
 B.T.P. No 59)

(i) SAND WAS LAID ACCORDING TO 1932 SOUNDINGS IN THE PROTOTYPE,
BEFORE STARTING THE EXPERIMENT.

(ii) WITH KING'S VANES OPPOSITE MITHRAO AND KHIPRO.

CONTOURS AFTER 3 HOURS FLOW (DURING WHICH SAND WAS INJECTED.)

SCALE 2=1 INCH

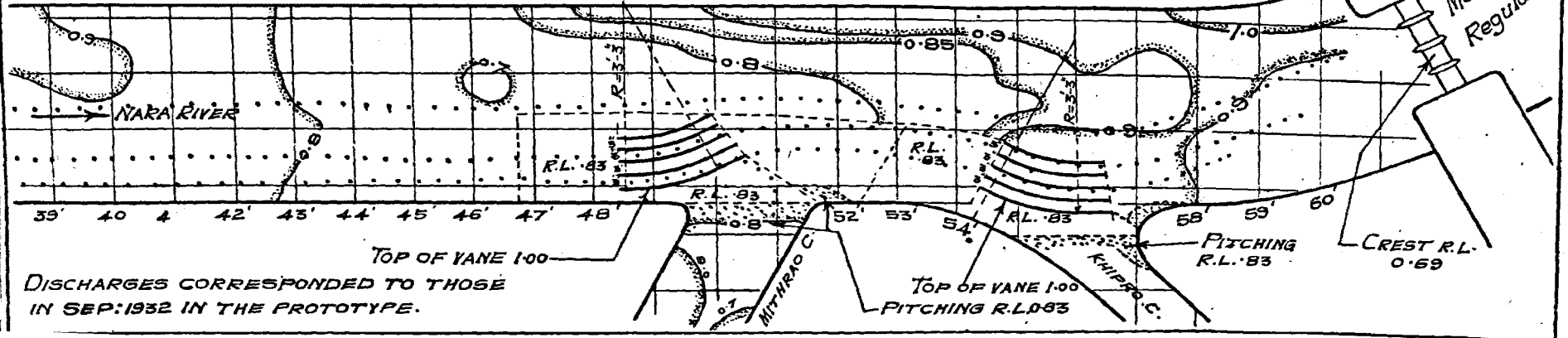
$Q (P/S \text{ OF MAKHI}) = 1.4 \text{ Cs. } (\approx 3,500 \text{ Cs.})$

$Q (MITHRAO) = 0.83 \text{ Cs. } (\approx 2,200 \text{ Cs.})$

$Q (KHIPRO) = 0.56 \text{ Cs. } (\approx 1,420 \text{ Cs.})$

Total 2.8 Cs. ($\approx 7,120 \text{ Cs.}$)

BED LINE OF FLOW -
 INDICATED BY SEEDS



DISCHARGES CORRESPONDED TO THOSE
 IN SEP:1932 IN THE PROTOTYPE.

EXCLUSION OF EXCESS BED SAND

6.19(c). DIVIDE WALL EXCLUDER

A. S. Gibb, who retired from the Punjab some 30 years ago, was probably the first person to understand this method of controlling silt. E. S. Crump has informed the author that according to Gibb, extra silt—more correctly sand—was drawn by off-takes because side eddies with vertical axes were bent towards the off-take, which caused the lower end of the eddy to rise abruptly, throwing bed material into suspension. Working on this theory, Gibb held that the right way to overcome excessive sand drawn was to divide water before it was affected by the draw of the off-take, by building a divide wall.

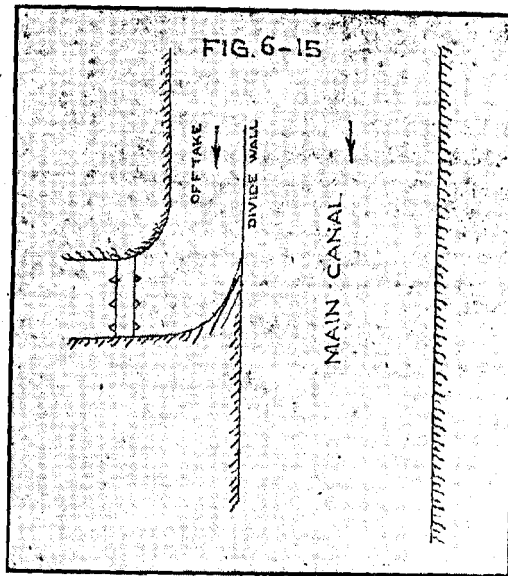


Fig. 6-15

(Fig. 4 from Bombay T. P. No. 52)

This question was not, however, fully clarified until 1935 when, after exhaustive experiments, the Author and D. V. Joglekar wrote a paper entitled "Factors affecting Exclusion of Bed Silt from Canals",* a summary of which will be found in Appendix 3.

In this experiment, a divide wall was constructed opposite the Mithrao to create "still pond" conditions † and a large-radius vane was built in continuation of the divide wall, to reduce the amount of bed sand entering the Khiprao Canal. The results of this experiment are compared with those for King's vanes in Table 6-I below.

Fig. 6-16
†
Photo Sheet
6-2
Table 6-1

*Bombay P. W. D. T. P. 52.

†Bombay P. W. D. T. P. 45 "Silt Exclusion from Canals—Still-pond versus Open-flow" by C. C. Inglis.

THE BEHAVIOUR AND CONTROL OF RIVERS AND CANALS

TABLE 6-I

Nara discharge above canals.	Rate of sand charges.				Remarks.
	Mithrao Canal/E. Nara d/s in		Khipro Canal/E. Nara d/s in		
	Experiment 7, King's Vanes.	Experiment 8, Divide Wall.	Experiment 7, King's Vanes.	Experiment 8, Divide Wall.	
(a) 7,120 Cusecs.	0.42	0.52	0.83	0.90	Discharges in experiments (a) and (e)—Mithrao Canal 2,200 cusecs (31 per cent); Khipro Canal 1,420 cusecs (20 per cent); Nara downstream 3,500 cusecs (49 per cent). Discharge in experiments (b) and (d) Mithrao Canal 1,928 cusecs (35 per cent); Khipro Canal 1,248 cusecs (23 per cent); Nara downstream 2,256 cusecs (42 per cent). In (c) Mithrao drew 1,280 cusecs (47½ per cent) and Khipro 1,193 cusecs (44½ per cent).
(b) 5,432 "	0.63	0.69	0.88	1.16	
(c) 2,682 "	1.08	0.32	0.65	0.34	
(d) 5,432 "	0.45	0.39	0.70	0.72	
(e) 7,120 "	0.29	0.20	0.60	0.43	

The lowest ratio of the velocity entering the pocket to that outside—that is the greater the width of the pocket at entry and the more constricted the flow in the canal outside the pocket—the more bed material will be excluded.

Photo Sheet 6-4
Fig. 6-17
6.20-(f). APPROACH CHANNEL TAKING OFF FROM OUTSIDE OF A BEND

In this design, part of the old river channel was re-opened to carry water to the Khiprao Canal and Makhi Regulator, and the existing (straight) channel was reduced in width to serve as an Approach Channel to the Mithrao Canal. Experiments in this connection were run for four periods of three hours each, and gave ratios of sand contained in water entering the Mithrao and Khipro to that passing down the Nara, per cusec, as shown in Table 6-II below.

Table 6-II

TABLE 6-II

Silt ratio during.	Mithrao/E. Nara d/s.	Khipro/E. Nara d/s.
1st period of 3 hours	0.09	1.11
2nd " " "	0.05	0.57
3rd " " "	0.04	0.53
4th " " "	0.07	0.44

Photo Sheet 6-4
Comparing the above ratios with conditions in 1932 (ratios = 2.15 and .81 respectively) it will be seen that the bed sand entering the Mithrao was almost wholly eliminated. This design was adopted—see Photo 1409, Sheet 6-4—showing the Eastern Nara on the left and the Approach Channel of the Mithrao on the right. As a result, "command" was at once restored and the drop into the Mithrao, at the head regulator—which had disappeared—increased to 1'6" within a few months.

Q DOWNSTREAM OF MAKHI = 1.4 Cs: ($\approx 3,500$ Cs.)
 Q (MITHRAO) = 0.84 Cs: ($\approx 2,200$ Cs.)
 Q (KHIPRO) = 0.56 Cs: ($\approx 1,420$ Cs.)

2.8 Cs: ($\approx 7,120$ Cs: IN THE PROTOTYPE)

DISCHARGES CORRESPONDED TO THOSE IN SEP: 1932 IN THE PROTOTYPE.

BED LINE OF FLOW INDICATED BY SEEDS.....

259

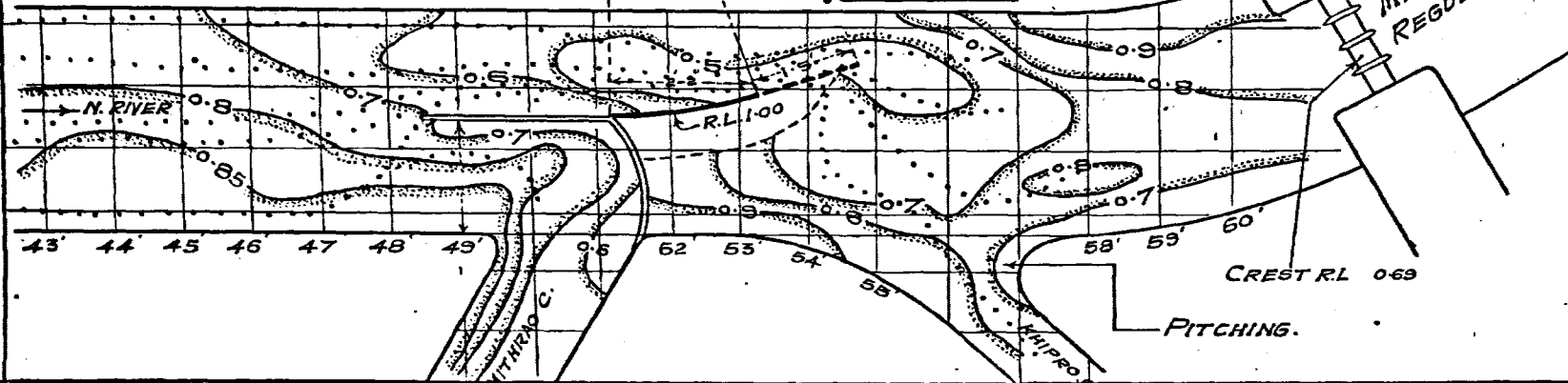


FIG: 6-16 PLATE II N^o 517
 MODEL OF THE EASTERN NARA AT MAKHI
 EXPERIMENT N^o 8 (a)_F - FALLING GAUGE.

(i) SAND was laid according to 1932 soundings in the prototype before starting the experiment.
 (ii) With a short 'divide wall' opposite Mithrao and a single curved Vane ($R=6'-8''$) o/s of the 'divide-wall' for Khipro. Contours after SAND had been injected for 3 hours.

Scale 2" = 1'

(FIG. 10 FROM B.T.P. N^o 59)

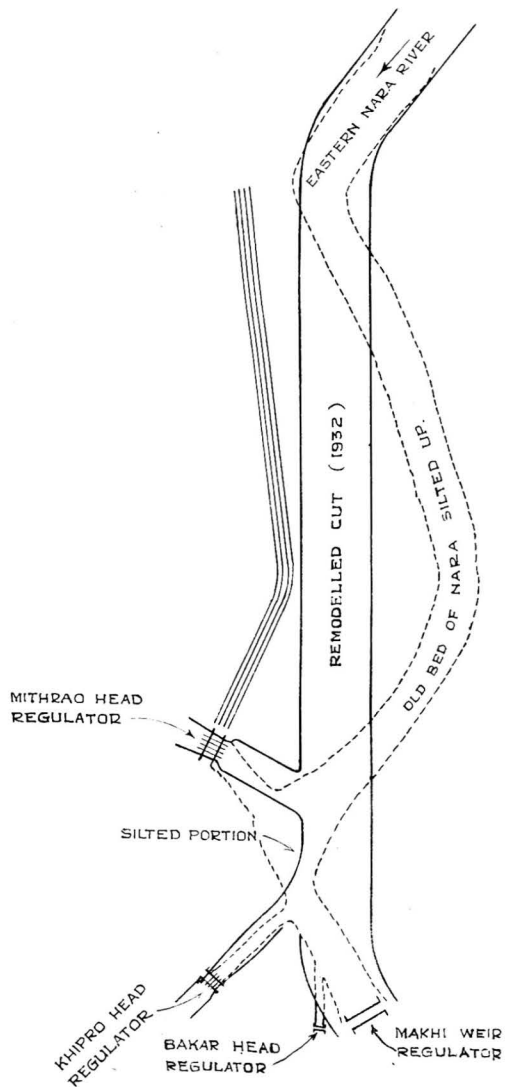
FIG:-6-18
 MITHRAO & KHIPRO EX-NARA
 AT MAKHI REGULATOR.

SCALE: 1" = 6' = 360'

(FIG. 30 FROM A.R.T. 1941-42
 OF C.I. & H.R. STN)

(a)

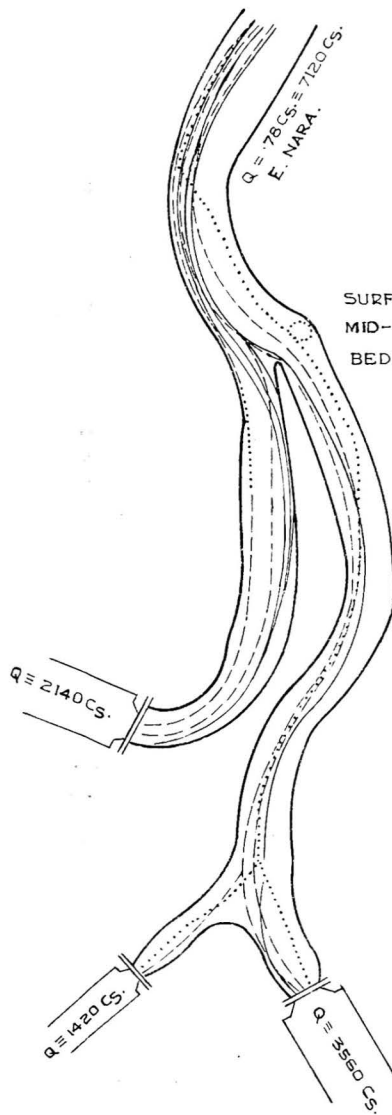
EASTERN NARA AT MAKHI AS
 IT EXISTED IN 1932



(b)

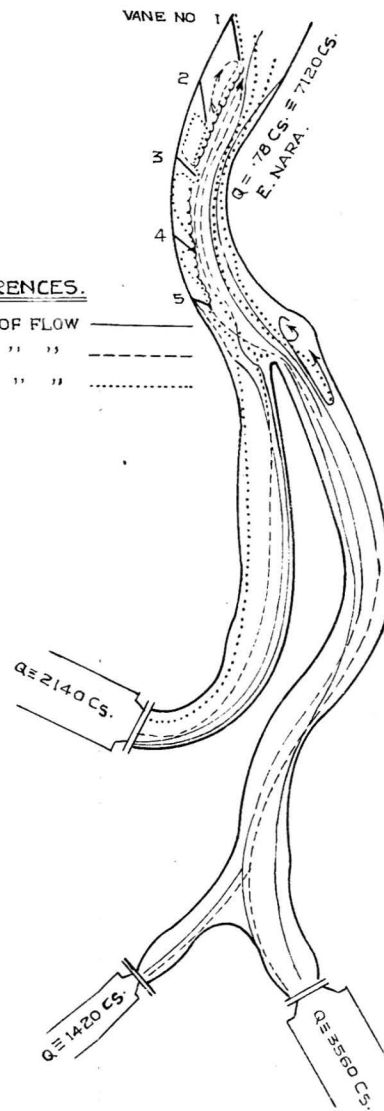
LINES OF FLOW IN 1/60:1/20
 V.E. MODEL.

LINES OF FLOW BEFORE THE
 VANES WERE CONSTRUCTED.



(c)

LINES OF FLOW AFTER THE
 VANES WERE CONSTRUCTED
 IN 1941.



REFERENCES.

SURFACE LINES OF FLOW ———
 MID-DEPTH " " " - - - -
 BED " " " ·····

EXCLUSION OF EXCESS BED SAND

Photos 882 and 883, Photo Sheet 6-5 and Fig. 6-17 show this experiment. Photo Sheet
6-5
Fig. 6-17

It was realized that as a result of excluding sand from the Mithrao, the Khipro Canal would get an excess charge of bed sand; but as the cut-off at its head was originally 3 ft. it was considered that it could stand this extra charge better than the Mithrao or the Nara; and though it was pointed out that sand entry into the Khipro could be prevented by supplying it from the Mithrao Approach Channel, the Author advised against this on the ground that the Nara bed was rising rapidly below Makhi Weir and would rise still more rapidly if most of the sand then carried by the Khipro were passed down the Nara. By the end of 1940, the position downstream of Makhi had become so serious that the Poona Station was asked to recommend, as an emergency measure, the best way to increase the sand charge entering the Mithrao Canal.

This was a complete reversal of the then existing design, which had been constructed in order to exclude bed sand by curvature of flow; and so the change presented difficulties. The conditions at the time are shown on Fig. 6-18(a). Two alternative designs were, however, found to be effective:— Fig. 6-18(a)

(i) Closing the Approach Channel mouth and making a cut from the Nara to the Approach Channel from a point 500 ft. downstream of the then Approach Channel mouth, thus drawing water from the inside of a bend;

or (ii) Constructing a series of groynes along the Right Bank of the Nara, pointing 45° downstream, starting from about 1,000 ft. upstream of the mouth of the Approach Channel. The object of these was to throw bed sand into suspension, to be subsequently carried into the Mithrao Approach Channel. Fig. 6-18(b)

The second alternative was adopted—mainly because it could be carried out rapidly and so would not interfere with irrigation, but also because it was relatively cheap. As constructed the first three groynes were 90 ft. long and the last two groynes which were subjected to severe attack, were 40 ft. long, the depth of the scour pits at their noses being 25 ft. Experiments with this design in the 1/60; 1/20 (V. E.=3.0) model at Poona showed that the surface water from upstream of groyne no. 1 flowed down the Nara and that the Approach Channel drew water from turbulence created by the groynes. These results have been reproduced in the Nara, *vide* a report made by the Executive Engineer, Mithrao Canal in December 1941, which reads:—

“The position with regard to Nara now is more favourable; Khipro has remained steady and effect of the groynes has obviously been beneficial to some extent so far as the Khipro and Nara downstream of Makhi are concerned. Visual observations of the effect of groynes during the last Kharif season showed that surface water was actually flowing back out of the Mithrao approach, and round the nose into the Nara. There was a pronounced reverse surface-flow for about 20 ft. downstream of the nose in the Mithrao approach, combined with a transverse flow towards the Nara side of the nose extending about 40 ft. across the Mithrao approach. This diversion of surface flow indicates that Mithrao is now drawing a much larger proportion of its supply from the bed filaments.”

The success of the design has been further confirmed by later data.

6.21. PERMANENT EFFECT OF TEMPORARY INFLOW OF COARSE SAND ON REGIME OF MITHRAO CANAL

Owing to the quantity of sand entering the canal being reduced, the slope was in excess of the energy gradient required to carry the finer bed material; so the silt which had previously deposited near the head was gradually scoured from the head reach and carried down the canal, to be dropped where the energy slope was insufficient to carry it any further.

As a result, bed accretion occurred and the banks had to be raised in the middle reaches of the Canal, where the scoured bed-silt had deposited. This fact does not, however, detract from the value of the re-modelling; because the "loss of command" resulting from sand having been allowed to enter the Canal had to be set right, and this could only be done by admitting clearer water which, in turn, led to coarser grades of bed-material passing down the Canal to a point where the gradient was insufficient to carry it further.

The lesson to be learned from this experience is the importance of preventing the entry of coarser grades than normal, even though this may continue only for a limited period of time.

6.22. DISTRIBUTION OF BED-LOAD BETWEEN OFF-TAKES

In the case of distributaries and branches taking off from a canal, the object is not mere exclusion of sand, but the suitable division of sand between the parent channel and the off-taking channels; so, correct quantitative distribution is required. Distributaries and branches generally take off from above a regulator; so that flow can be controlled to a considerable extent. Where, however, a branch draws off a large proportion of the discharge of the parent channel, the off-take should not be at right angles; but should face somewhat upstream [Fig. 6-19(a).]

Fig. 6-19 (a)

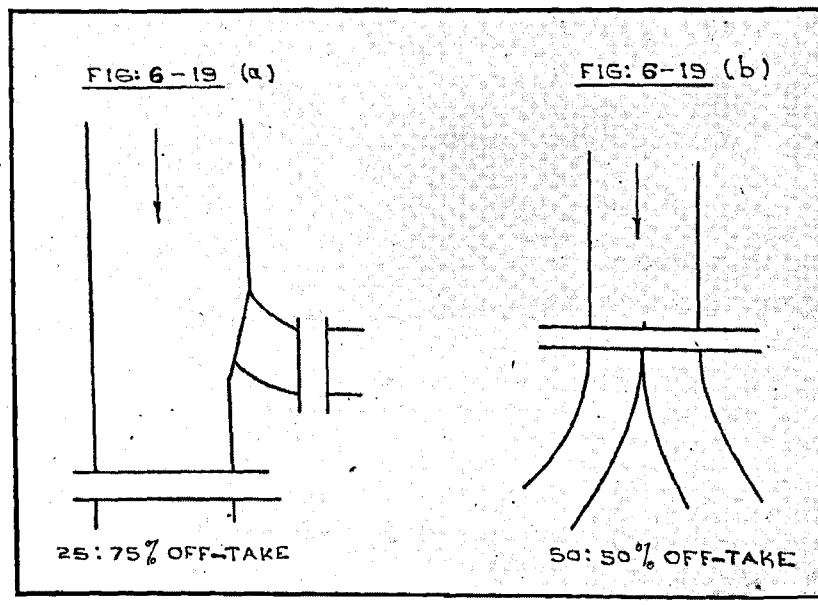


Fig. 6-19 (b)

When 50 per cent is drawn it should take off at the same angle as the tail channel [Fig. 6-19 (b)] where equal distribution of bed sand is desired.

EXCLUSION OF EXCESS BED SAND

In some cases it may be desired to run an off-take with a very flat gradient in order to feed high land near the tail of the channel. It is then necessary to design the off-take to draw a sub-normal charge. This can be achieved by super-normal exclusion by the methods already described.

6.23. TUNNEL-TYPE EXCLUDERS AT HEADS OF CANALS

In 1922, F. V. Elsdon originated the idea of tunnel-type excluders;* but it was not until 1934 that H. W. Nicholson built the first excluder at the Head of the Lower Chenab Canal at Khanki—now known as the Khanki type excluder. Several others have since been constructed, the principle in every case being to pass the slow-moving bed water, containing a heavy charge of the coarser grades of material, through tunnels, whose tops act as platforms at about canal sill level. These tunnels lead to under-sluice gates in the Barrage, which should be so regulated that

- (i) the tunnel does not get clogged with sand, and
- (ii) the bed water and top-water are divided without developing eddies which would throw the coarser grades into suspension.

This type of excluder has hitherto been restricted to the Punjab. Reference is invited to three papers on the subject:—

† "The silt exclusion arrangements provided at Trimmu Head works" by F. F. Haigh, Esq., C. I. E.†

‡ "Silt excluders and ejectors" by Dr. Harbans Lal Uppal‡ and ‡

§ "Shingle and silt excluders and ejectors" by Dr. H. L. Uppal.§ §

6.24. TAJEWALA TUNNEL-TYPE SHINGLE EXCLUDERS

In the last of the three Papers, a shingle excluder is described.* This was adopted after model experiments were carried out at Malikpur, with which the Author was associated. The principle is different from that in other types of tunnel excluders, in that successful exclusion depends on inducing optimum curvature of flow—as in the case of all the open-type excluders already described. Exclusion at Tajewala was extraordinarily successful—so successful that the design was *judged to have failed*; because it excluded such a heavy charge of boulders and pebbles that the water could not carry them down the river under the conditions ruling at that time; but the Author sees no reason to suppose that a satisfactory arrangement cannot be made to set this right; and considers that the design is eminently sound.

6.25. F. F. HAIGH ON EXCLUDERS

"The following advantages are common to all pocket excluders:—

(a) The head across the barrage is always available for operating them.

(b) Economy is secured by the use of the barrage gates and cisterns.

(c) Large orifices, unlikely to be choked by rolling or submerged floating debris are easily provided.

The disadvantages of this type are:—

(d) The difficulty of securing good approach conditions.

(e) The undersluice bays covered by the excluder cannot be used simultaneously for exclusion and their proper purpose of passing high river discharges.

(f) The work being subject to the river action has to be of very robust construction."

* "Irrigation Canal Headworks" Punjab Irrigation Branch Paper No. 25, 1922.

† C. B. I. A. R. (T) 1938-39, pp. 60-62.

‡ C. B. I. A. R. (T) 1941, pp. 149-157.

§ C. B. I. A. R. (T) 1943, pp. 100-103.

There are two types of pocket excluders, known respectively as the Khanki Type and the Trimmu (or Kalabagh) Type—see Figs. 6-20 and 6-21 (a) and (b).

Haigh appraised the relative merits as follows:—

“The excluder built in the left pocket at Trimmu—which has become known as the Kalabagh type, because it was originally designed for that headworks, is an attempt to eliminate the disadvantage (d) above. With the Khanki type the water approaching the excluder has to turn through a large angle and hence must be in a disturbed condition. If a divide wall exists, the approach will generally be *via* an S curve with intense local disturbances at the divide wall nose. The divide wall, therefore, appears to be definitely disadvantageous for this type. With the Kalabagh type a good approach can be secured if the divide wall is made of sufficient length, but this naturally involves heavy expense. It is doubtful if the length is sufficient to achieve this at Trimmu.

A disadvantage of the Kalabagh type as compared with the Khanki type is that the distribution of the canal supply at the edge of the diaphragm is not under control.”

6.26. EJECTORS

An ejector is designed to get rid of excess material after it has entered a canal. It consists of slots or apertures in the bed of a canal, through which a large part of the coarser particles is ejected by passing a relatively small quantity of canal water from near the bed of the canal.* The first ejectors were constructed by E. S. Crump, C. I. E. on the Upper Jhelum Canal and others have been installed in the lined Haveli Canal. No less than six are being installed in series near the head of the Thal Canal, which will take off the Indus at Kalabagh. Multiple ejectors have been adopted because it has been demonstrated by models that more sediment can be ejected for a given discharge escaped, by removing the coarser particles first, and then finer, and finer grades, than if a single ejector is used. As to whether this will work out so successfully in practice is doubtful; because the quantity of various grades in movement will be in the form of bed waves or dunes and will be constantly varying; but the multiple ejectors should, at any rate, give most valuable information with regard the design and best methods of regulating ejectors.

The main principle to be followed is that the bed upstream of an ejector should be smooth and have such a slope that with a suitably adjusted velocity the grade of material which it is desired to eject will be just moving along the bed, while finer grades will remain, so far as practicable, in suspension or saltating.

Where there is a single ejector with a cross regulator downstream, the water level can be regulated so as to attain any desired velocity; but where there are several ejectors in series and only one cross regulator, there can only be one regulation of velocity for all the ejectors, and a reduced efficiency must, therefore, be expected.

* Same references as for Excluders.

Fig. 6-20

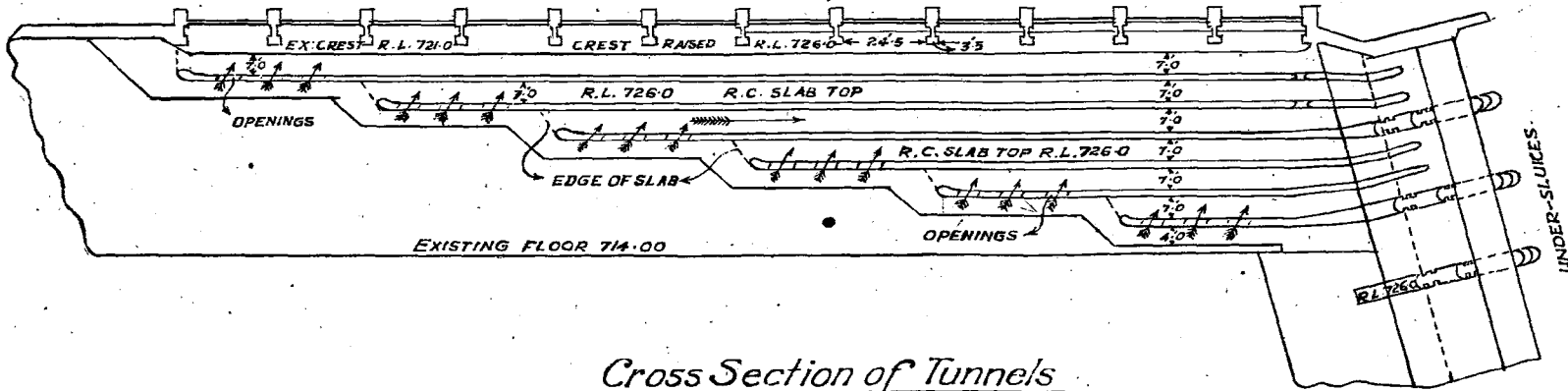
Khanki Excluder Plan of Tunnels

Scale. 1/500

No. 6148
Indian Waterways Experiment Stn.
POONA.

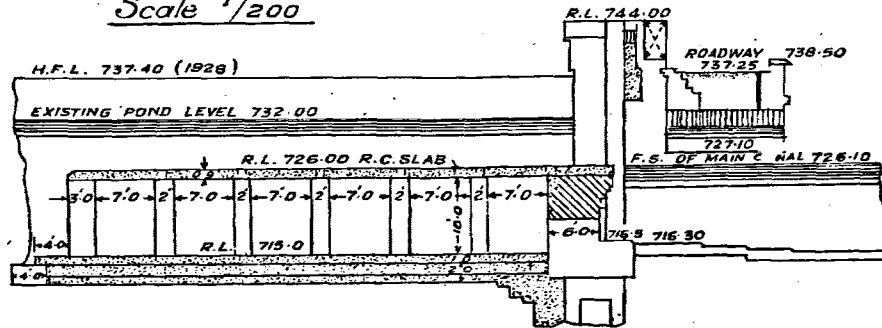
COPIED FROM:-

P.E.C. PAPER No 211 OF 1938.



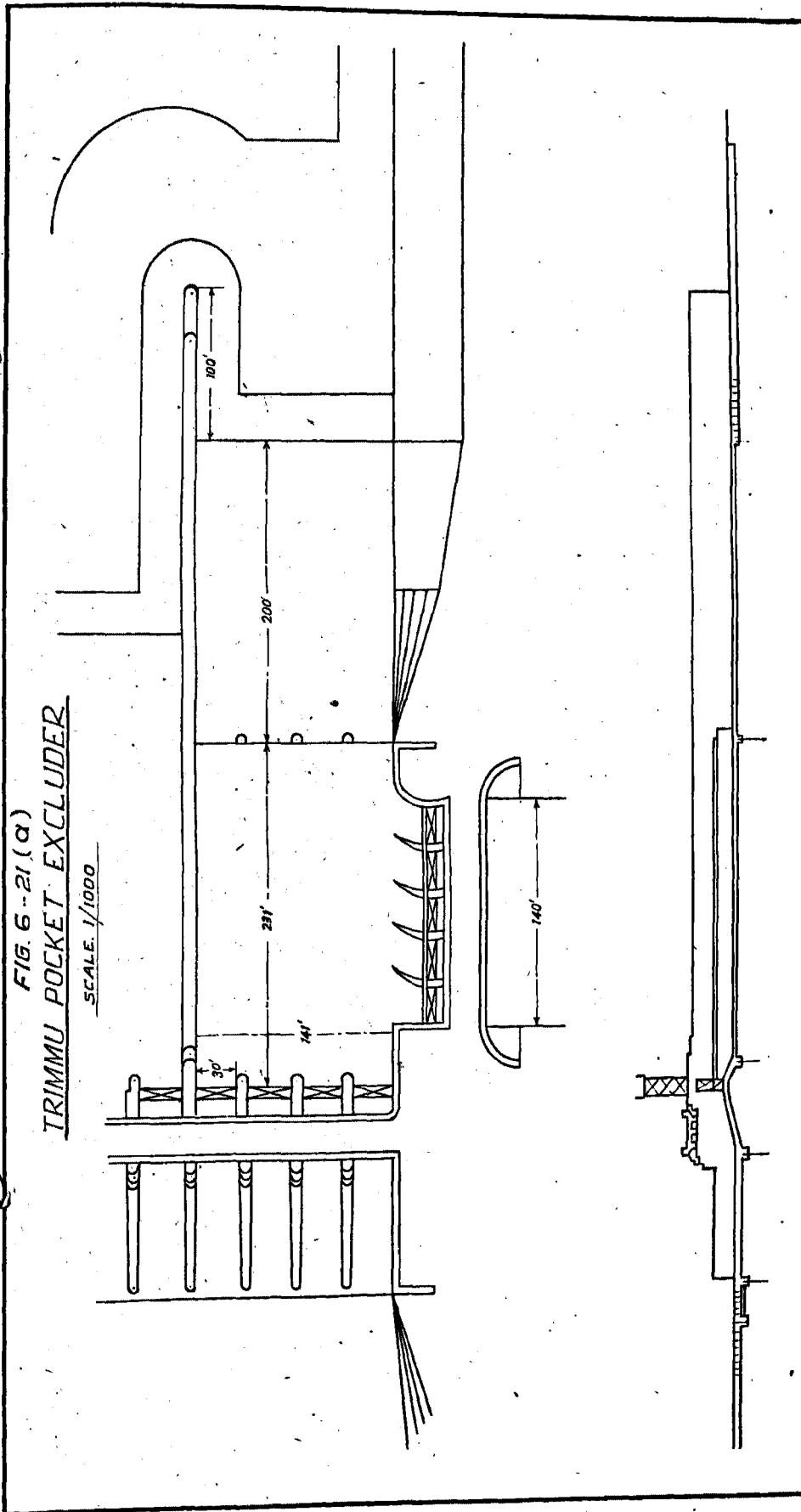
Cross Section of Tunnels

Scale 1/200



269

By Parkhi.
5th Aug. 1946.



© U.S. Army, 1967

EXCLUSION OF EXCESS BED SAND

6.27. HAIGH'S OPINION ON ADVANTAGES AND DISADVANTAGES OF EXCLUDERS AND EJECTORS

As regards the choice between ejectors and excluders, Haigh states that the advantages which can be claimed for ejectors, are:—

- (g) The ease with which good approach conditions are secured.
- (h) The relative lightness of the work.
- (i) The possibility of multiple extraction.

A long list of disadvantages may be cited such as:—

- (j) The difficulty of securing a working head with high river and low canal supplies.
- (k) The necessity of increasing the size of the head regulator to accommodate the escape.
- (l) The increase in size of the canal section above the extractors necessary to accommodate the escape.
- (m) The necessity of providing separate outfall works.
- (n) The comparatively small orifices are liable to be blocked by sunken debris and may necessitate the provision of a trash rack.

If, however, (j) can be overcome, the others are merely a matter of cost and are easily outweighed in comparison with the expense of the Kalabagh type. If, however, (j) presents difficulty and it is necessary to go a long way down the canal to secure the necessary working head (l) may be a serious matter."

6.28. ADDITIONAL DIFFICULTIES OF EXCLUSION WHERE CANALS TAKE OFF FROM BOTH BANKS

For satisfactory sand exclusion, curvature of flow—as found near the concave bends—is desirable. This can generally be achieved on one bank; but rarely on both. In this connection reference is invited to Fig. 6-4 showing the Indus at Sukkur Barrage, where the difficulty was overcome by an Approach Channel; but in a very wide shallow river—like the Son at Dehri—it may be possible to divide the flow between channels along the two banks*; but where curvature on one bank is unfavourable at a barrage it may be best to construct the head of a canal at a point about $\frac{1}{4}$ of a meander length upstream where favourable concave curvature prevails or can be induced.

Fig. 6-4

Fig. 6-22
Fig. 6-23

6.29. WHERE THE RATIO OF DISCHARGE OF OFF-TAKE TO THAT OF PARENT CHANNEL IS HIGH, EJECTORS MAY BE THE ONLY SOLUTION

Nearly the whole discharge of a river may be drawn by an off-taking canal. In that case the best solution is the one which requires the least waste of water to get rid of the excess sand. That means ejectors. This, in turn, means a considerable loss of head, and difficulty in getting rid of the ejected materials, due to the residual energy in the river being insufficient to carry the ejected material.

6.30. CONCLUSIONS

Each case must be considered on its merits. What may be most suitable where the material is relatively coarse and the river gradient steep, may be quite unsuitable where the material is fine, the slope flat, and "head" available small. In the latter case, open-type excluders have great merits, because they require practically no "head" and interfere much less with the natural movement of sand down a river than any alternative design.

* "Remodelling the Son at Dehri" C. I. and H. R. S., A. R. (T) 1941-42, p. 20-23 and I. W. E. S., A. R. (T) 1944, p. 58-59.

INDEX

A	Para. number.	Figure number.	Photo Sheet number	Page number of Pam.
A				
Accretion—Definition of—	(1)
" —Rice and North Western Canals	1-17, 6-89	46, 241
" —Rivers Indus at Sukkur	Chapter 7	283
" —Rivers Indus at Sukkur	1-28	75
" —Rivers Indus at Sukkur	Chapter 5	167
" —Time scales different for scour and—	13-38 (vii)	450
Adjustments in rivers	9-09, 4-07	88, 189
Afflux—Definition of—	(xvi)	5
Ajmer-Merwara. Unrepresented flood in	13-05	409
Alluvial channels. Factors affecting flow in	Chapter 7	283
Alluvium, flow in. Lacey Papers of 1929 and 1938	3-88	98
Alternate layers of sand and clay, Apron laid on	9-08	361
Amrawa Nadi (U. F. Kalai Bridge)—Maximum flood discharge in the—	12-05	409
Anaethema. Groyne as usually—to river training engineer.	10-01	279
" Training works long considered	12-01	407
Anar at Jaunpur (U. F.)—Maximum flood discharge in the—	13-04	409
Annual Reports (Technical)	Appendix 1	453
Annual run-off from catchments	12-08	418
Approach Channel. Effect of—on N. W. Canal	7-03, 7-05	289, 290
" " Effect of—on Rice Canal	7-04, 7-05	289, 290
" " Effect of—on water levels	8-14, 6-08	198, 284
" " " "	6-09	241
" " Mithrao Canal	6-14, 6-20,	6-17, 6-18,	6-4, 6-5	245, 253
" " Effect of—Sukkur	7-02	289
" " Right Bank—Sukkur	1-12, 6-07	...	6-1	51, 281
" " Outer bank of—	1-14, 6-1	...
Apron. Alternate layers of sand and clay laid in—	9-08	361
" Blow out in falling—	9-07	361
" Conditions under which a falling apron may fail	9-09	362
" Grade of stone required in—	9-12	370
" Launching of—	9-22	356
" Mixture of stone in—	Chapter 9	355
" " "	9-11	353
" Quantity of stone required in falling—	9-18	370
" Slope. Inclination of launched—	9-04	359
" Slip in falling—	9-07	361
" Reinforcing a fallen—	9-14	373
" Stones of mixed sizes in—	9-05, 9-06	360, 360
" Failure of Hardiage Bridge—	9-10	362
Aprons. Falling—	Chapter 9	355
Areas. Change in Datum—downstream of Barrage	5-18	5-12	...	184
" Change in sectional—for 300,000 cu sec	5-18	5-12	...	184
Attack reduced by attracting groynes	12-01	407

INDEX

B and C	Para. number.	Figure number.	Photo Sheet number.	Page No. of para.
Bose, Dr. N. K.—Ph. D., P. E. C. Paper 192 "Silt movement and design of channels,"	3-03	93
Bose. Flow formulae of 1936	3-03	98
Boundary layer. Separation of—	1-17	...	1-17	52
Brahmani (Or ass). Maximum flood discharge in the	13-05	409
Brahmaputra River. Model of—Bridge at Amingaon	1-22, 1-23	...	1-21	54, 54
Braiches. Prevention of—in river embankments in Sind...	1-27	76
Bridge piers. Maximum depths of scour round—	8-01 to 8-04	8-1 to 8-9	8-1	328 to 333
Buckley on Beileida equation for constant silt charge	3-23	133
Bukkur Gauge	Table 8-1	...	1-11	319
" Max. and min. flood levels	5-03	5-9	...	177
" Accretion in Gorge between Bukkur and Bohri	...	5-11
Bund, Bell	10-00, 10-02	379, 379
Bund Manual, Sind	5-23 (xvi)	206
C				
Canal. Eastern Nara—Variations in grade of bed material in—	7-06	290
" Jamrao—	3-15, 3-16	111, 119
" North Western—Effect of Approach Channel on	7-03, 7-05	289, 290
" Bico—Effect of Approach Channel on	7-04, 7-05	289, 290
" Rohri—Variations in grade of bed material in	7-06	290
Canals at Bukkur. Flat gradients of Left Bank	7-01	283
" at Bukkur. Steep gradients of Right Bank	7-01	283
Catchment formula. Inglis fan—	12-05	12-1, 2 and 3	...	409
Catchments. Annual run-off from—	12-06	418
" Cautley's flood run-off value for small—	12-05	407
" Inglis on run-off and floods of	12-05	409
Cautley's flood run-off value for small catchments	12-05	409
C. B. I. Paper No. 20 of 1939, Lacey's—	3-09	102
Channel at Bukkur. Effect of Approach—	7-02	289
" Definition of—	(xxv)	1
" Sand—experiments	1-16	...	1-16	63
Channels. Factors affecting flow in alluvial—	Chapter 7.	283
" And shoals in tidal models	13-14	439
" E. W. Lane's Paper of 1935 on Stable—	3-06	100
Characteristics of rivers. Factor determining—	2-11, 2-12	90, 90
Charge. Definition of—	(xiv)	2
" Significance of weighted mean (\bar{X}_w)	3-13	93
" To enter an off-take. Unsound to induce an excess.	12-03	403
Charge, terminal velocity and grade—their effect on area, velocity, slope, shape and width of a channel.	3-24	136
Charge (X) and terminal velocity (V_s) and their effect on Lacey formulae as modified by Sir Claude Inglis.	3-24	136
Chenab (Alexandria Bridge, Punjab)—Maximum flood discharge in the	12-05	409

INDEX

C and D	Para. number.	Figure number.	Photo Sheet number.	Page No. of para.
Chenab (Khanki Bridge, Punjab)	13-05	409
" (Manala, Punjab)	13-05	409
" (Kivan Bridge, Punjab)	13-05	409
" (Shah Shah Bridge, Punjab)	13-05	409
" Lower	3-15, 3-16	111, 119
Coefficient of discharge of remodelled Head Regulator of Nira L. B. Canal. High—	13-05	431
Coefficient of discharge of standing Wave Flumes	13-03	423
Coefficients. Lacey's early formulae with latest—	3-04	99
Commonsense not a guide to river behaviour	13-00	407
Concave submerged vane at Sukkur	1-11, 3-06	...	1-11	46; 230
Conclusions regarding hydrodynamic model experimentation.	13-39	453
Conditions under which a falling apron may fail	9-09	362
Confusion due to lack of precision in terminology	Symbols & Defs.
Continuity of progression of waterfalls	3-10	33
Continuation. Width—Definition of	(ix)	5
" Length— " "	(ix)	5
Control of rivers by repelling groynes	10-04	380
Coseys (Mohanpur Anicut, Bengal), Maximum flood discharge in the—	13-08	409
Crossing, Cross-over. Definition of	(xlii)	4
Curvature of flow (In many Chapters but see especially)	6-03, 6-05 6-20	6-2 (a) to (d)	...	217, 220 293
Curvature of flow. Application of—to river training	10-03	380
" " flow associated with meanders—	10-03	380
" " flow. Effect of—on sand exclusion	Appendix 3	...	A2-1, A3-2	471
Out-off. Definition of—	(xliii)	4
" Demonstration model of—	1-14	51
" Ratio. Definition of	(xliii)	5
Outback, Grijasa. Model of Mahanadi and Katjuri at—	1-24	...	1-22	64
D				
d=average depth=A/W	(ix)	2
D=a length characterising the effective depth of an alluvial channel—unless otherwise stated.	(v)	2
Damodar (Bengal). Maximum flood discharge in the	13-05	409
Damodar flood	13-05	12-5	...	409
Dasan River at Pahari (U. P.). Maximum flood discharge in the—	13-05	409
Datum area at Sukkur. Changes in—	5-13	5-12	...	184
Definitions. Symbols and—	Symbols & Defs.	...	1-3, 1-6	1
Deflector	1-05 and 1-09.	...	1-9	40 & 46
Delta. Seaward extension of—	5-05	171
Demonstration models.	1-13	...	1-15	51
Dumbly T-headed groynes	10-00	10-1	...	379
	10-01	379
Deposition of silt	5-03	169

INDEX

E and F	Para. number	Figure number	Photo Sheet number.	Page number of Para.
Exaggerated. Vertically—(A) tidal and (B) river models ...	13-13	439
Exaggerated length of standing wave in a vertically-exaggerated model.	13-34 (iii)	448
Excess charge entering Mithrao Canal. Effect of temporary—	12-08	408
" " Unsound to induce an—to enter an off-take...	12-08	408
" energy. Merits and demerits of Canal Falls and Regulators for absorbing—	12-04	408
" friction loss in rigid models	13-03	438
Excluders and ejectors. Relative merits of—	6-27, 6-80	275, 275
Excluders. Divide Wall—	6-19	6-15, 6-16	1-10, 6-3	257
" F. F. Haigh on—	6-25	267
" Open-type	6-08 to 6-13, 6-15 6-16	219 to 242, 245, 246
" Sand—	Chapter 6 Appendix 3	217 471
" Tunnel-type, shingle—at Tajewala	6-24	267
" Tunnel type—	6-02 6-28 to 6-25, 6-27	219, 267 to 267 275
Exclusion. Additional—difficulties when Canals 'take' off from both banks.	6-28	6-4, 6-22 6-23	...	275
Exclusion of bed sand at Head of Dim Minor, Jamrao Canal, Sind.	1-15	...	1-16	51
" Effect of curvature of flow on sand—	Appendix 3	...	A3-1 A3-2	471
" of bed sand from Canals, factors affecting—	Appendix 3 Chapter 6	A3-1 to A3-14	A3-1 A3-2	471 217
Experience. Importance of field—in mobilis' investigations.	1-31	76
F				
$f_{vs} : f_{ms} : f_{m} : Q$ relation in a canal system	8-16	119
Facilities available at Poona Research Station	1-02	86
Factors affecting exclusion of bed sand	Appendix 3	...	A3-1, A3-2	471
" affecting flow in alluvial channels	Chapter 7	283
Failure of Hardinge Bridge aprons	9-10	...	9-2	362
Fall. Remodelling of Tando Mastikhah	1-06	40
Falling aprons	Chapter 9	355
" apron. Conditions under which a—may fail	9-09	362
Falls and Regulators for absorbing excess energy. Merits and demerits of—	12-04	408
" Design of Flume meter—	1-06	40
" Scour downstream of flumed—	13-09	432
Fan catchment formula. Inglis'—	12-05	12-1, 2 and 3	...	409
Field inspection. Importance of—	1-31	76
Fish-like movement of sand particles	(ix)	476
Flared siphon outlet. Increased discharge due to—	13-10	433
Flat gradients of Left Bank Canals at Sukkur	7-01	283
Flexible design? Foolproof or—	12-00	407
Flood formula, Beale's—	...	12-3
" " Dicken's—	12-05	12-1	...	409
" " Gutmann's—	...	12-3
" " Inglis' fan catchment—	12-05	12-1, 2 and 3	...	409
" " Khangar and Gulhati's—	...	12-3

INDEX

F and S.	Para. number.	Figure number.	Photo Sheet number.	Page number of Para.
Flood formula Ryve's—	12-05	12-1	...	409
" " Whiting's—	...	12-1
" levels. Progressive rise of—in alluvial rivers	...	5-07
" of Baitarni River. Maximum—	12-05	409
" Unprecedented—in Khari River in Ajmere-Marwar...	12-05	409
" run-off value for small catchments. Cautley's	12-05	409
Floods. Damodar—	12-05	12-5	...	409
" Girna—	12-05	12-4	...	409
" Son—	12-05	409
" in Baluchistan	12-05	409
" of catchments. Inglis on Run-off and—	12-05	409
Flow. Distortion of—	12-02	407
" Bose's—formula of 1936	8-08	98
" Factors affecting—in alluvial channels	Chapter 7	283
" Historical Note on regime—	Chapter 8	95
" Lacey Papers of 1919 and 1933 on—in alluvium	8-02	96
" round a bend. Interference with natural—	12-02	407
Fluctuating discharges. Effects of—	Chapter 11	387
Fluming.	Chapter 11	387
" to attain modularity	11-01	11-1	...	387
" increase discharge	11-03, 11-04	11-4 to 11-13	...	387 333
" reduce cost	11-02	...	11-1	387
" of masonry works	1-04	40
Flume. Standing Wave—	1-04	40
Foolproof or flexible designs?	14-00	407
Formula of 1919. Lindley's—	8-01	95
" " 1936. Bose's—	8-03	98
" with latest co-efficients Lacey's—	8-04	99
F = Froude number = V^2/gD in terms of depth	(xix)	2
Fr = Froude number = V^2/gV in terms of width	(xix)	2
Francis Spring. Sir—	Chapter 9	355
Friction loss in rigid models	9-11	363
" Excess—	13-85	419
Friedkin's Vicksburg meander experiments	4-05	151
Froude number	13-13	439
G				
g=gravity	(xviii)	2
Gandak River	13-23	444
" " at Bagaha. Model of—	1-23	...	1-21, 1-19	54 409
Gangan (U. P.) Maximum flood discharge in the—	12-05	409
Ganges. Hardinge Bridge over—				
Experiments and recommendations	1-19, 10-C2	53, 379
" Hardinge Bridge. Models of—	1-19	...	1-18	63

INDEX

	Para. number.	Figure number.	Photo Sheet number.	Page number of Para.
Ganges held along Left Bank at Benares by outcrop of Lime Karak.	10-05	380
Ganges (U. P., Raiwala) Maximum flood discharge in the—	12-05	409
Ganges (U. P., Raiwala) " " " " " "	12-05	409
Garai (U. P., Karman) " " " " " "	12-05	409
Geometrically-similar models giving similar results	12 02 to 12-04	428 to 430
Geometrically-similar models not giving similar results	12-05	431
Geometrically-similar models which give partly geometrically similar and partly dissimilar results.	12-10	433
Ghat catchments, Bombay Presidency.				
Raintall to run-off relation in—	12-06	446
Ghatprabha at Dhupdal Weir (Bombay).				
Maximum flood discharge in the—	12-05	409
Gibb module	1-1	...
Gibson, Professor A. H.—	12-10	433
Girna floods	12-05	12-4	...	409
Girna near Malagaon, Nasik (Bombay).				
Maximum flood discharge in the—	12-05	409
Godavari at Nandur-Madhmeshwar (Bombay).				
Maximum flood discharge in the—	12-05	409
Gordon A	1-22	77
Gorge. Progressive accretion in—3 miles upstream of Sukkar Barrage between Rohri and Bukkur Island.	5 12	5-11	...	184
Grade of bed material in Eastern Nara Canal. Variations in—	7-06	295
" " Material in Rohri Canal. Variations in—	7-03	290
" " materials exposed on channel beds	Symbols and Defs.
" " Effect of reduction in—	Chapter 2	85
Grade, charge and terminal velocity—their effect on area, velocity, slope, shape, and width of a channel.	8-24	136
Grade of Stone required in falling aprons	9-12	370
" to discharge relation in a canal system, Inglis on ...	3-15	111
Gradients, Flat—of Left Bank Canals at Sukkur	7-01	283
" Steep—of Right Bank Canals at Sukkur	7-01	293
Groynes, Attack reduced by attracting—	12-01	407
" are usually anathema to river training engineer	10-01	379
" Attracting—	10-05	10-2	...	380
" Denchy T-headed—	10-00	379
" " "	10-01	10-1	...	379
" Misuse of—	12-01	407
" or spurs. Maximum depth of scour at—	8-05	447
" " "	Table 8—II	321
" Repelling—	10-04	380
" to protect Bilot Creek, Indus	10-01	379
Guide Banks	10-00	379
" " "	10-03	379
" " Maximum depth of scour at—	8-05	347
" " "	Table—II	321
Gulbati's flood formula. Khanger and—	...	12-3
Gutmann's flood formula	...	12-3

INDEX

H, I and J	Para. number	Figure number	Photo sheet number	Page number of para.
H				
Halgh F. F.—on excluders and ejectors	8-25	367
"	8-27	373
Harcourt Vernon	13-14	439
Hardinge Bridge Aprons, Failure of—	9-10	9-2	...	369
" " Experiments and recommendations	1-19	53
" " over Ganges	10-2	379
Hathmati at Khimatsagar (Bombay). Maximum flood discharge in the—	12-05	409
Hawes C. G.	1-82, 2-01	77, 86
Heading-up. Definition of—	(vi)	5
Hickin E. B. Construction of Approach Channel at Sukkur	6-09	241
High coefficient of discharge of remodelled Head Regulator of Nira L. B. Canal	13-05	431
High coefficient weirs	1-17	59
"	13-11	...	1-17	434
" level sill for excluding sand	6-11, 13-07	242
"	431
" sill off-take from Sukkur Approach Channel to exclude bed sand from Right Bank Canals	6-07, 13-07	...	6-1	231
"	431
Historical note on regime flow	Chapter III	95
Hooghly Tidal Model	1-25	...	1-23	68
Hydrodynamic model experimentation. Conclusions regarding—	13-39	431
I				
Inclination of launched apron slope	9-04	359
Incorrect throw-off in vertically-exaggerated models	13-33	447
Indus. Groynes to protect Biloi Creek	10-01	379
Indus (Attock, N. W. F. P.) Maximum flood discharge in the—	13-05	409
" (Ghazighat, N. W. F. P.) Maximum flood discharge in the—	13-05	409
" (Kaleabagh, N. W. F. P.) Maximum flood discharge in the—	13-05	409
Ingils' discussion on Dr. Bose's P. E. C. Paper	8-05	99
" fan catchment formula	12-05	13-1, 2 and 3	...	409
Ingils on $f_{v2} : f_{v3} : f_{v1} : Q$ relations in a canal system	8-16	119
" " Introduction of measure of normality and divergence from regime	8-05	99
" " Grade: discharge relation in a canal system...	8-15	111
" " $m \propto Q^{1/2}$ relation in a canal system	8-15	111
" " run-off and floods of catchments	13-05	409
" " Paper on "Divergence from Regime".	8-14	107
" " "meanders and their bearing on river training", Maritime and Waterways Paper No. 7, Instt. of C. E.	8-24	136
Inherent discrepancies in models	13-00	427
Instability of flow. Scour due to—	13-09	432
Interference with natural flow round a bend	13-02	407
Irrigation Development and Research Circle. Work done by	1-03	26
Isostatic depression	8-03	169
J				
Jamrao Canal	8-15, 8-16	111, 119

INDEX

J, K and L	Para number	Figure number	Photo sheet number	Page number of Para.
Jawai River at Erlinpura (Jodhpur State). Maximum flood discharge in the—	12-05	409
Jetting downstream of Sukkur Barrage	1-9	...
Jhelum. Lower—Canal	9-15, 9-16	111, 119
Jhelum (Keohala, Punjab) Maximum flood discharge in the—	12-05	409
" (Mangala, Punjab). Maximum flood discharge in the—	12-05	409
" (Rasul, Punjab). Maximum flood discharge in the—	12-05	409
Jamna (U. P., Faizabad) Maximum flood discharge in the—	12-05	409
K				
Kadwa at Palkhed (Bombay). Maximum flood discharge in the—	12-05	409
Kaira. Erosion of bank by Wataik River at—	1-20	...
Kakar Lora (Baluchistan) Maximum flood discharge in the—	12-05	409
Kalabagh tunnel-type excluder	...	6-21(b)
Kankar. Attraction of lime—	10-05	580
Karmanasa (Silkhat Weir, U. P.) Maximum flood discharge in the—	12-05	409
" (Deodari, U. P.)	12-05	409
Katjuri River at Cuttack. Model of—	1-22
Kennedy's formula 1895	2-90	95
Kou (U. P., Gangan). Maximum flood discharge in the—	12-05	409
Khadakwasla Dam, Lake Fife (site of C. I. & E. R. S. later I. W. M. S.).	Frontpieces.
Khaisar River (Baluchistan). Maximum flood discharge in the—	12-05	409
Khangar and Gulhati's flood formula	...	12-3
Khanki tunnel-type excluder, designed and constructed by the late H. W. Nicholson.	6-23	6-20	...	267
Khari River in Ajmere-Merwara. Unprecedented flood in—	12-05	409
Khigro Canal (See Mithrao Canal)	6-20	258
Kho (U. P. Headworks). Maximum flood discharge in the—	12-05	409
King C and Blench T.—Paper on "Effect of dynamic shape on Lacey relations" Annual Report (Tech), C. B. I. (India), 1941.	3-22	130
King G. Paper on "Practical design formulae for stable irrigation channels" Annual Report (Tech), C. B. I. (India), 1943.	3-22	130
King's Vanes for excluding sand	6-18	6-13, 6-14	6-3	246
Kosi Bridge. Thickness of stone at—	9-11	368
Koyana at Helwak (Bombay). Maximum flood discharge in the—	12-05	409
L				
Lacey. Gerald—	1-82, 2-02, 2-08, 2-07, 2-00	77, 86, 87, 96, 327.
Lacey's C. B. I. Paper No. 29 of 1939	2-09	102
" discussion on Lane's Paper	2-07	100.
" " " White's "	2-19	107
" early formulae with latest coefficients	2-04	99

INDEX

L and M	Para number.	Figure number.	Photo sheet number.	Page number of Para.
Lacey's Papers of 1929 and 1933 on flow in alluvium ...	8.02	96
"shock theory" ...	8.10	102
Lacey G.—Paper on "A general theory of flow in alluvium" journal Institution of Civil Engineers, November 1946, Paper No. 5518.	8.29	19a
Lalpur Tank (Rajkot State). Maximum flood discharge in the—	12.05	469
Lane, E. W.—Paper of 1935 on "Stable Channels".	8.06	120
Lane's reply to discussion	8.08	101
" Paper. Lacey's discussion on—	8.07	100
Launched apron slope. Inclination of—	9.04	359
" stone. Thickness of—on slope	9.11	363
Launching of an apron	9.02	356
Layers of sand and clay. Apron laid on alternate—	9.08	361
Lehri—Khamwan River (Baluchistan). Maximum flood discharge in the—	12.05	409
Length. Exaggerated—				
" of standing wave in vertically-exaggerated model	13.34 (iii)	448
" Scale ratios of—	13.25	445
Lindley's formulae of 1919	8.01	85
Lime Kanker. Attraction of—	10.05	380
Limitation of models	1.31, 13.39	78 451
Lines of flow at offtakes	13.03	6-2	...	493
List of Annual Reports (Technical)	Appendix 1	453
" " Bombay P. W. D. Technical Papers	Appendix 1	453
Longitudinal distortion	13.36 (v)	450
Lower Chenab Canal	8.15, 8.16	111, 119
" Jhelum Canal	8.15, 8.16	111, 119
" Sind Barrage.	10.05	380
M				
m = weighted mean diameter in mm.	(xxvi)	2
$m_{1/2} \propto Q$ in a canal system (Ingalls)	8.15	111
Mahanadi River Model	1.24	...	1-22	64
Mahanadi (Narej Weir, Orissa). Maximum flood discharge in the—	12.05	409
Makhi Regulator. See Eastern Nara and Mithrao	1-15	...
Malaprabha. Model of Bridge over—at Hol-Alur (M. & S. M. Railway)	1-15	...
Malhotra. Dr.—on "shock theory"	8.14	107
Maximum depth of scour round bridges	8.01, 8.04, Table 8-1	8-1 to 8-9	8-1	329, 338, 319
Maximum flood formulae	12.05	409
" " of Baitarni River	12.05	409
" " "	4.03	149
Meander experiments	380
" length determined by dominant discharge	10.03	380
Meandering of Rivers	See Chapter 4	143

INDEX

M and N	Para. number.	Figure number.	Photo sheet number.	Page number of para.
Meanders Adjustments to meet varying river conditions...	4-67	153
" Comparison of—in river models	4-04	150
" Conditions giving rise to—	4-06	152
" Curvature of flow associated with—	18-03	380
" Definitions of—	(xxxvii) to (xi)	4 to 4
" Factors affecting—	4-01	143
" Field data	4-02	144
" Vicksburg experiments	4-05	151
"Meanders and their bearing on river training" by Inglis, Maritime and Waterways Paper No. 7, Inst. of C. E.	3-24	136
Misuse of groynes	12-01	407
Mihrao Approach Channel	6-14, 6-20	6-12 6-18	6-2 to 6-5	245, 253
" Canal. Effects of temporary excess charge in—	12-03	408
" " Exclusion of bed sand from—	6-14 to 6-20	6-12 6-18	6-2 to 6-5	245 to 258
Mixed sizes of stone in aprons	9-05, 9-06, 9-11	360, 360, 36
Mobile river models. Vertically-exaggerated—	13-23	444
Models. Conclusions regarding—	13-39	451
" Discrepancies inherent in river—	13-09, 13-39	437, 451
" Equivalent sizes of stones and blocks in—	Appendix 4	485
" Exaggerated length of standing wave in vertically-exaggerated—	13-34 (iii)	443
" Excess friction loss in rigid—	13-35	449
" Incorrect throw off in vertically-exaggerated—	13-38	447
" Necessity of different scale ratios of rigid and non-rigid portions in vertically-exaggerated—	13-35 (iv)	449
" Rigid tidal—	13-15, 13-16	441
" River—	13-21	443
" Semi-rigid tidal—	13-17 to 13-20	441 to 442
" Vertically-exaggerated rigid river—	13-21	443
" Vertically-exaggerated mobile river—	13-23	444
" Vertically-exaggerated (A) tidal and (B) river—	13-12	439
Modularity. Fluming to attain—	11-01	11-1	...	387
Movement of bed ripples and bed dunes	Appendix 2	459
" Bed—The main difficulty in mobile models	13-28	444
Mula-Mutha at Jamsetji Bund, Poona (Bombay), Maximum flood discharge in the—	12-05	409
Mula at Chikalthan (Bombay). Maximum flood discharge in the—	12-05	409
Mula at Bahuri (Bombay). Maximum flood discharge in the—	12-05	409
Mula at Tea (Bombay). Maximum flood discharge in the—	12-05	409
Mutha at Khadakwasla Dam (Bombay). Maximum flood discharge in the—	12-05	409
N				
Nala upstream of M. & S. M. Railway Bridge, 445 (Guntakal-Berwade Section-mile 483). Maximum flood discharge in the—	12-05	409

INDEX

N, O and P.	Para. Number.	Figure Number.	Photo Sheet No.	Page No. of Para.
N = Coefficient of rugosity with subscript if necessary K = Kutter. M = Manning	(xxii)	2
Na = Lacey's coefficient of rugosity $Na = \frac{1.486}{V} R^{\frac{1}{2}} g^{\frac{1}{3}}$...	(xxiii)	2
Nara. Eastern—Canal	6-14	245
" " " Variations in grade of bed material	7-06	290
Naraj Weir. "Tilting" of—	13-37 (vi)	450
Nari Rivers (Baluchistan)—Maximum flood discharge in the—	12-06	409
Natural flow round a bend. Interference with—	12-02	407
Nature's way of excluding sand	6-03	219
Necessity of different scale ratios of rigid and non-rigid portions of vertically-exaggerated models.	13-35 (iv)	449
Nicholson, H. W.	1-22, 5-18, 6-23, 6-20	77, 301 267, 275
Nile at Beleida. Phillips equation for the—	8-23	139
Nira at Vir (Bombay). Maximum flood discharge in the—	12-05	409
Nira Left Bank Canal. Head Regulator. Effect of re-modelling of—	11-04	388
Nira Left Bank Canal. High coefficient of discharge of remodelled Head Regulator of—	13-05	431
Normality and divergence in flow by Inglis	3-05	99
North Western Canal. Effect of Approach Channel on—	7-03, 7-05	289, 290
North Western Canal. Effect of depth of sand on bed of—	6-09	6-8	...	241
North Western Canal at Sukkur	1-12	51
O				
Object of Report on the Behaviour and Control of Canals and Rivers.	1-01	85
Oderolal Regulator	1-10	...
Off-take. High sill—from Approach Channel at Sukkur to exclude sand.	13-07 6-07	431, 281
Off-take. Lines of flow at an—	13-09	6-2	...	432
One-man stone. Thickness of launched apron of—	9-11	363
One-man stone	9-11	363
Open-flow. See Still-pond.	6-19	6-16	6-3	257
Osborne Reynolds	13-12 13-13	439 439
Outer bank of Approach Channel	1-14	...
Outfall. Gauge at—cross-section	...	6-6 (e)
P				
P = wetted perimeter	(vi)	2
Paharpur Canal	10-01	379
Panjhra near Nahaed (Bombay). Maximum flood discharge in the—	12-05	409
Panjhra near Madhi Amalner (Bombay). Maximum flood discharge in the—	12-05	409
Parlat Nala (C.P. & Berar). Maximum flood discharge in the—	12-05	409
Pavement. Modification of— at Sukkur Barrage	1-07	45
Phillips equation—for the Nile at Beleida	3-23	134

INDEX

P, Q and R.	Para. number.	Figure number.	Photo sheet number.	Page No. of Para.
Fiern. Maximum depth of scour round bridge—	8-01, 8-04 Table 8-I	8-1 to 8-9	8-1	328, 333 319
Maximum depth of scour downstream of bridge—	8-06 Table 8-III	8-10	...	348 322
Models of Sukkur Barrage—	1-08	...	1-8	45
Ponding. Definition of—	(xlviii)	5
Effect of—on water levels upstream	5-06	173
Poonah (Punjab) Maximum flood discharge in the—	12-05	409
Pravara at Ojhar (Bombay) Maximum flood discharge in the—	12-05	409
Precision in terminology. Confusion due to lack of—	Symbols & Defs.
Q				
Q=Discharge in cusecs	(i)
q=Discharge per ft. width	(ia)
Quantity of stone required in falling aprons	9-13	370
R				
R=Hydraulic mean depth=A/P	(vii)
R _n =Reynolds number=(VD)/ ν	(xiv)
Rainfall. Relations in Ghat estohments Bombay Preci- sideopy between—and run-off.	12-06	418
R ₂₅ ° S°	8-15	107
Ratios. Scale—in vertically-exaggerated models	13-24 to 13-31	445 to 447
Ratmau (U. P., Dhanupuri). Maximum flood discharge in the—	12-05	409
Rayleigh	13-14	439
Rayleigh's wave relation $\sqrt{w} = \sqrt{gd}$	13-14	439
Reach. Definition of—	(xxxvi)	4
Recession of valleys	5-05	171
Reduction. Effect of—of charge	5-18	201
" " " discharge	5-18	201
" " " grade	5-18	201
Cause of—in silt charge at Bq ur and Kotri.	5-21	203
Regime and Stability	Symbols and Defs.
" flow. Historical note on—	Chapter 8.	95
" Inglis on divergence from—	3-14	107
" Lacey's definition.	2-02	86
" Lacey—formulae	2-03	87
Regulator. Effect of remodelling Nira Left Bank Canal Head—	11-04	388
Regulators for absorbing excess energy. Merits and demerits of—	12-04	408
Reinforcing a fallen apron	9-14	372
Remodelling. Effect of—Nira Left Bank Canal Head Regulator.	11-04	388
Remodelling. Effect of—pavement of Sukkur Barrage	1-07	45

INDEX

R and S.	Para. number	Figure number	Photo sheet number	Page No. of Para.
Repelling groynes above B. N. Railway Bridge over Rupnarain.	10-04	380
Control of river by—	10-04	380
Reports. List of Annual Technical—	Appendix 1	453
Retrospection. Definition of—	(xiv)	5
of water levels at Sukkur	5-09	178
Sympathetic—	5-18	301
Reynolds, Osborne—	13-13	439
"	13-18	439
" Criterion of turbulent flow	13-13	439
"	13-13	439
Rice Canal, Sind. Accretion in—overcome	7-04	289
" Depth of sand on bed of—	6-09	6-9	...	241
" Effect of Approach Channel on—	7-04, 7-05	289, 290
Rigid river models. Vertically-exaggerated—	13-21	443
Rigid tidal models. Vertically-exaggerated—	13-15	441
"	13-16	441
Ripple movement	2-10	2-2	2-2	89
Ripple turbulence	2-04	87
Ripples. Movement of bed—and bed dunes	Appendix 2	459
Rise of water levels due to Sukkur Barrage	5-09, 5-10	5-10	...	178, 182
River. Application of curvature of flow to—training	10-03	380
River. Experiments with—models	13-39	451
River models. See Chapter 13	13-21	443
River training engineers. Groynes are usually anathema to—	10-01	379
River Training—"Meanders and their bearing", by Inglis, Maritime and Waterways Paper No. 7, Inst. of C. E.	3-34	158
River. Vertically-exaggerated rigid—models	13-22	443
Rivers. Professor C. M. White's Paper of 1939 "The Influence of Transported Solids on—"	3-11	102
Rohri Canal. Variations of grade of bed material in—	7-06	290
Run-off and floods of catchments. Inglis on—	13-05	409
from catchments. Relation of Annual—to rainfall in that catchment, Bombay Presidency.	13-06	418
Rupnarain. B. N. Railway over—	10-04	380
" model	1-28	...	1-24	73
" Repelling groyne above—	10-04	380
Ryve's maximum flood formula	12-05	12-1	...	409
S				
S = slope = energy gradient	(xii)	2
S' = slope per 1,000 ft.	(xiii)	2
S' V: E _s S'	3-13	107
Salberg F. J. Discussion by—on Rupnarain model	10-01 12-01 1-16	379, 407 52
Sand channel experiments	6-07, 6-16	6-5 6-7 (a)	1-16	281, 246
Sand excluding sill	Appendix 3	...	A3-1 A3-2	471
Sand exclusion. Effect of curvatures of flow on—	Appendix 3	471

INDEX

S	Para. number.	Figure number.	Photo sheet number.	Page No. of Para.
Sand. Factors affecting exclusion of bed--	Appendix 3	...	A3-1, A3-2	471
Sand from Right Bank Canals. High sill offtake from Approach Channel at Sukkur to exclude bed--	6-07, 13-07	...	6-1	231, 431
Sand=material >0.10 mm. and <0.60 mm.	2-01	65
Sand movement	2-10 (d)	2-2	...	90
Sand Scale ratio of--in models	13-31	447
Sanding=deposition of sand	(iii)	5
Sarda Bridge. Thickness of stone at--	2-11	368
Sarda River. Model of groyne in--at Bambassa, to protect Right Afflux Bund.	1-20	...	1-19	54
	1-17	52
Scale effect
" ratio. Depth--	13-28	443
" " Discharge--	13-27	445
" " in vertically-exaggerated models	13-24 to 13-31	445 to 447
" " Length--	13-25	445
" " Sand--	13-31	447
" " Slope--	13-29	443
" " Time--	13-30	447
" " Width--	13-26	445
Scale ratios of rigid and non-rigid portions of vertically-exaggerated models. Necessity of different--	13-35 (50)	449
Scour--	Chapter 9	355
" and accretion. Time scales different for--	13-88 (vi)	450
" definition of--	(iv)	5
" downstream of flumed falls	13-09	432
" due to instability of flow	13-09	432
" Maximum depth of--downstream of bridge piers	8-06	8-10	...	348
	Table 8-III	322
" Maximum depth of--round bridges	8-01, 8-04	8-1 to 8-9	8-1	328, 333
	Table 8-I	319
" Maximum depth of--at guide banks, groynes (or spurs) and sharp heads.	8-05	347
	Table 8-II	321
Semi-rigid tidal models	13-17, 13-18, 13-19, 13-20	441, 442 442, 443
" " vertically-exaggerated models	13-12	439
Sheet movement	2-08	88
Shock theory. Lacey's--	3-10	102
" " Dr. Malhotra on Lacey's--	3-14	107
Shoals and channels in tidal models	13-14	439
Short spur at Bambassa to protect Right Afflux Bund	1-19	...
Silauria Nala (O. P. & Berar). Maximum flood discharge in the--	13-05	409
Sill. High--off-take from Approach Channel at Sukkur to exclude excess bed sand from Right Bank Canals.	6-07, 13-07	231, 431
Silt, silting.	Symbols & Defn.
" charge. Progressive reduction of--at Sukkur	5-16, 5-21	5-15, 5-16	...	189, 203
" and sand exclusion	6-10	6-5, 6-7(a) 6-10	1-16	242
Silting. Definition of--	(2)	5

INDEX

S	Para. number.	Figure number.	Photo sheet number.	Page No. of Para.
Silting in Right Bank Canals, Sukkur	1-05	40
Similar. Geometrically-similar models which give partly- and partly dissimilar results, siphon spillways.	13-10	433
Similar results. Models giving geometrically--	13-02 to 13-04	438 to 430
Sind Barrage. Lower--	10-05	380
Siphon. Increased discharge due to adding fixed outlet--	13-10	433
Siphon spillways	Frontis-piece.	...
" " Geometrically-similar models give partly similar and partly dissimilar results.	13-10	433
Size of stones and blocks. Equivalent--to be used in models.	Appendix 4	455
Slips in falling aprons	9-07	361
Slope exaggeration. Definition of--	(loi)	5
" scale ratio	13-29	448
Son at Dehri. Exclusion of sand from--	6-28	375
" Maximum flood in--River	13-05	409
Son (Bihar, Dehri). Maximum flood discharge in the--	13-05	409
Special Irrigation Division. Work done by--	1-03	86
Specific discharge gauge reading. Definition of--	(zzzi)	3
" discharge gauge reading downstream of Sukkur Barrage.	5-09	5-10	...	178
" discharge gauge reading. Short period variations in--	5-15	5-13, 5-14	...	189
Spring Sir Francis--	Chapter 9 10-00, 10-02	355 379
" " " "River Training and Control on the Guide Bank System" G. of I. Ry. Board T. P. 153 (1903).	10-00	379
Spurs and sharp bends. Maximum depth of scour at guide banks, groynes (or --)	8-05 Table 8-II	347 331
Stability. Definition of--	(zzz)	3
" Regime and--	Symbols & Defa.
Stable channels. E. W. Lane's Paper of 1935 on--	3-06	100
Standing wave. Enforcement of--	1-09	...	1-3, 1-4 1-5 to 1-9	46
" " flume at Sukkur Barrage	13-06	431
" " flumes	1-2 to 1-6	...
" " Coefficient of discharge of--	13-08	433
" " in vertically-exaggerated model. Exaggerated length of--	13-34 (ii)	448
Steep gradients of Right Bank Canals at Sukkur	7-01	283, 257
Still pond--see open flow	6-19	6-16	6-3	...
Stone. One man--	9-11	368
" Mixture of--of various sizes in apron	9-11	368
" Thickness of launched--on slope	9-11	368
" Thickness of--at Kosi Bridge	9-11	368
" Thickness of--at Sarda Bridge	9-11	368
Stones. Equivalent size of--in models	Appendix 4	455
" of mixed sizes in aprons	9-05, 9-06	360, 360
" Quantity of--required in falling aprons	9-13	370

INDEX

S and T.	Para. number.	Figure number.	Photo Sheet number.	Page number of Para.
Submerged vane. Exclusion by means of a—	1-11, 6-06 6-17	...	1-11	46, 220 246
Sukkur Barrage. Causes of accretion at—	1-11	46
" " Approach Channel to—	1-12	51
" " Modification of downstream pavement of— to exclude excess sand.	1-07	45
" " Position of standing wave at—	13-06	431
" " Rise and N. W. Canal (Right Bank) at—	1-05	40
" " Rise of water levels at—due to accretion ...	5-09, 5-10	5-10	...	178, 182
" " Rohri and E. Nara Canals (Left Bank) ...	7-01	283
" " Specific discharge gauge readings down- stream of—	5-09	5-10	...	178
Symbols and Definitions	Symbols and Definitions.
Sympathetic retrogression	5-18	201
T				
T-headed groynes. Dmaby—	10-00 10-01	379 379
Tajewala tunnel-type excluder	6-24	267
Tando Mastikhan Fall. Remodelling of—	1-06	40
" " Fall. Variable coefficient of discharge of—	18-04	430
Tapti River	13-33	444
Technical. List of Annual Reports—	Appendix I	453
" List of Bombay—Papers	"	453
Temporary. Effect of—Inflow of sand on regime	6-21	266
" Excess charge entering Mithrao Canal. Effect of—	12-03	408
Terminal velocity, charge, and grade—their effect on Area, velocity, slope, shape and width of a channel.	3-24	136
Terminal velocity (V_g) and charge (X) and their effect on Lacey formulae as modified by Sir Claude Inglis.	3-24	136
Terminology. Confusion due to lack of precision in—	Symbols and Def.
Theorem. Bernouilli's—	11-00	387
Thickness of launched stone on slope	9-11	366
" " Stone—Kosi Bridge	9-11	358
" " " —Sarda "	9-11	368
Throw-off in vertically-exaggerated models. Incorrect—	13-33	447
Tidal models. Hooghly—	1-26	68
" " Rigid—	13-15, 13-16	441 441
" " Bupnarain—	1-26	73
" " Scale ratios of—	13-14	439
" " Semi-rigid—	13-17 to 13-20	441 to 442
" " Shoals and channels in—	13-14	439
Tilt. Definition of—	(viii)	5
Tilting of Naraj Weir	13-34	446
Time scale ratio	13-30	447
" scales different for scour and accretion	13-33	450
Tortuosity. Definition of—	(xi)	4
Training works considered anathema	12-01	407

INDEX

T, U, V and W.	Para. number.	Figure number.	Photo Sheet number.	Page number of Para.
Transported Solids on Rivers." Professor G. M. White's Papers of 1939 on "The Influence of—	8-11	102
Trimmu tunnel-type excluder	6-28 to 6-28	6-21 (a)	...	267 to 267
Tunnel-type ejectors	6-28 to 6-29	6-21 (a)	...	268 to 275
" " excluders (Elsden-Nicholson) Khanki, Kalabagh, Trimmu, Tajewala.	6-02, 6-23 to 6-25	219, 267 to 267
Turbulent flow. Reynold's criterion of—	13-12 to 13-13	439 to 429
U				
Unprecedented flood in Khari River in Ajmer-Merwara	12-05	409
Unsound to induce an excess charge to enter an off-take	12-09	408
V				
Vaghur at Reipur (C. P. and Berar). Maximum flood discharge in the—	12-05	409
V = mean velocity	(iii)	2
V _g = terminal velocity. Definition of—	(xxvii)	8
V _w = \sqrt{gDB} (effect of shape and slope of channels on wave velocity.)	1-17	...
Valley. Recession of—	8-05	...	1-17	171
Vana. Effect of Left Bank attracting—near Rohri	1-11	48
" Submerged concave—for excluding sand	1-11, 6-16 to 6-17	1-11	...	48, 246 to 246
Vanes. King's—	6-18	6-13, 6-14	6-8	246
" Effect of adding splayed—at Tando Mastikhan Fall.	1-06	40
Vardha at Banwasi (C. P. & Berar). Maximum flood discharge in the—	12-05	409
Variable coefficient of discharge at Tando Mastikhan Fall...	13-04	480
Variations in grade of bed material in the Eastern Nara Canal.	7-06	90
Variations in grade of bed material in the Rohri Canal	7-06	90
Velocity. Effect of shape of channel on wave—	I-17	...
Velocity terminal, charge and grade—their effect on Area velocity, slope, shape, and width of a channel.	8-24	186
Velocity terminal (V _g) and charge (X) and their effect on Lacey formulae as modified by Sir Claude Inglis.	8-24	186
Vernon-Harcourt	13-12	489
Vertical exaggeration. Definition of—	(iv)	5
Vertically-exaggerated (A) tidal and (B) river models	13-12	439
" -exaggerated mobile river models	13-23	444
" -exaggerated rigid river models	13-21	448
" -exaggerated. Scale ratios in—models	13-24 to 13-31	445 to 447
Vicksburg meander experiments	4-05	151
W				
W = a length characterising the effective width of an alluvial channel except where otherwise stated.	(iv)	2
W = surface width, or weight of a cu. ft. of water	(viii)	3
Watrak River (Kaira). Model of—	1-21	...	1-20	54
Wave relation V _w = \sqrt{gd} —Rayleigh's	13-14	439
White, G. M.'s discussion on Sir Claude Inglis' paper on "Meanders and their beasing on river training", Maritime and Waterways Paper No. 7, Inst. of C. E.	8-24	186
White, Professor G. M. Paper by—(1939) on "The Influence of transported solids on rivers."	8-11	102