

ANNUAL REPORT

(TECHNICAL)

1947

PART II

**CENTRAL BOARD OF
IRRIGATION INDIA**



सत्यमेव जयते

PUBLICATION NO. 40

ANNUAL REPORT
(TECHNICAL)
OF THE
CENTRAL BOARD OF
IRRIGATION INDIA
1947

PUBLICATION NO. 40

PART II

EDITED BY
N. D. GULHATI, I.S.E., M.I.E. (INDIA)
SECRETARY



सत्यमेव जयते

GOVERNMENT OF INDIA
PUBLISHED BY ORDER OF THE BOARD

U. D. CLASSIFICATION
[532+551+608.4+626+627+(058)]

SIMLA
JANUARY 1948.

ADVERTISEMENT

The Central Board of Irrigation as a body is not responsible either for the statements made or for the opinions expressed in the following pages.

THE CENTRAL BOARD OF IRRIGATION
LIST OF MEMBERS AS ON DECEMBER 1, 1947.

Government of India

1. Shree A. N. Khosla, I.S.E., M.I.E., (Ind.), Chairman, Central Waterways, Irrigation and Navigation Commission.
2. Shree S. C. Majumdar, Member, Central Waterways, Irrigation and Navigation Commission.
3. Mohsin Ali, Esq., Member, Central Waterways, Irrigation and Navigation Commission.
4. H. M. Mathews, Esqr., C.I.E., Chairman, Central Technical Power Board.

Assam

5. Shree R. Chandra, I.S.E., Chief Engineer and Secretary, Public Works Department.

Biharwalpur

6. James L. Roy, Esqr., I.S.E., Chief Engineer, Public Works Department.

Baluchistan

7. Col. G. H. Vaughanlee, O.B.E., Chief Engineer and Secretary, Public Works Department.

Bihar

8. Shree D. Mehta, I.S.E., Chief Engineer and Secretary, Public Works Department.
9. Shree M. L. Bahl, I.S.E., Chief Engineer and Secretary, Public Works Department.

Bombay

10. Shree N. P. Gurjar, B.E., I.S.E., Chief Engineer and Secretary, Public Works Department.

Burma

11. U. Aye Moun, Esqr. I.S.E., Chief Engineer Irrigation.

C. P. and Berar.

12. Shree C.B., Roy, I.S.E., Chief Engineer and Secretary, Public Works Department.

Ceylon

13. S. G. Taylor, Esqr., B.Sc., (Eng.), M.I.C.E., Director of Irrigation.

East Bengal

14. J. F. Russell, Esqr., C.I.E., O.B.E., I.S.E., Chief Engineer Irrigation.

East Punjab

15. Shree Gita Ram Garg, I. S. E., Chief Engineer and Secretary to Government (Canals).
16. Sardar Sarup Singh, I.S.E. Chief Engineer and Secretary, to Government (Projects).
17. Shree S. A. Gadkary, B.E., M.I.E.E., (Lond.), A.A.I.E., C.P.E., Tech., M.I.E., (Ind.), Chief Engineer Electricity.

Gwalior

18. Major N. K. Bhonsle, Chief Engineer, Public Works Department.

Hyderabad

19. S. M. Younus, Esqr., B.A., Chief Engineer Irrigation, Waterworks and Drainage.

Madras

20. Shree A.R. Venkatacharya, I.S.E., Chief Engineer Irrigation.
21. Shree N. Govindaraja Ayyangar Chief Engineer Irrigation (Special).
22. Shree G. Sundaram, Chief Engineer Electricity.

Mysore

23. Rajakaryaprasakta M. Narasimhaiya Chief Engineer (Irrigation).

North West Frontier Province

24. Col. W. G. Lang Anderson, O.B.E. R.E., Chief Engineer and Secretary, Public Works Department.

Orissa

25. J. Shaw, Esqr., I.S.E., Chief Engineer and Secretary, Public Works Department.
26. Shree R. L. Narayanan, Chief Engineer Electricity.

Sind

27. Syed Ghulam Mustafa, I.S.E., Chief Engineer and Secretary, Public Works Department.
 28. Shree M. P. Mathrani, I.S.E., Joint Chief Engineer and Joint Secretary, Public Works Department.
- I. A. W. Foy—(on leave).

Travancore

29. Shree K. Raja Raja Varma, Chief Engineer, Public Works Department.

United Provinces

30. Kunwar Balbir Singh, I.S.E. Chief Engineer Irrigation (Eastern Canals).
31. Shree Sumer Chand, I.S.E. Chief Engineer Irrigation (Development).
32. Shree P. C. Agarwal, M.B.E., I.S.E., Chief Engineer Irrigation (Western Canals).

West Bengal

33. Shree S. K. Guha, I.S.E., Chief Engineer Irrigation.

West Punjab

34. T. Burt, Esqr., I.S.E., Chief Engineer and Secretary (Irrigation).
35. E. L. Protheroe, Esqr., I.S.E., Chief Engineer and Secretary (Irrigation).
36. M. A. Hamid, Esqr., I.S.E., Chief Engineer and Secretary (Irrigation).

RESEARCH OFFICERS

Bombay

1. Shree J. P. Naegamvala, B.S.E., (I), Executive Engineer.

Central Waterways, Irrigation and Navigation Research Station

2. Shree K. K. Framji, I.S.E., Director.

Ceylon

3. Dr. R. V. Burns, B.Sc., Ph.D., D.I.C., Divisional Engineer.

East Punjab

4. Shree S. L. Malhotra, I.S.E., Director.

Hyderabad

5. Dr. S. P. Raju, Director.

Madras

6. Jenab T. P. Kuttiammu Sahib Bahadur, B.E., A.M.I.E., Executive Engineer.

Mysore

7. Shree V. Ganesh Iyer, Director.

Sind

8. Shree T. R. Sethna, Executive Engineer.

United Provinces

9. Shree R. S. Chaturvedi, Research Officer.

West Bengal

10. Dr. N. K. Bose, M.Sc., Ph.D., Director.

West Punjab

11. Lt. Col. T. Blench, I.S.E., Director.
-

FOREWORD

This Annual Report contains an account of research and other technical investigations on subjects connected with irrigation, waterways, hydro-electric, navigation and allied subjects, carried out in India and other member countries during the year 1947. It is published for the information of engineers and scientists in India and abroad who are interested in the work of the Central Board of Irrigation (India).

Summary reports of the work carried out during the year 1946 at eight out of the ten research stations in the member countries, and papers on specific subjects selected by the Board, together with the discussions thereon, appear under the respective subject-heads. The discussions constitute the technical, proceedings of :

- (i) The seventeenth annual meeting of the Research Committee of the Board held in Simla from July 7 to 12, 1947 and
- (ii) The eighteenth annual meeting of the Central Board of Irrigation, held in Delhi from December 1 to 6, 1947.

The preliminary notes were written by the Secretary for the information of those attending the Research Committee and the Board meetings.

It is regretted that, due to lack of space, it has not been possible to include in full articles from the annual reports of the research stations contributed for discussion at the Research Committee meeting. Abstracts or abridgements of articles not included in full are, however, provided. In other cases where abstracts or explanatory introductions were not supplied by authors, the Secretary has provided introductory abstracts.

The report like that of last year is printed in two volumes, but is arranged according to the latest grouping of subjects approved by the Board : Hydrology, Theory of flow and design of channels, Hydraulic Works, Water power works, Soil science, Water utilization projects and General. In addition as usual there are two chapters on Meetings of the Board and the Research Committee during the year and summarised annual reports of the research stations given in the beginning.

The undersigned would repeat the request, contained in previous reports, for the supply of information relating to any of the problems mentioned in the publication. He will gladly furnish further details of any of the experiments investigations *etc.*, or other information relating thereto.

N. D. GULHATI,

Secretary,

Central Board of Irrigation.

Simla

Dated January 17, 1948.

CONTENTS

PART I

	Page
ask before the architect—by Shri N.V. Gadgil	1
the Presidential Address—by Shri A. N. Khosla	4
I—MEETINGS OF THE BOARD AND THE RESEARCH COMMITTEE DURING 1947	
Informal Winter Meeting of the Research Committee	15
Seventeenth Annual Meeting of the Research Committee	15
Formal confirmation of the minutes of Proceedings of the 16th Research Committee Meeting	22
Eighteenth Annual Meeting of the Board	22
II—SUMMARISED ANNUAL REPORTS OF RESEARCH STATIONS	
Indian Waterways Experiment Station, Poona	27
River Research Institute, Bengal	30
Poona Irrigation and Research Division, Poona	33
Irrigation Research Station, Madras	34
The Punjab Irrigation Research Institute, Lahore	36
The Development and Research Division, Sind	38
The United Provinces Irrigation Research Station	41
Hydraulic Research Station Krishnarajasagar	44
III—TECHNICAL PROCEEDINGS : SECTION A—HYDROLOGY	
1A—Rainfall Runoff	
Preliminary Note	49
(i) Total volume of runoff including the standardisation of hydrographs and their analysis	50
Unit hydrograph of the Brahmani Catchment above Nalhati	50
Rainfall runoff experiments in trays	54
Unit hydrograph of the Damodar Catchment above Chas Road Bridge Site	65
Runoff from catchment around Krishnarajasagar reservoir waterspread	65
Discussion by the Research Committee	68
Consideration by the Board	70
(ii) Maximum runoff discharge	70
Determination of the optimum number of rain recording stations required in catchments for estimating runoff—by V. Satakopan and V. Vittal Sharma	70
The measurement of precipitation in hilly catchments with special reference to problems of runoff and other hydrologic studies—by L.Y. Baker	76
Maximum flood discharges from small catchments—by A.R. Thomas	111
Discussion by the Research Committee	123
Consideration by the Board	129
2A—River Behaviour	
Preliminary Note	135
(i) Floods	137
Consideration by the Board	137
(ii) Stage discharge relations	140
Correlation between the gauge heights at the Sainthia railway bridge site and the Tilpara Ghat barrage site on the river Mor	140
Gauge-discharge curves for the river Brahmaputra at Jogigopa	141

	Page
Correlation between gauge levels of the river Damodar at Raneegunge and Rhondia and the gauge discharge curve at Raneegunge	142
High floods in Krishnarajasagar	143
Hydrology—the new science and its application—by Dr. N.K. Bose ..	144
Discussion by the Research Committee	149
Consideration by the Board	152
3A—Measurement of flowing water	
Preliminary Note	153
Comparison of velocity observations by the float method and the current meter method	154
Water surface profiles observed in a five feet wide standing wave flume with parallel sides for various stages of flow	157
Rating of meters	159
Correlation of surface velocity observations with current meter observations at 0.6 depth	159
Experiments conducted to find the effect of temperature on the results of ratings of current meter—by K.K. Mahadevan	161
Some experiments done at Krishnarajasagar to study the effect of turbulence on the velocity records obtained with cups and propeller types of meters—by K. K. Mahadevan	164
Discussion by the Research Committee	172
Consideration by the Board	175
4A—The Sampling of Bed and Suspended Load	
Preliminary Note	176
Suspended and bed load characteristics of the river Brahmaputra at the railway bridge site near Jogigopa	177
Experiments on the measurement of suspended load	183
Experiments on the measurement of bed load.. .. .	185
Silt surveys	189
Discussion by the Research Committee	191
Consideration by the Board	192
5A—Regeneration and Losses	
Preliminary Note	193
(i) Seepage Losses	195
(ii) Evaporation Losses	195
Estimation of evaporation losses in Reservoirs	195
Discussion by the Research Committee	195
(iii) Regeneration	197
Regeneration of the river Yamuna above Okhala	197
Return flow by seepage into natural drainages (<i>nallas</i>) in the canal areas ..	197
Discussion by the Research Committee	203
Consideration by the Board	204
6A—Soil denudation as an adverse factor on river supplies and agriculture	
Preliminary Note	205
Discussion by the Research Committee	206
Consideration by the Board	208
7A—Sub-terranean Waters	
Preliminary Note	209
(i) Yield	210
Transmission co-efficient of sand samples taken between ground level and 12 feet depth from three test borings and its verification with transmission co-efficient obtained from yield test results in the Damodar river at Sindri	210
Discussion by the Research Committee	210

	Page
(ii) Effect of tubewell pumping on sub-soil water level	211
(iii) Recharging the underground basin	211
Infiltration (absorption by ground) factors which determine the portion of rainfall which infiltrates while also considering the effects of plants, the configuration of the soil <i>etc.</i> , and the importance of these influences—by D.V. Rao	211
Discussion by the Research Committee	212
(iv) Evaporation and transpiration losses	213
Consideration by the Board	213
8A—Quality of waters in relation to (i) Crops, (ii) Soils, (iii) structures and (iv) Industries	
Preliminary Note	214
(i) Crops	214
Suitability of drainage waters for Irrigation (Shingwa area)	214
Suitability of drainage waters for irrigation (Baramati quadrats)	215
Well waters in the effluent zone	217
Discussion by the Research Committee	218
(ii) Soils	219
Effect of sewage irrigation on black cotton soil	220
(iii)* Structures	227
Note on trade waste experiments in the Bombay Province	227
Discussion by the Research Committee	228
(iv) Industries	228
Consideration by the Board	228
SECTION B—THEORY OF FLOW AND DESIGN OF CHANNELS	
1B—Flow in rivers and canals	
Preliminary Note	231
(i) Flow in channels with rigid and quasi-rigid boundaries	231
Stilling of whirlpools in the kondepudi gorge of the Godavari river	231
Discussion by the Research Committee	233
(ii) Flow in Channels in incoherent and coherent alluvium	233
Design of channels in the Mor Project	234
Formulae for velocity at the chas-road bridge site on the Damodar	235
Relation between different discharge elements for the river Brahmaputra at Jogigopa	236
Rate of sand attrition in channels	240
An analysis of hydraulic data from selected discharge sites along barrage canals during 1946	244
Discussion by the Research Committee	249
Consideration by the Board	252
2B—Hydraulic Data of torrents in boulder rivers and streams	
Preliminary Note	253
Consideration by the Board	253
3B—Design of Irrigation Channels	
Preliminary Note	254
Consideration by the Board	254
4B—Design of drains in irrigated and tidal areas	
Preliminary Note	255
Discussion by the Research Committee	256
Consideration by the Board	256

	Page
SECTION D.—WATER POWER WORKS	
D—Water Power Works	
Preliminary Note	259
Consideration by the Board	260
1D—Estimation of Water Power	
Preliminary Note	261
2D—Power House and Equipment	
Preliminary Note	262
(iii) Hydraulic Works	262
Foundation of Khatima Power House, Sarda Canal	262
Verification of the observed lowering of water table at Mohammadpur Power House by electrical analogy method	266
Model investigations regarding the project of Mohammadpur Power House	268
Model investigations regarding the design of Pathri Power House	269
To remedy silting above the left forebay of Nirgajni Power Station	271
Discussion by the Research Committee	273
Consideration by the Board	273
3D—Plant Accessories	
Preliminary Note	276
4D—Regulation	
Preliminary Note	277
5D—Transmission	
Preliminary Note	278
Spacing to be maintained between power transmission lines of the electricity branch and the telephone and telegraph lines of the Indian posts and telegraphs department—by S.A. Gadkary	278
Consideration by the Board	279
6D—Financial aspects	
Preliminary Note	282
Consideration by the Board	284
SECTION E.—SOIL SCIENCE	
1E—Soil Mechanics	
Preliminary Note	287
(i) Fundamental Research	289
A method of determining the shrinkage factors of soils	289
Protection of Usehat <i>Bund</i> for wave wash with stabilised soil	292
Experiments in connection with determination of (a) Optimum moisture content (b) Bearing capacity and (c) Calcium chloride (hygroscopic) required for optimum bearing power on different typical soils	294
Field capacity in terms of clay content	296
(ii) Earth dams and Embankments	297
Shape of dam profile	297
Type of material and its effect on hydraulic gradient	300
Effect of scale on dam models	301
(iii) Roads and runways	302
(iv) Foundations	302
(v) Retaining walls, revetments and trenches	302
Discussion by the Research Committee	303
Discussion by the Board	307
2E—Movement of moisture in saturated and unsaturated soils	
Preliminary Note	310
Report on soil moisture and evaporation investigations being conducted by the Agricultural Meteorology Section at Poona—by Dr. L. A. Ramdas	312
Experiments on transmission coefficients of sands in Sind	326
Seepage into the foundation trench of the Ramapadasagar Dam	328

	Page
Lysimeter experiments	331
Moisture movements	331
Discussion by the Research Committee	331
Consideration by the Board	333
3E—Land Reclamation	
Preliminary Note	334
(i) Saline lands	335
Penetration depth of different intensities of irrigation	335
Vertical movement of salts under reclamation waterings	336
Reclamation	337
Discussion by the Research Committee	337
(ii) Water-logged lands	340
Effect of heavy cane irrigation on the sub-soil water table in the deep soil area near Jinti distributary 18—Nira right bank canal	340
Effect of perennial cane irrigation on the sub-soil water table and consequent damage to lands	342
Discussion by the Research Committee	346
Consideration by the Board	346
4E—Water requirements of crops	
Preliminary Note	348
Experiments to determine water requirements for sugar-cane and wheat crops for the maximum yield with the most economical use of water	349
The amount of water utilized per acre of sugar-cane by sugar factories in the Bombay Deccan Canal areas	352
Discussion by the Research Committee	353
Discussion by the Board	358
5E—Soil Surveys	
Soil survey of the area commanded by the Mor project	359
Discussion by the Research Committee	361
Discussion by the Board	363
SECTION F.—WATER UTILIZATION PROJECTS	
1F—Study of individual projects	
Two-crore scheme for the restoration of minor tanks in the Mysore State—by Shri Rajakaryaprasakta M. Narasimhaiya	367
The project for constructing a dam at Gangapur near Nasik, Bombay—feasibility of constructing the dam in earth—by Shri N. P. Gurjar	369
The Meshwa Canal Project, Bombay—design of the weir and head Regulator on pervious foundations—by Shri N. P. Gurjar	369
Discussion by the Board	369
2F—Financial aspects of water utilization projects	
Preliminary Note	372
Nangal hydel project—by Shri S. A. Gadkary	372
Discussion by the Board	376
3F—Legal aspects of water utilization projects	
Preliminary Note	378
Consideration by the Board	379
4F—Anti-Malarial aspects of water utilization projects	
Preliminary Note	380
Discussion by the Research Committee	383
Discussion by the Board	383
4F—Miscellaneous	
Consideration by the Board	388

SECTION G.—GENERAL		Page
1G—River Pollution		
Preliminary Note		391
Consideration by the Board		391
2G—Spray Irrigation		
Preliminary Note		392
Discussion by the Research Committee		394
Consideration by the Board		394
3G—Definition of terms in common use		
Preliminary Note		395
Consideration by the Board		397
4G—Standard Notation		
Preliminary Note		398
5G—Nomograms		
Preliminary Note		399
6G—Standardisation of steel sheet piling		
Preliminary Note		400
Standardisation of the steel sheet piling bars in India		404
Discussion by the Research Committee		412
Written comments by Mr. Phiroz Kutar		418
Discussion by the Board		423
7G—Soil Conservation		
Preliminary Note		425
Consideration by the Board		425
8G—The Selection of subjects for discussion at the following meeting		
Preliminary Note		426
Grouping of subjects on the Agenda		427
Discussion by the Research Committee		429
Consideration by the Board		432
9G—Coordination of Research Work for the ensuing year		
Discussion by the Research Committee		433
Discussion by the Board		434
10G—Any other business		
Preliminary Note		436
Discussion by the Research Committee		436
Discussion by the Board		436

APPENDICES**Appendix I**

Minutes of a meeting held at the Meteorological Office, Lodi Road, New Delhi on 30th November, 1946 to discuss the scheme of investigating water balance of the Damodar Catchment	439
Minutes of proceedings of the second meeting of the sub-committee on water balance investigation of the Damodar Valley	440
Minutes of proceedings of the third meeting of the sub-committee on water balance investigation of the Damodar Valley	441

Appendix II

Item XII of the Administrative agenda, 1947—To urge on the Government of India, the importance and necessity of legislation to control and assist in the proper development and conservation of the water resources of the country and the desirability of an early enactment of a comprehensive waterways (including ground-water) Irrigation and Navigation act	444
List of existing Indian Legislations regarding the utilization and conservation of natural waters	444
List of Publications issued by the Central Board of Irrigation	(6)

PART II

SECTION C—HYDRAULIC WORKS

1C—Limitations of model experiments and comparison of models of prototypes	
Preliminary Note	449
Model limitations as regards bank erosion and accretion upstream and downstream of a tree Groyne	451
Model experiments in connection with scour hole below head regulator over Rohri Canal	452
Discussion by the Research Committee	452
Consideration by the Board	457
2C—River Control	
Preliminary Note	458
(i) Flood protection works	462
Re-distribution of sand load between the Mahanadi and Katjuri system at Cuttack (Orissa)	462
Flood absorption in Krishnarajasagar	473
An unprecedented flood in Mula and Pravara Valleys and estimate of loss of flood damage	481
Meandering rivers and flood control—by K. B. Ray	485
Discussion by the Research Committee	487
(ii) River training works	490
Model experiments for the prevention of erosion at Kurigram by the river Dharla	491
Erosion of the river Torsa	491
Model experiments for the proposed Mor barrage at Tilpara	491
Training the river Luni at Railway Bridge No. 3 of the Samdari—Raniwara branch of the Jodhpur Railway	493
Training of the Jumna river at Delhi gate pump house by means of a pitched embankment	494
Training of the Watrak River at Kaira (Bombay Presidency)	494
Training of the Sutlej river at Samasata, about six miles downstream of the Empress bridge at Adamwahan North Western Railway	498
The Rupnarian river upstream of the goods yard, Kolaghat Railway Station upstream of the Bengal Nagpur Railway Bridge	499
Training of rivers with permeable tree groynes	500
Beas river—Railway bridge at Dhilwan in district Jullundur	506
Ravi river—Spur at Baherian—district Gurdaspur	507
Rivers Chenab and Jhelum at Trimmu headworks	507
River Sutlej at Samasata Railway Station	508
Shape of spur apron for conventional types of Spurs	508
Effect of the angle of approach on the shape of the apron of a T-headed spur	508
Discussion by the Research Committee	509
(iii) Scour and erosion	517
Training of the river Indus above the Sukkur Barrage to prevent slips of the right bank below the nose of the approach channel	518
Emergent measures for preventing serious damage to the central sluices of the Mahanadi anicut at Cuttack, Orissa	518
Scour downstream of rigid structures in wide channels—prototype data regarding depth of scour downstream of bridges	519
Coffer dams for the Ramapadasagar Project	521
The Hagari aqueduct for the Tungabhadra Canal—Scour around piers	527
Discussion by the Research Committee	529

	Page
(iv) Spill	536
Model experiments for the revival of the river Chandana	537
Discussion by the Research Committee	544
(v) Meandering	545
Deterioration of the River Kultigong	546
Proposed railway bridge near Shahabad on the Ram Ganga river	551
Discussion by the Research Committee	563
(vi) Tidal action including that in estuaries	565
Model experiments for the revival of Matla—peali system	565
Measures adopted to reduce turbulence in the Hooghly river downstream of the knuckle at the Titaghar Jute Mill (Bengal)	569
Protecting the foreshore of the Anglo-India Jute Mill on the Hooghly river (Bengal)	575
Discussion by the Research Committee	577
Discussion by the Board	582
3C—Reservoirs	
Preliminary Note	584
(i) Sedimentation of reservoirs	584
Observations of suspended load at Dhukwan	584
Relation between rate of silting and rainfall	585
Relation between rate of silting and catchment area	586
Silting of Reservoirs —by B. P. Saxena	588
Discussion by the Research Committee	594
(ii) Capacity Surveys	597
Discussion by the Board	597
4C—Dams and Weirs	
Preliminary Note	599
(i) Investigation	601
(ii) Design	601
Stress analysis by method of least work	601
Experimental stress analysis—Photo-elastic methods	602
Variation of co-efficient of discharge with reservoir levels for full opening of sluices in Krishnarajasagar dam	603
Discussion by the Research Committee	603
(iii) Materials	610
Foundation rock of the Ramapadasagar dam	610
A note on <i>Surkhi</i> mortar—by M. C. Sampathiengar	611
Sweating from the Krishnarajasagar Dam	620
Discussion by the Research Committee	620
(iv) Construction	625
Consideration by the Board	625
5C—Dam Appurtenances	626
(i) Fish Passes	626
Preliminary Note	626
Problem of fish cultivation in irrigation and Hydro-electric Reservoirs—by Sunder Lal Hora	628
Discussion by the Research Committee	632
Consideration by the Board	632

	Page
6C—Spillways	
Preliminary Note	634
Scour downstream of a five-vented spillway dam for various combinations of shutter openings	637
Pressure distribution on Ogee spillways—Tungabhadra spillway dam	639
Pressure distribution on the Ramapadasagar spillway dam	640
Co-efficient of discharge for the Ramapadasagar spillway dam	642
Stilling basins for overflow dams	648
Roller bucket for the Ramapadasagar overflow dam	650
Siphon spillways for Pehuj Weir	657
Preventing scour below Ganesh Iyer volute siphon spillways by mutual impact of jets issuing from adjacent siphon outlets	665
Experiments on the effect of altering the shape and number of volutes in a volute siphon	667
Experiments on dissipating the energy of flow from volute siphons	669
Experiments on variation of co-efficient of discharge under varying degrees of sub-mersion of siphon outlets	671
Experiments on shaft spillways for tanks	672
Model investigations regarding the design of N gwa dam	676
Rasul Hydel Scheme spillway siphon	681
Bhakra dam spillways	682
Discussion by the Research Committee	682
Discussion by the Board	699
7C—Hydraulic Structures on Permeable foundations	
Preliminary Note	701
Pressures on Lloyd Barrage floor	702
The Poondi Regulator	706
Discussion by the Research Committee	706
Discussion by the Board	707
8C - Canal Falls	
Preliminary Note	709
Experiments on one span (part model) of siphon in connection with proposed additions and alterations to the existing Tando Mastikhan falls at R. D. 119,117 of Robri Canal.. .. .	710
Canal drop with counteracting jets for the Neivasal Tenpathi Channel	712
To find a device for controlling side erosion at foot bridge at mile 8—4—232 Marikan branch—Mirzapur Canals	712
Rasul bydel Scheme falls	713
Fall at R. D. 129, 500 Pakpattan Canal	713
Design of canal falls	713
Discussion by the Research Committee	714
Consideration by the Board	717
9C—Distributary heads	
Preliminary Note	718
Basic experiments to investigate the effect on silt distribution of (a) angle of off-take and (b) varying discharges	719
To find a suitable device for eliminating bed scour below Ghaggar Canal head	725
Basic experiments for determining co efficiencies for radial gates.. .. .	728
Model experiments in connection with scour below cross regulator over Sind canal at Rustom.. .. .	733
Model experiments in connection with silting of Saidkhan distributary ex-Hyderabad branch	736
Model experiments in connection with silting of D'ore and Patol distributaries ex-Jamrao Canal	741

(xiv)

	Page
Model experiments in connection with silting of Jamarao Canal.. ..	746
Discussion by the Research Committee	751
Consideration by the Board	753
10C—Excluders and Ejectors	
Preliminary Note	754
Model experiments for exclusion of silt from Damodar Canal	755
Analysis of observations on silt exclusion at the headworks of Damodar Canal at Rhondia	760
Silt exclusion works at Lloyd Barrage at Sukkur—by T. R. Sethna ..	764
Design of lower Sind Barrage at Kotri for effective Sand exclusion from canals	770
Optimum shape of the nose of Keti Island at Kotri for the Lower Sind Barrage	764
Model experiments on the exclusion of sand from a feeder Channel taking off from Pennar river	790
Sediment investigation—river Sutlej at Hussainiwala headworks ..	792
Sediment investigation—river Jumna at Tajewala headworks ..	792
Sediment investigation—river Ravi at Dhanna Weir	792
Discussion by the Research Committee	793
Discussion by the Board	797
11C—Outlets or Sluices	
Preliminary Note	801
Discussion by the Research Committee	801
Consideration by the Board	801
12C—Other Works	
Preliminary Note	802
Fluming the Tungabhadra low level canal between miles 27/4 and 28/4 ..	802
Reduction of afflux at sluices, culverts and inverted siphons	813
Siphoning of the Cumbum tank sluice	815
Discussion by the Research Committee	816
Consideration by the Board	817
13C—Staunching of canals and Embankments	
Preliminary Note	818
(i) Materials	819
Staunching of Canals (the use of Usar Soil)	819
Experiments on staunching of canals with <i>Surkhi</i> mortar	820
Discussion by the Research Committee	821
(ii) Economics	822
Consideration by the Board	822
14C—Grouting	
Preliminary Note	823
Consideration by the Board	823
15C—Tubewells	
Preliminary Note	824
(i) Types	825
Radial tubewells	825
Discussion by the Research Committee	826
(ii) Design	829
(iii) Construction	829
Tubewell borings with 18 inches un-reinforced cement concrete pipes—by Rameshwar Saran	829
Discussion by the Research Committee	838
Discussion by the Board	839
List of publications issued by the Central Board of Irrigation	(i)

SECTION C—HYDRAULIC WORKS

M51CBI

447-448

1C—Limitations of Model Experiments and Comparison of Models and Prototypes

PRELIMINARY NOTE

This subject has been on the agenda of the Board since 1939, with the object of comparing the results obtained from works built or remodelled in accordance with indications obtained from model experiments in order to assess the value and limitations of the use of models in solving problems connected with irrigation and river training works. It is particularly desired to make a study of such cases where the results on the prototype are different from those on the model with a view to developing the technique of experimentation.

At the 1945 Board Meeting the following resolution was passed :—

“ Resolved that a guide to the construction and operation of hydraulic models would be of value to all Research Stations, that this guide be prepared in the office of the Central Board of Irrigation, based on contributions by research officers. To this end, it may be necessary to place a research officer on special duty in the office of the Board. ”

In response to this resolution, the Indian Waterways Experiment Station contributed a comprehensive note on “ Methods followed in fixing model scale ratios and over-coming model limitations, based on experience gained at the Indian Waterways Experiment Station, Poona.⁽¹⁾ ”

The Engineering Research Laboratories, Hyderabad have forwarded two pamphlets as a contribution on the subject. Directors of the Sind and the Punjab Research Organisations have sent in advance copies of their contributions which have yet to be approved by their respective Chief Engineers Bombay, United Provinces, Ceylon and Madras have no contribution to make. Replies from Bengal and Mysore are still awaited.

The note on “ The Development of Electrical Analogy Methods in India ” promised by Dr. V. I. Vaidhianathan is still awaited.

The following items were discussed at the 1946 Research Committee meeting :—

(1) Measures taken to control erosion of the left bank of the Rupnarain upstream of Bengal Nagpur Railway Bridge at Kolaghat by means of a deflecting groyne.

(2) Behaviour of the river Sutlej at Suleimanki headworks, Sutlej Valley Project, in the presence of a pitched island and its comparison with model results.

(3) Behaviour of the river Indus at Kalabagh headworks, Thal Project, during the river diversion and its comparison with the model results

(¹) Indian Waterways Experiment Station Poona, Annual Report Technical 1945, page 119—158.

(4) Methods followed in fixing model scale ratios and in overcoming model limitations based on experience gained at the Indian Waterways, Experiment Station, Poona.

Research Officers met in an informal conference on July 1946, and discussed the various problems in connection with hydraulic model experimentation. The general consensus of opinion was that it would be desirable to lay down certain broad principles dealing with such model experiments. These were therefore, discussed and laid down⁽²⁾.

The Research Committee approved of the principles and the Board accepted them.

Recent Literature.

1. Blaisdell, F. W. and Muff, A. N. : Report on tests made on three types of flume entrance—U. S. Deptt. of Agriculture Soil Conservation Service August 1944.

2. Krumbein, W. C. : Shore Currents and sand movement on a model beach—War Department Corps of Engineer, U. S. Army Washington, D.C. Technical Memo. No. 7, September, 1944.

3. Inglis, C. C. : Training works constructed in the Rupnarain river in Bengal—after model experiments—to prevent further bank erosion endangering the Bengal Nagpur Railway line linking Calcutta with Bombay and Madras, 1946.—Institute of Civil Engineers London, 1946

4. Harris, C. W. : Hydraulic Models—Bulletin University of Washington Engineering, Experiment Station Series, Bulletin No. 112 June 1944.

5. Chatley, H. : The use and validity of models in harbour and estuary investigations—Dock and Harb Auth. Vol. 27 N. 311, September 1946).

6. Peckover, F. L. : Design and construction of an Hydraulic model of the Fraser river at New Westminster, B. C.—National Research Council of Canada Division of Mechanical Engineering, Hydraulic Section, Report, No. MH 6 Copy No. 9 Ottawa, August 6, 1946.

7. Tingley, S. F. : The application of stress models to specific structural problems—Proceedings of the Society for Experimental Stress analysis, U. S. A. 1945.

The Year's Work

The following items were discussed at the 1947 Research Committee meeting :—

(1) Model limitations as regards bank erosion and accretion upstream and downstream of a tree groyne.

(2) Model experiments in connection with scour hole below head regulator over Rohri Canal.

⁽²⁾ Central Board of Irrigation, Publication No. 38.

(1) MODEL LIMITATIONS AS REGARDS BANK EROSION AND ACCRETION UPSTREAM AND DOWNSTREAM OF A TREE GROUYNE (3)

A striking example in which a model could not reproduce bank erosion was found in the case of the model of the river Sutlej at Samasata(4). In this case, in order to accelerate the development of a cut-off an "attracting" impermeable groyne, facing 60° downstream, was tested. It was expected that this groyne would attract the flow, thus tending to increase the bank erosion upstream and thus inducing more river curvature and reducing the distance of the 'neck'—conditions favourable for a cut-off. In the model, the banks consisted of consolidated, incohesive sand. Velocities upstream of this groyne and curvature of flow increased; and bank erosion occurred very rapidly to start with. After a short time, however, the bank assumed a flatter slope of 2 : 1—the natural angle of repose of the material—there by reducing the velocities and bank erosion became imperceptible.

Actually, in the river it is expected that banks would continue to erode rapidly—continuing to maintain almost vertical sides—and at the same time, bed material will be thrown on the opposite convex side, and the river would shift towards the right concave side. This could not, naturally, occur in the model.

Lower down, in the same model, banks consisted of stiff material (clay) but as the velocities in the model were naturally not high enough to cause scour of such material, only high velocities persisted along the bank and no appreciable recession of bank occurred, as would in the river.

An experiment was also done in which the banks were built up of alternate layers of sand and clay; as in the river, such formation is often found on outside of curves. Actually, however, it was found that some undermining of the stiff material of clay layers occurred in the model due to washing away of the sand layer, but this action in the model was comparatively, very limited, due to sand layer assuming the natural angle of repose (2 : 1) : whereas in the river the undermining action is, generally, very severe and rapid during the floods, especially in the falling stage, great chunks of clay falling into the bed of the river in quick succession as soon as the sand underneath has been washed away.

Another obvious limitation in the model is the length of the model with an imposed entry length. The model would, indeed, have to be very long to simulate correct meander movements of the Sutlej river. This, on top of the above limitation of bank erosion, makes movements many years ahead impossible to predict directly from model results.

In Sind, tree groynes have been found very successful :—

- (a) to divert or deflect the current which is directly threatening an erosion of the *bund* ;
- (b) to deaden the flow in one bye-river near the *bund* in order to open out another bye-river away from the *bund* ; or
- (c) to silt up a channel of the river at its source by checking the flow in it.

(3) Indian Waterways Experiment Station, Poona, Annual Report (Technical) 1946, pages 5 & 6

(4) See page 498.

In the case of the Sutlej model referred to above, a permeable tree groyne was tested to check the erosion of the left bank upstream. This groyne consisted of small branches of *Babul* trees (*acacia arabica*). From Sind experience it is known that when the river first rises during freshets, it brings in a large quantity of sand moving along the bed and silt in suspension; this is trapped in the branches and the twigs of the trees and hence upstream and downstream of the groyne the river bed rapidly accretes, and the permeable groyne in a very short time approximates to an impermeable groyne and thus deflects the high velocity flow. In the model, however, there was very little silt in suspension and whatever amount of sand was moving along the bed was not enough to cause this accretion; hence the channel continued to flow through the groyne and, comparatively, little throw-off occurred. Any quantitative conclusions from the model itself, without regard to the behaviour of a tree groyne from actual field experience, would, therefore, be misleading.

(2) MODEL EXPERIMENTS IN CONNECTION WITH SCOUR HOLE BELOW HEAD REGULATOR OVER ROHRI CANAL (5)

The modifications which were suggested⁽⁶⁾ did not give satisfactory results on the prototype. The scour downstream of the regulator has not shown any sign of silting. It was felt that the deflector as provided actually brought about increased velocity at the exit end, owing to restriction in the Waterway and a *pacca* platform immediately downstream of the deflector preventing any roller action in building the earthen slope below the deflector. It was, therefore decided to depress the pavement and remove the deflector. As it has now been found that the flow has worsened, further experiments are in progress to find a solution to the problem.

DISCUSSION BY THE RESEARCH COMMITTEE

MR. K. K. FRAMJI, introduced item (1) and said that it was useful to recount for future guidance three striking model limitations observed in the model of the river Sutlej at Samasata.

(1) A model could not show progressive bank erosion such as occurred in a river in nature on the concave side of bends and the extent of bank erosion could not therefore, be directly assessed from model experiments. When the banks consisted of consolidated incohesive sand, to start with, bank erosion occurred very rapidly with almost vertical sides as in nature, but after a short time the bank assumed a flat slope 2 : 1—the natural angle of repose of the material—slowing down the velocities and the bank erosion became almost imperceptible.

When banks of stiff material (clay) were reproduced in the model, the velocities in the model were not high enough to cause scour of such material and no appreciable recession of bank occurred as would in the river: it was by the high velocities persisting along the bank that bank erosion must be estimated.

(5) Report of the work done in the Research Division, Sind, for the year ending December 1946, pages 1 and 2.

(6) Report of the work done in the Research Division, Sind, for the year ending December 1945, pages 1 and 9.

With banks built up of alternate layers of sand and clay, such as often found on the outside of curves in nature, it was found that some undermining occurred of the stiff material of clay layers by the washing out of the sand layer, but this action in a model was comparatively, very limited due to the sand layer assuming the natural angle of repose (2 : 1).

(2) Another obvious limitation observed was of the length of the model *vis-a-vis*, the imposed entry length. The model would, indeed, have to be very long to simulate correct meander movements. Thus, such models could not be expected to give direct predictions of results many years ahead.

(3) In nature, it was known, when a river commenced to rise during freshets it brought in a large quantity of sand moving along the bed as well as in suspension; a tree groyne halted and trapped the rolling sand and the suspended silt; and the permeable groyne in a very short time became thickly staunched and approximated in its action to an impermeable spur. In the model, however, there was very little silt in suspension and whatever amount of sand was moving along the bed was not enough to simulate the staunching occurring in the river with the result that the channel continued to flow through the groyne with relatively less check and, therefore, with comparatively less throw-off.

THE PRESIDENT asked Mr. Framji, if there was no way of simulating suspended silt in a model.

MR. FRAMJI replied that simulation in models was done by injecting silt. But this silt also passed through the permeable groynes in the model. The suspended silt is so fine that it would pass through any permeable obstruction in the model.

DR. N. K. BOSE asked Mr. Framji, if he had any instance to show where the model did not agree with the prototype. Mr. Framji's observation, he felt might be merely hypothetical. He also enquired how this paper could come under this subject. As he understood, only those results of model experiments that were reproduced in the prototype could be discussed under this heading.

MR. T. R. SETHNA introduced item (2) and explained the modifications carried out in the December closure of 1946 and the reasons for these modifications not being successful on the prototype though they looked promising on the model. He added that further modifications were carried out in December 1946 closure, on the prototype, without model experiments. The floor was depressed at the exit end to reduce exit velocity but the flow had worsened. Model experiments which were now in progress showed that a depressed floor with a deflector of $0.1 D$ height was promising. This would be recommended for adoption.

DR. BOSE enquired from Mr. Sethna whether the four feet sill improved conditions of flow.

MR. SETHNA replied that sufficient length was not available on the downstream side. The distance being short, every thing seemed well on the model, but actually it was not so. Arrangements had now been made to distribute the discharge equally in the width.

MR. H. P. MATHRANI explained that there was some defect in the head regulator. An 'S' curve had been put in to improve conditions of flow in the centre and experiments had been carried out with uneven openings of the regulator. The 'S' curve with baffle teeth improved the conditions a little. The regulator is safe but the flow is still uneven.

DEWAN BAHADUR GOVINDARAJA AYYANGAR asked Mr. H. P. Mathrani to state the effect of the work on the canal itself.

MR. H. P. MATHRANI replied that at the bridge the canal was 200 feet wide and a huge ambayment had formed below the bridge. The safety of the bridge had been endangered. After making the changes the side scour had decreased.

DEWAN BAHADUR GOVINDARAJA AYYANGAR asked if the full supply level was still four feet below designed.

MR. H. P. MATHRANI replied that there was no irrigation above the bridge. The working depth was 12 feet as designed but the water level was four feet lower than designed.

THE PRESIDENT remarked that in other words the slope of the canal had flattened.

MR. H. P. MATHRANI said that the slope was designed as 1 in 12,000 but it was now 1 in 17,000.

DEWAN BAHADUR GOVINDARAJA AYYANGAR asked what the soil was like.

MR. H. P. MATHRANI replied that the soil was very fine.

MR. T. P. KUTTIAMMU asked what the velocity was.

MR. H. P. MATHRANI replied that the velocity was about 3.5 feet per second and agreed that it was a bit too high for the particular soil.

THE PRESIDENT remarked that in any case the trial in the laboratory had not yet been reproduced on the prototype. It was not known how the results would compare.

MR. T. R. SETHNA corroborated the President's remarks.

MR. H. P. MATHRANI said that all that could be said at the moment was that there was some improvement.

MR. FRAMJI referring to item (2) said that the so-called deflector was obviously unsatisfactorily designed with a glacis upstream of it instead of a flat pavement, its relative height far too much, and the arrangement as a whole causing greater turbulence in already difficult conditions because of the high velocities due to the low depth. Mr. Framji asked if this deflector was obtained from experiments carried out with the designed canal depth, downstream, of 12 feet of the more severe *actual* conditions of eight feet obtaining due to retrogression of water levels. It was not known whether correct entry conditions upstream of the regulator to reproduce cross flow and correct exit conditions with bellies and retrograded bed were reproduced in the model with a mobile bed, as were absolutely necessary in such a case as this to the same extent as for devising measures for excluding entry of excess sand. In fact, it appeared essential to reproduce a part of the upstream river course as well as the drawoff of the other head regulator.

MR. H. P. MATHRANI replied that retrogression was allowed for.

MR. T. R. SETHNA said that cross currents had also been represented on the model.

DR. N. K. BOSE remarked that the measuring weir was much too near the crest and, therefore, the heading up affected the observations. He wanted to know the width of the weir and the canal.

MR. SETHNA said that space limitation had narrowed down the floor. In the recent experiments needles had been put in. The weir was three feet wide and the canal eight feet.

DR. S. P. RAJU wanted to know the slope of the apron.

THE PRESIDENT asked Mr. Sethna what was the improvement effected in the head regulator.

MR. SETHNA replied that the roller that was being formed was shortened and the intensity of scour was less.

MR. H. P. MATHRANI said that the water hugged the sides more.

THE PRESIDENT enquired whether the trouble had shifted from the bed on to the sides.

MR. H. P. MATHRANI replied in the negative and said that the trouble at the sides was much less.

THE PRESIDENT enquired from Mr. Mathrani as to what the overall result was.

MR. H. P. MATHRANI said that the trouble had been reduced.

MR. SETHNA said that this was a different case.

MR. R. S. CHATURVEDI said that he differed from Mr. Framji and that with a suitable material for the model he had been able to reproduce side scour on a two years hydrographs.

THE PRESIDENT asked Mr. Chaturvedi if he would be able to reproduce the mix.

MR. CHATURVEDI replied that he had on record that an embayment was reproduced quantitatively and referred to his previous year's report on the Deoha model.

THE PRESIDENT remarked that Mr. Chaturvedi had not been able to verify the results on the prototype as yet.

In reply to Mr. Framji's question Mr. SETHNA said that all the four canals at the site had not been reproduced for the entry conditions.

DR. RAJU said that short entry conditions and short terms results referred to by Mr. Framji were not understood.

MR. FRAMJI replying to the discussion referred Dr. Bose to pages 500 to 506 of this report where three actual cases of tree groynes constructed in Sind and what happened to them had been given. The case was not merely hypothetical as pointed out by Dr. Bose. Information regarding model technique and limitations was as important as information relating to comparison of behaviour of model and prototype.

Replying to Dr. Raju, MR. FRAMJI said that it was not correct for anyone to try to predict from a river model with a year or two's course and hydrograph what would happen after 10-15 years. That was what he meant by short term results. If long term results were required, then data for a long period had to be serially reproduced on the model and not the data for one or two seasons.

MR. H. P. MATHRANI asked Mr. Framji if better results would be obtained by having impermeable groynes in the model.

MR. FRAMJI replied that if the conditions on the river model were required to be correctly simulated, allowance had to be made for the fact that the groynes were permeable.

DR. BOSE said that if enough suspended silt was not introduced in the model the results obtained would be entirely different. There should be as much of suspended silt as possible. If a long range view of the whole thing was required the model must be a long one. Generally, Dr. Bose said, he took twice the meander length which in turn depended on the discharge. DR. BOSE emphasized that the entry condition was very important.

MR. CHATURVEDI asked Mr. Framji how were the trees and leaves reproduced on the model.

DR. FRAMJI explained how it was possible to reproduce to scale the trees and leaves but silt size could only be reduced within practical limits.

DR. VAIDHIANATHAN said that the silt could be reproduced on the model.

RAO BAHADUR JOGLEKAR said that they had gone down to 0.05 mm. He asked what was the limit.

DR. VAIDHIANATHAN said that by precipitation one could get as fine particles as one liked and there was no difficulty.

MR. FRAMJI said that it was not a practical proposition in the size of models in use.

DR. VAIDHIANATHAN said that if cost were not the consideration it could be done.

THE PRESIDENT asked Dr. Vaidhianathan if he could give a rough estimate of the cost.

DR. BOSE said that the precipitation method had been used by Gibson in Manchester and he had obtained accurate results on the Severn Model. Probably the Hooghly's muddy water could be used.

THE PRESIDENT asked Mr. Framji how he had reproduce the *babul* trees.

MR. FRAMJI said that the rough dimension of *babul* trees were actually known. *Babul* leaves and twigs were actually made upto scale so as to represent the trees as faithfully as possible.

THE PRESIDENT asked Mr. Framji if he made the thing really dense.

MR. FRAMJI replied that this was exactly the point that he had tried to make. There was an essential difference between prototype flow and the model flow. In the model in spite of the groynes being made as dense as is practicable the flow passes through the trees and the silt was not trapped to nearly the same extent as it does in nature.

THE PRESIDENT remarked that in other words it had not yet been possible to reproduce the trees correctly.

DEWAN BAHADUR GOVINDARAJA AYYANGAR asked how extensive the use of tree groynes was in practice.

MR. FRAMJI replied that though the use was fairly wide and increasing, the potentialities were even larger than the actual use. The impermeable spur had several defects. One great advantage of a tree groyne was that it did not cause scour at its nose.

MR. GOVINDA RAO suggested that the leaves might be repelling the clay particles.

DR. BOSE in reply to Dr. Raju said that Professor Gibson had used alum to increase sedimentation.

DR. VAIDHIANATHAN said that Mr. Govinda Rao was right, the leaves had electric charge and hence the clay particles were repelled. If Mr. Framji introduced an electric field in the running model he may be able to get better results.

MR. FRAMJI replied that this might add to the complications by undue precipitation.

THE PRESIDENT said that the results were yet inconclusive and that the subject be kept on the agenda.

CONSIDERATION BY THE BOARD

THE SECRETARY explained that this was a new name for the subject which was previously called "Field results of measures adopted after model experiments" and described the work done at the Research Committee meeting:—

Regarding progress on the Board publication on the subject, the Secretary drew attention of the members to the preliminary note on page 449.

It was decided that the subject should continue on the agenda.

2C—River Control

PRELIMINARY NOTE*

This subject first appeared on the agenda of the Research Committee 1942 under the head "River Behaviour, Training and Control" though the subject "Meandering of Rivers" had been under discussion for many years past. The subject is to be considered under the following sub-heads:—

- (i) Meandering
- (ii) Scour
- (iii) Spill
- (iv) Tidal action including that in estuaries
- (v) Flood Control †
- (vi) Local training
- (vii) Stage discharge relation †

(i) FLOOD PROTECTION WORKS*

This sub-head appeared on the agenda for the first time in 1945. Certain aspects of this subject were discussed previously in different forms *e.g.* in 1939-40 as "The collection and study of statistical data pertaining to river flood control".

The following items were discussed at the 1946 Research Committee Meeting:—

- (1) Experiments with a Model of the Mahanadi and its branches at Cuttack Orissa.
- (2) Cost of damage caused by floods in the Narbada and Tapti rivers.

A sub-committee was appointed by the Board in 1946, to draft a resolution on the subject calling upon all provinces and states to collect data regarding floods and the manner in which it should be collected.

Recent Literature

1. Mayott, C.W. : Report on Flood Control, Navigation, and Hydro-Electric Developments on the Watersheds of the Connecticut, Thames and Housatonic Rivers—The Hartford Electric Light Co., Hartford, Connecticut, September, 15, 1945.
2. Elgar W. H. The Seasonal and periodic occurrence of maximum flows in water courses in Great Britain.
3. Ziemer, G. L. :—Conservation factors in flood control plannings—Agricultural Engineering Vol. 26, No. 9, September 1945.
4. Laverty, F. B. :—Correlating flood control and water supply, Los Angeles Coastal Plain—Proceedings of the American Society of Civil Engineers, Vol. 71 No. 6, June 1945.

*This preliminary note was written for the Research Committee Meeting. At that meeting the subject head was divided into the following subheads (i) Flood protection works (ii) River training works, (iii) Scour and erosion, (iv) Spill, (v) Meandering, and (vi) Tidal action including that in estuaries.

† The sub-heads "Floods" and "Stage-discharge relations" have been included under Section A—Hydrology item (2) River behaviour and the remaining sub-heads have been arranged according to the new arrangement of grouping of subjects under discussion recommended by the Research Committee.

5. Guide for Public Works Department and district floods officers for flood protection measures during the flood season—Public Works Department, Cuttack.

6. Schuleen. E.P. :—Control of floods at Pittsburgh planned—Civil Engineering, Vol. 15, No. 10, October 1945.

7. Rangaiya, M.G. :—River flood control Current Science, Vol. 14, No. 11, November 1945.

8. McMillin, M. :—Flood Control in San Bernardino Valley—Excavating Engineer, Vol. 40, No. 3, March 1946.

9. Ockenden, G. H.—Flood Control—The Surveyor, Vol. 104, No. 2775, March 30, 1945.

(ii) RIVER TRAINING WORKS

“ Training ” was the original subject head but it was later divided into

- (a) falling aprons,
- (b) the heads of guide-banks, spurs *etc.*, and
- (c) the results of measures adopted in training rivers.

In 1942, the Board passed the following resolution :

* * * * *

• “ Resolved further that Mr. Inglis be requested to write a note on ‘ meandering ’ with the data already collected by him and that the Secretary to collect data from the provinces and the railways on ‘ Scour ’ and ‘ Training ’ and pass them on to Mr. Inglis for analysis.”

Accordingly a form was issued for the collection of data of launched aprons; which data it was intended to analyse and compare with the assumptions made by Spring ⁽¹⁾ and observations made on models. Most of the data available have been received except that from Sind. The data have been passed on to the Punjab Irrigation Research Institute for examination and presenting it to the Research Committee.

As regards the collection of other data for the proposed Board Publication on River training, the United Provinces, Bengal, and Madras have sent in their contributions.

The Railway Board also have supplied five notes on the subject and two more notes are expected.

Central Provinces and Berar, Orissa, Punjab, Ceylon, Bahawalpur, and Mysore have nothing to contribute. Replies from the rest are awaited.

The following items were discussed at the 1946 Research Committee Meeting :—

- (1) Training of Luni river to prevent it damaging Jodhpur Railway Line.
- (2) Training of river Jumna at Delhi Gate Pumping station with pitched embankment on the right bank .
- (3) Training of river Deoha at Duni Barrage.
- (4) Training of river Adjai at E.I. Railway bridge.
- (5) Proposed remodelling of the Anderson Weir (Part A) .

(1) Spring, F.—“River training and Control on the guide bank system ” Railway Board, Technical paper No. 153, Simla, 1903.

- (6) Relieving the strain on the Bherian spurs on the river Ravi, by a pitched island.
- (7) Protecting the left bank of river Sutlej near Samasata Railway Station.
- (8) A note on River Behaviour and Control by Moti Ram, Esq., I.S.E.
- (9) Training of tidal creeks which have only small local area to drain by D. N. Sen Gupta, Esq., I.S.E.

Recent Literature.

1. Training the Yellow River.—The Engineering, Volume CLXXXII, No. 4739, November 8, 1946.
2. River Control projects in the United States—Indian and Eastern Engineer, Vol. 96, No. 6, June 1945.
3. Rao, T. S. N.—History of the Hardinge bridge across the Lower Ganga (up to 1941)—Journal of the Institution of Engineers, India, Vol. 26, No. 1, September 1945.
4. Niyogi, K. C.—River training in U.S.A.—Science and Culture, Vol. II, No. 8, January and February 1946.
5. Senour, C.—New project for stabilising and deepening lower Mississippi River—Proceedings of the American Society of Engineers, Vol. 72, No. 2, Part I, February 1946.
6. Cozzens, H. F.—Steel rails for bank protection on Salinas River California—Civil Engineering, Vol. 16, No. 3, March 1946.
7. Thomson, O.G.—Principles of well foundations river training and protection works—Proceedings of the Punjab Engineering Congress, Paper No. 278, 1946.
8. Inglis, C.C.—Training works constructed in the Rupnarain river in Bengal after model experiments to prevent further bank erosion endangering the Bangal Nagpur Railway line linking Calcutta with Bombay and Madras—The Institution of Civil Engineers, London.

(iii) SCOUR AND EROSION

The Indian Waterways Experiment Station is collecting data on scour below bridges and it is hoped that the results will be presented to the Research Committee as soon as they are available.

The following items were discussed at the 1946 Research Committee Meeting.

- (1) Experiments in connection with dissipation of energy downstream of Lower Sind Barrage.
- (2) Scour downstream of rigid structures in wide channels.
- (3) Experiments in connection with the proposed barrage at Guhagram.
- (4) Proposed remodelling of the Anderson Weir.
- (5) Causes of accretion of the river bed at Sukkur Barrage and how to reduce rate of rise—Part II.

Recent Literature.

Permeable pile jetties stop river bank erosion—Construction Methods, Vol. 27, No. 9, September 1945.

(iv) SPILL

This sub-head was introduced for the first time in 1943.

There was no contribution under the sub-head last year.

(v) MEANDERING

It is understood that a note on the subject has been included as a chapter in the book on "River Behaviour and Control" written by Sir Claude Inglis which is under publication by the Indian Waterways, Experiment Station.

The following items were discussed at the 1946 Research Committee Meeting:—

- (1) A study of the meanders of the Indus between Sukkur and Kotri.
- (2) Deterioration of the river Kultigong.

The following resolution was approved by the Board at its 1946 annual meeting:—

"Resolved that the dominant formative discharge concept in non-boulder rivers may be examined by the various provinces and research organizations in the following respects—

- (i) The tentative definition of the dominant formative discharge as the discharge which is sufficiently large in magnitude and which is exceeded a sufficient number of times in the flood season so as to have its effect on the formation of the river course and bed ;
- (ii) The estimation of this dominant formative discharge from the mode of the smoothed frequency curve of the daily discharges exceeding the discharge at which bed movement begins in the river ;
- (iii) The estimation of the discharge at which bed movement begins in the river from hydrographs showing the datum areas, that is the cross sectional areas below a given datum ;
- (iv) An approximation to this estimate of the dominant formative discharge from the average of the appropriate flood discharges ;
- (v) The comparison of the dominant formative discharge with the bank-full stage discharge and maximum discharge wherever data regarding bank-full stage discharges are available or can be observed or estimated ;
- (vi) The correlation of the meander characteristics with the dominant formative discharge and maximum discharge of the year".

Mr. Framji supplied a note (2) in explanation and amplification of the resolution for the information of those who were not fully conversant with the terms used in the resolution.

Recent Literature.

A laboratory study of the meandering of alluvial rivers. War Department, Corps of Engineers, U.S. Army, U. S. Waterways Experiment Station, Vicksburg, Mississippi, May 1, 1945.

(vi) TIDAL ACTION INCLUDING THAT IN ESTUARIES

This sub-head was introduced for the first time in 1943.

The following items were discussed at the 1946 Research Committee Meeting:—

- (1) Training of the Hoogly river at Dunbar Cotton Mills with a by-pas channel.
- (2) Experiments on a model of the Peechaboni Sluice.

The Year's Work

(i) FLOOD PROTECTION WORKS

The following items were discussed at the 1947 Research Committee Meeting:—

- (1) Redistribution of sand load between the Mahanadi and Katjuri systems at Cuttack-Orissa.
- (2) Flood absorption in Krishnarajagar.
- (3) An unprecedented flood in Mula and Pravara valleys and estimate of loss of flood damage.
- (4) Meandering rivers and flood control by K. B. Ray.

(1) RE-DISTRIBUTION OF SAND LOAD BETWEEN THE MAHANADI AND KATJURI SYSTEM AT CUTTACK (ORISSA) (3)

ABSTRACT

About 12 miles above Cuttack, the Mahanadi with its peak flood of 15 *laks* cusecs passes through the gorge at Goil Bank to flow through a wide stretch, upstream of the second gorge at Naraj about five miles above Cuttack Figure 2C. 1. Below the Naraj gorge it bifurcates into the Katjuri, the Mahanadi flowing northwards for about a couple of miles and then turning eastwards to flow along the North bank of Cuttack. The Mahanadi further bifurcates into the Birupa river just upstream of the Mahanadi Anicut at Jobra. The Katjuri, after taking off from the Mahanadi at the Naraj weir, is separated from the Mahanadi by a dividing embankment about two miles long. At Naraj, 56 per cent. (8.4 *laks* cusecs at peak flood) of the high river discharge flows down the Katjuri. On its left bank, the Katjuri spills 4.67 per cent. of the flood (70,000 cusecs at peak flood) into the Mahanadi, through the Birnasi gap between the dividing embankment and the Cuttack town whereas on the right bank, it spills 6.67 per cent. (=1.0 *lakh* cusecs at peak) of the flood, which find its way, mainly, back to the Kuakhai river above the Kuakhai Railway Bridge. Downstream of the Banki Road spill, the Katjuri bifurcates into the Kuakhai, Surua and Katjuri loop. A series of qualitative experiments, carried out to find the effect on the distribution of sand load due to different modifications are described.

(3) Indian Waterways Experiment Station, Poona Annual Report (Technical) 1946, pages 44-52.

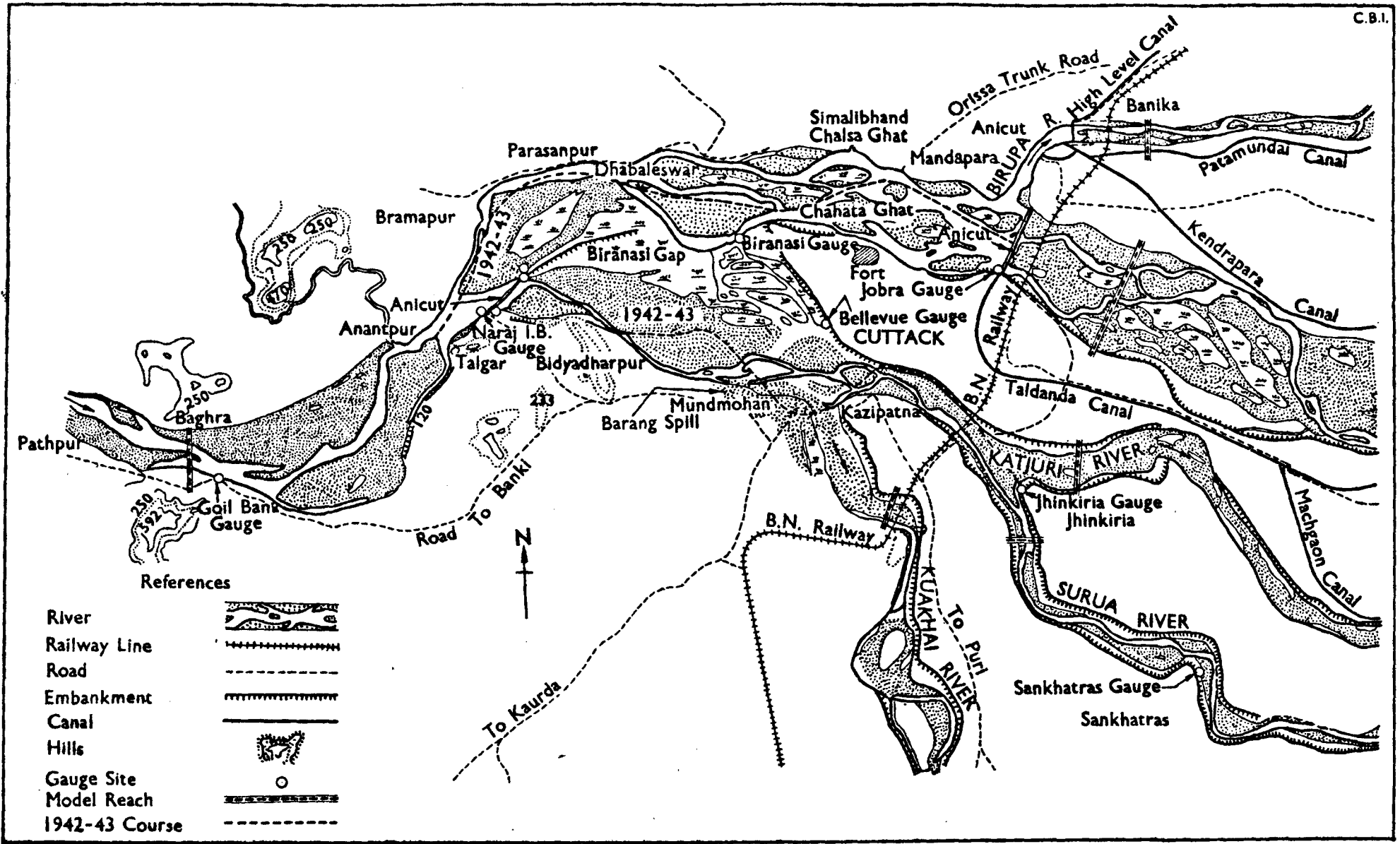


Figure 2 C.1 Showing Mahanadi and its branches at Cuttack(1930)

Scale 2 1 0 2 4 Miles

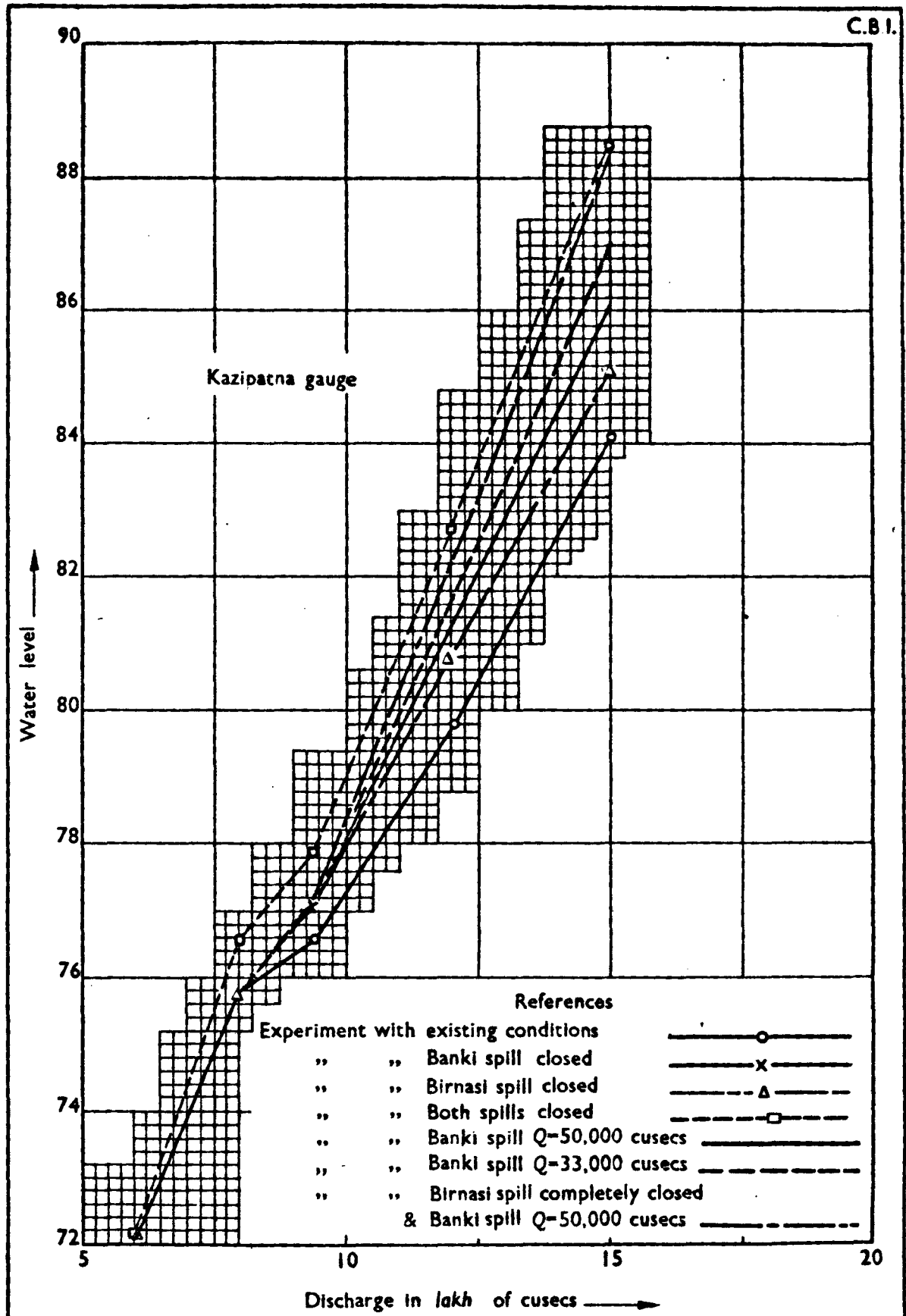


Figure 2C.2:— Showing water levels observed at the Kazipatna gauge at the Kuakhai head in the experiments of series I

THE MODEL

The model reproduced a reach of the river from Goil Bank down to two miles below the Jobra Weir on the North of Cuttack and two miles below the Katjuri Railway Bridge on the South of Cuttack. In this year's experiments, the length of the reach of the Kuakhai river was increased from two miles to five miles below its head. The 5.5 miles long reach, between the gorges at Goil Bank and Naraj, served as an entry length for the model and was found to be adequate.

The scale ratios of the model were :—

Discharge scale ratio	= 1/160,000
Length scale ratio	= 1/400
Vertical scale ratio	= 1/66
V.E. (vertical exaggeration)	= 6.06
B.S.E. (bed Slope exaggeration) = S.E. (Surface slope exaggeration)	= 5

Though from the point of view of ease in operating the model and interpreting water levels *etc.* it is convenient to have V. E. = S. E. = B.S.E., yet as has been observed at this Station, it is very rarely that this condition is obtained and almost all the river models at this Station are in effect "tilted" and it is only by chance, when the bed material available for the models is just right to give the desired slope for the charge injected, that we get V.E. = S.E. = B.S.E. It may be argued that by manipulating charge we can get any desired slope within a range ; but, in that case the width of the channel becomes either too small or too large depending on whether the charge is less or more than the correct one. Moreover, it is only at one stage of designed discharge that one can obtain V.E. = S.E. = B.S.E.

In the present case, the scale ratios noted above were found to give satisfactory conditions.

EXPERIMENTS

The previous experiments ⁽⁴⁾ were mainly done to find out the afflux caused by shutting of the Banki Road and Birnasi spills either singly or both together. It had been, however, observed that shutting of the spills would cause excessive afflux in the Katjuri system, which could be set right by reducing its discharge at the Naraj weir.

It was, however, argued that it may be too drastic a step to shut off the spills completely, because, being high level spills, they carry relatively sand-free water which eventually goes to make the mix in the Mahanadi and Kuakhai leaner than it would otherwise be. Further by reducing the discharge of the Katjuri system, the changed discharge conditions may cause a curvature of flow which may be more adverse to the Kuakhai than at present, thus making it deteriorate rapidly. Moreover, closing Banki Road spill completely was objectionable, because occasional flooding in the Puri District, into which the Kuakhai spills, was considered a desirable feature.

(4) C. B. I. Publication No. 38, Annual Report (Tech.) 1946.

A series of *qualitative* experiments were, therefore, carried out to find the effect on distribution of sand load due to the various modifications.

They were :—

SERIES I

With unreduced length of the Naraj Weir.

- Experiment 1 : With existing conditions.
- Experiment 2 : With Birnasi gap full open and Banki Road spill discharge reduced to 50,000 cusecs.
- Experiment 3 : With Birnasi gap full open and Banki Road spill discharge reduced to 33,000 cusecs.
- Experiment 4 : With Birnasi gap closed by an embankment ending in a spur (*) and Banki spill full open.
- Experiment 5 : Same as Experiment 4, but Banki Spill discharge reduced to 50,000 cusecs.
- Experiment 6 : Same as Experiment 4, but Banki Spill discharge reduced to 33,000 cusecs.
- Experiment 7 : With both Birnasi and Banki spills closed completely.

SERIES II

With length of Naraj Weir reduced so as to cause no afflux in the Katjuri system at peak flood of 15 lakhs cusecs above Naraj.

- Experiment 1 : With Birnasi gap open and Banki Road spill discharge reduced to 50,000 cusecs.
- Experiment 2 : With Birnasi gap open and Banki Road spill discharge reduced to 33,000 cusecs.
- Experiment 3 : With Birnasi gap closed and Banki Road spill discharge reduced to 50,000 cusecs.
- Experiment 4 : With Birnasi gap closed and Banki Road spill discharge reduced to 33,000 cusecs.

PROCEDURE

In each experiment the model bed was moulded according to the survey of 1943 and flood cycles, rising from 6 lakhs cusecs to 15 lakhs cusecs and falling back to six lakhs cusecs again, were run till the quantity of bed sand collected in the sand traps at the downstream end of each branch remained the same for two consecutive cycles. Due to the very short duration of model flood discharges, it was not possible to measure the quantity of sand at each successive stage of the flood. It was, however, considered that reasonably correct results could be obtained by measuring the sand accumulated in the sand pits at the end of the final flood cycles.

DISCUSSION OF RESULTS

(i) Figures 2C. 2, 2C. 3 and 2C. 4 show the water levels observed in the model at the Kazipatna, Collectorate and Bellevue gauges of the Katjuri in the experiments of Series I.

(*) *Ibid.*

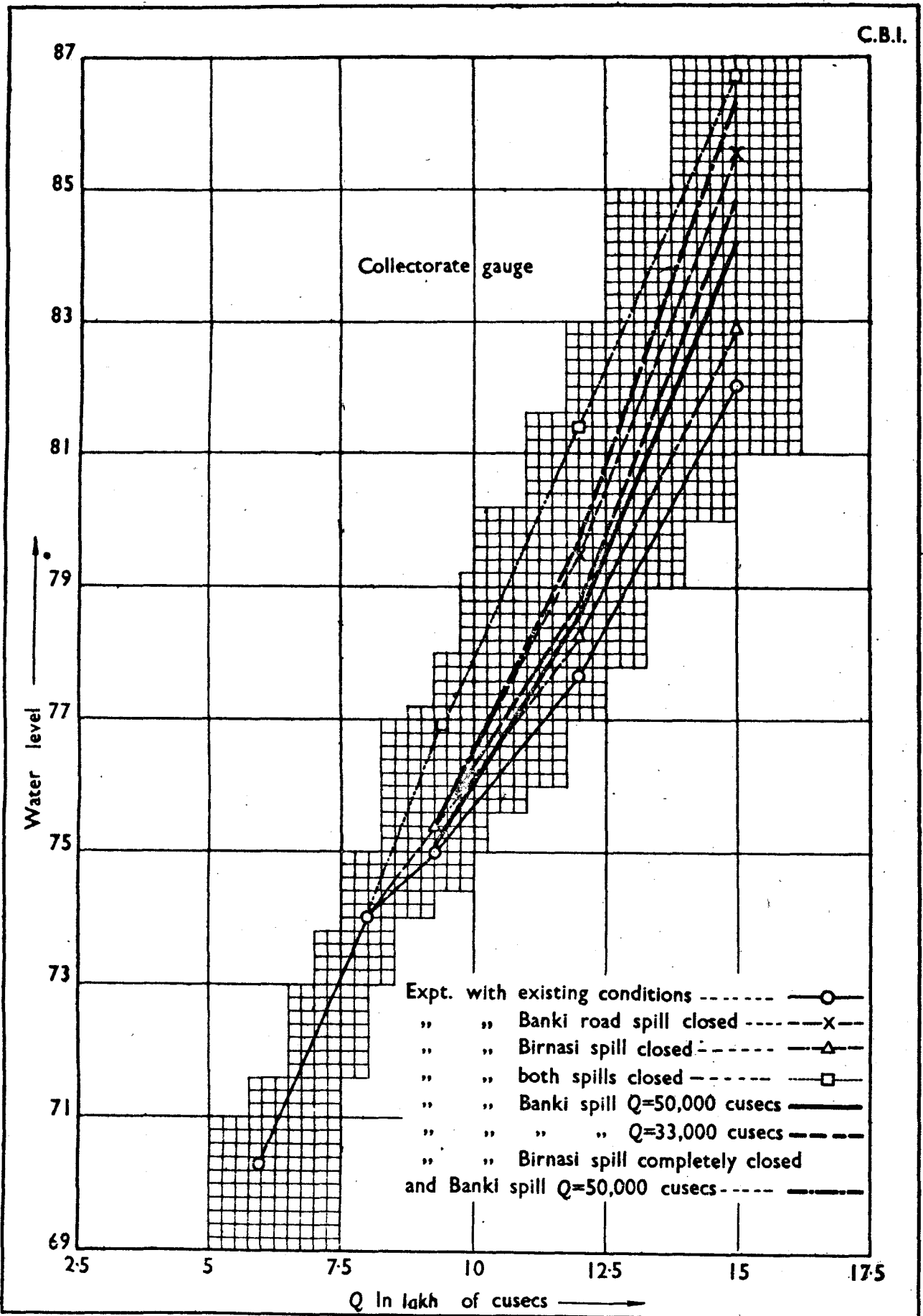


Figure 2 C.3:— Showing water levels observed at the Collectorate gauge on south side of Cuttack, in the experiment of series I

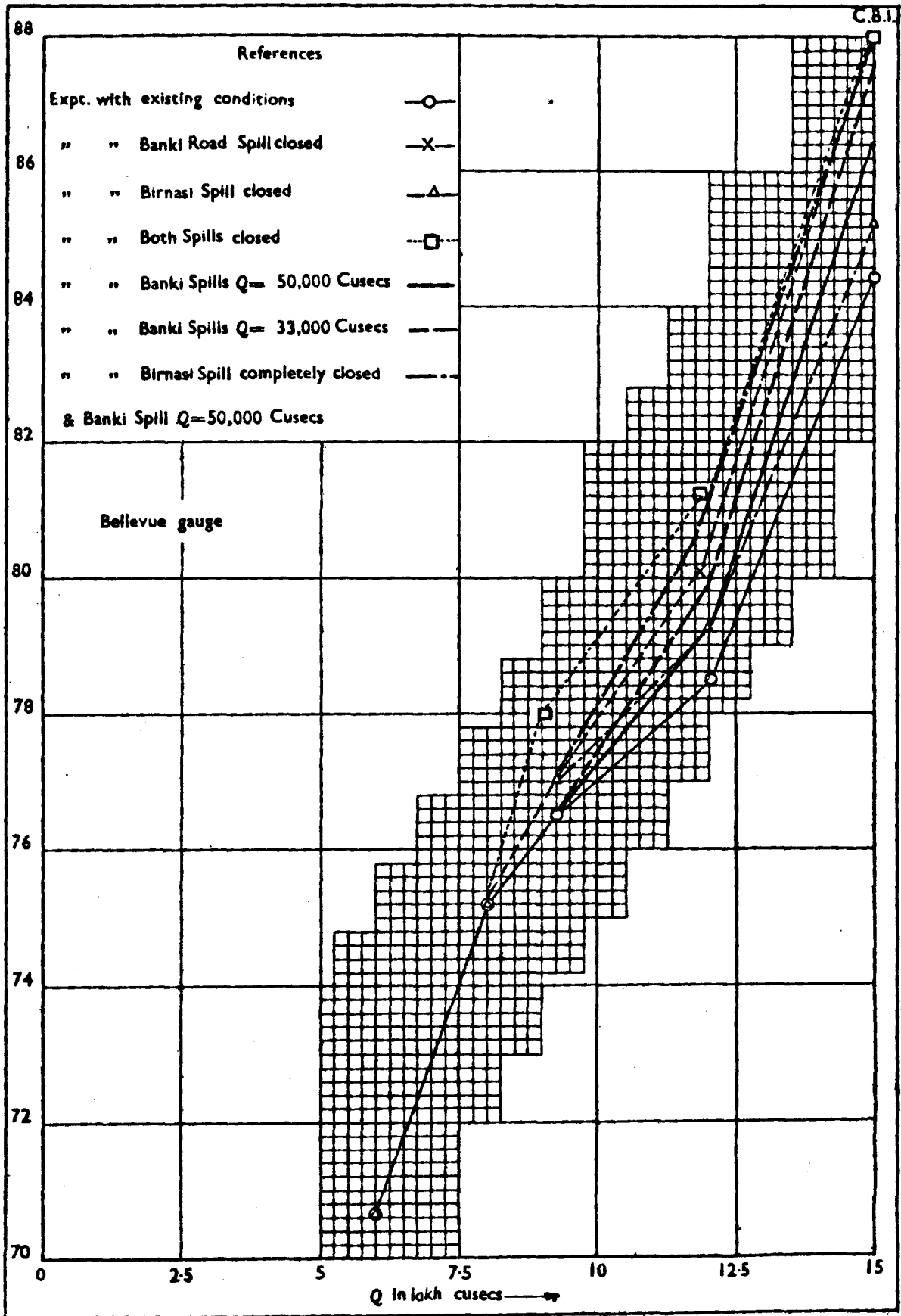
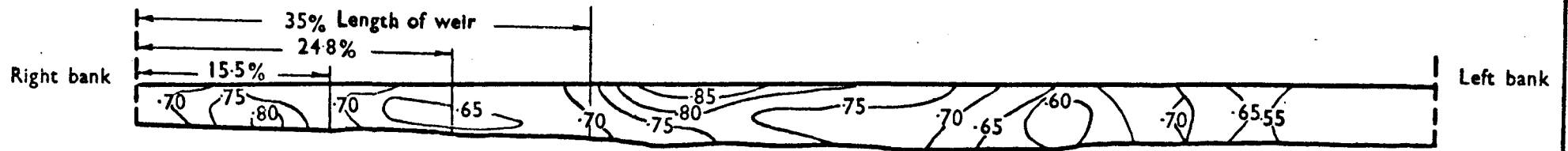
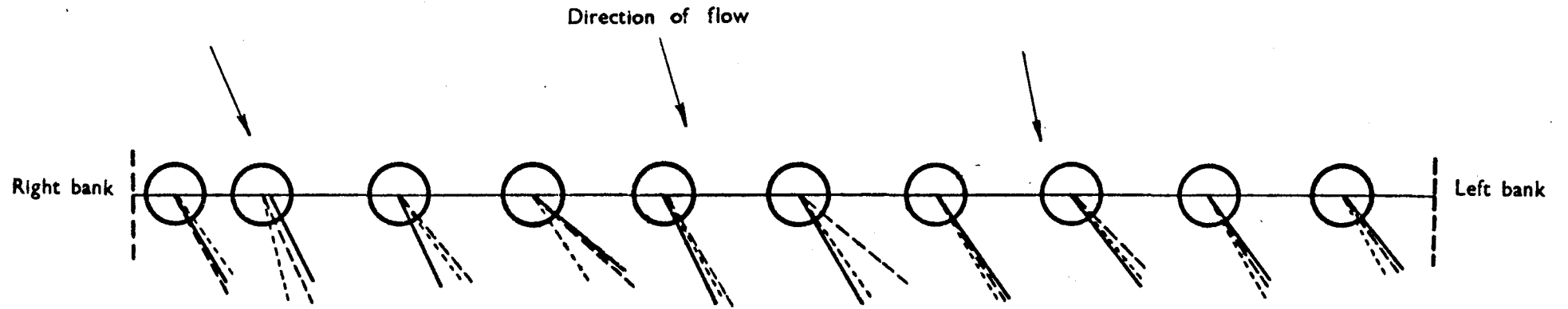


Figure 2C.4:- Showing water levels observed at the Bellevue gauge in the experiments of series I



Isovels showing velocity components at right angles to the Naraj weir

References :-

- Surface velocities
- Mid depth velocities
- Bed velocities

Figure 2C.5 :- Showing the velocity distribution at C.S. \equiv 200' U/S of and parallel to the Naraj weir

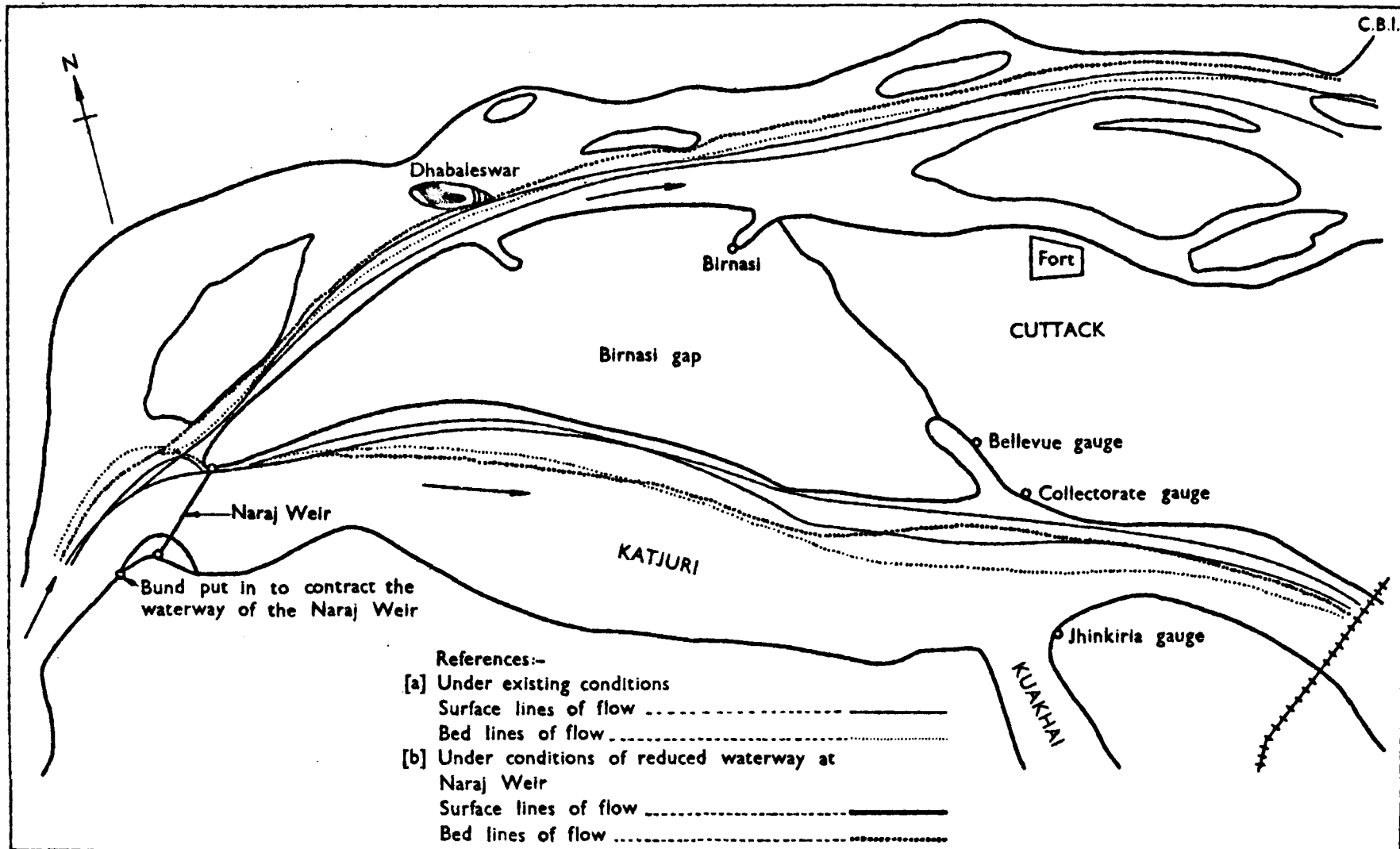


Figure 2 C.6 :-Showing the divide of the surface and bed under existing conditions and with Naraj Weir waterway reduced by 15.5%

Discharge below Naraj = 6 lakh cusecs

It will be seen that with Birnasi spill shut off completely and the Bank Road spill discharge reduced to 50,000 cusecs, the afflux at the Kazipatna gauge was 4.4 feet; whereas that at the Collectorate gauge was 4.2 feet and that at the Bellevue gauge was 3.6 feet. With only the Banki spill discharge reduced to 50,000, the afflux was two feet which increased to three feet when the Banki spill discharge was further reduced to 33,000 cusecs.

Considering the present conditions with the peak flood levels, this high order of afflux would be undesirable.

In Series II, when the waterway of the Naraj Weir was reduced to counteract the afflux in the Katjuri system, the gauges on the upstream side of the Naraj Weir and at Jobra recorded higher levels than under existing conditions.

(ii) *Increase in the afflux at the Jobra gauge of the Mahanadi Anicut.*

Figure 2 C.5 shows the velocities observed in the model, with peak flood along a section parallel to and 200 feet upstream of the Naraj weir. Computing from this the discharge that flowed through the reduced portion of the Naraj weir, *i.e.*, 35 per cent. (*) with both spills completely closed, 24.8 per cent. in experiment 3 (Series II), and 15.5 per cent. in experiment 1 (Series II), the additional discharge passed down the Mahanadi on reducing the weir length would be—

1.42 lakhs cusecs	with 35 per cent. reduction of Naraj weir waterway.
0.985	do. 24.8 do. do.
0.523	do. 15.5 do. do.

In the experiments 1 and 3 of series II, therefore, the discharge of the Mahanadi below Naraj and below the Birnasi gap was as shown in Table 2C.1 below:—

TABLE 2C. 1

Showing the discharges of the Mahanadi below Naraj and below Birnasi gap in lakhs of cusecs.

	Total above Naraj	Due to reduction of Naraj weir	Mahanadi below Naraj	Birnasi gap	Mahanadi below Birnasi gap
According to Rhind (existing conditions).	15.00	Nil	6.58	0.64	7.22
Series II, Experiment 1, Birnasi gap open, Banki Road spill Q=50,000 cusecs and Naraj Weir length reduced by 15.5 per cent.	15.00	0.523	7.103	0.64	7.743
Series II, Experiment, 3, Birnasi gap closed, Banki Road spill Q=50,000 cusecs and Naraj weir length reduced by 24.8 per cent.	15.00	0.985	7.565	Nil	7.565

Assuming that the Mahanadi and the Birupa share the discharge below the Birnasi gap in the same ratio as under existing conditions, *viz.* 6.1 lakhs cusecs in Mahanadi and 1.12 lakhs cusecs in Birupa, then the discharge of the Mahanadi in Experiment 1 of Series II=6.62 lakhs cusecs and in experiment 3 of Series II=6.46 lakhs cusecs.

(*) *Ibid.*

Under existing conditions, the average coefficient of the Mahanadi anicut works out to 2.35 as shown below :—

Length of weir=6373.5 feet.

Maximum water level =75.0.

Sill R. L. =64.27.

h_v or head due to velocity of approach = $\frac{(7.4)^2}{50} = 1.10$ feet.

$D = D_1 + h_v = 10.73 + 1.10 = 11.83$ feet.

Q over weir = 6.10 lakhs cusecs.

6.1×10^5

$\therefore C = \frac{6.1 \times 10^5}{6373.5 \times (11.83)^{1.5}} = 2.35.$

Hence, to pass 6.62 and 6.56 lakhs cusecs, the head required will be 12.49 feet and 12.30 feet respectively. Thus, the afflux in experiment 1, would be $12.49 - 11.83 = 0.66$ feet and in experiment 3 it would be $12.30 - 11.83 = 0.47$ feet.

As, however, the weir is short-crested and is in a drowned condition, the actual 'C' will be less than 2.35 and the calculated afflux figures would be slightly higher. The afflux observed in the model was, however, 0.945 feet in experiment 1 of Series II and 0.806 feet in experiment 3 of Series II. Both the figures were higher than their computed values, which was due to scale effect.

The afflux in the prototype is, therefore, likely to be about 0.75 feet and 0.50 feet under conditions of experiments 1 and 3 respectively.

A test was also made with the proposed road bridge, 1/2 mile below, parallel and similar to the existing Katjuri Railway bridge, to see if any increase in the afflux is caused by it. The model did not show any increase in the afflux.

(iii) *Afflux at the Naraj upstream gauge*

When the length of the Naraj weir was reduced in various experiments, there was a rise in the water level at the upstream Naraj gauge as compared with the existing conditions as shown in Table 2 C. 2.

TABLE 2 C. 2

Showing the rise in water level at the Naraj upstream gauge, at peak flood of 15 lakhs cusecs

Experiment No. Series III	Experiment	Reduction in Naraj weir waterways	Rise in water level	
			Model	Prototype (equivalent rise)
1	Birnasi gap open and Banki $Q=50,000$ cusecs	15.5 per cent.	0.021 foot	1.38 feet
2	Birnasi gap closed and Banki $Q=50,000$ cusecs	24.8 per cent.	0.029 foot	1.91 feet

The model rise compares fairly with the calculated rise, thus :—

Under existing conditions, the conditions at the Naraj weir with peak flood of 15 lakhs cusecs are as follows :—

$$Q=8.40 \text{ lakhs.}$$

$$B, \text{ i.e., length of weir} = 3,900 \text{ feet.}$$

$$\text{Sill R. L.} = 72.6$$

$$\text{Water level with peak flood} = 91.0$$

$$h_v \text{ i.e. velocity of approach} = \frac{(10.5)^2}{50} = 2.21 \text{ feet}$$

$$\therefore D = D_1 + h_v = (91 - 72.6) + 2.21 = 20.61 \text{ feet.}$$

$$\therefore C = Q/BD^{1.5} = 2.30$$

• When the Naraj waterway could be reduced by 24.8 per cent. or 15.5 per cent. the discharge in the Katjuri would be reduced by 0.985 lakhs cusecs or 0.523 lakhs cusecs, respectively ; with $C=2.30$ the head required to pass the remainder from the 8.4 lakhs cusecs will be as in Table 2C. 3.

TABLE 2C. 3

Reduction in waterway at Naraj	Q passing into the Katjuri (Cusecs)	$D = \left(\frac{Q}{2.3 \times B} \right)^{\frac{2}{3}}$ feet	Increase in afflux (feet)
Nil	8.4	20.61	—
15.5 per cent.	7.877	22.10	1.49
24.8 per cent.	7.415	22.90	2.29

The model rise was slightly less (see Table 2C. 2). Model figures may, however, be taken as more correct, considering that the weir in the model was adjusted to get the relative position of standing wave downstream and the coefficient of discharge the same as in the prototype ; besides, in the above calculations for the prototype, a straight approach is assumed, whereas there is a sharp curvature of flow upstream of the weir which the model simulates.

(iv) *Distribution of sand load in the various branches*

Table 2C. 4 shows the quantity of sand collected in the downstream sand pit of each branch, the "time average" discharge, i.e. the weighted mean discharge weighted in respect of time, and the sand charge in the different branches relative to the time average discharge in various experiments both (a) as observed and (b) as corrected for final completed stability when the injected charge equals that collected downstream. Thus, the figures denoted by (a) in Table 2C4 are the quantities actually collected in the pits and the corresponding sand

TABLE

Showing the ratio of the sand drawn by the Mahanadi

Details of experiments	Total quantity of sand collected in all branches	Mahanadi		
		$\frac{M}{QT}$ T	Q_s	X
1	2	3	4	5
Series I—With unrestricted waterway of Naraj Weir				
Experiment 1 with existing conditions. i.e. Birnasi gap Q=64,000 Bank spill Q=1,00,000 flood.	(a) 14·000 (b) ..	1·390 ..	6·000 ..	0·820 ..
} at peak flood				
Experiment 2 with Birnasi gap full open. Banki Spill Q=50,000 cs. (at peak flood).	(a) 14·375 (b) 14·000	1·390 ..	6·000 5·850	0·820 0·799
Experiment 3 with Birnasi gap full open, Banki Spill Q=33,000 cs. (at peak flood.)	(a) 14·100 (b) 14·000	1·400 ..	6·000 5·960	0·814 0·808
Experiment 4 with Birnasi gap Q nil. Banki spill Q=1,00,000 cs. (at peak flood).	(a) 14·500 (b) 14·000	1·360 ..	6·000 5·795	0·840 0·811
Experiment 5 with Birnasi gap Q nil Banki spill Q=50,000 cusecs (at peak flood).	(a) 15·125 (b) 14·000	1·370 ..	8·000 5·550	0·838 0·773
Experiment 6 with Birnasi gap Q nil. Banki spill Q=33,000 cs. (at peak flood).	(a) 16·000 (b) 14·000	1·380 ..	6·000 5·250	0·826 0·722
Experiment 7 with Birnasi gap Q nil. Banki spill Q=nil (at peak flood).	(a) 16·750 (b) 14·000	1·390 ..	6·250 5·220	0·855 0·714
Series II—With Naraj Weir Waterway reduced.				
Experiment 1 with Birnasi gap full open. Banki spill Q=50,000 cs. at peak flood. Naraj waterway reduced by 15·5 per cent.	(a) 15·000 (b) 14·000	1·500 ..	7·000 6·550	0·880 0·825
Experiment 2 with Birnasi gap full open ; Banki spill Q=33,000 cs. at peak flood and Naraj weir waterway reduced by 16·85 per cent.	(a) 16·750 (b) 14·000	1·580 ..	7·500 6·260	0·900 0·752
Experiment 3 with Birnasi gap closed. Banki spill Q=50,000 cs. Naraj weir waterway reduced by 24·8 per cent.	(a) 14·50 (b) 14·000	1·540 ..	7·500 7·240	0·925 0·893
Experiment 4 with Birnasi gap closed. Banki spill Q=33,000 cs. Naraj weir waterway re- duced by 28·5 per cent.	(a) 17·25 (b) 14·000	1·600 ..	8·000 6·500	0·950 0·771

(a) figures denote the quantity of sand and the ratios as observed in the model.

(b) figures denote the quantity of sand and the ratios after correction to make total sand.

Q = discharge in cusecs; Q_s = quantity of sand collected in cubic feet; T = time in minutes;

$$\frac{M}{QT} \quad \frac{M}{QT}$$

X = charge in, parts per 1,000 (by weight) = Ratio of Q_s to T where T is the taking the sum of the products of each stage discharge and its time of duration.Col. 5 = $0.19 \times$ col. 4 / col. 3.

2C. 4

and its branches to the time average discharge

Katjuri loop			Surua			Kuakhai		
$\frac{MQT}{T}$	Q_s	X	$\frac{MQT}{T}$	Q_s	X	$\frac{MQT}{T}$	Q_s	X
6	7	8	9	10	11	12	13	14
0.769	3.000	0.740	1.526	4.000	0.498	0.516	1.000	0.370
"	"	"	"	"	"	"	"	"
0.775	3.250	0.796	1.538	4.125	0.510	0.510	1.000	0.372
..	3.166	0.775	..	4.010	0.496	..	0.974	0.362
0.772	3.250	0.797	1.530	3.500	0.435	0.510	1.350	0.503
..	3.230	0.791	..	3.470	0.432	..	1.340	0.498
0.770	3.500	0.865	1.550	4.000	0.490	0.506	1.000	0.373
..	3.380	0.830	..	3.860	0.473	..	0.965	0.360
0.800	3.750	0.890	1.620	4.250	0.498	0.528	1.125	0.403
..	3.470	0.823	..	3.930	0.460	..	1.040	0.372
0.820	4.000	0.930	1.630	4.500	0.525	0.544	1.500	0.524
..	3.500	0.814	..	3.940	0.459	..	1.310	0.459
0.830	4.500	1.030	1.650	4.375	0.506	0.550	1.625	0.560
..	3.760	0.861	..	3.660	0.423	..	1.358	0.468
0.750	3.000	0.760	1.420	4.000	0.535	0.450	1.000	0.423
..	2.800	0.710	..	3.720	0.499	..	0.934	0.394
0.770	3.250	0.801	1.320	4.500	0.648	0.490	1.500	0.580
..	2.720	0.670	..	3.720	0.541	..	1.255	0.485
0.735	3.000	0.775	1.450	3.000	0.393	0.470	1.000	0.405
..	2.900	0.749	..	2.900	0.379	..	0.965	0.391
0.720	3.500	0.923	1.410	4.500	0.607	0.440	1.250	0.540
..	2.840	0.750	..	3.650	0.493	..	1.010	0.436

collected equal to sand injected.

weighted mean discharge with respect to time, computed by and dividing by the total time of the flood cycle.

to weighted-mean-discharge ratios in the various experiments : while those denoted by (b) are corrected values to enable a fair comparison by making the total sand collected equal to the total sand injected in all the branches (14 cubic feet) as in Experiment 1 of Series I (*i.e.* existing conditions). Obviously, these corrections become less and less justifiable as the difference between the aggregate sand collected and injected becomes larger.

Experiments 1, 2 and 3 of Series I done earlier during the year of report, were repeated so as to form a complete comparative series. This was found necessary because, due to constant running of the model, fine particles in the bed sand had been washed away and the model sand, which had originally its mean diameter of 0.314 mm., now gave $m=0.389$, with consequent observed increase in slope exaggeration from 5.00 to 5.44 and reduction in observed vertical exaggeration from 6.06 to 5.13.

Analysing the corrected figures, *i.e.* (b) of Table 2C.4 it will be seen that in Experiments 1 to 3, Series I the Mahanadi improved, as compared with the existing conditions, as the Banki spill discharge was reduced. This was apparently because of the Birnasi gap passing increased quantities of the surface, sand-free water, thus making a leaner mix in the Mahanadi consequent upon, the rise in the Katjuri levels due to the Banki Road spill being curtailed.

When the Banki Road spill was reduced to 33,000 cusecs like the Mahanadi, the Surua also improved ; but the Katjuri loop and the Kuakhai drew relatively richer mixture—the Katjuri loop getting 6.9 per cent. richer and Kuakhai as much as 34.4 per cent. more relative sand charge as compared with Experiment 1. The Kuakhai drew a markedly greater quantity of relative charge of bed sand.

In Experiment 7, with Birnasi gap closed by means of an embankment ending in a spur (*), the effect as regards relative sand load in the Katjuri loop was bad, whereas that in the Kuakhai became even worse than in Katjuri loop when Banki spill discharge was reduced to 33,000 cusecs. The Surua, however, drew 7.84 per cent. less relative sand charge.

The Mahanadi showed progressive improvement in these experiments, even though its share of the relatively sand-free Birnasi gap spill discharge was cut off. This may probably have been due to the afflux in the Katjuri system, which in Experiment 7 was as high as 4.5 feet at Cuttack, diverting bigger sand quantities into the Katjuri system. This is also clear from the fact that the total quantity of sand drawn by the Katjuri system in Experiment 4 was 8.5 cubic feet as observed (and 8.25 cubic feet as corrected) which gradually increased to 10.5 cubic feet as observed (and 8.78 cubic feet as corrected) in Experiment 7.

(*) *Ibid.*

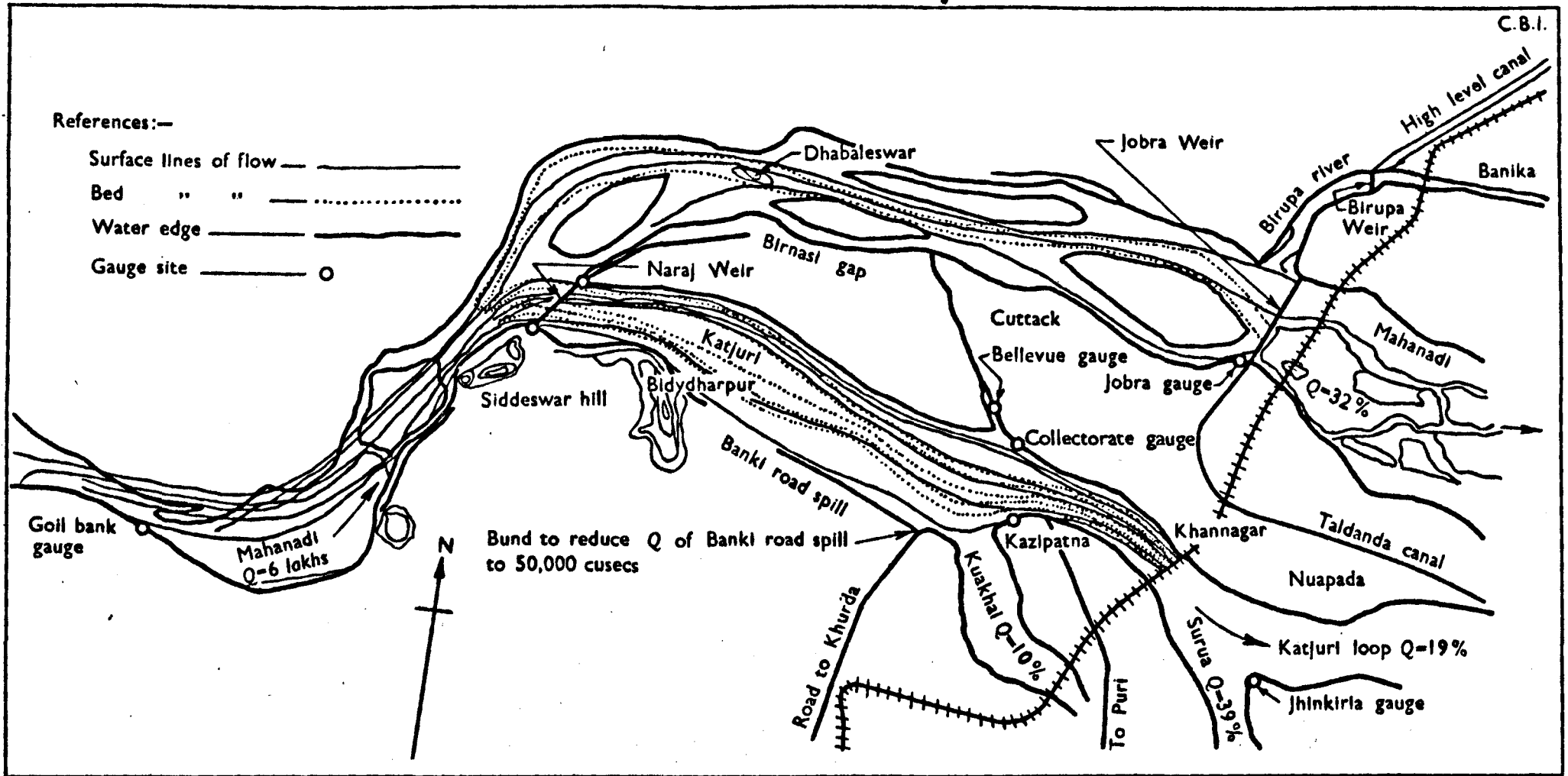


Figure 2C.7:— Showing lines of flow with 6 lakh Cusecs flood (above Naraj Weir) in the experiment with Banki Spill discharge reduced to 50,000 Cusecs, Birnasi gap open & Naraj Weir waterway as at present

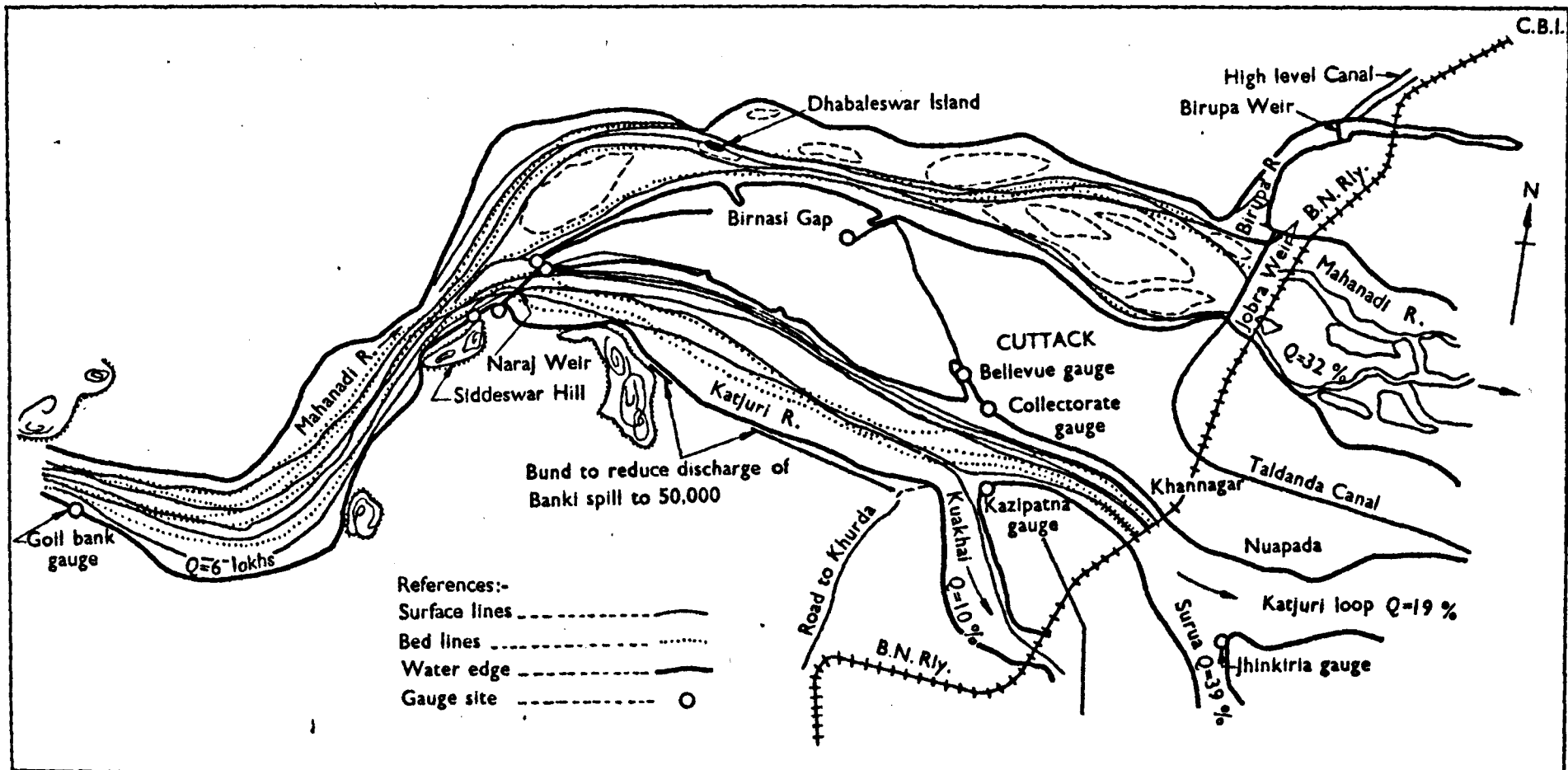


Figure 2 C.8:—Showing lines of flow with 6 lakhs flood in the experiment with Banki spill discharge reduced to 50,000 cusecs, Birnasi gap open and Naraj Weir waterway reduced by 15.5%

In Experiments 1 to 7 of Series I, a general feature observed is that the Banki Road spill discharge need not be reduced to more than 50,000 cusecs to maintain existing sand charge conditions in the Kuakhai ; and also that, on account of large discharges passed down, the Katjuri loop below the Surua head was the main "sand-drawing" off take.

In Series II, the discharge of the Katjuri system was reduced by contracting the waterway of the Naraj weir from its upstream right end ; so as to counteract the increase in afflux which would otherwise be caused in the Katjuri system by shutting off the spills. The immediate effect of this was to divert a greater portion of the bed water into the Mahanadi than it draws under existing conditions Figure 2 C. 6.

Comparing corrected relative sand charges of Experiment 1 (Series II) with Experiment 1 (Series I) it will be seen that whereas in Experiment 1 (Series II) the Mahanadi and the Surua sand charge was almost the same, the Katjuri loop drew slightly less and the Kuakhai 6.45 per cent. more.

In Experiment 3 (Series II), the Mahanadi took 8.9 per cent. more relative sand charge as compared with Experiment 1 (Series I). This was partly due to the shutting off of the Mahanadi share of the clear Birnasi gap water and partly to the diversion of the bed water into the Mahanadi due to shutting off the 24.8 per cent. waterway of the Naraj weir. In the Katjuri river system, the Surua improved, conditions in the Katjuri loop were almost the same as in Experiment 1 of Series I, and the Kuakhai drew, relatively, 5.7 per cent. more sand charge.

Results of both Experiments 2 and 4 appear to be anomalous, probably due to the large differences between the aggregate collected and injected sand quantities. The anomaly is in respect of Mahanadi which improved in both experiments, whereas the Surua which in none of the experiments drew bigger sand charge than in Experiment 1, Series I, did so in Experiment 2. These experiments need repetition.

(v) *Effect of reducing the Naraj weir length*

Figures 2C. 7 and 2C. 8 show the lines of flow with 6 lakhs cusecs flood in the experiment with Birnasi gap open and Banki spill discharge reduced to 50,000 cusecs, (a) with existing Naraj weir waterway, and (b) with Naraj weir waterway reduced by 15.5 per cent.

It will be seen that the surface lines entered the Katjuri at a more oblique angle across the weir to follow the left bank and the bed lines tended to follow the right bank, when the Naraj weir waterway was reduced. At Kazipatna, the surface flow was towards Cuttack side, *i.e.* left bank, whereas the bed flow was towards the Kazipatna side, *i.e.* right bank.

The effect of reducing the Naraj weir length on the Katjuri system was to increase surface flow to the left bank and bed flow to the right bank.

Except at the peak flood, the river levels in the Katjuri system will be lower than under existing conditions, thereby giving a longer period for the river to flow within its banks and scour away the excess sand already accumulated.

On the Mahanadi side there did not appear to be any marked change in the conditions except that there was more bed flow in it due to its diversion caused by the reduction of Naraj weir length.

CONCLUSIONS

(i) The experiments carried out were with unrestricted flow of sand and water from the catchment of the river down to the sea. When the proposed dams and barrages across the river are constructed they will alter the sand load and discharge of the Mahanadi system and by themselves may modify conditions at Cuttack to an unpredictable degree, and may necessitate further experimentation in the light of the altered conditions. The conclusions given below apply to the existing conditions of distribution of sand and water; but may be made useful, with due modifications in light of the altered conditions, if the distribution are not significantly affected.

(ii) An analysis of Table 2 C. 4 would appear to show Experiment 1 of Series II—with Naraj waterway reduced by 15.5 per cent. Birnasi gap fully open and the Banki Road spill discharge reduced down to 50,000 cusecs—as most promising. In this case, however, the beneficial effect of the embankment ending in a spur, in coercing a deep low water channel towards the south of Cuttack will be absent; but conditions of distribution of sand relative to discharge of water in the Mahanadi would remain more or less as they are at present while those in the Katjuri system will improve. The improvement is likely to be better and quicker than indicated in the model; because, whereas, the effect of passing more clear water was to increase the bed sand movement in the model, in the river, where much of the sand goes in suspension the effect will be to make a leaner mix at the bed, thereby reducing sand entry into the offtake. The area covered by the Birnasi spill cannot, under this proposal, be reclaimed for the possible extension of Cuttack.

If it is adopted the Naraj weir should be first restricted and if time does not permit it, the Banki Road spill may be closed partially in the following year, without causing any abnormal conditions in the Katjuri system.

(iii) From the point of view of increasing the carrying capacity of the Katjuri and coercing a deep channel towards the Collectorate gauge on the south of Cuttack, it would appear better to shut off the Birnasi spill by the embankment ending in a spur—*vide* Experiments, Series II—but this is apparently harmful to the Mahanadi itself as it will then get a richer sand mix. (See Table 2C. 4).

If this proposal is adopted, it would be necessary to reduce the Naraj weir waterway by 25 per cent. and also simultaneously to shut off the Birnasi spill. The Banki spill also has to be closed, but this can be done next season if there is not sufficient time to do all the three works in the same year.

(iv) The optimum solution would probably be a modification of (iii) above, *i.e.* to allow a certain portion of the surface water of the peak flood from the Katjuri into the Mahanadi by providing adequate high-level openings (sluices) in the embankment, say by keeping Birnasi gap half closed and Naraj weir accordingly reduced by about 20 per cent. Tests for this proposal are in hand.

(v) The reduction of bed sand entering the Katjuri, because of its eventual leaner mix would lead to retrogression and straightening in the length where retrogression would occur. This, however, cannot be shown by the model due to inherent model limitations. Downstream of this there will be increased secondary meandering⁽⁵⁾ in a limited length. Generally, however, the conditions in the Katjuri system may be expected to improve.

⁽⁵⁾ Indian waterway Experiment Station Annual Report (Tech), 1942-43, page 51.

(2) FLOOD ABSORPTION IN KRISHNARAJASAGAR (6)

ABSTRACT

Flood absorbing capacity is the sum of the discharges from the outlets and spillways, together with the discharges bottled up in the reservoir between full reservoir level and maximum water level. It is calculated on the reservoir level at the start of the flood and rates of inflow and outflow corresponding to a particular hydrograph of inflow. It is expressed as the value of the peak discharge in the hydrograph. It is shown in this paper that flood absorbing capacity of a reservoir depends not only on the rates of increase of flood but also on its duration. The total capacity of the reservoir between full reservoir level and maximum water level which may be sufficient to absorb a particular intensity of flood may take in a flood of higher intensity provided its duration is proportionately short. Correct data of the relation between high floods and their duration is also necessary for determining the flood absorbing capacity of a reservoir.

The Krishnarajasagar Dam is 130 feet in height and its full reservoir level is at + 124 feet. Its maximum water level may be taken to be at + 126 feet for which storage the dam has been calculated to be stable. The maximum flood ever recorded was 279,089 cusecs on July 26, 1924.

Accurate records of floods are available for thirty years from 1916 to 1946. During floods observations have been recorded once every three hours. In all calculations, the unit of time has, therefore, been taken as three hours. The following data have been taken into account for computing the flood absorbing capacity:—

- (i) The margin in feet between full reservoir level and maximum water level.
- (ii) Capacity of the lake between full reservoir level and maximum water level available for flood absorption.
- (iii) The discharging capacity of the outlets from the reservoir.
- (iv) Maximum rate of rise of flood recorded and
- (v) Duration of flood.

CAPACITY OF THE RESERVOIR AVAILABLE FOR FLOOD ABSORPTION

During the time, when a flood is rising, the reservoir level is always kept low to have a greater margin for flood absorption, but not too low to miss picking up level to full reservoir level should the floods fall rapidly. In these calculations it has been assumed that the full reservoir level will be at + 123 feet at the time of a rising flood. The margin between full reservoir level and maximum water level will be three feet, the capacity between them being as follows:—

		Million Cubic feet
Capacity between	+123 and + 124 feet	1,366
do.	+124 and + 125 "	1,396
do.	+125 and + 126 "	1,425
Total		4,187

(6) Hydraulic Research Station, Krishnarajasagar, Report for the year 1946, pages 2-15.

DISCHARGING CAPACITY OF THE OUTLETS

These have been worked out for the various sluices for every increase of 0.1 foot in reservoir level (vide Table 2 C. 5). The coefficients of discharge for various sluices for different heights of vents opening and different reservoir levels have been determined after conducting accurate experiments. The discharging capacity of all the outlets at + 123 feet is 370,100 cusecs.

MAXIMUM RATE OF RISE OF FLOOD

Records of high floods which have occurred so far have been analysed and it has been found that the maximum rate of rise ever recorded is a rise of 16,120 cusecs in three hours. This occurred during the floods of June 9, 1941. For purposes of calculations, the maximum rate of increase has been assumed to be 18,000 cusecs in three hours. The rate of increase is never uniform in actual floods. The rate of increase begins to decrease as the flood nears its peak. For calculations the maximum rate of increase of flood has been taken as the rate of increase for the complete duration of the flood thus getting a slightly lower figure for the flood absorbing capacity.

DURATION OF FLOODS

As the discharging capacity of the outlets is 370,100 cusecs and the maximum flood which has occurred so far is only 279,089 cusecs, there has been no occasion when the flood absorbing capacity of the reservoir had to be depended upon for routing a flood. Table 2.C6 shows flood absorbed in the capacity of reservoir between + 123 feet and + 126 feet for different rates of rise of flood.

The frequencies suggest that the relation between n and g is of the form $n=ag^{-b}$ as g becomes larger n becomes smaller and as g tends to zero n tends to infinity.

$$\therefore \log n = \log a - \log g$$

and putting $y = \log n$, $x = \log g$

$y = \log a - b x$, which is a linear equation and a straight line can be fitted to $\log n$ and $\log g$

The preliminary work is shown in Table 2C.6. The equation with the help of which we have to determine a_0 i.e. $(\log a)$ and a_1 i.e. $(-b)$ are

$$54.1853 - 11 a_0 - 10.2097 a_1 = 0$$

$$47.5751 - 10.2097 a_0 - 11.1240 a_1 = 0$$

Solving these in the usual way we have

$$a_0 = 6.4574 ; a_1 = -1.65$$

TABLE 2C. 5

Showing the discharge in cusecs through the several sluices located in the dam for reservoir water levels varying by 0.1 foot from 123 to 126

R. L.	+ 12 Sluices 8 of 6 feet × 12 feet	+ 50 Sluices 3 of 6 feet × 15 feet	+ 80 Sluices 16 of 10 feet × 20 feet	+ 103 Sluices 48 of 10 feet × 8 feet	+105 Sluices 40 of 8 feet × 12 feet	+114 Sluices 48 of 10 feet × 10 feet	Irwin Canal discharge and evapora- tion losses	Total issue h throu g all the sluices	Increase in cusecs above the dis- charge capacity at + 123	
1	2	3	4	5	6	7	8	9	10	11
									Diffe- rences	Total
123.0	29,872	11,226	119,760	99,552	68,320	38,880	2,500	370,110	..	—
.1	29,872	11,241	119,968	99,792	68,520	39,552	..	371,445	1,335	1,335
.2	29,904	11,256	120,176	100,032	68,960	40,128	..	372,956	1,511	2,146
.3	29,904	11,256	120,384	100,512	69,160	40,848	..	374,564	1,608	4,434
.4	29,936	11,268	120,592	100,800	69,560	41,568	..	376,224	1,660	6,114
.5	29,936	11,268	120,800	101,040	69,760	42,144	..	377,448	1,224	7,338
.6	29,960	11,283	121,008	101,280	70,200	42,864	..	379,095	1,647	8,985
.7	29,960	11,298	121,216	101,760	70,100	43,440	..	380,574	1,479	10,464
.8	29,992	11,298	121,216	102,000	70,800	44,160	..	381,966	1,392	11,856
.9	29,992	11,310	121,424	102,288	71,000	44,928	..	383,442	1,476	13,332
124.0	30,016	11,310	121,632	102,528	71,200	45,504	..	384,690	1,248	14,580
.1	30,016	11,325	121,840	103,008	71,640	46,272	..	386,565	1,875	16,455
.2	30,048	11,337	122,048	103,248	71,840	46,848	..	387,869	1,304	17,759
.3	30,048	11,337	122,256	103,536	72,240	47,616	..	389,533	1,664	19,423
.4	30,080	11,352	122,464	103,776	72,440	48,240	..	390,852	1,319	20,742
.5	30,080	11,367	122,464	104,016	72,880	49,008	..	392,315	1,463	22,205
.6	30,080	11,367	122,688	104,496	73,080	49,776	..	393,987	1,672	23,877
.7	30,104	11,379	122,896	104,736	73,280	50,400	..	395,295	1,308	25,185
.8	30,104	11,379	123,104	105,024	73,680	51,168	..	396,959	1,664	26,849
.9	30,136	11,394	123,312	105,264	73,880	51,792	..	398,278	1,319	28,162
125.0	30,136	11,409	123,520	105,504	74,320	52,608	..	399,997	1,719	29,887
.1	30,168	11,409	123,520	105,744	74,520	53,232	..	401,093	1,096	30,983
.2	30,168	11,421	123,728	106,272	74,720	54,048	..	402,857	1,764	32,747
.3	30,192	11,421	123,936	106,512	75,120	54,672	..	404,353	1,496	34,243
.4	30,192	11,436	124,144	106,752	75,320	55,488	..	405,832	1,479	35,722
.5	30,224	11,448	124,352	106,992	75,520	56,160	..	409,196	1,364	37,086
.6	30,224	11,448	123,560	107,232	75,960	56,976	..	408,900	1,704	38,790
.7	30,256	11,463	124,768	107,473	76,160	57,600	..	410,219	1,319	30,109
.8	30,256	11,463	124,768	108,000	76,360	58,464	..	411,817	1,598	41,706
.9	30,280	11,478	124,976	108,240	76,760	59,166	..	413,370	1,553	4,326
126.0	30,280	11,490	125,184	108,480	76,960	59,808	..	414,702	1,332	44,592

TABLE 2C. 6

Curve fitting to express the relationship between the rate of increase of flood and the duration of flood for absorbing the capacity between + 123 and + 126

Rate of increase of flood in cusecs for every three hours <i>n</i>	Duration of flood necessary for absorbing of the capacity between +123 and +126 contour in hours <i>g</i>	Log <i>n</i> = <i>y</i>	Log <i>g</i> = <i>x</i>	<i>y x</i>	<i>x²</i>
5,000	48.00	3.6990	1.6812	6.2188	2.8264
10,000	30.00	4.0000	1.4771	5.9084	2.1818
18,000	21.00	4.2553	1.3222	5.6264	1.7482
50,000	12.00	4.6990	1.0792	5.0712	1.1647
100,000	7.50	5.0000	0.8751	4.3755	0.7658
150,000	6.00	5.1761	0.7782	4.0280	0.6056
200,000	5.25	5.3010	0.7202	3.8178	0.5187
250,000	4.50	5.3979	0.6532	3.5259	0.4267
300,000	4.00	5.4771	0.6021	3.2978	0.3625
350,000	3.50	5.5441	0.5441	3.0165	0.2960
432,277	3.00	5.6358	0.4771	2.6888	0.2276
..	..	54.1853	10.2097	47.5751	11.1240

Therefore the straight line is

$$y = 6.46 - 1.65 x$$

from which we have

$$n = 10^{6.46 - 1.65 g}$$

$$n = 2,866,750^{-1.65 g}$$

$$\therefore y = 2,866,750 x^{-1.65}$$

$$\text{or } y x^{1.65} = 2,866,750$$

It will be seen, therefore, that the time taken by the flood to get absorbed in the lake varies inversely as its intensity. It can be written in the form of an equation.

$$y x^{1.65} = 2,866,750 \quad \dots \quad (2C.1)$$

where y = rate of increase of flood in cusecs

x = time taken by the flood for absorption in the capacity of the lake between +123 and +126 feet contours.

FLOOD ABSORPTION IN THE RESERVOIR

The method of calculating is explained below and in Tables 2C.7, 2C.8 and 2C.9.

Data

(i) Capacity of reservoir between 123 and 124 feet	= 1,360 million cubic feet
124 and 125 feet	= 1,396 million cubic feet
125 and 126 feet	= 1,425 million cubic feet
(ii) Discharging capacity of all the sluices at +123	= 370,100 cusecs
Rate of rise of flood above 370,100 cusecs in every three hours	= 10,000 cusecs
(iii) Margin between full reservoir level and maximum water level available for flood absorption	= 3 feet
Discharging capacity at + 123	= 370,100 cusecs.
Increase in flood at the end of first three hours	= 10,000 cusecs

As a First Approximation :—Assume the entire increase in discharge to get impounded.

The quantity impounded	= $3 \times 3600 \times 10,000$	= 108 million cubic feet
The reservoir level corresponding to holding up 108 million cubic feet above + 123.		= 123.08 feet
Discharging capacity of all the outlets at 123.08		= 371,168 cusecs.
Increase in discharges capacity of all the outlets at 123.08 over that at + 123 = 371,168—370,100		= 1,068 cusecs
Quantity held up = 10,000—1,068 cusecs		= 8,932 cusecs

As a Second Approximation :—Assume this entire 8,932 cusecs get impounded.

Quantity impounded.	= $8932 \times 3 \times 3600$	= 97 million cubic feet
Reservoir level corresponding to holding up 97 million cubic feet above 123 feet		= 123.07 feet
Increase in discharging capacity of all the outlets at + 123.07 feet over that at + 123		= 935 cusecs
Quantity held up = 10,000—935		= 9,065 cusecs

TABLE 2C.7

Sample calculation sheet for a rate of rise of floods of 10,000 cusecs every three hours

Period or time	Increase of flood in cusecs	Approximate reservoir level	Increase in issue above plus 123 in cusecs	Quantity held up in cusecs	Equivalent in million cubic feet for Col. 5	Total quantity held up above plus 123 in million cubic feet	Equivalent reservoir level
1	2	3	4	5	6	7	8
3 hours	10,000	123.0	1,068	8,932	108	108	123.08 (April 1)
		123.08	935	9,065	97	97	123.07 (April 2)
6 hours	20,000	123.07	935	19,065	182	303	123.22
		123.22	3,168	16,832	112	279	123.20
9 hours	30,000	123.20	2,846	27,154	293	572	123.42
		123.42	6,359	23,641	255	534	123.39
		123.39	5,992	24,408	259	538	123.39
12 hours	40,000	123.39	5,992	34,008	367	905	123.66
		123.66	9,873	30,127	325	863	123.63
		123.63	9,429	30,571	330	868	123.63
27 hours	50,000	123.63	9,429	40,571	448	1,316	123.96
		123.90	14,080	35,920	388	1,256	123.92
		123.92	13,582	36,418	393	1,261	123.92
30 hours	60,000	123.92	13,582	46,418	501	1,762	124.28
		124.28	19,161	40,839	441	1,702	124.25
		124.25	18,773	41,227	445	1,704	124.25
21 hours	70,000	124.25	18,773	51,257	553	2,259	124.64
		124.64	29,397	45,603	492	2,198	124.60
		124.60	23,877	40,123	498	2,204	124.60
24 hours	80,000	124.60	23,877	50,123	606	2,812	125.04
		125.04	30,327	49,673	536	2,742	124.79
		124.99	29,887	50,113	541	2,747	124.99
15 hours	90,000	124.99	29,887	60,113	649	3,396	124.00
		125.44	36,266	53,734	580	3,327	125.39
		125.39	35,586	54,414	588	3,355	125.39
18 hours	100,000	125.39	35,586	64,414	696	4,031	125.86
		125.86	42,637	57,303	620	3,955	125.84
		125.84	42,327	57,673

Second Three Hours :—i. e. six hours after water level has reached full reservoir level.

First Approximation:—

Increase in discharging capacity above +123 Full Reservoir Level =935 cusecs
 Increase in flood at the end of six hours. =20,000 cusecs
 Quantity impounded in three hours 20,000—935=19,065 cusecs
 =19,065 × 3 × 3,600 = 206 million cubic feet
 Total quantity held up above +123=97 × 203 in the first three hours and second three hours = 303 million cubic feet
 Reservoir level due to holding up of 303 million cubic feet above +123 = 123.22 feet
 Increase in discharging capacity of all the outlets at +123.22 over that at +123. = 3,168 cusecs
 Quantity held up in the reservoir=20,000—3,168 = 16,832 cusecs
Second Approximation :—
 Quantity impounded 16,832 × 3 × 3,600 = 182 million cubic feet
 Total quantity held up above +123 }
 Quantity held up in the first three hours + the quantity now held up } 97+182 = 279 million cubic feet
 Reservoir level due to holding up 279 million cubic feet above +123 = 123.20 feet
 Increase in discharging capacity of all the outlets at +123.20 feet above that at +123 feet = 2,846 cusecs

TABLE 2C.8

Abstract of the results of calculation for flood absorption

(Assumption) Rate of rise of floods in cusecs for every three hours	Flood absor- bed in the capacity of the reservoir between full reservoir level and maximum water level i.e. between plus 123 to 126 feet (cusecs)	Discharging capacity of the outlet at plus 123 full reser- voir level (cusecs)	Total flood absorbing capacity Col. 2 plus 3	Time taken for absorbing of the capacity between plus 123 and 126 in (hours)	Remarks
1	2	3	4	5	6
5,000	80,000	370,100	450,100	48	
10,000	100,000	..	470,100	30	
15,000	120,000	..	498,100	24	
50,000	200,000	..	570,100	12	
100,000	250,000	..	620,100	7½	
150,000	300,000	..	670,100	6	
200,000	350,000	..	720,100	5½	
250,000	375,000	..	745,100	4½	
300,000	400,000	..	770,100	4	
350,000	408,330	..	778,430	3½	
432,277	432,277	..	802,377	3	

TABLE 2C.9

Curve fitting to express the relationship between the rate of increase of flood and the flood absorption capacity

Rate of increase of flood in lakhs of cusecs for every three hours	Flood absorption capacity quantity in lakhs of cusecs above the full discharging capacity of all the dam—sluices at +123 (370,100 cusecs)						
y	x	x^2	x^3	x^4	x^5	yx	yx^2
0.05	0.00	0.64	5.12	0.4096	0.328	0.04	0.032
0.10	1.00	1.00	1.000	1.0000	1.000	0.10	0.100
0.18	1.26	1.59	2.000	2.5204	3.176	0.23	0.286
0.50	2.00	4.00	8.000	16.0000	32.000	1.00	2.000
1.00	2.50	6.25	15.625	39.0625	97.656	2.50	6.250
1.50	3.00	9.00	27.000	81.0000	243.000	4.50	13.500
2.00	3.50	12.25	42.875	150.0625	525.219	7.00	24.500
2.50	3.75	14.06	52.734	197.7539	747.577	9.38	35.150
3.00	4.00	16.00	64.000	256.0000	1,024.000	12.00	48.000
3.50	4.08	16.65	67.932	277.1026	1,130.579	14.28	58.275
4.32	4.32	18.66	80.611	348.2851	1,504.592	18.66	80.611
18.65	30.21	100.10	362.289	1,369.1966	5,009.127	69.69	268.7042

We shall assume for the cubic parabola the equation....

$$y = a_0 + a_1 x + a_2 x^2 + a_3 x^3$$

Since (0, 0) is a point on the curve $a_0 = 0$

∴ The equation is $y = a_1 x + a_2 x^2 + a_3 x^3$

where a_1 , a_2 and a_3 are to be determined with the help of the following equations according to the method of least squares.

$$18.65 - 30.21a_1 - 100.10a_2 - 362.29a_3 = 0 \quad (2c2)$$

$$69.69 - 100.10a_1 - 362.29a_2 - 1369.20a_3 = 0 \quad (2c3)$$

$$268.70 - 362.29a_1 - 1369.20a_2 - 5309.127a_3 = 0 \quad (2c4)$$

Solving these equations in the usual way we have

$$a_1 = 0.005, \quad a_2 = 0.04 \text{ and } a_3 = 0.04$$

Therefore the equation to the cubic parabola is

$$y = 0.005x + 0.04x^2 + 0.04x^3$$

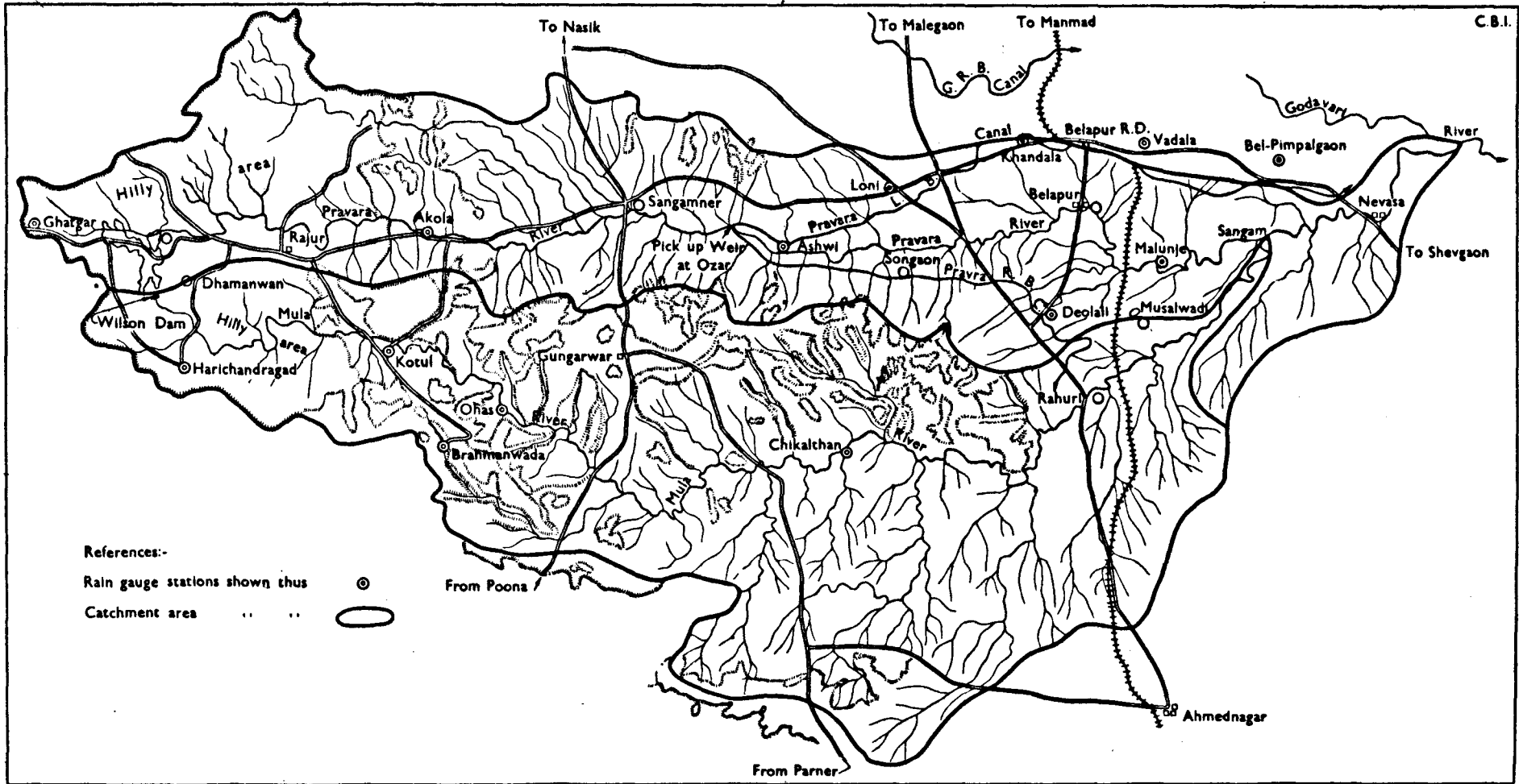


Figure 2C.9:-Showing catchment area of Mula & Pravara Rivers

It is seen from Table 2C.9 that the maximum rate of rise of flood assumed i.e. 18,000 cusecs every three hours and for a capacity of 4,187 million cubic feet gives a margin of three feet below mean water level. The flood absorbing capacity of the reservoir works out to 496,100 cusecs.

RELATION BETWEEN RATE OF INCREASE OF FLOOD AND FLOOD ABSORPTION

The effect on the absorption capacity of the reservoir for different rates of increase of flood has been worked out in Table 2C.8. It is seen therefrom that greater the rate of increase of flood greater is the flood absorbing capacity of the lake. If the flood rises every three hours by 5,000 cusecs more than the discharging capacity of the outlets at +123, that is if the flood is 370,100 cusecs at the start and goes on increasing to 375,100 cusecs three hours after and 380,100 cusecs during the next succeeding three hours and so on, it is seen the flood absorbing capacity works out to 450,100 cusecs. If the rate of rise is 10,000 cusecs every three hours, the flood absorbing capacity is 470,100 cusecs and if the rate of rise is a *lakh* of cusecs every three hours it is 720,100 cusecs. The maximum capacity is that which fills up the lake in three hours after allowing for the outlets to discharge full at the maximum water level. This works out to 802,377 cusecs.

The relation between the rate of increase of flood and the flood absorbing capacity of the reservoir is expressed by the equation,

$$y = 0.005x + 0.04x^2 + 0.04x^3 \quad \dots \quad (2C.5)$$

Where y = rate of increase of flood in *lakhs* of cusecs every three hours.

x = quantity in *lakhs* of cusecs above the full discharging capacity of all the outlets at +123.

It is thus seen that higher the flood, shorter is its duration. Greater the rate of rise of flood greater is also the flood absorbing capacity of the reservoir.

The maximum flood at Krisnarajasagar is 279,089 cusecs. The discharging capacity of all the outlets at +123 feet is 370,100 cusecs or 13 per cent. of the very maximum.

If the flood absorbing capacity of the reservoir is also considered, the floods that can be negotiated can be between 161 per cent. and 288 per cent. of the extra-ordinary flood that occurred in 1924, the exact magnitude depending on the duration and rate of increase of flood.

(3) AN UNPRECEDENTED FLOOD IN MULA AND PRAVARA VALLEYS AND ESTIMATE OF LOSS OF FLOOD DAMAGE (*)

ABSTRACT

In the Bombay Deccan June to October is the main rainy season. In the eastern part of the Province there occur heavy showers in November also. In the month of November 1946, the catchment area of Pravara and Mula Rivers in Ahmednagar District received a fairly long spell of rainy weather of about ten days which ended with a heavy down pour, varying from four inches to seven inches in about 12 hours. This coming on top of a week of soaking rain caused unprecedented floods in the valleys of Mula and Pravara. Figure 2C. 9 shows the valleys in question with rain gauge stations. Table 2C. 10 gives the data

(*) Poona Irrigation and Research Division, Poona, Report on Irrigation and Allied Research 1946, pages 82—85.

about rainfall in the area. Figure 2C.10 shows the flood levels at three important points viz. Rahuri Belapur, and Kolhar.

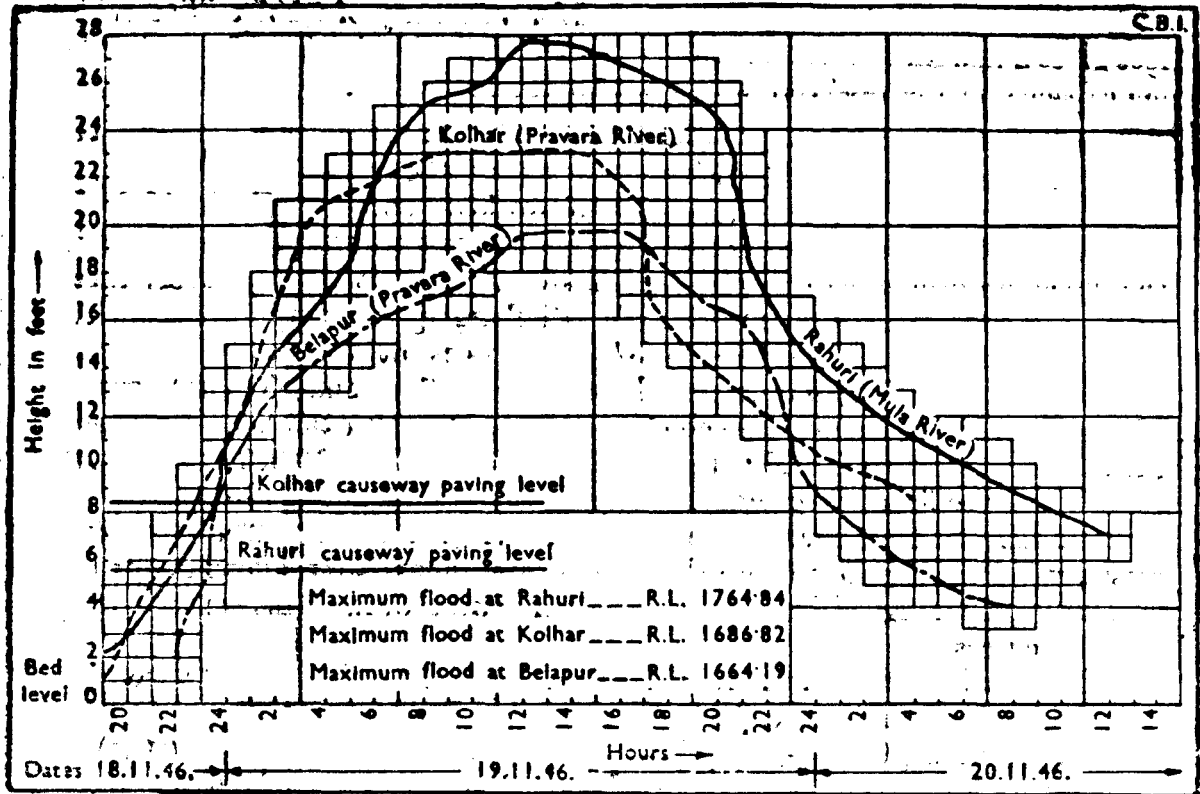


Figure 2C. 10: Showing rise of water level during the high flood on November 18-19, 1946 at Rahuri, Kolhar and Belapur in Ahmednagar District.

AREAS AFFECTED

Talukas affected by these floods were (i) Rahuri (ii) Newasa (iii) Belapur (iv) Shrigonda (v) Sangamner (vi) Parner. Rahuri and Newasa Talukas had to bear the main brunt of these floods. The total number of villages affected by the floods was 117.

FLOOD DAMAGE

The damage to life and property was assessed by a special officer of the Revenue Department and the following gives brief details—

(a) *Human life*.—The number of human lives lost was 28, of which Rahuri Budruk alone lost 15.

(b) *Animal life*.—Goats, bullocks, fowls, horses *etc.* lost in floods were 707 in number. This large toll in animal life was due to the main on-slaught of flood coming in at night when the animals tethered to stakes could not break loose, all human efforts being centred in the rescue of men, women and children.

(c) *Houses*.—Damage to habitations due to floods in the six Talukas was as under:

- (i) No. of houses completely damaged : 197
- (ii) No. of huts completely damaged : 171
- (iii) No. of houses partly damaged : 1,149
- (iv) No. of huts partly damaged : 10

TABLE 2C.10

Showing rainfall at various stations in the Mula and Pravara catchments for periods from November 9 to 21, 1946.

Rain gauge stations	River catchment	November 1946												
		9	10	11	12	13	14	15	16	17	18	19	20	21
Bhandardara	Pravara	0.10	0.8	1.90	0.54	0.80	0.33	4.67
Kotul		5.10
Brahmanwada	Mula	6.50
Sangamner	Pravara	1.61	0.25	2.08	0.16	0.22	..	0.69	4.75
Songaon		..	1.20	0.14	1.00	0.90	0.07	0.20	7.50	..
Ashwi	..	1.50	0.05	1.10	0.55	0.30	6.50
Loni		..	3.05	0.40	0.70	1.15	0.65	..	0.30	6.10	..
Khandala	..	0.85	0.50	0.62	2.20	0.83	3.50
Vadala		..	0.70	0.30	1.25	1.10	0.65	..	0.60	6.00	..
Belpimpalgaon	Mula	0.70	0.65	2.00	2.23	0.20	0.70	8.30
Rahuri		0.26	..	0.40	6.10
Ghatgar	Pravara	3.44
Akola		5.70
Belapur town	..	0.95	0.15	0.50	1.35	0.5	0.47	6.00
Malunje		..	0.35	0.20	0.65	2.10	0.55	..	0.50	3.80
Deolali	..	0.63	0.21	0.80	2.07	1.25	0.21	..	0.31	5.95
Musalwadi		..	1.25	0.50	1.15	1.60	0.10	..	0.20	0.05	..	0.60	6.30	..
Harischandragad	Mula	0.60	1.30	1.55	1.50	0.60	1.14	5.70
Dhamanyan		1.00	1.00	..	0.50	1.11	1.00	3.00	5.00	..
Chas Pimpaldari	..	0.32	1.35	0.85	0.45	8.42
Chikaltah		1.15	0.55	0.45	8.10

RIVER CONTROL

(d) *Damage to lands and crops*: There was no appreciable erosion of lands any where except in Rahuri, but the sheaves of harvested *bajri*, which were collected into heaps in open areas were carried away by floods. In Rahuri Taluka, *Mosambi* trees with fruit on them were much damaged as many such gardens are situated very close to the river banks.

(e) *Moveable property*—As stated above much of the rise in water level occurred in night time and the public could not foresee the calamity in store for them. Thus movable property was also lost during night time.

Estimate of damage:—As regards, loss of property including houses, valuables and crops etc. the following are the approximate estimates for the villages affected by floods.

						Rs.
(i) Sangamner taluka	—(a) Sangamner	50,000
	—(b) Other villages	50,000
(ii) Shrigonda taluka	31,000
(iii) Parner taluka	21,900
(iv) Belapur taluka	1,61,000
(v) Newasa taluka	(a) Newasa Kd.	1,57,600
	(b) Newasa Bd.	2,71,600
	(c) other villages	1,36,000
(vi) Rahuri taluka	(a) Rahuri town	6,00,000
	(b) other villages	8,00,000
(viii) Loss of Government property including damage to roads, buildings, ferry boats etc.	2,00,000
Total						24,79,100

As a result of these floods about 17,000 persons have become destitute and Government has sanctioned about three *lakhs* of rupees for granting free rations to destitutes, supplying materials such as bamboos, galvanised iron sheets and granting *tagai* loans to housing societies and deserving persons.

FLOOD DISCHARGE

From Figure 2C 10 it will be seen that at Rahuri causeway the maximum flood level was eight feet higher than the previous recorded level. There was 22 feet depth of water over the top of causeway. Efforts are being made to estimate approximately the discharges of the Pravara river at Newasa, and the Mula at Rahuri. The bed fall in these rivers at above places is about one foot per mile. Rough data collected so far in Ahmednagar Division show that the discharges must have been about 3,00,000 and 2,00,000 respectively at Newasa and Rahuri. There is no recent record available of any high flood damage in these areas. Enquiries indicate that such high floods have not been heard of during the last 100 years or so. No definite data about frequency of floods can, therefore, be given at this stage.

POSSIBILITIES OF CONTROL

One of the two valleys *viz.* that of Pravara has a storage reservoir at Bhandardara in the *ghats*. This lake is, however, situated near the head of the valley and was not of any use in controlling this flood.

Its contents have since been eased by 373 million cubic feet or about 4,300 day cusecs during the flood period. Two storage tanks of a total capacity of 18,000 million cubic feet contemplated in the Mula valley, if constructed, would help

to an appreciable extent in reducing the high flood discharge of the type that occurred in November. In normal years draw off from such lakes begins by the beginning of October and hence heavy floods in November can to a certain extent be absorbed in the lakes.

(4) MEANDERING RIVERS AND FLOOD CONTROL

by

K. B. Ray

ABSTRACT

There is a belief that a river follows a tortuous course and finds its own length, to attain a state of equilibrium. But in doing so, with the reduction in velocity the discharging capacity is also reduced, there is overtopping of banks, water flows over the country side causing flood damages. It is not at all necessary to reduce the velocity of flow to prevent erosion of river banks which leads to a tortuous course. There is no erosion when a rapid current flows parallel to an erodible bank. Erosion is due to the oblique impingement of the running water on river banks. Such oblique impingement is caused by obstructions in the river bed. A higher velocity of flow with its greater power of erosion, erodes and removes such obstructions and restores the flow to a state of uniform motion in a straight line. The straighter channels in the upper reaches of a river or at high water levels, is explained by the higher velocity of flow. The higher velocity, in deepening and straightening the channel, increases the hydraulic mean depth and slope of water surface, thus maintaining a better channel with a higher discharging capacity, which can accommodate the maximum discharge within its channel without overtopping of banks and flood damages. Rivers in this country have been allowed to assume any alignment and sectional area, but it has not helped them to attain a stable condition. On the contrary there has been progressive diminution in their discharging capacity with overtopping of banks, and floods have increased in intensity and frequency. Embankments are not a permanent remedy for curing floods. What is necessary is to increase the discharging capacity of the river channels. This can be ensured by deepening the rivers and eliminating tortuous alignments. For sluggish rivers, diversion channels will be necessary to avoid tortuous courses, while for live rivers, it will be necessary to train the flowing channel to an alignment with straights joined by flat curves.

There is a belief that a river follows a tortuous course, in its natural attempt to reduce the velocity below what will not erode the fine alluvial soil, so as to attain a state of equilibrium, by increasing its length and thus reducing the slope of water surface. The statement that a river tries to find its own length is also based on this belief.

There is no doubt that erosion of river banks, with consequent formation of bends and taking a tortuous course, increases the length of the channel, reduces the slope of water surface, progressively reduces the velocity of flow to a limit which will not erode the fine alluvial soil. But in attaining this stage, with the reduction in velocity the discharging capacity of the river is also considerably reduced. For example, the maximum velocity at high water level, even in a small river, is as high as six feet per second. This velocity has to be reduced to as low as six inches per second to prevent erosion of the fine

alluvial soil of river banks. But the reduction in velocity of flow from six feet to six inches per second also reduces the discharging capacity of the river to $1/12$ th. Consequently the balance of $11/12$ ths of the discharge cannot be carried by the channel, and there is overtopping of river banks—water flows over the country side and there are flood damages.

But the question is : Is it at all necessary to reduce the velocity of flow in a river to prevent erosion of the fine alluvial soil of river banks ? It is noticed that there is no erosion when a rapid current flows alongside of, and parallel to an erodible bank. This is explained by the fact that there is little frictional resistance between the river bank and the water flowing parallel to it. It is only when the fluid filaments impinge obliquely on a river bank and the direction of flow is changed after the impact that erosion of banks takes place the action being governed by Newton's Third Law of Motion. The force required to change the direction of motion of the fluid filaments is derived from the frictional resistance between the water and the river bank, which is equal and opposite to the force of impact of the fluid filaments on the river bank. This frictional resistance causes the surface of the bank to be eroded. But the caving in of the banks is caused by the fact that the deflected current leaves a patch of dead water between it and the bank, which is turned by it into an eddy ; this scours deep causing the bank slopes to slide into it. The erosion of bank and formation of bends is, therefore, due to the oblique impingement of the running water on the river banks.

But why does a current of water, which normally according to Newton's First Law of Motion should move in a straight line like any other moving body, deflect and impinge obliquely on the river bank ? An observation of flow in a river will show that obstructions in the river bed, such as stranded tree or a protruding bank of clay or even a shoal, deflects the current, causing it to strike the river bank. The power of erosion of running water varies as the square of its velocity. A higher velocity with its greatly increased power of erosion, is thus able to erode and remove such obstructions in the river bed, thus restoring the current to flow " in its state of uniform motion in a straight line ". This explains the fact why a river channel having a higher velocity, as in the steeply sloping plains below the foot hills such as in the upper reaches of the Manas—Beki, has a straighter channel. Even the tortuous dry season channel of an alluvial river assumes a straighter course during high water level in the rainy season owing to the higher velocity of flow.

The higher velocity erodes the river bed and deepens the channel. A deeper and straighter channel has a higher hydraulic mean depth and a greater slope of water surface, conditions which lead to higher velocity of flow and consequently a higher discharging capacity. The river, under such conditions, can accommodate the maximum discharge within its channel. There is then no overtopping of banks, no flow of water across the river side lands and no flood damages. The higher velocity in the river erodes and removes all obstruction from its bed and flows parallel to the river banks. There is thus no oblique impingement of the fluid filaments on river banks, no erosion and no formation of bends. Such a condition of flow enables the river to maintain its channel in a stable condition and accommodate and drain the maximum discharge within its channel.

The rivers in this country have been allowed to assume any alignment, having any width, depth and sectional area, in the belief that by so doing, the natural flow of water is not being interfered with. This has gone on for hundreds of years, but has it helped the rivers to attain a stable condition? On the contrary, due to the tortuous course assumed and consequent progressive reduction in the velocity of flow, there has been progressive diminution in the discharging capacity, as a consequence of which flooding of river side lands have increased in intensity and frequency. Not only do the inhabitants suffer from such floods, but the Government has to spend enormous sums of money in repairs to damage roads, bridges *etc.*, as well as in giving relief to the inhabitants.

Embankments along river banks may prevent flood waters from flowing across the country. But as is well known such marginal embankments are not a permanent remedy. What is necessary is to increase the discharging capacity of the river channel, so that the maximum discharge can be accommodated in the channel and drained through it. This can be effected by increasing the velocity of flow. As already explained a higher velocity of flow with its higher power of erosion erodes and maintains a deep channel, and by removing all obstruction from river bed prevents deflection of the current and oblique impingement on river banks. A state of uniform motion in a straight line is thus maintained. Any change in the direction of flow should follow a flat curve, the radius of the curve depending on the width of the flowing channel and the velocity of flow. To ensure a uniform velocity, the sectional area of the channel must remain uniform and for this the width and depth must remain uniform. For rivers with a sluggish flow, having well defined channels with tortuous alignments, it will be necessary to have diversion channels which will ensure an alignment for each river with straights joined by flat curves. For the live rivers, which usually have eroded very wide beds only a fraction of which is occupied by the flowing channel, it will be necessary to train the flowing channel to an alignment with straights joined by flat curves.

DISCUSSION BY THE RESEARCH COMMITTEE

MR. GHOTANKAR introduced item (1) and said that the Mahanadi river near Cuttack had four branches in addition to two spills and those branches which were on the convex side of curvature, drew excess sand. In this case, the Katjuri below Naraj weir was one of the excess sand-drawing offtakes. To arrest deterioration, therefore, either

- (a) its sand carrying capacity should be increased ; or
 - (b) less sand must be admitted into the Katjuri at head by reducing the waterway of the Naraj weir ; or
 - (c) combination of both (a) and (b).
- (a) could be achieved by shutting off the spills but this was not permissible because it caused an afflux of $4\frac{1}{2}$ feet.

It was also considered that closing the Banki Road spill entirely was not desirable because occasional flooding of the Puri district where the Banki Road spill waters find their way is considered essential and so experiments were

carried out with either spill reduced partially, singly or both together, and also with Naraj waterway reduced so as to off-set the afflux caused by stopping the spills partially.

The anyalsis of the experimental data showed that satisfactory conditions would be obtained by reducing the Banki Road spill discharge by one half *i.e.* to 50,000 cusecs and Naraj waterway from its right and by 15·5 per cent.

In addition to the reduction of the Banki Road spill to 50,000 cusecs, closing the Birnasi spill by an embankment ending in a spur had its advantage in increasing the carrying capacity of the Katjuri river and scouring deep channel towards Cuttack ; yet it was harmful to the Mahanadi which had its relatively clear water supply of Birnasi spill cut off. Probably the optimum solution might be to allow a reduced discharge through the Birnasi gap, with the Banki Road spill reduced to 50,000 cusecs and the Naraj weir length suitably reduced to counteract afflux caused by reducing these spills. This was being tested.

The experiments were carried out under existing conditions of sand load and discharge distribution above Naraj. In the event of the proposed dams and barrages across the Mahanadi, the proposals would have to be suitably modified after further experimentation to conform to the changed distribution of sand and discharge.

MR. GOVINDA RAO introduced item (2) and said that flood absorbing capacity of a reservoir is the sum of discharges from the outlets and spills together with the discharges bottled up in the reservoir between full reservoir level and maximum water level. It not only depended on the maximum rate of rise of flood but also on the duration. For the flood absorbing capacity of reservoir, four factors were taken into consideration ;

- (1) depth in feet between full reservoir level and maximum water level
- (2) capacity between full reservoir and maximum water level.
- (3) discharge capacity and
- (4) rate of rising of floods.

Equations had been worked out on the basis of these factors, but it appeared that these were not sufficient. The relation between the duration of flood and its intensity had to be worked out. This was very important.

Record of floods occurring in Krishnarajasagar for the past thirty years had been maintained. It showed that the maximum rate of rise of flood had never exceeded 16,000 cusecs in three hours. The sluices in the dam had on the aggregate discharging capacity of 370,100 cusecs at full reservoir level. The flood absorbing capacity of the reservoir could be any where between 370,100 cusecs to 802,317 cusecs depending on the rate of rise and duration of flood. Hence the importance arose of taking into consideration the duration of peak floods in addition to the other factors usually taken in such calculations.

RAO BAHADÜR MAYDEO introduced item (3). He said that this was only a report giving the areas affected, the extent of damage, the amount of flood discharge and the possibilities of control by constructing some tanks for flood absorption.

Item (4) by Mr. Ray was taken as introduced in the absence of the author.

MR. FRAMJI commenting on Mr. Ray's paper said that the author was right in decrying the reduction of slope and velocity of a river as means of arresting meandering, though not for the reasons he had given; not so much because it would reduce the discharging capacity of the river, with adverse consequences, but because there would result *more*, not less meandering. As experiments in Poona have shown^(*) the primary cause of meandering was insufficient (not excessive) slope—*i.e.* insufficient energy—for the river to transport a given sand charge with a given discharge so that a part of the charge was thrown down on one bank and curvature develops.

On the other hand, if the slope was in excess of that required to carry the charge with the discharge available, scour occurred and this scour increased the induced charge which then became excessive for the slope and secondary meandering occurred further downstream.

Nor was it more than wishful thinking to expect to attain and maintain equilibrium conditions where the charge was just what the channel could carry with the slope and discharge available, preventing sub-charge on the one hand and super-charge on the other, not only because the charge and discharge in a river were always changing and fluctuating, but because scour due to inequilibrium was always accompanied by a rising bed; so that although there might have been a point or a short reach where the charge could just be carried, this did not persist along the river but conditions passed from a sub-equilibrium charge for a deeper channel to a super-charge for a shallow channel. This explained why all rivers tend to meander, even incised rivers.

Hence, increasing the discharging capacity of a river, though useful for flood control, if and where it could be done, would not solve the problem of meandering. Nor was it clear how precisely Mr. Ray proposed to train the flowing channel of a live river to an alignment with straights joined by flat curves. Would he design the curves for maximum or dominant formative discharge; and what width would he allow between the straights? A suitable width and curvature for one magnitude of discharge and charge would not be suitable for another and rivers unfortunately did not adhere to any constant or predictable discharge and charge.

MR. FRAMJI presume that the diversion channels—Mr. Ray's panacea for sluggish rivers—meant excavation of entirely new courses. In addition to the same difficulties of design as for the remodelling of live rivers, there would be the question of suitable alignments.

Had Mr. Ray considered the practicability of his suggestions in respect of expense and balanced the likely gain against the outlay?

MR. CHATURVEDI invited a reference to a problem in hand at the United Provinces Research Station concerning the training of river Rapti in Balrampur Estate. That problem immediately went to show that the arguments introduced by Mr. Ray with a view to get more slopes were very fallacious. A $\frac{3}{4}$ mile long short-out made in the river to eliminate loop of about three miles resulted in the river ascerting fresh meanders. The cut was 20 feet wide and now it has become about a thousand feet wide and takes almost the entire supplies during the floods. Pumping Stations for two lift canals downstream

(*) Indian Waterways Experiment Station, Poona, Annual Report (Technical) 1942-43, page 59.

of this cut are now facing trouble—one being left high and dry and the other in the danger of being outflanked. It is thus not very desirable short circuiting these meanders haphazardly.

In reply to the Chairman (R. B. Venkatacharya) MR. CHATURVEDI said that the old course was silted up and was functioning only as a high level channel.

DR. UPPAL asked why the cut-off could not be stabilized by an Island.

MR. FRAMJI pointed out that experience on the Mississippi and the Indus had also shown that cut-offs were often followed by fresh meanders, particularly on the upstream side and stabilizing cut-off was not a simple matter.

(ii) RIVER TRAINING WORKS

The following items were discussed at the 1947 Research Committee Meeting:—

- (1) Model experiments for the prevention of erosion at Kurigram by the river Dharla.
- (2) Erosion of the river Torsa.
- (3) Model investigations for the proposed Mor Barrage at Tilpara.
- (4) Training the River Luni at Railway Bridge No. 3 of Samdari-Raniwara Branch of the Jodhpur Railway.
- (5) Training of the Yamuna River at Delhi Gate Pump House by means of a pitched embankment.
- (6) Training of the Watrak River at Kaira (Bombay Presidency).
- (7) Training of the Sutlej River at Samasata, about six miles downstream of the Empress Bridge at Adamwahan North Western Railway.
- (8) The Rupnarain river upstream of the goods yard, Kolaghat Railway Station upstream of the Bengal Nagpur Railway bridge.
- (9) Training of rivers with permeable tree groynes.
- (10) Beas River—Railway Bridge at Dhilwan in District Jullundur.
- (11) Ravi River—Spur at Baherian—District Gurdaspur.
- (12) Rivers Chenab and Jhelum at Trimmu Headworks.
- (13) River Sutlej at Samasata Railway Station.
- (14) Shape of spur apron for conventional types of spurs.
- (15) Effect of the angle of approach on the shape of the apron of a T-headed spur.

(1) MODEL EXPERIMENTS FOR THE PREVENTION OF EROSION AT KURIGRAM BY THE RIVER DHARLA (9)

SUMMARY:

Model experiments to find out permanent remedy to stop the erosion were started. Experiments described show that the model simulated the prototype conditions faithfully.

(2) EROSION OF THE RIVER TORSA (10)

SUMMARY

Describes a study of the behaviour of the River Torsa near about the town of Cooch Bihar. Makes suggestions for the collection of data for a detail study of models.

(3) MODEL EXPERIMENTS FOR THE PROPOSED MOR BARRAGE AT TILPARA (11)

SUMMARY

It has recently been decided to raise the whole barrage structure by two feet and reduce the total waterway of the barrage by 60 feet by eliminating one weir bay of 60 feet. It has also been pointed out that the barrage will be designed for a maximum river discharge of 300,000 cusecs. The total waterway of the barrage will be eight bays of 60 feet each in the two undersluices and seven bays of 60 feet each in the weir portion in the middle. With the barrage as previously designed the total energy level for a river discharge of 300,000 cusecs was expected to be 212.0 feet (R.L.), with 345 cusecs per foot run over the undersluice section and 281 cusecs per foot run over the weir section. This has been calculated from the curves of discharge co-efficient obtained from the results of experiments on two models. The result of the change in design will be to raise this total energy level to 215.0 feet (R.L.)

The effect of raising the whole structure by two feet on the scour produced downstream of the structure has been investigated by model experiments on the undersluice section. The results are :—

1. For discharge of 300 cusecs per foot run, the upstream water-level is raised from 208.0 feet to 210.0 feet. For the same downstream water-level in the river, therefore, the afflux increases by two feet. The expected downstream water-level for this discharge after retrogression is 206.0 feet, so that the expected afflux is four feet. The structure may have to be designed for this afflux, if the retrogression is accepted as a probability. This increase of afflux, however, will not increase the maximum depth of scour downstream when there are staggered friction blocks on the downstream floor. The depth of sheet pile, therefore, need only be increased by two feet, the amount by which the whole structure is raised.

(9) Annual Report of the River Research Institute, Bengal 1946, pages 6 to 9.

(10) Annual Report of the River Research Institute, Bengal, 1946, pages 23 to 26.

(11) Annual Report of the River Research Institute, Bengal, 1946, pages 27 to 39.

2. For discharge of 200 cusecs per foot run, the upstream water-level was maintained at 207·0 feet by the application of the gates in the previous design. When the structure is raised by two feet, the upstream water-level obtained without application of gates is 206·5 feet, so that the gates will have to be just lowered to raise this level to 207·0 feet. The effect of this change on the standing wave and scour produced downstream when friction block system (B)* is present, is shown in Table 2C. 11.

TABLE 2C.11

Mor Barrage—Undersluice Section—Model scale $\frac{1}{16}$

Discharge=200 cusecs per foot run ; Upstream water-level maintained by gate=207·00

Friction block system " B " on the downstream floor

Down-stream water level R.L.	Nature of standing wave		Max. depth of scour in feet below the downstream floor level at distance (mentioned) from the end of the pucca floor)	
	Original design	Whole structure raised by two feet	Original design	Whole structure raised by two feet.
202·00 ..	Not observed ..	Forming on the glacis at 19 feet from the crest	Not observed	13·0 at 136
200·00 ..	Forming on the glacis at 20 feet from the crest	Shooting away ..	11·3 at 128	21·2 at 176
198·00 ..	Shooting away ..	Shooting away ..	22·7 at 208	16·4 at 144

Table 2C.11 shows that for the same downstream water-level, the standing wave is pushed downstream, if the structure is raised by two feet. The maximum depth of scour is not appreciably affected.

3. For discharge of 100 cusecs per foot run, the upstream level is to be maintained at 207·0 feet by application of gates. It has been found that with downstream water-level at 195·0 feet, the standing wave formed on the glacis at 18 feet from the centre of the crest in the original design, while the standing wave shoots away when the structure is raised by two feet. The effect on scour produced downstream is not appreciable, if there are friction blocks on the downstream floor.

These results show that so far as the scour downstream is concerned, raising the structure by two feet will only require increasing the depth of sheet pile by two feet.

* Two rows of blocks at the of glacis and two at the end of pucca flow, size of blocks being 4 feet × 3 feet × 3·5 feet high.

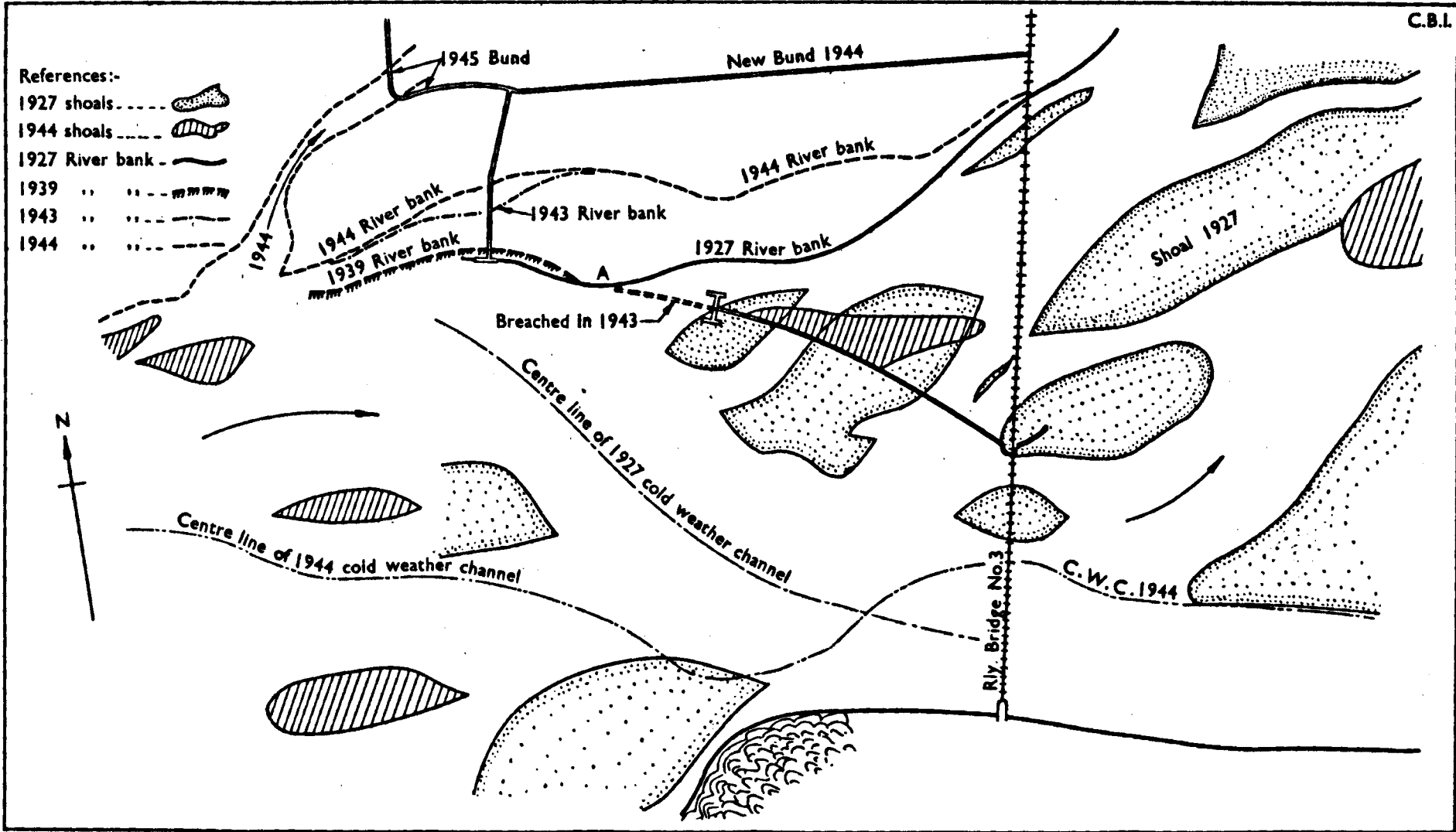


Figure 2 C.11:- Luni River at Railway bridge No. 3

Scale 600 300 0 600 1,200 Feet

(4) TRAINING THE RIVER LUNI AT RAILWAY BRIDGE NO. 3 OF THE
SAMDARI—RANIWARA BRANCH OF THE JODHPUR RAILWAY ⁽¹²⁾

SUMMARY

Experiments were carried out with the object of devising measures to prevent the occurrence of trouble due to floods such as those experienced in 1939 and 1944 by reducing the afflux and centralising and distributing the flow evenly in all spans of the Railway bridge. Increased water way with a flared guide bank gave promising results. The experiments also showed that the alignment of the left guide bank had an important bearing on the prevention of dangerous scour on the left abutment and on the flow conditions at the bridge.

Introduction :—

The average width and depth of the river near the Railway bridge, in flood, are 3,800 feet and eight feet respectively, and its slope is 1 in 1,500. The bridge is 1,295 feet in length and has a clear waterway of 1,160 feet with 29 piers ; while before the construction of the bridge, in 1927, the river width at the bridge site was 3,100 feet. Thus, the present clear waterway is only 0·374 times the normal width of the river, and is only 0·856 times the Lacey's minimum stable width of waterway [based on the estimated maximum flood discharge of 1944 = 261,000 cusecs].

When the bridge was constructed, the high bank line was continuous up to A, shown in Figure 2C.11 ; a concave guide *bund* was constructed from this point to the left abutment for prevention of flooding of the low level area behind the bund. Since then the river has been gradually eroding this bank.

In 1939, high floods occurred in the river, the maximum recorded water level at the bridge being R. L. 443·5. The guide *bund* breached at the head ; and another breach occurred in the railway bank. Before the floods of 1944, this was repaired and the head of the damaged *bund* was strengthened by building a *T* head. A new retired *bund* was also constructed for checking the tendency of developing new channels.

During the flood of 1944—the maximum recorded since 1917—the water level rose to 447·2, *i.e.* up to the underside of the end spans ; the flow was obstructed in the middle portion, where the R. L. of the underside of the girders was 446·08. Large breaches resulted in the guide bunds and a 612 feet length of the railway embankment was severely damaged. Heavy damage was also caused to the three end piers on the left bank, one of which collapsed altogether.

Experiments :

Preliminary experiments ⁽¹³⁾ were carried out to reproduce existing conditions. These experiments were continued during the year to obtain optimum flow conditions at the Railway bridge.

⁽¹²⁾ Indian Water-ways Experiment Station Annual Report (Technical) 1946, pages 52-60.

⁽¹³⁾ Indian Waterways Experiment Station, Poona, Annual Report (Technical) 1945.

A series of experiments were conducted when various schemes and devices were tried. These provided information regarding the inadequacy of artificial, pitched "attractive" islands in training flashy rivers like the Luni, though such islands are recommended⁽¹⁴⁾ under more favourable circumstances. As a result of a long series of experiments⁽¹⁵⁾, it was concluded that

(i) new spans providing about 300 feet of additional waterway should be constructed on the left side.

(ii) consideration of the safety of the piers dictates that the new piers should be carried down either to hard stratum or at least to R. L. 397.00.

(iii) new guide banks, upstream and downstream of the bridge, on the left bank should be constructed. Suitable alignments for these are shown in Figure 2C-12. Two alternative alignments of the guide bank are proposed. The upstream guide bank should preferably be carried up to the retired bund and pegged into it, thus protecting the left approach bank and the New Bund (part) against any possible attack by floods; or it may be ended (as shown in Figure 2C-12.) at an angle of 120°, if this is found to be substantially more economical.

The above measures, it is hoped, will considerably improve the present conditions at the bridge and lessen the action all along both the banks.

(5) TRAINING OF THE JUMNA RIVER AT DELHI GATE PUMP HOUSE BY MEANS OF A PITCHED EMBANKMENT⁽¹⁶⁾

SUMMARY

As a result of the existing curvature, the low water channel of the Jumna river at Delhi Gate Pump House recedes after the floods towards the opposite left bank away from the pump house, necessitating the heavy expenditure in bunding it off and dredging the channel to feed the pump and the power house.

A pitched embankment which cut a portion of the existing down stream bank of the power house was found effective in inducing a left water channel both under existing and post-barrage conditions. This alignment was unsuitable to Delhi Gate Power House authorities as it cut from their land, so another alignment further inside the river has to be evolved from model tests.

After the tests, an alignment of the embankment was evolved such as to be effective under existing conditions and also with the barrage.

(6) TRAINING OF THE WATRAK RIVER AT KAIRA (BOMBAY PRESIDENCY)⁽¹⁷⁾

SUMMARY

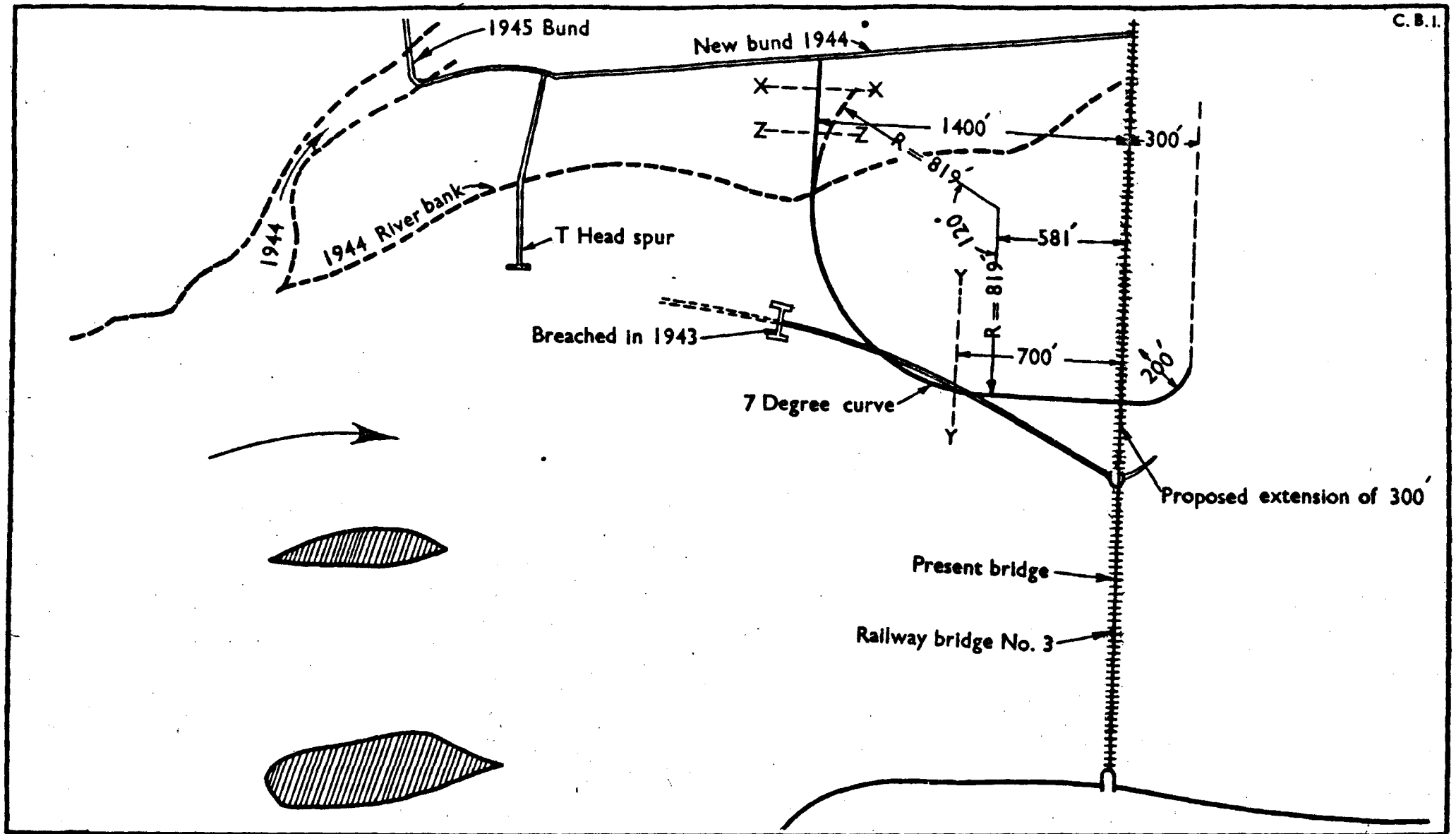
Severe erosion has been going on left bank upstream of the low Fort Wall of the town of Kaira and part of the road running left bank was washed away.

⁽¹⁴⁾ Indian Waterways Experiment Station, Poona, Annual Report (Technical) 1944, page 87.

⁽¹⁵⁾ Indian Waterways Experiment Station, Poona, Annual Report (Technical) 1945, pages 52—60.

⁽¹⁶⁾ Indian Waterways Experiment Station, Poona, Annual Report (Technical) 1946, pages 60—64.

⁽¹⁷⁾ Indian Waterways Experiment Station, Poona, Annual Report (Technical) 1945, pages 64—67.



C. B. I.

Figure:- 2C.12:- Showing present Railway bridge over Luni & the alterations recommended

In view of the urgency of the problem, temporary solution for training the river with tree groynes was recommended as follows :—

TREE GROYNES

Each tree groyne should be 200 feet long and originally laid to face some 60° upstream with reference to the bank ; so that when the groyne launches and becomes sand-bound it assumes a position still facing slightly upstream, say 70° to the bank. Unlike an impermeable groyne which is generally made to face 60° upstream, a permeable groyne should not face so much upstream since it would collect floating debris against its face ; converting it to an almost impermeable type, with attendant disadvantages in this case. After launching, it should also not, however, face downstream lest it " attracts " the flow to the bank upstream of it, and only induces siltation below itself. The suggested initial alignment and position of the three tree groynes is shown in Figure 2C-13. They are approximately 300 feet distant—a distance which should be easily protected by 200 feet long groynes, as a groyne generally protects two to three times its length.

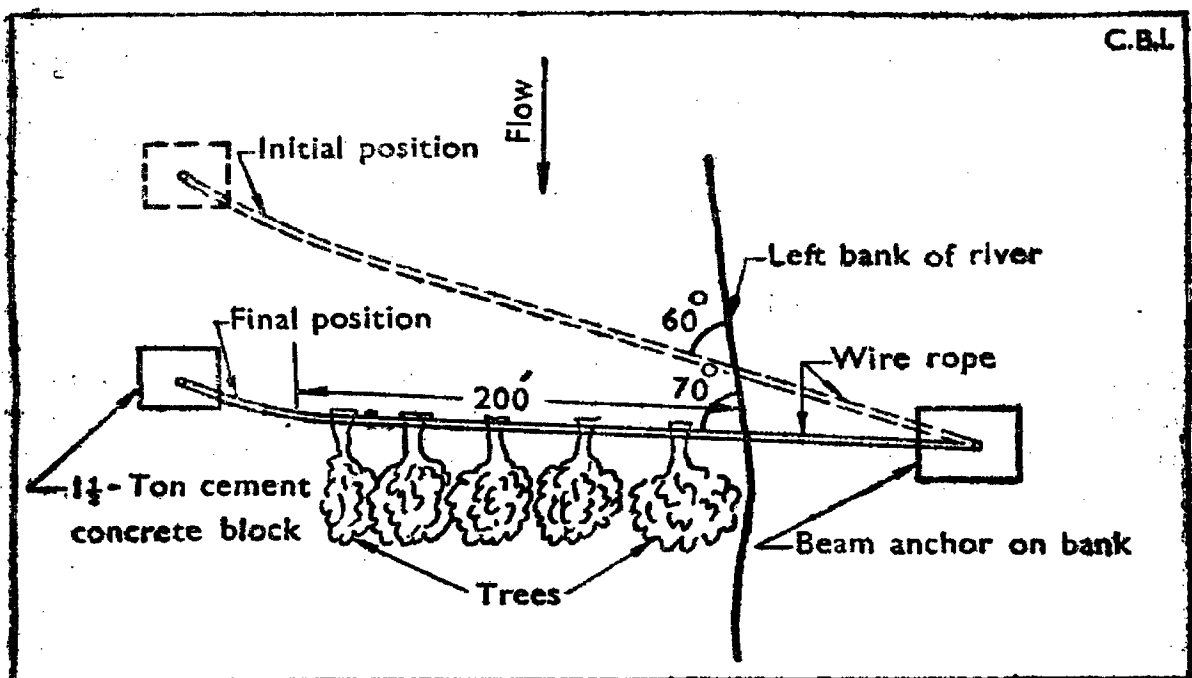


Figure 2C-13 : Sketch of permeable tree groyne.

Each tree groyne should consist of a thick wire rope ($\frac{3}{4}$ inch diameter) very firmly anchored at one (*fixed*) end to the bank to a sufficiently strong girder or beam or block of concrete, deeply embedded into hard ground ; and tied at the other, *free* end to a heavy buoy or concrete block (about $1\frac{1}{2}$ tons in this case). The length of the main wire rope will be the length of the groyne, allowing extra for the sag which will occur due to the weight of the sand-bound trees ; approximately one tree (20 to 25 feet long) per running foot of groyne should be provided. In construction the free end of the wire rope attached to the buoy would be dropped into the river somewhat upstream of the position it is finally

to take ; then from the bank the wire rope is pulled taut by a winch and anchored to the fixed end. This end has ultimately a tremendous strain on it and must be strong enough to take the full load. The work of fixing trees must then begin from the shore end, proceeding *outwards* to the free end and *upwards* from the bed to above water level. Whole leafy trees with abundant branches, such as will not quickly rot in water (*e.g. babuls, "acacia arabica"*), are required for the purpose ; it is the root, branches and twigs, not the stem or the leaves, that help to trap and reinforce the sedimentation, by retarding the velocity of flow and intercepting the sand and silt. About a foot up the stem, a hole is drilled through each tree, through which an iron ring is drawn and attached either directly or by subsidiary pendant wire ropes to the main rope. The trees soon sink when they become heavy with detritus. The trees should be packed as thickly as possible both close to each other and over each other from the bed to above water level.

The tree groynes should be completed before the early freshet floods in the river ; as it is during these freshet-floods transporting a super-charge of bed-sand and debris that the groynes get earth-bound and best serve to deflect the high velocity flow away from the bank. Such tree groynes constructed in Sind under very arduous conditions have been successful. Hardly had the groynes been completed when there was heavy sedimentation, both upstream and downstream of them, completely embedding the trees ; so that they were later fully reinforced under extremely severe conditions not merely to withstand erosion of a severe flood, but to deflect the current away from the bank to be protected.

TIMBER PILE GROYNES

Main verticals.—Country timber piles, eight inches minimum diameter, should be embedded a minimum of eight feet (and preferably ten feet or more) with top of half the length of groyne at R. L. 80.0. This top level is suggested with a view to restricting the required pile lengths to 25 feet without reducing the penetrations ; with longer piles being available there should be one foot extra penetration to every two feet increase of height ; and with shorter piles or larger depths the top level may be lowered, decreasing the efficiency, but penetration must *not* be reduced. For ease of driving it is suggested that the piles should be tapered at the base and, if possible, shoed with 30 lb. iron shoe. As the bed along the high bank is probably nearly dry ; driving will present little difficulty.

Intermediates, transverses and struts.—There will be two rows of verticals, spaced not more than four feet apart in each row, the rows being also four feet apart. In between two main verticals there will be two intermediates (two to four inches in diameter) embedded not less than four feet below bed. Each row should be either closely intertwined by "*lai*" brushwood branches going in and out around each vertical, or horizontal wallings, four to six inches in diameter rafters spaced an equal distance apart, should be nailed or screwed on the main vertical, or an expanded metal mesh fixed to the upstream row. The upstream row should be braced back to the downstream row by both transverses and diagonals, which should be of rafters four to six inches in diameter. Every other

main vertical of the rear row should be strutted, the strut being embedded a minimum of eight feet below bed (measured vertically) and squarely clamped and bolted to the main vertical, Figure 2 C-14.

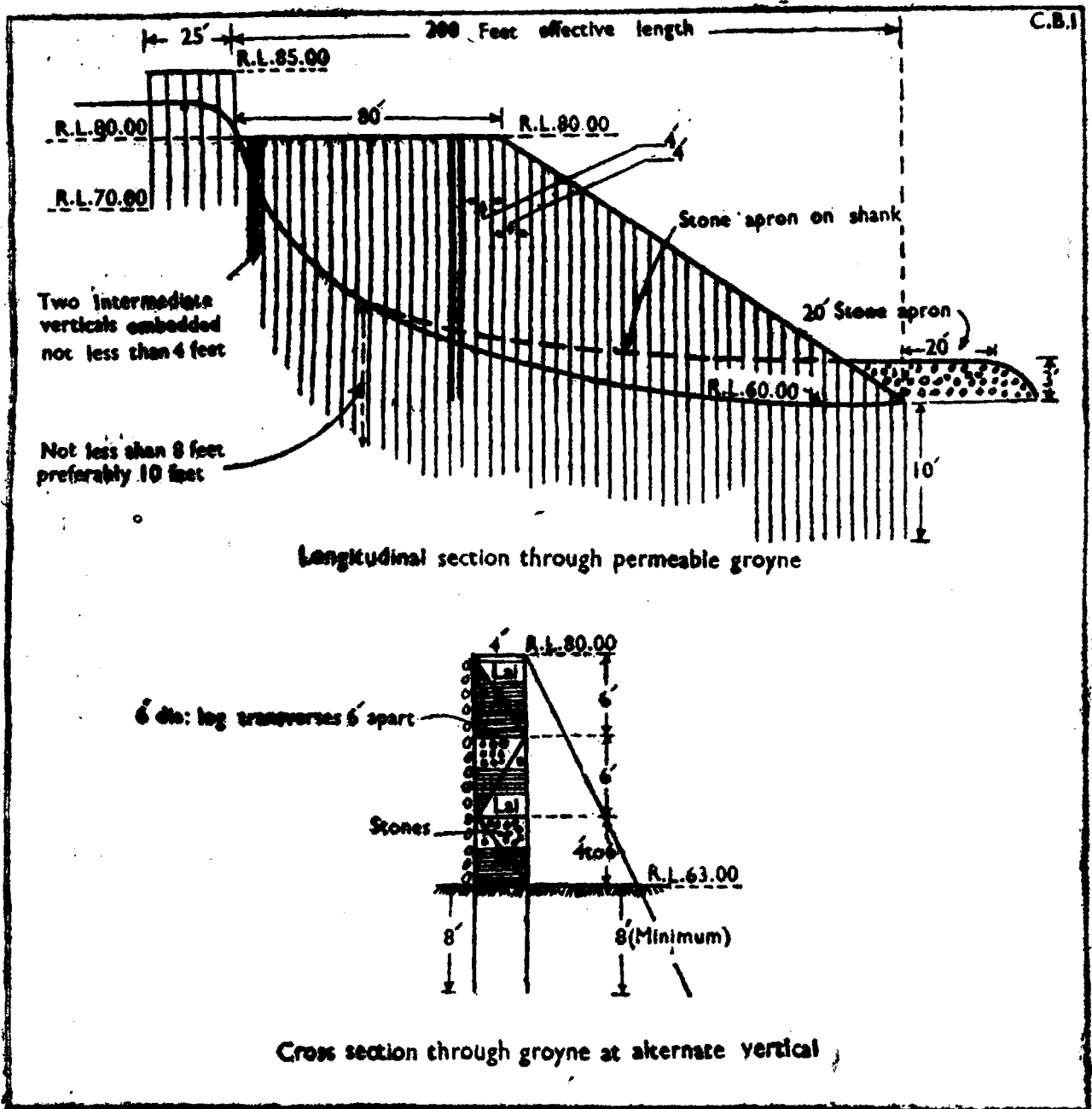


Figure 2C.14 : Timber pile groyne.

Filling.—Between the two rows there will be filling of brushwood, preferably "lai" branches, closely packed and tamped. Preferably the entire quantity of filling should be "lai" brushwood ; but if sufficient and suitable "lai" for tight packing of all the three groynes is not available, the filling should comprise alternate layers of six feet thick brushwood weighted down by two feet thick rubble stone (one-man-stone). The whole filling should not be of stone since the groyne is intended to be, to start with a permeable, not a semi-permeable or impermeable groyne.

Mattress or apron.—When properly designed, constructed and functioning as a permeable groyne, there should be no tendency to scour round the nose of the groynes or for the piles to scour out ; but because debris will collect upstream, and the groynes will even get sand-bound, the groynes become more or less impermeable in time. To protect against scour occurring under such conditions, it is desirable to protect the bed, both upstream and downstream of the groyne, and around the nose with a mattress or stone apron three feet thick for a width of ten feet along the shanks and 20 feet along the nose.

This type of groyne has the advantage that it can be built out stage by stage, but for satisfactory construction it is best built in dry or when the depths of flow are small.

**(7) TRAINING OF THE SUTLEJ RIVER AT SAMASATA, ABOUT SIX MILE.
DOWN STREAM OF THE EMPRESS BRIDGE AT ADAMWAHAN,
NORTH WESTERN RAILWAY ⁽¹⁸⁾**

SUMMARY

To check the rapid erosion of the left bank about half mile above the Samasata Station, a repelling spur was constructed before the 1945 floods season. This spur acted efficiently until it breached in the following floods of the same season where the downstream portion of the shank had not been armoured, as it should have been. After the season, a new regime of the river having by them established, it was thought expedient to abandon the breached spur and to construct a new spur on a position a little upstream of the first. The latter was however never constructed, but the old groyne was repaired instead. The repaired spur breached again in 1946 with resulting further in roads by the river towards the station yard.

After several trials, a 1,750 feet long repelling spur down stream of the existing breached one with nose pointing 70° up stream and designed to prevent gulletting on the sides gave the optimum results. An impermeable spur would have to be strongly protected both on its up stream and down stream faces and would depend for its success on its stability. Should it not be possible to complete it together with the necessary protection, and apron, it would be advisable to construct instead a permeable tree groyne as the next best solution ; failing which as a last resort, the model showed that temporary protection could be afforded by putting five short repelling tree groynes along the left bank in the bend and an attractive groyne about the entrance of the Sharabi Nala on the right bank, the latter to accelerate a cut off.

To prevent gulletting, a nose with 3 : 1 longitudinal slope for top 1/3rd depth of water and 5 to 1 for the lower 2/3rd depth and a stone apron roughly conforming the shape of scour in the model were recommended, the latter to reduce distortion in river curvature. To avert the immediate danger it was also suggested that an apron should be provided to the ring bund of the station yard.

⁽¹⁸⁾ Indian Waterways Experiment Station, Poona, Annual Report (Technical) 1946 pages 68 to 78.

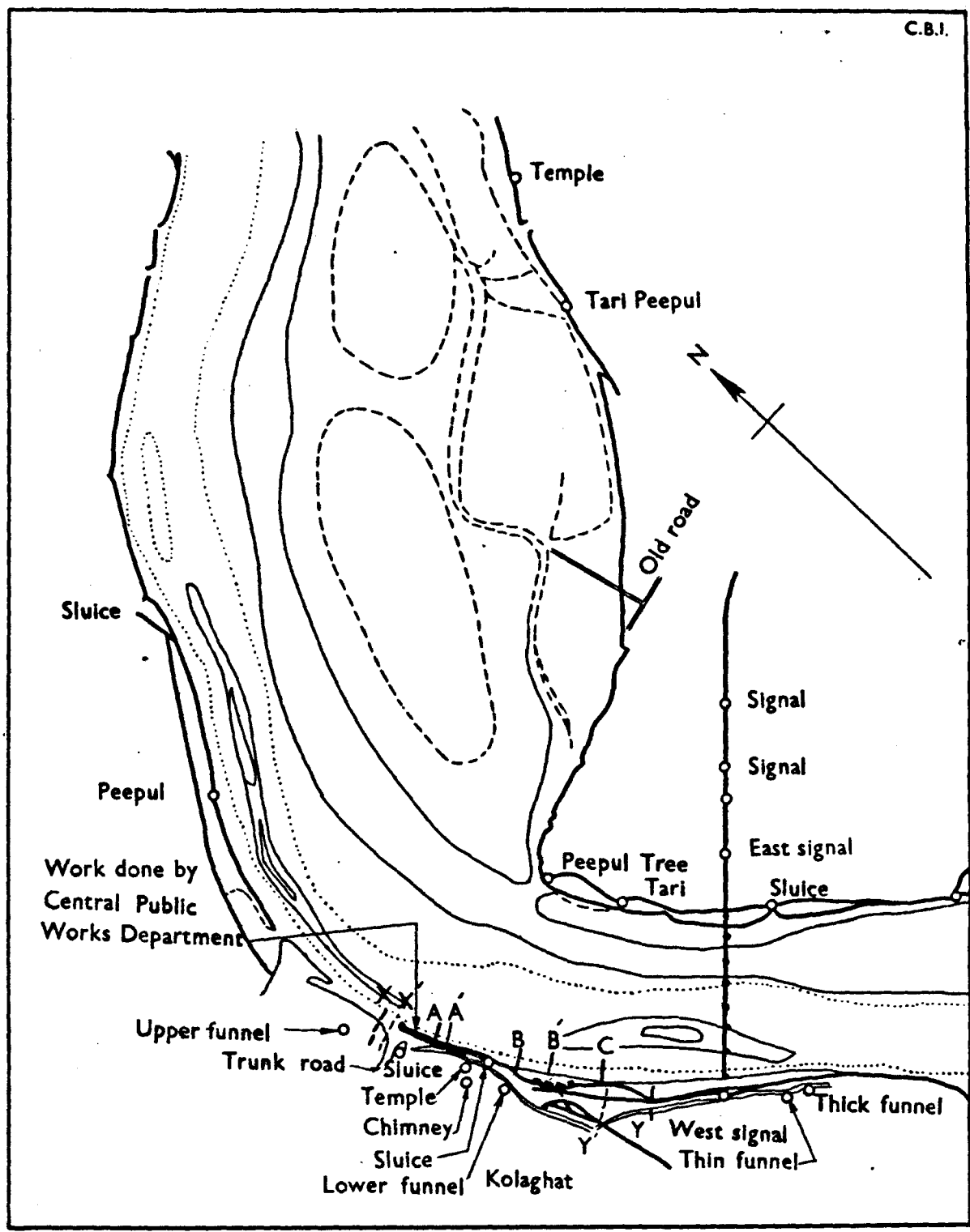


Figure:- 2C.15. Showing alternative positions of permeable tree groynes (A, B, C, or A', B',)

Rupnarain river [Surveyed 17 th to 19 th April 1946]



(8) THE RUPNARAIN RIVER UPSTREAM OF THE GOODS YARD, KOLAGHAT RAILWAY STATION (UPSTREAM OF THE BENGAL NAGPUR RAILWAY BRIDGE). (19)

Since the construction of the spur on the left bank, about 4,200 feet upstream of the sharp bend above the railway bridge, deposition of sand has taken place all along the left bank both upstream and downstream of the spur to a considerable depth. Prior to construction of the spur, this bank was subject to severe erosion; most of the flood tide now goes up the river along the right bank, in contrast to conditions of flow prior to construction of the spur, when flood tide used to split up at the sharp bund above the bridge, part going up along the left bank and part along the right bank, the right bank channel has now developed into the main channel. Absence of freshet floods of appreciable intensity for the last two years has also indirectly helped the development of the right bank channel into a main channel by retarding the desired development of the central channel; as a result the bed levels, in general, in the right bank channel have been deeper.

It was feared that due to increased depth and consequent increase of concentration of ebb flow along the right bank in the approach channel to the bridge, the pitching of the right bank, protecting the goods yard at Kolaghat might fail and measures to check the increase of concentration of flow on the right bank may become necessary.

The present conditions were studied in the pilot model (horizontal scale = $\frac{1}{3,000}$; vertical scale = $\frac{1}{300}$). The model bed was moulded to April 1946 survey and ebb and flood flows equivalent to 200,000 cusecs were run in the model. It was considered undesirable to control the river at this site by permanent spurs; only semi-permeable tree groynes were tested in the model. These groynes* have been found very effective in warding off attack in Sind; moreover they can be easily constructed in a short time, and at much lower cost.

After several trials in the model, it was found that two or more effectively three, such tree groynes, extending about 300 feet in the river, at positions shown in the Figure 2C-15, would reduce the attack, if any, and protect the river bank between XY—chainage 1,500 feet to 4,500 feet from the Bridge.

The local officers, however, after further examination of the right bank, considered that the danger is not immediate; and so have not adopted the recommended design; but it is advisable to keep the arrangements ready so that the groynes could be launched without delay in case of further development.

(19) Indian Waterways Experiment Station, Poona, Annual Report (Technical), 1946 page 78.

* For details of construction and launching of groynes please see pages 500—506 of this Report.

(9) TRAINING OF RIVERS WITH PERMEABLE TREE GROYNES ⁽²⁰⁾**ABSTRACT**

Tree groynes have been very successfully used in Sind for river training under suitable conditions.

Impermeable groynes—constructed either wholly of stone or in sand with stone armoured shanks and nose—may either be “repelling” or “attracting” according to their design and position and their most effective position can, with our present knowledge, only be entirely satisfactorily determined by model tests. They are more or less of a permanent nature and may produce far-reaching effects. It is also absolutely essential to provide sufficient protection to the nose and shanks of these groynes—not only upstream but also downstream of the spurs : otherwise there is a danger of their being out flanked. In urgent cases it may thus not be possible to collect in time the materials required for impermeable groynes ; in other cases, impermeable groynes may have secondary results which must be avoided or which cannot be sufficiently clearly foreseen ; in all such cases tree groynes would be very useful.

It is often advisable to decide the position, alignment and optimum lengths of long tree groynes by model tests—much the same as for impermeable spurs but the disposition of short tree groynes can generally be fixed by inspection of plans. The most suitable tree for these groynes is *Babul* (*acacia arabica*). The thorns, twigs and branches of this tree catch the sand moving along the bed and also trap material in suspension ; after the first freshets, the permeable groyne approximates to an impermeable groyne, if properly located, and is then most effective in protecting the banks.

Tree groynes are permeable spurs and are built for the purpose of training the river by causing accretion behind, or in front of, the line of the groynes. There are three rather distinct purposes for which more or less different types of tree groynes have been used in Sind with varying success :—

Type (a) : to divert or deflect the current which is directly threatening an erosion of the bund ;

Type (b) : to deaden the flow in one bye-river near the bund in order to open out another bye-river away from the bund ;

Type (c) : to silt up a channel of the river at its source by checking the flow in it.

⁽²⁰⁾ Indian Waterways Experiment Station, Poona, Annual Report (Technical) 1946, pages 79 to 85.

Each of these types is described below :—

TYPE A.—TREE GROYNES FOR LOCAL DEFLECTION OF CURRENT

This type was constructed at mile 3/5 of Ghoraghat Bund in 1940-41 and has proved quite successful.

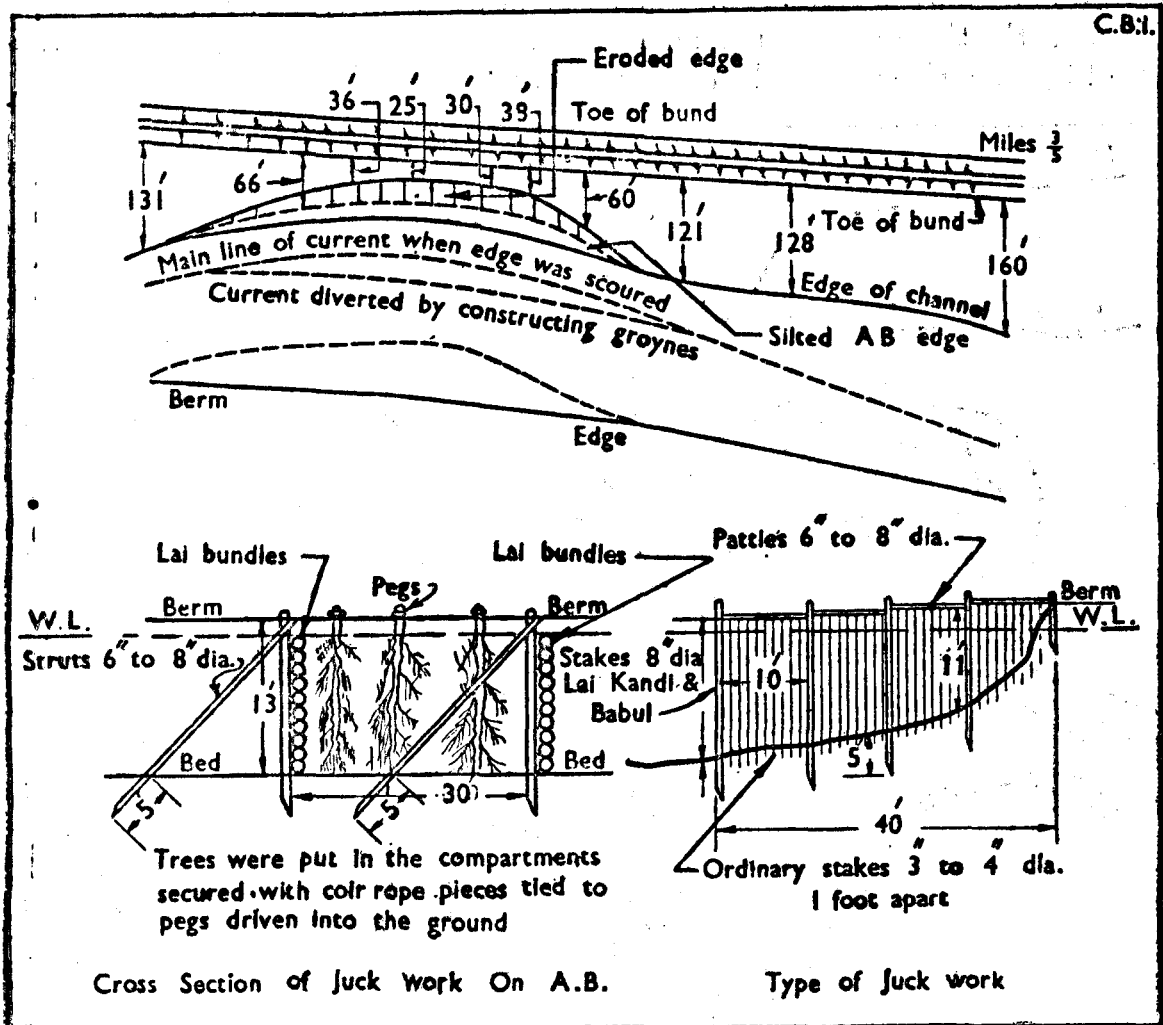


Figure 2C.16 : Tree groynes for local deflection of current Type (a)

Jucks (cross groynes) were constructed about 30 feet apart, Figure 2C-16. Each of these groynes consisted of strong vertical stakes (*munas*), driven about five feet in the grounds, at ten feet centres, held in position by strong, back diagonal stays and guy-ropes secured to strong pegs well embedded in the pucca bank. The "*munas*" (main verticals) were connected to each other by transverse pieces (*patties*) having holes drilled in them to take the tapered ends of the verticals. "*Panjars*" (intermediate verticals), one foot apart, were placed between the vertical posts. The entire juck-work was made water-tight by lining the groynes, on their upstream side, by "*lai*" bundles two feet in diameter, well tamped down by divers treading them in with their feet. The space between two such rows of groynes was thickly filled in by trees, held in position by means of coir rope pieces tied to pegs driven in the pucca ground on the bank. The depth of scour was roughly seven feet.

The result was successful : the current was deflected and a portion of the berm width which had been previously scoured was reclaimed and the edge of river held as desired.

TYPE (b)—TREE GROYNE FOR CLOSING UP ONE RIVER CHANNEL AND OPENING OUT ANOTHER

This type was constructed across the river at MN, Figure 2C-17 opposite mile 21 of the Sukkur-Begari Bund in the cold weather of 1939-40. It did not succeed in its purpose of opening out the other bye-river JK; it, however, deflected the thread of the main current outward and prevented a more rapid inroad by the river towards the river bund.

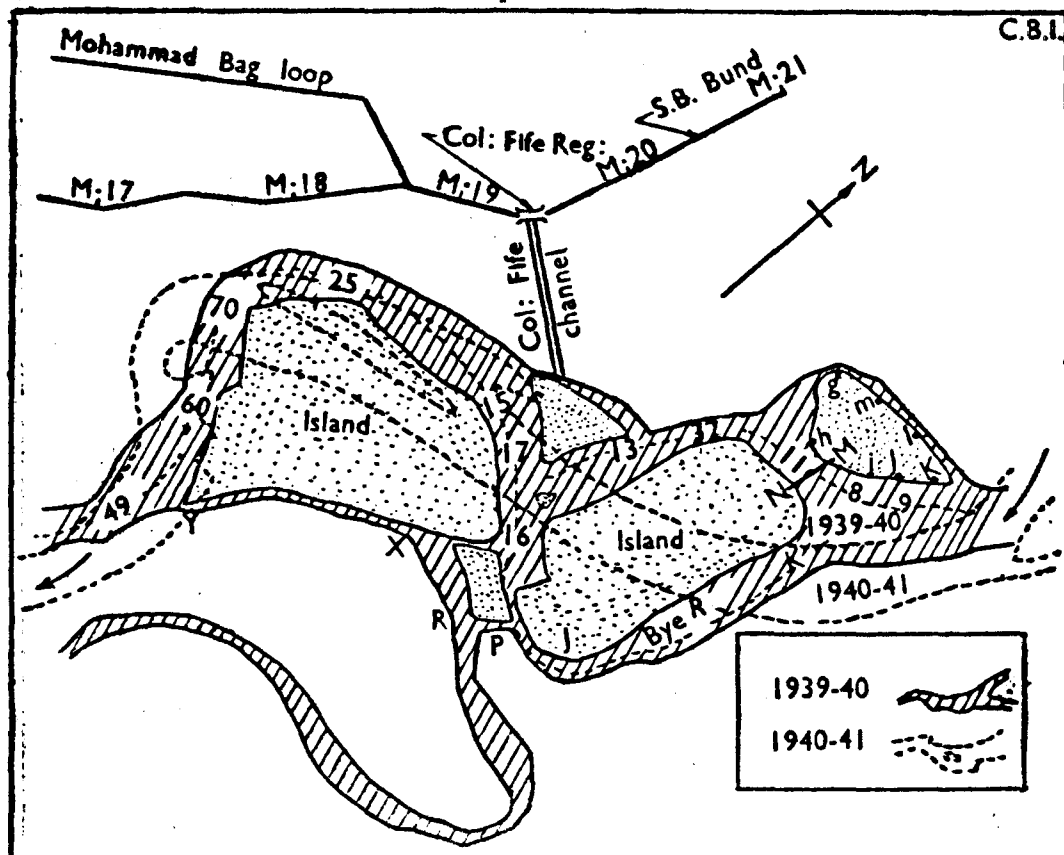


Figure 2C.17 : Tree groyne Type (b).

It also afforded useful experience of the difficulties that are to be encountered in constructing such a groyne in the "katcha" on account of difficulties of finding a suitable anchorage and of constructing the groyne, *in situ*, in the middle of a wide river bed.

The method of construction was, briefly, as follows :—

The site was selected on three considerations ; (a) the tree groyne was required to oppose the main current as near the entrance of the bye-river as possible ; (b) the reflex action should not lead to another curve below the site of the groyne ; and (c) the tree groyne was required to lead the river into one of the old river courses. Accordingly, the site MN was selected for the groyne. Inspection of the site also showed the possibility of developing the "dhor" (low river channel) XY through the course PRX, because the soil between it and the pre-1938 river course was sandy and appeared such as

would be easily scoured out. This meant deepening XR in a central 20 feet width and widening of "dhoru" RP to about 400 feet,—the widening to be done on the left bank which was in clay to admit through it a scouring discharge, which, it was hoped, would open up the "dhoru" owing to the existence of a deep "kumbh" (pit) at R.

Anchorage consisted of two concrete piles, 100 feet apart, driven in series by jetting with water under pressure in the sandy *katcha*. It was hoped to sink these piles to 40 feet depth but with the pumps available, they could only be taken to a depth of 21 feet, the tops thus being only two feet below *katcha* level. On this account, a subsidiary anchorage was provided by connecting the piles to another anchor made up of tree trunks, laid in the *pucca* as in Figure 2C-18. The trenches housing this anchor were filled up. The anchors were connected by $1\frac{1}{2}$ inches cable with trees tied on to it, at ten feet intervals, as in case of a tree groyne. It was, thus, intended to prevent deflection of the current to the island NJ.

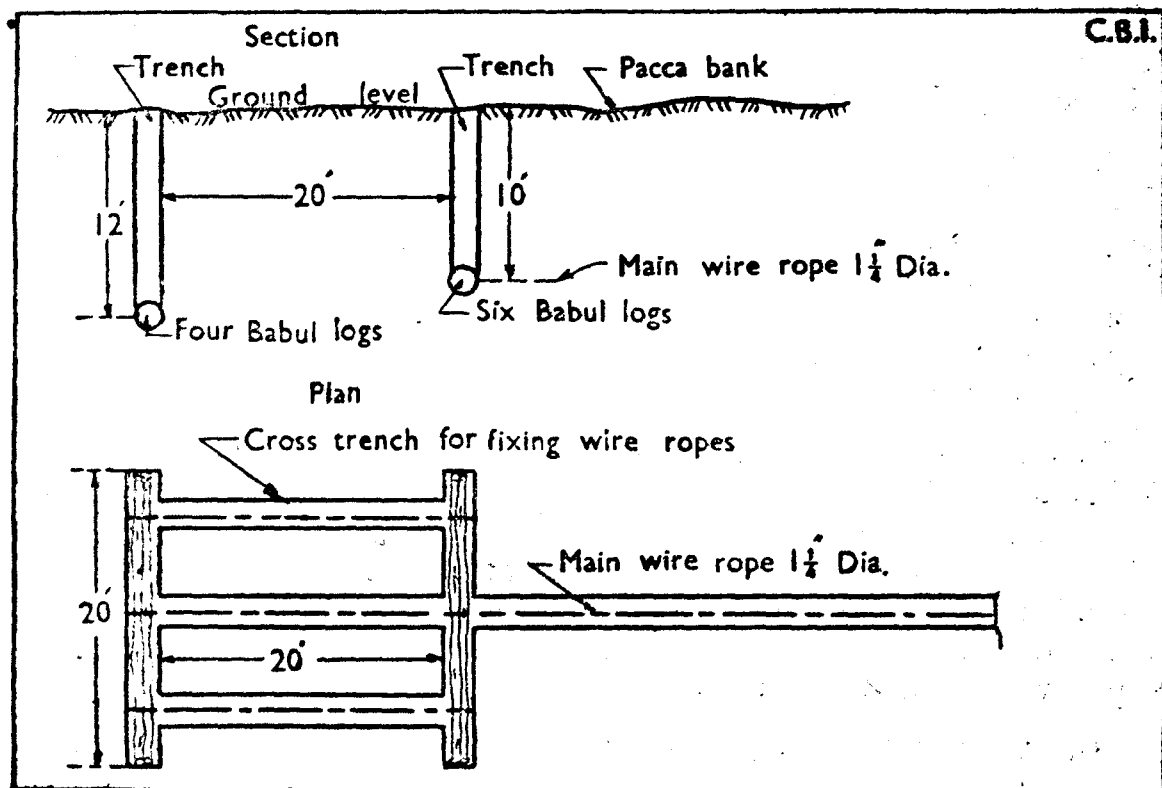


Figure 2C. 18.

The groyne was fabricated at the edge of the stream because of the difficulty of keeping the cable stable across a stream 2,000 feet wide with semi-rigid anchorage, and tying the trees *in situ*. The tree groyne was made up of whole green *babul* trees, about 20 feet to 30 feet high, spaced 5 feet apart, tied to a G. I. Cable $1\frac{1}{2}$ inches diameter, anchored at end M as described. Suitable arrangements for tying the trees to the cable by stirrups and every fifth tree by a clamp and hold-fast bolt were made. It was intended to pull the tree end upstream at 45° to the direction of the current by the aid of power craft and then allow the current to launch the groyne at roughly 60° to the direction of the

current. This was intended to be done in three stages. Actually, however, with the first pull two-thirds of the groyne launched itself more or less in the alignment desired. Efforts to pull the remaining third caused loops near its land end, which could not be disentangled. It has been estimated that this resulted in the effective length being about ten per cent. less than the designed length.

After the groyne was launched, the current actually was diverted to the bye-river for a time. But later, the main river again became stronger and active erosion of the island resulted. The net result was merely a bodily shifting of the meander.

Such groynes in the *katcha*, unless outflanking can be prevented, are not likely to be effective for the purpose of closing up one bye-river and opening out another.

TYPE (c)—TREE GROUYNE FOR SILTING A BYE-RIVER AT ITS OFFTAKE

Type (c), constructed after the *abkalani* of 1937, opposite mile 5/3 of Kassimpur-Nabishah Bund, on the other hand, appears to have been an unqualified success in silting up the bye-river (in which it was constructed) at its source, Figures 2C-19 and 2C-20.

The site A B (Figure 2C-19) was selected, after a detailed river survey, where there had been active erosion and the main current was directly striking the *pucca* edge. However, the launch (carrying the free end of the wire-rope) drifted down and the wire rope was actually fixed along C. B. On account of the downstream sag of the wire rope and because the floating buoys along with anchors were dragged down, the tree groyne after completion actually adopted the line BEDC.

Double B. G. rail 12 feet long was first fixed in cement concrete (1 : 3 : 6) block, 5 feet \times 5 feet \times 8 feet, at point B in the island, 260 feet away from the water edge, to serve as a temporary anchor and double B. G. rail was fixed similarly on the *pucca* at C, 600 feet from the water edge. For stretching the wire rope, a ten ton winch was firmly fixed in the *pucca* near C, by means of wooden sleepers buried deep in the ground and loaded by gunny bags full of earth. The sleepers were connected to the winch by means of wire ropes.

An old, flexible, steel wire one inch diameter rope made up of separate small lengths into one length, 1,850 feet, was first tried but snapped; and a new rope $1\frac{1}{8}$ inches diameter, 1,850 feet long, had to be used instead. One end of the wire rope was fixed to the rail block in the island at B and the other end was then taken across the river by gradually releasing the wire rope which was kept coiled on S. L. "Madge". The other end was then fixed to the ten ton winch in the *pucca*, after being taken through rings, made up from old chains, for tying trees on to it. The wire rope was supported at regular intervals by boats and pontoons in order to reduce the added strain on the winch when the wire was submerged and to facilitate stretching. It was then stretched by the winch. Part of the load (after trees were fixed on) was released from the winch, by connecting the wire rope to the rail block by a separate piece of wire connected to the main wire by means of U bolts.

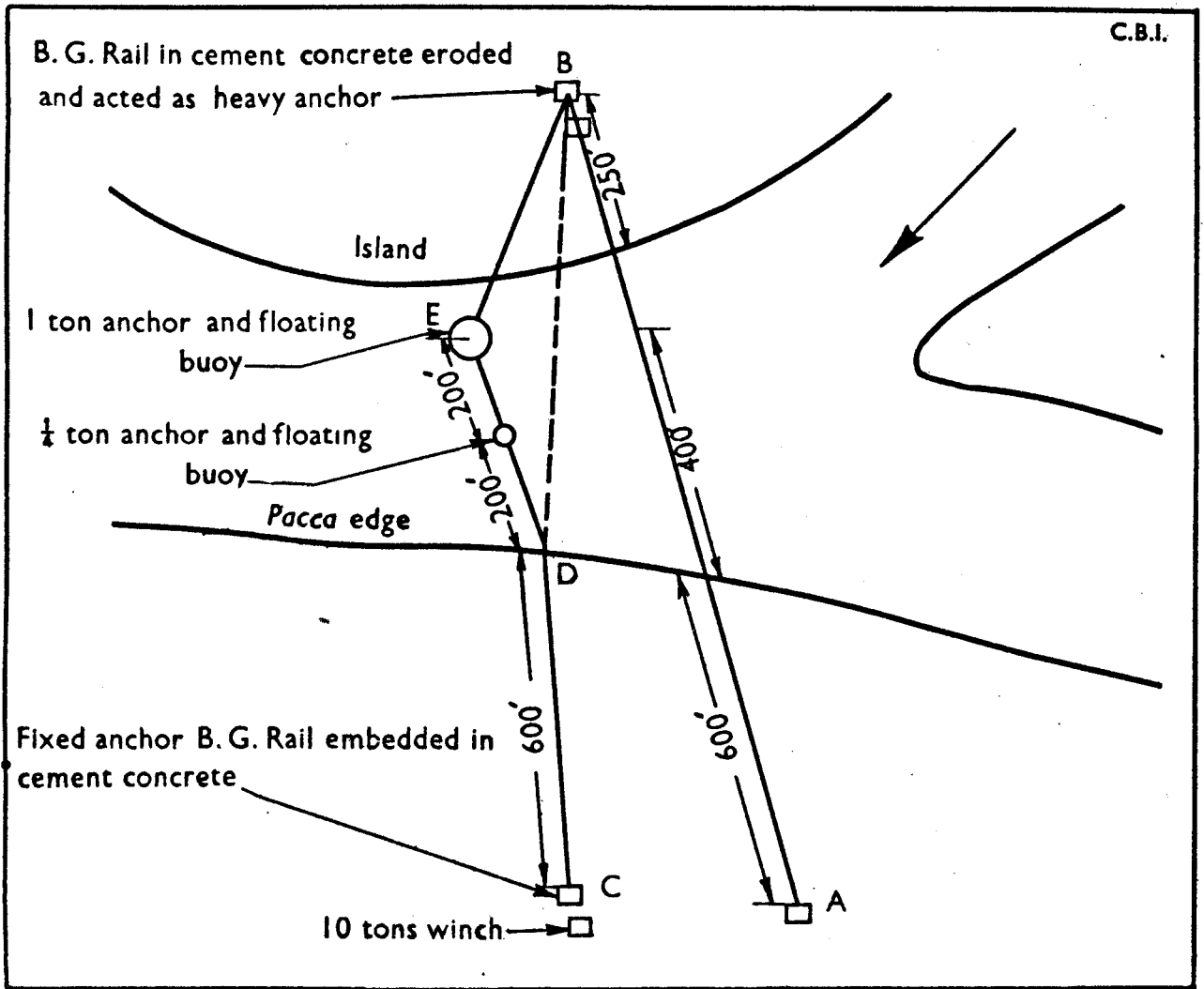


Figure:- 2C.19. Showing general arrangement of tree groyne type C

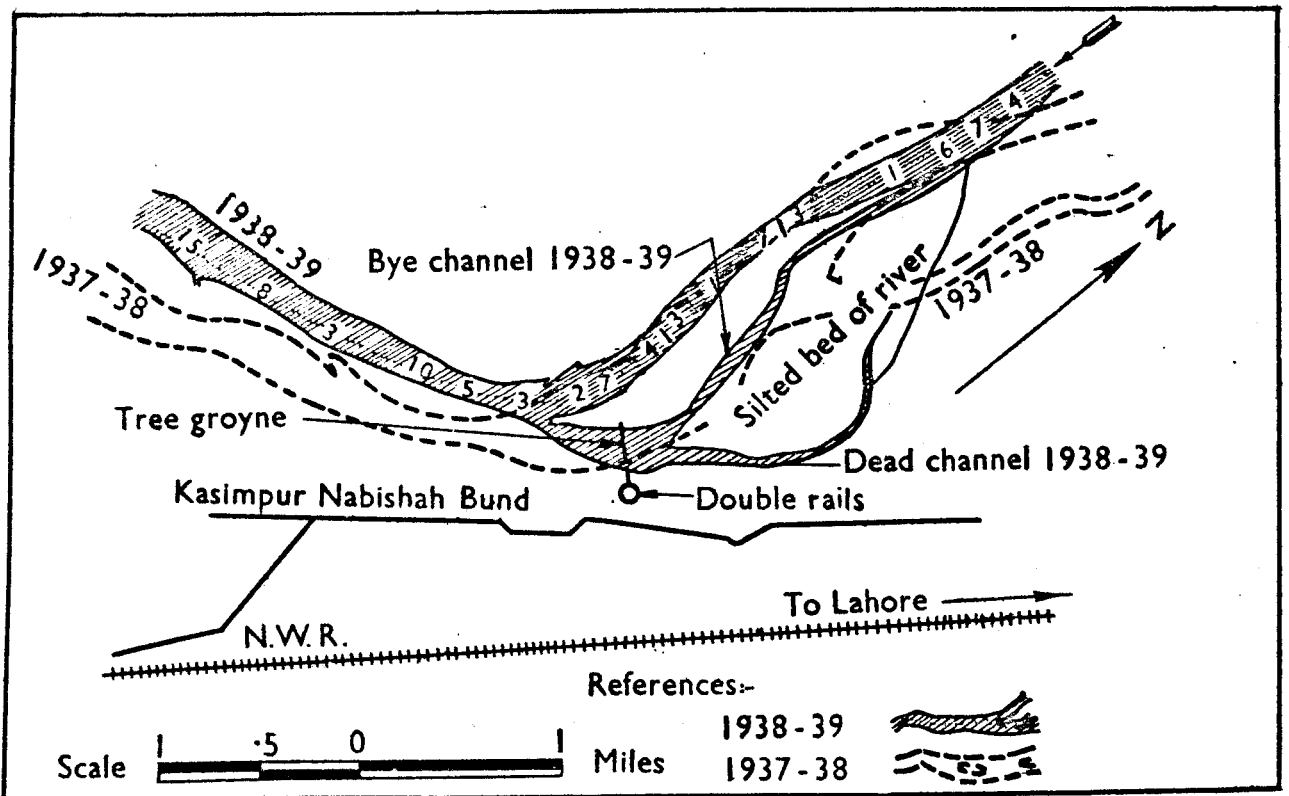


Figure:- 2C.20. Showing success of tree groyne type C in silting up the bye channel upto it's offtake

Two floating buoys, connected to heavy anchors with strong iron chains five feet long and shackles, were placed on the upstream side of the wire rope line, one with a five cwt. anchor 200 feet away and another one ton anchor 400 feet away from the *pucca* edge of the river, to hold the line of groynes in position. The anchors and buoys were fixed with due regard to the drag downstream by the current pressing on the loaded trees. The buoys and anchors were fixed to the main wire rope with separate wire pieces and U bolts.

Babul trees, 2½ to three feet girth, 25 to 35 feet long, complete with branches, were selected from a forest above the tree groyne site, so as to facilitate haulage of the tree on country boats. The trees were cut nearly at ground level and the whole trees were carted on bullock carts with branches over hanging at the back, the carts being hauled by a team of bullocks and men. The trees were loaded on the boats by pulling the trunks up by means of pulleys, the branches left to overhang on the side of boats.

The trees were unloaded and carted to the site in the same way. Holes one inch diameter, 2 feet above the bottom, were drilled through the trunks. They were then hauled on pontoons towed by S. L. "Madge" to the upstream side of the wire ropes and tied by wire rope pieces ten feet long, 3/8 inches diameter, to the iron rings or even to the main wire rope (the rings proving insufficient as a larger number of trees were required). The trees floated horizontally on the downstream side, until when overloaded with the deposit of silt and katchra trapped in their branches, they began to sink and get nearly vertical. It was found that as many as one tree per running foot of the groyne was ultimately actually required against an estimated provision of one tree per ten running foot. *Kandi* trees, though much lighter, and, therefore, easier to transport, are not nearly so effective as the more thickly branched *babul*.

With the load, the sag and drift of the wire rope increased, causing a large strain on the rail block B in the island, and on the winch. The wire rope had, therefore, to be eased from time to time at the winch, causing more and more sag. On constriction of the waterway, the island began to erode, as was the intention. Ultimately, the block B got eroded and submerged. The main rope, intended to be cut at E, was allowed to remain along with the rail block in the island, acting as a heavy anchor and preventing the groyne from moving downstream to any large extent.

Throughout the operation the main wire had to be kept above water level by shifting the pontoon ahead as the fixing of the trees progressed. When the buoy began to sink, the pontoon was moved towards the island. Part of the main wire rope was completely submerged with the weight of the trees. A subsidiary wire rope was, therefore, stretched from the first buoy to the point where the main wire rope was above water level and trees were fixed by stirrups to the subsidiary wire rope. Any remaining gaps were filled by trees, *lai* brushwood *etc.* being wedged in place and tied by strings and coir rope where necessary.

After the tree groyne was completed, the winch and pontoon were removed and the main wire rope fixed to the rail block in the *pucca* at C.

Soundings taken before and after the groyne revealed a marked improvement. Since that year there has been no further erosion of the *pucca* and the flow along the bye-river is so sluggish as to indicate that the river has silted at its source. Figure 2C-20 shows how successful the tree groyne has been in silting up the bye-river up to its off-take.

Experience on the groyne has taught the following lessons :—

- (i) Instead of attempting to fix the tree end B as a temporary anchor to a rail block in the *katcha*, it should be attached to a very heavy anchor, say a ten ton block 10 feet \times 5 feet \times 3 feet, where desired. This will not obstruct boat traffic and will prevent accidents which may lead to failure.
- (ii) An ordinary pontoon is rather powerless for keeping the wire rope up; one of the 80 ton barges available in the Barrage Division would be more useful.
- (iii) Far more trees than estimated are required and provision should be made at one tree per running foot. *Babul* trees about 3 feet in girth and 25 to 30 feet long are the best.
- (iv) This kind of groyne can be successfully made in depths of water about 15 to 20 feet. It must have at least one rigid, unyielding anchor fixed in *pucca* soil.
- (v) The cost of the groyne was Rs. 14 per running foot in 1937-38.

(10) BEAS RIVER—RAILWAY BRIDGE AT DHILWAN IN DISTRICT JULLUNDUR⁽²¹⁾

Figure 2C-21 shows the general site with a discharge of 15,000 cusecs. The position of the deep channel is indicated by a white line with arrows. The problem was to stop the erosion at the location marked X.

Details of model and prototype are :—

	Discharge in cusecs	Sediment diameter mm	Charge cubic feet	Time	Length
Prototype	5,000 to 250,000	0.25	..	5 months	9,580
Model	0.56 to 27.95	0.25	2	60 hours	95.80
Ratio	$\frac{1}{8,944}$	1.4	..	$\frac{1}{60}$	$\frac{1}{100}$

(21) The Punjab Irrigation Research Institute Lahore, Annual Report 1946, page 1

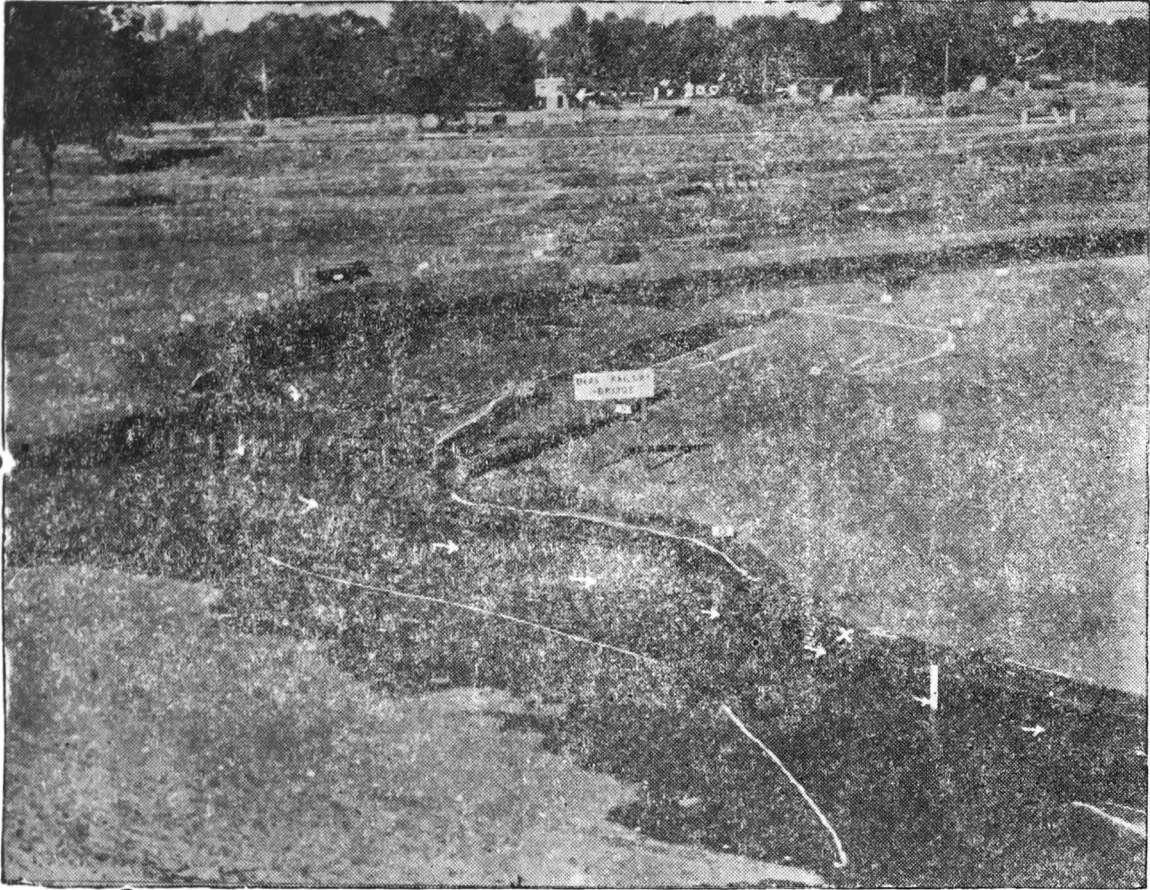


Figure 2C. 21 : *Model of the river Beas near Beas Railway Station—(looking upstream) the river, the Railway Bridge and the guide banks. Main flow is on the left side downstream of the bridge and is attacking the Dhillwan Timber Depot situated near the gauge—X shown in the model.*

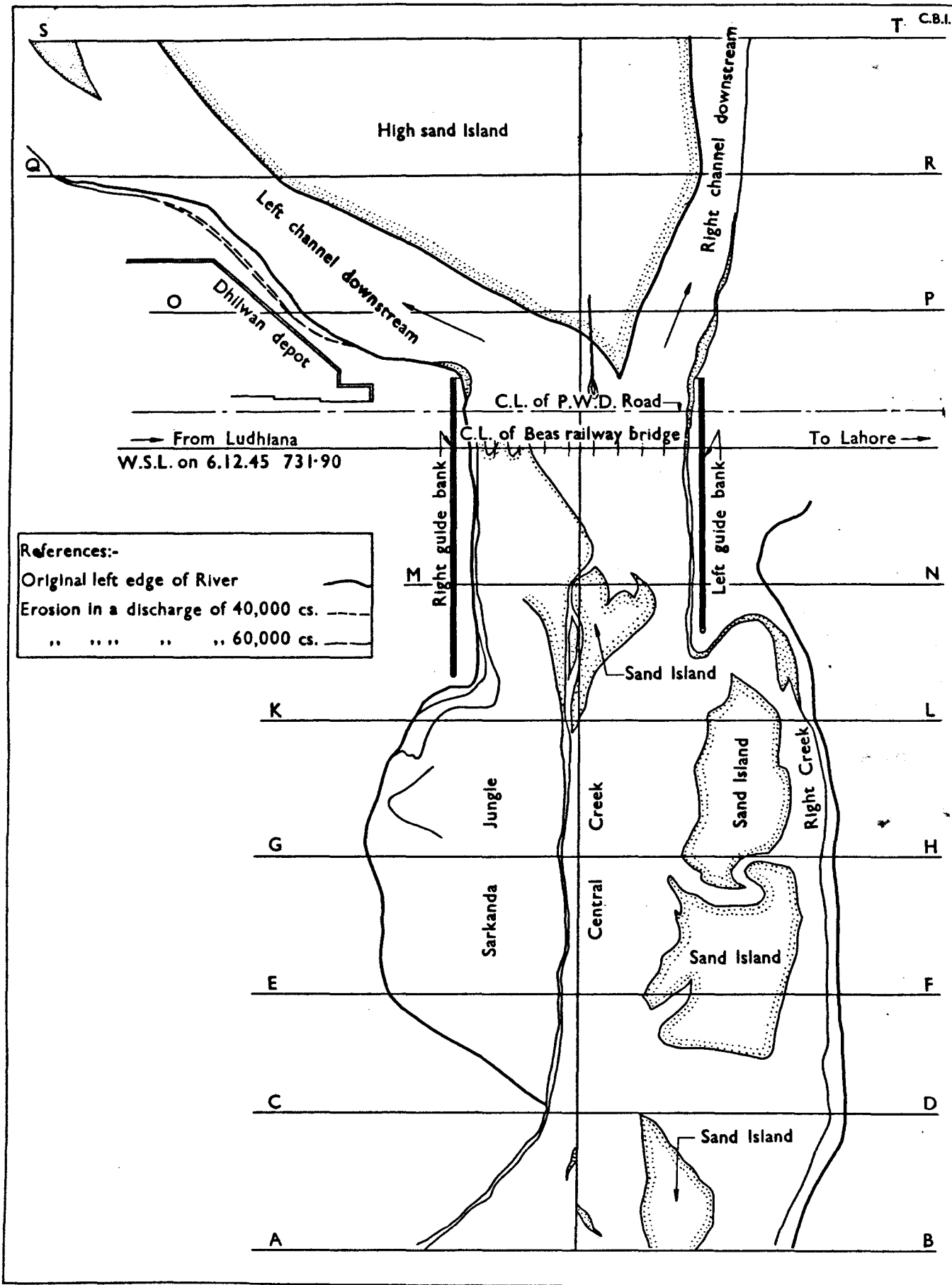


Figure 2C.22:-Model studies of the River Beas at Railway bridge site for the protection of Dhilwan depot. Showing of left edge and the position of the Deflector and Bund of 600 feet

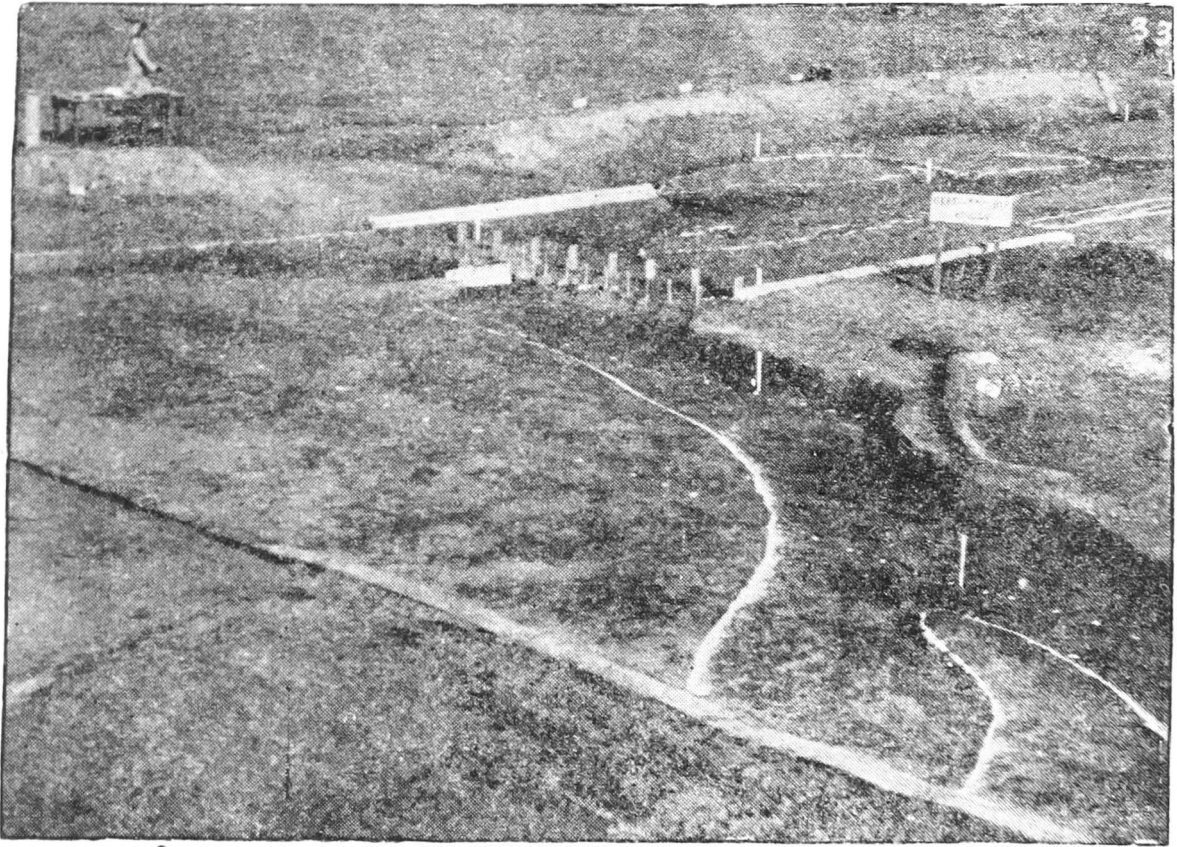


Figure 2C.23 : *Model of the river Beas at Railway Bridge at Dhilwan—Existing protective embankment extended by 600 feet downstream of the island*

Pitched islands were tried at various places to divert the objectionable current, but were not satisfactory.

Finally a deflector was built as shown at Y in Figure 2C-22 and the existing protective embankment was extended by 600 feet downstream of the island as shown in Figure 2C-23. This solution reduced the velocity along the original bed area; it was combined with a rounding of the upstream nose of the right bank to reduce the effect of attack there.

A report was prepared. The North Western Railway has since started work on the rounding of the upstream nose.

(1) RAVI RIVER—SPUR AT BAHERIAN, DISTRICT GURDASPUR (22)

Work reported last year (22) was continued. The object of the tests was to distribute the flow uniformly in front of the Baherian spurs and thereby relieve the strain on the spurs. After running the model for ten hours with a discharge equivalent to 50,000 cusecs a creek developed on the right side. As this was undesirable, various pitched island positions were tried with the object of discouraging the tendency. The most suitable position of the island was determined.

(12) RIVERS CHENAB AND JHELUM AT TRIMMU HEADWORKS

At the confluence of the river Chenab and the Jhelum upstream of the Trimmu Headworks, there has occurred the heavy concentration of flow at the nose of the Right Guide Bank. Several methods were examined to shift the confluence to the left (which at present has a strong tendency to move towards the right) and thereby to relieve the strain at the nose of the right Guide Bank. Pitched islands were also used to develop the extreme left creek on the left bank. Work is now being continued to induce a cut off to occur in the extreme right channel.

(22) The Punjab Irrigation Research Institute Lahore, Annual Report 1948, page 2.

(*) *Ibid.*

(13) RIVER SUTLEJ AT SAMASATA RAILWAY STATION ⁽²³⁾

The river continued to erode the left bank and the Chief Engineer North Western Railway desired to find the maximum depth of erosion of the left bank which would occur opposite the Samasata Railway Station after one year. Tests were made on a model in the presence of the mole head at section ten downstream. The indications obtained were that :—

- (i) Erosion of left bank opposite Samasata Railway Station will not be heavy.
- (ii) Sharabi Nalla will silt up and a cut-off in the river can occur from the Sharabi head position if soil conditions permit.

Detailed recommendations will be made in a special report being prepared for the Chief Engineer North Western Railway.

(14) SHAPE OF SPUR APRON FOR CONVENTIONAL TYPES OF SPURS ⁽²⁴⁾

Experiments were carried out in the river tray to determine the optimum shape of apron from the scour pattern for some conventional types of spurs—namely straight, hockey, inverted hockey and T-headed. The spurs were tested in a channel, trapezoidal in section with a side slope of 1 : 1, a bed width of 1.25 feet and a depth equal to 0.35 feet. The model was run for three hours with a discharge of 0.3 cusecs.

Results of the test are given below in Table 2C.12

TABLE 2C.12

Serial No.	Spur shape	Max. depth of scour below water-level (feet)	Area of apron square feet
1	Straight	0.52	0.44
2	Hockey	0.52	0.85
3	Inverted Hockey	0.46	0.61
4	T-headed	0.50	0.37

It was concluded that the T-head spur was more economical than the other conventional types.

(15) EFFECT OF THE ANGLE OF APPROACH ON THE SHAPE OF THE APRON OF A T-HEADED SPUR ⁽²⁵⁾

Experiments were made to study the effect of the angle of approach channel on the scour pattern around a T-headed spur. The important observations were :—

⁽²³⁾ The Punjab Irrigation Research Institute, Lahore, Annual Report 1946, page 3.

⁽²⁴⁾ The Punjab Irrigation, Research Institute Lahore, Annual Report, 1946, page 7.

⁽²⁵⁾ The Punjab Irrigation, Research Institute Lahore, Annual Report, 1946, page 8.

- (1) The area of apron increased with the angle of approach for angles greater than 45° .
- (2) The difference in scour depths at the upstream and the downstream noses decreased as the angle of approach increased. Therefore, while the apron should taper in width from the upstream head of the downstream and of T-head for a straight channel, it should be of almost uniform width in the case of right angled approach.

The detailed results of the above two investigations were published in the "Indian and Eastern Engineer" for March 1946.

DISCUSSION BY THE RESEARCH COMMITTEE

MR. PRAMANIK introduced item (1) and said that these experiments were undertaken to find the cause of the erosion of the town of Kurigram by the river Dharla for the last ten years and to devise remedial measures. On inspection of the site it was found that a new channel was developing as a short cut about $2\frac{1}{2}$ miles above Kurigram. This new channel had shifted the point of attack from below Kurigram to the right bank at Kurigram proper. The model had also proved this fact.

The length of the river from about ten miles to about two miles below Kurigram was incorporated in the model. The model (Horizontal Scale= $1/200$; Vertical Scale= $1/50$) was found to reproduce prototype conditions well. It had been found that three downstream spurs at Kurigram (spaced between cross section Nos. 11 and 13 on the right bank) would be sufficient to protect the town from further erosion.

DR. BOSE introduced item (2) and said that he was invited by the Cooch Behar Durbar to inspect the river Torsa and to advise the State on the preventive measures to be adopted for stopping the serious erosion that was taking place near the Palace on the bank of the river Torsa. He inspected the river from about 16 miles upstream of the town of Cooch Behar to about six miles downstream. In the first reach he found that the river was between high banks but at a point known as Barabhita, the river entered the wide basin whose left bank was well defined but the right bank allowed spills at high stages. At a point lower down near a temple a channel known as Buda joined the Torsa. For the last two or three years there had been a breach on the left embankment of the river higher up and considerable volume of water used to come down this stream Buda and join Torsa upstream of the temple. Since then the erosion at the Palace had intensified. The inspection showed that on every bend down stream of the temple the erosion was getting severe. The State had been advised to stop this overflow into the Buda so that immediate danger to the palace would be over.

DR. BOSE in reply to a query said that the river used to flow *via* Mora Torsa to the west of the town. In about 1886 the river had threatened the Palace and the Maharaja had asked the State Engineer to divert the river by means of a cut in 1887. The river since then had been regaining its length as could be seen from the meander line about the 1887 cut.

MR. PRAMANIK introduced item (3) and said that the problem had been tackled in two models. The river model was undertaken to find the best

position of the barrage and the training works necessary. The sectional model was undertaken to find the protection works necessary on the downstream floor of the barrage to make the designed section safe against uplift pressure and downstream scour.

The experiments on the sectional model of the under sluice portion had shown that staggered blocks of size 4 feet \times 3 feet \times 3.5 feet arranged as shown on page 140 would be the best under the circumstances. They would reduce the downstream scour from 40 feet to 26 feet for the maximum discharge. The scour could not be reduced any further by friction blocks.

The experiments were still in progress.

MR. FRAMJI commenting on item (1) said that the inlet and exit lengths of the model appeared to be too short, as there should be an effective length upstream of the entrance to the new channel and downstream of Kurigram and Pataswari. He wanted to know the slope at which the model was laid and what were the vertical exaggeration and slope exaggeration as laid and as obtained. What was the mean grade of bed material in the river—the mean grade in model being 0.3 mm.—and what was the quantity of sand injected.

One of the alternatives suggested in the model was to close the new channel altogether. Was that practicable seeing that the channel had now sufficiently developed? In a similar case of the Malaprabha River at the M. and S.M. Railway Bridge near Hole-Alur, it was considered impracticable to close off a cut-off channel which had developed downstream of the bridge and alternative measures had to be evolved by model experiments at Poona. The factors affecting the erosion were reasonably apparent and the design of remedial measures such as a long repelling spur down-stream of Kurigram to create afflux and avert the development of the new channel as well as to deflect the current away from the right bank could be easily evolved from a correctly designed model.

DR. BOSE replying said that in the first survey supplied they had overlooked the existence of this channel. By 1945, when the second survey was made this channel had developed considerably. The main problem was how to protect the town of Kurigram. They have already put in several spurs. About entrance and exit lengths provided in the model, he thought they were sufficient for Kurigram. Regarding exaggeration in the model it had been worked out by trial and error method.

Commenting on item (3) MR. FRAMJI said that although Figure II -18* of the River Research Institute Bengal did not substantiate it, it was stated that the Khatanga site was chosen originally as the river had a straight reach above and below it. A straight reach was not so important a consideration in the selection of a site for a Barrage as a *stable* reach. Was stability of river-course at the proposed sites studied?

Although the supplies from Pushkar nalla might be of use, when the river was in short supply, the *nalla* would also bring with it coarse sand and shingle and cause complications of shoaling or require expensive excluding or ejecting devices. So it could not be taken as an unmixed blessing.

The principal considerations governing the selection of a site were—

- (i) Potentialities of sand exclusion, if possible on both banks:
- (ii) Freedom from accretion, not merely freedom from local shoaling just upstream of the Barrage; and

* Not printed.

(iii) Stability both before and after the Barrage.

The approach to the selection of the suitable site for the Barrage from model tests thus left much to be desired. Firstly, no mention was made of experiments without a Barrage to indicate flow under existing conditions—although it was necessary to test intensively the stability of the site. Secondly, merely deciding the position of a Barrage on considerations of extent of the return flow region was not enough. The first model was stated to have given no bed movement below river discharge of 100,000 cusecs. This made the case all the worse for choosing the site by considering regions of return flow only. Figure 2C-19 appeared to indicate that the forward flow was through a very narrow strip which must have caused deep scour along the bank, if the model, was not faulty.

The position No. 4 was recommended although as was clear from Figure II 17* of the River Research Institute, Bengal the curvature was apparently excessive for the right bank canal so that water did not follow it during high floods and the left Bank Canal would certainly give insuperable trouble or be costly in satisfactory exclusion, particularly as the ratio of canal discharge normal river discharge was as high as 28 per cent.

• Spurs just upstream of the off take would create turbulence churning up the bed and surface flow so that they would seriously detract from the efficiency of exclusion by natural curvature of flow. A model was unable to show the loss in efficiency of exclusion due to the inter-mixing consequent upon the turbulence and this would make the proposals uncertain as well as less efficient. An important consideration should be sweet and smooth flow along the concave bank and this could be achieved, at least on the right bank, by suitably aligning and extending the right guide bank up to the point necessary to insure stability and permanency of the flow. The alignment could be decided on the model to eliminate the return flow region on the right bank, which would also eliminate or reduce the region of return flow on the left bank. The shape of the left guide bund did not appear satisfactory either in point of high river discharge following the bank or for the purpose of aiding exclusion. It was possible with the width of river available at this site so to design in the guide bund and offtakes on the left Bank—if necessary with an island and approach channel in between—as to create artificial curvature on that bank also.

The Barrage profile was designed for free overfall conditions only. Was no accretion anticipated downstream or upstream? For, if accretion occurs and there was even partial drowning the co-efficient of discharge would reduce and the discharging capacity of the Barrage would deteriorate.

If the deflector blocks, were correctly positioned and designed and worked as a deflector with a horizontal roller, sand should pile up against the deflector at a slope of 1 in 3 and no undermining should result. There was therefore, little justification of increasing the depth of the downstream pile which was mainly provided against undermining because deepening it increased uplift pressures.

DR. BOSE in reply said that the position of the barrage had been fixed by the engineer in charge who was told plainly that the site was likely to give silt trouble. The best site according to DR. BOSE was at Khatanga where the river channel was straight.

DR. BOSE further said that islands had been tried to train the river but as the river was generally low during most part of the monsoon season, these islands did not come into operation. The right bank canal had been found not to give any trouble of silting but heavy silting was apprehended on the left bank canal. By means of the four spurs in the embankment of the right bank, it had been possible to throw the intense concentration of the current on the right bank to the centre of the river but even so it had not been possible to have direct flow into the left hand pocket. The supply to the left bank canal was maintained by a flow from the right bank across the barrage and the left divide wall. This was known to be extremely unsatisfactory. Similar cases were known in many of the Punjab headworks.

MR. FRAMJI introduced item (4) and said that in the floods of 1939, and 1944, heavy damage was caused to the left guide bank, left hand piers of the bridge and the Railway embankment. Experiments were, therefore, conducted with the object of devising means of preventing the damage done by the floods by reducing the flood intensity and as far as possible, centralising and distributing it. The construction of an artificial pitched island was considered. The experiment proved that for any flashy river like Luni, which had a high discharge running for a short period only, the pitched island was not successful. In such cases an island did not have the same scope as in other rivers with high floods running for sufficiently long duration. In this particular instance, a symmetrical guide bank gave the best results.

Introducing item (5) MR. FRAMJI said that as a result of the existing adverse curvature the low water channel of the Jumna river receded after the floods towards the left bank away from the Power House, necessitating heavy expenditure in bunding or dredging. The pitched embankment suggested in last years' report was found not suitable due to the projected extension of the Power House. So fresh experiments were made this year. Some important discrepancies in the plans showed that the survey supplied was incorrect. The matter was referred to the Delhi authorities and a fresh survey was made and supplied. Fresh experiments were conducted taking into account the proposed barrage at Purana Quilla. The results showed that the new alignment of pitched embankment, with or without intake piers (shaped like 'attracting' islands) was the best solution.

MR. DILDAR HUSSAIN asked whether they have done any experiments to find out why the river receded away. Was it due to the series of spurs?

MR. FRAMJI said that the spurs there acted as attracting and not as repelling spurs.

RAO BAHADUR JOGLEKAR introduced item (6) and said that the Watrak flows in an alluvial bed of fine sand ($m=0.26$ mm). There is a sharp right angled bend upstream of the town Kaira. The river had passed a critical stage and the rate of erosion upstream of the town was accelerating. The river had already cut the road leading to Mehmedabad. In 1937 the Station was approached for finding measures to arrest further in roads of the river into the left bank. Experiments were carried out on three possible alternatives.

(1) A bridge was proposed projecting from the North West corner of the town :-

(2) A groyne 180 feet long sloping down 1 in 5 facing 30° upstream. Both these alternatives were meant to repel the attack of the river from the left bank ; and

(3) Which was the cheapest but not a good solution, was to strengthen the corner of the damaged fort wall by pitching.

The Bombay Government accepted the second proposal of the groyne ; but later, on account of the cost, they adopted the cheapest method of protecting the corner of fort wall with pitching. The station had warned at that time that this was not a good solution, because it would perpetuate the dangerous swing upstream and downstream and that it will hold the river permanently at the corner of the fort wall. Inspection carried out in 1940 revealed that the conditions were not satisfactory and that further experiments were suggested to be carried out ; but the local officers considered it better to wait and watch the effect of local pitching. Subsequently in 1942-43, a concrete bridge was constructed downstream of the town ; but the bridge spans were located towards the left bank *i.e.* away from the right bank ; the cold weather channel, which was along the right bank at the bridge, had to be blocked by an embankment, with the idea of forcing the river towards the left bank. In the next flood season this embankment was severely attacked and breached, virtually outflanking at the bridge. This was subsequently repaired at a huge cost of Rs. 2,35,000. The bridge had distorted flow to such an extent that Harijala village on the right bank was being rapidly eroded and an avulsion of the river into the low lying area on that bank was feared. The Director was again called for advice and after inspection, short tree groynes were suggested as a temporary measure. The lengths and alignments of the groynes suggested were such that they would not conflict with the final solution which would be decided by model experiments.

Introducing item (7) MR. FRAMJI said that to protect the left bank and the railway line downstream of the bridge the railway authorities had constructed a series of spurs, which only resulted in pulling the river towards Samasata. When the question was referred by the Railway to the Station for advice in 1943, a repelling spur duly armoured at nose and shanks was suggested : In the words of the Railway Officers, this spur worked entirely satisfactorily—repelling the attack away from the bank, causing some siltation and working towards the much desired cut-off—until a breach occurred in the spur due to the required armouring along the shanks being incomplete. The truncated nose of the breached spur acting as a pitched island considerably changed the regime of the river ; hence a new spur was recommended. Acting on contrary advice, the new spur was not constructed but the old spur was repaired ; as had been predicated by the Station, however, it again breached in the following flood season. The Director was requested by the Chief Engineer, North Western Railway, to visit the site and, as a result, detailed experiments were done at the request and cost of the Railway with and without mole head of the breached spur, to determine the likelihood of the development of the existing Sharabi Nala as a prospective cut-off channel. The experiments showed that the Sharabi Nala is adversely located for a cut-off to occur along it, particularly when unassisted by a spur. The Station recommended an impermeable stone spur about 1,500 feet long alternatively, if it was not

possible to complete the spur, together with the necessary armouring to shanks and nose, before the flood season, a permeable tree groyne was recommended in its place, or short permeable tree groynes as a purely temporary and protective measure.

DR. UPPAL said that Bhawalpur State approached them with this problem. They had been warned that the spur proposed by Sir Claude Inglis would not stand and it so happened. The detailed survey was supplied by the State and they have done several model experiments with the mole-head in position. They had not recommended a repelling spur and they visualized that the cut-off was taking at the same place if the soil permitted.

DR. BOSE said that the repelling spurs had got some inherent defects. He cited the instance in the river Beas.

MR. FRAMJI held that it was practicable to design a repelling spur to withstand the attack on it, however severe, such as at Samasata— provided it was properly located, aligned and armoured. Badly designed repelling spurs might fail but the fault is not so much with the type of spur as with the design or construction not being adequate. He would not recommend a repelling spur of the impermeable type at a point where it could not be held, if duly armoured.

Commenting on the work done in the Punjab on this model, MR. FRAMJI said that interpretation of model results should be given in firm and unequivocal terms. The conclusion arrived at was that a cut-off in the river would occur from the Sharabi head if soil conditions permitted. This he said, was not conclusive and suggested lack of confidence in the results.

He further asked what were the scale ratios, slope exaggeration as adopted and obtained, the grade of the material, and the injection used in the model in the Punjab.

MR. BLENCH explained that the Railway Department had approached both the Poona and Lahore Stations with the Samasata problem; but as the levels were inadequate to describe the problem fully the two stations had made slightly different models. The Poona Station gave levels that prevented occurrence of a cut-off, the Lahore Station model permitted cut-off. He found that cut-off could occur and develop. He disagreed with Mr. Framji's views on what constituted a firm and unequivocal statement of results. When a model of noncohesive sand showed a cut-off developing the conclusion was that it would develop in a prototype of similar material and corresponding levels. The engineer's attention had to be drawn to this, otherwise he might hope for a cut-off to occur with non-corresponding levels and in material that was either cohesive or bound by vegetation. The Lahore Station's recommendation contained a suggestion (not mentioned in the Annual Report) on how to assist a cut-off to occur.

The moral of the disagreement in recommendations of the two Stations seemed to be that model work required better information than was normally supplied; he had recommended to his own Government that important river model work should not be undertaken without an air-survey.

MR. FRAMJI then introduced item (8) and said that the spur constructed on the left bank had fully served its functions of repelling attack from the left bank and the right abutment of the bridge. But it was feared that increased depth and increased concentration of ebb flow along the right bank in the approach channel to the bridge might cause some danger to the goods yard at Kolaghat. They could arrest the action by inserting permeable spurs. The positions and lengths of tree groynes were tested in pilot models. The recommendations have, however, not been carried out because the local officers consider the action at the goods yard as insignificant.

DR. BOSE commenting on item (8) said that so far as the latest information went, extensive scouring was going on in the right bank of the river upstream of the railway bridge. Bengal Irrigation Officers had reported the matter to the Bengal Nagpur Railway authorities for immediate action.

MR. FRAMJI introducing item (9) said that tree groynes had been successfully used in Sind for river training under suitable conditions. There were three types of groynes for three different purposes. This note gave successes and failures. In his view, such groynes had large potentialities because they had several advantages over the types.

DR. BOSE enquired as to how these tree spurs were laid.

MR. FRAMJI briefly explained the method of launching these tree groynes as done in Sind.

DR. UPPAL referring to item (14) and (15) said that they studied the efficiency and suitability of various types of spurs, such as hockey spurs, T-spurs etc. The criterion was taken as the amount of scour that each produced. Among these the T-head spur was found to be the best. Next the angle of approach was studied—six different approaches were tried. It was found that the approach should not be more than 6° .

SECRETARY requested for more details regarding dimensions of spurs.

MR. FRAMJI said that a spur primarily depended for its action on the behaviour at or near the bed while only lines of flow at surface appeared to have been studied. Bed flow lines were more important. Each kind of spur had its own use or function and it was wrong to generalize on the relative efficiency of a spur from the amount of scour it caused. For example, scour would be a desirable feature in an "attracting" spur to quite the same extent as it would be undesirable in a "repelling" spur. He asked what was exactly meant by the area of apron. Was it the extent of the scour hole?

DR. BOSE asked whether the experiments had been carried out in flumes

DR. UPPAL replied that these had been carried out in glass models.

In reply to Rao Bahadur Joglekar, DR. UPPAL said that the model was geometrically similar.

In reply to a query as to how scour depth could be reproduced correctly in the geometrically similar model. DR. UPPAL stated that the models were very big—to 1/10th scale. In reply to the Secretary's question DR. UPPAL stated that the total discharge was 0.3 cusec.

MR. FRAMJI remarked that the model could not have been very large, as the discharge was only 0.3 cusecs. Results obtained in a straight, rigid flume were of little quantitative value for use with curved channels where spurs are used.

DR. UPPAL said that they had actually constructed such spurs in the Punjab and had found that T-headed spurs were the best. The area of apron or the extent of scour was used for assessing the comparative merits of various spurs.

MR. FRAMJI remarked, that at the most the experiments could be claimed to show that where minimum scour was desirable T-head spurs were the best.

DR. UPPAL explained the experiments conducted during the year regarding item (12). After the confluence of the two streams the river took a circuitous turn towards the Trimmu head-works. At Jhelum the river was going farther and farther away. They first put in bulley islands and tried moving them. All the experiments gave no solution. Now they were leaving the river to develop a straight reach by itself.

MR. FRAMJI said that this was a striking example of the criticisms against building barrages 'in the dry' epitomized in Mr. Montagu's dictum "when you build a barrage you sign its death warrant".

No comments could be made on the model as no data was furnished regarding scales *etc.*

DR. UPPAL explained the experiments done regarding the Beas river to lead the flow in the centre of the Railway Bridge at Dhilwan. The Railways wanted to protect the creosoting plant and depot on the left bank of the river. A wall was constructed joining the third pier of the bridge. Swirl formation was observed at the farther end of the wall which disappeared after the construction of nose to the spur. The conditions of flow were observed to be better after the nose.

Here MR. FRAMJI objected to different terminology being used in the research reports. For example the term 'deflector' had a recognized meaning. One always used a deflector to create a beneficial bed roller below a rigid structure; if its function was to divide water between the right and left channels to distribute the flow and thus reduce concentration or action, it should be more correctly called a 'dividing embankment'. On the other hand if its purpose was to deflect the current, it should be called a 'repelling spur'; but in the present case, it was more likely to be serving the purpose of an "attracting" spur than a "repelling" spur.

Continuing MR. FRAMJI remarked about the Beas experiments that the depth scales and vertical exaggeration were not given and when worked out from discharge scale, it was presumed that depth scale was 1/20 and hence $V.E.=5$ and probably model $B.S.E.=V.E.$

What was the slope exaggeration obtained during experiments and did the adopted depth scale tally with the observed depth scale.

Figure 1* showed pitching on the left bank downstream of the bridge. Actually in the model the pitching was not reproduced as seen by the recession of the left bank in Figure 3*. These experiments should have been repeated with pitching because the training measure without pitching might show bigger effect in the model than with pitching on the opposite bank.

He enquired as to what was the design of the so-called deflector. Very severe action was to be expected at its nose, being on the outside of the curve of the left channel. Other training measures if tried had not been described ; but certainly there were other more promising alternatives.

DR. UPPAL replied that experiments were done without pitching and actually it was not put in practice.

He then introduced item (11). The pitched islands at the spur in the Ravi at Bherian had worked satisfactorily in protecting the right bank from being cut by the river.

MR. FRAMJI asked as to what was the design (side slopes *etc.*) of the pitched island adopted. In the same position they had found at Poona that it was possible to improve the attracting effect of an island by providing a steeper slope on the side on which it was desired to attract flow.

It was better, he said, to have a plan to show the position of the " pitched attracting island " relative to the meanders and high banks. He then enquired as to—

- (1) the percentages of discharges on the side of the *creek* before and after the construction of the island ;
- (2) whether the flow was uniform in front of the Baherian spurs after stabilization with the island and
- (3) whether there was no likelihood of transferring the attack down stream of the Baherian spurs.

DR. UPPAL replied that the design of islands was the same as was used at Suleimanki with side slopes 2 : 1 and tail slopes 5 : 1. By these changes the discharge in the two creeks which was originally 5% later on developed to 30%.

MR. FRAMJI suggested it was desirable to distinguish between dividing and attracting islands according to the functions intended to be served by them. The former one were meant merely for dividing flow, while the latter were for attracting the river to its nose or sides as a training measure.

THE SECRETARY said that regarding terminology, the pitched island as used in the Punjab was an attracting island and the dividing one was called a groyne.

(iii) SCOUR AND EROSION

The following items were discussed at the 1947 Research Committee Meeting :—

- (1) Training of the river Indus above the Sukkur Barrage to prevent slips of the right bank below the nose of the approach channel.

- (2) Emergent measures for preventing serious damage to the central sluices for the Mahanadi Anicut at Cuttack, Orissa.
- (3) Scour downstream of rigid structures in wide channels—prototype data regarding depth of scour downstream of bridges.
- (4) Cofferdams for the Ramapadasagar Project.
- (5) The Hagari Aqueduct for the Tungabhadra Canal—Scour around piers.

(1) TRAINING OF THE RIVER INDUS ABOVE THE SUKKUR BARRAGE TO PREVENT SLIPS OF THE RIGHT BANK BELOW THE NOSE OF THE APPROACH CHANNEL (26)

SUMMARY

The primary object of these experiments was to reduce the concentration of flow at the nose and the depth of scour inside the approach channel, thereby establishing forward flow throughout the approach channel. This would prevent slipping of right guide bank opposite the nose consequent upon the side slopes not becoming steeper than the natural angle of repose. This would also, incidentally, improve the sand exclusion at the entrance to the approach channel. Various devices were tried and experiments are still in progress.

(2) EMERGENT MEASURES FOR PREVENTING SERIOUS DAMAGE TO THE CENTRAL SLUICES OF THE MAHANADI ANICUT AT CUTTACK-ORISSA (27)

SUMMARY

The design of the Central Sluices of the Mahanadi Anicut being defective for dissipation of excess energy necessitated heavy recurring expenses over purely protective measures; nevertheless serious damage occurred to the downstream floor threatening the dwarf walls in the floods of 1945. The trouble was diagnosed as occurring due to the flat downstream pavement which did not hold the standing wave when the downstream water level was low. Suitable dissipating devices *viz.*, a glacis followed by a series of baffles to dissipate the excess energy were suggested.

Due to a practically flat pavement downstream of the Central Sluices of the Jobra Anicut on the Mahanadi at Cuttack an unstable standing wave formed on or downstream of the loose stone protection below the *pacca* pavement in low floods, when the river discharge passed through the sluices was only of the order of 50,000 cusecs. This was due to very low water level in the river downstream obtained at the end of a flood season, *viz.* R. L. 61.0 with a comparatively high upstream water level, at R. L. 68.5 and an unuitable and inadequate pavement for these conditions. A deep scour-hole used to form downstream of the loose stone protection, in which the stones sank unevenly aggravating the trouble; the *pacca* pavement was also threatened by undermining. The scour-hole continued to form each year in spite of heavy local repairs to the loose stone protection.

(26) Indian Waterways Experiment Station, Poona, Annual Report (Technical) 1946 pages 17 to 27.

(27) Indian Waterways Experiment Station, Poona, Annual Report (Technical) 1946 pages 27 to 37.

In the hope of dissipating energy, three rows of staggered blocks were constructed by the local officers in 1944-45, this being the first attempt to treat the cause rather than the symptom; the downstream divide walls were extended and loose rubble stone in *bullah* cribs and crates were also dumped in the scour-hole.

(3) SCOUR DOWNSTREAM OF RIGID STRUCTURES IN WIDE CHANNELS—PROTOTYPE DATA REGARDING DEPTH OF SCOUR DOWN STREAM OF BRIDGES ⁽²⁸⁾

Experiments were done in the year 1944-45 ⁽²⁹⁾ to correlate depth of scour downstream of a rigid structure with intensity of discharge and grade of bed material.

For the substantiation of these results, analysis of similar prototype data is necessary.

Such data are being observed for the last 20 years on 21 important bridges on alluvial rivers on North Western Railway. The North Western Railway authorities have kindly made this data available for analysis, Table 2 C. 12.

The depths, at present, are observed only at the bridges, in floods, to see if the scour around the piers exceeded the safe limit. Actually, a deeper and more dangerous scour-hole often occurs further downstream of the bridge, due to the increased velocities and turbulence arising from the restriction of the water way, as was observed in the case of Hardinge Bridge over the Ganges ⁽³⁰⁾

The Railway and canal authorities have, therefore, been requested to furnish the following data :—

- (a) Depths at 150 feet, 300 feet and 450 feet downstream of the bridge in addition to the depths observed at the bridge, this being necessary to get maximum depth of scour which is expected to occur within this range downstream of the bridge due to the angle of repose of the bed material : correctly a contour plan of soundings over a distance of 1,000 feet downstream of bridges, made from probings at frequent intervals during a flood season is required. This would require much additional staff and equipment and so, for the present, cross-sections, 150 feet apart, till beyond the point of maximum scour, have been requested, for, in case of only eight out of 21 bridges.

⁽²⁸⁾ Indian Waterways Experiment Station, Poona Annual Report (Technical) 1946, page 114.

⁽²⁹⁾ Indian Waterways Experiment Stations, Poona, Annual Report (Technical) 1945.

⁽³⁰⁾ Central Irrigation and Hydrodynamic Research Station, Poona, Annual Report (Technical) 1941-42, page 12.

TABLE 2 C-12

Important Bridges on alluvial rivers on the North Western Railway

Serial No.	Name and site of Bridge	Place at which discharge is observed	Distance between the site of of bridge and place where discharges are observed (Miles)	Whether upstream or downstream of bridge	Remarks
1	2	3	4	5	6
1	Jhelum at Jhelum ..	Rasul Head works	21	Downstream	
		Mangla ..	22	Upstream ..	
2	Jhelum at Haranpur	Rasul Head-works	28	Do.	
3	Jhelum at Khushab	Chela ..	85	Downstream	
4	Chenab at Chund	No observations in the vicinity
5	Chenab at Wazirabad	At the bridge	
6	Chenab at Chiniot E.	} Rivaz bridge	56	Downstream	
7	Chenab at Chiniot W.				
8	Chenab at Shershah	At the bridge	
9	Ravi at Lahore ..	Sidhnai	5	Do.	
10	Ravi at Abdul Hakim	At the bridge	
11	Ravi at Dera Baba Nanak	Shahadara ..	72	Downstream	
		Madhopur Head-works	59	Upstream	
12	Beas at Beas ..	Mandi plain	30	Downstream	
13	Sutlej at Phillaur	Rupar Head-works	
14	Sutlej at Gidaripindi		
15	Sutlej at Adamwahan	At the bridge	
16	Jumna at Delhi ..	Do.	} No record in Lahore Discharge Division
17	Jumna at Kalanour	
18	Indus at Kotri ..	At the bridge	
19	Indus at Sukkur ..	Do.	
20	Indus at Kalabagh ..	Do.	
21	Kurum at Thanedarvala	No record

- (b) Discharge observations, upstream or downstream of the bridge (to be obtained from the Discharge Division), for correlating discharge intensity, grade of material and maximum depth of scour.
- (c) If it is not possible to obtain bed-sand samples under flood conditions from the scour-hole, they can be obtained by borings (made in the winter) up to the maximum depths of scour, in the region of the scour-hole. This additional data will be observed for the eight important bridges (shown in italics in Table 2C-12) where daily discharge measurements are made by the Discharge Division.

(4) COFFER DAMS FOR THE RAMAPADASAGAR PROJECT ⁽³¹⁾

ABSTRACT

The width of the river Godavari at the site selected for constructing the Ramapadasagar Dam is about 5,000 feet. There are substantial rocks at both flanks and also a hill, called the Mahanadi Hill, in the centre of the river. The main dam will be in two parts abutting against the flanking rocks and joining at the central hill in a hockey-stick alignment. The river bed at the site is sandy for great depths and firm rock is met at 150 to 200 feet below bed. The summer flow is about 20 feet deep and during maximum flood the depth is about 65 feet. The maximum flood discharge is as much as 2.4 million cusecs. There are no facilities for diverting the river through saddles or tunnels and the construction of the dam foundations has to be taken up in two stages enclosing one half of the river within coffer dams in each stage. The Mahanadi Hill will form the anchorage for the coffer dams. In the first stage the right half of the river at the dam site will be enclosed within coffer dams and the dam built up to about the bed level. Next the left half will be enclosed by coffer dams for construction of the dam up to bed level. Thereafter the entire dam will be raised in stages.

The coffer dam itself will consist of a solid masonry wall just over the central hill, of earthen *bunds* with or without rip-rap protection at the margins and of cellular sheet piles across the sandy portions. These cells will be about 60 feet diameter each, made of interlocking sheet piles, going down to about 100 feet below bed for reducing seepage into the trench, and rising above flood water level for preventing over topping.

The model experiments were carried out for

- (i) determining the afflux caused by the contraction of water way and thus fixing the top levels for the coffer dams ;
- (ii) studying the flow patterns and examining the presence of dangerous eddies, if any ;
- (iii) estimating the velocity distribution in the surface currents ;

⁽³¹⁾ Irrigation Research Station, Poondi, Madras, Annual Report for 1946 pages 7 to 21.

- (iv) finding out the incidence, development and pattern of scours in the region affecting the safety of the coffer dams ; and
- (v) generally examining the soundness of the alignments.

DESIGN OF THE MODEL

The smaller model was designed as a pilot model and as it was desired to have it constructed inside the laboratory, protected from wind and rain, the scale was fixed as 1/300 taking into consideration the space, time and discharge available for the studies. The model was intended essentially for the study of flow pattern for which similitude with respect to both gravitational and frictional forces had to be obtained. A geometrically-similar model giving a high Reynold's number and having its friction adjusted to scale was best suited for the purpose. Adjustment of friction was effected by selecting a suitable grade of sand by a process of trial and error for moulding the bed. There was no sand injected at the upstream end of the model. The sand used had, therefore, to satisfy the conditions (1) that without the coffer dam in position there would be no movement of the bed, (2) that for the same condition the prototype surface falls would be reproduced in the model and (3) that when the coffer dam was put in and thereby the velocities of flow increased, scours would be produced within reasonable time. River sand sieved to pass through 30 and retained on 50 mesh per inch (0.5 to 0.3m.m.) was found suitable and was adopted. Numerical values of the scale ratios were :—

Linear scale	1 : 300
Velocity scale	$1 : \sqrt{300} = 1 : 17.25$
Time scale	$1 : \sqrt{300} = 1 : 17.25$
Discharge scale	$1 : 300^{\frac{3}{2}} = 1 : 1.559 \text{ millions}$
Friction scale	$1 : 300^{\frac{1}{2}} = 1 : 2.587$

CONSTRUCTION OF THE MODEL

Salient features of the model are shown in Figure 2C-24. The overall size of the model proper was 40 feet \times 25 feet included between nine inches thick brick walls built on the concrete floor of the laboratory and comprised the reach of the river 6,500 feet upstream of the dam line and 3,000 feet downstream. The reach 6,500 feet to 5,000 feet upstream, forming five feet length of the mode at the head, was moulded rigid so as to preserve the correct direction of the approaching stream. The last cross-section was made up in masonry. The central hill and the non-erodible margins on both sides were also correctly moulded in masonry. The rest of the model representing the sandy bed was moulded in sand.

The model was provided with a forebay, stilling racks, a tail bay and a tail gate. The water-supply was drawn by gravity from the reservoir (or by pumping when the reservoir was low) into an eight feet diameter well which was provided with an over flow arrangement so as to maintain a constant head

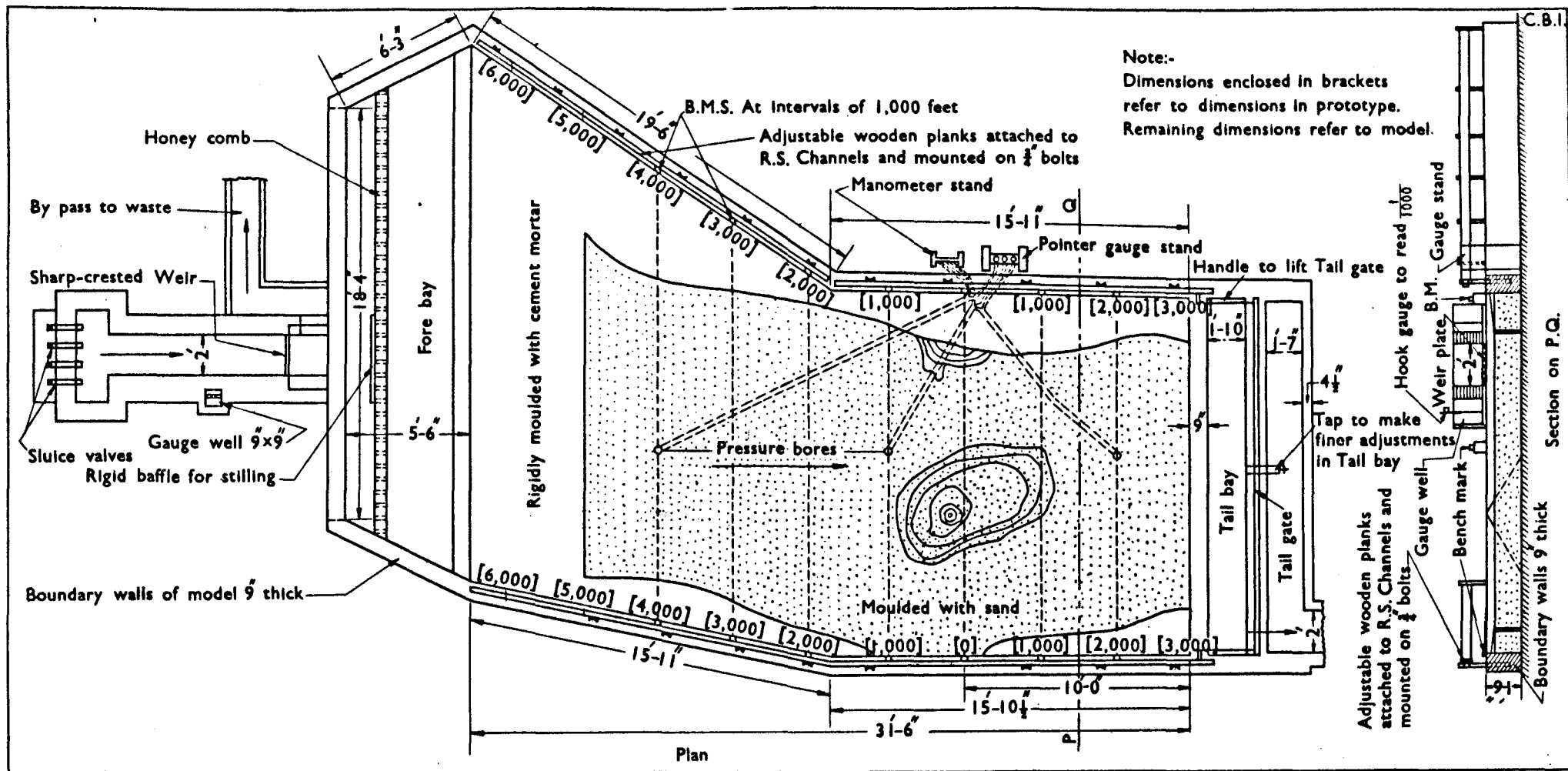


Figure 2C.24. Showing layout of model [Ramapadasagar Cofferdam-Model $\frac{1}{300}$]

Scale 5 2.5 0 5 10 15 Feet

through out the progress of experiment. To this well were connected four inches diameter pipes, each provided with a sluice valve, which delivered the supply to the model. Before admission to the model the water was stilled and gauged over a two-foot wide sharp edged weir which had been previously calibrated. Beyond the weir a bye-pass was provided through which the water could be diverted when not required to be let into the model.

Pressure boxes built of perforated brass sheeting covered by wire mesh were embedded underneath the sandy bed of the river at points on the axis corresponding to 2,000 feet downstream, 1,000 feet upstream and 4,000 feet upstream respectively of the dam line. These boxes communicated the water level elevations at the points through $\frac{1}{2}$ inch diameter brass pipes also running underneath the model bed. These pipes all converged to a point near the side wall of the model where they were taken out and connected with Y-pieces. One limb of each Y-piece was connected by rubber tubing to a brass can and the other to a manometer glass tubing. The free surface of the water in the brass cans and glass tubes indicated the water surface elevation at the relevant point in the model. The cans and tubes were mounted together on two racks and the readings were taken by pointer gauges reading to 1/1,000 feet.

Several bench marks were fixed at various parts of the model and accurately levelled by means of "water levels" and all adjustments of the river bed, water surface, *etc.*, were made with reference to these bench marks.

Wooden railings, fixed in brackets running along the top of the model walls, served to support "male templates" cut in galvanized iron sheeting which were used for moulding the mobile bed.

A cradle was constructed just over the dam line from which photographs of candle floats could be taken for getting the direction and magnitude of the surface velocities.

OPERATION OF THE MODEL

For any given coffer dam arrangement the tests were conducted in the following manner. The tail gate was raised and the dry model was flooded using an auxiliary hose pipe supply line. This was necessary to ensure that no scour occurred in the model bed before correct flow and tail water conditions were obtained. The desired discharge was then set on the measuring weir and the correct tail water elevation corresponding to this discharge was obtained on the gauge 2,000 feet downstream of dam line. The gauge discharge relationship adopted is given in Table 2C-13. For each set of experiments four discharges corresponding to 0.091, 1.36, 1.85 and 2.25 million cusecs were passed. When flow conditions got steady in each case, the gauge reading were taken. Flow pattern was observed and the presence and magnitude of eddies detected by means of confetti, aluminium powder and floats. Seven candle floats held two feet apart from one another were released simultaneously across the model at cross sections 5,000 feet upstream and these indicated the general pattern of surface currents. These were photographed. The model was run for the desired length of time and at the conclusion of the test the water-supply was stopped without causing any disturbance to the model.

TABLE 2 C-13

Gauge discharge Table for the river Godavari at the Ramapadasagar dam site

Discharge in million cusecs						Water surface elevation in normal river at dam line
2.25	+90.0
1.85	+88.7
1.36	+87.2
0.91	+84.2
0.53	+73.0
0.22	+62.5

Where scour contours had to be traced the water from the model was lowered to the required levels in stages, the boundary of the water giving the contour in each case. The configuration so formed was determined by a system of co-ordinates obtained by stretching strings across the model. This was found convenient and especially suited for the present purpose where to transfer the undulations of a bed full of ripple by the usual method of taking cross sections of the bed would have been less accurate and more tedious.

The model was finally unwatered completely, to facilitate which the end cross section wall had been provided with a number of holes filled with lean porous concrete, and photographs revealing the position and shape of scour creators were taken. The disturbed bed was then remoulded and the next experiment taken up.

DESCRIPTION OF TESTS

Altogether twenty-two experiments were conducted on this model. The first few experiments were for verification of the model and for selecting the bed material. Then two preliminary designs for the west (right) side and east (left) side coffer dams respectively were tried on the model. In the final set of experiments two designs suggested by Dr. Terzahi were examined. These are briefly explained below :—

(i) *Normal river.*—The last experiment of the first series carried out on the model without putting in the coffer dam reproduced the surface fall of $2\frac{1}{2}$ feet per mile and indicated no sheet movement of sand nor local scours. The flow pattern around the central hill indicated by confetti is shown in Figure 2C. 25. The water surface elevations at the three gauges observed on the model are furnished in Table 2 C. 14.

(ii) *Coffer dams*—(a) *Preliminary designs.*—The alignment of the west side coffer dam as designed in the first instance is shown in Figure 2C. 26. In this figure the masonry portion of the coffer dam, the cellular sheet piles and the earthen embankments are all separately indicated. Figure 2C. 27 furnishes a view of the model before admitting water and Figure 2 C. 28 shows the flow pattern obtained on this model. The approach of the currents towards the constriction of water way was fairly uniform, there was no rapid current alongside the coffer dam and beyond the nose of the coffer dam on the central hill the flow line nearest the coffer dam got deflected away from the nose. There were no harmful eddies anywhere and no bed scour. From a hydraulic point of view the design was quite good. But the length of the coffer dam was excessive and though the alignment followed a ridge in the sub-surface rock, a reduction in length, if possible, would be more economic.

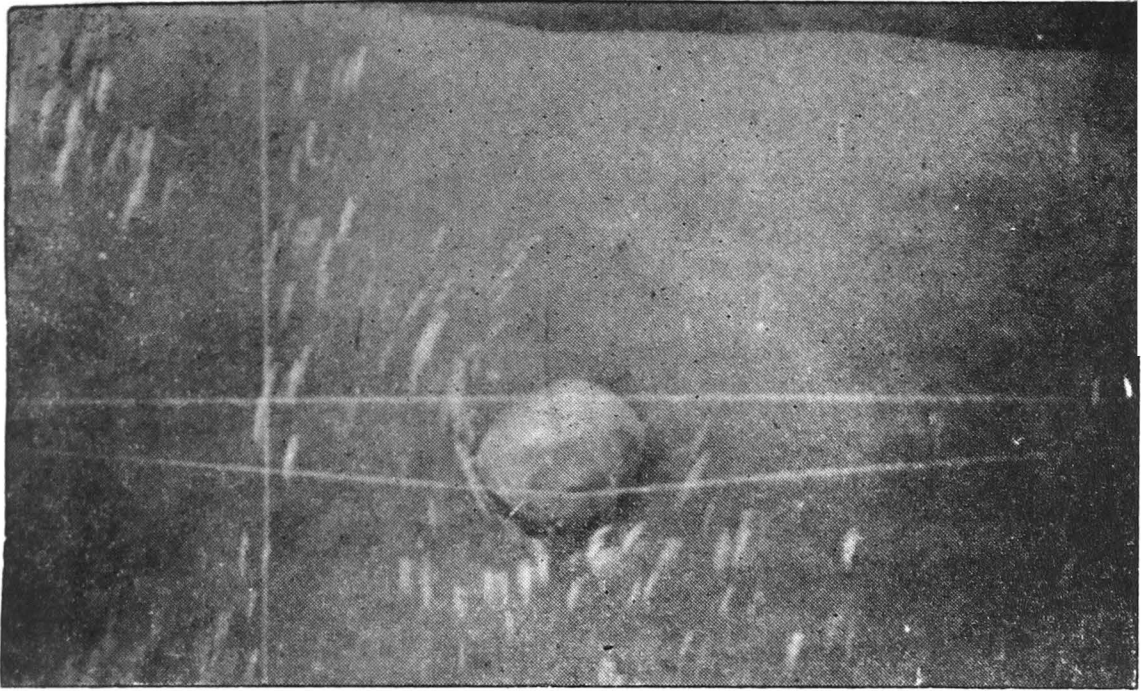


Figure 2 C. 25 : *Ramapadasagar coffer dam model 1/300—Photograph taken from the cradle from the Central Hill looking downstream. Flow pattern round the Hill indicated by confetti.*

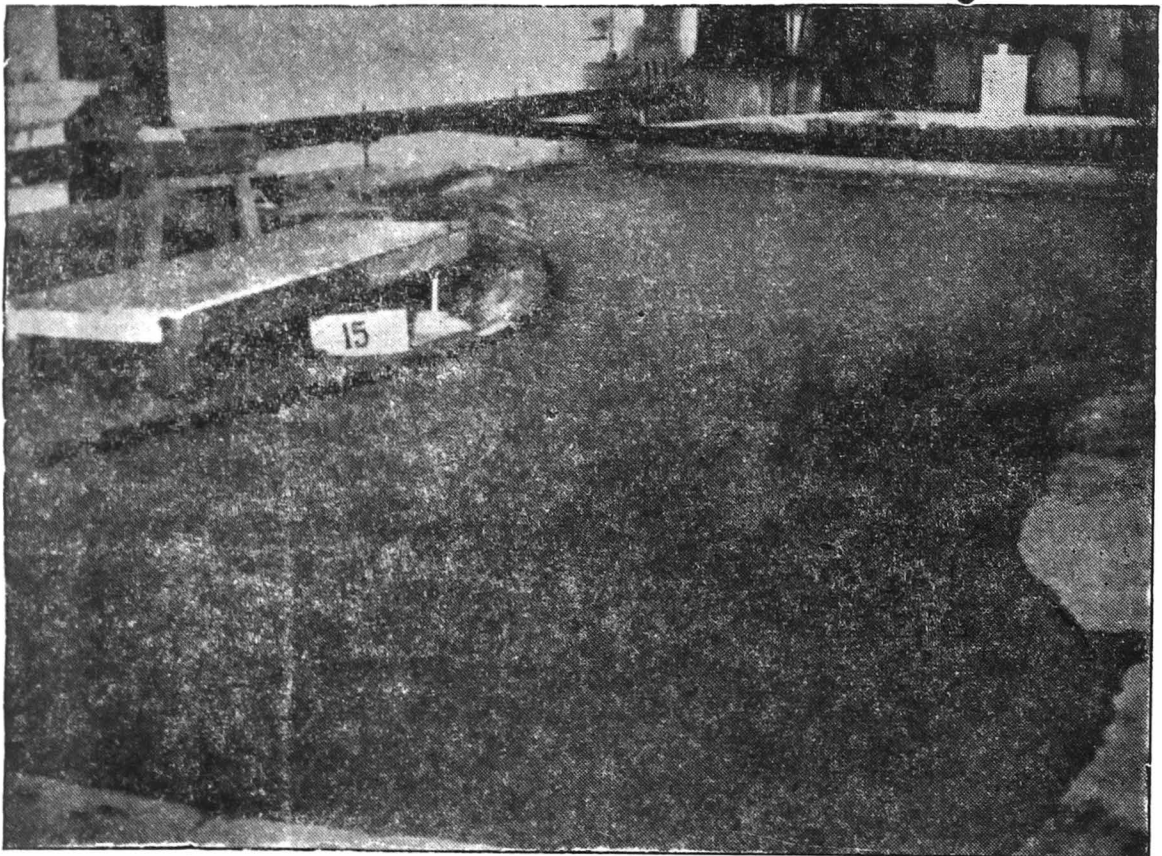


Figure 2 C. 27 : *Ramapadasagar coffer dam model 1/300—Photograph taken looking upstream. Alignment of west side coffer dam—Preliminary design—the number plate 15 is kept downstream of Central Hill.*

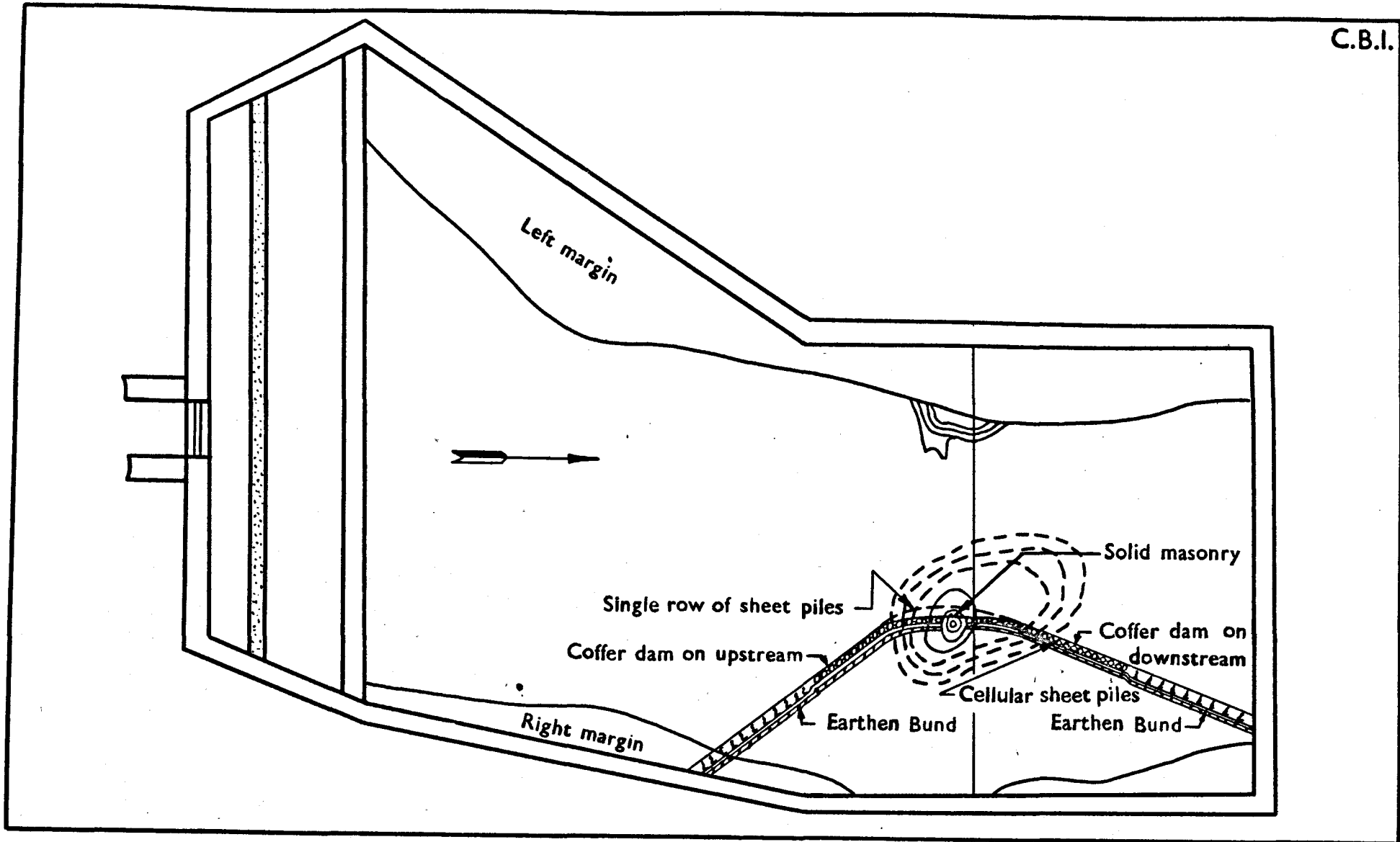


Figure 2C.26:-Showing alignment of west side coffer dam preliminary design [Ramapadasagar coffer dam model $\frac{1}{300}$]

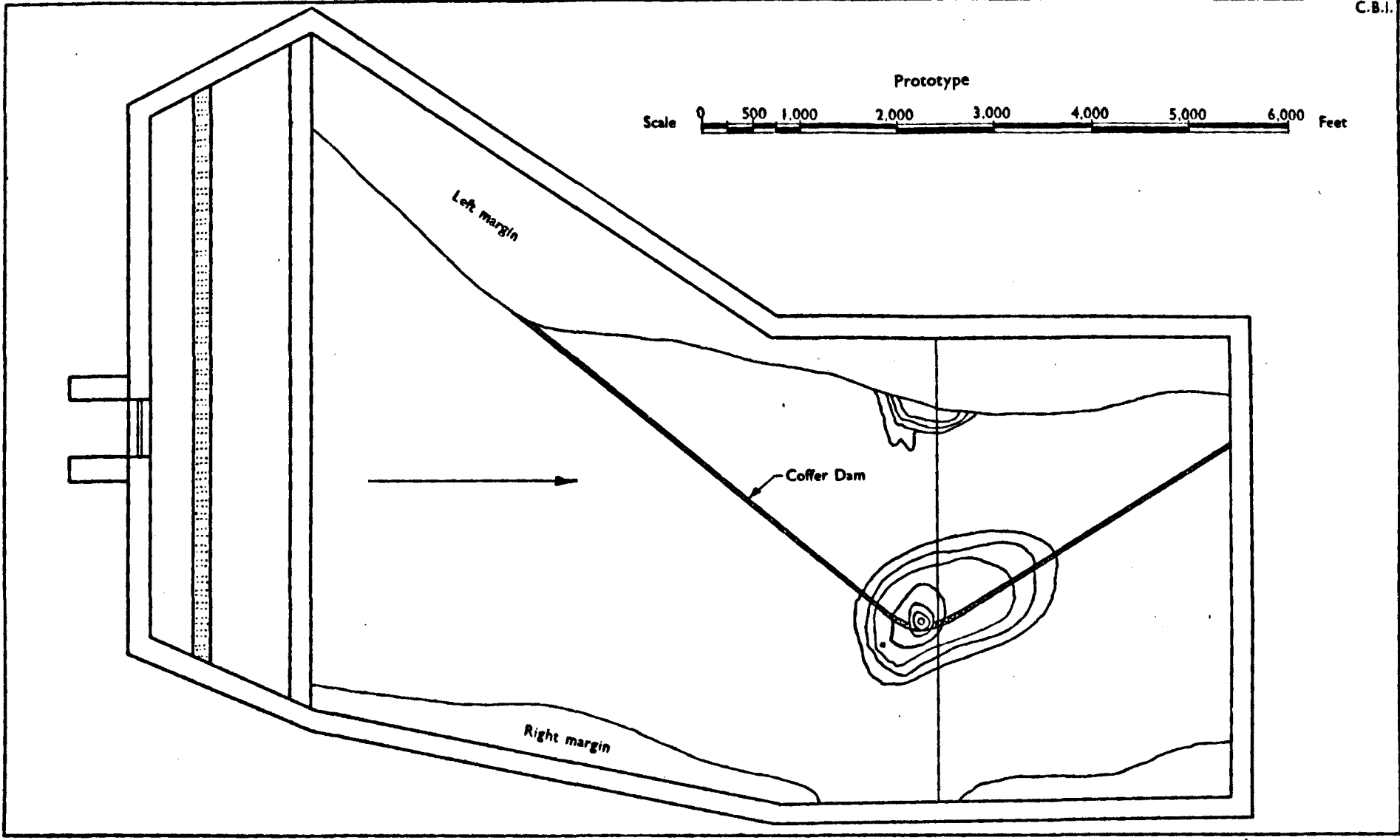


Figure 2C.29:- Alignment of east side Coffer Dam-preliminary design (Ramapadasagar Coffer Dam model $\frac{1}{300}$)

Reg. No. 316 HE (C) '49-1260 (H.L.O.)

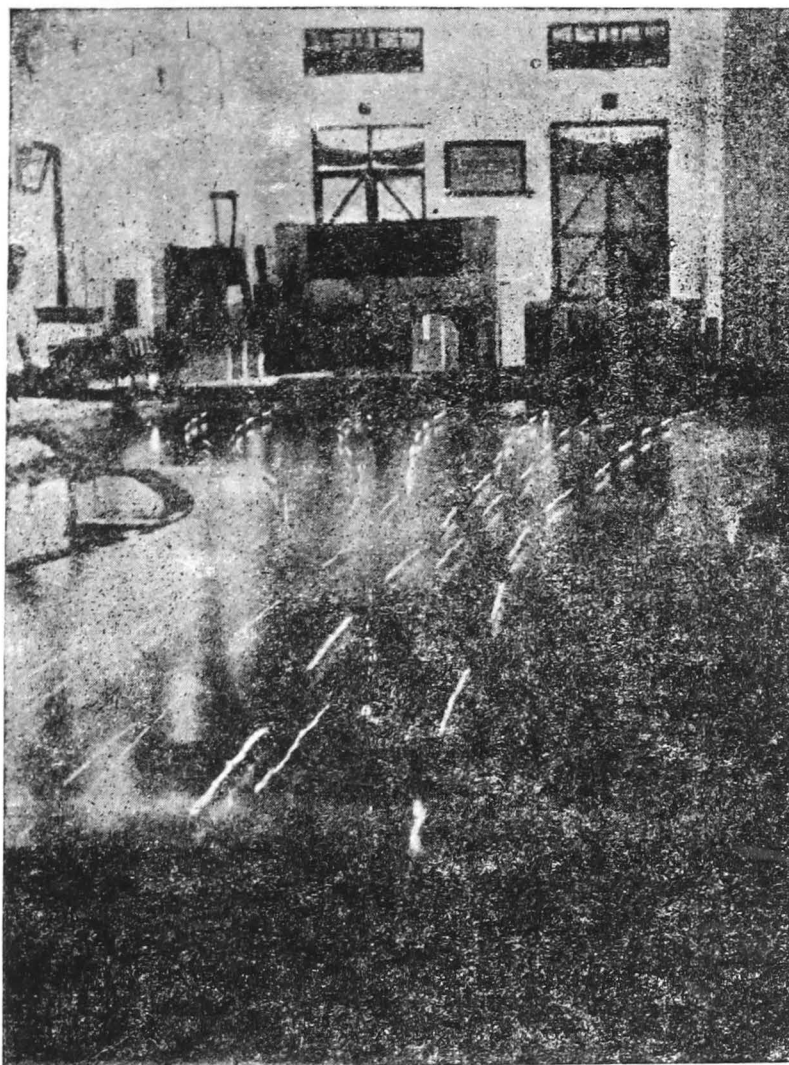


Figure 2C. 28: *Ramapadasagar coffer dam model 1/300.—Photograph taken looking upstream—west side coffer dam—Preliminary design—Flow pattern indicated by candle floats shown by dotted lines.*

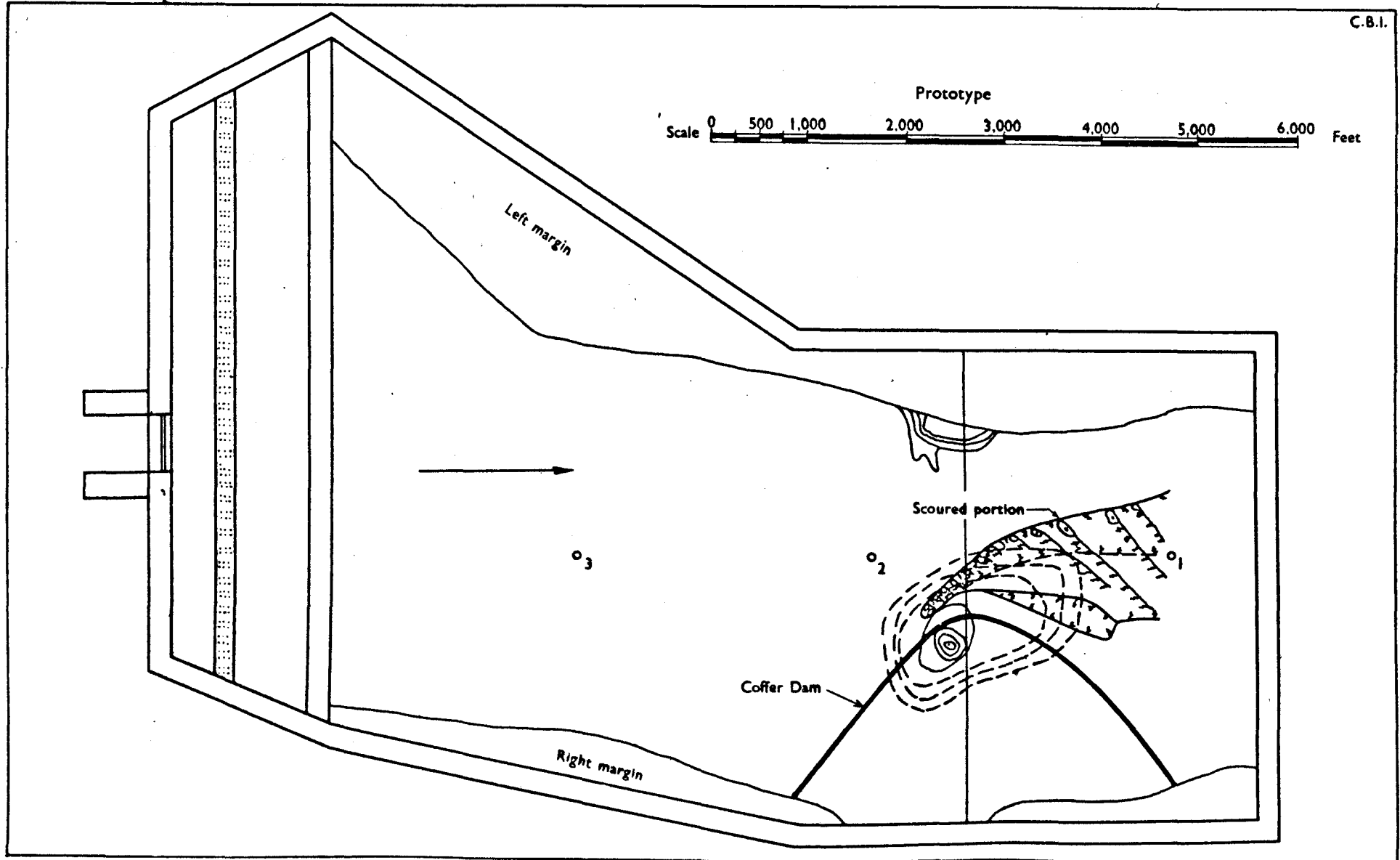


Figure 2C.30:— Alignment of the final design of the west side Coffer Dam showing scour after 4½ hours run (Ramapadasagar Coffer Dam model $\frac{1}{300}$)

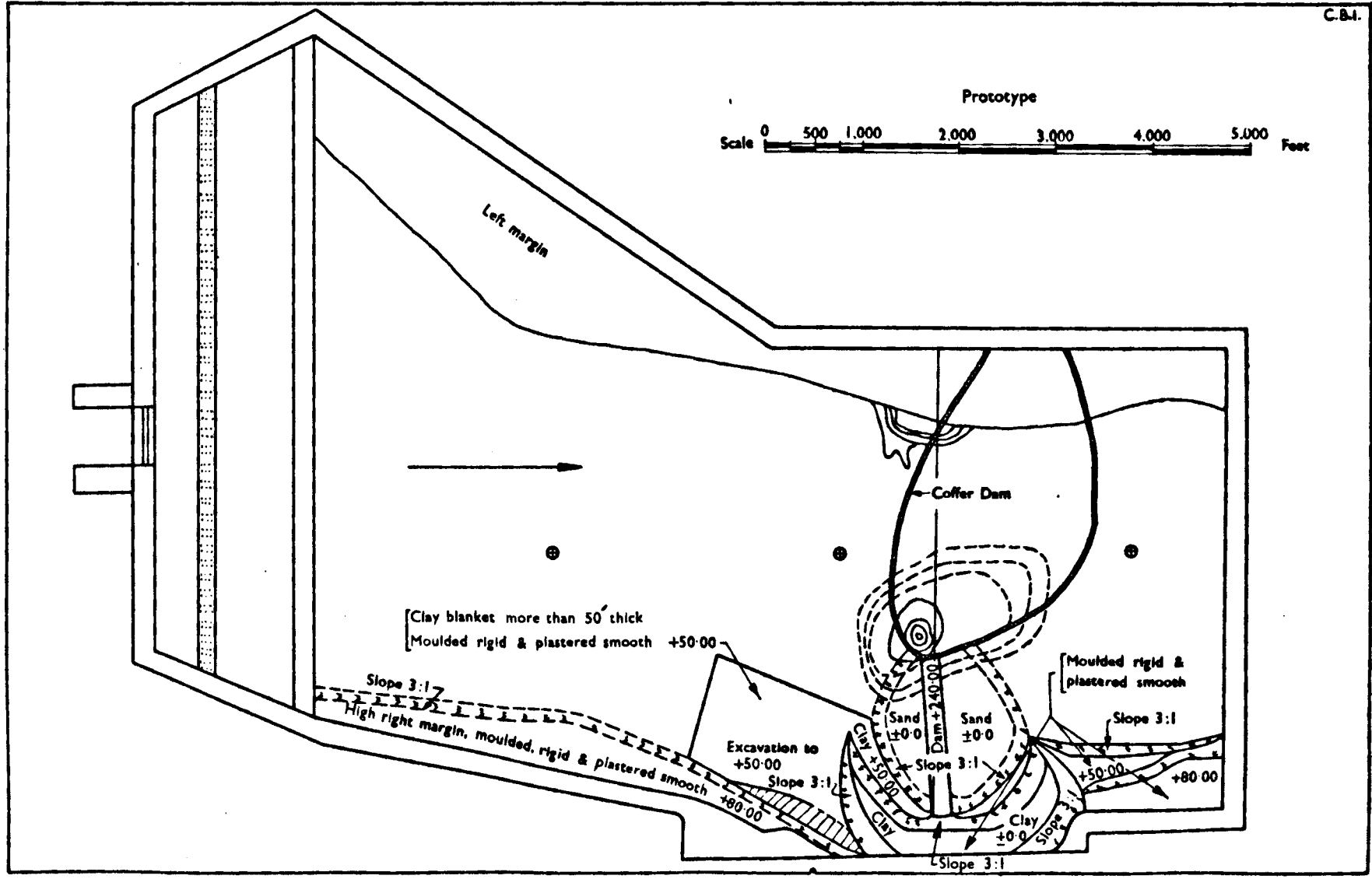


Figure 2C.31:— Alignment of east side Coffor Dam-final design (Ramapadasagar Coffor Dam model $\frac{1}{300}$)

TABLE 2 C-14

Comparative statement of water surface elevations for normal river and with west and east coffer dams.

N.B.—All figures relate to prototype.

Discharge million cusecs	Conditions	Water surface levels at			Remarks
		No. 1 gauge 4,000 feet up- stream of dam line	No. 2 gauge 1,000 feet up- stream of dam line	No. 3 gauge 2,000 feet down- stream of dam line	
2.25	Normal river ..	+92.7	+91.2	..	Water surface elevations, reduced to common datum as represented by No. 3 gauge reading.
	West side coffer dam	+97.2	+94.2	+90.0	
	East side coffer dam	+100.2	+97.8	..	
1.85	Normal river ..	+90.8	+89.6	..	
	West side coffer dam	+93.8	+91.4	+88.7	
	East side coffer dam	+98.0	+96.3		
1.36	Normal river ..	+88.0	+87.7		
	West side coffer dam	+89.4	+88.1	+87.2	
	East side coffer dam	+93.5	+92.3		
0.91	Normal river ..	+85.1	+84.8		
	West side coffer dam	+86.3	+84.8	+84.2	
	East side coffer dam	+88.4	+87.5		

(b) A similar design for the east side coffer dam was also tested. The alignment is shown in Figure 2C. 29. In this the current distribution was not so satisfactory as in the previous model. The deep course in the river at the dam site was on the left of the central hill and when this (left) half was cut off by coffer dam a sharp curvature was naturally induced. There was rapid current alongside the coffer dam and eddying near the nose on the hill and scouring took place at this point. This alignment also had the disadvantage that it was excessively long.

(iii) *West side coffer dam—Final design.*—Final studies on the model were carried out for testing two designs suggested by Dr. Terzaghi after his study of the topography and boring results. His designs for the west side coffer dam is shown in Figure 2 C. 30. The flow pattern obtained was satisfactory. The stream lines approached the constriction with a fairly uniform speed, crowding towards the centre of the open stream and then travelled down with increased velocity. The lines nearest the coffer dam face got deflected away towards

the centre of the river. There were no undesirable eddies. The water surface elevations for the various discharges tried are furnished in Table 2.C. 14. The rise in water level at the gauge 4,000 feet upstream of dam line was 4.5 feet for a discharge of 2.25 million cusecs. Scours obtained for a run of 4½ hours are shown in Figures 2 C. 30. There was no scour near the coffer dam face. Scour in the river-bed started about 500 feet away from the coffer dam nose (on the Mahanandi Hill) and developed towards the centre of the river. The depths varied from 20 to 40 feet.

(iv) *East side coffer dam—Final design.*—The design suggested by Dr. Terzaghi for the east side coffer dam is shown in Figure 2C.31. As this would be taken up after the right half of the dam is constructed up to bed level, the details of the changed conditions of the waterway on the right as might be expected are also indicated in the plan. The dam would be built up to bed level, say plus 24.00. The foundation trench excavated for the construction of the bit would get filled up to say plus 0.0 during the low floods. In order to provide additional area of waterway the bed would be lowered as indicated on the plan for some width along the right margin. In addition to this a passage 500 feet wide with its invert at plus 0 would be cut in the rock at the right flank. The model was built up introducing all these details carefully. The dam was built in masonry up to plus 24.0. Where removal of top sand was proposed, the bed was formed rigid and, plastered smooth to represent nonerodible clay. The foundation trench on right was filled up with sand up to plus 0.0.

The model was first tried without the 500 feet wide passage at the right flank and with a few minor alterations. Figure 2 C. 32 shows the flow pattern obtained on this model.

The study was repeated with the passage cut and the waterway arranged as shown in Figure 2 C. 31. This indicated an improvement in the flow lines. The sharpness of the curvature of the currents was eased by the diversion and the development of the main course was shifted farther from the coffer dam. Still there was some scouring near the coffer dam face on the central hill and downstream of it. The scour contours obtained after a 4½ hours run are shown in Figure 2 C. 33. Water surface elevations were everywhere higher in this than observed with the west side coffer dam. The values are furnished in Table 2 C. 14. For the discharge of 2.25 cusecs million the rise of water level at gauge 4,000 feet upstream of dam line was 7.5 feet.

The water surface elevations for the various discharges obtained with the final designs of the coffer dam are also shown in the graphs in Figure 2C. 34.

CONCLUSION

The final design tried for the west side coffer dam was generally satisfactory. The water surface 4,000 feet upstream was plus 97.2 for a discharge of 2.25 million cusecs. The top of the coffer dam should be kept above plus 100.0 to obviate over topping. No undesirable eddies were formed. Scours developed away from the coffer dam.

The final design tried for the east side coffer dam including the 500 feet wide waterway on the right flank was fairly satisfactory. The water surface 4,000 feet upstream was plus 100.2 for 2.25 million cusecs and the top of the coffer dam should therefore be kept above plus 103.0 to prevent overflow.

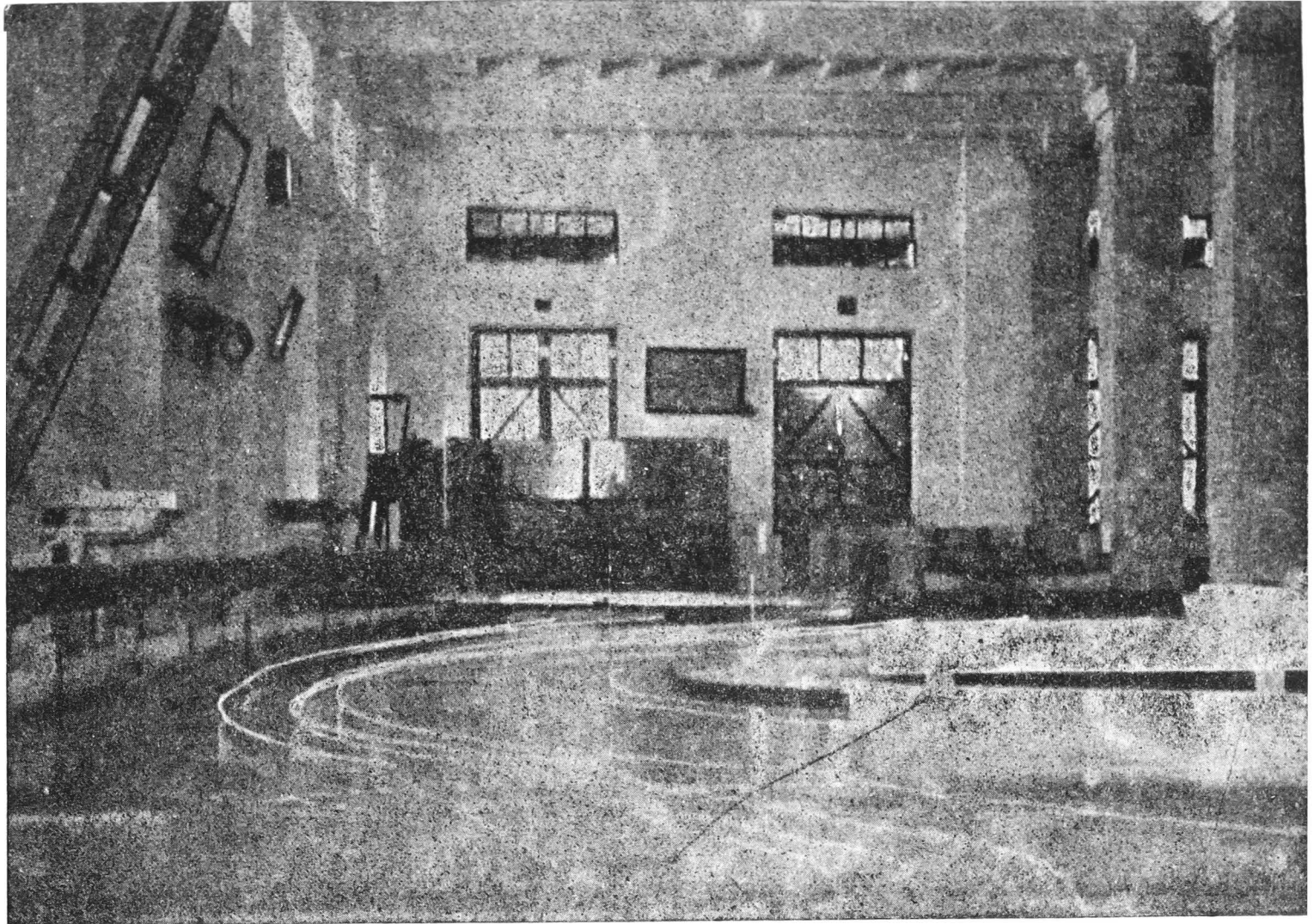


Figure 2C. 32 : *Ramapadasagar coffer dam Model 1/300. Photograph taken looking upstream East side coffer dam—Final design—flow indicated by candle floats shown by continuous lines.*

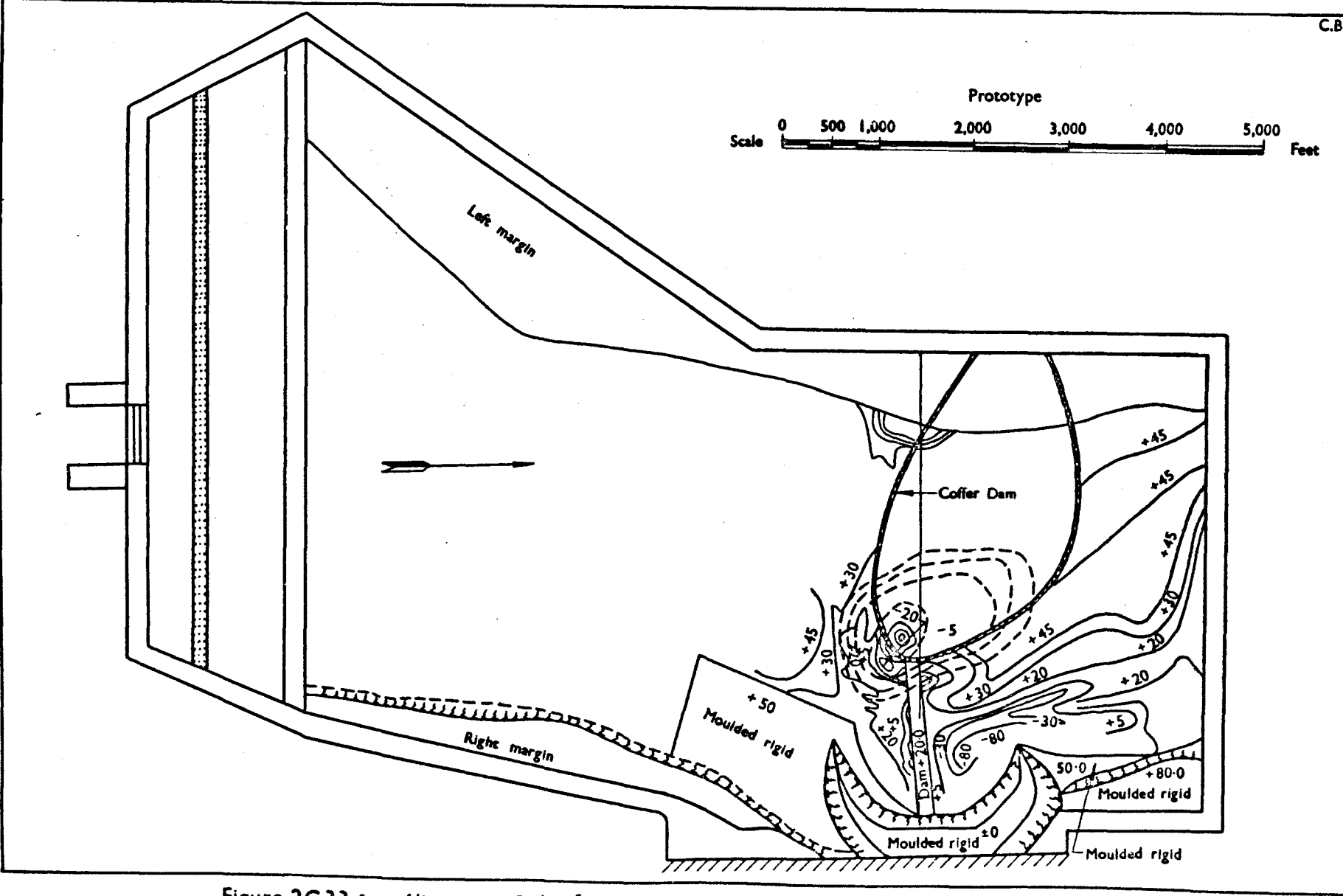
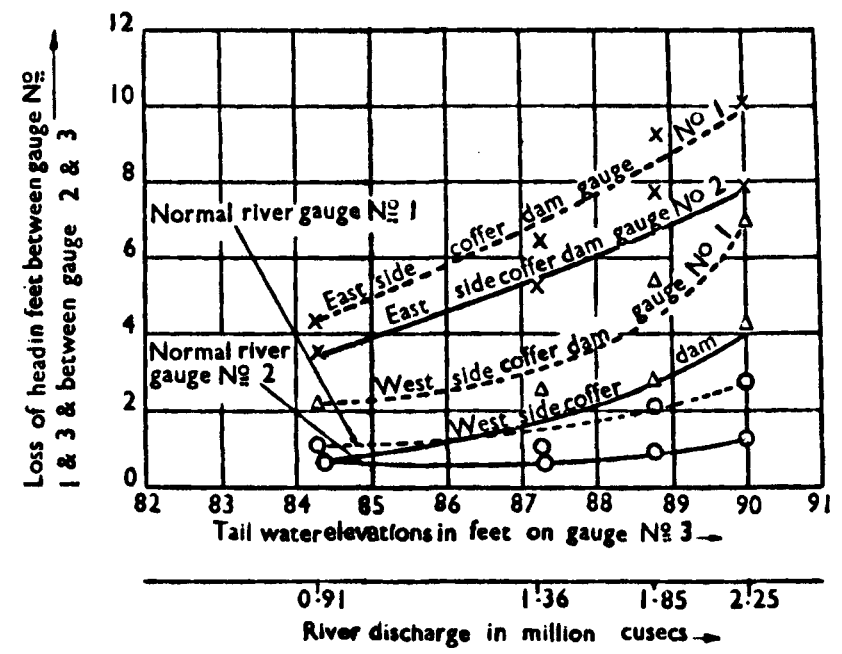
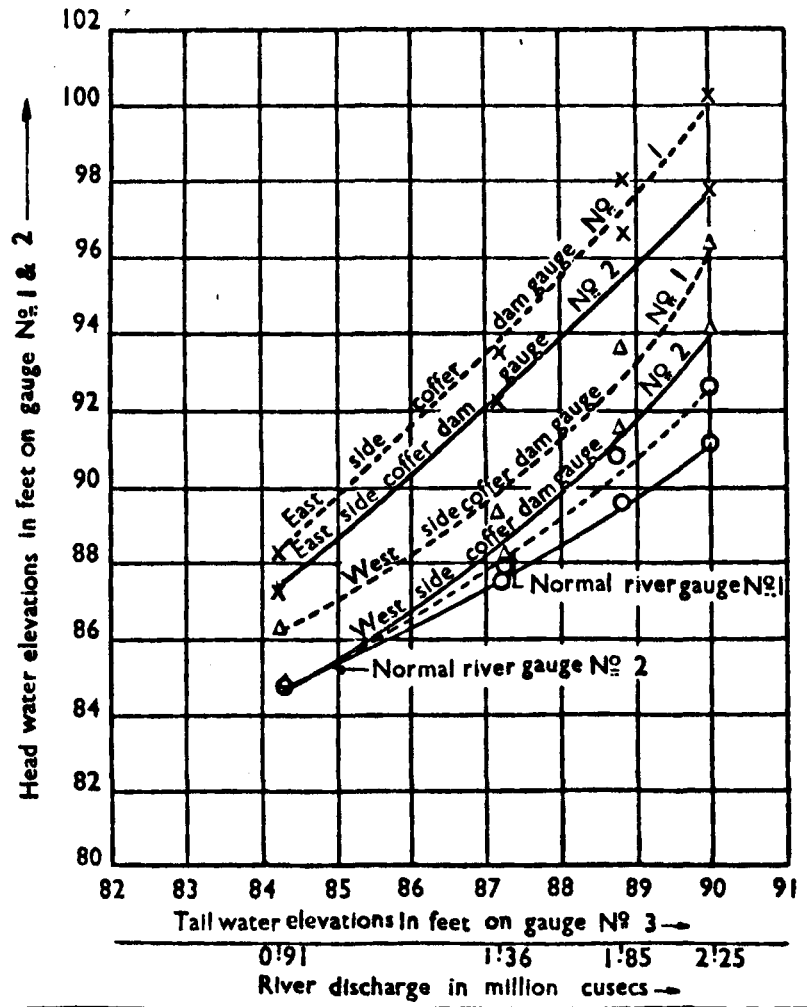


Figure 2C.33 : — Alignment of the final design of the east side Coffer Dam showing contours of scours after $4\frac{1}{2}$ hours run [Ramapadasagar Coffer Dam model $\frac{1}{300}$]



Position of gauges

Gauge No 1 4000 feet upstream of dam line

" " 2 4000 " " " " "

" " 3 2000 " downstream of dam line

Figure 2C.34:- Showing water surface elevations & loss of head between gauges

[Ramapadasagar coffer dam model $\frac{1}{300}$]

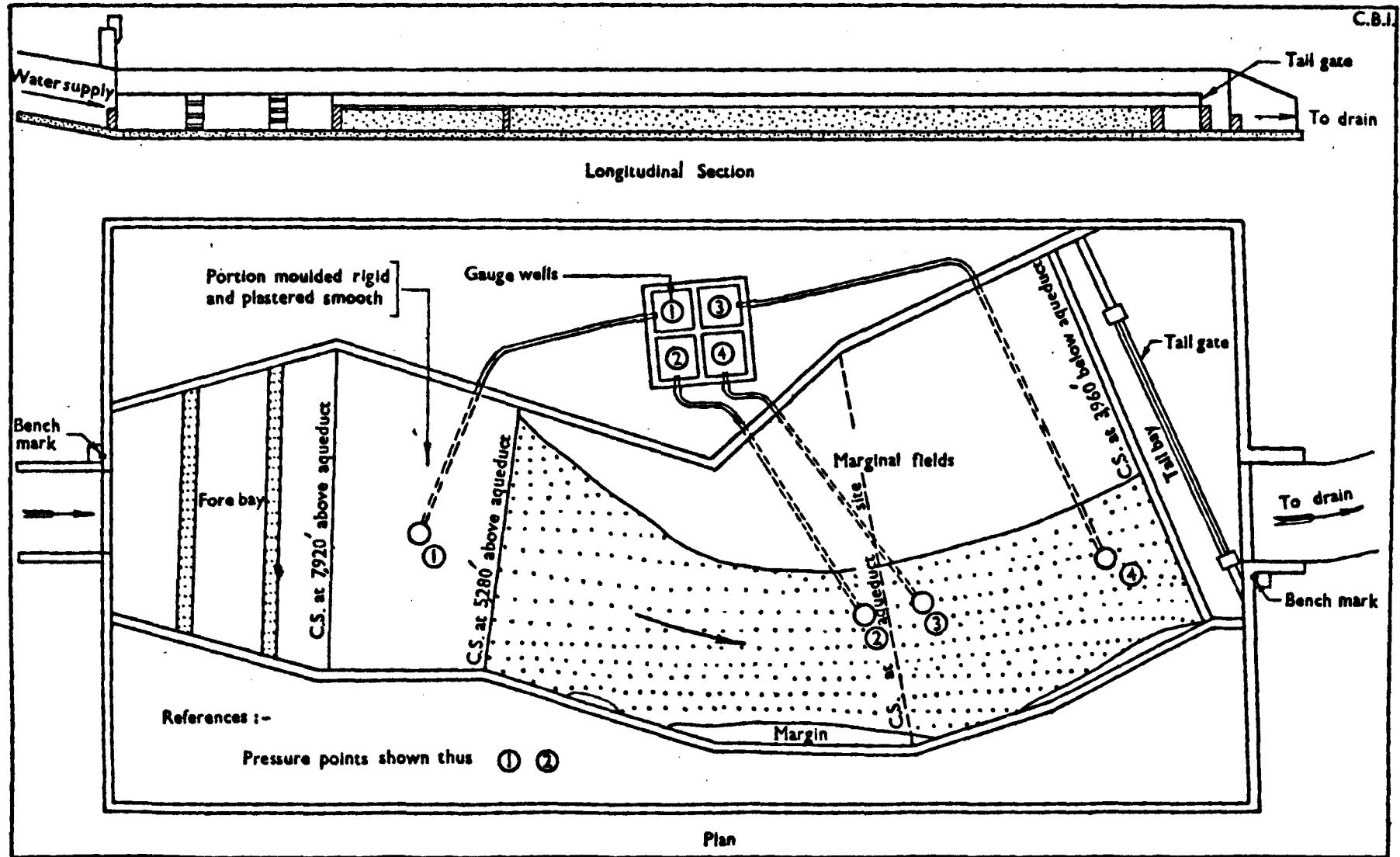


Figure 2C.35:- Showing layout of Hagari river model $[\frac{1}{400} \times \frac{1}{100}]$

Though the main scouring developed away from the work, there was scouring indicated near the same on the central hill and downstream. The design should take into consideration such scours going down to about 40 feet below bed. These inferences will be verified on the bigger model under construction.

(5) THE HAGARI AQUEDUCT FOR THE TUNGABHADRA CANAL— SCOUR ROUND PIERS ⁽³²⁾

ABSTRACT

The Tungabhadra low level canal is to cross the Hagari river by an aqueduct. This was proposed to have 73 spans of 30 feet each, with piers 5 feet thick having semi-circular cutwaters, founded on wells taken down to about 20 feet below river bed. No flooring or other protection was proposed for the pier foundations. Studies on the formation and development of scours around the piers and on devices to minimise the same if necessary were taken up with the help of models and this describes the first series of tests carried out on one model.

DESIGN OF THE MODEL

Scour round bridge (or aqueduct) piers is influenced by the nature of bed material shape of pier, discharge intensity and curvature of flow. It being impossible to reproduce all these effects in one model it was proposed to study the effect of curvature of flow alone on a full model in the first instance and for this purpose a vertically distorted scale was selected.

The only data available were (i) a survey of the river and cross sections, (ii) maximum observed flood discharge=150,000 cusecs, (iii) gauge reading corresponding to this 1,319, (iv) maximum computed, flood discharge=185,000 cusecs (v) gauge reading corresponding to this assumed as + 1,320, (vi) bed and surface fall=five feet per mile. The model was run only for maximum flood conditions. It had a mobile bed indicating local scour only and there was no injection of silt upstream.

Considerations of space and discharge available led to the selection of scale 1/400 horizontal \times 1/100 vertical though a much bigger model would have been preferred. Bed material was selected experimentally and by having recourse to tilting of bed, sieved sand of 24/30 mesh per inch was found suitable. With these it was found (a) that the surface fall was reproduced in the model, (b) that no movement of bed was noticed before the aqueduct was introduced and (c) appreciable scour was indicated after the aqueduct was put in.

CONSTRUCTION OF THE MODEL

The model was constructed in a tray which is 40 feet \times 20 feet \times 2 feet and has a concrete bed. The river was reproduced for a length 7,920 feet upstream and 3,960 feet downstream of the aqueduct, Figure 2 C. 35. The portion 2,640 feet long at the inlet end was moulded rigid and plastered smooth to preserve the approach conditions in the model. The non-erodible marginal lands were also moulded rigid. The remaining portions of the river were moulded in sieved sand 24/30 mesh per inch. A forebay with stilling screens, a tail bay and a tail gate were provided for ensuring smooth entry and exit conditions and correct water levels. The retaining wall between the model and tail

(32) Irrigation Research Station, Poondi, Madras, Annual Report for 1946, page 21 to 29.

bay was built porous to facilitate easy unwatering of the model at the conclusion of an experiment. The mobile portion of the river bed was moulded by using male templates cut in galvanized iron sheeting representing cross sections at intervals of three to five furlongs. Water supply was drawn by gravity from the static tank and measured over a precalibrated, knife edged weir. Suitable bench marks were installed to check the elevations of the bed and water levels. Four pressure boxes were buried underneath the sandy bed at points 6,000 feet and 300 feet above and 300 feet and 3,000 feet below the aqueduct site and were connected by piping to four gauge wells located in one spot. The water levels were read off in these wells by point gauges reading to 1/1,000 feet.

The canal embankment was constructed in cement mortar and plastered smooth and the aqueduct piers were made of well seasoned and polished teak-wood.

OPERATION OF MODEL

The tail gate was lifted and water admitted at a very slow rate so as not to disturb the mobile bed due to initial onrush of water. The model was thus flooded to a suitable level and all leaks closed. The static water level in the model was determined by referring to the bench marks. The pointer gauges were then read and could thus be inter-related. Water was then admitted to the model corresponding to the maximum flood discharge with the tail gate still in position. After the water level in the model was about the same as maximum flood in the river the tail gate was gradually lowered till under steady conditions the necessary maximum flood level was obtained in the tail reach as read by the pointer gauge. The model was run for one hour corresponding to 40 hours in the prototype. The surface flow pattern was studied with confetti. The model was unwatered and the scour pattern studied.

DESCRIPTION OF EXPERIMENTS.

Altogether 12 experiments were carried out. Of these, the first three were preliminary tests for selecting the scale, bed material *etc.*, and for proving the model. Subsequent studies were as below:—

(i) The model was run without putting in the aqueduct. Discharge corresponding to 185,000 cusecs was passed and maximum flood level corresponding to + 1,320 at the aqueduct site was maintained. A surface fall of five feet per mile was reproduced in the model and no scour was formed.

(ii) The experiment was repeated with the aqueduct constructed in place. For maximum flood level conditions, the flow was not confined to the mobile bed, but spilled over to the marginal fields on the left side. The aqueduct was located in the mobile portion of the river bed only and the canal embankment spanned across the marginal fields. This embankment prevented any flow along the fields and induced a cross flow in the spill and consequent eddying near the left abutment. Scour developed immediately at this abutment and extended up to the eleventh pier towards right. The scour was about 0.1 foot=10 feet deep, Figure 2. C. 36.

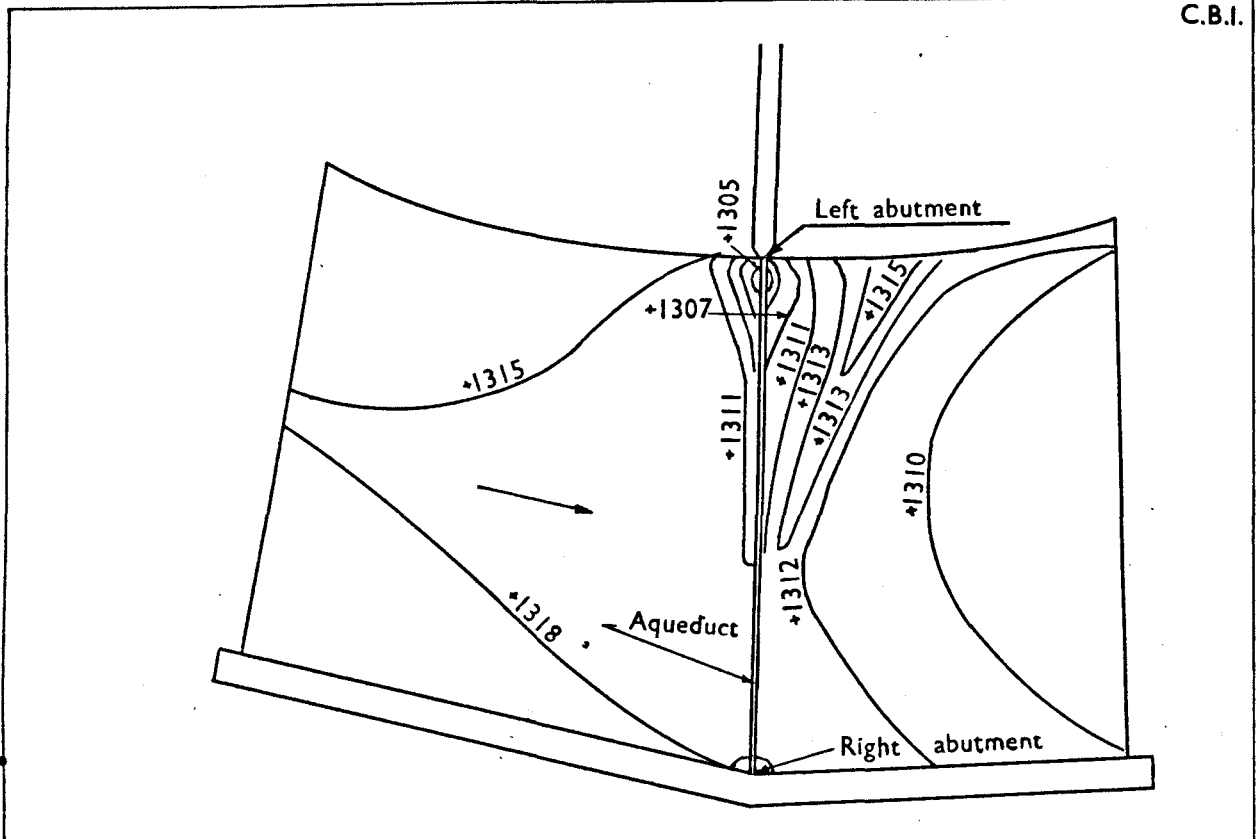


Figure 2C.36:- Showing Scour contours with aqueduct in position [Hagari river model]

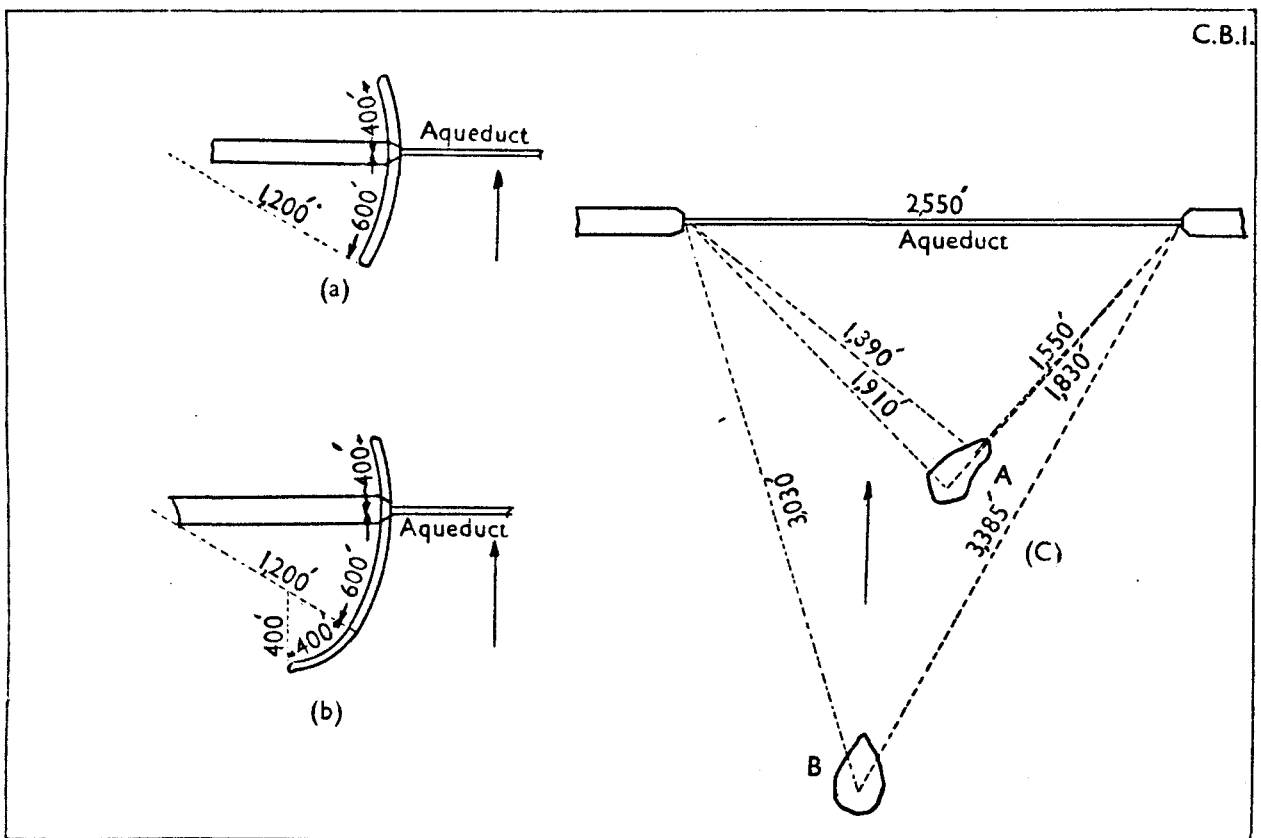


Figure 2C.37:- Showing Training bund and pitched Island [Hagari river model]



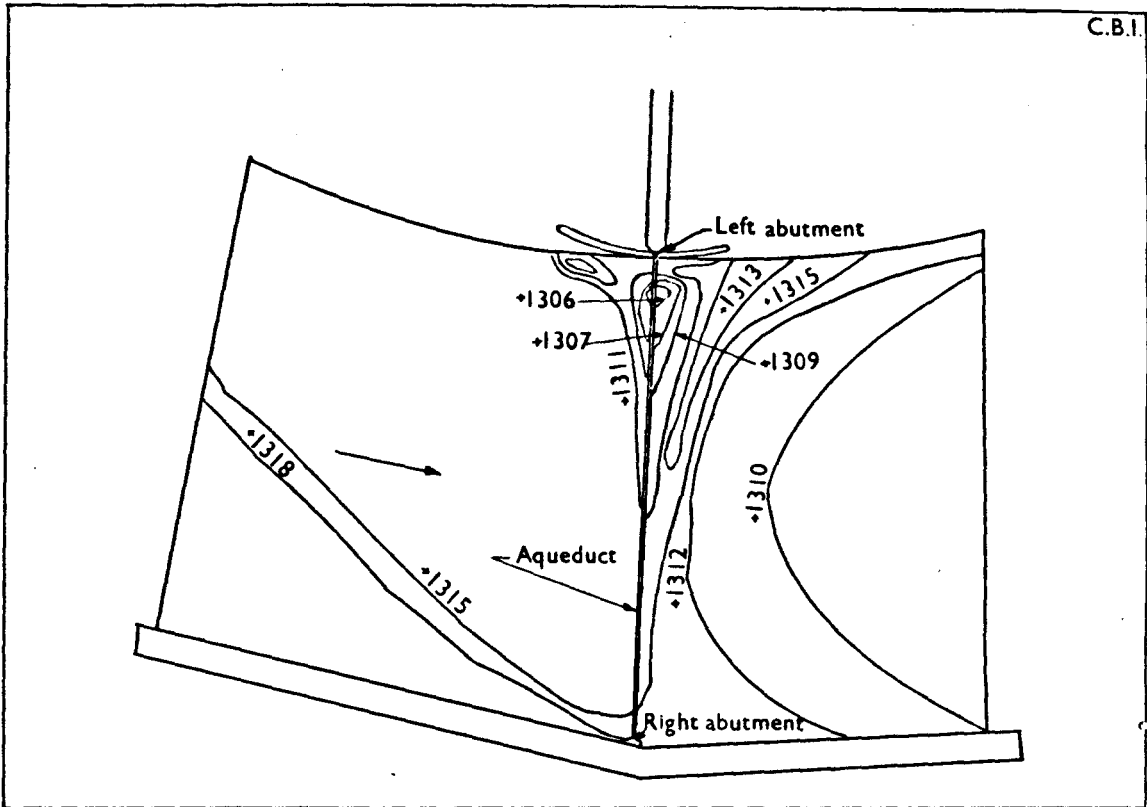


Figure 2C.38:- Showing Scour contours with Training bund (1st design)
(Hagari river model)

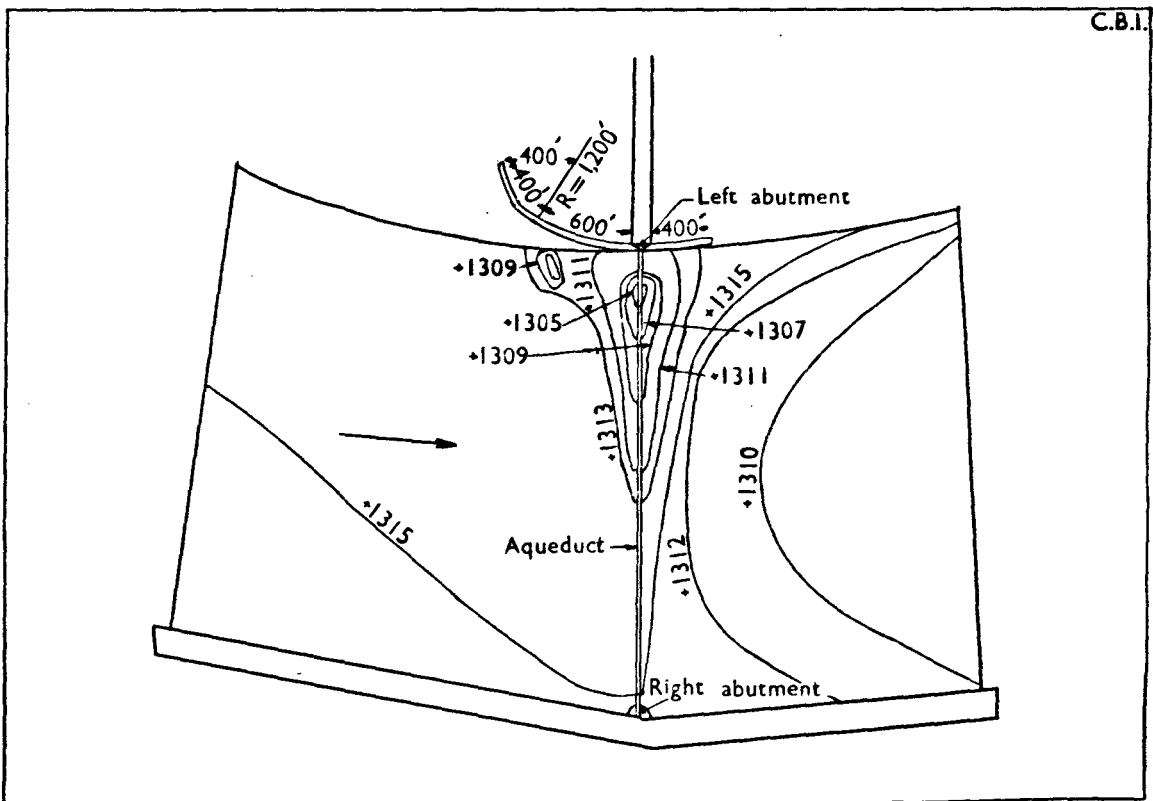


Figure 2C.39:- Showing Scour contours with Training bund (2nd design)
(Hagari river model)



(iii) In order to eliminate the sharp cross flow at the left abutment training bund of the dimensions indicated in Figure 2 C 37 (a) was next put in and the experiment repeated. The scour got slightly shifted away towards the right, Figure 2C. 38.

(iv) The training bund was then extended upstream as shown in Figure 2. C. 37 (b) and the experiment repeated. This had the effect of mitigating the scour to some extent. The scour started upstream and developed away from the abutment, Figure 2C. 39.

(v) With a view to distributing the flow more evenly into the spans and to obviate the scour crater on the left, a pitched island was constructed in the portion indicated by 'A' in Figure 2C. 37 (c). This had the effect of attracting the flow towards itself and producing a more even flow towards the aqueduct but the magnitude and extent of the local scour near the left abutment remained unaffected.

(vi) A second pitched island was then added upstream of the first as indicated by 'B' in Figure 2C. 40. This produced no improvement.

CONCLUSIONS

(a) For the conditions tried, viz., maximum flood discharge of 185,000 cusecs, which produced considerable spill over the left marginal fields, there was cross flow and eddying at the left abutment causing harmful scour.

(b) The scour could be shifted away from the abutment and its magnitude reduced by means of a training bund located on both sides of the abutment.

(c) A pitched island located upstream of the aqueduct and slightly towards the right of the axis of the stream had the beneficial effect of ensuring a more even flow towards the aqueduct, but had no influence on the local scour at the left abutment.

DISCUSSION BY THE RESEARCH COMMITTEE

MR. FRAMJI, introducing item (1) said that when model experiments were originally made for the approach channel, it had been anticipated that there would be deeper scour at the nose of the approach channel on the river side; but it had not been anticipated that on the approach channel side the scour would be as extensive as has actually occurred. This scour had caused strong return flow in the approach channel along the outer bank and slack return flow on the right bank and high velocity flow in the centre; with the result that the scour extended up to the right bank. When the pond level was dropped the berm slipped into the scour-hole. Filling of the scour-holes by stones was actually tried on the model also and was promising but was very costly and figures show that the Station's proposals were much cheaper. It is believed that if the approach channel bank is extended so as to allow greater width of entry as well as to have a flatter slope to the outer bank nose then the scour would not extend up to the point where the right bank could slip in. This was tried in the model and found successful; but it would be a job to try and extend the approach bank across the scour-hole at the nose. The model, however, showed that if only the truncated nose of the required extension was constructed beyond the scour-hole the results would also be effective.

Introducing item (3) he said that they had been collecting certain data through the good offices of the North Western Railway and the data they had been supplied gave only the scour at the piers. The actual maximum depths of scour occurred further downstream and this really required a contour plan of the scour for the purpose of scientific correlations but as contour plans would be difficult to get, the station had asked for cross-sections at intervals of 150 feet in the reach where the maximum scour was expected to occur.

Mr. S. T. GHOTANKAR introducing item (2) said that the design of the Mahanadi Central Sluices consisted of a flat pavement followed by a very flat glacis causing hyper-critical velocities. When on the falling floods, weir crest shutters were raised and the whole flood discharge of the order of 50,000 cuseca was passed down the sluices, this caused severe scour downstream. After the 1944 floods staggered blocks were fixed but were ineffective and caused more dangerous erosion, because they did not withstand the hyper critical velocity and were unevenly dislodged.

On inspection by the Director the design recommended was a five feet high glacis followed by baffles and a deflector with a 1 in 3 loose stone apron downstream of it. The divide walls were also asked to be raised so as to prevent cross currents from the sluices spilling on them. The design had worked completely satisfactorily.

Mr. M. P. MATHRANI referring to item (1) wanted to know whether these results were brought out when the first model was worked. If they were not, what were the results? As Mr. Framji had pointed out, the river bed went on changing every year; short entry conditions gave short time results. The scour did not occur in the first or second year but in the third year. The first thing that should be done was to bring about this changing course in the model. What was actually happening was that the accretion was causing silt islands. He felt that unless the results were brought out for all the years since this model was made, that was to say river course changes from year to year the results now obtained might not be conclusive. So unless the experiments were carried out with the river bed changing from year to year for about eight or ten years, the mere construction of the nose as proposed might produce danger somewhere else. He liked to know whether this model test had been carried out with the existing conditions of the river. He also requested Mr. Framji to explain fully as to how this model test had been carried out.

Mr. H. P. MATHRANI explaining with sketches on the board said that the proposed extension of the nose might cause much restriction of flow which would cause more heading up and finally worsen the conditions at the barrage.

Mr. SETHNA said that during winter when canals run full, silt was being carried from the scour hole, and there was a reverse flow at the place. This might be due to widening of the section. Had this been taken into account in the model?

Dr. Bose enquired if these experiments were carried out previously in a part-width model also. He thought that the difference in behaviour of the model and the prototype was due to the model experiment having been carried out in the part-width model. In the part-width model it was extremely

difficult to reproduce the current directions and intensity at all points of the model. Specially in those cases where the bed currents and the surface currents followed different directions, the reproduction in the part-width model was almost impossible.

MR. CHATURVEDI asked whether the effect of the groynes had been taken into account.

MR. FRAMJI, replying to Mr. M. P. Mathrani, said that his apprehensions were unwarranted in this case because the entry conditions were fixed by the Bukkur gorge—so the entry length used, up to and beyond the gorge, was quite sufficient. From the detailed annual surveys of the river above the Barrage the worst meander course, as most affecting the scour holes, return flow and slipping, was determined and used for further experiments. Mr. Mathrani had referred to the islands formed in the river by accretion. The model reproduced these islands and where necessary to impose more severe conditions, these islands were exaggerated in the model.

In reply to Mr. H. P. Mathrani, MR. FRAMJI said that the proposed extension was aligned to preclude any possibility of restriction or masking of waterway. It had been stressed in the Report that there was no additional afflux the additional length of the approach being more than compensated by flaring the approach.

About Mr. Sethna's suggestions of training the approach channel by spur, MR. FRAMJI referred to the Report where an exhaustive account was given of the different types of spurs used and the insufficient results obtained therefrom.

Regarding Dr. Bose's question, MR. FRAMJI said that experiments were done in part as well as full width models, both now and before. So long as entry conditions in the part-width models were imposed as obtained from the full-width, correctly exaggerated models, and both had been 'proved' to simulate correctly the conditions on the prototype, the part-width model is not only unobjectionable but is often necessary to give detailed conditions in a given area.

THE PRESIDENT asked at what period of the year did these slips take place.

MR. FRAMJI said that slipping occurred in two years somewhere about November or December, in one of these years the pond was suddenly lowered.

MR. M. P. MATHRANI asked Mr. Framji, "when you say that you have carried out model tests under the worst conditions, is accretion not going to take place with the result that what is happening may continue further? When the river is very low will that accretion stop or will it still continue still raising the bed level. If the accretion is going to take place will there be any further trouble?" The other thing he wanted to know was about the extension of the bund by 500 feet length. "Will it not create a new flow channel which may strike the bank and begin scouring?" He wanted to know whether all these factors had been taken into account and whether really worst conditions were depicted in the model.

MR. FRAMJI replied that when he had said the worst conditions had been tried, he had referred principally to the meander conditions. The point now raised was whether additional discharge might not come down this channel, striking the bank obliquely and causing scour at its toe and slips. The model showed that it was unlikely, probably because the flatter slope to the nose on the approach channel side would decrease the attraction of flow into it; nevertheless, the model had been tested for the conditions in respect of discharge also, both for present conditions and for potential worst conditions.

MR. H. P. MATHRANI said that he had allowed in the model the maximum local discharge to come in. So far as he knew it was limited to the discharge going into this channel. This approach channel was very closely regulated and the discharge into it was already restricted. Further increasing the length of the wall might block more spans of the barrage and might cause greater damage to the barrage.

MR. FRAMJI replied that it was not clear to him why additional afflux should be feared when it is well known that head can be recovered by fluming. As far as the model could show, he could say that the flaring appeared fully to counteract the effect of an addition of about 500 feet in the existing length of over 5000 feet. The alignment of the proposed extension took care that waterway of the Barrange was not masked.

THE PRESIDENT enquired whether it was not better to strengthen the banks rather than lengthen the wall.

MR. H. P. MATHRANI said that there was already stone pitching for the banks in that section.

MR. KUTTIAMMU introduced item (4) and said that in the Ramapadaagar project a spill way dam was proposed across the Godavari. This would be about a mile long, having a hockey stick alignment, the two sections meeting on a hill in the centre of the river. The river bed was sandy and foundations were to go down about 150 feet below the river bed. The proposal was to coffer dam one half of the river for construction work allowing the river to flow in the other half. The central hill provided the anchorage for the coffer dam. This coffer dam would consist partly of cellular sheet piles driven about 100 feet in sand. The stability of these piles depended on the proper design of the coffer dam alignment. Various particulars relating to this aspect of design were examined on a 1/300 model constructed inside the laboratory. With the right side coffer dam the current deflected away from the nose of the coffer dam and scour developed towards the middle of the channel. There was no eddying or scouring near the cellular piles. The east side coffer dam design was not quite satisfactory. The current kept hugging the coffer dam in its tendency to flow towards the deep course of the river which was on the left. There was bad scouring near the coffer dam. By providing extra water way, by having a cut in the right flank rock, this could be somewhat improved.

These experiments were done on a 1/300 natural scale model. So movement of bed for normal river flow was not reproduced. This had given exaggerated values of afflux and the shape and depth of scours were not also correctly indicated. But as a pilot model it gave valuable results required for design purposes. To get more accurate values a larger model on 1/150 scale was being worked now with slope exaggeration and silt injection.

Introducing item (5), he said that it was distorted model 1/400 : 1/100 but movement of bed before putting in the aqueduct was obviated and injection of silt avoided by tilting of bed. The aqueduct was proposed to be constructed across the river at a bend. In this reach there was a spill on the marginal fields and regular flow along these fields. This flow was stopped by the new canal embankment and consequently sharp curvature of flow and eddying were created near the left abutment and deep scours were developed. This could be minimised and thrown farther away from the abutment by having a training bund as shown in Figure 2C 39.

MR. FRAMJI said that in the Ramapadasagar model only surface flow lines and surface eddies appeared to have been studied. In coffer dam design surely it was the direction and magnitude of bed flow that were important. Surface eddies were not dependable indications of bed disturbances. It was the bed flow and bed velocities that should have been investigated, at the maximum discharges up to which the coffer dams would remain *in situ*. Under the existing conditions without a coffer dam there must be bed-movement in the river. Hence the first condition adopted in the selection of sand, namely that there was no bed movement without coffer dams, was incorrect and, therefore, no great reliance could be placed on the development and pattern of scour with coffer-dams. A scale of 1/300 for depth would give for the maximum river depth of 65 feet, a depth of 0.22 feet in the model; for normal summer depth of 20 feet, only 0.07 feet; and for the full range between low water and high water gauges of 27.5 feet, an elevation of 0.09 feet—thus it was hardly a suitable scale for experiments on afflux and scour where depths were the principal parameters.

It would be better MR. FRAMJI said, if instead of proposed 1/150 model there were two models, one with vertical exaggeration to study bed pattern and then a second large scale geometrically similar model to get afflux with bed pattern reproduced.

The duration of flow of various magnitudes of discharge in the river should be studied to determine the worst conditions to which the coffer dam would be subject, unless during the period of 4½ hours after which measurements for scour had been made, the model had stabilized to its maximum scour depth and extent.

With regard to the Hagari Aqueduct for the Tungabarda Canal, MR. FRAMJI said that his criticisms of these tests were the same as made for the tests on coffer dams of the Ramapadasagar Project, namely (1) the direction and magnitude of bed flow—all important to scour—did not appear to have

been investigated ; and (2) model was designed so that there was no bed movement for existing conditions before the aqueduct was introduced ; in all model work, it was essential to simulate bed movement throughout the model by correct bed slope and surface slope exaggeration.

Two additional defects might be mentioned ;

- (a) Insufficient entry and exit length ; without the former, correct curvature of flow could not be expected in the reach under investigation ;
- (b) It was only permissible in geometrically similar models to have the same number of piers in the models as in the prototype, even when allowance for excess friction had been made in the model. In vertically-exaggerated models, unless the number of piers was small, experience at Poona had shown that the number of piers should be reduced to have approximately the same width : depth relation of waterway as in the river, preferably having the number of piers in the model even or odd as in the river.

As a consequence, the curvature of flow had not been fully developed in the model and whatever curvature was reproduced in the model was counteracted by the reproduction of the full number of piers ; and this probably had deflected flow more to the left causing more intense scour near the left abutment.

Nothing had been mentioned, concluded Mr. FRAMJI regarding the stated main purpose of the experiment, namely the formation and development of scour around the piers.

Dr. BOSE said that according to the report, the length of the model taken compared to the width of the river was rather small. It should have been much longer. Secondly, the scale chosen if one wanted to make a geometrical model, should be at least 1/150 to facilitate taking any readings. That was why no sand movement could be observed.

It had been stated that pressure readings were observed in the middle of the river model by means of pipes put on the bed of the model and those compared with the readings taken in the river. Dr. BOSE enquired if the gauges were read in the middle of the river in the prototype. If they were not read in the centre of the river why were they taken in the centre in the model ?

Col. BLENCH agreed with the principles enunciated by the previous speaker Mr. Framji regarding depth and width.

Dr. RAJU agreed also that the scalar model was too small with a discharge of $2\frac{1}{2}$ cusecs. The bigger model with a discharge of 12 cusecs would work better.

Mr. GOVINDA RAO said that there was a rocky out crop on the left bank. He enquired whether these did effect the pattern of flow and scour.

DR. UPPAL observed that one important point was that the scales were too small *viz.*, 1/300. For such work one wanted distorted models. If they wanted to have a model of this size, they should use materials such as cinder and coal instead of sand. The result then could have been obtained far better. He asked Mr. Framji why he advocated changing number of piers in distorted models. Instead the friction co-efficient could be changed by changing the bed material and the same number of piers could be kept.

MR. FRAMJI replied that the number of piers were required to be changed not only for rugosity, but to keep the width to depth ratio correct which is impracticable with a vertically-exaggerated model unless the number of piers is small.

MR. CHATURVEDI enquired as to what were the arguments of Prof. Terzhagi for the change of alignment of coffer dams.

Replying MR. KUTTIAMMU said that he had already agreed that the scale of the model *viz.*, 1/300 was too small and could not give accurate reproduction of afflux, scour *etc.* It was only a pilot model that was constructed inside the laboratory. They had to be satisfied with it and get a few results very quickly, and the model did give much information required for the design. This was in 1946. Since then they had constructed the bigger model 1/50 in one of their outdoor sheds and that had given them better results. In this bigger model they were taking the direction of bed currents also. They had found that they kept more or less parallel to the surface currents. As regards Dr. Bose's remarks about the approach length, the first two cross sections in the model represented a reach where the flow was more or less straight. Even so, they tried to create artificially by means of vanes *etc.* certain alterations in the direction of entry, but found that it had no material effect on the flow near the coffer dam. As regards bed movement, they proposed to reproduce the same by tilting the bed and injecting silt. When these results became available they would know the comparative benefits of the several models.

MR. Govinda Rao had referred to the projections at the left margin. These were rocky. They did have considerable effect on the development of scours. They had now obtained very accurate contours of this rock and were examining the effect of removing these in the very first instance.

Mr. Chaturvedi had enquired why the alignment shown in Figure 2C. 26 was changed to that shown in Figure 2C. 30. The former showed the alignment fixed after the preliminary survey. At that stage they did not have much data about the sub-soil conditions namely the levels of clay and rock. Based on the data available at that time the coffer dam was aligned along a ridge line of subsurface rock. Since then several hundreds of penetrometer reading tests had been made and fuller data giving contours of rock, clay layers *etc.* became available. A more economic alignment was then worked out, which resulted in the alignment shown in Figure 2C. 30.

DEWAN BAHADU GOVINDARAJA AYYANGAR agreed that no doubt the size of the model they used was too small, but once when they could produce some of the scours in the small model in-side on a laboratory basis, they thought they could expand it. They wanted to study preliminarily on small models. Even for this small scale model they had no proper sheds which were necessary under Madras conditions of weather. He did think that the scours were being reproduced pretty faithfully, and that morning when he heard that it was a geometrical model, that was likely to reproduce quantitatively and qualitatively more accurate results, he thought that more accurate results might be given by their geometrical models. It was suggested to Mr. Kuttiammu that they could even reproduce the actual scours that they have noticed in the river by changing the size of sand for forming the bed and various layers so that by passing those particular discharges, they could reproduce quantitatively the scours that they noticed in the river. They had not got even 1/100th of the data that they should like to have of the past conditions of the rivers, and so how could they reproduce them in the models? That was the case with Godavari also. Investigations had been started only 2½ or three years back. The rock out-crop mentioned on the left was very small. They have decided to remove that for better streamlining of the river on the left side. They also changed the alignment of the dam, because the left side rock was not very hard. They had contours of the clay in the bed of the river. The design of the coffer dam had not been finalised. It was being done and it was being changed to suit the levels and soils. The levels at which the different types of soil such as sand, clay etc. which go to form the over burden in the river bed, and the details regarding the levels and the thicknesses of the clay lense occurring in the sandy bed of the river at the sight had since been determined by having 200 to 300 numbers of penetrometer soundings, which in some instances had been taken to depths as large as 200 feet. The change in the alignment of the coffer dam had been made so as to make the best use of the clay lense and to avail of slightly higher rock levels so as to reduce percolation. Further exploration work was being done, and the dam alignment would be modified in the best manner possible.

Other ideas that they had were that they could put up a cross bund about four furlongs above the dam site and another six furlongs below and completely divert the river during the summer season, this would reduce the seepage into the coffer dam.

THE PRESIDENT expressed great satisfaction over the useful discussion and thanked members of the Committee for the very clear exposition of the work they had been doing and he was sure that they would continue to do so in future as well.

(iv) SPILL

The following item was discussed at the 1947 Research Committee Meeting :

- (1) Model experiments for the revival of the River Chandana.

(1) MODEL EXPERIMENTS FOR THE REVIVAL OF THE RIVER CHANDANA ⁽⁸³⁾

ABSTRACT

Due to the deterioration of the river Chandana all the villages round about have gone down in prosperity and deteriorated in health. It has been proposed to revive the river and maintain a steady flow throughout the year: For the resuscitation of the river it will be necessary (1) to improve its offtake from the Padma (2) to increase its discharge (3) to increase its capacity. Describes model experiments.

The river Chandana is a small spill channel of the river Padma carrying a maximum discharge of 1,558 cusecs during rainy season and completely drying up during the winter (Figure 2C. 40). In some reaches it acts as a

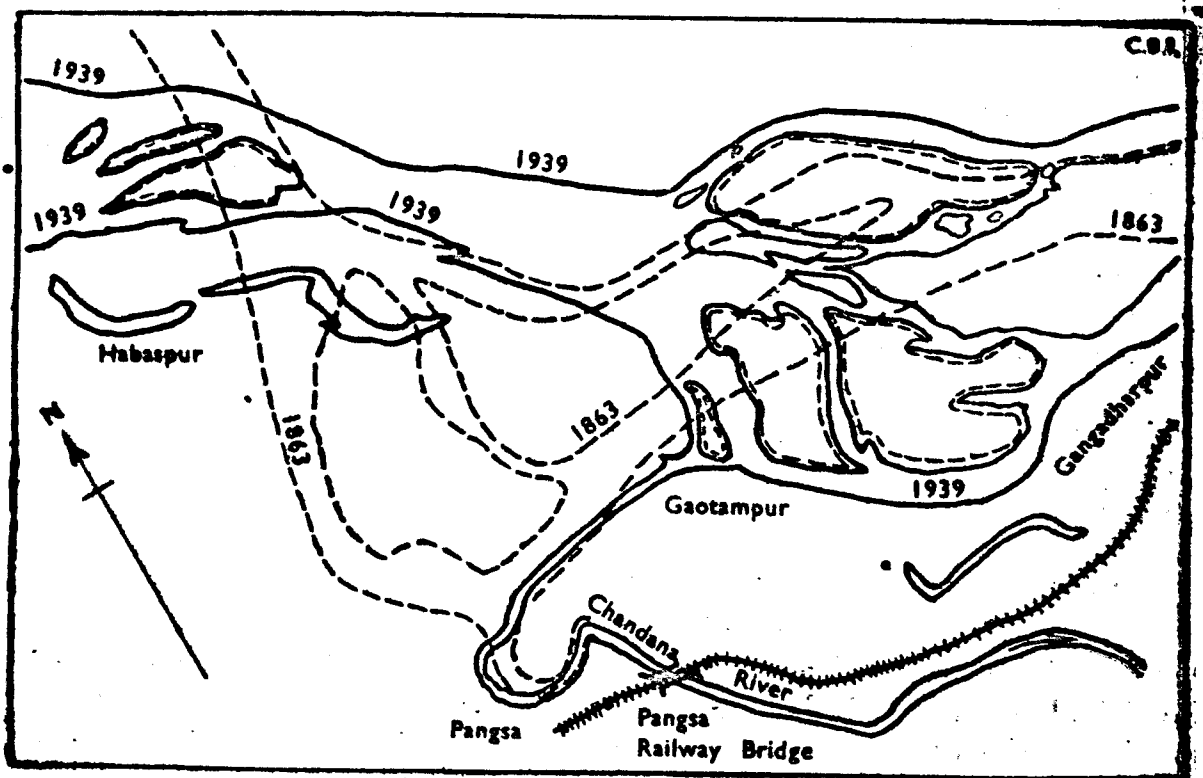


Figure 2C. 40: Plan showing the courses of the rivers Padma and Chandana. drainage channel into the Padma. The river comes out of the Padma near Gaotampur in the Faridpur district from the right bank. After traversing a meandering course of nearly 37 miles it bifurcates at Arkandi one branch flows in the river Gorai at Kamarkhali and the other into a small river named Kumar at Majurdia after passing through low lying *bils*. The total length of the river will be approximately 42 miles. About 1863 the course of the Padma was on its right bank near the offtake of Chandana and Chandana was in a healthy condition. At present the Padma has deserted the right bank near the offtake of Chandana and swung towards the left (Figure 2C. 40). A big *char* (island) of one mile width has been formed in front of the present offtake.

⁽⁸³⁾ Annual Report of the River Research Institute, Bengal, 1946, pages 15 to 21.

During the winter season the connection with the Padma is completely cut off. With the approach of rainy season back water enters the river through a loop channel of the Padma. As the river rises water also enters Chandana by spilling over the sandy *char*.

The river Chandana used to flow through a stretch of the country which was once very prosperous and healthy. Due to its deterioration all the villages have gone down in prosperity and deteriorated in health. It has been proposed to revive the river and maintain a steady flow throughout the year. For the resuscitation of the river it will be necessary (1) to improve its offtake from the Padma (2) to increase its discharge, and (3) to increase its capacity. A model of the river Padma and Chandana was constructed to determine the best position and angle of the offtake, and the consequent increase in discharge. The model was constructed to a scale of $H=1/200$ and $V=1/64$, and the discharge scale (Froude) comes out to be $1/102,400$. Eight miles of the river Padma from *char* Habibpur to Gangadharpur on the right bank was reproduced in the model. This represents five miles of the river Padma from Gautampur offtake of Chandana and three miles downstream. The Chandana was reproduced from Gautampur to below Pangsa railway bridge, a distance of six miles and 320 feet.

This model was built to study the effect of the position of the offtake on discharge and silt entry into the Chandana. It was therefore decided to take only the offtake of the Chandana up to a point where the river is on firm land and is not encroached upon by the shifting course of the river Padma. A gauge was being read at the railway bridge at Pangsa and discharges were also observed occasionally. It was therefore decided to extend the length of the Chandana upto this bridge.

In the model the length of Padma comes out 211 feet and width varies from 70 feet to 105 feet. (In the prototype, varies from 14,090 feet to 21,140 feet.) The length of Chandana is 160 feet. The breadth varies from 6 inches to 25 inches. The bed of Padma in the model was made of river sand (Hooghly sand) and the average diameter is 0.238 mm. The bed of Chandana was constructed in weak cement mortar and the proportion of sand and cement was 10:1 over a concrete slab of four inches thick in order to avoid cracking. The surface of the river course was suitably polished. As constructed the bed of the Chandana is fixed.

The bed of the river at site consists of very stiff clay with no scouring action. The river carries very fine silt in suspension beyond the first $\frac{1}{2}$ mile of the offtake. As the section of the Chandana in the model was going to be very small and in nature it was more or less rigid, the bed was made of very weak cement. The bed was properly roughened to get the gauge and discharge readings true to prototype conditions.

The model bed was moulded according to the survey that was carried out during winter of 1943-44 in the case of Chandana and that during 1946 (February to April) in the case of Padma. As the discharge below Pangsa railway

bridge in Chandana was small in the model, a chamber was constructed below the tail end of the Chandana river to measure the discharge by volumetric method. Further to observe the levels, at various points of the Chandana between Gautampur and Pangsa railway bridge pointer scale measuring devices were installed at the following points along the Chandana river :—

- (1) Between 6,000 feet—7,000 feet (measured from Gautampur).
- (2) Between 18,000 feet—19,000 feet (measured from Gautampur).
- (3) Between 27,000 feet—28,000 feet (measured from Gautampur).

Velocities, discharges and silt contents of the river Chandana were observed during 1943, 1944 and 1945. The discharge of the river Padma at Paksey for the above period was received from the Executive Engineer, Paksey, B. A. Railway.

The discharge of river Padma varies from 62,000 cusecs to 1,462,000 cusecs and the river Chandana gets its first supply when the Padma discharge is near about 400,000 cusecs. Between Paksey and Chandana offtake there was an important distributary of the Padma known as the Gorai. Discharges equivalent to the discharge at Paksey minus the discharge taken off by the Gorai was introduced into the model. Corresponding gauges were obtained in the model at Gautampur, three miles downstream in the Padma, and at Pangsa railway bridge.

The model was run for 847,000 cusecs discharge in the Padma as observed on August, 15, 1945 at Hardinge Bridge and the corresponding levels on the same date at Gautampur and at Pangsa railway bridge was maintained. For the above discharge in the Padma there was a record of discharge, velocity and water surface levels in the Chandana. The above discharge is in the beginning of the flood season in the Padma when she spills over the sandy *char* and the velocity at the offtake are not sufficient to move the sand of the *char*.

In the early stage of these experiments, the model was built in the following scales :—

Vertical	1/40
Horizontal	1/200

The higher distortion was adopted in the model as it was anticipated that without this higher distortion it will not be possible to make the Padma sand move in the model. When the model work was started, it was expected that with the five pumps installed it will be possible to get a supply of 15 cusecs in the model. These pumps were brought in 1938 and had not been used since then. After installation it was however found in 1946 that not more than 8.5 cusecs could be had for the model experiments. This was a great disappointment and with the above scales of the model it was not possible to run discharges higher than 300,000 cusecs in the Padma. At this discharge

Padma water just entered the Chandana and no sand movement was noticed. Further as this was too low, a discharge for the Chandana it was decided to reduce the distortion with the following scales :—

Vertical	1/64
Horizontal	1/200

With these scales it has been possible to run a discharge equivalent to 850,000 cusecs in the Padma corresponding to which there are a number of prototype observations in the Chandana. By increasing the vertical scale however the velocities have been reduced considerably and no silt entry could be obtained.

The levels at the other points and the velocities, slopes and discharges which are observed in the model are given in Table 2C.15. It was noticed while running the experiment that the big loop in the Chandana on which Pangsa village is situated acts as a bottle neck and retards the flow of the channel. It was therefore decided to short circuit this loop in the model by means of a chord channel.

TABLE 2C.15

Chandana River Model Experiments

Comparison of Gauges, Slopes, Velocities and Discharges in Old and the New Courses of the Chandana

Detail No.	Site of observations along the river Chandana.	Gauges in feet in P.W. D. Datum.		Slope per 1,000' old course	Proposed new course	Velocity in feet per second		Discharge in cusecs below Pangsa Railway Bridge		Discharge in cusecs in gauges at Hardinge Bridge
		Old course	Proposed new course			Old course	Proposed new course	Old course	Proposed new course.	
1	2	3	4	5	6	7	8	9	10	11
1	Gantampur 9'	33.65	33.65	1.92	2.63	847,000
2	6,000'—7,000'	32.91	32.33	0.123	0.23	2.91	4.25
3	18,000'—19,000'	31.66	..	0.104	..	1.01
4	27,000—28,000' (12,000'—13,000) ..	31.06	31.06	0.6	0.21	1.24	1.82
5	31,000'—32,000' (16,000'—17,000')	31.20	31.20	1.33	1.98	1,263	1,834	..

A map showing the old course of Chandana and the proposed new course is given in Figure 2C.41. The bottom level of the new course at Point A (shown in the map) is 18.48 R. L. The section of this portion is the extension of the existing section at 7,000 feet and 27,000 feet of the old Chandana course.

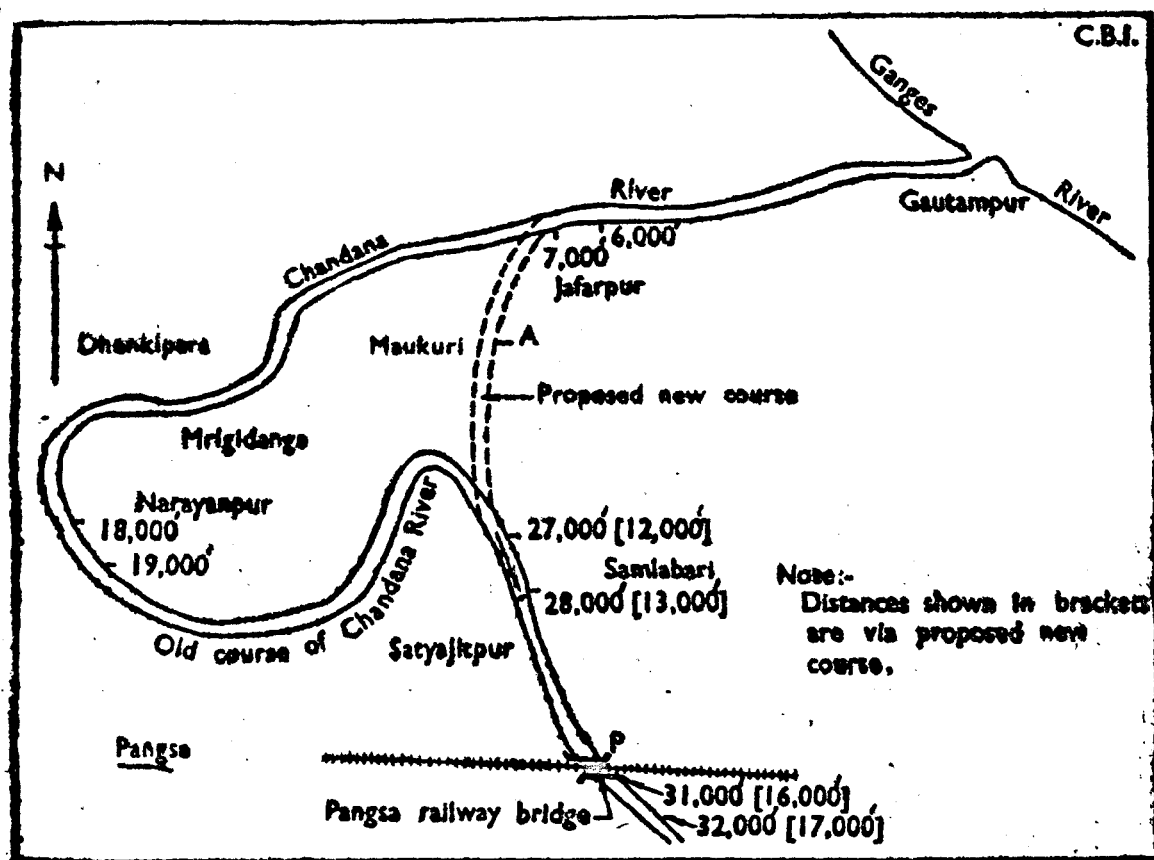


Figure 2 C. 41 : Plan showing the existing course of the river Chandana and the proposed new course

The reduced length of Chandana due to short circuiting is 17,000 feet that is, three miles 1,160 feet up to Pangsa railway bridge. The reduced length is given within the bracket in the Table 2C.15. The levels, velocities, slopes and discharges in the Padma are also given in Table 2.C.15. There is a sudden increase in velocity below 6,000 feet in the proposed new course and there might be retrogression of bed level in this part of the river. It may, therefore, be necessary to regrade the river bed in this reach.

Further as it is proposed to excavate the lower portion of Chandana below Pangsa railway bridge by removing the bottom silt, there will be a drop in the water-level at Pangsa railway bridge and consequent change of level and slope from Pangsa railway bridge to Gautampur. In order to study the effect of this bed lowering the levels at Pangsa railway bridge were lowered in the model by quantities equivalent to one, two and three feet in the prototype maintaining the Padma discharge the same. The observed levels, surface velocities and discharge in the model at different points in the old and the proposed new course are given in Table 2C.16 and 2C.17 respectively.

TABLE 26.16

Chandana River Model Experiments

Slope, Velocities and Discharges in old Course at different levels below Pangsa Railway Bridge

Serial No.	Site of observation along the river Chandana	Gauge in P. W. D. Datum due to lowering of levels below Pangsa Railway Bridge				Slope per 1,000 feet due to lowering of levels below Pangsa Railway Bridge				Velocity in feet per second due to lowering of levels below Pangsa Railway Bridge				Discharge in cusecs due to lowering of levels below Pangsa Railway Bridge				Discharge in cusecs at Hard-finge Bridge	Remarks
		By				By				By				By					
		Original condition	1'	2'	3'	Original condition	1'	2'	3'	Original condition	1'	2'	3'	Original condition	1'	2'	3'		
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
1	Gantampur 0'	33.65	33.65	33.65	33.65	1.92	2.94	2.61	847,000	Velocity was not observed during lowering of level by 3 feet
2	6,000'—7,000'	32.91	32.58	32.32	32.50	0.123	0.178	0.221	0.191	2.91	4.46	4.64	
3	16,000'—19,000'	31.66	31.17	30.91	31.00	0.104	0.117	0.117	0.225	1.01	1.54	1.66	
4	27,000'—28,000' (12,000'—13,000')	31.06	30.02	29.45	28.35	0.06	0.128	0.162	0.294	1.24	2.16	1.94	
5	31,000'—32,000' (16,000'—17,000')	31.20	30.20	29.18	28.42	1.33	2.14	1.82	..	1,263	1,839	1,930	1,945	..	

TABLE 2C.17

Chandana River Model Experiments

Slopes, Velocities and Discharges in the new Course at different levels below Pungsa Railway Bridge

1	Gautampur 0'	..	33.65	33.65	33.65	33.65	2.63	3.81	4.00	4.21	847,000
2	6,000'—7,000'	..	32.33	31.49	30.66	30.24	0.22	0.36	0.50	0.55	4.25	6.64	8.88	8.88
3	18,000'—19,000'
4	27,000'—28,000' (12,000'—13,000')	..	31.06	30.08	29.50	28.35	0.21	0.23	0.19	0.33	1.82	2.72	3.36	3.63
5	31,000'—32,000' (16,000'—17,000')	..	31.20	30.14	29.50	28.29	0.01	1.98	2.96	4.00	4.00	1,834	2,499	2,735	2,785	..

RIVER CONTROL

All the three parameters—slope, velocity and discharge—increase due to the lowering of levels below Pangsa railway bridge both in the old and the proposed new course. The maximum effect is obtained by lowering the levels by one foot—further lowering is not proportionately effective.

The increase of discharge in the old course due to lowering of level below Pangsa railway bridge by one foot (which can be attained by silt clearance) is practically the same as due to short circuiting the loop [see Tables 2.C.15 and 2.C.16 columns (10) and (16) respectively]. Further if the proposed short circuiting is made and the river course below Pangsa railway bridge is improved thereby and the gauge at this point is lowered by one foot, the overall increase of discharge from its existing condition will be over 100 per cent. [see Tables 2.C.15 and 2.C.17 columns (9) and (16), respectively].

Figure 2.C.42 shows the bed level of river Chandana and the water level of the old course and the proposed new course for a discharge of 847,000 cusecs in the Padma.

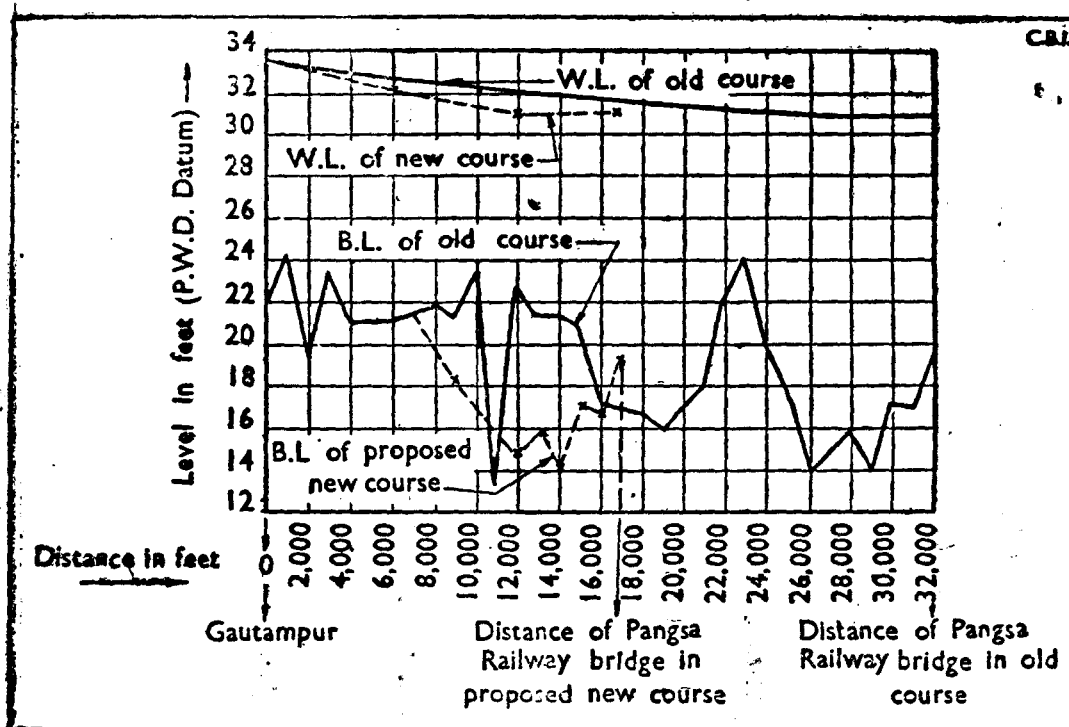


Figure 2C.42 : Bed level and water surface level of the river Chandana in old and proposed new course

DISCUSSION BY THE RESEARCH COMMITTEE

DR. BOSE introduced the subject.

MR. FRAMJI said that it had been stated that at present the Padma had deserted the right bank near the offtake of Chandana and swung towards the left, i.e., the offtake was virtually on the inside of the curve which obviously explained the deterioration of the river as it must draw a relatively excessive

sand charge. The remedy suggested was as bad : in addition to retaining the adverse position of offtake of the Chandana from the Padma—the main source of the trouble—a cut-off channel taking off from the wrong side of a curve had been suggested. This would not only attract or induce greater sand-entry into the Chandana at its offtake from Padma due to the greater slope and velocity, but because it would carry much more charge as well as more discharge—the latter only temporarily—the chances of its development would be less than indicated by the model and its prospects appeared to be uncertain.

The problem was principally one of movement of bed material while no mention appeared to have been made of studies of lines of bed flow from the Padma.

If the cut-off was effected, the Chandana cut-off would increase its length upstream and downstream, particularly upstream. This was likely to result in even worse entry-conditions from the Padma and might cause greater obliquity at the Pangsa Railway Bridge. Model limitations precluded indications of the nature of lengthening of a cut-off channel ; these must, nevertheless, be foreseen from experience.

A more promising solution appeared to lie in locating the offtake of Chandana, if feasible, at the outside (concave side) of a curve in the Padma and to train the river to remain concave at the point of offtake.

DR. BOSE said that the Chandana was a spill channel of Padma. The characteristics of a spill channel were that only when the level of the parent river rose above a certain point, the spill channel could get any supply. In the case of the river Chandana this happened when the Padma discharge was over 300,000 cusecs. The main channel of the Padma near about the offtake of the Chandana was away on the left bank and Chandana got its supply from a point much lower down. There was a big sandy island in front of the offtake. It did not allow flow into the Chandana to take place directly. The problem was how to increase the duration and intensity of flow into the Chandana by some amount without drawing more silt. In the present experiments only the easiest and cheapest method had been tried. Experiments were in progress. He also said that they had already tried the remedies suggested by Mr. Framji and because of local conditions they had not been successful. The main idea of improving the Chandana was to increase the discharge of the river from 1,800 cusecs to 4,000 cusecs, and also to increase the duration of this from 2½ months to four months. A spill channel was different from a tributary. It flowed only when the river was full. It might change its position of offtake yearly. Generally the zone of offtake was very wide. In the case of Chandana this was nearly 40 miles. They were trying to keep the offtake as it was and at the same time increase the discharge. They also wanted to study the best position of the offtake.

MR. FRAMJI asked whether in the present position it had come to the limiting point for the worst offtake and whether he wanted to find out a solution which would keep the present offtake and for how long this would remain.

(v) MEANDERING

Two items were discussed at the 1947 Research Committee Meeting :

- (1) Deterioration of the River Kultigong.
- (2) Proposed railway bridge near Shahabad on the Ramganga River.

(1) DETERIORATION OF THE RIVER KULTIGONG⁽³⁴⁾

In order to ascertain the origin of the new course sand that is being deposited on the sides of the river Kultigong as well as on the islands ⁽³⁵⁾ that are visible in low water a survey of the banks of the river *chars** and the islands of the river was undertaken. This type of sandy deposit was always found on the inside of a bend of the river where there was a chance for the sand to be deposited. Previously the deposits usually consisted of clay and silt with some sewage material but this new menace, that is, the deposition of a sand layer on the bank soil is of a recent occurrence as is evident from the fact that when a bore hole was made on the bank deposits the surface layer only was found to be consisting of sand the thickness of the layer varying from a few inches to two to three feet below which there was always clay and silt. This deposition of sand is very harmful to the river as the effect of the scouring action of the river on heavy sand particles is less in comparison to that on the lighter clay and silt particles.

A previous survey of the river showed altogether nine *chars** within the stretch between Haroa to Kalinagar marked in the index map, but afterwards as the present survey has shown few more *chars** were visible and identified in low water some in the condition of development. The collection of bank soil samples and *char** samples was started from Haroa. The samples were analysed for the percentages of sand as well as for organic matter, moisture and salt contents. The places of collection and the percentages of sand obtained are shown in Figure 2C.43.

Bank soil samples were collected from the mouth of the Indali sluice as well as from inside the sluice which on analysis showed 13.45 and 2.32 per cent. of sand, respectively. Surface samples of *char** No. 1 marked in the map on the left bank revealed 91.45 per cent. of sand on analysis. The next bend down the river showed *char** No. 2 in the map. This is a big *char** on the left bank of the river about five to six feet above the low water and about 500 feet in length extending up to three quarters of the river width. Here also the percentage of sand was very high (95.0). At the river bend just downstream *char** No. 2 a new *char** was developing on the left bank. This *char** was not shown in the previous survey. Here the sands are just beginning to be deposited on the surface and the percentage of coarse sand was less being 46.46. The second bend from *char** No. 2 down the river has got sand deposited on the right bank in a way almost identical with the previous deposits and therefore no sample was collected. The third bend down *char** No. 2 showed another small but well formed *char** on the left bank, percentage of sand being 80.51. Sample was collected from a fairly extensive *char** on the right bank

⁽³⁴⁾ Annual Report of the River Research Institute, Bengal, 1946, pages 10 to 14

⁽³⁵⁾ Annual Report of the River Research Institute, Bengal, 1945, pages 7 to 19

* Islands

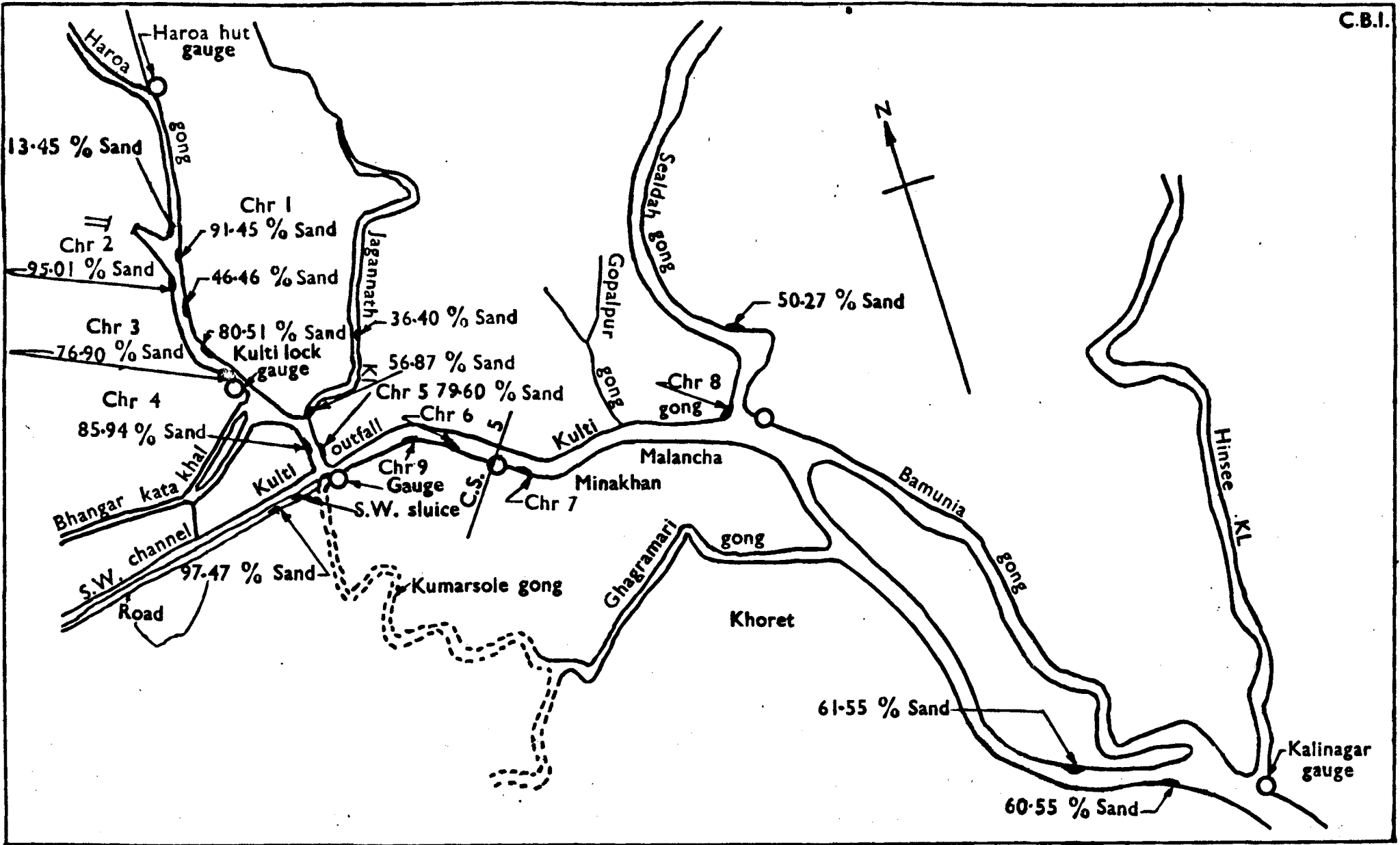


Figure 2C.43:- Index map of Kulti gong showing percentages of sands obtained from different points

fourth bend below *char** No. 2. This is *char** No. 3 in the map and the percentage of sand here is 70.9. The fifth bend down *char** No. 2 opposite Bhangar Kata *khal†* lock showed a very small deposition and therefore no sample was collected. Sand samples were also collected from the mouth of Jagannath *khal†* as well as from one mile inside the Jagannath *khal†* the percentage of sand being 56.87 and 36.4, respectively. Inside the Jagannath *khal†* the samples were only clayey and it is unlikely that sand could come down that channel. Further downstream the South Western channel of the Calcutta Corporation is met. Samples from the mouth of the South Western channel give a value of 45.85 of sand on analysis. This value is low in comparison to the high percentage of sand 97.47 obtained from a sample collected from about one mile inside the South Western channel. This is the highest value obtained so far on analysis of all the samples. Samples of bank deposit obtained from below cross section No. 28 of Calcutta Corporation left bank showed another stretch of sand deposit percentage of sand being 82.42. Samples from a nose opposite Brahmapurkur sluice has been examined. There is no *char** formation but a slight deposition of sand. The inside bend upstream of cross section F left bank has been eroded and very slight deposition of sand has been observed. Sample has been collected from opposite Atpukur gauge below standard cross section III on the right. Here the deposit is about 100 feet in length and about 20 feet in width the percentage of sand being 75.01. *Char** No. 7 marked in Figure 2C43 is not a *char** but the remnant of a sandy bank which is being washed away. There is a slight deposition of sand just upstream of Minakhan sluice—No. II. There is a caving bank here and inside this protected area some sand has been deposited. Sample from cross section J near Gopalpurgong has been examined. Here only oily layers existed on the surface of both the banks. Sample has been collected from about $\frac{1}{2}$ mile inside Sealdahgong from a bend where the percentage of sand was found to be 50.27. Samples have been collected from the lower junction of Bahmaniagong and Hatgatchagong upstream of Hinshi *khal†*. This is a big *char** on the right bank. The length of the *char** is about a mile and it is about 100 feet in width. Three samples were taken, one from the surface layer and another from a depth of $1\frac{1}{2}$ feet and the third sample from a line separating the sandy deposits from the previously existing clay, the line of demarcation being easily discernible. The width of the *char** is about 25 to 30 feet. The percentage of sand determined on analysis for the above three samples being 60.55, 72.45 and 39.87, respectively. About $\frac{1}{2}$ mile upstream from the point from where these samples were collected a small *char** was observed on the right bank of the river Kultigong and the deposit collected from this place gave a value of 79.77. *Char** No. 4 and *char** No. 5 marked in the map revealed 72.6 and 85.94, per cent. of sands, respectively.

As is to be expected the amounts of moisture and organic matter associated with these samples are generally less as the percentage of sand increases. The results are given in Table 2.C.18.

*Islands

†Channel

TABLE 2C. 18

Results of the Chemical Analysis of Coastal and Char Samples Collected from
Kultigong River

Date of sampling (1946)	Serial No. of sample	Description of samples	Sand	Salt	Moisture	Organic matter
			Per cent.	Per cent.	Per cent.	Per cent.
March 5	1	Coastal soil from the mouth of Indali Sluice during ebb tide.	13.45	1.536	5.11	3.14
March 6	2	Coastal soil from inside the Indali Sluice during ebbs tide.	2.32	1.252	3.35	3.22
March 7	3	Sands from <i>char</i> No. 1 marked in the map on the left bank.	91.45	0.1	5.79	1.8
	4	Sands from <i>char</i> No. 2 marked in the map on the right bank.	95.01	0.1	1.89	0.799
	5	Sand from a bend just below the <i>char</i> No. 2 which is developing into a new <i>char</i> on the left bank.	46.46	1.728	1.3	1.97
	6	Sand from 3rd bend down <i>char</i> No. 2 left bank.	80.51	0.16	0.19	0.4
	7	Sand from a fairly extensive <i>char</i> on the right bank 4th bend below <i>char</i> No. 2 This is <i>char</i> No. 3 in map.	76.9	0.0	0.16	0.26
	8	Sand from the mouth of Jagannath <i>Khal</i> .	56.87	0.27	1.2	1.5
	9	Sand from the mouth of S. W. Channel	45.85	0.56	0.2	2.08
	10	Sand from one mile inside the mouth of S. W. Channel.	97.47	0.06	0.19	0.39
	11	Sample from below cross-section 28 of Calcutta Corporation from the left bank.	82.42	3.0	2.63	2.8
	12	Samples from right bank opposite At-pukur gauge below standard Cross Section III.	75.01	0.144	0.66	0.83
	13	Cross-section "J" near about Gopal-purgong only the oily layer was taken.	..	.03	0.506	1.8
	14	Sample about $\frac{1}{2}$ mile inside Sealdah-gong.	50.27	1.128	2.3	2.55
	15	Sample from the lower junction of Bahamaniagong and Hatga chagong upstream of the junction of Himshi <i>Khal</i> . This is a big <i>char</i> —				
		(a) Sample from surface ..	60.55	2.34	2.46	2.3
		(b) Sample from inside ..	72.45	1.122	1.23	2.15
		(c) Sample from the line up the width of <i>char</i> where sand ceases to exist and there is clay.	39.87	0.25	2.14	2.34
	16	Samples from a <i>char</i> on the left bank about $\frac{1}{2}$ mile upstream of sample 15.	61.55	4.66	2.14	2.34
	17	Sample from inside the breach on the right bank of the River Kultigong.	79.77	2.4	0.82	4.1
	18	<i>Char</i> just opposite S. W. Sluice <i>char</i> No. 5 in the map.	72.6	0.82	0.83	1.02
	19	<i>Char</i> between Govt. Sluice and S. W. Outfall <i>Char</i> No. 4 in the map.	85.94	0.16	0.04	0.4
April 14	20	From one mile inside Jagan nath <i>Khal</i>	36.4	..	2.38	2.5

The results obtained so far indicate that the origin of the sand may be traced in between Indali sluice and Minakhan sluice No. III because both above and below this stretch very little sandy deposits have been observed though there were enough scope for deposition.

The sands thus obtained were submitted to siltometer analysis and it was expected that the concentration of particles of higher diameters in these samples would throw some light on the origin of these coarse sand deposits. The results of the siltometer analysis are given in Table 2C.19. These data go to show that the highest value of the mean diameter in mm. (0.199) was obtained for the sample from the *char** between Government sluice and South Western outfall, that is, for *char** No. 4, while comparatively higher values were given by *char** No. 3, 1 and 2 and *char** No. 5 just opposite the South Western sluice, the values being 0.188, 0.160, 0.158, and 0.156, respectively. Therefore the origin of the new coarse sand may be expected to be in this locality.

TABLE 2C. 19

Results of Siltometer Analysis of Coastal and Char Samples Collected from Kultigong River

Serial No.	Date of sampling (March 1946)	Date of analysis (April 1946)	Description of the samples	Mean Diameter in mm.
1	5	2	Coastal soil from the mouth of the Indali Sluice ..	0.070
2	5	2	Sand from <i>char</i> No. I in the map	0.160
3	5	4	Sand from <i>char</i> II	0.158
4	5	4	Sand from a bend just below <i>char</i> II	0.117
5	5	5	Silt from 3rd bend below <i>char</i> II	0.143
6	5	5	Silt from <i>char</i> III in the index map	0.188
7	5	8	Sand from the mouth of Jagannath <i>Khal</i>	0.133
8	5	9	Sand from the mouth of S. W. Channel	0.091
9	6	1	Silt from 16th mile S. W. Channel	0.141
10	6	10	Sample from below cross-section 28 of Calcutta Corporation	0.138
11	6	12	Samples from the bank opposite Atpukur gauge ..	0.146
12	6	11	Samples from $\frac{1}{2}$ mile inside Sealdahgong	0.07
13	7	12	Sample from lower junction of Bahmaniagong and Hatgachagong—	
14	7	16	(a) from surface	0.077
15	7	16	(b) from inside	0.06
			(c) from the line separating sand and clay ..	Very fine silt sample, too small for analysis
16	7	17	Sample from $\frac{1}{2}$ mile upstream of above sample ..	0.07
17	7	23	Sample from inside the breach on the right bank of Kultigong River.	0.083
18	7	16	Sample from <i>char</i> just opposite S. W. Sluice <i>char</i> five in the index map.	0.156
19	7	16	Sample from <i>char</i> between Government Sluice and S. W. Outfall. <i>char</i> No 4 in the Index map.	0.199

*Island

Bed silt samples of the South Western channel were also collected by taking samples at every mile interval starting from Bantola and analysed to study the mechanical composition or the size distribution of particles that were found in the bed of the channel. The results obtained are given in Table 2 C. 20. The maximum size of the coarse sand was obtained from a point at a

TABLE 2 C. 20

Results of the Analysis of the Bed Sample of S. W. Channel at Different Points Collected on 14th April, 1946

Serial No.	Distance of the sample collected from Bantola Sedimentation Tank in miles	Sand per cent.	Salt per cent.	Moisture per cent.	Organic matter per cent.
1	1	26.74	0.92	0.62	3.4
2	2½	39.49	0.47	0.57	4.46
3	3	23.0
4	4	35.52	1.28	0.96	5.1
5	5	8.54	0.76	2.92	7.2
6	7	43.15	1.21	0.81	4.1
7	8	30.3	1.03	1.4	6.08
8	9	56.3	1.15	1.87	4.2
9	10	46.2	0.12	0.74	1.22
10	12	33.45	.91	4.48	4.63
11	13	52.2	1.04	2.14	3.43
12	14	50.0	1.17	1.92	2.86
13	14½	73.9	0.46	0.75	1.33
14	15	88.2	0.21	0.24	0.41
15	16	11.7	0.53	2.67	3.0

distance 15 miles from Bantola sedimentation tank and about one mile from the Kulti Outfall the value being 88.2 per cent. and that for another sample 14½ miles distance from Bantola is 73.9 per cent.

The result of the siltometer analysis of these samples are given in Table 2 C. 21.

TABLE 2 C. 21

Results of Siltometer Analysis of Bed Samples of S. W. Channel at Different Points Collected on 14th April, 1946

Serial No.	Date of Analysis October 1946	Description	Mean diameter in mm
1	14	Bed slit from 15th mile of S. W. Channel ..	0.185
2	15	Bed silt from 14th mile of S. W. Channel ..	0.158
3		Bed silt from 14½ mile of S. W. Channel
4	14	Bed silt from 4th mile of S. W. Channel
5	15	Bed silt from 7th mile of S. W. Channel	Quantity of sand too small for analysis

The remaining samples are below the siltometer range

CONCLUSIONS

The sands found in these *chars* must have been thrown up from the deep scour holes that have formed in the river bed between Indali sluice above Kulti and the Malancha Hat. Sand samples obtained at mile 15-16 of the S. W. channel indicate that there is a very extensive layer of coarse sand below the normal bed level of the river. If a concentrated supply of desilted water is let into the river at any point such as the Government sluice or S. W. channel sluice then deep scour holes will form in the bed of the river immediately downstream of these points which may reach this sand layer and churn up the coarse sand particles and carry them to different reaches of the river. Deposition of sand will then take place at favourable points such as the insides of bends. These sand depositions at different bends of the river will tend to increase the meandering tendency of the river.

(2) PROPOSED RAILWAY BRIDGE NEAR SHAHABAD ON THE RAMGANGA RIVER ⁽³⁶⁾

ABSTRACT

Presents results from theoretical considerations regarding : (1) The minimum distance between the river edge at the loops and the proposed railway embankments, which will be sufficient to keep the embankment safely away from the effect of erosion in the event of the meanders acquiring a swing towards the embankment. (2) The estimation of the maximum flood discharge, and (3) the assumed depths of scour.

THE MINIMUM DISTANCE BETWEEN THE RIVER EDGE AT THE LOOPS AND THE PROPOSED RAILWAY EMBANKMENT

Unfortunately, no records of discharges of the river at the site were available; so assessment of the maximum and dominant discharges had to be made from the inspection of plans or from various formulas. This *minimum* distance is governed by the following two considerations:—

- (i) The difference between the existing amplitude of the river course loops, in the state they are, and the ultimate maximum amplitude (*i.e.* semi-meander belt) the loops would assume, under the worst condition when the arc/chord ratio increases to the maximum ratio observed on the river in question before a cut-off occurs, with a

⁽³⁶⁾ Indian Waterways Experiment Station, Poona, Annual Report (Technical) 1946 pages 6 to 17.

sufficient margin of safety for an increase in the meander belt for unforeseeable reasons, is the minimum safe distance the loops should be located from the approaches.

- (ii) Where a river is controlled by a fixed exit, the full meander belt may occur on either side of the axis of the river ; and hence the meander belt tends to be twice that on the river at other places where the exit is not fixed. For example, from a study of the river courses during the last 30 years in Sind, it has been observed that the river swing is generally about four to six miles, but just upstream of the Sukkur Barrage the swing is much greater, being nearly 12 miles ⁽³⁷⁾.

The longer of the two distances, calculated according to the above two considerations, should be the minimum distance.

(1) A detailed study of the meander loops in the river was, therefore, made. (Table 2 C. 22). Only seven out of 150 years river courses had been supplied ; so it had to be assumed that the seven river courses shown on the two inch to one mile plan were faithfully representative of all the years for which data were available. The correct estimation of arc/chord ratio is of considerable importance, both for the positioning of the approach embankments and for fixing the length of the guide banks.

Figure 2 C. 44 shows meanders on this river in four different stages of development. It will be observed that as the arc/chord ratio increases from 1.26 to 2.67, M_b (the length of the meander belt) also increases from 1.25 to 1.80 miles. Hence, according to the first consideration, if an existing loop happens to be in the condition of the 1795 loop (*i.e.* arc/chord=1.26, $M_b=1.25$), it can develop to as much as $M_b=1.80$ miles, and the minimum distance between the edge of the loop and the proposed embankment should, therefore, be (1.80—1.25) miles *plus* a certain margin of safety (say 25 per cent. of M_b max.) to allow for the possibility that the maximum arc/chord ratio might not yet have occurred, or been spotted, or that a meander might not cut-off even at the highest arc/chord ratio but might further increase due to the incidence of certain unforeseeable disturbing factors. Accordingly, the minimum distance for

a loop such as of the year 1895 would be $1.80 - 1.25 + \frac{25}{100} \cdot 1.8 = 1.0$ mile, while the minimum distance from the existing 1945-46 loop

($\frac{B}{C} = \frac{\text{length along bend}}{\text{chord}} = 2.11$, $M_b=1.75$.) would be (1.80—

$$1.75 + \frac{25}{100} \cdot 1.8) = 0.5 \text{ mile.}$$

Thus, from this consideration, the line of the proposed embankment should be AB (Figure 2 C. 45), so far as it is possible to fix it with the incomplete meander courses available.

(2) Though ordinarily, where a river is controlled by a fixed exit and constrained from free movement on either side, a swing equal to the full meander belt—instead of half the meander belt—may be expected on either side of the river axis, in the present case there is an off-setting factor which must

⁽³⁷⁾ Indian Waterways Experiment Station, Poona, Annual Report (Technical) 1943, page 13.

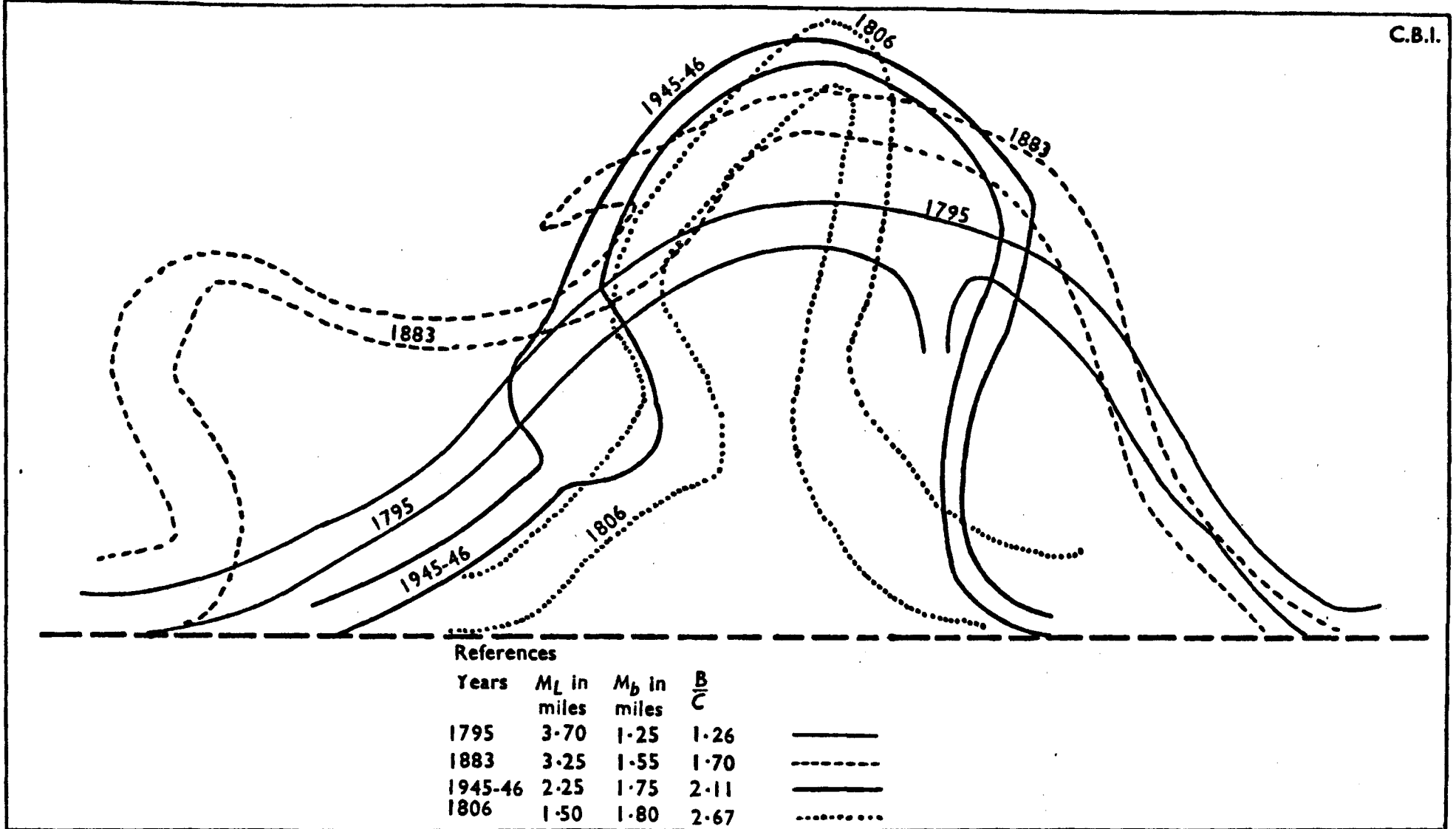


Figure 2C.44:— Showing comparative increase in Meander belt and decrease in Meander length with rising

$$\frac{B}{C} \text{ i. e. } \frac{\text{Length along bend}}{\text{Length along chord}} \text{ Ratio}$$

References-
 Alignment of the Railway Embankment proposed by the Director, Poona Experiment Station -----
 Alignment of the Railway Embankment based on the considerations stated in Director's note -----

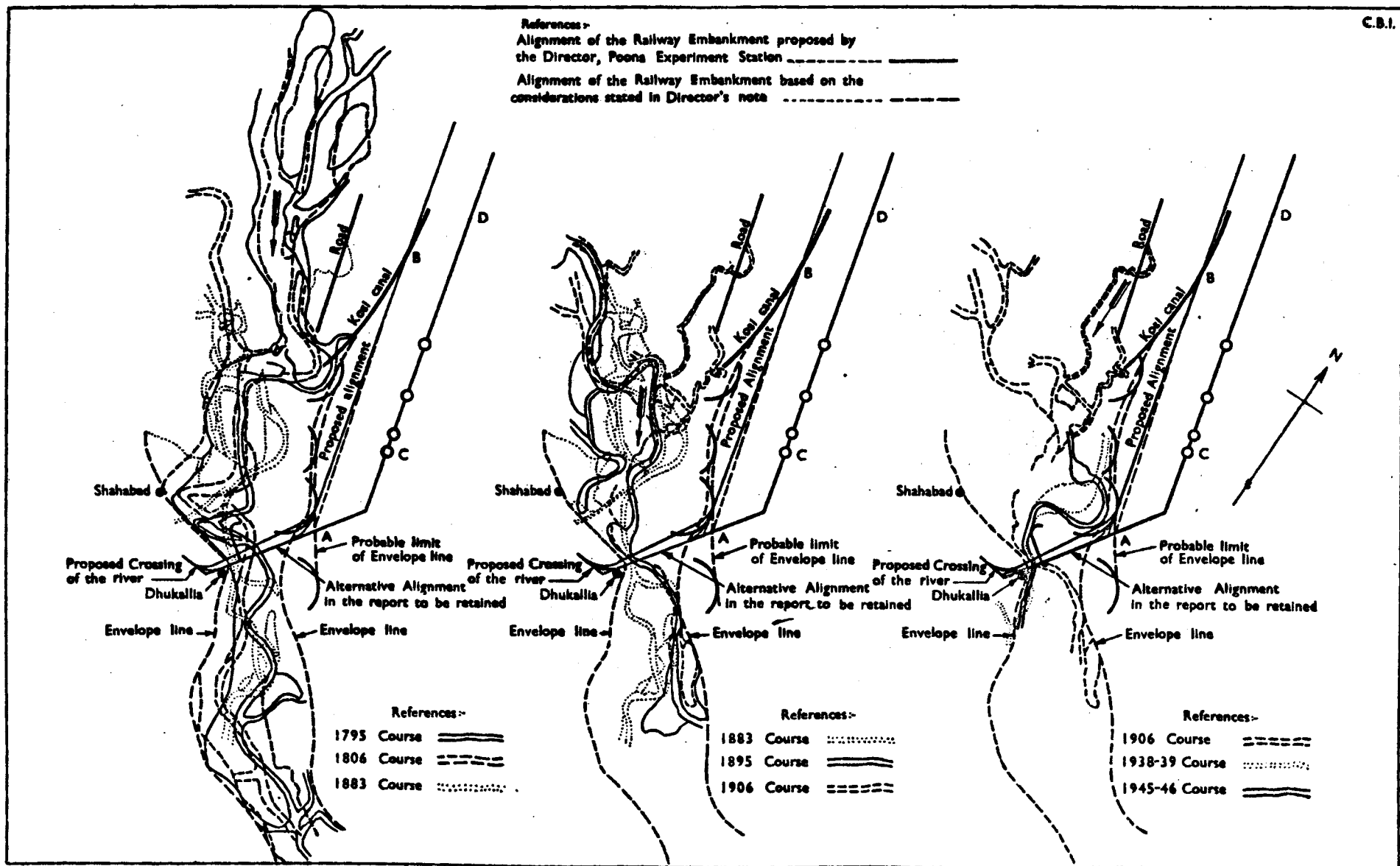


Figure 2C.45:- Showing Courses taken by Ramganga River between 1795-1946

Scale 2 1 0 2 4 6 8 10 Miles

not be lost sight of. On the right (or west) bank of the river exits, at and just above the proposed bridge site, a naturally hard inerodible bank which has stood without appreciable change for many years ; so that, even under existing conditions, the river was not free to move without restraint on that side and it cannot be assumed that, in the reach of this natural hard bank, the river has only swung up to half a meander belt on either side of the axis while after the construction of the bridge it will swing to the extent of the full meander belt. Moreover, the guide bank on the east side will increase the attack on the west side in the immediate vicinity of the bridge in certain years, and to this extent the meander swing on the east side may be expected to decrease.

But, just beyond the stretch of this hard inerodible bank on the right bank, the argument applies ; thus, assuming the figure of river swing of a approximately four miles near Shahabad owing to incompleteness of river courses it is not possible to verify this figure—the approach should nowhere be less than four miles away from the river axis, in a reach of, say eight to ten miles upstream of Shahabad. (The hard inerodible bank on the right side appears to terminate at Shahabad). This gives, approximately, the line C D in Figure 2.C. 44. As CD is the outer of the two lines, other considerations being equal, the approach bank was recommended to be located about one mile further inland.

THE CALCULATIONS OF THE FLOOD DISCHARGE

In the absence of flood discharge data the flood discharge was calculated by almost all the well-known empirical formulae,—Dickens, Ryves, Inglis, *etc.*, which gave widely varying results.

The discharge was estimated by the Mississippi formula, recommended by Sir Francis Spring, assuming depth=27.69 feet with ten feet scour at high flood level which was on the low side. With depth= 35 feet velocity (for slope of 6.76 inches per mile)=6.3 feet per second, area =20,200+ 17.7/10 × 10,000 =37,500 square feet, discharge=37,500 × 6.3=2,36,000 cusecs. Making the per cent. allowance for error in reported high flood level the discharge would be 2.5/1.72 × 2,36,000=3,43,000 cusecs.

The flood discharge was also estimated on the analogy of the Bareilly Bridge, assuming a velocity of six feet per second, while for rivers of this size the mean velocity in flood would more likely be at least eight feet per second ; velocity, the discharge on this basis would be 2,86,896 cusecs.

In the Inglis' formula, no allowance for shape had been made, while the formula is specifically applicable for fan catchments only. With due allowance

for shape, $(^{38}) Q=K \frac{n 7000A}{\sqrt{A+4}}$ where for Deccan catchments $n=1/3$; but

United Provinces catchments are known to have less run-off for the same area of catchment.

so $n=1/2$ may be assumed. Shape factor, $K=\frac{2}{\sqrt{\pi}} \frac{\sqrt{A}}{L} = \frac{2}{1.77} \times \frac{7.07}{168} = 0.54,$

$:Q=K^{\frac{1}{2}} \frac{7000A}{\sqrt{A+4}} = 0.54^{\frac{1}{2}} \times 4,95,800 = 0.735 \times 4,95,800 = 3,64,000$ cusecs.

(³⁸) Indian Waterways Experiment Station , Annual Report (Technical) 1942-43, page

TABLE 2.C. 22

Course of year	No. of defined meanders	M_l Length of the meander in miles	M_l Average length for the year in miles	M_b Meander belt in miles	M_b Average meander belt for the year in miles	Ratio $\frac{B}{C}$ B=length along bend C=chord	Radius of curvature "r" in miles		$\frac{r}{VQ_{max}}$ Where values of "r" are as given in 8 (b) x 5,280	$\frac{r}{VQ_{dom}}$ Where values of "r" are as given in 8(b) x 5 280
							As measured from plan	As calculated by formula I		
1	2	3	4	5	6	7	8a	8b	9a	9b
1795	4	3.70	..	1.25	..	$\frac{9.25}{7.3} = 1.25$	1.05	1.13	10.11	12.38
		2.25	2.77	0.77	1.53	$\frac{5.50}{4.5} = 1.22$	0.45	0.77	6.89	8.44
		1.80	..	1.75	...	$\frac{8.0}{3.6} = 2.20$	0.10	0.51	4.57*	5.58 *cut-off appears to have occurred.
		3.40	..	2.35	..	$\frac{13.75}{6.8} = 2.02$	0.60	0.86	7.70	9.40
1806	3	6.0	..	1.3	...	$\frac{13.25}{12} = 1.11$	1.40	2.48	22.20	27.15
		1.50	3.33	1.8	1.61	$\frac{8}{3} = 2.67$	0.20	0.47	4.20	5.15
		2.50	..	1.75	..	$\frac{8.5}{5} = 1.7$	0.60	0.63	5.64	6.90

1888	2	2.50	2.87	0.85	1.20	$\frac{7.5}{5} = 1.5$..	0.84	7.52	9.20	Development imperfect
		3.25	..	1.55	..	$\frac{11}{6.5} = 1.7$	0.80	0.84	7.52	9.20	Do.
1885	3	4.50	..	1.00	..	$\frac{11.4}{9} = 1.27$	1.80	1.94	17.37	2.12	
		1.25	2.87	1.35	1.37	$\frac{5.5}{2.5} = 2.2$	0.20	0.36	3.22	3.94	
		2.85	..	1.75	..	$\frac{10}{5.7} = 1.76$	0.95	0.71	6.35	7.77	
1906	3	5.0	..	0.9	..	$\frac{11}{10} = 1.1$	1.80	3.70	33.15	40.50	
		1.75	3.34	1.43	1.36	$\frac{6}{3.5} = 1.72$	0.20	0.45	4.03	4.93	cut-off occurred
		3.25	..	1.75	..	$\frac{11}{8.5} = 1.7$	0.95	0.82	7.34	8.96	
1938-39	1	3.00	3.00	1.20	1.20	$\frac{7.75}{6} = 1.29$	0.92	0.84	7.52	9.20	
1945-46	2	3.25	..	1.50	..	$\frac{10}{6.5} = 1.54$	0.50	0.85	7.61	9.30	
		2.25	2.78	1.75	1.10	$\frac{9.5}{4.5} = 2.11$	0.60	0.58	5.18	6.35	

Average 2.99 miles = 15,800 feet 1.34 miles = 1,075 feet 1.67 0.77 miles = 4,075 feet 1.05 miles = 5,520 feet

Note (1)—
 (i) Worst $\frac{B}{C} = 2.57$ (Year 1806)
 (ii) Four acute meanders $\frac{B}{C} = 2.0$ to 2.2
 (iii) Cut-off appears to have occurred between 1895 and 1906 at $\frac{B}{C} = 2.3$

Note (2)—
 For 'r' formula below is used:

$$\left(\frac{M}{L}\right)^2 = \frac{Mb}{2} \frac{FW}{2} \left(2R - \frac{Mb - W}{2}\right) = \frac{Mb - 1,500}{2} \left(2R - \frac{Mb - 1,500}{2}\right)$$

Note (3)—
 Maximum discharge adopted for calculations in Col. 9 (a) = 350,000 cs., and dominant discharge adopted for calculations in Col. 9 (b) = $\frac{2}{3}$ Qmax 233,500 cs.; as actual measured discharges for these years are not known.

The flood discharges obtained by the various methods are set out in Table 2.C. 23 :—

TABLE 2 C. 23

Formula used	Computed maximum discharge (cusecs)	Remarks
1	2	3
(i) Dicken's ..	490,100	
(ii) Dredges ..	214,000	
(iii) Inglis'	495,800	Corrected for shape factor, assuming $n = \frac{1}{2}$, discharge will be 364,000 cusecs
(iv) Cousins ..	619,655	
(v) Ryves' ..	196,500	
(vi) Mississippi	172,354 (250,000 cusecs assumed for error in reporting high flood level)	Total depth has been elsewhere calculated as 39.9 feet as against 27.7 feet assumed here. With depth of 35 feet (later verified), maximum discharge would be 343,000 cusecs.
(vii) From Bareilly bridge section and assumed flood velocity of six feet per second.	215,172	With 8.85 feet per second mean velocity, as calculated below from Lacey's formulæ for $R = 35$ feet against six feet per second assumed discharge would be 317,000 cusecs.
(viii) From Moradabad Bridge section plus tributaries and assumed flood velocity of six feet per second	259,176	With 8.85 feet per second velocity (<i>vide</i> Lacey's formula, <i>see</i> below) discharge would be 382,000 cusecs.

Based on consideration of the figures given in column 2, it was proposed to take $Q_{max} = 500,000$ cusecs for calculating the bridge opening and 3,00,000 cusecs for other calculations, *e.g.* maximum depths of scour. It is difficult to understand the validity of this discrimination: if anything maximum scour should be calculated on the highest-ever discharge while waterway may be on the normal high flood discharge.

There are two other ways of calculating the maximum discharge of the river one by Lacey's formulæ, and the second from the correlation between meander length and maximum discharge. ⁽³⁹⁾.

⁽³⁹⁾ Indian Waterways Experiment Station Poona, Annual Report (Technical) 1939-40 page 10 and 1942-43 page 54.

Lacey's Method.—Requires a correct estimation of the hydraulic mean depth below high flood level and of the energy slope. For these cross-sections taken during high flood would be of the greatest value. An enquiry was, therefore, made of the value of R and bed slope (bed slope being nearer energy slope than surface slope); but the reply received appears to give R calculated from cross sections taken after the floods, which is of little use without very questionable assumptions in regard to allowance for scour below the bed occurring during high floods. Therefore, R had also to be assumed and subsequently verified, by trial and error thus —

Assuming $R=40$ feet.

By Lacey's "normal" flow formula $V = 1.6 R^{1/3} S^{1/3}$ where S is the slope in parts per thousand, with energy slope taken as equal to the given bed slope viz. 6.76 inches per mile, i.e. 0.1065 feet per 1,000 feet.

$$V = 1.6 \times 40^{2/3} \times (0.1066)^{1/3} = 8.85 \text{ feet per second} \quad \dots (2C.6).$$

And, by Lacey's "general" flow formula,

$$V = \frac{1.3458 R^{3/4} S^{1/2}}{N_a} \text{ where } N_a = 0.0225 f^{1/4}.$$

Now $f = 0.77$ (not 1.0 see below—)

$$\therefore N_a = 0.0225 (0.77)^{1/4} = 0.0211.$$

$$\begin{aligned} V &= \frac{1.3458 \times 40^{3/4}}{0.0211} \times \frac{(0.1065)^{1/2}}{(1000)^{1/2}} \\ &= \frac{1.3458 \times 15.8 \times 0.0103}{0.0211} = 10.4 \text{ feet per second} \quad \dots (2C.7) \end{aligned}$$

The average of (2 C. 6) and (2 C. 7) gives—

$$V = \frac{8.85 + 10.4}{2} = 9.6 \text{ feet per second}$$

Area given up to high flood level = 20,220 square feet when R is about 17.7 feet and 10,000 square feet extra for ten feet additional scour; hence for $R=40$ feet the area would be—

$$20,220 + \frac{40 - 17.7}{10} \times 10,000 = 20,220 + 22,300 = 42,500 \text{ square feet.}$$

under maximum scour at maximum high flood level

$$\begin{aligned} \text{This would give discharge} &= A \times V = 42,500 \times 9.6 \\ &= 4,08,000 \text{ cusecs.} \end{aligned}$$

$$\begin{aligned} \text{For this discharge, } R &= 0.473 \left(\frac{Q}{f} \right)^{1/3} = 0.473 \left(\frac{4,00,000}{0.77} \right)^{1/3} \\ &= 37.8 \text{ feet.} \end{aligned}$$

Hence $R=40$ feet assumed is apparently too much.

Assuming $R=35$ feet from results in (2C. 6) and (2 C. 7) above,

$$\begin{aligned} \text{from "normal" flow equation, } V &= \frac{8.85 \times 35^{2/3}}{40^{2/3}} = 8.1 \text{ feet} \\ &\text{per second.} \end{aligned}$$

and from "general" flow equation

$$V = \frac{10.4 \times 35^{3/4}}{40^{3/4}} = 9.6 \text{ feet per second.}$$

$$\text{Average velocity} = \frac{8.1 + 9.6}{2} = 8.85 \text{ feet per second.}$$

$$\begin{aligned} \text{Area, corresponding to 35 feet depth} &= 20,200 + \frac{35 - 17.7}{10} \times 10,000 \\ &= 37,500 \text{ square feet.} \end{aligned}$$

Hence, discharge = $37,500 \times 8.85 = 3,32,000$ cusecs.

$$\text{For this discharge } R = 0.473 \left(\frac{3,32,000}{0.77} \right)^{1/3} = 35.7 \text{ feet.}$$

Hence, according to Lacey's formulae, maximum river discharge would be about 3,30,000 cusecs. This has been obtained from post-flood bed slopes. The energy slope during high flood would be, relatively, somewhat larger and the maximum flood discharge, according to Lacey's formulae, may, therefore be taken as 3,50,000 causes.

Flood discharge determined from meander-length

From Table 2C. 23 the average meander length of the seven years for which river courses were available = 2.99 miles, or 15,000 feet, and the maximum meander length of any year is 3.34 miles, or 17,600 feet. Now, the average meander length over a long period of years is generated by the "dominant" discharge, which varies little from a mean value from year to year; while the maximum meander length of any year may be taken to correspond to the maximum discharge.

The formulae (39) correlating maximum meander length with Q (maximum) is:—

$$\begin{aligned} M_L &= 29.6 \sqrt{Q_{\max}} \\ Q(\max) &= \frac{17,600^2}{29.6} = 3,53,000 \text{ cusecs} \quad \dots (2C.8) \end{aligned}$$

Thus, from meander length considerations, the flood discharge works out to be of the same order as from Lacey's formulae.

In fact, for a river with so little available data there is a surprising degree of agreement between the various estimates made of the flood discharge:—

Inglis' formula (with correction for shape factor)	=	364,000	cusecs
Mississippi formula (for $R=35$ feet)	=	343,000	"
Vide Bareilly bridge ($V=8.85$ feet per second)	=	317,000	"
Vide Moradabad bridge and tributaries (8.85 feet per second)	=	382,000	"
Lacey's formulae	=	350,000	"
Meander length formula	=	353,000	"
Average	=	351,500	cusecs

Say, 350,000 cusecs

Hence the estimate of the flood discharge would be 350,000 cusecs.

(39) Ibid

DEPTHS ASSUMED FOR SCOUR

These are considered below, separately for the depth of scour to be adopted in the design of the guide bank and bridge piers based on the recommendations of Spring, Gale and Lacey.

A. *Maximum scours in design of guide banks*

At the nose or head of a Guide Bank there is more scour than along the shanks and tail ; so the scour must be separately considered for each position of the guide bank.

(a) According to Spring

(i) *Head of guide bank.*—Spring recommends that at the head the thickness of the apron should be doubled and the breadth be kept *twice* the depth of scour instead of $1.5 \times$ depth of scour. This means that the design should be based on 1.7 times " the greatest discoverable scour below low water level ", which is reported to be 35 feet. This depth has been obtained under natural conditions of cutting at a soft bank : making the same allowance of 33 per cent, as recommended by Gale for cutting adjacent to a hard or rigid bank, the scour at the head would be $1.7 (35 + 11.5) = 79.0$ feet below low water level, or 91.0 feet below high flood level the difference between high flood level and low water level being 12 feet.

(ii) *At the Shanks.*—According to Spring the apron should be designed for the greatest discoverable scour, plus due allowance for greater scour adjacent to a hard or rigid bank, *i.e.* $35 + 11.5 = 46.5$ feet below low water level, or 58.5 feet below high flood level.

(b) According to Gale

For class A Rivers (*i.e.* discharges ranging from 2,50,000 to 7,50,000 cusecs) the percentages to be added to the deepest known scour at a soft cutting bend (35 feet in this case) are 33 per cent. of the basic depth for scour adjacent to a hard bank ; and 50 per cent. of the basic depth for the head and 25 per cent. for the body and tail.⁽⁴⁰⁾

(i) *At the head.* Thus, at the head, maximum scour $= (35 + \frac{33}{100} \times 35) + \frac{50}{100} (35 + \frac{33}{100} \times 35) = 69$ feet below low water level or 81 feet below high flood level.

(ii) *At the shanks,* maximum scour $= (35 + \frac{33}{100} \times 35) + \frac{25}{100} (35 + \frac{33}{100} \times 35) = 58$ feet below low water level or 70 feet below high flood level.

(c) According to Lacey

$$R_{\text{Lacey}} = 0.473 [Q/f]^{1/3}.$$

The mean diameter of the bed-sand used in calculations of normal depth by the Lacey formula is 0.323 mm. for the stated reason that the sand at site is a mixture of medium and very fine sand. However, there is a predominance of very fine sand, and if the sieving results given in the reports are based on sieves of British Standard Specifications, the mean diameter is only 0.191 mm., which will give a bigger depth of scour.

⁽⁴⁰⁾ Institution of Civil Engineers, London, Paper No. 167, page 167.

$\therefore f=0.77$, not 1.0 as assumed.

$$\therefore R_{\text{Lacey}} = 0.473 \left[\frac{3,50,000}{0.77} \right]^{\frac{1}{3}} = 36.4 \text{ feet below high flood level.}$$

(i) *At head*

Deepest scour ⁽⁴¹⁾ = 2.0 to 2.5 R ; the smaller depth being for the larger radius curves. Assuming depth below high flood level = 2 R , maximum scour depth below high flood level = 72.8 feet.

From an examination of data of existing bridges, however, the scour at the head of large radius noses of guide banks is found to be 2.75 R . ⁽⁴²⁾ Accordingly, maximum scour to be adopted in design of aprons would be $2.75 \times 36.4 = 100$ feet below high flood level.

(ii) *At shank* ⁽⁴¹⁾—Deepest scour = 1.5 $\times R$ 54.6 feet below high flood level.

Recapitulating, according to various authorities, the maximum scour in feet below high flood level would thus be :—

Position	Spring	Gale	Lacey	Indian waterways Experiment Station
Nose or head of large-radius guide bank	91	81	72.8	100
Along shank	58.5	70	54.6	

It was, therefore, recommended that the aprons should be designed for 100 feet depth of scour at the nose and 60 feet along the shanks.

B. *Maximum scour depth and depth of foundations for Bridge piers.*

(a) *According to Spring* :—

Maximum scour adjacent to hard or rigid surface = $35 + \frac{33}{100} \times 35 = 46.5$ feet below low water level, or 58.5 feet below high flood level. According to Spring's curve the foundations should be taken down to 103 feet below high flood level.

(b) *According to Gale* :—The deepest known scour at a soft bank being 35 feet, deepest anticipated scour in vicinity of bridge = $35 + \frac{33}{100} \times 35 = 46.5$ feet.

Add for Class A rivers, 25 per cent. of 46.5 = 11.6 feet.

Add for grip in Class A Rivers 50 feet.

\therefore Depth of foundation below low water level = 108 feet.

\therefore Depth of foundation below high flood level = 120 feet.

⁽⁴¹⁾ Central Board of Irrigation, Publication No. 12 page 155.

⁽⁴²⁾ Indian Waterways Experiment Station Poona, Annual Report (Technical), 1944 page

(c) *According to Lacey.* For piers without high-level protections, depth of scour = $2R = 72.8$ feet below high flood level; and allowing, according to current practice ⁽⁴³⁾ a grip of R , the level of the foundation of the piers should be $3R$ below high flood level.

(d) At the existing bridges on the Ramganga River, the depth of foundation below high flood level appears to be 100 (below low water level) + 12 = 112 feet at Moradabad, and 85 + 12 = 97 feet at Bareilly.

In consideration of (a), (b) and (c) above, it was recommended that the foundations should be taken to a depth of 110 feet below high flood level, basing particularly on the results of the examination of depths of scour at piers of existing bridges ⁽⁴²⁾ if the width of the bridge is made 2,000 feet as proposed which corresponds to Lacey's minimum stable waterway for 5,00,000 cusecs, as against 3,50,000 estimated maximum cusecs flood discharge, discharge intensity (allowing for 20 per cent. concentration as usual) $q = \frac{120}{100} \cdot \frac{(3,50,000)}{2,000}$

= 210 cusecs and $R = 0.91$ $q^{\frac{2}{3}} f^{\frac{1}{3}} = \frac{0.91 (210)^{\frac{2}{3}}}{(0.77)^{\frac{2}{3}}} = 35.5$ feet hence, depth of foundations required = $3R = 106.5$ feet below high flood level. The fact that the Bareilly bridge foundations carried only 100 feet below high flood level are safe should not be made to influence the required depths at another site: on the other hand, the figure of 120 feet (required according to Gales) may be reduced to 110 feet, since it only means that the grip of 50 feet required according to Gale is reduced to 40 feet, which according to modern concepts, is quite sufficient for a maximum depth of scour of 120 feet below high flood level. Hence, if the bridge is made 2,000 feet long, the foundations were recommended to be taken 110 feet below high flood level.

Length of guide bank.

The detailed study of the meanders made for the first point of reference revealed that the proposed length of guide bank—namely $L = 1,800$ feet, being the clear waterway of the bridge—was insufficient according to Gale's theory on which it was stated to be based. Even Gale himself finally proposed a length of $1\frac{1}{4} L$ for A Class rivers with an arc/chord ratio of 1.75 and radius of curvature of nose of 3,300 feet ⁽⁴⁴⁾; for the Kosi, with arc/chord ratio of 2.25 Gales recommended a guide bank length of $2.33 L$; ⁽⁴⁵⁾ while in the present case of the Ramganga, the maximum arc/chord ratio is 2.67 (see Table 2C. 22).

It would be difficult to justify a shorter length of guide bank than necessary according to Gale's theory. It would be also unsound to provide a shorter length of guide bank in the first instance and increase the length later, if or when found necessary; because irreparable damage might occur within one flood season, and the price of benefitting by experience would prove too high and the experience would come too late.

(43) McIntyre, Major T. Well foundations for bridges, Railway Board Quarterly Technical Bulletin No. 78.

(42) *Ibid.*

(44) The Journal of the Institution of Civil Engineers, London, December 1938, page 315

(45) The institution of Civil Engineers London, paper No. 5167, page 15.

Following Gale's theory, the guide bank should be so long that a cut-off will occur before the sharpest possible meander can threaten the approaches. From Table 2C. 22 taking the worst meander of 1806, arc/chord = 2.67, $M_1 = 1.5$ miles = 7,920 feet, $M_b = 1.8$ miles = 9,504 feet, and estimated $Q_{max} = 3,50,000$ cusecs, the radius of curvature can then be computed thus ⁽³⁹⁾.

$$\begin{aligned}
 & A B^2 = AC (2R - AC) \\
 \therefore \left[\frac{W_e}{2} \right]^2 &= \frac{M_b - W}{2} (2R - \frac{M_b - W}{2}) \\
 \therefore \frac{7920^2}{4} &= \frac{9504 - 1500}{2} (2R - \frac{9504 - 1500}{2}) \\
 \therefore 1980^2 &= \frac{8004}{2} (2R - 4002) \\
 \therefore R &= 2,490 \text{ feet.}
 \end{aligned}$$

Figure 2C.46 shows that, allowing a margin of safety of $L/2 = 900$ feet in the conditions at Ramganga, $B/C = 2.67$, $R = 2,490$ feet, a guide bank of length 4,700 feet = $2.6 L$ is necessary; and even without any margin of safety—though some is essential—the guide bank would be 3800 feet = $2.1 L$.

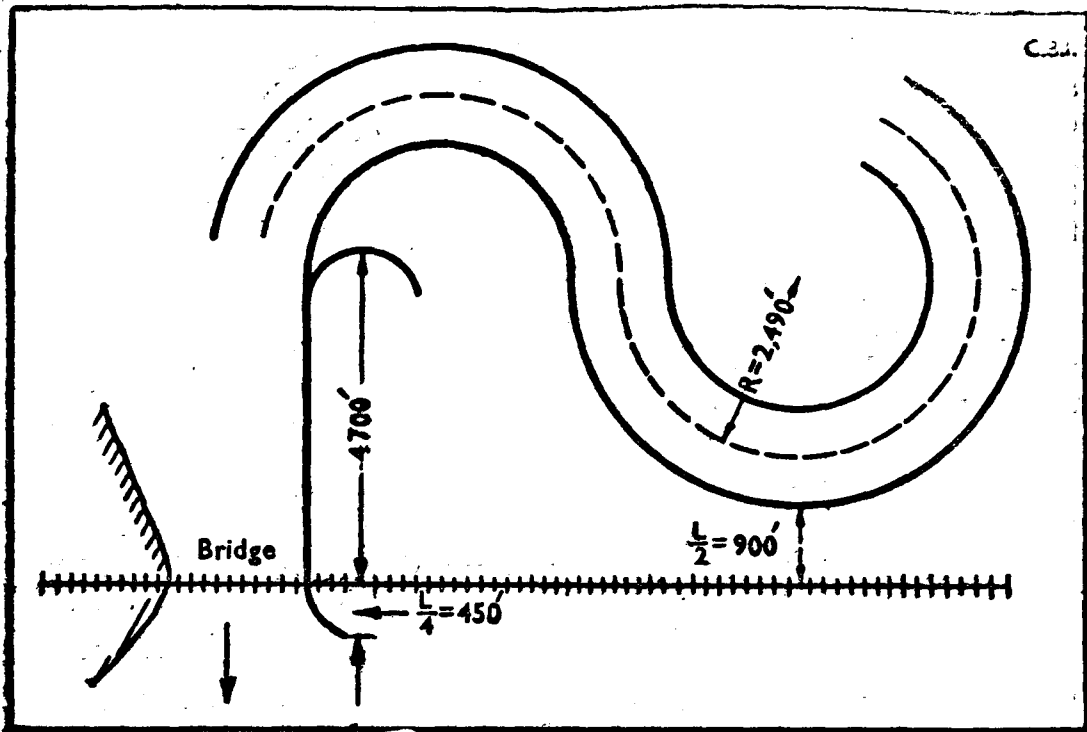


FIGURE 2C. 46—Showing the guide bank required to keep off the worst theoretical meander from Railway Embankment by $L/2 = 900$ feet.

NOTE:—

Ratio of arc: Chord	2.67 feet
M_1 = Meander Length	7,920 feet
M_b = Meander Belt	9,504 feet
R = Radius of Bend	2,490 feet

³⁹⁾ *ibid.*

It was, therefore, recommended that unless model experiments are made—whether the 1945-46 embayment has a cut-off before the construction of the bridge or not—the full length of the guide bank 2.5 *L* should be constructed along with the bridge; the guide bank and left approach, as proposed are shown, in relation to the river courses, in Figure 2 C. 45. The falling aprons of the guide banks should, as far as possible, overlie a sub-soil stratum of sand only; and small adjustments should be made in the alignment of the guide bank, if necessary, to secure this advantage.

It was made very clear that the above recommendations are subject to the following limitations:—

(i) They are based on the data of seven years' river courses (presumed to be representative) covering a limited reach of the river above the proposed bridge which were all that were supplied for analysis.

(ii) The information supplied did not indicate whether there is any general easterly or westerly trend of the river, the existence of which would affect the recommendations substantially.

(iii) There was some discrepancy observed upon superimposing the two plans supplied; the large scale plan was assumed as the correct one for showing the proposed alignments of the Railway Line.

It was advised that the superficial evidence of stability of a river course is not a sufficient criterion to justify the final election of a site of an important bridge on an alluvial river; and that, therefore, it is most essential to take trial borings along the bridge line, extending far into both the banks, to confirm the apparent stability.

DISCUSSION BY THE RESEARCH COMMITTEE

DR. BOSE introduced item (1) and said that he had already explained to the Research Committee the peculiar circumstances of the river Kultigong. At the request of the President, DR. BOSE reiterated some of the peculiarities for the benefit of new members. The sewage water and the storm water of the city of Calcutta used to be passed on to the river Bidyadhari which was a tidal creek but when the river Bidyadhari deteriorated and ultimately died out, this sewage and storm water was diverted by an artificial channel into another tidal creek known as Kultigong. Of recent years this creek had also been showing signs of deterioration. Investigations of the last three years had shown that the meandering of the river had been increasing. The long section of the river Kultigong taken from Haroa Hat to Kalinagar showed the points of very deep scour on both sides of the point where sewage and storm water from Calcutta is passed on to Kultigong. But all the sewage does not go into the Kultigong. Between Calcutta and Kultigong there are a large number of fisheries. These fisheries take away major portion of sewage and allow desilted water into the river. This desilted water has a tendency to scour. About a mile and a half downstream and a mile upstream the bed of the river has scoured down to -49 whereas the usual river bed level is -21. Investigations have shown that below a clay crust of about 29 to 30 feet there is in this region a very thick layer of comparatively coarser sand and as the scour holes have developed in the Kultigong, the bed has pierced the deep clay layer and churned up the layer of coarser sand which

has been thrown up and carried away by the tidal currents. On the inside of the bends where the current naturally is slack, quantities of coarser sand have been deposited. This has forced the river to increase its meander.

MR. H.P. MATHRANI enquired as to how the fisheries managed to take the sewage from the channel.

DR. BOSE replied that since the level of the channel carrying sewage was at a higher level, it was easy for the fisheries to cut the bank and steal the sewage.

MR. H. P. MATHRANI asked whether it was not possible to stop this cutting of the banks.

DR. BOSE said that it was possible to stop this cutting, but then again there were vested interests involved in these fisheries. One more thing he wanted to state was that the sedimentation tanks were also not working satisfactorily.

DR. HOON asked whether sedimentation by mechanical means had been tried.

DR. BOSE replied that it was very costly.

RAI BAHADUR SETHI enquired why the sewage was not being utilised for farm irrigation.

DR. BOSE said that there was not any irrigable land available in that locality.

THE PRESIDENT enquired whether Dr. Bose had made any concrete suggestions to Government about how to stop the meandering of the river.

DR. BOSE replied in the negative.

THE PRESIDENT said that the study was therefore to be continued.

MR. FRAMJI in introducing item (2) said that in the absence of sufficient data from the Railway authorities he had to estimate the various factors from well known empirical formulae. Thus, flood discharge as calculated from Lacey's and the Station's meander formula worked out to be between 3 to 3½ lakh cusecs, the same as by the empirical formulae used in the Report when duly moderated for local conditions. Similarly, the scour depths and protection of guide banks were calculated from Lacey's, Spring's and Gale's formulae.

DR. BOSE wanted an explanation of some points in the Report.

MR. FRAMJI referred to Figure 2C. 47 and said that where there was a natural gorge or a bridge which fixed the position of the meander the tendency for the river was to have a full meander belt on either side of the axis because it was restricted from free movement at that particular point. This was the result whatever the theory might be and was based on actual investigations of river Indus meanders. Where constrained from movement on one side, this double meander belt may occur further upstream.

THE PRESIDENT enquired whether this kind of investigation had been conducted elsewhere and whether it had been proved.

MR. FRAMJI said that there was no such data of annual river surveys for a long period as in the case of the Indus.

MR. M. P. MATHRANI asked whether the result was applicable to the Kotri bridge.

MR. FRAMJI said that so far as he could recollect this was some distance above the Kotri bridge near Saidabad Figure 2C. 147.

MR. H. P. MATHRANI said that it was not so at the Kotri bridge.

RAO BAHADUR JOGLEKAR said that if there was hard bank on one side, the result would be different.

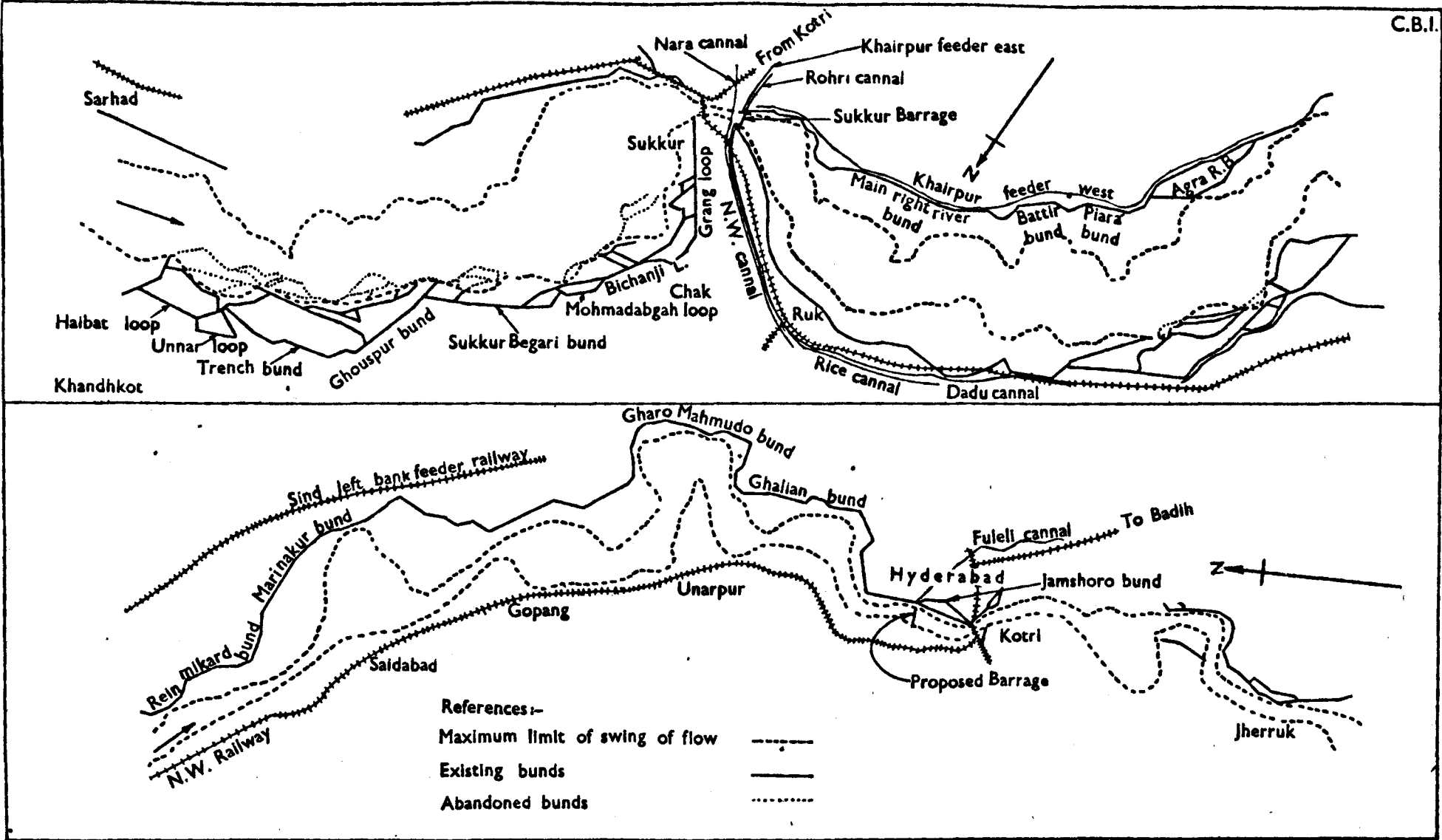


Figure 2C.47:- Showing maximum swing of deep channel of the Indus (1911 to 1942) near Sukkur & Kotri



(vi) TIDAL ACTION INCLUDING THAT IN ESTUARIES

The following items were discussed at the 1947 Research Committee Meeting.

- (1) Model experiments for the revival of Matla-Peali system
- (2) Measures adopted to reduce turbulence in the Hooghly river downstream of the knuckle at the Titaghur Jute Mill (Bengal)
- (3) Protecting the foreshore of the Anglo=India Jute Mill on the Hooghly river (Bengal)

(1) MODEL EXPERIMENTS FOR THE REVIVAL
OF MATLA—PEALI SYSTEM ⁽⁴⁶⁾

ABSTRACT

Peali one of the important spill channels takes off from the Matla near Kultali about 19 miles downstream of Canning. There are considerable difficulties of drainage through the Peali due to sand locking of the drainage sluices on the channel side and the vast low lying areas to the south-east of Calcutta which are waterlogged throughout the year. In some areas the ground levels are lower than the lowest water level of the tides due to premature reclamation.

To remove this drainage congestion and to improve the land for the purpose of cultivation it is proposed to re-excavate the Peali from the Peali railway bridge to the Kultali outfall to the proper section so as to carry the drainage water. It is the experience with tidal rivers in Bengal that such channels very seldom maintain their regime for sufficiently long time unless sufficient spill area is provided for them. It is, therefore, proposed in the present case either (1) to exclude tidal water by *bunding* Peali and constructing a lock and a sluice in channel connecting the Peali higher up with Matla or (2) to provide sufficient waterway in the re-sectioned channel so as to accommodate both the drainages and the ebb flow of the Peali. Describes experiments.

The Matla is one of the biggest tidal estuaries of South Bengal. Its upper reaches above Canning however is now practically dead, Figure 2C. 48 (Index map of Matla series of rivers). The width of the estuary at Canning which is 28 miles from Calcutta is 3,200 feet. The estuary used to take a sharp bend near Canning and flow upto the Great Salt Lake areas adjoining Calcutta. Previously this tidal estuary with its innumerable spill channels used to spill over the vast low-lying areas adjoining Calcutta and build the land.

About 100 years ago the spill areas were closed by raising marginal embankments. The Matla with its important spill channels Bidyadhari and Peali were then used as drainage-*cum*-navigation channel for the above areas. The sewage water of Calcutta used to be drained in the Bidyadhari and the upper reaches of Matla which rapidly silted up and was abandoned as a sewage disposal channel in 1937. The bed of this channel is now higher than the surrounding countryside and no drainage water flows into this channel.

The Matla now is still active from Canning downwards. Peali one of the important spill channels takes off from the Matla near Kultali about 19 miles downstream of Canning. It again joins the upper reaches of Matla which was once the Bidyadhari. Total length of Peali channel is 34 miles and the

(46) Annual Report of the River Research Institute, Bangal, 1946, pages 20 to 22.

drainage of 330 square miles of the area south of Calcutta is dependant on the activity of this channel. Even before the deterioration of the Bidyadhari was complete in 1935 deterioration had already started in the upper reaches of the Peali. It has travelled down to a place known as Uttarbhag where the charnel has dwindled down to a very small section.

At present there are considerable difficulties of drainage through the Peali due to sand locking of the drainage sluices on the channel side and the vast low lying areas to the south-east of Calcutta which are waterlogged throughout the year. In some areas the ground levels are lower than the lowest water level of the tides due to premature reclamation.

To remove this drainage congestion and to improve the land for the purpose of cultivation it is proposed to re-excavate the Peali from the Peali railway bridge to the Kultali outfall to the proper section so as to carry the drainage water. It is the experience with tidal rivers in Bengal that such channels very seldom maintain their regime for sufficiently long time unless sufficient spill area is provided for them. It is, therefore, proposed in the present case either (1) to exclude tidal water by bunding Peali and constructing a lock and a sluice in a channel connecting the Peali higher up with Matla as shown in Figure 2C. 48, or (2), to provide sufficient waterway in the resectioned channel so as to accommodate both the drainage and the ebb flow of the Peali. It is expected in the latter case that as there will be sufficient waterway for the ebb water and the drainage supply to flow down without interference no obstruction need be apprehended to the flow of the tides and hence the channel will maintain its regime.

A model of the Matla-Peali system was therefore constructed to test whether—

- (a) the proposal under (2) above will maintain the channel in regime or
- (b) the proposal under (1) above while by restricting the spill which goes from the Matla to the Peali will act to the detriment of the former.

The model was built to the following scales :—

Horizontal $l=1/500$.

Vertical $h=1/50$

Time scale $\frac{\sqrt{h}}{l}=1/70.7$

The river was at first laid to the above scales on the ground which consisted of stiff clay but as the distortion was high the banks could not be held and started slipping in. The bed therefore was then cut to the proper section and then plastered over with a very weak mixture of cement and sand. The proportion of sand and cement is 10 : 1. The model bed was moulded according

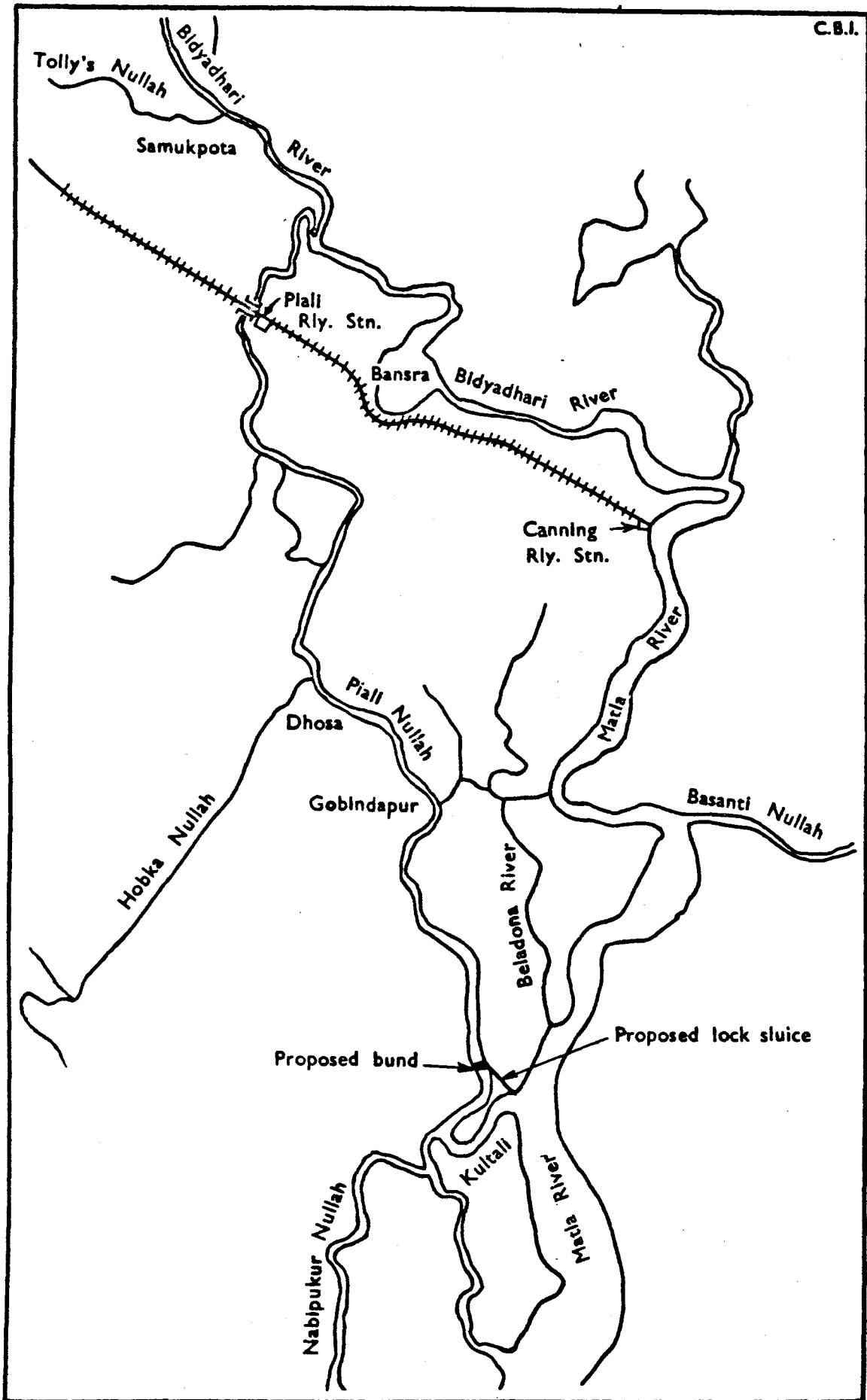


Figure 2C 48:- Index Map of Matla Piali River



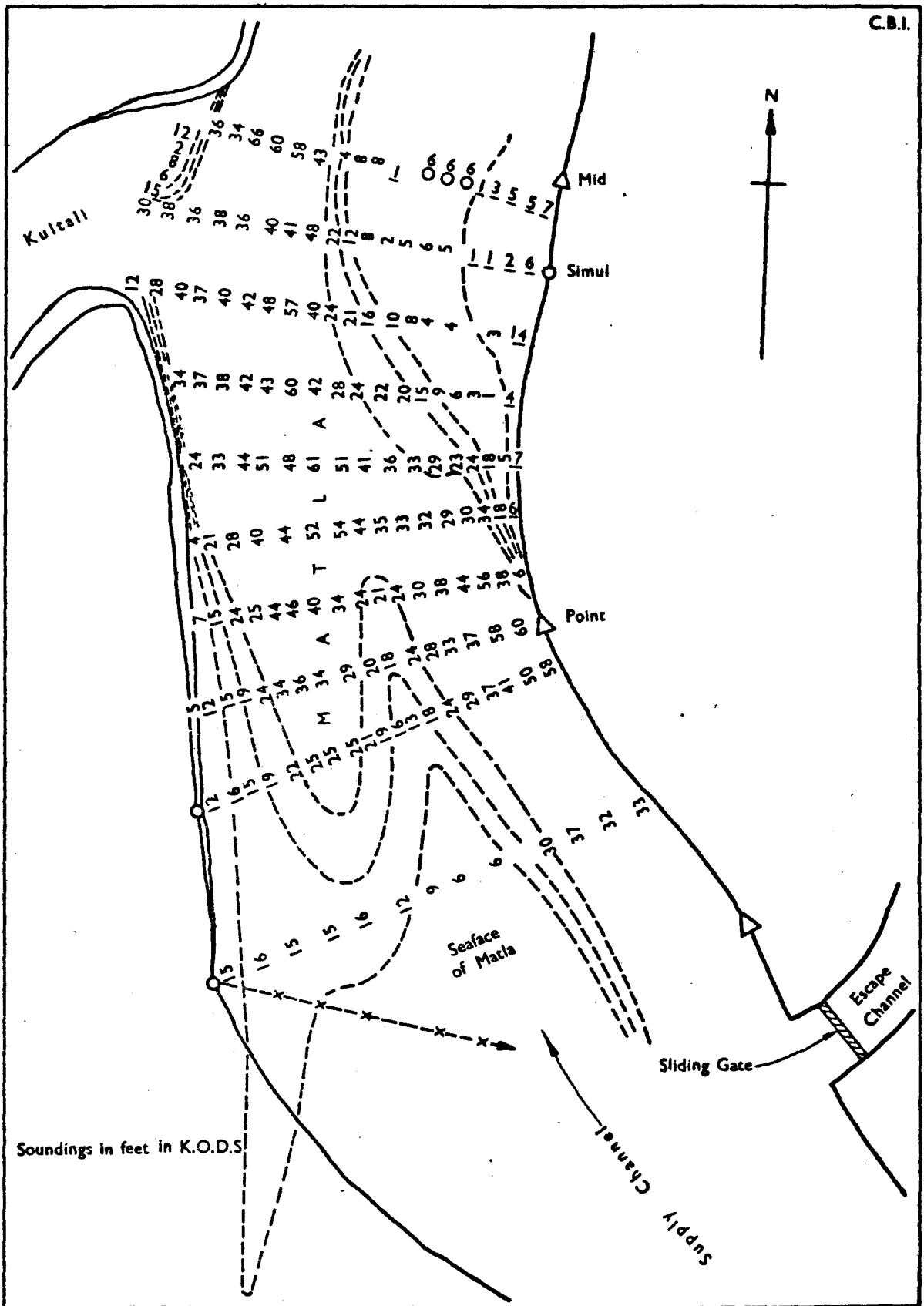


Figure 2C.50:- Showing arrangement of Supply Channel, Sliding Gate and Escape Channel of Matla Priali Model

Scale 6 4 2 0 6 12 18 Feet (Model)

to the survey of the Matla carried out during 1945 by the Commissioners for the Port of Calcutta and [to the survey of the Peali river carried out during 1946 by the Executive Engineer, Canals Division.

The length of the Matla taken in the model is about 19 miles and that of Peali river is about 30 miles. It includes a stretch about one mile below the outfall of Kultali and up to the furthest limit on the Canning side. Total length of the Peali up to the railway bridge has been included. The average width of Matla is seven feet in the model (3,500 feet in prototype) and varies from 2.4 feet to 13.2 feet in model. The width at Canning is 6.4 feet and at Kultali is 11.2 feet. The width of Peali river varies from 0.42 foot to 7.8 feet in model. The depth of model in some places is two feet. The maximum range of spring tide of Matla during rainy season at Kultali is 19.7 feet observed during 1945.

There is no simultaneous gauge readings of the Matla at Canning and Kultali. During 1945 some gauges were observed at Kultali outfall but there is no corresponding reading at Canning. During the January of 1946 one fortnight's gauge readings at Canning were taken by the Commissioners for the Port of Calcutta but there is no corresponding reading at the Kultali outfall. The model therefore could not be proved rigorously. Different cycles of gauges at Kultali have been reproduced in the model and the corresponding gauge readings obtained at Canning have been compared with the readings obtained in the prototype during January 1946 by the Port Commissioners. The agreement is fairly good. The gauge curves as obtained in the model and the prototype are shown in Figure 2C. 49.

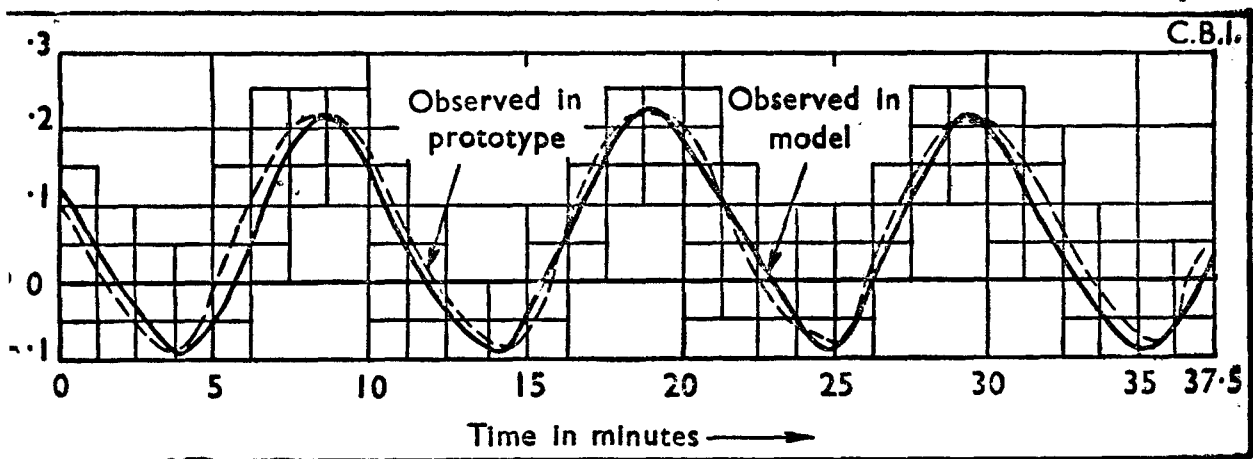


Figure 2C. 49 : *Gauges of the river Matla at Kultali.*

GENERATION OF THE TIDES IN MODEL

A constant discharge is maintained in the supply channel of the model and the actual discharge that enters the model is controlled by means of a sliding gate operated by manual labour. The sliding gate is fixed at the side wall of the supply channel. Excess water spills over the sliding gate and passes through an escape channel. The arrangement showing the supply channel, sliding gate and the sea face of the Matla river is shown in Figures 2C. 50, 2C. 51, and 2C. 52. During flow tide as the supply enters the model from the sea end, one observer reads the gauge of the river at Kultali and slides up the gate to maintain the correct gauge according to prototype value. When the maximum

level is attained, the constant supply is closed and the gate is lowered slowly. Discharge from the model escapes over the sliding gate and the ebb tide follows. At the Canning side of the river the tide spills over the lowlying areas in the prototype and in order to reproduce the gauge at Port Canning end, the flow tide in the model spills into a closed chamber while during the ebb tide the water returns from the chamber into the model. The other gauges in Matla and Peali are automatically reproduced in the model. The only gauge that is maintained by the operation of the gate is that at Kultali outfall. There are a number of spill channels above Kultali. The volume of water entering these spill channels and the gauges maintained at their outfall into the Matla had to be adjusted to get the tidal curve at Canning correctly. The model required very careful adjustment before Canning gauge could be properly reproduced.

The main difficulty in this model has been the paucity of data. As mentioned before no simultaneous gauge readings exist at Kultali outfall and Canning. Further no gauge readings exist at the outfalls of any other important spill channels. No survey exists of these spill channels. No discharge observations are available either of the Matla or any of its spill channels. Simultaneous gauge readings have now been started at the Kultali outfall and Canning. As soon as these are available, it will be possible to compare the model values with the prototype more rigorously.

The model having now been proved it is proposed to test the second alternative first. The model will be run to get the Kultali outfall gauge correctly. Then the following gauges in the Matla and its spill channels will be noted :—

- (1) Canning gauge.
- (2) Basanti *Khal* gauge (maximum and minimum value).
- (3) Harbhanga *Khal* gauge (maximum and minimum value),
- (4) Dhusa and Uttarbhag gauge (Peali gauges).

The exact discharges that pass into the model for different stages of the flow and ebb tides will also be noted.

The Peali channel will then be *bundled* up and the same sequence of discharges passed into the model. Then the following gauges in the Matla and its spill channels will be noted :—

- (1) Kultali Outfall,
- (2) Canning,
- (3) Basanti (maximum and minimum),
- (4) Harbhanga *Khal* (maximum and minimum).

On comparing these two sets of gauge readings it will be possible to say if the *bundling* of Peali is going to be harmful or not.

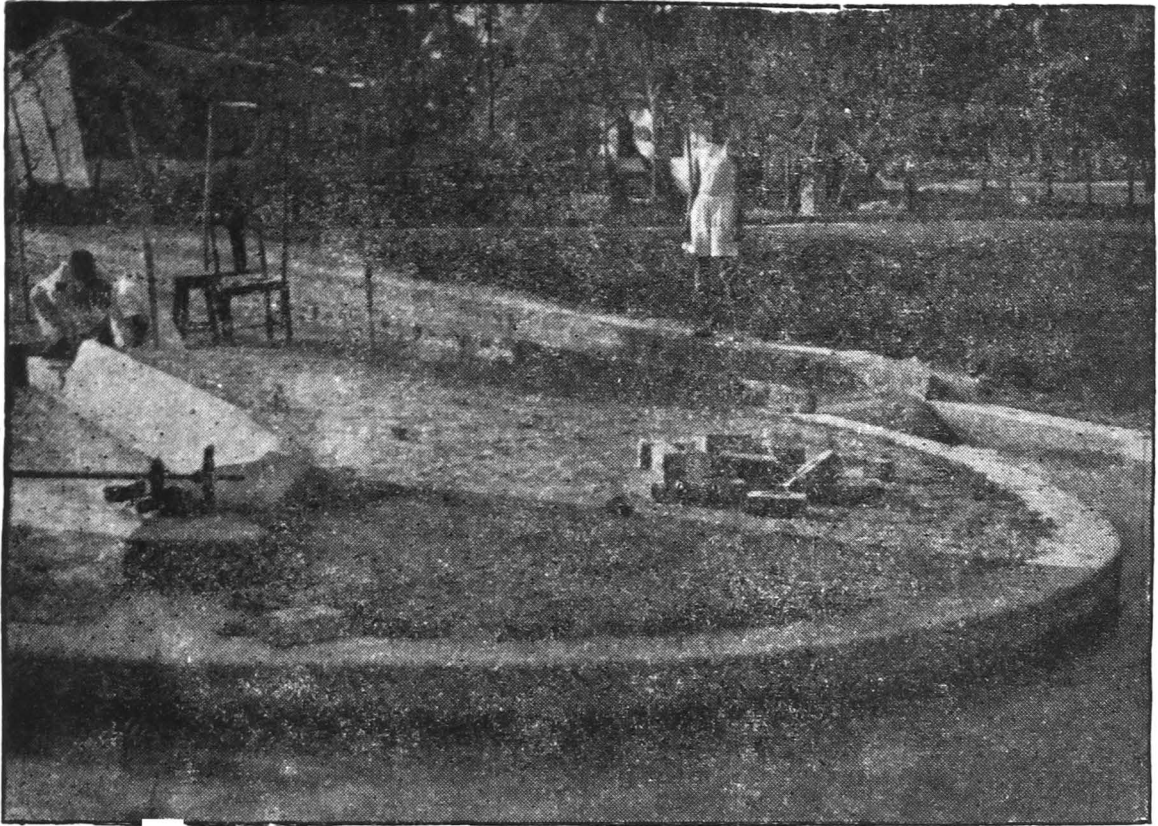


Figure 2C. 51 : *General view—Matla-Peali River Model (tidal).*



Figure 2C. 52 : *General view—Matla-Peali River Model (tidal).*

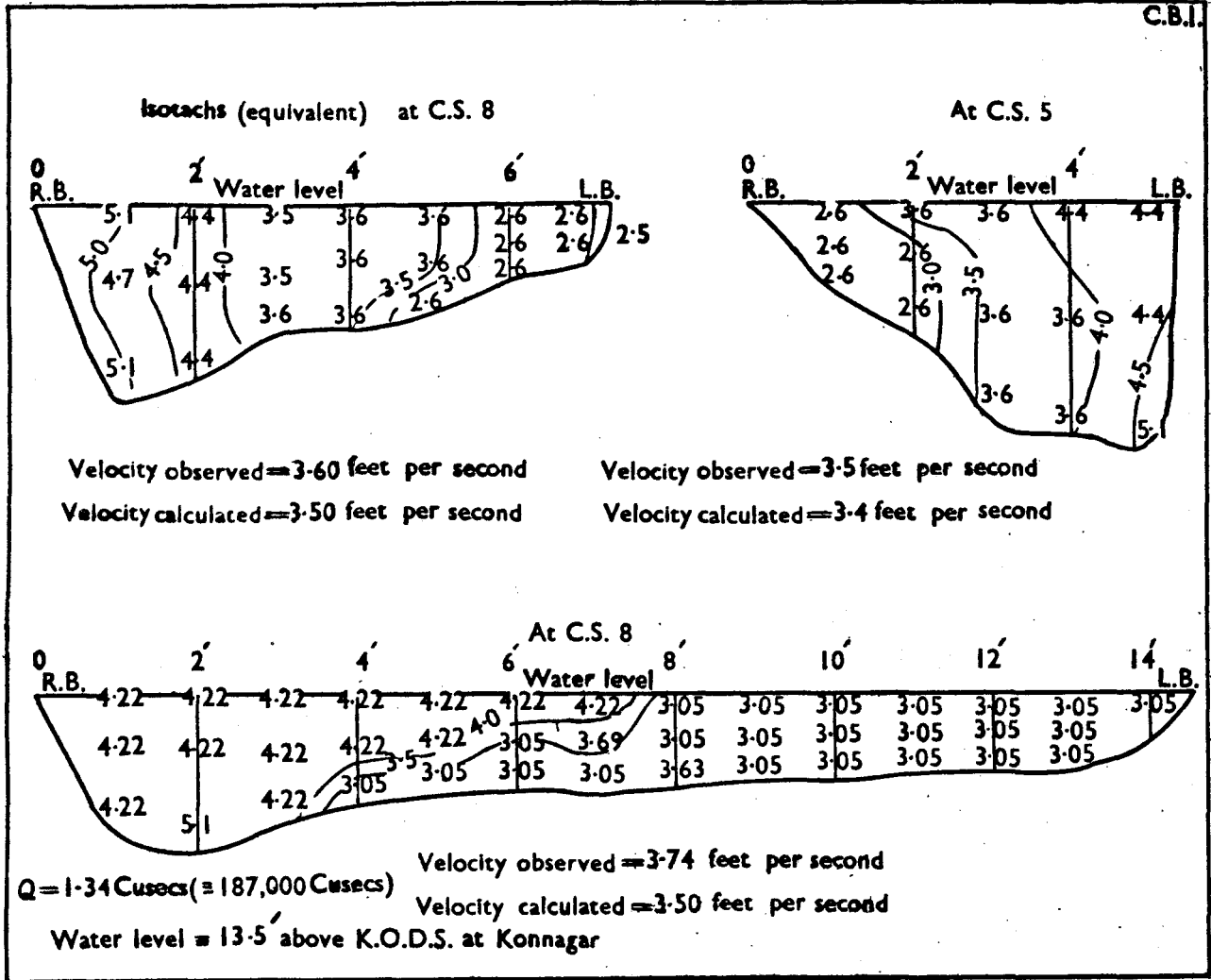


Figure 2C.53(a):- Showing velocities observed and calculated [at Titagarh Jute Mill ($\frac{1}{300} : \frac{1}{60}$ Hoogly model)]

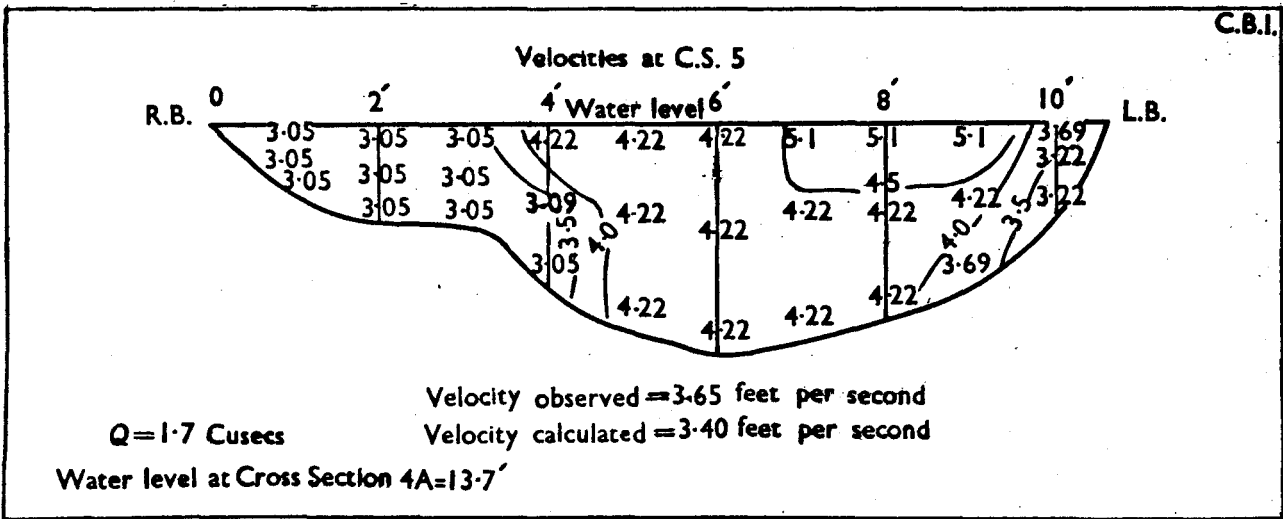
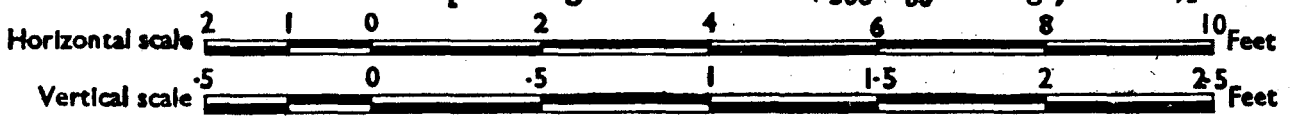
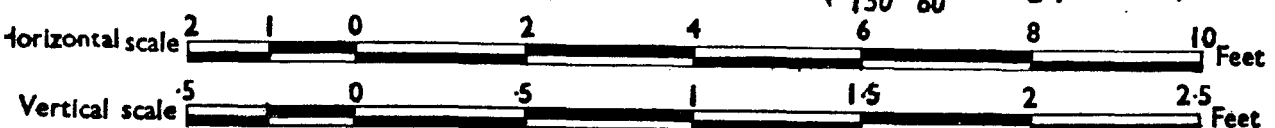


Figure 2C.53(b):- Showing velocities with grid upstream & Londha sand on bed ($\frac{1}{150} : \frac{1}{60}$ Hoogly model)



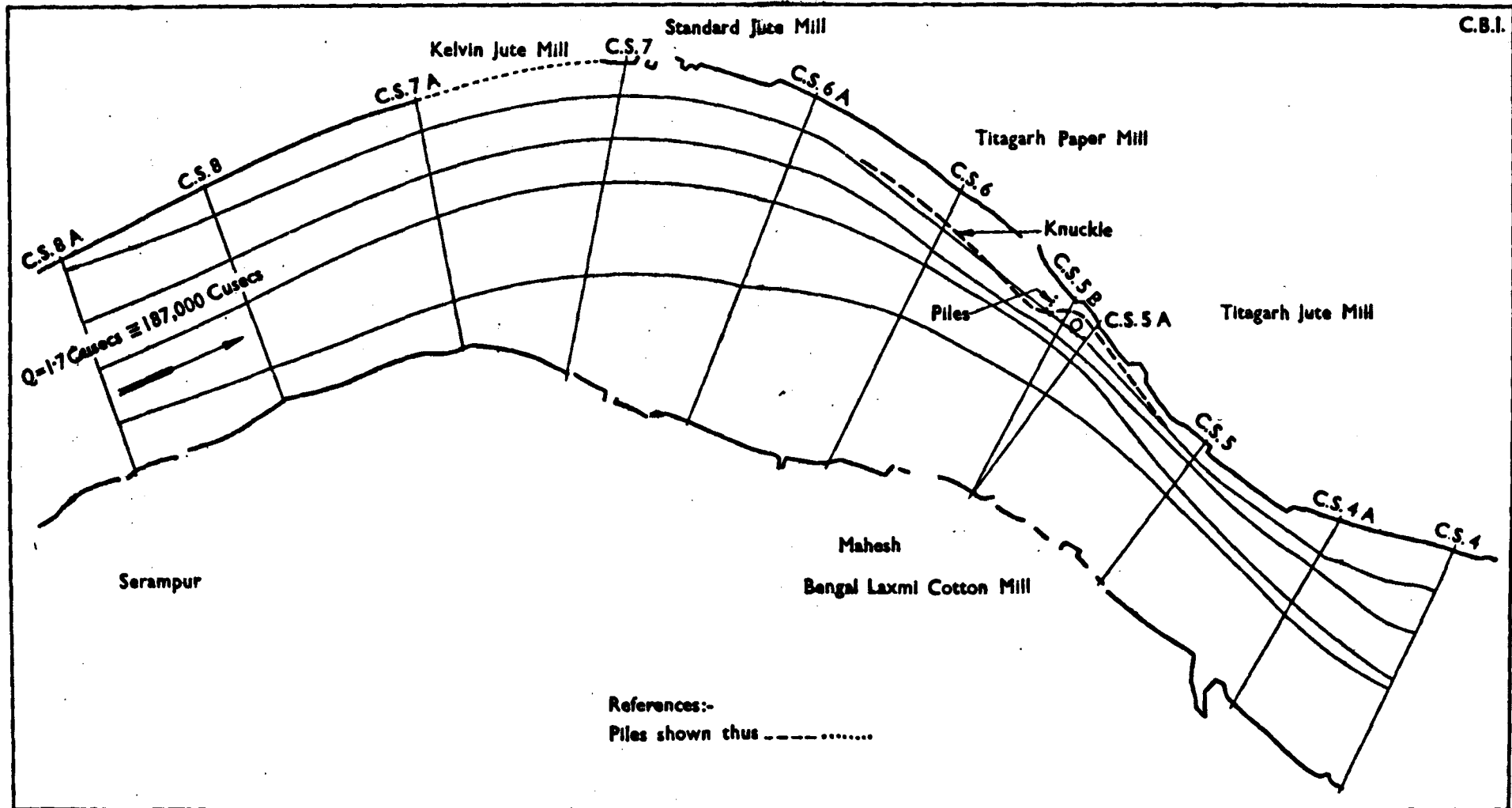


Figure 2 C.55:- Surface lines of flow with the L-Piles in position No. 1

(2) MEASURES ADOPTED TO REDUCE TURBULENCE IN THE HOOGLHY RIVER DOWNSTREAM OF THE KNUCKLE AT THE TITAGHAR JUTE MILL (BENGAL)⁽⁴⁷⁾

ABSTRACT

The note describes experiments carried out with 1/300 : 1/60 scale model and 1/50 : 1/80 part model of the Hooghly.—

- (a) to investigate the present condition downstream of the knuckle ;
- (b) to see if accretion can be induced in the scour-hole downstream of the knuckle by means of piles *i.e.* to see whether the treatment recommended for the Dunbar Cotton Mill ⁽⁴⁸⁾ would be equally effective in this case.

As results in (b) were not promising,

- (c) experiments were done with pitched “ attracting ” islands so placed as to attract the flow towards the opposite right bank, thus reducing turbulence in the knuckle. These experiments are still in progress

PRELIMINARY EXPERIMENTS IN THE 1/300 : 1/60 MODEL AND IN THE PART MODEL TO DETERMINE THE SCALE RATIOS OF THE LATTER

Lines of flow and velocities were observed in the 1/300 : 1/60 scale model, which was laid in V-sand to the 1939 survey and the continuous ebb discharge of 1.34 cusecs ($\equiv 1,87,000$ cusecs observed in 1941) was run with water level at Konnogar $\equiv 13.5$ feet above K. O. D. S. (Kidderpore Old Dock Sil as datum).

A horizontal scale of 1/150 was chosen for the large, part model, so as to represent 1.8 miles upstream of Titaghar Jute Mill and 2.1 miles downstream. When this model was run under identical conditions of contours, discharge and water level as in 1/300 scale model, with a depth scale of 1/100, (*i.e.* $V. E. = 1.5$),

and $Q = 1.25$ cusecs $\left(\equiv \frac{1,87,000}{LD\sqrt{D}} = \frac{1,87,000}{150 \times 100\sqrt{100}} \right)$, heavy scour occurred.

So dam sand was replaced by Londha sand ($m = 0.30$ mm.); the bed movement was very low. Hence the depth scale was altered to 1/80 ($V. E. = 1.8$), and $Q = 1.7$ cusecs ($\equiv 1,87,000$ cusecs) and the movement of bed material improved. A wooden grid was also used at the upstream end of the model to improve the velocity distribution. Considering the distortion of cross sections in the large model, isotachs and lines of flow compare fairly well with those in the 1/300 model, Figure 2 C. 53 (a) and (b).

PART I—EXPERIMENTS WITH PILES IN 1/150 : 1/80 MODEL

Introduction

After completing the experiments to prove the behaviour of the part model, it was decided to see the effect of submerged piles in various positions. The same procedure of experiments was adopted as in the case of experiments at the Dunbar Cotton Mill knuckle.

Londha sand ($m = 0.22$ mm) was injected in all experiments at the rate of two cubic feet per hour (*i.e.* 0.4 parts per 1,000).

Duration of experiment was 24 hours $\equiv 16$ days in all experiments.

(47) Indian Waterways Experiment Station, Poona, Annual Report (Technical), 1946, pages 38 to 42.

(48) Indian Waterways Experiment Station, Poona, Annual Report (Technical), 1944, page 7.

Experiments

Experiment No. 1.—In this experiment the L (longitudinal) row of piles was constructed as shown by position 1 in Figure 2C. 54. The alignment of cross-row of piles was arbitrarily fixed keeping the top four feet below K.O.D.S. allowing thereby ten feet draft at low water. The longitudinal piles were constructed with the top level 32 feet below K.O.D.S.

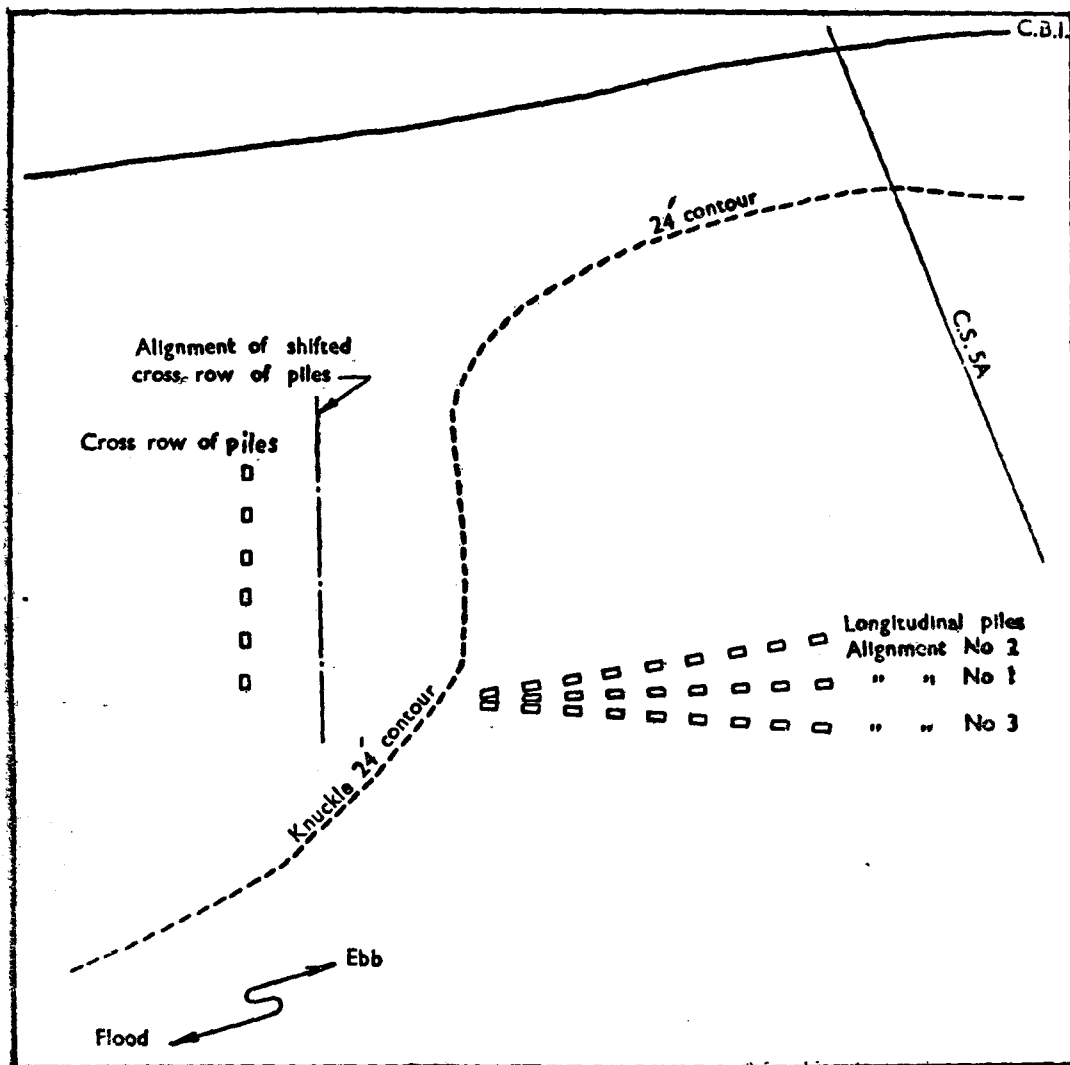


Figure 2 C. 54 : Plan showing the different position of L-piles tried.

Observations

(a) Lines of flow are shown in Figure 2C. 55. As compared with the experiment without any piles, and under similar conditions, the general nature of surface lines of flow did not show any marked change.

(b) Bed contours are shown in figure 2C. 56 (b). When compared with the previous experiment without piles and under similar conditions Figure 2C. 56 (a) it was seen that the 50 feet contour extended towards the right bank showing thereby the tendency to scour on the right side of the knuckle. Scour also occurred downstream of the knuckle at C.S. 5, 4A, etc.

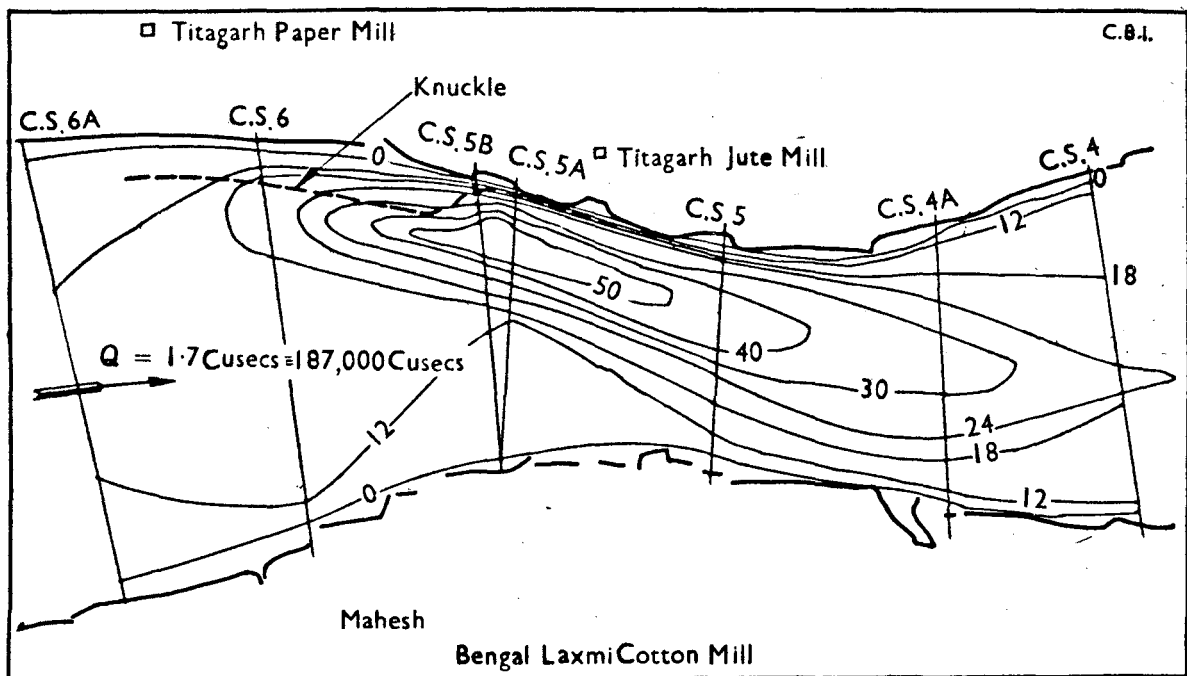


Figure 2C. 56 (a):- Showing $\frac{1}{150} : \frac{1}{80}$ Hoogly River Model Bed Contours after 24 hours of injection at 2 cubic feet per hour (0.4 parts per 1,000)
 $Q = 1.7 \text{ cusecs} = 187,000 \text{ cusecs}$
 Water level at 4A $\equiv 13.9$ feet above K. O. D. S.

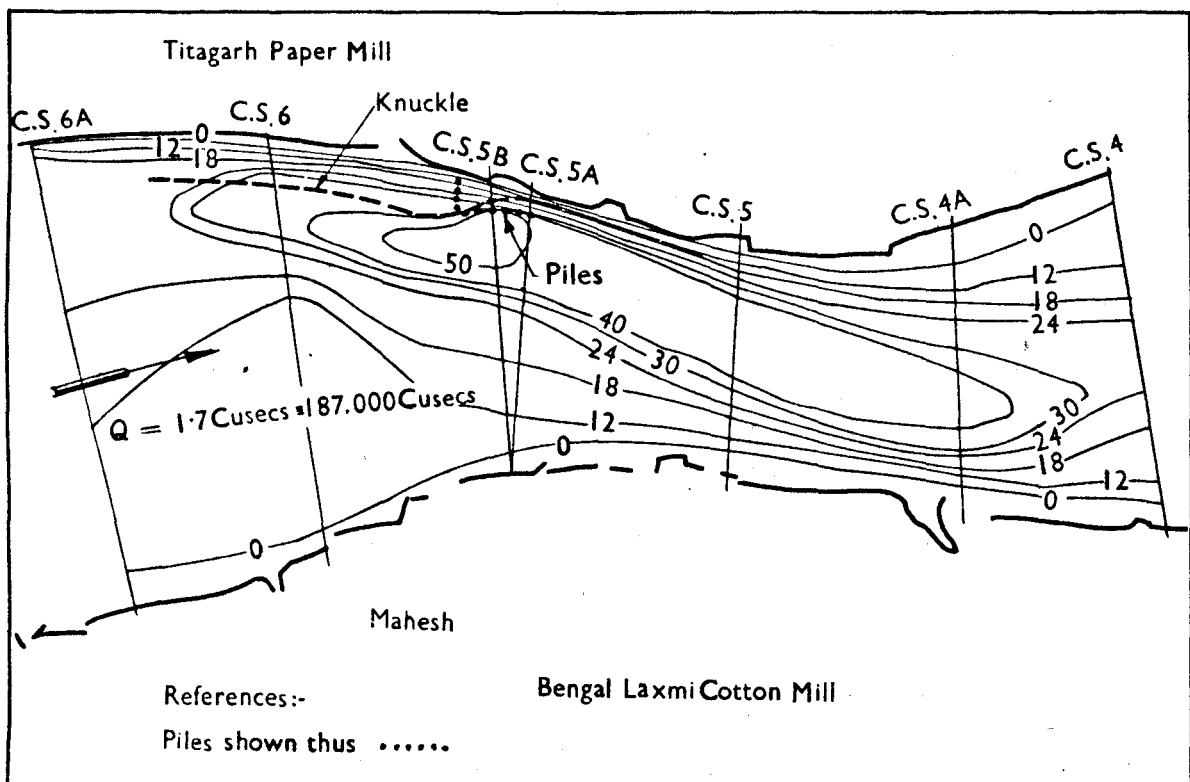


Figure 2C. 56 (b):- Showing Bed Contours after 24 hours with Piles in position I
 $Q = 1.7 \text{ cusecs} = 187,000 \text{ cusecs}$
 Water level at 4A $\equiv 13.9$ feet above K. O. D. S.

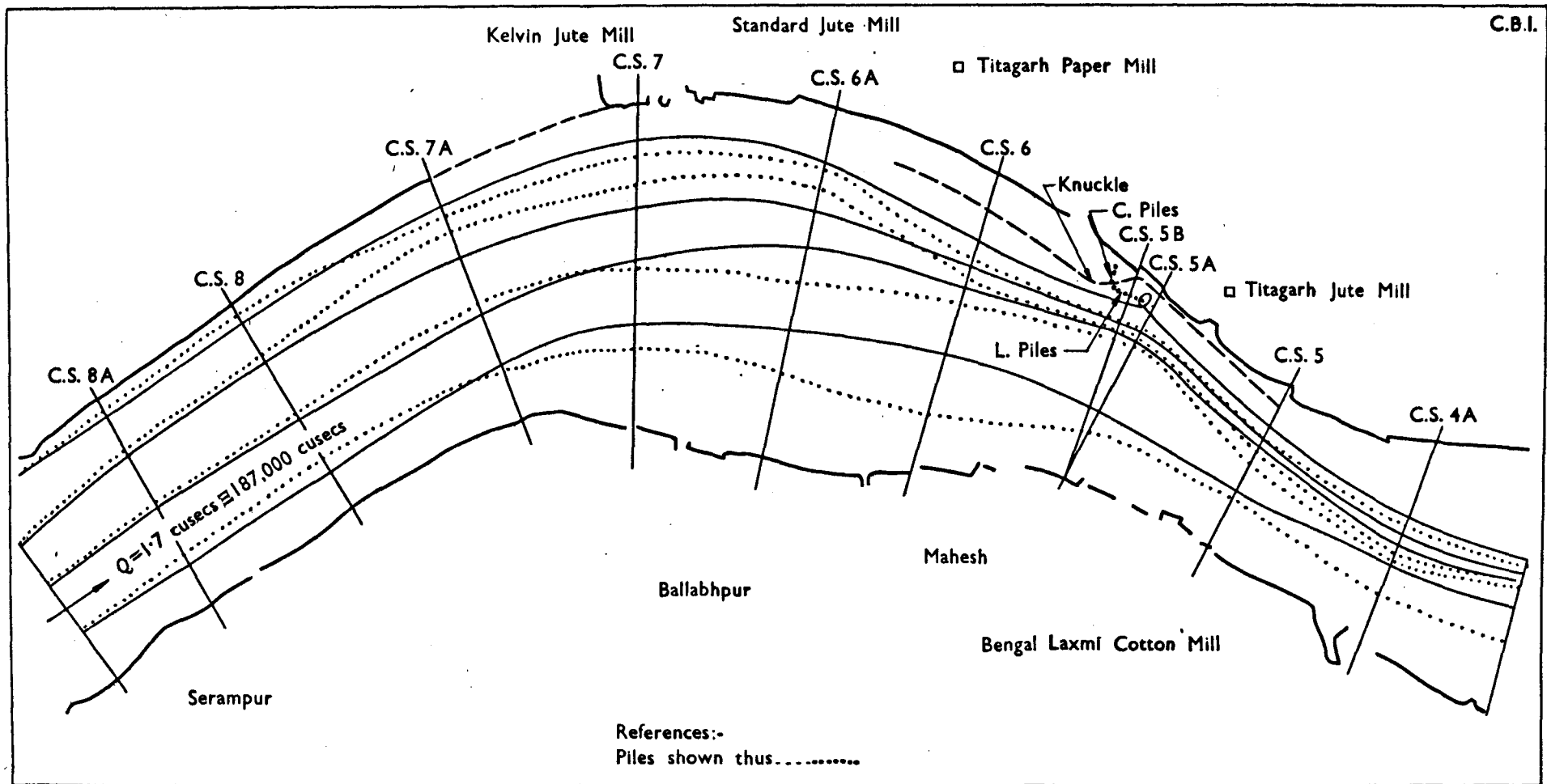


Figure 2 C.57:-Lines of flow with C-Row of Piles shifted downstream and a L-Row of Piles, Position 2 [$\frac{1}{150} : \frac{1}{80}$ Hoogli River model] Water level at C.S. 4 A \approx 13.9 feet above K.O.D.S.

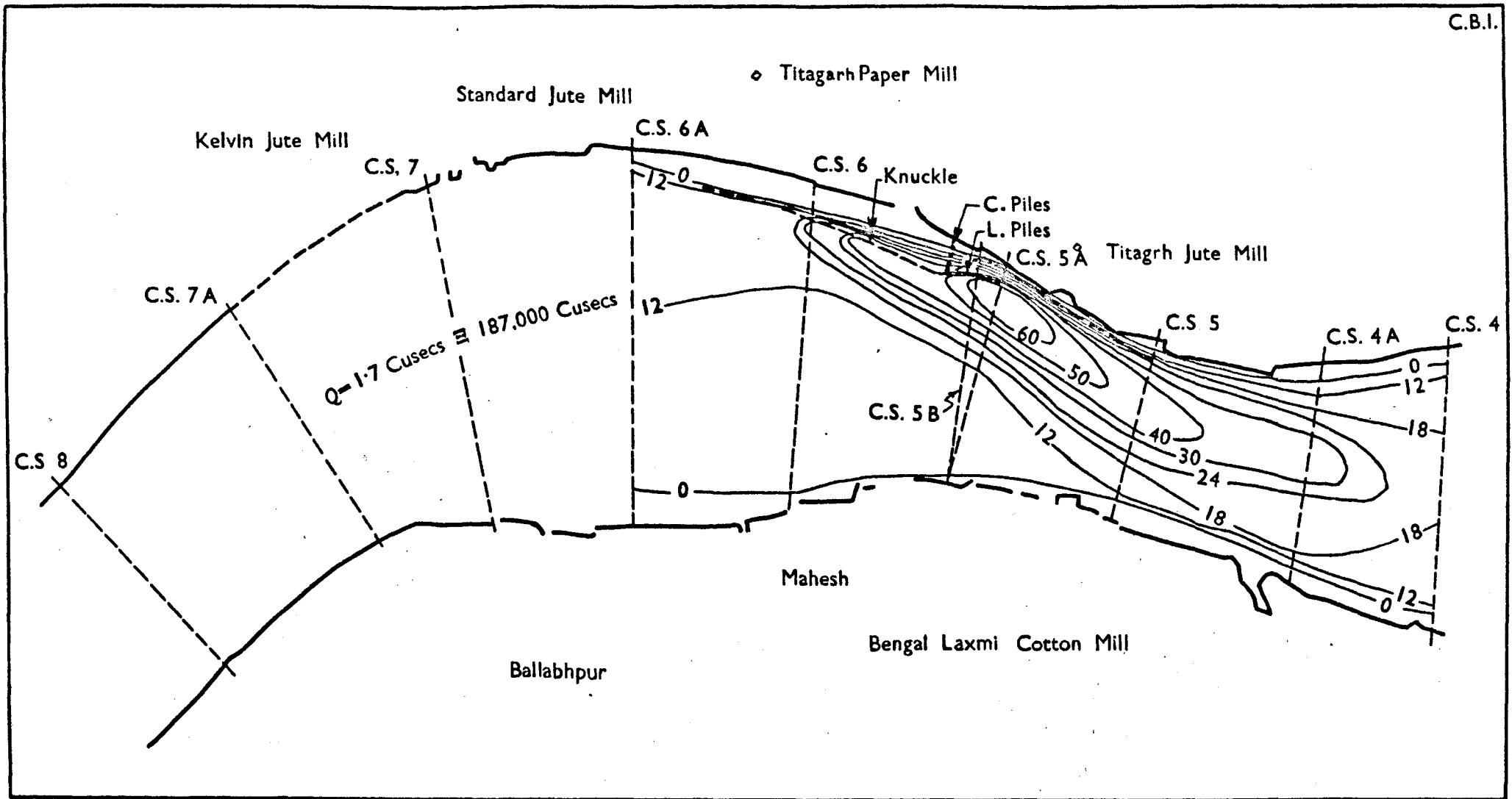


Figure 2C.58:- Showing Bed Contours with C-Row of Piles shifted downstream and a L-Row of Piles Position 2 [$\frac{1}{150} : \frac{1}{80}$ Hoogly River model] Water level at C.S. 4A $\approx 13.9'$ above K.O.D.S. $Q = 1.7 \text{ Cusecs} \approx 187,000 \text{ Cusecs}$

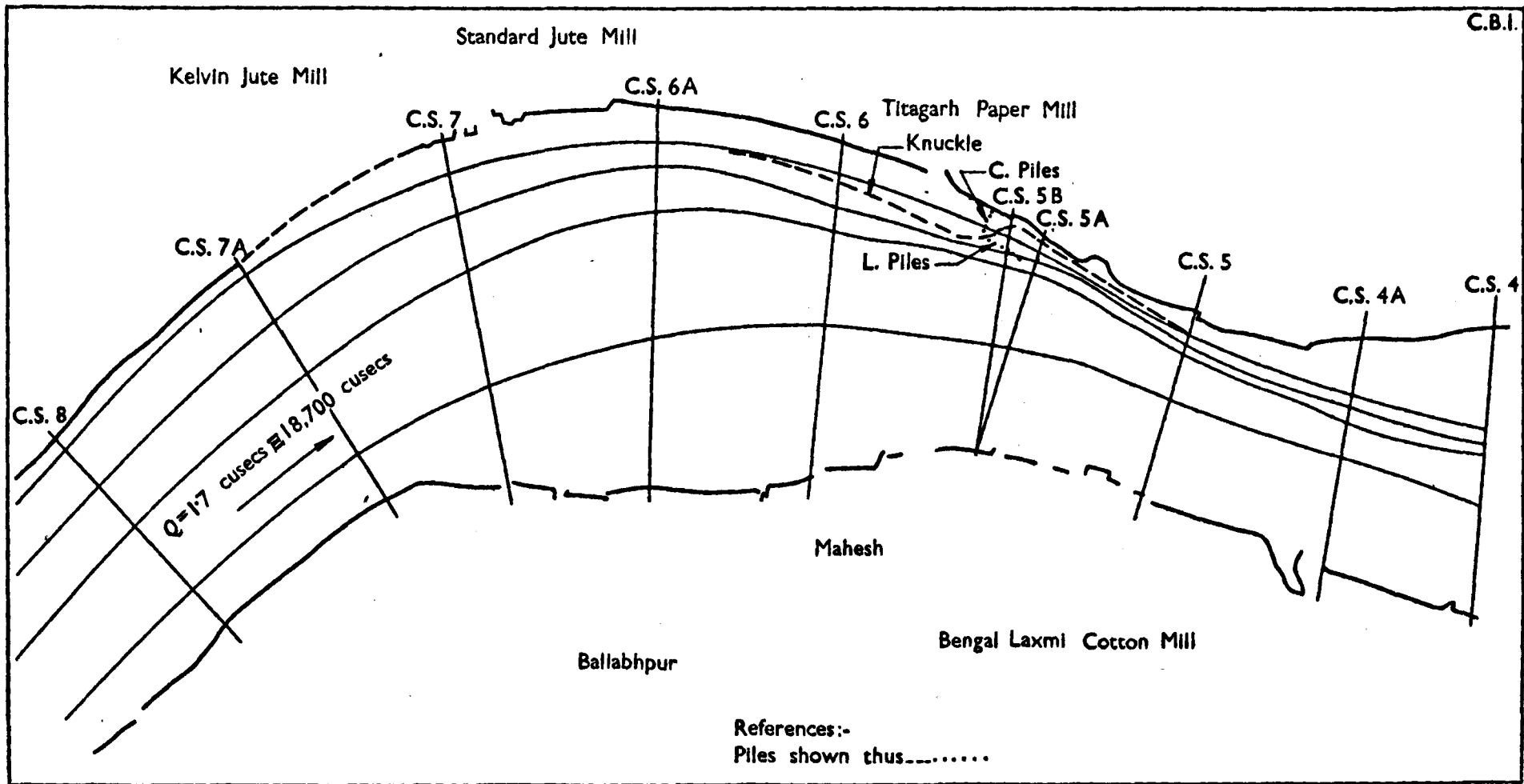


Figure 2 C.59:- Surface lines of flow with the L-Piles as in experiment I A
 Position 3. [$\frac{1}{150} : \frac{1}{80}$ Hoogli River model]

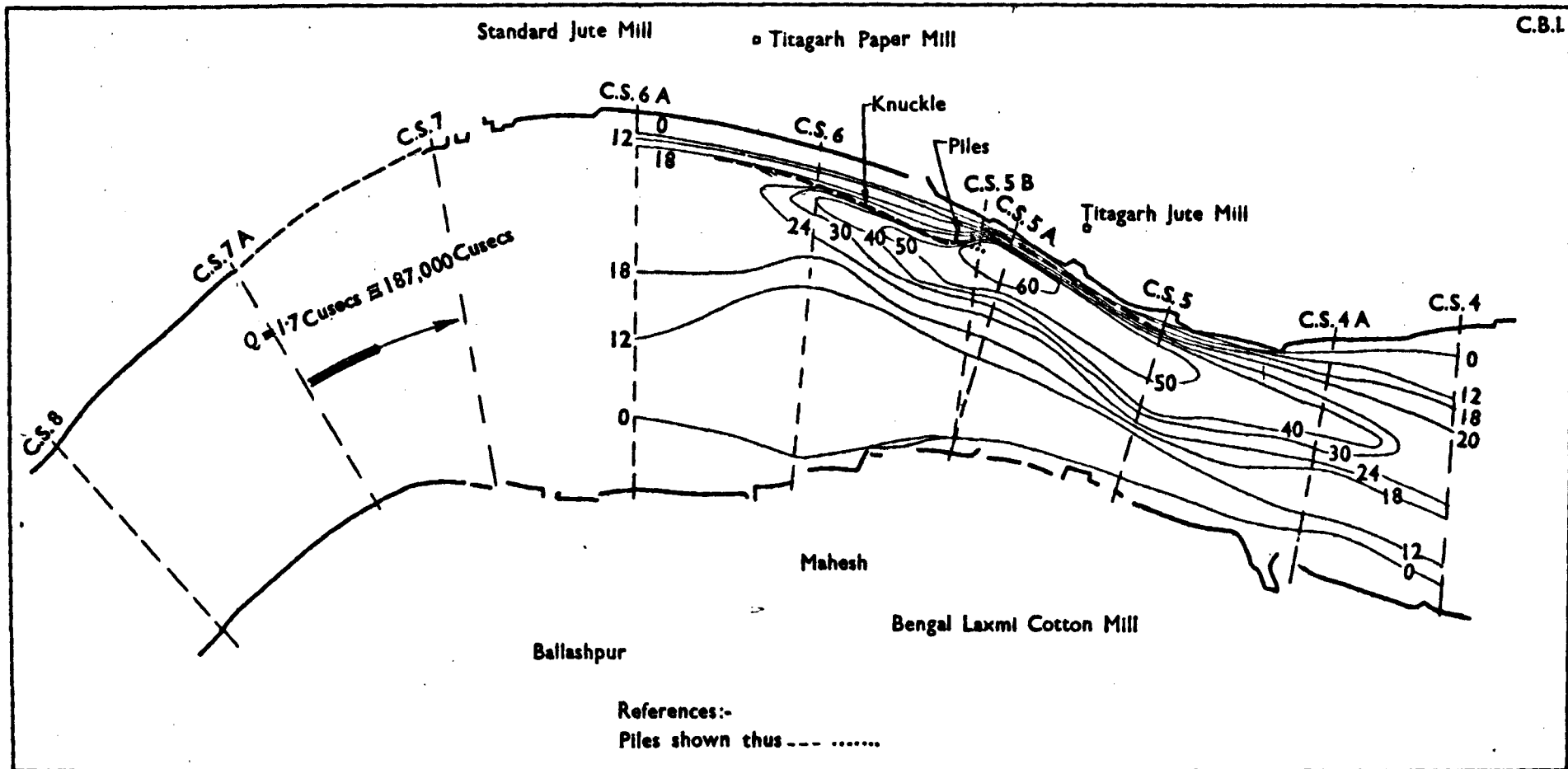


Figure 2 C.60:- Showing bed contours with L-Row of Piles In Position No.3
 [1/150 : 1/80 Scale Hoogli River model]

(c) Soundings were taken at C.S. 5B to see the amount of silting downstream of the knuckle. There was silting and scour within the bight ; but silting was not uniform. The depth of silting also was small, only of the order of nine inches or so in prototype .

It was, therefore, decided to shift the cross piles downstream, where the depth is greater—thereby increasing their height and making them more effective, while keeping the top at the same level as before.

Experiment No. 1A.

Cross piles were shifted \cong 30 feet downstream keeping the L. piles in the same position. The bed was adjusted to the original conditions and the experiment was run for 24 hours with injection of sand as before.

In this experiment, soundings at C.S. 5 B were only taken. These showed uniform silting in the bight, though not sufficiently deep.

Experiment No. 2.

In this experiment, the cross-row of piles was kept as in experiment 1A and the alignment of L. piles was changed to position No. 2 as shown in Figure 2C. 54.

Observations :—

(a) The surface lines of flow downstream of the knuckle shifted away from the knuckle, Figure 2C. 57.

(b) Bed contours show that there was scour between C.S. 5A and 5 towards the left Bank : 60 feet contour which was not obtained in the previous experiment was obtained in this experiment, Figure 2C. 58.

(c) Soundings at cross section 5B show that there was no tendency to silt. Slight scour was, however, observed.

Experiment No. 3.

In this experiment, the cross-row of piles was as in experiment 1A and L. row of piles as is shown by position No. 3, Figure 2C. 54.

Observations :—

(a) Lines of flow showed a concentration of flow at C.S. 5, which might result in a formation of knuckle at that place, Figure 2C. 59.

(b) Bed contours showed that there was scour within the bight and outside the L. pile — to the right of L. piles. 60 feet contour was obtained in the bight Figure 2C. 60.

Sounding at C.S. 5 B confirmed the scour in the bight.

Experiment No. 2A.

As the scour was observed outside the L. piles, an additional row of four piles, with the tops eight feet below the tops of L. piles, was constructed. Figure 2C. 61 shows the effect of such piles on the detailed lines of flow. The bed flow outside the L. piles was deflected away and, consequently, would tend to protect these piles.

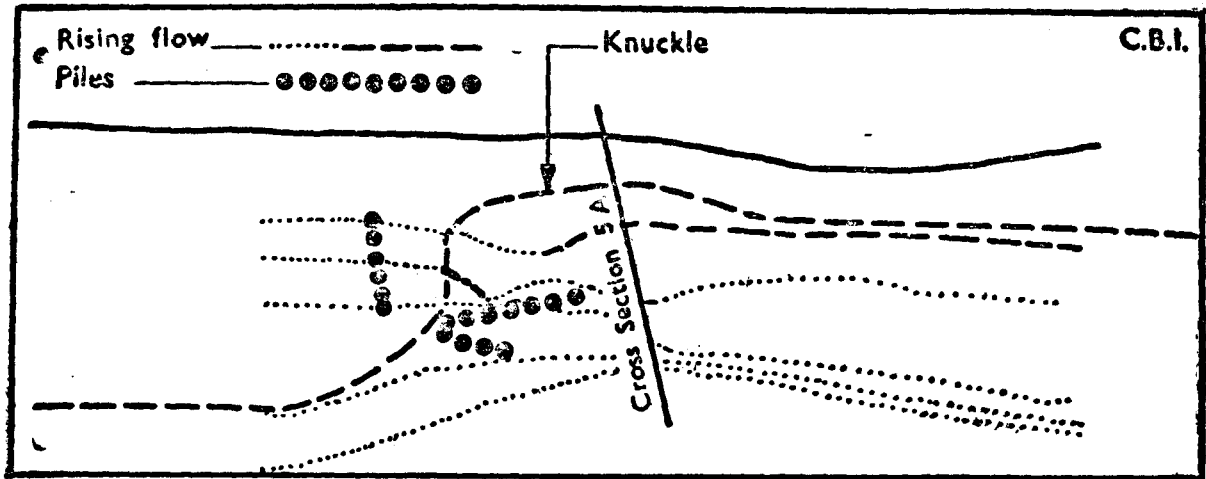


Figure 2C. 61 : Showing bed lines of flow.

Experiment No. 4.

In this experiment, one row of L. piles was constructed as in position 1 and two rows of cross piles were constructed as against one in the previous experiment. This was done to see if the silting tendency could be accelerated.

The sounding at C.S. 5 B show that silting was not satisfactory.

Conclusion :—

Position No. 1 of L. piles was the best of the three positions tried ; but depth of silting obtained in the bight was equivalent to only one to 1.5 feet in the river with a continuous ebb tide running for 15 days, which was not satisfactory or sufficient.

The reason why piles did not prove satisfactory in the case of the knuckle at Titaghar Jute Mill appears to be due to the "change over" of the flow occurring immediately upstream of the knuckle, leading to more turbulence downstream, whereas in the case of the knuckle at the Dunbar Cotton Mill, the change over takes place long before the flow approaches the knuckle ; and the difference in velocities along the left bank upstream and downstream of the knuckle is less at the Dunbar Mill than at the Titaghar Mill. Moreover, while at the Dunbar Mills the trouble is only due to too sharp a curve for the discharges, at Titaghar, the trouble is also due to throttling and distortion immediately downstream. It was considered that "attracting" islands might be effective in causing the "change over" of flow further upstream of the Titaghar Jute Mill knuckle.

PART II—EXPERIMENTS WITH ISLANDS

Introduction

Experiments with islands described in the note were done in 1/300 : 1/60 model of the Hooghly so as to get quick and comparative results with various positions of islands under tidal flow conditions, the time required for completing one tide being much less in this model (*viz.* 19 minutes \equiv 12 hours 8 minutes for one tide) than in the 1/150 : 1/80 model (*viz.* 43 minutes). In all experiments, the size of the island was kept the same, *viz.* length=300 feet with

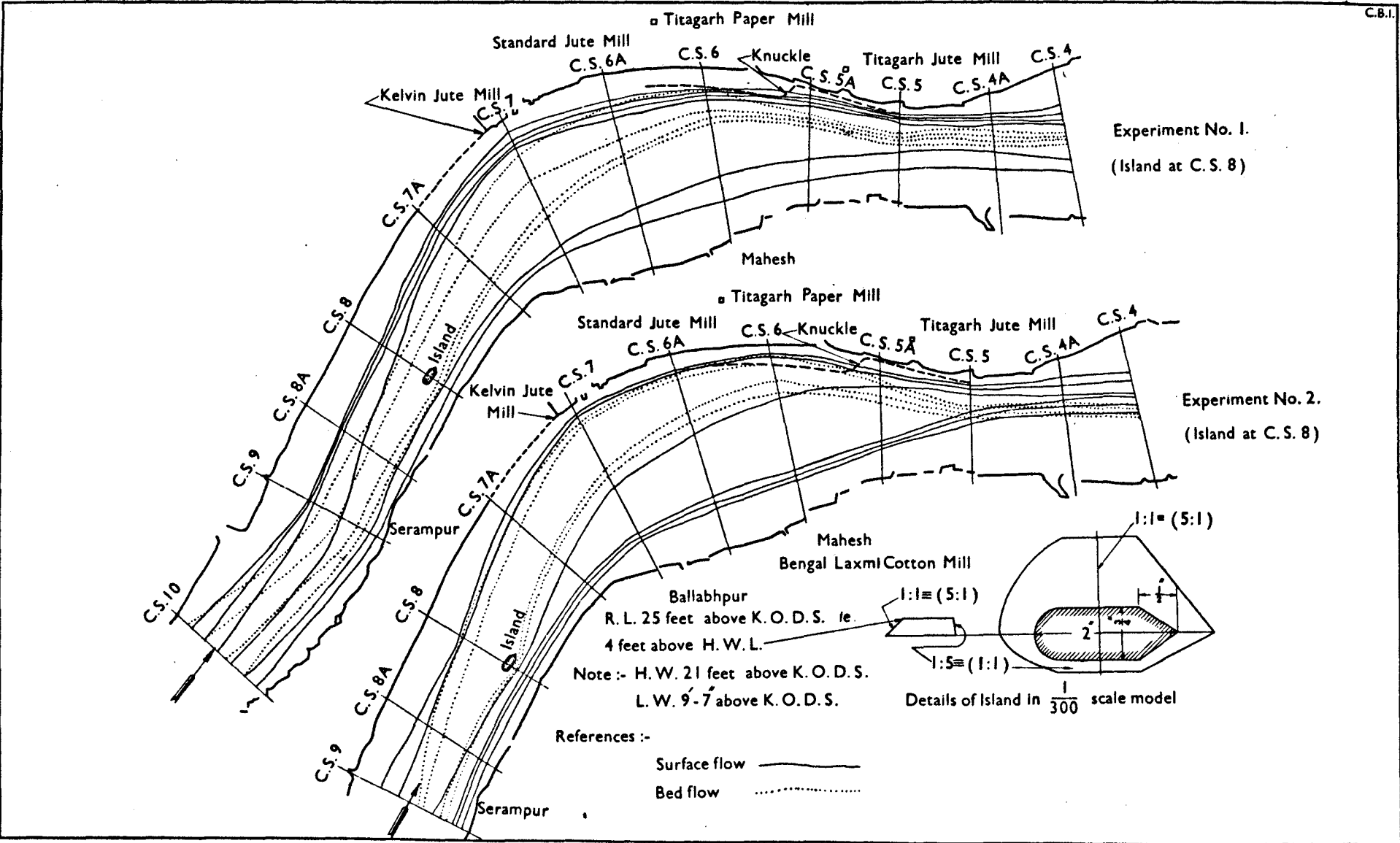


Figure 2C.62(Top)
 Figure 2C.63(Bottom) *Showing lines of flow with Island [$\frac{1}{300} : \frac{1}{60}$ Hoogly river model]*

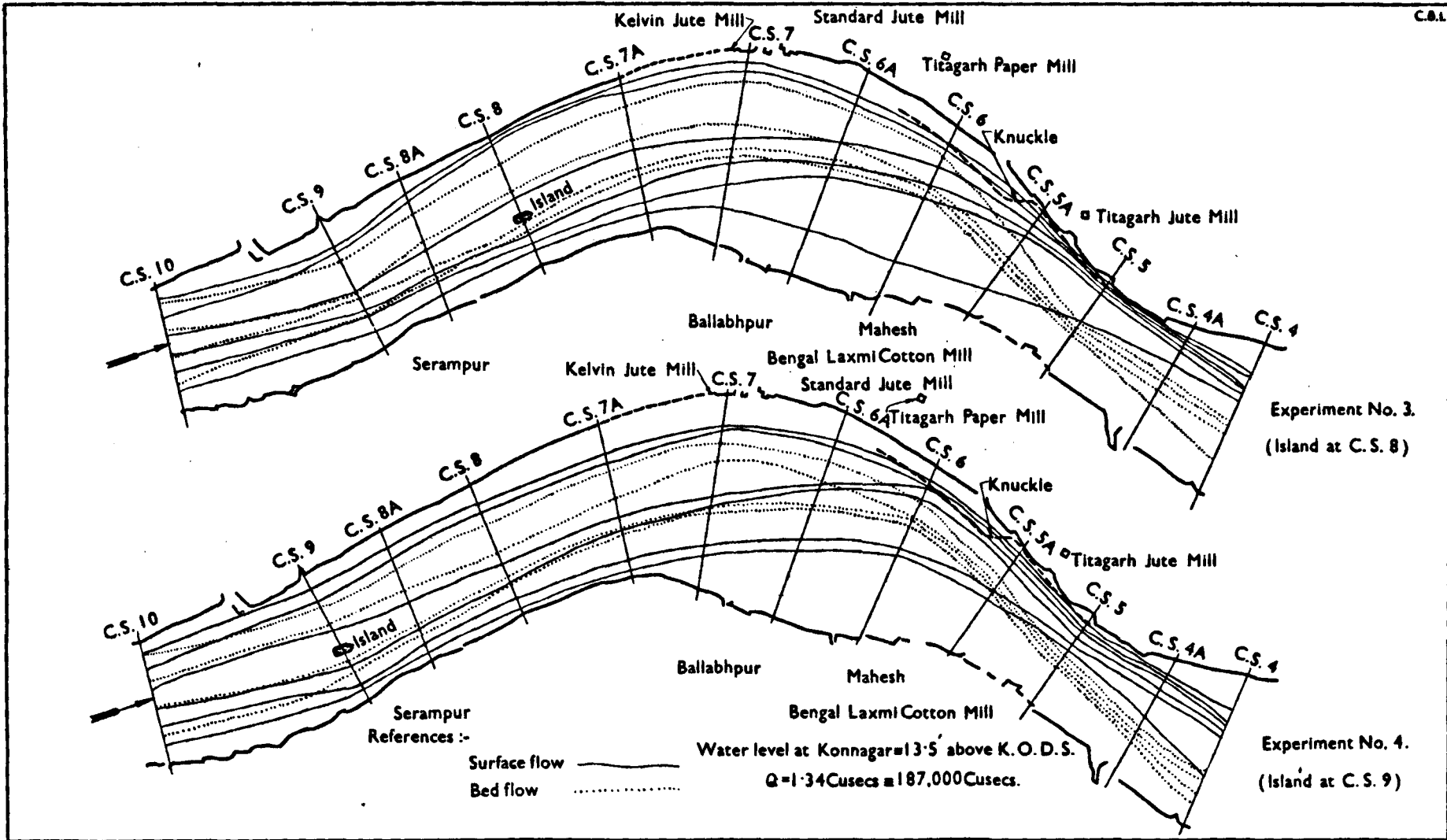


Figure 2C.64(Top) Figure 2C.65(Bottom) Showing lines of flow with Island (Ebb flow) [$\frac{1}{300} : \frac{1}{60}$ Hoogly river model]

top 25 feet above K.O.D.S., *i.e.* four feet above high water level width \cong 112.5 feet (semi-circular cut-water and 45° ease-water are included in the above length). This size can be modified later.

It is believed this is the first time "attracting" islands are ever being considered for river training of tidal and navigational rivers. The islands would be located on existing shoals, far from the reach of the navigational channels; so that there can be no warrantable objection of obstruction to navigation, while "attracting" islands have been decried as being unsuitable for training flashy rivers such as the Luni near Samdari (Jodhpur Railway) (49) where the high formative discharges occur for too short a duration to allow the island to leave its impress on river configuration, it is considered that islands of suitable design, thoughtfully located and aligned in relation to the river meanders and the flow lines, have great potential promise on tidal rivers, where the high and prolonged freshet flow gives them ample scope as in the case of non-tidal rivers with a long duration of flood flow. The islands can be conveniently constructed out of dredged cuts, the cuts themselves located and designed in the right places and of the right size, so that they would help, *not* hinder, the action of the islands in regulating and maintaining the deep water channels. The action of "attracting" islands in tidal rivers is similar to that already explained for non-tidal rivers except that their design and location must take into account the two-way flow, where both are important in their effect on the river course.

Experiments

The model was laid to the 1939-40 survey, V-sand ($m=0.20$ mm.) was used as bed material. Freshet spring tide observed in the river on September 23, 1941 was re-produced and repeated ten times in each experiment.

Lines of flow were invariably observed under 3/4 ebb flow conditions, after ten tides were completed in each experiment.

Water level recorders were fixed at Damurdaha (upstream end of model) at Cossipore (downstream end of model) and at Konnagar near the knuckle of the Titaghar Jute Mills.

Experiment No. 1.—

The island was constructed at C.S. 8 as shown in Figure 2C. 62. Out of the six surface lines, two lines passed over the shoal near the right bank upstream of the island.

Experiment No. 2.—

The downstream end of the island was pulled towards the right bank so that the island faced the high velocity flow more obliquely than in the previous case. Three surface lines passed over the shoal on the right bank, thus indicating a definite improvement. Bed lines were also deflected towards the knuckle Figure 2C. 63.

Experiment No. 3.—

As the island in experiment No. 2 was constructed in a deep channel where the depths were 16 to 20 feet below local zero (*i.e.* 20 to 24 feet below lowest low

(49) Indian waterways Experiment Station, Poona, Annual Report (Technical) 1946, para II vi. 1. (see page 494 of this publication).

water level), it was considered desirable to shift the island towards the left bank where the depths were considerably less. The results were not satisfactory, as the island did not act, Figure 2C. 64.

Experiment No. 4.—

In this experiment, the island was shifted upstream so as to be at a distance of M1/4 from the knuckle. The island was not, however, effective in attracting the flow towards the right bank opposite the knuckle Figure 2C. 65. Lines of flow were observed in this experiment, also under flood condition, Figure 2C. 66. The island was then removed and the lines of flow again observed under flood condition, Figure 2C. 67. This experiment clearly showed that any island so far upstream of the knuckle would have too little effect on flow conditions near the knuckle.

Experiment No. 5.—

In this experiment, two islands were tried : (1) Island A as in experiment 2 and (2) Island B (between C.S. 7 and 6A). The high velocity flow near right bank at C.S. 8 flowed along the right side of the island B ; but further downstream , it swung back towards the knuckle, Figure 2C. 68.

Experiment No. 6.—

In this experiment, the island B was constructed more towards the right bank than in experiment 5. Results were fairly satisfactory, as scour occurred on the shoal near right bank between C.S. 7 and 5 A, Figure 2 C. 69.

Experiment No. 7.—

In this experiment, a third island called 'C' was constructed as shown in Figure 2C. 70, the object being to get advantage of scouring action during flood tide. So the semi-circular end of the island was made to face the flood flow.

Figure 2C. 70 shows the lines of flow under flood condition. All the flow was hugging the left bank and there was no indication of scour on the shoal by the addition of this island ; and so the third island was ineffective.

Experiment No. 8.—

It was considered that islands A and B were too far apart ; so a third island 'C' was constructed in between, Figure 2C. 71.

The islands were so constructed so as to form a continuous line. This, it was considered, would result in a formation of continuous deep channel, attracting part of the flow on right bank opposite the knuckle and reducing concentration of flow on the left bank.

On the right side of island A, where already a deep channel existed and with a steeper side slope given to the island, deep scour took place. The flow being the last of ebb which (without the islands C and B) had a tendency to move towards the knuckle from island A, hit the island C on its left face and in spite of the flatter slope given to it to reduce scour, deep scour was obtained on this side (left side).

Between islands C and B, similar action took place. The effect, in short, was deeper scour on the left sides of islands C and B and hence the high velocity flow was still away from the right bank—opposite to what was required.

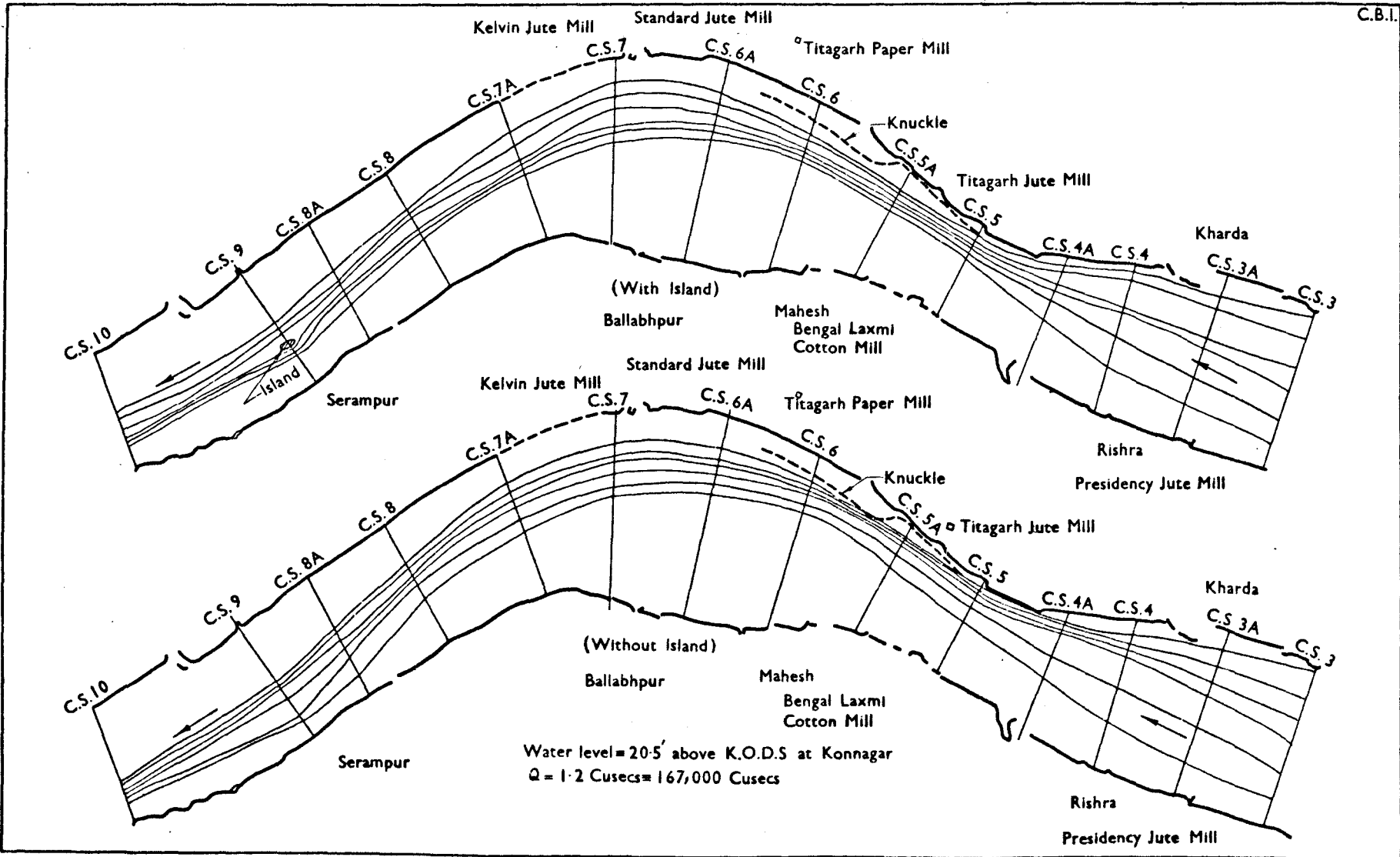


Figure 2C.66 (Top) Showing surface lines of flow under flood condition $[\frac{1}{300} : \frac{1}{60}]$ Hoogly river model expt. no 4]
 Figure 2C.67 (Bottom)

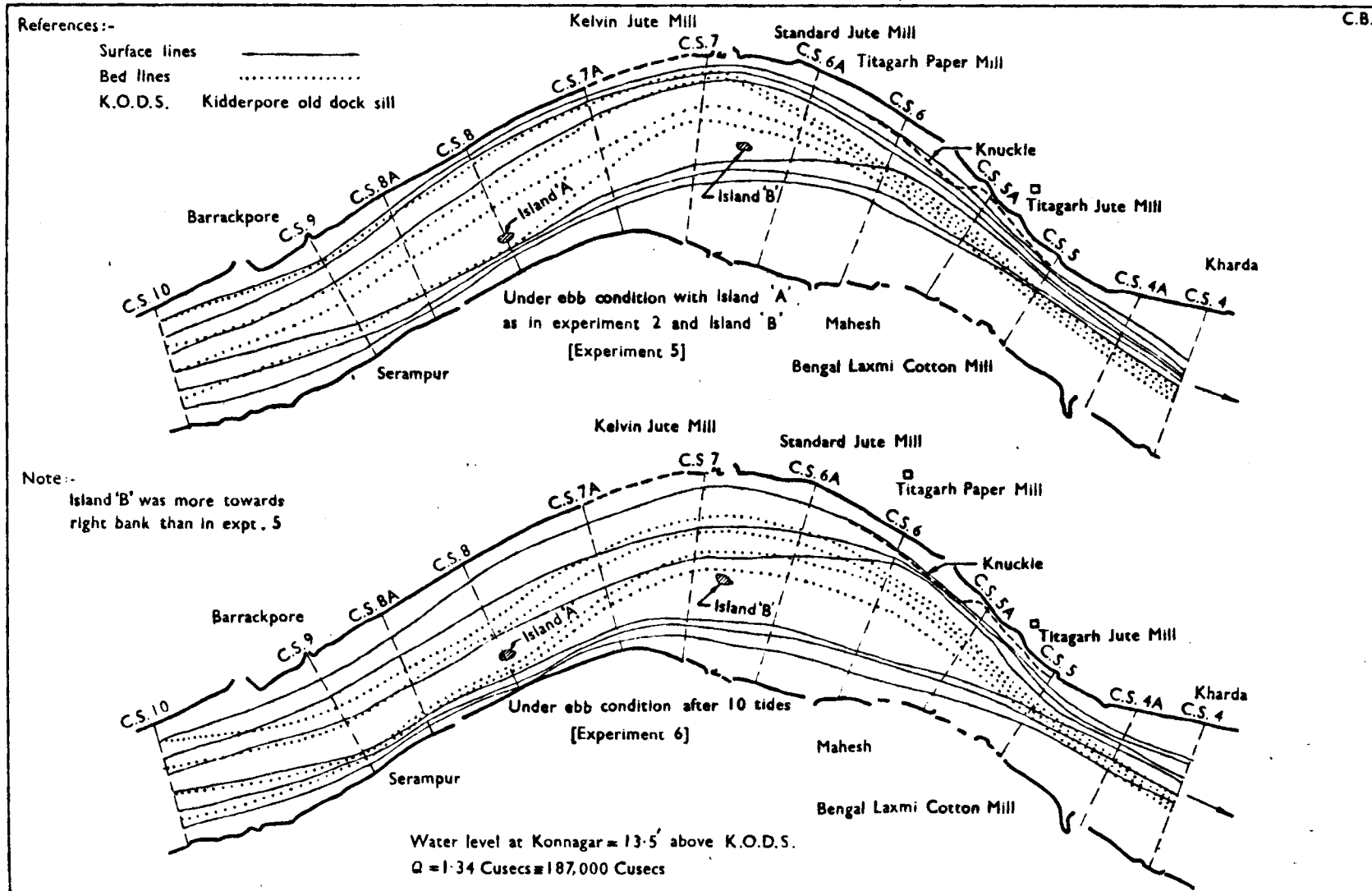


Figure 2C.68 (Top) Showing lines of flow with Islands 'A' and 'B' [$\frac{1}{300} : \frac{1}{60}$ Hoogly river model]
 Figure 2C.69 (Bottom)

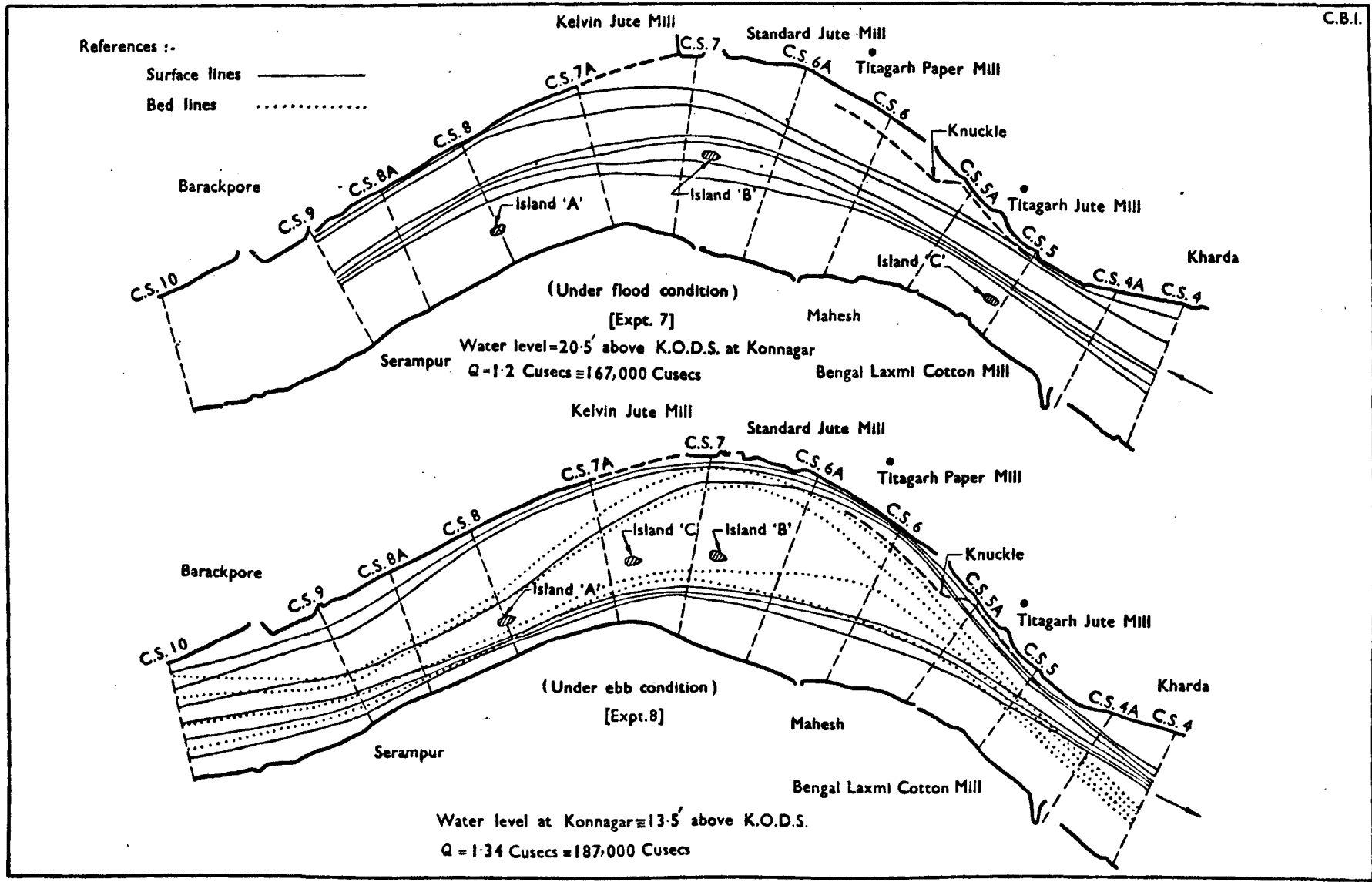


Figure 2C.70 (Top)
 Figure 2C.71 (Bottom) *Showing lines of flow with Islands A, B & C [1/300 : 1/60 Hoogly river model]*

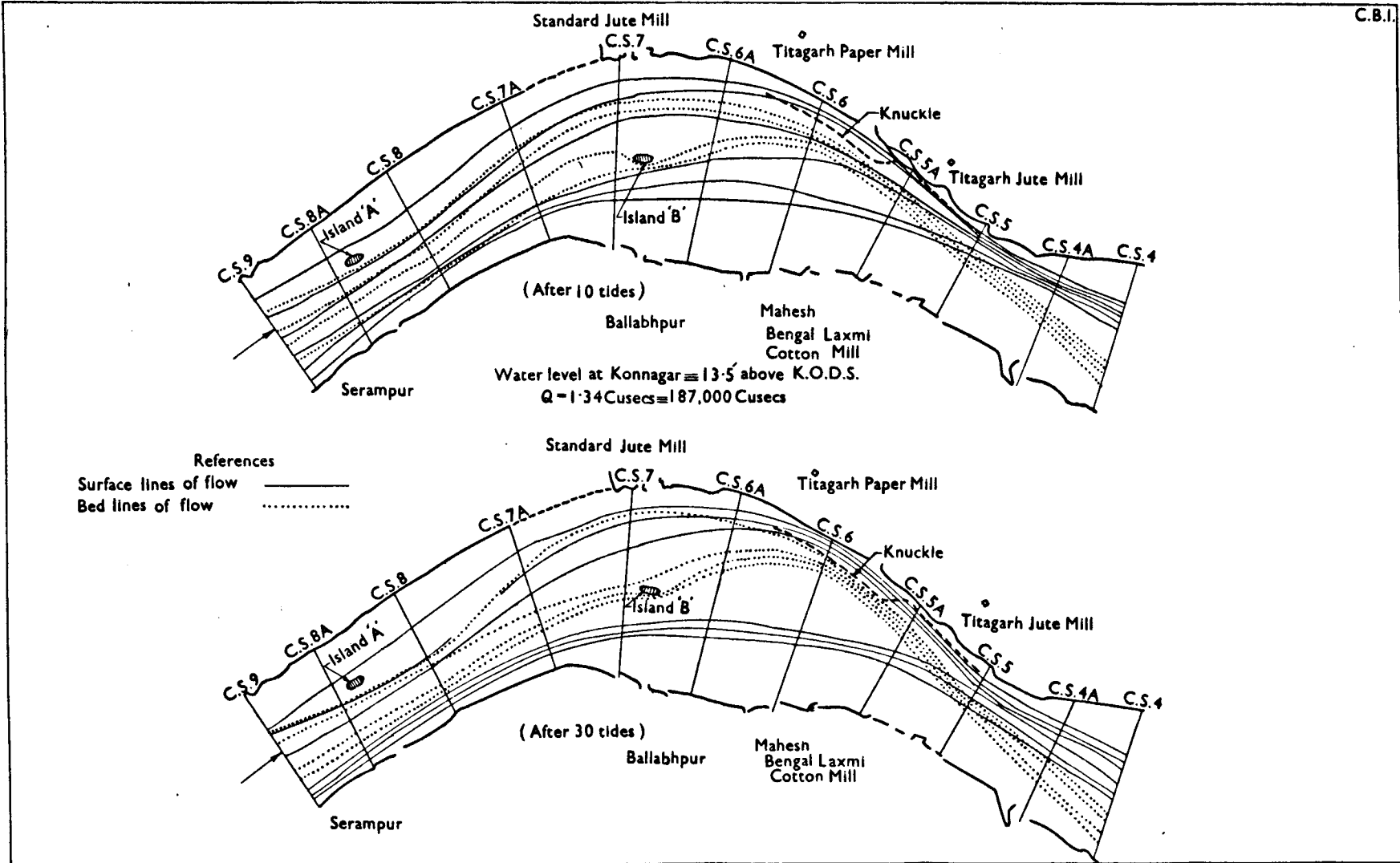


Figure 2C.72(Top) Showing lines of flow under ebb condition with Islands A & B [1 : 1 Hoogly river model]
 Figure 2C.73(Bottom)

Experiment No. 9.—

The position of the island A in experiments No. 1 and 2 was such that the depth at that place was 30 to 35 feet below lowest water level. The construction of such an island would be difficult and so the island A in experiment 9 was constructed on the shoal and the right side of the island was dredged to the practical possible depth (*i.e.* 20 below K. O. D. S. or 30 feet below lowest water level). Dredging was also done from C.S. 8A to 8—a distance of about 1/3 mile.

Figure 2C. 72 shows the lines of flow after ten tides. Island A was not in action. Island B was in good action and showed appreciable movement on the shoal on the right bank. The ripples were then removed and ten more tides were run. The movement was again observed on the shoal. The ripples were again removed and the procedure was repeated for the third time. This experiment showed the possibility of the development of a channel on the existing shoal.

Figure 2C. 73 shows the lines of flow after 30 tides. The surface flow was drawn towards the right bank.

Results of experiment 9 appear to give most promising results so far except that the island A to be really effective should be moved towards the right bank; and to keep down the cost of construction, it may be a submerged island with its top ten feet below low water level so as not to interfere with shipping, provided submerged islands are found equally effective in tests proposed to be made.

The same experiment will have to be carried out in the large, part model (1/50 : 1/80); island A, cannot, however, be constructed due to insufficient upstream length in this model; so lines of flow and velocity distribution, say at C. S. 7A obtained in the 1/300 model will be simulated in the 1/150 model and it will be seen by injecting fine material whether silting will occur in the bight of the knuckle.

Further positions and behaviour of islands are also under investigation.

(3) PROTECTING THE FORESHORE OF THE ANGLO INDIA JUTE MILL ON THE HOOGHLY RIVER (BENGAL) ⁽⁵⁰⁾.

INTRODUCTION

Figure 2C. 74 shows the plan of the Hooghly from six miles upstream of Bansbaria to Konnagar. This figure clearly shows that upstream of the Victoria; Jubilee Bridge the river is fairly straight; whereas downstream of the bridge the river is meandering into sharp curves. This has been found to be mainly due to human interference with the natural river movements. There are protruding jetties at numerous places belonging to the various factories on both banks of the river and several brick kilns erected at many places; the general tendency being to encroach upon the river as much as possible to have maximum facilities of water frontage *etc.* All these rigid encroachments act as groynes and pitched banks, which attract the flow and upset the flow conditions; in places such as just below the Titaghar Mills, the distortion caused by excessive contraction is substantial.

(50) Indian waterways Experiment Station, Poona, Annual Report (Technical) 1946 pages 42 to 44.

The river is practically held on both banks—especially on the outside of curves—where stone revetments have been constructed for bank protection. In may 1945 the following observations regarding erosion of bank on outside of curves were made.

Banks where erosion was taking place are shown in hatched lines in Figure 2C. 74.

- (1) Along the left bank upstream and downstream of Punhatti.
- (2) Left bank near the Titaghar Jute Mill. The downstream end of the drain upstrain of the second jetty had collapsed. "The Sea Wall" of the Mill was badly undercut in several places.
- (3) Very severe erosion was taking place along the right bank at Ballampur and Chatra, that at Chatra being the most severe, where in many cases the "Sea Wall", walls of buildings and *ghats* were found severely damaged.
- (4) Severe erosion along the left bank between Palta Waterworks and Nawabganj.
- (5) Erosion near Dunbar Cotton Mills.
- (6) Erosion along the right bank between Gondalpara and Chandranagore. New embayment had formed along the right bank opposite Anglo Indian Jute Mills.
- (7) Erosion along the right bank upstream and downstream of the Victoria Jubilee Bridge.

Three cases for protection against severe threats were referred by the following authorities :—

- (1) Anglo India Jute Mills,
- (2) Dunbar Cotton Mills,
- and
- (3) Titaghar Jute Mills.

Of these, experiments for the Dunbar Cotton Mill have been completed ⁽⁵¹⁾. Those in connection with the Titaghar Jute Mill are in progress. On inspection at the site showed that the trouble in the case of the Anglo India Jute Mills was purely of a local nature and that the river conditions upstream and downstream were not at all severe and could not have led to the collapse of the foreshore by 12 feet on February 21, 1942. The collapse of the bank on March 3 and 4, 1944 was very severe, being more than that which occurred on February, 21, 1942. Both these occurred about the first of neap tides and the previous springs had been "Apogee" springs and consequently had small range without any bore or even strong rush of tide. It was suggested that the trouble might have been due to the high subsoil water level. Subsequently, pits were taken at various places in and around the mill area and water levels were observed at frequent intervals. The data of water levels in the pits were analysed and it was found that the subsoil water level was 26 feet above the local zero, the local zero itself being 4.5 feet above K. O. D. S. (Kiddespoore Old Dock Sill). The least water level determined from simultaneous tidal curves observed on March, 25, 1940

⁽⁵¹⁾ Indian Waterways Experiment Station, Poona, Annual Report (Technical) 1944. page 7.

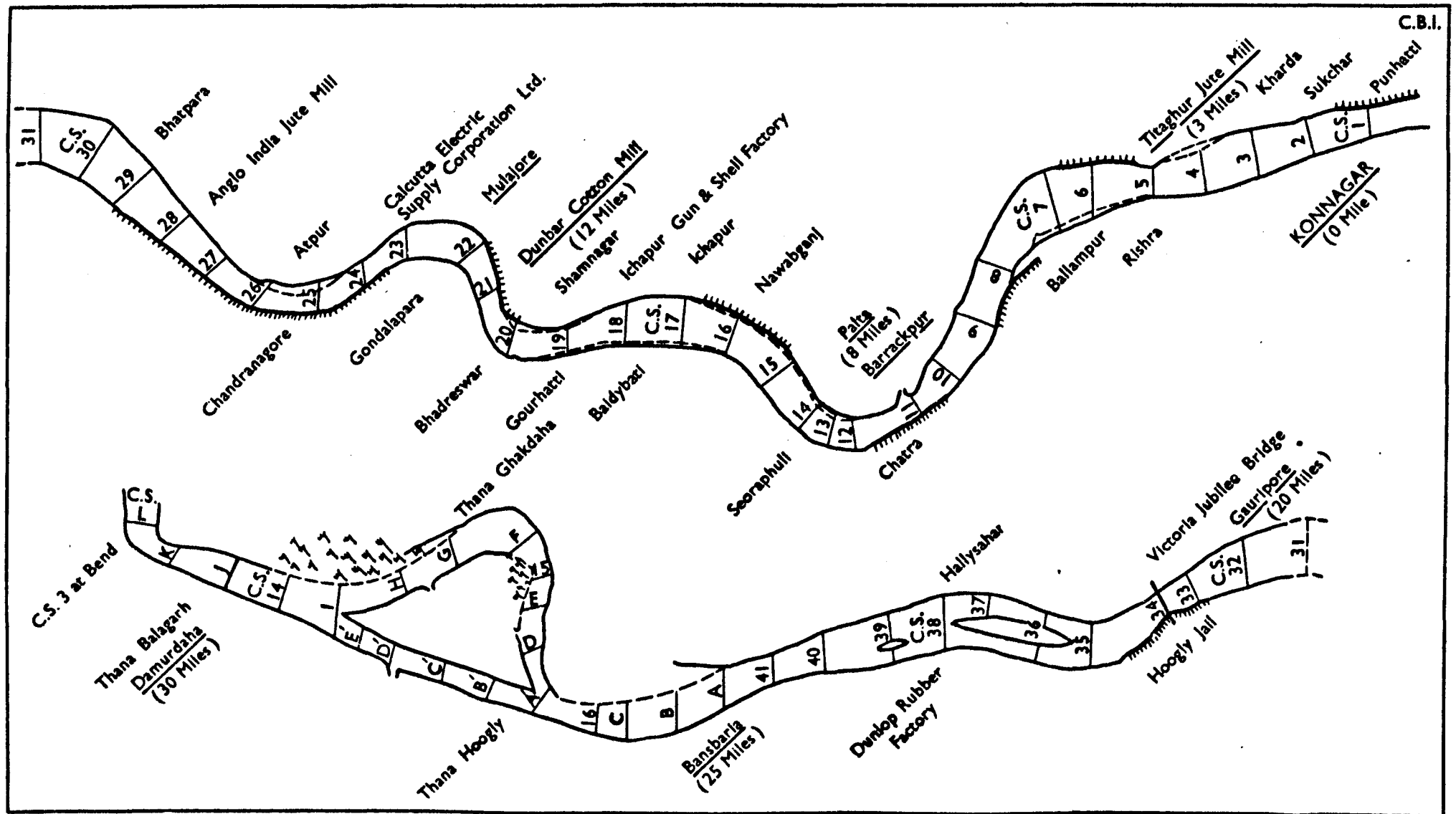


Figure 2C.74:— Showing Survey of Hoogly River from Konnagar to 6 miles upstream of Bansbaria (1940)



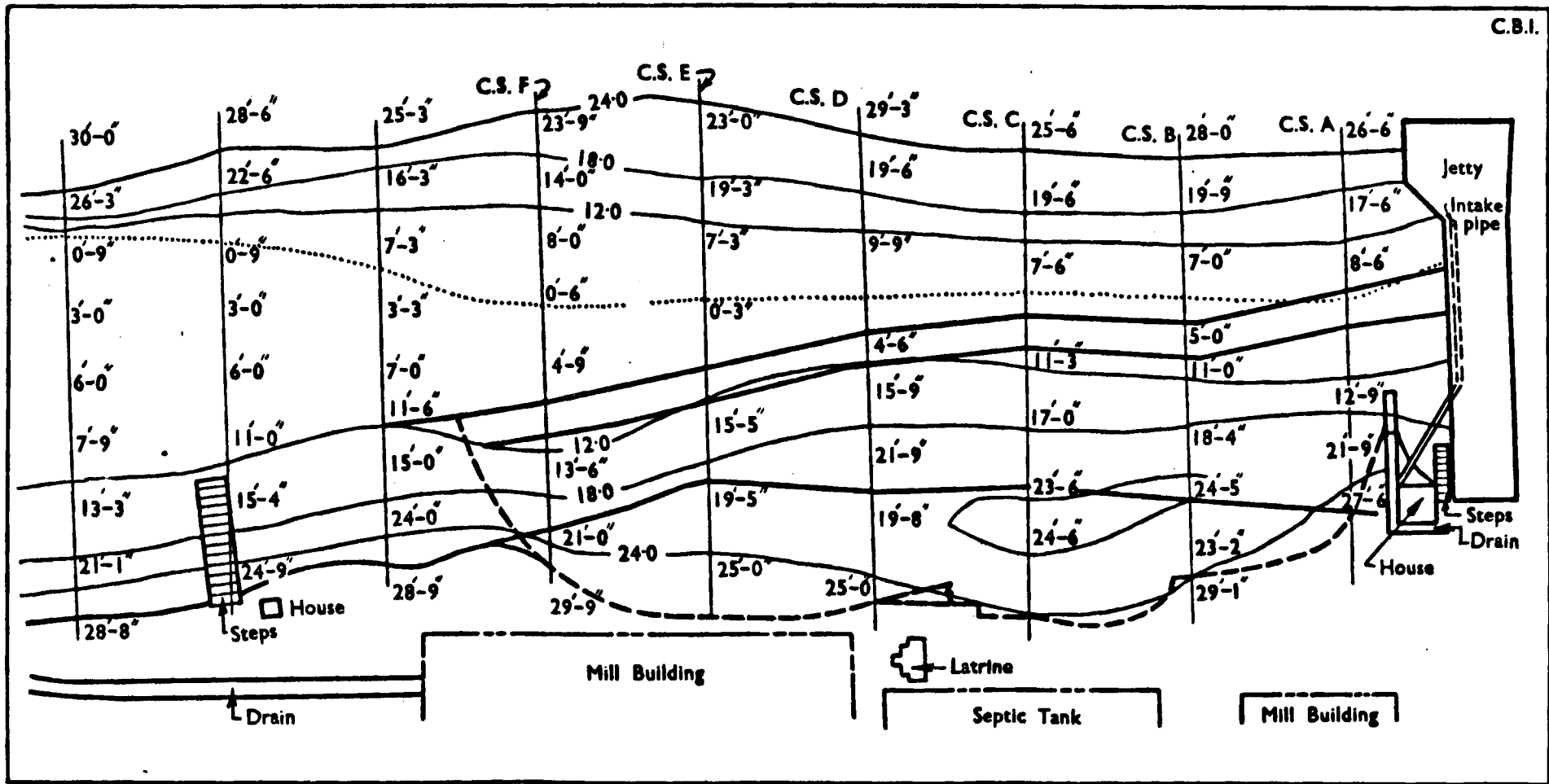


Figure 2C75:- Foreshore of Anglo India Mill No.3 showing Collapse of the Bund in thick dash line and the proposed Stone Pitched Bank in thick line Surveyed by G.F.Willcox on 14.3.45

Scale 40 20 0 40 80 120 160 200 Feet

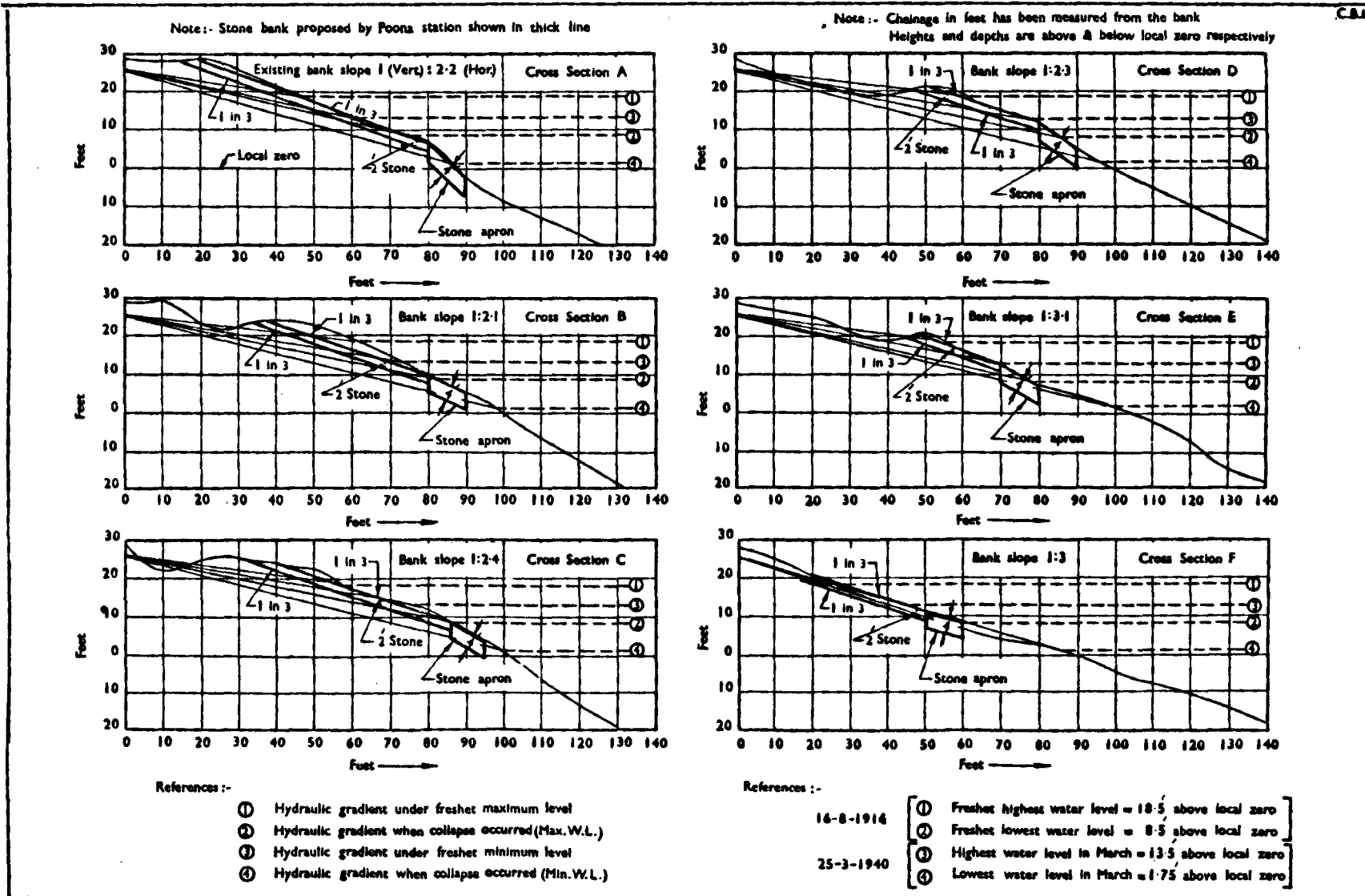


Figure 2C.76:- Showing Foreshore sections & Hydraulic Gradients at various stages [Anglo India Mill No.3]

was 1.75 feet above local zero. This gave the maximum head due to subsoil water equal to 24.25 feet and a saturation gradient of about 1:4. During the inspection, it was found that the floor of one of the mill buildings which is quite near the river bank had subsided in many places. It was also noted that the foreshore was made up of ashes, clinkers, *etc.* These also contributed to the collapse of the foreshore as particles of these materials offered very little frictional resistance and were thus unstable. Figure 2C. 75 shows the foreshore of Anglo India Jute Mill and the line of collapse. Figure 2C. 76 shows cross sections.

RECOMMENDATIONS

The existing material, namely, ashes, clinkers, *etc.*, should be removed as far as possible to a depth of two to three feet and new banks in sand sloping in 1 in 3 should be made as shown in Figure 2C. 76. This should be covered with two feet thick "one maund stone" (weighing 70 to 90 lbs.) This should be followed by sloping stone apron four feet thick near the toe, which will protect the lower portion of the bank and will also lower the saturation gradient by suitable drainage material at the toe. This apron will, of course, have to be laid at the lowest water level. It is also recommended that below the bottom of two feet thick stone, there should be a six inch layer of graded stones as in a water filter, so that there is no danger of sand filling up the interstices of stone and obstructing the free drainage. It will also be noted that the stone pitched bank proposed above is designed so as not to interfere with navigation.

DISCUSSION BY THE RESEARCH COMMITTEE

DR. BOSE introduced item (1) and said that in dealing with problems of tidal rivers by models the model work could be divided into two parts ;

Part I.—in which the hydrodynamical interaction between the boundary of the channel with the tidal flow in the channel was to be determined. In some simple cases of channel configurations the motion had been solved mathematically. When the channel configuration became complicated as was usual with tidal systems, the help of scale model was being increasingly taken to solve the problem.

Part II.—in which the boundary was moveable and there was fine colloidal material in suspension. In tidal rivers generally the fluid medium was saline which accelerated the rate of precipitation of the suspended colloidal material. No mathematical solution was possible in this case. Experiments by scale model were the only means by which exact quantitative solution could be obtained. Professor Gibson's Severn Model was the only model in which efforts were made to take into account all the above factors in model experiments.

There were however, certain hydrodynamical characteristics of tidal channels that preceded the general phenomenon of deterioration of their regime. These were supported by theoretical reasonings as well as observations. When a tidal channel showed tendencies of deterioration—

- (a) the values of maximum and minimum levels showed a tendency of going up,
- (b) the total energy of flow across a section of the tidal channel fell off.

In the experiments on Matla-Peali system the first part of the experiment was undertaken. It was proposed to take up the second part later on.

The first part of the experiment was now complete. The experiments showed that by closing Peali all the gauges in Matla and its other spill channels showed definite rise both in the high water and low water levels. But the total energy increase and the velocities at corresponding phases of the tide also increased. Though the rise in the gauges indicated deterioration, the increase in energy and velocity might counteract it. The results were therefore not conclusive. Further experiments with bed sand and suspended silt will now be undertaken.

MR. ROY gave further details of the experiment and said that Matla was a tidal river. The tides on the model were generated by the operation of a sliding gate at the downstream end of the model at Kultali. During the flow tide constant discharge was passed through the supply channel and this was the maximum required by the model. The supply required at other periods was controlled by the manipulation of the gate at Kultali. During ebb as the supply in the model was not required, it was by passed through a side channel. In both tides, that is, ebb and flow, the gates were controlled always by noting the gauge at Kultali and the time sequence of operation of the gates was noted. When once the gauge was reproduced, the gates were no longer controlled by the noting of gauge readings but were manipulated according to the time sequence.

The cycle was repeated more than 100 times and the result was statistically tested. It was found that the cycle was fairly well repeated. The standard deviation was found to be small and the readings consistent. Gauge readings were taken at interval of 30 seconds with the existing conditions of the river and simultaneously the readings of discharge in the two flumes were also taken in order to find out the discharge in the model. The energy of both flow and ebb was also calculated at this stage.

After this Peali was closed and the model was run with the same sequence of discharge as before. 100 cycles were repeated and analysed statistically. It was found that the cycles were well repeated and standard deviation was not significant.

It was found that High Water Level was increased nearly 12% at all places and Low Water Level was also increased at Kultali and this increase diminished as it went up along the river.

Energy of flow and ebb was also calculated at this stage at Kultali.

The experiment was in progress.

RAO BAHADUR JOGLEKAR introduced item (2) and said that Figures 2C. 56(a) and 2C.56 (b) showed the conditions of flow at the Titaghar Mill. It would be seen that the river was sharply curving upstream at the Titaghar Mill and immediately downstream there was severe contraction. This resulted in a very intensive diving flow, which had undermined the sand layers of the bank leaving the clay material behind, giving the shape of a knuckle. The conditions at the Mill were dangerous for navigation. The object of these experiments was to find out best methods to remove the attack from the left bank at the Mill,

Experiments were carried out with two models :— one 1/300 : 1/60 model which was laid in sand and the other larger, part-length model (1/150 : 1/80). Lines of flow were observed in the former, by running the model with a continuous ebb of the freshet spring tide observed on September 23 1941. This gave a discharge of 187,000 cusecs, the equivalent discharge of model being 1.34 cusecs.

Several preliminary tests had shown that severe conditions at the knuckle occurred only under the above tidal conditions and hence other tidal period was not considered. Lines of flow at surface and bed were closely observed in this model and the same were reproduced in a second, large, part-length model in which the horizontal scale was 1/150 and depth scale obtained experimentally = 1/80 with a discharge of 1.7 cusecs = 187,000 cusecs. Under these conditions there was movement of bed material. The object of repeating these experiments in a large model was to obtain qualitative results as regards accretion downstream of the knuckle, in the same way in which results were obtained with the Dunbar Mill knuckle described in the last year's Annual Report. The entry and exit lengths of this large model were sufficient viz. 20 feet and 30 feet respectively; also a grid was put at upstream end so as to get the correct distribution of velocities and discharge intensity. Various alignments of longitudinal and cross rows of piles were tested in these experiments and sand was injected upstream to see whether any accretion took place downstream of the knuckle. The results were not satisfactory showing that the treatment which was found successful at the Dunbar Mill was not feasible in this case. The chief reason was that the conditions at this mill were much more severe, in that, the change of flow took place, immediately upstream of the knuckle leading to more turbulence downstream. Besides as stated above, the mill property had been pushed too much inside the river causing a very severe contraction downstream. So the experiments were begun in the small 1/300 scale model with the object of diverting a part of the flow towards the right bank opposite the Titagarh Mill by means of islands.

'Attracting' islands had been tested in models and prototypes in the case of one-way rivers so far ; it was considered that they might yield useful results even in tidal and navigational rivers. 'Attracting' islands had been described by the Station as being unsuitable for training rivers such as the Luni near Samadari (Jodhpur Railway) where high formative discharges occurred for too short a duration to allow the island to leave its impress on river configuration; this Station considered that islands of suitable design, thoughtfully located and aligned in relation to the river meanders and the flow lines, had great promise on tidal rivers, where the high and prolonged freshet flow gave them ample scope as in the case of non-tidal rivers with a long duration of flood flow.

In the case of tidal and navigational rivers, one had to be very careful in properly locating the islands so that—

- (1) they should not obstruct the existing navigation channel ; and
- (2) should be located on shoals to minimise cost of construction.

The shape and alignment of islands and number of islands were varied in the experiments based on experience of previous experiments at the Station ; the islands were given a steeper slope on the side opposite right bank and flatter slope facing the left bank, so that more scour would take place on the steeper

slope side, which in turn, would attract more water to flow on the shoal near the right bank. Each alternative was tested after running ten freshet spring tides. It was found that a single island between cross sections No. 6 and 7 shown in Figure 2C. 72 gave better results than the island in conjunction with islands upstream and downstream.

MR. FRAMJI introduced item No. (3) and said that the problem was purely of a local nature and the slipping was not due to river attack. The river conditions upstream and downstream were not at all so severe as to cause the slips. There were slips just like those occurring usually in river *bunds* due to insufficient side slopes and insufficient drainage at the toe. He suggested giving a flat slope to the banks and recommended removing the cinders and ashes in the *bund* and providing a suitable stone apron.

RAO BAHADUR JOGLEKAR requested that the statement in the Bengal Report "It is the experience with tidal rivers in Bengal that such channels very seldom maintain their regime for sufficiently long time unless sufficient spill area is provided for them" be further elucidated. Spill water contained only fine materials in suspension and if this water was allowed to spill on the adjoining area, there would be less discharge to carry heavier material on bed. It was not understood how the channels would be in regime if spill areas were allowed. A river with embankments was expected to delay deterioration; where as the river without embankments would deteriorate earlier. The excess bed material was the main consideration, as this led to deterioration of the river and its meanders.

DR. UPPAL referring to the papers introduced by Mr. Framji asked whether he had carried out any other full scale model tests. Since he had only a limited section of the model of the river upstream of the island, he wanted to know how the direction of flow upstream of the first island was maintained.

MR. FRAMJI replied that the conditions upstream of the island were maintained correctly as obtained from the long model which had been proved against the prototype. As regards the second point whether entry conditions changed with worse river conditions and whether the island would then be suitable, he said that Dr. Uppal probably was not aware of the almost semi-rigid conditions of the Hooghly, which changes very little if at all.

DR. UPPAL wanted to know whether it would continue to be effective. MR. FRAMJI thought it would, since bank conditions on the Hooghly do not change appreciably.

RAO BAHADUR JOGLEKAR requested Dr. Uppal to refer to Figure 2C. 74.

MR. FRAMJI said that the signs of deterioration of a stream or a river according to the Bengal Report that the maximum and minimum gauges vary for the same discharges and secondly that the energy of flow across any section of the channel was reduced were secondary and stated that the primary sign, so far as engineers were concerned, was the sedimentation of the channel.

DR. BOSE commenting on items (2) and (3) said that he called these experiments unreal, in so far as they did not take into consideration the realities of tidal channels of Bengal. In the Ganges Delta one was extremely chary out introducing any sort of obstructions, not to talk of islands in total channel. Professor Gibson has also found in his experiment on Severn and Dee that the

introduction of bridge piers in these channels raised both the high and low water levels and at the same time reduced the tidal energy of flow. Both these were first indications of the deterioration of a tidal channel. In Bengal if any boat or any other snag sank in a tidal channel, particular care was taken to remove the same at once. If they were not removed immediately, an island started forming with the *snag* as the nucleus which by the end of the season could divert the main channel altogether. This happened simply because the tidal rivers of Bengal carried such a heavy quantity of fine silt in suspension that whenever there was any obstruction to the flow, sedimentation started immediately.

Further in these rivers there were two periods of "slack" in a tidal cycle of 12 hours 30 minutes. They were known as "High Water Slack" and "Low Water Slack" whose duration generally varied between half an hour to one and a half hours each time. Sedimentation took place mostly during these periods. In the experiments with the islands, no fine colloidal material was added to the water which he presumed was not saline. In these experiments with islands two most fundamental characteristics of tidal flow were omitted—the absence of fine colloidal material in suspension and the action of saline water on their rate of sedimentation.

In the experiment on Hooghly river at Konnagar a continuous ebb flow of a certain intensity was run in the model.

It was well known that 'scouring' took place not so much due to the actual magnitude of the velocity of flow but due to the change in the velocity either in space or time. In tidal scours both these factors were operative. They were more pronounced during flow than during ebb because of the fact that duration of flow tide was much shorter than that of the ebb and in consequence the change of velocity was much more rapid. These factors had also been ignored in these experiments.

Further action of a steady one-way current on a channel was bound to be very much different from that of a current which changed direction of flow every 5 to 7 hours. In a tidal curve, as the water level rose from P to Q Figure

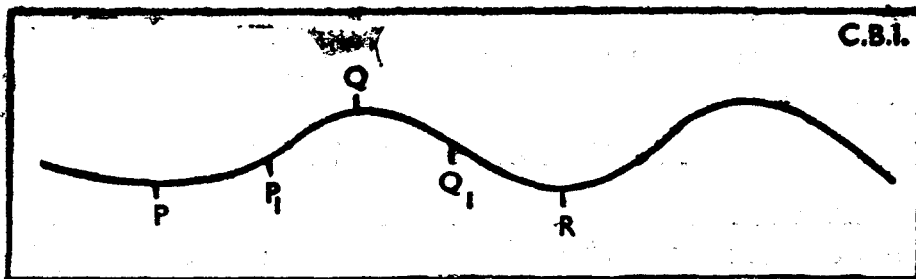


Figure 2 C. 77

2C. 77 the flow was upstream, while as the water level fell from Q to R the flow was downstream. Even in this upward movement of five hours the velocity changed from zero at P to maximum at P. 1 and then to zero again at Q. The same thing happened during the downward movement of $7\frac{1}{2}$ hours from Q to R. If the channel was straight, there could have been a chance of attaining uniform steady flow even for a short while but with a tortuous channel which

a tidal river always was, such chances were very remote. Further it was mentioned that a continuous ebb-discharge of 1.344 cusecs (=1,87,000 cusecs observed in 1947) was run with water level at Konnagar 19.5 feet above K. O. D. S. Though this one-way flow might represent one phase of the ebb-tide (say $\frac{3}{4}$ ebb) at Konnagar it would not correctly reproduce the ebb flow at other points of the river represented in the model. In this connection, see Figure 2C. 57. Though the ebb flow at cross section 4A might represent the prototype condition correctly, there might be high water slack at cross section 8A at the same time. In any case, the condition of ebb tide at the different sections would be different. This was not reproduced correctly in the model by running a steady uniform flow corresponding to a guage at one cross section 4A only.

MR. FRAMJI differed from the opinion expressed by Dr. Bose. Firstly, he said, Dr. Bose in effect appeared to rule out training measures altogether since he objected unconditionally to the introduction of any obstruction in tidal channels. Secondly, MR. FRAMJI contended, since the problem was not so much one of sedimentation as of scour, the omission of colloids and salinity would give the solution on the right side; whatever measure succeeded in eliminating the attack leading to the knuckle would have greater success in filling up the height, had colloids been injected and the model been running with saline water.

Thirdly, MR. FRAMJI continued, while he would agree with Dr. Bose that in certain tidal problems it would be wrong to test with a steady one-way current, in the present case action was much more severe at the knuckle with ebb flow of 1/87,000 cusecs at the head of the model than under any other condition. When preliminary experiments had established the fact beyond doubt—and when observations on the prototype by experienced local officers had confirmed it—, it would be wrong to persist with tidal flow and thus not subject the knuckle to the severest action.

THE CHAIRMAN (RAO BAHADUR VENKATACHARYA) closed the discussion and advised that in future every one should try to submit the report as complete as possible with all possible information supplied.

RAO BAHADUR JOGLEKAR stated that sliding gates used in Dr. Bose's experiments to create tides were according to the design in use at and supplied by the Poona Station.

DISCUSSION BY THE BOARD

THE SECRETARY referred to his note and said that the sub-heads 'Floods' and 'Stage-discharge relations' had been dealt with under Section A—Hydrology—2. 'River behaviour'. The rest of the sub-heads were to be examined here.

(I) FLOOD PROTECTION WORKS

The work done at the Research Committee Meeting was noted by the Board.

(II) RIVER TRAINING WORKS

THE SECRETARY described the discussion held at the Research Committee Meeting.

It was decided that the subject should continue to remain on the Agenda.

(iii) SCOUR AND EROSION

THE SECRETARY said that five items were discussed at the Research Committee Meeting and that there was a lot of useful discussion.

MR. K. K. FRAMJI referred to item (2) discussed at the Research Committee Meeting and requested Mr. D. Mehta to let the house know the results of the measures adopted on the Mahanadi anicut.

MR. SENAPATHY said that the scour hole had completely silted up.

DR. BOSE asked if there was any trouble due to siltation.

MR. SENAPATHY replied that there was no trouble at all now.

The Board noted the position.

(iv) SPILL

It was decided that the subject should continue on the Agenda.

(v) MEANDERING

The Board noted the work done at the Research Committee Meeting.

(VI) TIDAL ACTION INCLUDING THAT IN ESTUARIES

It was decided that the subject should continue on the Agenda.

3C-RESERVOIRS

PRELIMINARY NOTE

The Research Committee at their 1946 meeting recommended to the Board that the subject " Silting of Reservoirs " be changed into " Reservoirs " with two sub-heads (i) Silting of Reservoirs and (ii) Losses from reservoirs. The Board has since accepted the recommendations of the Research Committee.

(i) SILTING (SEDIMENTATION OF RESERVOIRS)*

This subject has been on the agenda since 1939.

At its 1946 meeting, the Research Committee recommended for adoption a set of standard instructions on the subject. The Board at its last meeting adopted a resolution (1) on the subject based on the recommendations of the Sub-Committee.

The following items were discussed at the 1946 Research Committee Meeting.

- (1) Observations at Dhukwan.
- (2) Pimpalgaon Tank in the Ahmednagar District.

(ii) LOSSES FROM RESERVOIRS*

This subject was introduced only last year (1946).

Recent Literature.

1. Lutz, J. F., and Hargrove, B.D.,—Soil movement as effected by slope discharge, depth and velocity of water, Agricultural Experimental Station, North Carolina, State College of Agriculture and Engineering, Raleigh, N. C. 1944.

2. Brown, Carl B. The Control of Reservoir Silting, Miscellaneous Publication No. 521, United States Department of Agriculture.

The Year's Work

(i) SEDIMENTATION OF RESERVOIRS

The following items were discussed at the 1947 Research Committee Meeting :—

- (1) Observations of suspended load at Dhukwan.
- (2) Relation between rate of silting and rainfall.
- (3) Relation between rate of silting and catchment area.
- (4) " Silting of Reservoirs " by B. P. Saxena.

(1) OBSERVATIONS OF SUSPENDED LOAD AT DHUKWAN (2)

ABSTRACT

Observations of suspended load at Dhukwan, initiated with a view to determine the rate of silting, have been continued. The suspended silt entering and retained by Dhukwan Reservoir has been calculated for the monsoon of 1946.

* Sedimentation of reservoirs and capacity surveys of reservoirs are the two sub-head proposed by the Research Committee this year (1947). The previous sub-head " losses from reservoirs " goes under Section A—Hydrology item (5) Regeneration and losses.

(1) Central Board of Irrigation, Journal, April 1947, page 136.

(2) United Provinces Public Works Department, Irrigation Research Station, Annual Report on Research Progress, 1946 Technical Memorandum No. 17 page 5.

TABLE 3C. 1
Showing weight of silt deposit at Dhukwan

Serial No.	Year	Average rainfall of the catchment in inches	Total weight of dry suspended silt entering the reservoir in lb	Total weight of dry suspended silt retained in the reservoir in lb
1	1941	20.6	4,848,228,000	1,146,499,200
2	1942	54.6	26,311,333,356	2,458,051,596
3	1943	33.8	8,074,146,564	2,348,302,176
4	1944	28.0	7,007,305,536	1,603,147,392
5	1945	34.0	19,853,834,400	3,703,552,200
6	1946	38.9	9,188,834,200	477,914,200

Figure 3C. 1 shows the relation between silt entering and silt retained in the reservoir, Table 3 C. 1 and average rainfall. There appears to be a possi-

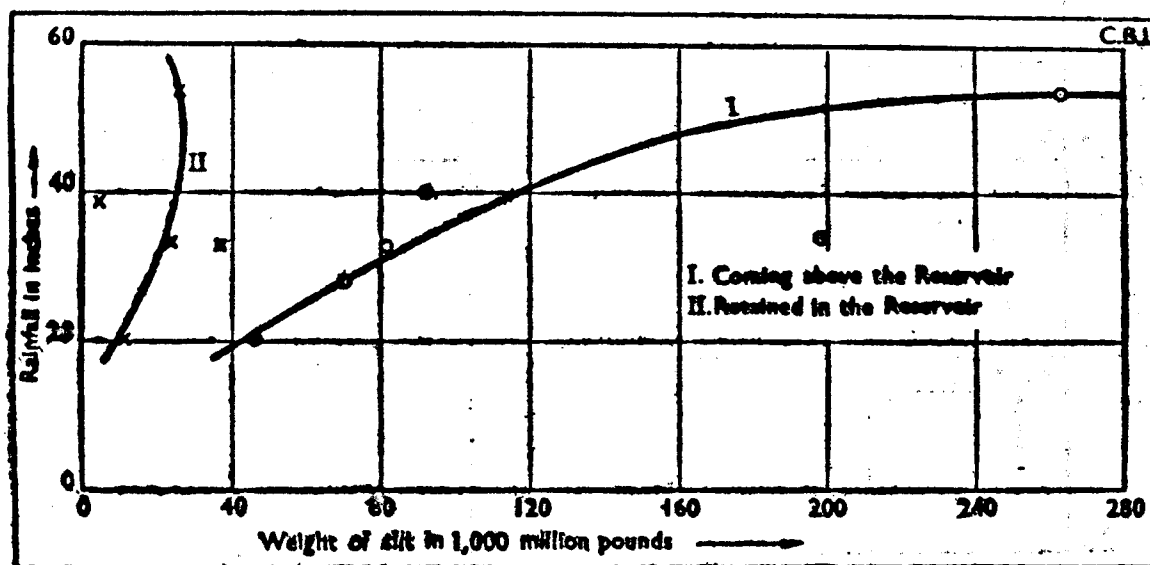


Figure 3C. 1: Relationship between rainfall over Dhukwan Catchment.

bility of relationship between the silt brought down and the annual rainfall which will be worked out when more data is collected. As in previous years an attempt has been made to determine the weight of dry silt contained in a cubic foot of deposit at different sites of the reservoir. The dry density of the reservoir deposit varies from 77.5 lb. to 71.0 lb. per cubic foot, as its moisture content increases from 33 per cent. to 40 per cent. Assuming the density of deposited material to be 77.5 lb. per cubic foot on account of compaction done by the superimposed load. The total volume of deposit for the year works out to be 6.17 million cubic feet or 141.3 acre feet.

It is intended to continue these observations for some years. Similar observations have also been started at the Nayar River to estimate the silt charge of the river above the site of the proposed dam.

(2) RELATION BETWEEN RATE OF SILTING AND RAINFALL (3)

Last year, a graph (4) was plotted, giving the variation of silt entering and silt retained in Dhukwan Reservoir with rainfall in the entire catchment

(3) United Provinces Public works Department Research Station Annual Report on Research Progress 1946, Technical Memorandum No. 17 P. 6.

(4) United Provinces Technical Memorandum No. 16, Graph No. 8, Page 8.

This year, the same relationship has been attempted for a number of Bundelkhand reservoirs for which continuous records of total annual accumulation of silt as well as rainfall were available.

A graph has been drawn showing rate of silting, *i.e.*, annual accumulation of silt in acre feet per 100 square miles of catchment against annual rainfall in inches for different reservoirs, *vide* Figure 3C. 2. Figures have been taken from columns (6) and (8) of Table 3C. 2. It may be seen that except for Koncha

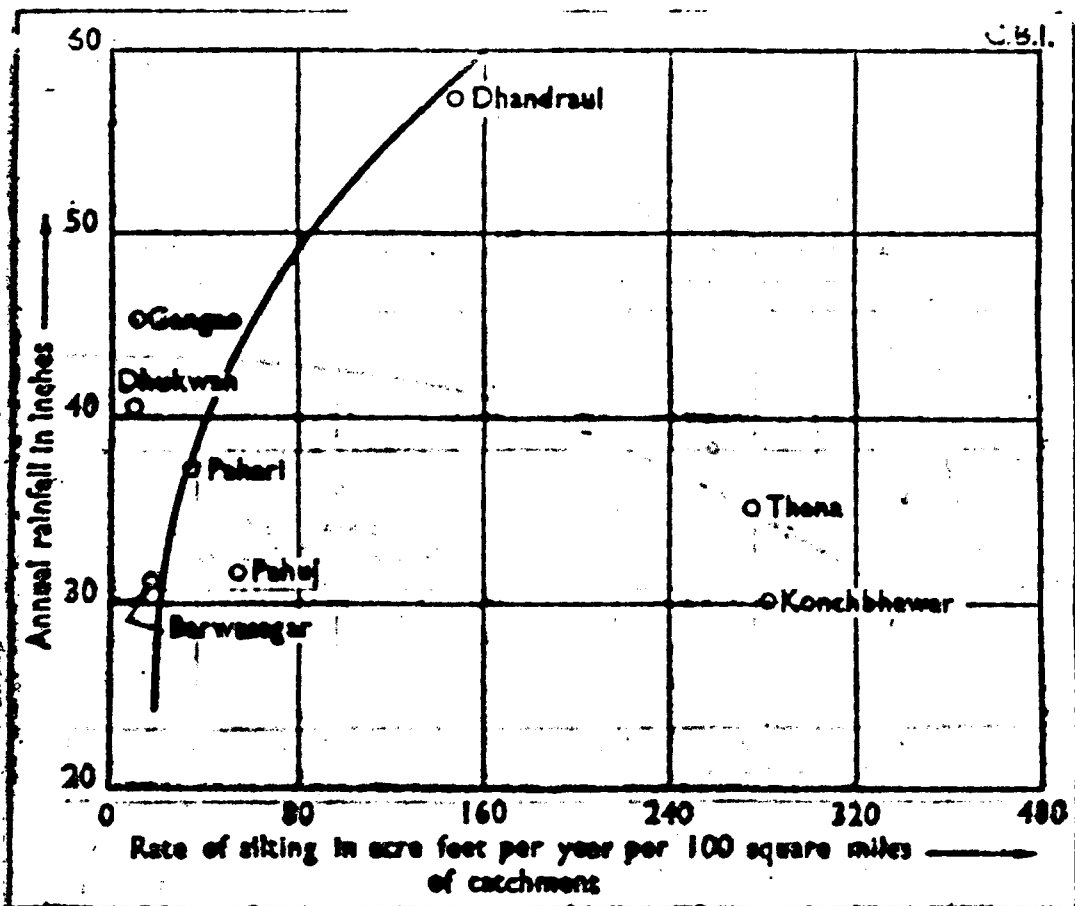


Figure 3 C. 2 : Graph showing rate of silting in acre feet and annual rainfall in inches

Bhawar and Thana, which are very small reservoirs, there is some degree of association between rainfall and rate of silting though it is clear that rainfall alone cannot be the main factor responsible for silt accumulation. It may also be concluded that very small reservoirs, whose catchment area is of the order of three square miles and less, behave differently from larger reservoirs in that the rate of silting of the former is very much higher.

(3) RELATION BETWEEN RATE OF SILTING AND CATCHMENT AREA (5)

The formula (6) expounded by Sir Claude Inglis was tested for Bundelkhand catchments. Sir Claude assumes :

(5) United Provinces, Public Works Department, Research Station, Annual Report on Research Progress, 1946, Technical Memorandum No. 17, page 6.

(6) Relationship between Silt Volume accumulated in Basins and Reservoirs and Area of Watershed in C.B.I. Publication No. 24.

TABLE 3C. 2

Statement giving rate of silting and value of $C = \frac{S}{A} \cdot 75$ for various Catchments in the United Provinces.

Serial Number	Name of catchment	Nature of catchment	Catchment area in square miles.	Mean annual run off/Reservoir capacity	Average annual rainfall in inches.	Average monsoon rainfall in inches.	Rate of silting per 100 sq. mile of catchment in acre-feet	Average annual accumulation of silt in acre-feet	Value of $C = \frac{S}{A} \cdot 75$	Time over which measured (years)	Remarks
1	2	3	4	5	6	7	8	9	10	11	12
1	Dhukwan ..	Hilly and Rocky ..	8,240	46.4	40.60	36.66	7.60	626.24	0.7241	35	Based on capacity survey of 1907 and 1942
2	Barwasagar ..	Hilly covered with forests	71.00	4.00	31.29	29.69	16.00	11.36	0.4644	73	..
3	Gangao ..	Hilly and rocky with little cultivation	7,199	92.50	..	45.30	8.86	637.78	0.8160	27	Based on capacity surveys of 1912 and 1939
4	Dhandraul ..	Hilly with steep slope	110	1.30	57.28	49.73	147.80	162.56	4.7860	12	Based on capacity surveys of 1933 and 1945
5	Pahari ..	Hilly with bare steep slope	3,026	23.40	37.34	32.69	34.23	1035.86	2.5390	28	Based on capacity surveys of 1911 and 1939
6	Thana ..	Partly hilly wooded with steep slope	3.20	0.51	35	29.28	275	8.8	3.679	73	..
7	Pahuj ..	Plains and Hilly ..	120	..	31.72	28.70	53.30	71.20	1.7640	5	Based on capacity surveys of 1940 and 1945
8	Koncha Bhawar ..	Plains ..	2.70	1.60	30.30	28.36	282.00	7.61	3.6140	22	..
9	Bara Tal ..	Plains ..	2.00	0.00	402.00	8.04	4.7300	27	..

Note :—Monsoon rainfall has been taken for annual rainfall in case of Gangao

$S = 0.5 A^{0.75}$ acre feet per year for rocky catchments,
 $S = 1.7 A^{0.75}$ acre feet per year for normal catchments,
 and $S = 5.5 A^{0.75}$ acre feet per year for soil catchments,
 with intermediate values of C for intermediate types of catchments, where
 S = silt accumulated per year in acre-feet,
 A = catchment area in square miles.

Values of $C = S/A^{0.75}$ were worked out for various Bundelkhand catchments and are given in column (10) of Table 3 C. 2 the figures being calculated from columns (4) and (9), which give A and S respectively. It may be seen that except for Dhandraul, Pahari and Thana, the values of C for most of the reservoirs, whether big or small, agree fairly well with Sir Claude's constants, which may, therefore, be adopted for Bundelkhand catchments.

CONCLUSION

Sir Claude Inglis's formula $S = CA^{0.75}$ for annual accumulation of silt is recommended as a useful guide for Bundelkhand catchments.

(4) SILTING OF RESERVOIRS

by

B. P. Saxena, I.S.E.,

Superintending Engineer, U.P., Irrigation Department.

ABSTRACT

Gives an analysis of observations of the quantitative study of the probable rate of silting of some of the reservoirs in the United Provinces over a long period by capacity surveys. A few typical examples of the smaller reservoirs have also been incorporated.*

INTRODUCTION

It is well known that glacial, torrential and other streams do carry varying silt loads, depending on the physical characteristics of the catchment, the vagaries of rain and snowfalls and peculiarity of dispersing basins. In the case of impounding reservoirs, the transported sediment is trapped by the dams, and its accumulation in the storage basin gradually reduces their capacity. Experience gained in recent years shows that the loss of storage by silting is a major problem, not just because some sediment is deposited in our lakes, but because it is deposited in such quantities as to cause economic loss to the present as also to future generations.

Water storage is a vital factor in the economic structure of United States of America, where nearly five billion dollars have been invested on the construction of some 9,000 larger dams and impounding reservoirs. They provide one third of country's electric power, furnish water supply to areas where a fifth of population resides and irrigate lands that produce five per cent. of its agricultural products. Observations indicate that many of the reservoirs are losing from one to five per cent. of their capacity every year, and some of them have got completely choked up in a period ranging from one to five years. Some 2,000 smaller reservoirs have got filled to the top of the dam

*Not printed but included in C.B.I. Publication No. 43, Silting of Reservoirs.

with the debris brought by the streams and the annual loss due to reduction of storage is computed to be upwards 50 millions dollars. Naturally, the subject of silting has assumed great importance in the United States and is being studied on scientific lines.

Here in India, keen interest is now focussed on the problems allied with the silting of reservoirs, greatly for the consideration of Post-war Schemes of Dam construction all over the country. Huge stakes are involved in the contemplated projects, which are proposed to provide irrigation for large unirrigated areas and to generate power for the industrialisation of the country. The capital outlay may run into millions of pounds, and the subject for study is of all India importance in which this Province has its due share.

ORIGIN AND NATURE OF SILT

Silt is produced by the process of disintegration and erosion of rocks and soil in a catchment under the action of wind and water, and is transported by flowing water into streams, in suspension and as bed load. The principal factors leading to the production of sediment are (1) sheet erosion or the removal of surface soil by overland flow, (2) gulying or the cutting of channels in soil by concentrated runoff, (3) the washing away of material from cuts, falls and surfaces of lines of communications and (4) the scouring of material by concentrated flow from the beds and banks of streams in valleys. The quantitative and qualitative yield of silt is dependent on watershed characteristics of the catchment and on the factors which influence erosion in the different basins. If precipitation and high rates of runoff, turbulence of flow, steep country slopes and extreme variations of temperature tend to increase erosion, protective cover, impermeability, adhesion and low runoff form a resisting force against it.

TRANSPORTATION OF SILT

The material produced by erosion varies from large boulders to fine particles of sand and clay. It is carried through by movement as suspended and as bed load by a complicated process. Its disposal along the course of the carrying stream depends on the nature of the flow ; accumulation of the material is caused when the velocity is checked whereas its removal is accentuated by the acceleration in the velocity. The finer particles are in suspension near the surface of the flow while the proportion of coarser material which is pushed and rolled along the bed increases according to the depth of water. A rising river overcharged with sediment tends to deposit its load ; a falling river with a deficit of silt tends to scour. If the river is turbulent, the bed silt is carried up to the surface and the distribution of sediment in a volume of water is more uniform than it is when the river flow is regular and gentle. Theoretically, there is no clear demarcating line to determine the exact relative proportions of suspended and bed loads, although experiments indicate that there is an empirical and unique division between the grain sizes carried by each mode of transportation (?).

(?) Einstein, H. A. and others " Distinction between bed load and suspended load in natural stream. "

SUSPENDED LOAD

Silt in suspension varies greatly in weight and volume, depending on the source of sediment, intensity of rainfall and rate of runoff, so that the measurements of a silt load for a given discharge will not indicate that the same load may be carried by an equal discharge at another time. The degree of concentration of silt is governed by the size of particle, the discharge of the stream and by fluid turbulence.

Suspended sediment is continuously tending to settle down but is being thrown back into the stream by scour from the bottom, due to turbulence. There is equilibrium as long as rate of scour is equal to rate of settlement, but sediment concentration increases when rate of scour is greater. On the other hand, the stream unloads when rate of unloading is related with the size and the settling velocity of particles in suspension as also with the depth of the magnitude of turbulence. Thus, it is seen, that the degree of turbulence is connected with the distribution of sediment concentration, and E. W. Lane and A. A. Kalinske (8) have developed analytical expressions for total sediment load in unit sections, which may give more accurate estimates of suspended sediment than the averages of a few measurements at arbitrary sampling points.

The common practice is to determine the suspended silt by standard methods of sampling and analysis. As mentioned by Khosla (9), a sample taken from six-tenth depth will give the mean percentage of silt in the vertical within the limits of permissible error, and that the mean of the results from three tests made at the centre of the section and at distances of one-sixth the width from each edge of the stream will give mean percentage for the cross section.

The Development and Research division, Sind had been conducting experiments on estimating suspended load in alluvial channels and in their report for December, 1945, they have recommended "tenpoint precise method" for rivers which usually have a departure from uniform flow conditions.

BED LOAD

Whereas analytical and practical methods have been evolved to estimate suspended loads with a fair degree of accuracy, bed loads are so variable under different conditions that accurate computations are not possible. Particles which are too large for movement in suspension are carried by transporting forces as bed load, and an estimate for particular discharge and slope conditions may be made by analysing bed surface material, immediately after a flood. Large scale experiments have been conducted on American rivers, and although it is indicated that bed load unlike suspended load is directly related to the discharge at all times, the investigations are inconclusive in determining a relation between bed load and the total charge of a stream. Investigations for the design of the All-American Canal indicated that Schoklitseth bed-load formula gives satisfactory results. This is :

$$G = \frac{86.7}{D^{\frac{1}{2}}} S^{(Q-bq)}$$

(8) Lane E. W., Kalinske A.A., "Engineering calculations of suspended sediment."

(9) Khosla, A. N., "Silting of Reservoirs".

G is total bed load in pounds per second ; D is effective grain diameter ; S is slope of the energy gradient ; Q is total instantaneous discharge in cubic feet per second ; b is width of river in feet ; and q is critical discharge in cubic feet per second per foot width at which movement begins and is equivalent to $\frac{0.00532 D}{S^{\frac{1}{2}}}$. The bed load can roughly be calculated by assuming a single repre-

sentative grain size for the bed load sediment and means values for the slope and discharge. Accurate computations are, however, not possible owing to variations of bed material from time to time and shifting effective bed widths for which necessary assumptions have to be made. There are two useful contributions which discuss the theory of suspended and bed loads and which have been published (40).

According to the numerous investigations made on American rivers and streams, the bed load has been variously estimated to vary from one to 25 per cent. of the suspended load. There is, however, no mathematical relation between the quantities of bed load and suspended load as the two are entirely dependent on the nature of catchment and on the transporting capacity of the stream.

PHENOMENON OF SILTING IN RESERVOIRS

The process of sedimentation commences from the time when an artificial storage is created by the erection of a barrier to the normal flow of a stream. Velocity of flow is reduced due to its impinging upon the pond of water in a reservoir and the unloading of the fluid charge starts. The heavier sediment, whether in suspension or moving as bed load is deposited first, tending to form a sort of a delta at the junction, gradually extending upwards into the river at a flatter gradient and partly into the reservoir at a steeper slope. The form of the delta, however, is affected by variation in reservoir water level ; and no deltas may be formed if the stream does not carry coarse sediment. Silt laden flow, on its entry into the still-waters of a storage basin, has a greater specific gravity. It tends to settle down as it advances over a fan shaped area towards the dam, displacing clear water of the storage which rises up and is the first to be discharged over the spill-way as inflow increases beyond the capacity of the basin. Dissipation of transporting forces and denseness of the load naturally lead to deposition of sediment, in the deeper areas of the reservoir, in relation to the shape of the basin. If it has a regular shape, the deposits are formed uniformly along the axis ; if it is shaped irregularly, the silt is distributed heterogeneously. The heavier particles, in any case, are deposited first, the smaller sizes are carried further and may not be deposited at all, depending on the volume of inflow and the relative turbidity of the fluid escaping from the basin. When a reservoir has enough capacity to accommodate the incoming floods with sufficient time to permit all the sediment to settle down, the basin acts as a trap ; on the other hand, if the basin can accommodate only a fraction of the total inflow, relative deposition of suspended load will depend on the intensity of runoff and the turbulence of flow, even if the whole of the bed load may be intercepted.

(10) The Central Board of Irrigation, Quarterly Bulletin No. 15, 1936 and Journal, October 1944.

It has to be noted that the storage basin of a reservoir exerts a certain influence on silting in its own way. When the flood velocity receives a cushioning effect according to the height of dam, sedimentation starts due to loss of velocity. The velocities at a cross section will vary from the centre towards the flanks. Farther are the flanks apart, the greater will be the difference in velocities. Consequently, a fan type basin is likely to have greater sedimentation. On the other hand, if the reservoir is confined within narrow and high banks, the variation in velocities is lesser and greater part of the silt load is carried away, oftener when the capacity of reservoir is only a small part of the annual inflow.

ANALYSIS OF THE SILTING OF RESERVOIRS IN BUNDELKHAND

The composition of watershed catchments of Bundelkhand reservoirs is more or less similar, but the trend of silting is different. The comparative rate of sedimentation has been given in Table 3C.3. The conclusions are :

(a) Reservoirs like Dhukwan, Pahari and Gangao form one class. There are no reservoirs above them and their sluicing capacity with respect to normal floods is too small for consideration. Rate of sedimentation is high in the first 15 to 20 years, varying with the nature of the catchment from nine to 37 acre foot per 100 square mile catchment per year. Later, the rate rapidly falls down and may ultimately be negligible. This is easily explained. The runoff capacity ratio is high being roughly 100 to 1, and the river spates require a certain section to maintain the flow. The obstruction by the dam causes the dips and the flanks of the storage basin to fill up with the silt in the early years. A stage comes when the river section is adjusted to carry the normal flood discharge. Under such a condition, the disposal of the suspended load in the area of the lake is harmonised with the conditions of flow as they are in the other parts of the river and silting is greatly reduced. Besides, the progressive development of deltas above reservoir level helps in trapping some of the silt load, and shrinkage combined with settlement of silt deposits due to weather and due to superimposed load of additional silt reduces the silt volume in the storage basin.

(b) Subsidiary reservoirs like Lachura below Pahari, Bariarpur below Gangao, and Pahuj below Garhya and Palanpur show insignificant silt deposit in the first 15 or 20 years, when the upper reservoirs had been picking up the maximum volume of silt. As the lakes above are forming a stable section for the passage of normal floods, more silt is travelling downwards to subsidiary reservoirs where rate of silting is now six to seven acre feet, per year per 100 square mile catchment, and is showing a rising tendency.

(c) Bara Tal and Koncha Bhanvar tanks are the worst silters. Capacity runoff ratio is unity and so, the normal runoff along with the entire bed and suspended load is accommodated in the lakes almost every year. In these two cases, silt is being deposited at the rate of eight acre feet annually ; the capacity of the lakes is about 800 acre feet each, and at this rate they are likely to get partially choked up in a period of 100 years. The rate of silting in later years may, however, rapidly decrease when the reservoir silts up to a section required for normal flood discharge, and the useful life of the lakes may be somewhat prolonged.

TABLE 3 C. 3

	Reservoir		Catchment area (square miles.)	Height of Dam (feet.)	Storage capacity			Annual Rainfall (inches)	Annual run-off (Acre feet)	Run off per square mile catchment (acre feet)	Years of capacity			Silt deposited		Silt deposited		Remarks
	River	Name			Foot acres	Per cent. of amount run off	Per square mile of catchment				First	Last	Period in years	Total (acre feet)	Annual (acre feet)	Per cent. of original capacity per annum	Per cent. square miles of catchment per annum	
1	Betwa ..	Dhukwan ..	8,240	50	56,252	1.00	6.9	42	5,660,000	687	1907	1942	35	24,870	801	1.24	8.5	
2	Ken ..	Gangao ..	7,199	53	48,645	0.81	6.8	45	6,000,000	833	1912	1945	33	19,872	802	1.24	8	
3	Dhasan ..	Pahari ..	3,026	54	38,284	1.32	12.7	38	2,760,000	912	1911	1939	28	23,454	838	2.19	27	Has a high percentage of cultivated area in the catchment
4	Betwa ..	Paricha ..	10,380	55	50,305	0.70	5.0	41	7,200,000	694	1912	1943	31	-10,411	-336	Scour	-10	Has continuously scoured. Subsidiary reservoir
5	Dhasan ..	Lachura ..	3,240	57	16,728	0.56	5.2	38	2,960,000	913	1908	1945	37	4,891	132	0.30	4	Subsidiary reservoir
6	Ken ..	Bariarpur ..	7,699	26	4,669	0.07	0.6	45	6,400,000	331	1906	1945	39	1,177	30	0.64	Insignificant	Subsidiary reservoir
7	Pahuj ..	Pahuj ..	120	35	7,460	14.1	62	39	53,000	442	1914	1945	31	345	11	0.15	9	Has two small reservoirs above in the catchment
8	Barwar ..	Barwasagar	71	24	8,300	27.7	117	36	30,000	422	1868	1933	65	900	14	0.17	20	
9	Barwar ..	Bara Tal ..	2	..	1,020	113	510	37	900	450	1906	1933	27	220	8	0.80	400	
10	Barwar ..	Kunch Bhanvar	2.7	..	77	64	285	36	1,200	444	1911	1933	22	170	8	1.04	296	

GENERAL OBSERVATIONS

Sedimentation in reservoirs is governed by a large number of factors, such as, the physical characteristics of the catchment, capacity runoff ratio transporting power of the runoff, the form and shape of the storage basin, the escaping capacity of the sluices with respect to normal flood discharge, *etc.* The rate of sedimentation per year per 100 square miles of catchment of Bundelkhand reservoirs during some 30 years of existence ranges from 4 to 27 acre feet when runoff capacity ratio is as high as 1 in 80 to 1 in 100. The case of reservoirs having capacity runoff ratio approaching unity is, however, different. Such reservoirs with small catchments of a few square miles, show a high rate of silting, *i.e.*, about four acre foot per square mile catchment per year, as the entire silt of a season is readily trapped in the storage basin which is within an easy approach. On the other hand, reservoirs with bigger catchments and capacity runoff ratio still nearer unity, have a greater area for the dispersal of transported silt load. The rate of silting in such cases, is more controlled although the entire runoff is accommodated in the lake.

No formula can be laid down for the rate of silting, but it is observed that the extent of the cultivated and other erodable area as also the low runoff capacity ratio are likely to give a high rate of sedimentation. Rai Bahadur A. N. Khosla has suggested the maximum rate of silting as 75 acre feet per 100 square mile of catchment, and he has probably based his conclusions on such data of Indian reservoirs that he had and on the study of the reservoirs of other countries. The statistics compiled for the reservoirs of this Province show that the maximum rate of silting, as on Pahari reservoir, is 28 acre feet per 100 square mile catchment on an average of 38 years. This rate has a downward tendency, and as time passes by the average rate per year is likely to be less than 28 acre feet, as after all, the reservoir should be left with an equivalent river section to carry normal spates.

DISCUSSION BY THE RESEARCH COMMITTEE

MR. CHATURVEDI introduced items (1) to (3). He said that in the United Provinces they had started experiments on the determination of suspended load and also on silting of the reservoirs in the Province. For the Dhukwan reservoir, data for six years was now available. The results were not conclusive. But they gave an indication of the silting for this period. For this particular reservoir the ratio of water stored to the total run-off was rather small, five or six per cent. The silting was definitely reduced with higher rain-fall. Figure 3C. 1 showed that as the rain-fall increased the silting tendency did not go up proportionately. They had also found that the relation between the rate of silting and the catchment area worked out by Sir Claude Inglis ten years back was applicable to the reservoirs in the United Provinces also.

MR. SAXENA introduced item (4). He observed that his paper related to reservoirs in the United Provinces only. The capacity of reservoirs had been considered for a number of years, from which a reduction in silting had been noticed. He tried to get similar data regarding the reservoirs in the South and in Ceylon, but unfortunately they had no regular record of this kind and so he could not compare the rate of sedimentation with other provinces. His

conclusions were two-fold, one that the rate of sedimentation depends on (1) the shape of the reservoir and (2) on the capacity and run-off ratio. The entire static silt was carried into the reservoir. Laboratory experiments of bed and suspended load were carried out and useful data was arrived at. Rai Bahadur Khosla after his return from America had written a note on silting in reservoirs, in which he had analysed the rate of silting and given the maximum rate as 75 acre feet per 100 square miles. His conclusion was that the rate of silting for reservoirs of the United Provinces was to the extent of 28 acre feet per 100 square miles. Such reservoirs which have capacity-runoff ratio tending to unity have a much higher rate of silting. On the other hand, there is one case of Paricha reservoir where the capacity has appreciably increased due to erosion of bed and sides.

RAI BAHADUR BIJAWAT said that in Bundelkh and the question of finding out the rate of silting was taken up a very long time ago, probably in 1912. In 1913-14, they made some observations at Dhukwan and Pahari for finding out the quantity of silt carried by the river. At that time they thought that the life of the Pahari reservoir would not be more than 40 or 50 years. Already nearly 35 years have passed and the capacity for Pahari reservoir had not very much changed. Therefore, there was something wrong in the calculation of the rate of silting that was then determined. He agreed with Mr. Saxena that there were many other factors besides those which were then considered. He hoped that the experiments which were now being carried out would take into consideration the fact that reservoirs which were expected to fill up in 40 to 50 years time as per theoretical calculations had not yet been effected very appreciably.

MR. BLENCH said that the colloidal fraction of a river's transported sediment could account for a major portion of a reservoir's loss of capacity from sedimentation. Sand and silt settle in the delta, at a gross density of about 92 lbs. per cubic foot, but the clay size material passes forward in a density current and will pass through a reservoir even as long as the one at Boulder Dam provided the means are provided to remove water from the base of the Dam. If the means are not provided, then the material will gradually settle into a pasty mass of bulk dry density about 24 lbs. per cubic foot that is, for a given mass, the clay-size material will waste four times as much reservoir storage space as will non-cohesive deltaic material. If, as in the Lower Mississippi the clay content of sediment is about 40 per cent. by weight, then a dam that provides means to remove the density current would increase the life of its reservoir nearly three times. He had recommended density current withdrawal for Bhakra Dam, where the colloidal content of the water was probably only five per cent., mainly because the collection of that material in the reservoir would deprive the irrigation water of the Sutlej Valley of its fertility; he had heard adverse comment on such deprivation when visiting the Imperial Valley area below Boulder Dam.

Referring to the river sediment observations and bed surveys in the Elephant Butte Reservoir, U. S. A., he pointed out that when their results were quoted to prove a certain averaged density of deposited material they ignored the differentiation that must have occurred actually. In a reservoir

long enough to permit a density current to form, the mean density of deposit could be used to estimate the clay fraction if deposition in the delta and the after-delta were taken to have the densities he had quoted ; clay subjected to alternate wetting and drying, or to load, would, of course, increase its density.

MR. COCHRANE said that though the Central Provinces had not done much research in this direction some investigations were carried out in 1940-41 after a bad monsoon, combined with heavy draw-offs for irrigation purposes, had left a number of tanks dry. The depth of silt deposit was ascertained directly by digging numerous test pits in the tank beds. The results indicated an annual deposit rate of the order of from 6 to 12 acre-feet per 100 square miles of catchment area, but were by no means conclusive.

MR. CHATURVEDI said that he wished to confirm what had been said by Mr. Blench. He also considered that density was to be taken at two or three places. In reply to Rai Bahadur Maydeo the speaker said that the higher the floods, the higher was the rate of silting. Also older the reservoir, the less was the volume of silt accumulation and hence less was the percentage of silting.

MR. BLENCH then moved the following resolution :—

Resolved that the Research Committee recommends to the Board to request all provinces and states that :—

(1) At discharge-observation sites on main and tributary stream, sediment discharge observation, with analysis into principal sizes, should be made concurrently with every water discharge-observation. The result should be correlated annually with erosion intensity surveys done by the Forest Department.

(2) Every project for damming a river system should contain a report detailing; in quantitative terms, the whole relevant sediment data and the economics of action required to protect the reservoirs from rapid loss of capacity, and the rivers from undesirable regime changes. This report should be prepared by conservation experts working in liaison with engineers from the time of conception of the project.

MR. BLENCH said that he had used the word sediment because the term "silt" had now acquired the specialised meaning of "material of diameter between 0.002 and 0.02 millimeters".

He described how, in the U. S. A., reservoirs had been formed without consideration of sedimentation and how, in 1935, the U. S. A. Government had to create a Soil Conservation Department to assess the magnitude of past errors, and recommend corrective and preventive action. He described to the meeting the tremendous damage that could be done to the physics of a country by unconsidered tempering with rivers, and depicted the situation that would arise when a community that had built its prosperity on the water of a reservoir found that its source of prosperity was no longer of use against periods of drought and would vanish altogether within one or two generations. He said that, in the U. S. A., Sediment Surveys and co-operation between departments were now recognised as essential and were being put into effect although current literature had not yet given these actions publicity ; and he was anxious that India should start its projects with adequate data and with proper cooperation

between those who built works and those who understood their consequences. He was of the opinion that suitable soil conservation actions, enforced in time on a country, could often, after allowing for their cost, produce successful conditions for reservoir projects that otherwise would have ended in failure; hence the need for data that would show the locus for conservation action, and for conservation experts and engineers to work together from the time a project was contemplated.

After some discussion, the resolution moved by Col. Blench was passed unanimously.

(ii) CAPACITY SURVEYS

There was no item for discussion under this sub-head.

DISCUSSION BY THE BOARD

THE SECRETARY said that the two new sub-heads were (i) sedimentation of and (ii) capacity surveys. He recalled that a comprehensive resolution on this subject was passed by the Board at its last meeting in 1946. This resolution has been published in the C. B. I. Journal for April 1947. It was also forwarded to all administrations. The replies received from Sind, Cochin, the United Provinces and Bombay showed that instructions had been issued to the officers concerned for implementing the recommendations of the Board. The Central Provinces, Madras and Ceylon had replied that due to shortage of staff it was not possible for them to take up the observations for the present.

Continuing THE SECRETARY stated that Chief Engineer, Jodhpur had raised a few points regarding the resolution of the Board as under—

1. Due to shortage of staff Jodhpur could not take up the observations in accordance with the recommendations of the Board but if the information pertaining to measured depths of silt deposit obtained by digging the beds of tanks when they were dry, the ages of which were known accurately would be of value, the Chief Engineer was prepared to supervise the obtaining of it personally.

2. Regarding the advancement of delta into tank beds, he suggested that the only satisfactory way of recording would be by means of air photos taken at five years interval over a representative series of tanks; this should not be either a difficult or costly matter to arrange.

3. Since there was very little silt in Rajputana rivers, yearly measurement of silt by levelling or soundings would not show significant changes he suggested that the interval should be five years.

4. For flashy rivers, silt sampling is difficult to arrange and the information to be collected would not commensurate with the cost and trouble involved.

The Board examined the points raised by the Chief Engineer, Jodhpur and after discussion, decided as follows :—

1. The information so obtained would be very useful, and Chief Engineer, Jodhpur be requested to collect it. He should also be requested to ascertain if possible the dry bulk density of silt *in situ*, by layers, in its wet (spongy) state and whether it is possible to distinguish the various layers—yearly or otherwise.

2. A good suggestion which might be adopted for all reservoirs. The period between two successive photographs would vary for each case.

3. The period would vary for different places and must be determined by local officers.

4. Agreed.

Continuing the Secretary intimated that as pointed out in his report the Executive Committee had approved the publication of "Silting of Reservoirs" by Rai Bahadur A. N. Khosla, and other connected data.

(i) SEDIMENTATION OF RESERVOIRS

The Secretary described the work done at the Research Committee meeting and requested the Board to examine the resolution passed by it.

After some discussion, the following resolution was passed by the Board :—

Resolved that all provinces and states be requested that —

(i) As an ultimate objective, but for the present as far as possible, at discharge observation sites on main and tributary streams, sediment discharge observations, with analysis into principal sizes, should be made concurrently with every water discharge-observation. The results should be co-related annually with erosion intensity surveys done by the Forest Department.

(ii) Every project for damming a river system should contain a report detailing, in quantitative terms, the whole relevant sediment data and the economics of action required to protect the reservoirs from rapid loss of capacity, and the rivers from undesirable regime changes.

* The Board further recommends that soil conservation experts be appointed from the investigation stage, to each reservoir project.

(ii) CAPACITY SURVEYS

There was no discussion

4 C—Dams and Weirs

PRELIMINARY NOTE

At the last (1946) meeting of the Board the subject was sub-divided as follows :—

- (i) Design.
- (ii) Materials.
- (iii) Construction.
- (iv) Investigations.
- (v) Fish passes.*

Only the first two-sub-heads existed at the time of the last (1946) Research Committee Meeting.

(I) DESIGN

The following items were discussed at the 1946 Research Committee Meeting :—

- (1) Critical depths and velocity for stage flow in dam tunnels.
- (2) Experiments to determine a suitable profile of the lower Sind Barrage to cause minimum afflux.
- (3) Note on value of ' C ' to be adopted for prototype as determined from results of experiments on models of Hajipur Barrage carried out by Indian Waterways Experiment Station, Poona.
- (4) The determination of economic flood and earthquake probabilities for the design of dams by A. R. Thomas.
- (5) A simplified method of earthquake allowance in design of gravity dams by A. R. Thomas.
- (6) Development of photo-elastic methods for the analysis of stresses.
- (7) An Economic type of surplus weir.

Recent Literature.

- 1. Huge Earth Fill Dam will Enlarge Eildon Reservoir—The Commonwealth Engineer, Volume 34, No. 4, November 1, 1946.
- 2. Case School Engineers devise Novel Energy Dissipator for Mt. Morris Dam—Engineering News Record, Volume 137, No. 7, August 15, 1946.

(II) MATERIALS

The following items were discussed at the 1946 Research Committee Meeting :—

- (1) Laboratory studies on *Kankar* and *Kankar* limes.
- (2) Studies on segregation of coarse aggregate in a concrete mix.
- (3) Hydraulic gradients in earthen dams.

*Introduced under this subject head provisionally by Secretary. This sub-head now goes under Dam Appurtenances.

- (4) The effect of the addition of argillaceous materials to cement concrete I: Segregation of coarse aggregate, by R. C. Hoon, Mohinder Singh and Raghbir Singh.
- (5) Laboratory studies on *Kankar* and *Kankar* limes by Raghbir Singh, J. C. Bahri and R. C. Hoon.

Recent Literature.

1. Billig, K: Electro Concrete, Journal of the Institution of Civil Engineers, No. 8, October 1945.
2. Bean, L. and Litvin, A: Effect of Heat on Portland Cements Containing Vinsol Resin—American Society of Testing Material, Bulletin No. 135, Augsut, 1945.
3. Alderman, A. and Vivian, H. E.: A qualitative Test for Cement Aggregate Reaction, The Commonwealth of Australia—Journal of the Council of Scientific and Industrial Research, Volume 18, No. 4, November, 1945.
4. Wartz, J. G.: Aluminium Treated Concrete Cuts Building Costs, Construction Methods, Volume 8, No. 3, March, 1946.
5. Building Regulation for Reinforced Concrete—Journal of the American Concrete Institute, Volume 16, No. 6, June, 1945.
6. Concrete Containing Air-Entraining Agents—Reprinted from Journal of the American Concrete Institute, June, 1944.
7. Recommended Practice for the Design of Concrete Mixes. American Concrete Institute, A. C. I. Mix Design Standards 613, Detroit 2, Michigan, 1944.
8. Mercer, L. B.: Concrete Expansion Due to Alkali Aggregate Reaction—The Commonwealth Engineer, Volume 33, No. 11, June, 1946.
9. Purandase, N. N.: The Effects of Revibration on the Strength of Concrete—Indian Concrete Journal, Volume XX, No. 9, September, 1946.
10. Peabody, D: The Design of Reinforced Concrete Structures—John Wiley and Sons, Inc. New York, 1946.
11. Mensoh, L. J.: Advantages of High Strength Steel as Reinforcement for Concrete—Engineering Journal of Canada, Volume 29, No. 4, April, 1946.
12. High Alumina Cement in Hot Climates—Concrete and Constructional Engineering, Vol. XLI, No. 8, August 1946.
13. Some Development in Design Theories—Concrete and Constructional Engineering, Vol. XLI, No. 7, July, 1946.
14. Some development in Design Theories—Concrete and Constructional Engineering, Vol. XLI, No. 7, July, 1946.
15. Curing Concrete with Protective Coatings—Concrete and Constructional Engineering, Vol. XLI, No. 9, September, 1946.

(iii) CONSTRUCTION

Recent Literature:

1. Conn Creek Dam to Provide Napa Valley with Water.—Extract from Western Construction News, October, 1945.

2. Shasta Dam and its Builders—Pacific Constructors, Inc. April, 1945.
3. Pauls, A. L. : Fort Loudoun Dam—Proceedings of the American Society of Civil Engineers, Volume 72, No. 10, December, 1946.
4. Taylor, T. F. : Douglas Dam—Proceedings of the American Society of Civil Engineers, Volume 72, No. 10, December, 1946.

(iv) INVESTIGATIONS

Recent Literature.

Investigation of Crack Development in the Concrete in Parkar Dam—U. S. Department of the Interior, Bureau of Reclamation Laboratory Report No. Ce 31, Denver, Colorado, January 20, 1941.

The Year's Work

(i) INVESTIGATION

No contribution was made at the 1947 Research Committee Meeting under this sub-head.

(ii) DESIGN

The following items were discussed at the 1947 Research Committee Meeting :—

- (1) Stress analysis by method of least work.
- (2) Experimental stress analysis—Photo-elastic methods.
- (3) Variation of co-efficient of discharge with reservoir levels for full opening of sluices in Krishnarajasagar dam.

(1) STRESS ANALYSIS BY METHOD OF LEAST WORK ⁽¹⁾

Calculation of stresses in the non-overflow and overflow sections of the proposed Tungabhadra Dam were originally made by using the well-known methods given by Unwin and Bouvier. The designs so obtained were analysed by the method of Least Work ⁽²⁾. A few stresses obtained by this method and by the classical methods of Unwin and Bouvier are furnished in Table 4C. 1.

TABLE 4C.1

*Tungabhadra Dam—Overflow and non-overflow sections
maximum stresses obtained by different methods of calculation*

Description	Non-overflow section			Overflow section		Method least of Work
	Bouvier's method	Unwin's method	Method of least Work	Bouvier's methods	Unwin's method	
Maximum tension at Upstream face in tons of square feet	Nil.	Nil.	2.759	0.45	..	—0.312
Maximum compression Downstream face in tons per square foot	8.57	10.21	6.87	3.70	..	—2.79

N.B.—Weight of masonry = 147 lb per cubic foot. Maximum water level = +1,632.5
Weight of water = 62.43 lb per cubic foot. Level of bottom section = +1,510.0
 $m = 8$. K for non-overflow section = 0.66 and K for overflow section = 0.8

(1) Irrigation Research Station, Poondi, Madras, Annual Report 1946, page 58.

(2) Jacobson, Transactions of the American Society of Civil Engineers, 1932.

The results showed that the stresses obtained by the Method of Least Work were different from those obtained by the classical methods. While there were certain good points about the former, there were many weak points as well. The entire calculations were based on the "Equivalent Profile" which determined "K" in the formulae involved in the Method of Least Work. This, however, was not uniquely defined and had to be obtained by trial and error. In the present calculations the values taken were 0.66 and 0.8 respectively for the non-overflow and overflow sections. These were obtained by trial and error; by successive approximations more correct values could probably be obtained and for each set of values there would be a different set of stress analysis. The triangular equivalent profile so chosen (determined by K) might overlap the practical profile for certain regions and extend beyond its boundaries while in other regions it might not cover the practical profile at all. Again any material added at the top in the practical profile would result in the widening of the base of the "equivalent triangle" and the effect produced by balancing would have an adverse value in the matter of estimation of stress. The special advantage of the classical method is that it will take into consideration the exact shape of the profile. Yet another objection to the method is according to Hanna and Kennedy⁽³⁾, that "it fails to include the effects of the deformations of the foundation plane. Much further experimental and mathematical work remains to be done before a satisfactory solution of the method can be obtained. In the meantime the design should adhere to the classical theory."

(2) EXPERIMENTAL STRESS ANALYSIS—PHOTO-ELASTIC METHODS ⁽⁴⁾

The methods for determining the fringe constant of photo-elastic material were perfected.

The principles of flexure, compression and tension were used. The calibration was done by observing and counting monochromatic fringes with changes in stress. The variations of fringes in case of flexure and compression are shown in figure—

A case of an arch and a beam both loaded in the centre were analysed. Attempts were made to use gelatine models for stress investigation of earthen dams. Stress fringes observed in a rectangular gelatine mass are shown in Figure 4C.1.

⁽³⁾ Hanna and Kennedy, Design of Dams, page 86.

⁽⁴⁾ The Punjab Irrigation Research Institute, Lahore, Annual Report, 1946, page 19.

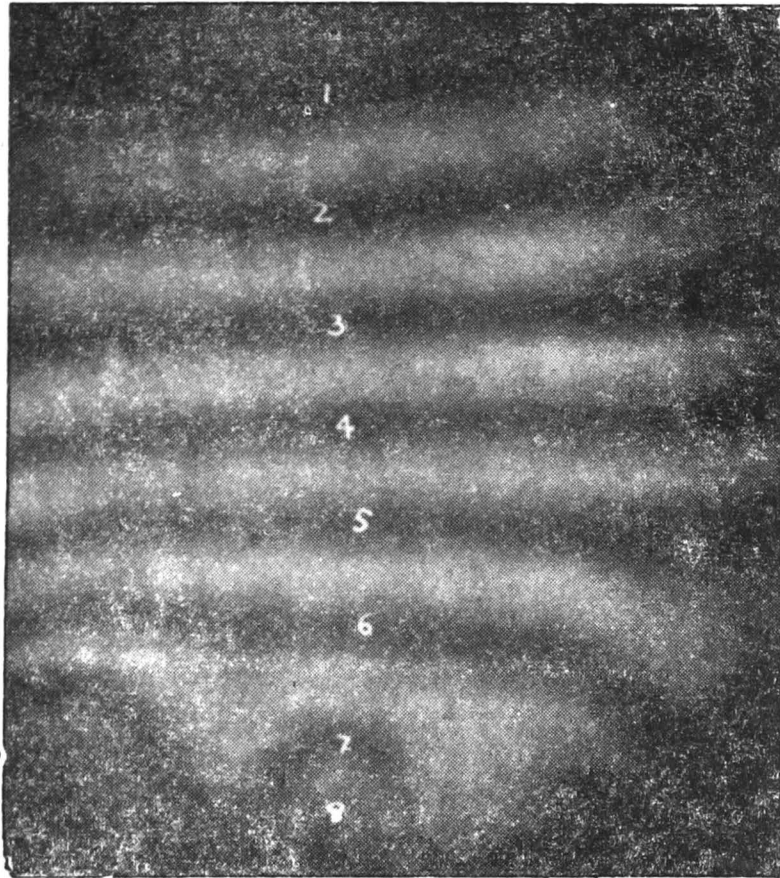


Figure 4C.1 : *Showing stress fringes observed in a rectangular gelatine mass*

(3) VARIATION OF CO-EFFICIENT OF DISCHARGE WITH RESERVOIR LEVELS FOR FULL OPENING OF SLUICES IN KRISHNARAJASAGAR DAM⁽⁵⁾

ABSTRACT

Presents a study of the variation of the co-efficient of discharge with reservoir levels for full openings of gates in the case of three sets of sluices in the dam.

The Krishnarajasagar dam has a full reservoir storage depth of 124 feet above bed of the river, Cauvery. Experiments have been conducted to determine the co-efficients of discharge of the various sets of sluices in the dam. They were determined by letting down water at different heights of openings of gates at different reservoir levels for known intervals of time. The discharge was collected in a basin formed below the dam and its volume was determined accurately after making allowances for losses due to seepage, evaporation *etc.* Experiments were conducted during a period of eight summer seasons.

The co-efficients of discharge so obtained have been tabulated in the appendices issued to the explanatory notes to the rules of regulation, Krishnarajasagar.

The results of the experiments show that the co-efficient of discharge varies (a) with the reservoir levels for any particular vent opening (b) with different openings of gates for the same reservoir level, and (c) on the nature of the bell mouth given in front of the sluices.

A study of the variation of the co-efficient of discharges with reservoir levels for full openings of gates in the case of three sets of sluices in the dam has been made in this paper.

One set of sluices are three in number each six feet by 15 feet and are placed with their sills at 50 feet above the bed of the river (+50 sluices). They have been fitted up with Stoney's patented shutters moving on rollers. The gates are 6.5 feet from the front face of the dam and have no bell mouth entry.

There are sixteen opening each 10 feet by 20 feet placed at 80 feet above the bed of the river (+80 sluices). These also have Stoney's patented shutters placed 6.40 feet from the front face of the dam. Bell mouth entry has been given at the sides and top, the angle of divergence being 100°. The bottom is flat.

The gates placed at +103 feet are 48 in number each being ten feet by eight feet. These have waggon type cast iron shutters placed 4 $\frac{3}{8}$ feet from the front face of the dam. Bell mouth entry has been given on all the sides that is at the bottom, top and sides. The angle of divergence is 6 $\frac{1}{4}$ °.

A comparison of the variation of the co-efficients in the three sets of sluices have been made only for their full openings when the gates are raised clear of water. A comparison for half or other partial openings is not possible as the effect of bell mouth at the top and bottom is more pronounced in the case of smaller heights of openings than in case of bigger openings.

(⁵) Hydraulic Research Station, Krishnarajasagar, Annual Report, 1946 pages 28 to 34.

Similarly a comparison of the variation of co-efficients is only possible with the head of water measured from the top of the vent opening to the free surface of water in the reservoir. A comparison with the head of water measured from the sill or from the centre of the openings would not be correct as in either case, the operating head would include the depth of flow equal to the height of vent opening which is not effective in increasing the co-efficient of discharge. For example in the reservoir of 50 feet depth, the operating head if measured from the sill would be the same both for a vent openings say 5 inches \times 20 inches or if measured from the centres of the openings. The difference in the operating heads between the two vent openings would be only five feet, but the co-efficient of discharge realised in the smaller openings is very much more than in the bigger vent. When the gate is raised clear of water the percentage of the area of issuing jet to area of vent opening is very much more in the case of the smaller opening than in the bigger opening. As this percentage of area of discharging jet to the vent area increases as the head of water over the vent opening increases, this has been taken for purposes of comparison of co-efficients realised in the three sets of gates.

Table 4C.2 shows how, for the similar depth of water above sluice openings the co-efficients of discharge vary with the nature of the bell mouth entry.

For $12\frac{1}{2}$ feet depth of reservoir level over the vent opening co-efficient $C\sqrt{2g}$ for the +50 sluices where no bell mouth entry is given is only 4.68, or +80 sluices with bell mouth entries at top and sides is 5.80 and for +103 sluices where bell mouth entries have been given on both the sides top, and bottom it is 6.48.

The relationship between the reservoir levels and the values of $C\sqrt{2g}$ can be expressed in the form of a simple parabolic equation of the type

$$Y = a_0 + a_1X - a_2X^2 \quad \dots \quad (4C.1).$$

In the case of the +50 sluices, Table 4C.3, with no bell mouth entries the equation works out to

$$Y = 5.029 + 0.0785X - 0.013X^2 \quad (4C.2).$$

With origins for the X and Y co-ordinates at $X = 92.50$ (reservoir level) and $Y = 5.03$ which is the value of $C\sqrt{2g}$ for the corresponding reservoir level of 92.50.

The equation which can be fitted up for the +80 sluices, Table 4C.4, which have bell mouth entries at sides and top only is

$$Y = 5.81 + 0.169X - 0.007X^2 \quad \dots \quad (4C.3).$$

and for the +103 sluices, Table 4C.5, which have bell mouth on all the sides, it is,

TABLE 4C. 2

Statement of co-efficient of discharge $C\sqrt{2g}$ for various depths of water above sluice openings.

Depth of water above sluice opening	Maximum F. R. L. +124 feet Sill level of sluice. +50 feet size of vents. 6' x 15' (With no bell mouth entry)	+124 feet +80 feet 10' x 20' (Bell mouth entry at sides and top)	+124 feet +103 feet 10' x 8'. (Bell mouth entry at sides; top and bottom)
1.50	—	—	4.67
3.50	—	—	4.95
3.00	—	4.74	—
3.50	—	—	5.23
4.50	—	—	5.51
5.00	—	5.06	—
5.50	—	—	5.77
6.50	—	—	6.00
7.00	—	5.29	—
7.50	4.49	—	6.18
8.50	—	—	6.31
9.00	—	5.48	—
9.50	—	—	6.40
10.50	—	—	6.46
11.00	—	5.65	—
11.50	—	—	6.48
12.50	4.68	—	6.48
13.00	—	5.81	—
15.00	—	5.96	—
17.00	—	6.10	—
17.50	4.83	—	—
19.00	—	6.25	—
21.00	—	6.38	—
22.50	4.94	—	—
23.00	—	6.52	—
27.50	5.03	—	—
32.50	5.09	—	—
37.50	5.12	—	—
42.50	5.14	—	—
45—50	5.14	—	—

$$Y = 5.99 + 0.187X - 0.017X^2 \dots \dots (4C.4).$$

Similar equations are also proposed to be worked out for the variation of co-efficients of discharge for different heights of openings of gates, for any particular reservoir level.

TABLE 4C. 3

Curve-fitting to express the relationship between the reservoir levels and the value of $C \sqrt{2g} Y$ where C is the sluice co-efficient for the Plus 50 sluices

Reservoir levels in feet	Value of $C \sqrt{2g} Y$	Reservoir level Approximate mid values X	Value of X with the change of orifice	X^2	X^3	X^4	Xy	X^2y
70-75	4.49	72.5	-4	16	-64	256	-17.96	71.84
75-80	4.68	77.5	-3	9	-27	81	-14.04	42.12
80-85	4.83	82.5	-2	4	-8	16	-9.66	19.32
85-90	4.94	87.5	-1	1	1	1	-9.94	4.96
90-95	5.03	92.5	0	0	0	0	0	0
95-100	5.09	97.5	1	1	1	1	5.09	5.09
100-105	5.12	102.5	2	4	8	16	10.24	20.48
105-110	5.14	107.5	3	9	27	81	15.42	46.26
110-113	5.14	112.5	4	16	64	256	20.56	82.24
Total	44.46	..	0	60	0	708	4.71	292.29

Assuming a simple parabolic relation between the variables, we can take the equation of the quadratic parabola in the form $Y = a + bX + cX^2$ where the constants a, b, c , are to be determined with the help of the following equations:—

$$\begin{array}{rclcl} 44.46 & 9a & 60c & .. & (i) \\ 4.71 & 60b & & .. & (ii) \\ 292.29 & 60a & 708c & .. & (iii) \end{array}$$

Solving these in the usual way, we obtain the following values of the constants:—

$$a \ 5.029 \quad b \ 0.0785 \quad c \ 0.013$$

Hence the curve of best fit connecting the variables is given by the equation

$$Y = 5.029 + 0.0785 X - 0.013 X^2.$$

TABLE 4C. 4

Curve-fitting to express the relationship between the reservoir levels and the value of C where C is the sluice co-efficient for the plus 80 sluices

Reservoir levels in feet	Value of C Y	Reservoir level approximate midvalues X	Value of X with the change of orifice	X ²	X ³	X ⁴	XY	X ² Y
102—104	4.74	103	—5	25	—125	625	—23.70	118.50
104—106	5.06	105	—4	16	—64	256	—20.24	80.96
106—108	5.29	107	—3	9	—27	81	—15.87	47.61
108—110	5.48	109	—2	4	—8	16	—10.96	21.92
110—112	5.65	111	—1	1	—1	1	—5.65	5.65
112—114	5.81	113	0	0	0	0	0	0
114—116	5.96	115	1	1	1	1	5.96	5.96
116—118	6.10	117	2	4	8	16	12.20	24.40
118—120	6.25	119	3	9	27	81	18.75	56.25
120—122	6.38	121	4	16	64	256	25.52	102.08
122—124	6.52	123	5	25	125	625	32.60	163.00
Total	63.24	..	0	110	0	1958	18.61	626.00

Assuming a simple parabolic relation between the variables, and following the method of least squares the curve of best fit connecting the variables is given by the equation $Y = 5.81 + 0.169X - .007 X^2$.

TABLE 4C. 5

Curve-fitting to express the relationship between the reservoir levels and the value of C where C is the sluice co-efficient for the plus 103 sluices

Reservoir levels in feet	Value of C Y	Reservoir level approximate midvalues X	Value of X with the change of orifice	X ²	X ³	X ⁴	XY	X ² Y
112.3—113	4.67	112.5	—5	25	—125	625	—23.35	116.75
113—114	4.95	113.5	—4	16	—64	256	—19.80	79.20
114—115	5.23	114.5	—3	9	—27	81	—15.69	47.07
115—116	5.51	115.5	—2	4	—8	16	—11.02	22.04
116—117	5.77	116.6	—1	1	—1	1	—5.77	5.77
117—118	6.00	117.5	0	0	0	0	0	0
118—119	6.18	118.5	1	1	1	1	6.18	6.18
119—120	6.31	119.5	2	4	8	16	12.62	25.24
120—121	6.40	120.5	3	9	27	81	19.20	57.60
121—122	6.46	121.5	4	16	64	256	25.84	103.36
122—123	6.48	122.5	5	25	125	625	32.40	162.00
123—124	6.48	123.5	6	36	216	1296	38.88	233.08

Assuming simple parabolic relation between the variables and following the method of least squares, the curve of best fit connecting the variables is given by the equation $Y = 5.99 + 0.187 X - .017 X^2$.

DISCUSSION BY THE RESEARCH COMMITTEE

MR. KUTTIAMMU introduced item (1) and said that he had nothing to add to what had been given in the report.

MR. BLENCHE asked how the method was applied. Whether it was trial load analysis.

MR. KUTTIAMMU said it was not by trial load analysis, but by method of least work as explained in American Society of Civil Engineers Proceedings Vol. 132.

DR. VAIDHIANATHAN introduced item (2) and said that this subject was first introduced last year and the speaker could just show then what a photo-elastic picture was like.

In the first instance he remarked that there was little that had been done this year. First they wanted to study experiments on earthen dams, for this work they wanted a substance which should not be very hard, which must be transparent, and which must give the stresses and strains in a fairly accurate manner. The material in which a section of the earthen dam was finally constructed was crude gelatine obtained from the bazar and treated for a period of five to six months. It was black in the beginning and at the end of the treatment it was a fairly transparent stuff. Any material other than gelatine would not show anything under its own weight whereas for dams made of masonry one could use any transparent material, for earthen dams the choice was restricted.

He showed a picture of stress diagram of the earthen dam obtained by photo-elastic method and said that the picture gave an insight, into the stress distribution in an earthen dam, acting under its own weight. The magnitude of the stress could be calculated from another experiment. He explained the picture, and said that there were six fringes. A particular fringe had a particular value, another had got another. To interpolate middle values between fringes was difficult. Closer fringes could be got by using white light, and using colour photography. He then exhibited another set of pictures showing the photo-elastic study of the stress distribution in a dam and its foundations and said that the technique in this case had been changed entirely.

The possibility of using bakelite or any other material, e.g. glass, instead of gelatine was discussed at length. DR. VAIDHIANATHAN spoke of the advantage of gelatine over bakelite for this particular case.

THE SECRETARY enquired what the dark spot under the foundation meant.

DR. VAIDHIANATHAN said that he did not know how it occurred. He, however, said that it might be due to a number of maximum stress concentrations at those points which caused that.

MR. DILDAR HUSSAIN enquired whether he had adopted any particular height.

DR. VAIDHIANATHAN said that only shape of the dam had been taken into account and the first thing was to get a picture so that one could prove any model on a hypothetical basis.

RAO BAHADUR VENKATACHARYA asked if the photograph was to full scale or drawn to any scale and what was the thickness of the model across.

DR. VAIDHIANATHAN said that the picture showed the full size of the model. It did not represent any particular dam. The cross thickness of model was about $2\frac{1}{2}$ inches to three inches.

DR. RAJU wished to see a sample of gelatine and asked whether he had made sure that there was not any initial stress—whether he had done any annealing.

DR. VAIDHIANATHAN said that there was always going to be some initial stress due to its own weight. The thing that one should ensure himself was that there was no cross polarisation.

Next DR. VAIDHIANATHAN showed another picture got from a model in bakelite. He said that in this model one could not get stresses due to its own weight. It was an arch with 32.5 lb. weight acting on its middle point. The analysis sheet was also shown, from which it was explained that the fringe value for that particular bakelite model was 182 lb. per square inch.

THE PRESIDENT enquired whether the previous cases could also be done with bakelite.

DR. VAIDHIANATHAN said that bakelite could be used in the case of dams also to get stresses due to superimposed loads and not for stresses due to its own weight.

While it would be possible to use bakelite for arches, it was contended that it might be possible to use for dams as well something in between gelatine and bakelite.

It was possible to work on gelatine in cold countries whereas in a hot country it was very difficult; with great difficulty these experiments had been attempted in the cold months of December in Lahore. It was further stressed that arrangements would have to be made to keep the temperature constant to suit the preservation of gelatine for the special work.

MR. BLENCH explained that photo-elastic work was now well established. His Institute was interested in it because of the possibility of the Province taking up a high dam programme. The method was useful in showing how stress concentrations occurred in shapes to which mathematical analysis could not be applied readily *e.g.* galleries in dams, dam of one Young's modulus built on rock of another. So far it was limited to two dimensions and to purely elastic material; however, if its limitations were understood, it served a useful purpose, and was liked by the engineer as it substituted pictures for calculations that could be understood and checked by experts only. The lines in the photos shown by Dr. Vaidhianathan represented loci of equal shear stress, and did not indicate principal stresses.

RAI BAHADUR HANDA said that he was very much impressed with the use of photo-elastic methods by the Bureau of Reclamation while he was there for six weeks. In reply to questions, he learnt from them with very great satisfaction the qualities of the system. It was possible by means of these photo-elastic experiments to verify the stresses if they were correct quantitatively and qualitatively. He hoped work would be greatly facilitated by the adoption of these photo-elastic methods. This enabled the mathematical treatment

of the subject to be visually seen by an engineer. Americans were saying that they, by this method, were able to get results 100 per cent. correct qualitatively and within ten per cent. correct quantitatively.

DR. VAIDHIANATHAN said that among instruments they had got Bably Compensators. The other instruments ordered for could not be obtained due to dollar difficulties. If these methods were sure to give results within ten per cent. it was worthwhile trying to get the remaining instruments also.

DR. RAJU said that Hyderabad was getting a $8\frac{1}{2}$ inches aperture polariscope for their laboratories. The difficulty seemed to be about the material for the models. He saw the material used in the Indian Institution of Science where even the suppliers did not exactly state its nature, wishing evidently to keep it a secret.

DR. VAIDHIANATHAN said that he would be very grateful for further information on the subject. Regarding material, there were many materials in the market. Any transparent homogeneous material could be used, as a matter of fact.

The first time he got a piece of resin last time, he got it from Bangalore. He sent it to America to find out what it was. Any number of sheets could be found and any number of them could be polarised. Transparent bakelite was one of common substances used.

MR. GOVINDA RAO introduced item (3) and said that the data collected was unique in as much as experiments had been done with the actual services and the reservoir, and not on models. Experiments had been made during eight summer seasons.

RAO BAHADUR VENKATACHARYA remarked that the experiments had been made jointly with Madras.

THE PRESIDENT said that there were useful indications from these photo-elastic studies, and he hoped that these investigations would be pursued to see how far it could be useful in further stages. He thanked the members participating in the discussion.

(iii) MATERIALS

The following items were discussed at the 1947 Research Committee Meeting.

- (1) Foundation rock of the Ramapadasagar Dam.
- (2) A note on *Surkhi* mortar by M. C. Sampathiengar.
- (3) Sweating from the Krishnarajasagar Dam.

(1) FOUNDATION ROCK OF THE RAMAPADASAGAR DAM(*)

The thermal properties of one sample of the foundation rock at the Ramapadasagar Dam site were determined in the Physics and soil Mechanics Laboratory. A disc of the rock was cut and ground to uniform thickness from the core sample obtained. Using this disc the specific heat, co-efficient of heat conductivity and co-efficient of diffusivity were experimentally determined. The values obtained are given below :—

Specific heat—0.19 cal./gm./°C=0.19, B. Tu./lbs./°F.

(*) Irrigation Research Station Poondi, Madras, Annual Report 1946, page 59.

Coefficient of thermal conductivity = $0.003 \text{ cal./sec./sq.cm./}^{\circ}\text{C} = 0.88 \text{ B.T.U./hr./ft./}^{\circ}\text{F.}$

Coefficient of diffusivity $0.0063 \text{ cm}^2/\text{sec.} = 0.024 \text{ ft.}^2/\text{hr.}$

Density— $3.04 \text{ gm./cc.} = 190 \text{ lb./c.ft.}$

Being the only sample tested, these values could not of course be taken as representative of the entire body of the foundation rock; they just indicated the order of the respective values generally.

A NOTE ON SURKHI MORTAR

by

Mr. M. C. Sampathiengar, B. E.,

Executive Engineer, Krishnarajasagar Division.

ABSTRACT

Presents results obtained from tests carried out on different proportions of mortar prepared out of *Surkhi*.

EVOLUTION OF SURKHI MORTAR

It is the common belief that in situation where great strength and durability are called for, cement mortar only has to be used. But in Mysore State, for all important masonry works such as the construction of the Vanivilas Sagar Dam the Marikanave Dam, the Krishnarajasagar Dam, the Hirebhasgar Dam and so on, the State Engineers have used no cement mortar but employed a kind of mortar called 'Surkhi Mortar' obtained by intimately mixing burnt hydraulic lime with burnt brick powder. This mortar is dull red in colour and an un-experienced eye is likely to mistake it for mud-mortar.

SURKHI MORTAR STARTED AT VANI VILAS SAGAR

Mysore State started for the first time on an extensive scale, the use of *Surkhi*-mortar on its dam construction at Marikanave called Vanivilas Sagar situated in the North eastern district of the State in the year 1899. The height of the dam above the river bed was 142 feet. Normally, cement mortar would have been thought of for the construction. But the Engineers responsible for its construction decided not to use cement because :—

1. Cement had to be obtained from either England or Germany as its manufacture in India was still in its infancy. The supply of mortar materials from foreign countries for dam construction could never be depended upon and consequently no programme of construction could be kept up. Not working upto programme would have spelt disaster on the works, as floods would have over taken the construction work causing heavy damages and prolongation of the works and making it costly too.

2. The sand available in the river was not of good quality for cement mortar work.

3. The combining of cement and water for hydration is an exothermic reaction and the heat thus developed if not allowed to escape might cause serious complications.

Investigations were, therefore, started to prepare a suitable hydraulic mortar from the lime stone available locally. Different samples of *kankar* available from quarries within a radius of six miles were tested and the best *kankar* was found to be that which was got from a quarry about six furlongs from the work. On an analysis it was found to contain the following ingredients :—

	Per cent.
Moisture	0.400
Insoluble matter in hydrochloric acid	9.300
Ferric oxide and alumina	2.600
Lime	49.448
Magnesia	0.216
Carbonic anhydride	37.000
Sulphuric anhydride	Traces
Allies	1.256
	100.000

This lime being only slightly hydraulic, it became necessary to add *surkhi* to make the mortar hydraulic. After elaborate tests conducted at site, details of which are given in Tables 4C.6 and 4C.7, it was found that the best mortar was obtained with one part quick lime unslaked and two parts *surkhi* slightly under burnt. But as regards strength, the difference between that mixture and one lime three *surkhi* was so trifling that it was decided to use for dam construction works, a mixture of one unslaked burnt lime, with three parts by volume of powdered *surkhi* fairly well burnt and ground in power mills. This proportion was fixed on account of its cheapness.

The mortar as used for the dam were made into briquettes everyday and tested after 14 days immersion in water. The briquettes in all cases were immersed 24 hours after moulding. The results obtained during the month of June 1899 are found in Table 4C.8. They indicate that there would be ample margin of safety even after 14 days' setting as the maximum intensity of pressure allowed in the dam design is only 8.024 tons per square foot equivalent to 125 lb. per square inch nearly, and the strength developed in the mortar samples tested was on an average above six cwts. per square inch. Further tests of the mortar used on dam construction were also continued and they too point to the same conclusion. These results are found in Table 4C.9. As an experiment, during the months of November and December 1899, slaked lime in place of unslaked lime was used, and the tests showed a deterioration in the strength of the mortar thereby indicating that fresh unslaked lime only has to be used for the *surkhi* mortar manufacture.

TABLE 4C. 6

Showing the results of tests conducted of mortar with different proportions of Lime Kankar with under burnt Surkhi at Vanivilas Sagar Works—Mysore State.

Composition of mortar	After 1 hour		24 hours		3 days		7 days		14 days		22 days		3 months		6 months		Percent age of water
	T.S.	C.S.	T.S.	C.S.	T.S.	C.S.	T.S.	C.S.	T.S.	C.S.	T.S.	C.S.	T.S.	C.S.	T.S.	C.S.	
1 Lime, 1½ Sand and 1½ Surkhi.	55	224
1 Lime and 1 Surkhi	5	52	14	65	33	197	48	378	81	694	138	1,281	154	1,456	50
1 Lime and 1½ Surkhi	9½	..	20	60	57	318	82	659	123	986	153	1,280	44
1 Lime and 2 Surkhi	19	..	17	..	35	106	33	197	136	886	166	1,282	289	2,240	38
1 Lime and 3 Surkhi	17	..	11	..	21	113	40	247	125	752	157	918	182	1,848	190	2500	33
	After 2 hours		Lime kankar with ordinary surkhi lb.														
1 Lime 1½ Sand and 1½ Surkhi	45	220
1 Lime and 1 Surkhi	75	..	17	35	45	235	34	55	90	896	151	1,243	183	2,464	50
1 Lime and 1½ Surkhi	17	..	21	51	41	110	61	359	51	229	70	340	44
1 Lime and 2 Surkhi	9	..	10½	..	18	..	29	107	85	512	70	336	230	1,400	38
1 Lime and 3 Surkhi	5	..	15	..	12	..	58	444‡	110	855	65‡	336‡	172	1,736

‡ This is the best
 N.B.;—T. S. tensile strength (Breaking) in lb per square inch. C. S. Crushing strength (Breaking) in lb per square inch

TABLE 4C. 7

Showing the strengths of mortars composed of various proportions of Lime, Sand and surkhi Vanivilas Sagar Works, Mysore State.

No.	Ingredients	After 7 days		After 14 days		After 1 month		After 3 months		Percentage of water	Remarks
		T.S.	C.S.	T.S.	C.S.	T.S.	C.S.	T.S.	C.S.		
1	1 Lime and 1 Surkhi	44	840	66	1,230	95	2,000	131	2,390	54	
2	1 Lime and 2 Surkhi	48	812	62	1,372	104	1,792	123	2,240	50	
3	1 Lime and 3 Surkhi	47	504	90	1,196	122	1,900	149	2,632	43	
4	1 Lime and 4 Surkhi	55	850	76	1,568	130	2,240	190	3,248	45	
5	1 Lime and 5 Surkhi	67	840	96	1,288	128	2,240	191	3,024		
6	1 Lime and 6 Surkhi	56	700	84	1,176	125	1,900	163	2,880		
7	1 Lime and 1 Sand	8	278	10	336	20	500				Not continued as mortar was inferior
8	1 Lime and 2 Sand	6	224	12	300	14	460				
9	1 Lime and 3 Sand	10	336	21	448	27	500				
10	1 Lime and 1½ sand and 1½ Surkhi ..	52	868	79	1,344	108	1,900	160	2,800		
11	1 Lime and 1 sand and 2 Surkhi	52	933	93	1,500	107	1,650	166	2,744		
12	1 Lime and 2 sand and 1 Surkhi	37	600	62	836	77	2,240	119	2,184		

TABLE 4C. 8

Showing the results of mortar tests after 14 days conducted in June 1899 at Vanivilas Sagar Works, Mysore State.

Date	Tensile Strength in lb per square inch	Crushing Strength in lb per square inch	Remarks
3rd	102	7 Cwt.	
4th	80	6½ "	
7th	100	10½ "	
8th	92	10½ "	
9th	80	5 "	
10th	79½	4½ "	
14th	100	7 "	
16th	102	6½ "	
17th	85	5½ "	
18th	108	5½ "	
22nd	Very bad
23rd	111	7½ "	
25th	101	7½ "	
26th	100	7½ "	

TABLE 4C-9

Showing the average both for Tensile and Crushing Strengths for each month of Surkhi Mortar (tested after 14 days) used at Vanivilas Sagar Works (Mysore States).

Month	Tensile Strength in lb per square inch	Crushing Strength in lb per 1.5 square inches	
September 1899	81	896	
October 1899	92	896	
November 1899	79	784	} Unslaked lime
December 1899	60	672	
January 1900	87	900	
February 1900	80	784	
March 1900	70	896	
April 1900	75	1,120	
May 1900	72	950	
June 1900	80	1,230	
July 1900	88	1,230	
August 1900	78	880	

Tests were also made in Vanivilas Sagar Dam Works, to ascertain whether *surkhi* mortar really deteriorated by being kept for two or three days before use and the results have revealed that it does not deteriorate if kept even for five days when kept damp. The results of these tests are to be found in Table 4C.10. Even though these tests indicate no deterioration in the strength of mortar, in practical construction, mortar older than 24 hours is not generally used as it becomes difficult to break the mortar lumps that have started setting, and convert it into a smooth paste free from lumps for use on works.

TABLE 4C.10

Showing the strength of surkhi mortar kept unused upto six days Vanivilas Sagar Works (Mysore State).

Age	After 7 days		After 14 days		After 1 month	
	T.S.	C.S.	T.S.	C.S.	T.S.	C.S.
Fresh	49	616	75	1,064	106	1,568
24 hours old	42	560	76	896	87	1,288
2 days	55	896	80	1,288
3 days	33	560	55	896	80	1,288
4 days	30	504	52	784	80	1,288
5 days	29	504	58	784	79	1,288
6 days	20	448	49	672	64	1,176

T.S.—Tensile strength in lb per square inch, and

C.S.—Crushing strength in lb per 1.5 square inches

KRISHNARAJASAGAR SURKHI MORTAR

The pioneer work done by the Vanivilas Sagar Engineers in the manufacture of *surkhi* mortar was pursued vigorously by Krishnarajasagar Engineers. Ever since the construction work at this dam started, various tests with different proportions and ingredients have been conducted. The lines on which the tests were proceeded resemble more or less with those conducted at Vanivilas Sagar. They are :—

(a) Determination of the best hydraulic lime available in the neighbourhood.

For this purpose, natural lime stone from four different quarries were obtained, burnt, slaked briquettes made and tested upto 12 weeks' curing for determining the breaking tensile and compressive stresses. The results of these tests are given in Table 4C.11. They indicate that the lime stone obtained from a quarry called 'Baby Quarry' was the best and 'Hebbal' and 'Undavady' quarries stones were also fair.

(b) Determination of the best proportion of *surkhi* and sand to be added to improve the hydraulicity of the mortar. Experiments were conducted with briquettes made of unslaked burnt lime mixed and ground with different proportions of *surkhi* up to six parts by volume and also with mixture containing an admixture of both *surkhi* and sand in different proportions, Tables 4C.12 and 4C.13.

These tests have indicated that *surkhi* mortar with the addition of sand gains in compressive strength. The mortar of proportions one lime : two *surkhi* and one lime : three *surkhi* gives higher results than the other proportions. But these richer proportions are costlier and such rich-mortar is also liable to contain free unslaked particles of lime. Hence the proportion 1 : 4 is a sage proportion, satisfying all requirements. This is the proportion of the mortar used in Krishnarajasagar dam construction.

(c) Determination of the time of grinding :—

The *surkhi* mortar used in Vanivilas Sagar was manufactured thus. First a sufficient quantity of water was poured into the pan enough to bring the lime to the consistency of cream. Then 3½ cubic feet of lime was put in and ground well for seven minutes. Then 10½ cubic feet of *surkhi* was added, in buckets and also water to keep the mortar to the proper consistency. The mixture

TABLE 4C.11

Results of pure hydraulic lime briquette conducted at Krishnarajasagar

Name of Quarry	1st week		2nd week		3rd week		4th week		6th week		8th week		12th week	
	T.S.	C.S.	T.S.	C.S.	T.S.	C.S.	T.S.	C.S.	T.S.	C.S.	T.S.	C.S.	T.S.	C.S.
1. Hobbal	17	—	25	—	Not tested	Not tested	49	67	64	146	83	314	90	437
2. Undavady	17	—	35	—	56	—	64	112	82	168	112	280	125	382
3. Baby	31	—	50	—	30	235	87	302	109	359	120	426	129	483
4. Madanahally	10	—	24	—	38	—	42	34	45	101	56	157	66	246

TABLE 4C.12 Result of lime—Surkhi mortar of different proportion conducted at Krishnarajasagar

Lime—Surkhi		1st week		2nd week		3rd week		4th week		5th week		8th week		12th week	
		T.S.	C.S.	T.S.	C.S.	T.S.	C.S.	T.S.	C.S.	T.S.	C.S.	T.S.	C.S.	T.S.	C.S.
1	0	31	—	50	—	80	235	187	302	109	358	120	426	129	482
1	1	65	560	109	907	148	1,288	184	1,456	190	1,792	219	2,173	256	2,240
1	2	52	268	100	784	138	1,053	160	1,444	185	1,758	Not tested	Not tested	260	2,262
1	3	72	612	133	1,154	166	1,165	184	1,422	200	1,758	240	1,971	284	2,363
1	4	79	425	122	896	150	986	152	1,444	204	1,702	205	1,826	240	1,936
1	5	76	531	128	885	142	1,176	174	1,366	183	1,702	194	Not tested	234	1,886

T. S.—Tensile strength (Breaking) in pounds per square inch
 C. S.—Crushing strength (Breaking) in pounds per square inch

was then ground removed, the operation occupying half an hour. The output of mortar was found to be $11\frac{1}{2}$ cubic feet. The percentage of water used was about 35.

At Krishnarajasagar, experiments were conducted regarding the time of grinding. Mortar with 25 minutes, 30 minutes, 35 and 45 minutes grinding in power mills were tested up to 12 weeks curing and the results, (Table 4C.14) indicated that mortar of 25 minutes grinding was not strong and grinding after 35 minutes did not improve the quality of mortar. A suitable specification has been drawn up for the manufacture of *surkhi* mortar for dam construction and the same is found at the end. It will be seen from the specification that for the manufacture of *surkhi* mortar *surkhi* in the form of 1 to $1\frac{1}{2}$ cubes has been used at Krishnarajasagar, in place of powdered *surkhi* employed at Vanivilas Sagar. In the former case, the powdering of the brickbats is done in the mills only.

CONCLUSIONS

The use of *surkhi* mortar on big and important irrigation and hydro-electric works in Mysore State has come to stay, as it can be depended upon for its strength *etc.* A safe compressive stress of eight tons per square foot is being allowed generally in all structures where this kind of mortar is used. In the manufacture of this kind of mortar, care is taken to use only the best natural hydraulic lime stone available at or near the work. The lime stone is burnt at the works and used. Unslaked and as fresh as possible, the slaking being done before mixing *surkhi* in the mill itself. For *surkhi* well burnt bricks are only used, even though slightly under burnt bricks are found to give better results. The manufacture of the mortar is done by departmental agency to ensure quality and not entrusted to any contractor.

Mysore State has used this type of mortar for the last 45 years nearly. The best proportion of *surkhi* and lime to be used for the manufacture is determined by taking tests from samples of different proportions and adopting the suitable among them from point of strength, cost *etc.* Further research is necessary to find out what exactly should be the ingredients of the lime stone and *surkhi* brick that are used in the preparation of best mortar. This enables, anyone after chemically analysing the lime stone and burnt brick available near the works, to fix upon the proportions to be mixed to obtain the best results.

TABLE 4C.13

Results of mortar tests conducted at Krishnarajasagar.

Lime Surkhi Sand Mortar			1st week		2nd week		4th week		6th week		8th week		12th week	
Lime	Surkhi	Sand	T. S.	C. S.	T. S.	C. S.	T. S.	C. S.	T. S.	C. S.	T. S.	C. S.	T. S.	C. S.
1	4	0	79	426	122	896	152	1,445	204	2,702	205	1,826	240	1,938
1	3	1	66	614	102	794	160	1,358	192	1,638	210	1,820	226	2,016
1	2	2	49	210	66	364	91	610	110	796	126	950	152	1,120
1	1	3	33	200	56	336	82	588	110	760	130	910	156	1,100
1	0	4	19	56	34	98	53	168	64	224	71	266	75	324

TABLE 4C.14—Surkhi Mortar—1 Lime 4 Surkhi :—Ground in Power Mills from 25 to 45 Minutes

Time	1st week		2nd week		4th week		6th week		8th week		12th week	
	T. S.	C. S.	T. S.	C. S.	T. S.	C. S.	T. S.	C. S.	T. S.	C. S.	T. S.	C. S.
25 Minutes	53	225	91	500	128	896	152	1,120	171	1,232	202	1,500
30 Minutes	60	312	100	648	141	1,008	163	1,120	180	1,454	211	1,680
35 Minutes	69	714	123	972	163	1,344	195	1,710	213	1,904	231	2,040
40 Minutes	76	500	130	820	170	1,300	200	1,700	220	1,860	240	2,000

T. S. — Tensile strength (Breaking) in lb per square inch
 C. S.—Crushing strength (Breaking) in lb per square inch

(3) SWEATING FROM THE KRISHNARAJASAGAR DAM (7)

The Dam at Krishnarajasagar across the River Cauvery is a masonry dam of gravity type. The entire masonry is of granite stone in *surkhi* mortar. When water was commenced to be stored in the reservoir sweating on the rear face of the dam was observed. A systematic study of the water percolating through the rear face was taken up in the year 1929 and observations are being recorded from that year and onwards.

In the river bed portion of the dam a small drain was constructed for a length of 66 inches and divided into three sections. Water percolating down the rear face was allowed from these three sections to three cisterns respectively. This water flowing down in a definite time was collected in a bucket and measured in cubic inches. The time of collection in seconds, the time of observation, the reservoir level at that particular time, atmosphere temperature were all noted and tabulated. From the section of dam the sloping area of the rear face for the length and reservoir level as calculated and the sweating in cusecs per million square feet was deducted.

It has been observed with the results so far obtained that the value of sweating decreases with the age of construction and depends on the reservoir level, temperature and humidity of the atmosphere. As part of the sweating goes away in evaporation which has not been taken into account, whatever is collected is only the net amount of water that flows down the rear face after evaporation. The sweating has been observed to be high for high reservoir levels. It varies from 0.09 cusec in average for a reservoir range of +120 to 124 and to 0.004 cusec for a range of 95-100. The variation in sweating for different seasons of the same year for the same reservoir level is not much appreciable. Another fact of observation is that the sweating of the same reservoir level depends upon the period of high reservoir level prior to the day of observation. In one or two observations recorded, instead of the sweating showing a diminution with age for a particular reservoir level, an increase has been recorded owing to the long period of high reservoir level occurring prior to the day of observation.

DISCUSSION BY THE RESEARCH COMMITTEE

MR. KUTTAIMMU introduced item (1) and said that some stone blocks for use in the construction of the dam were tested for compression tests. They were taken from the cores of the foundation rock. The result of tests had been given for information.

THE PRESIDENT asked how these compared with the results of rocks used in other dams.

MR. KUTTAIMMU replied that he had not studied the thing comparatively.

MR. M. P. MATHRANI asked if any shear tests for the material had been made.

MR. KUTTIAMMU replied that tests had not been completed. In reply to Mr. M. P. Mathrani, MR. KUTTIAMMU intimated that a 100 ton Bucktons testing machine was in use.

(7) Hydraulic Research Station, Krishnarajasagar, Annual Report, 1946, pages 24 to 27.

THE PRESIDENT remarked that the strengths seemed rather low.

DEWAN BAHADUR GOVINDARAJA AYYANGAR explained that the testing was done in the laboratory at the College of Engineering, Guindy in a Bucktons machine. They were establishing a separate laboratory for the Public Works Department at Madras and a 300,000 lbs. Universal testing machine was coming to the site.

THE PRESIDENT asked whether they had made any test after soaking the specimens under water?

MR. KUTTIAMMU said that they had not done that so far. They had carried out a few experiments only. The laboratory would be coming into being in another three to four months.

THE PRESIDENT referring to the work done at Bhakra said that on some rock tests where a strength of three tons was obtained on isolated samples they got nine to ten tons per square inch *in situ*. In one or two cases where there were slight fissures, the results were exactly the same as on isolated blocks. Whether one could utilise these results on isolated blocks in design was a different matter. Since the tests *in situ* gave as much as three to four times the value of isolated samples—being something like triaxial strengths, he would recommend such tests to be made. He also suggested tests on soaked specimens.

DR. RAJU suggested tests on different sizes of samples with a view to estimating the possible strength in the undisturbed rock.

RAI BAHADUR HANDA said that in the Punjab they had selected a 400 ton capacity machine for their work and that was the one they should use. In the Bureau of Reclamation they use a still bigger one of one million lbs. capacity. Regarding the other question, *i.e.* making test about the strength of rock at site they had ordered 200 ton capacity jacks for the purpose.

DEWAN BAHADUR GOVINDARAJA AYYANGAR said how far the foundations act along with the dam must have been studied elsewhere, *e.g.*, with respect to Boulder or other dams.

THE PRESIDENT said that the difference between isolated samples and of rocks *in situ* would be very big. He gave illustrations of some of the experiments carried out by him. In 1942-43 they carried out lot of tests; they had dug out a tunnel about seven feet high and four to five feet wide. It was fine rock. They wanted the strength of the rock to be tested. Spaces 3 inches \times 3 inches, 4 inches \times 4 inches, 6 inches \times 6 inches, 12 inches \times 12 inches and various other sizes, at the top were chiselled out and covered by a layer of mortar. Pressure was applied at the top as well as at the bottom. In certain cases they did not go up to the yield point but they reversed the pressure and got deflection readings. They found that deflection of the mortar was greater than the deflection of the rock itself. In another set of experiments they made two baffles one at the bottom and one at the top and got the deflection of the rock direct without the use of any mortar.

Such experiments could give a relation between the strength of rock *in situ* and that of a cut out sample, it would be very useful for preliminary calculations

DEWAN BAHADUR GOVINDARAJA AYYANGAR said that the results given in Madras report were for Tungabhadra and not for Ramapadasagar. The maximum stress could not be more than ten tons. A few tests had been carried out in respect of Ramapadasagar dam also. The stones were taken out from the quarries, but they had not yet gone down 100 to 200 feet below the bed of the river. They were also doing chemical analysis of the rock.

MR. GOVINDA RAO introduced item (2) and said that *surkhi* mortar had been used in Mysore for three big dams, viz., Vanivilas dam 142 feet high, Krishnarajasagar dam 124 feet high and Hirebhasgar dam. To start with, they used *surkhi* mortar because cement was not available. With *surkhi* mortar there was not much expansion and contraction on account of variation in temperature and, therefore, both in Krishnarajasagar dam, 8,000 feet in length and Vanivilas dam, 4,000 feet in length, no provision was made either for contraction or expansion joints. He gave details of how the mortar was made and said that experiments had been conducted to ascertain whether the composition of mortar used in dams was sufficiently strong, and the results had been favourable. The proportion generally employed was one of lime to three of *surkhi* optimum but in dam construction 1 : 4 had been used. Experiments had also been conducted to find out the strength of the various compositions. Unlike cement they could take their own time in these constructions. The mortar could be used even after 72 hours after grinding though they tried to use it within 24 hours. Unslaked lime developed better strength than slaked lime. Experiments were now in progress to find out what would be the optimum content of clay in *surkhi* and hydraulic lime, which would develop best strength. They had taken up analysing the lime also.

THE PRESIDENT enquired what was the chemical composition of lime. Were they all hydraulic limes ?

MR. GOVINDA RAO said that they were all hydraulic limes and referred to the detailed analysis on page 612 of this report. In reply to Mr. H. P. Mathrani, MR. GOVINDA RAO said that no sand was used. Hydraulic lime and *surkhi* without the use of sand had been found to develop better strength.

MR. KARIM said that experience with different limes would be different.

MR. GOVINDA RAO said that was why they were carrying on further experiments to find out the optimum contents of lime and clay for best strength.

DR. HOON asked how the chemical compositions of various limes varied.

MR. GOVINDA RAO said that the clay contents of various quarries varied from 15 to 40 per cent.

DR. HOON asked whether they used lime along with *kankar* lime and *surkhi*. Had they tried to find out by any method what suitable proportion of *surkhi* had to be added to *kankar* lime, because from his experience of *kankar* lime, he found that it was very much dependent on the free lime content which was present in it ; for instance the *kankar* lime used in these experiments contained calcium carbonate and clay. Did they work out the ratio and quantity that would be suitable for any particular lime ?

MR. GOVINDA RAO referred him to Table 4C. 6. and said that the ratios Vanivilas Dam ; lime : *surkhi* : : 1 : 3

Krishnarajasagar Dam ; lime : *surkhi* : : 1 : 4
were fixed as a result of daily experiments.

MR. M. P. MATHRANI asked if they had tried lime plus *surkhi* plus sand mortars.

MR. GOVINDA RAO said that the strength of such mortars with various proportions of *surkhi* and sand were given in Table 4C. 7.

MR. COCHRANE observed that since a period of at least 14 days must elapse between the date on which the mortar was used in the work and the date on which the sample could be tested for strength there could be no real control over the quality of the mortar and no action could be taken (short of dismantling the work done) to set right any defects in composition.

MR. SAXENA said that he had an occasion to visit Mysore and his impression was that *surkhi* was dug out from the hills. It was very difficult to say whether it contained some parts of sand because they were not making any analysis of the *surkhi*. He was convinced that because of the existing sand contents, they did not consider any addition of sand suitable.

MR. GOVINDA RAO said that *surkhi* used in Mysore was not dug out earth, but powdered burnt bricks.

THE PRESIDENT asked what was meant by slaked and unslaked lime.

RAO BAHADUR MAYDEO enquired how did the cost of this mortar compare with cement.

The difference between slaked *kankar* lime and unslaked *kankar* lime was explained by MR. GOVINDA RAO. He also stated that according to the present local rates, the use of *surkhi* was 50 per cent. cheaper than that of cement.

THE PRESIDENT enquired if Mr. Govinda Rao followed Dr. Hoon's work presented to the last research committee meeting.

MR. GOVINDA RAO said that they had gone further than that. But they went the other way about. They did the analysis of *kankar* lime after burning and not from the quarry. There were certain advantages in doing so.

MR. BLENCH favoured full scale attempts to produce a uniform *kankar* lime. He quoted Hiwassee Dam of the TVA as an example where suitable aggregate and cement had permitted construction, without cooling arrangements, of a moderately high dam, and gave his opinion that a low-heat cement such as could be manufactured from *kankar* might be used for relatively high dams and save cooling arrangements. If *kankar* lime could be produced of fair uniformity then its strength could be increased greatly, on a manufacturing scale, by adding just sufficient puzzolanic material (brick dust being the most readily available) to neutralise the free calcium oxide. In a large mass concrete dam made with Portland type of cement with its high heat of hydration, the secondary stresses arising from shrinkage, cooling, and restraint, were of the same order of magnitude as the stresses calculated in terms of pure elastic theory and only the applied external loads; so a weaker concrete that did not induce such stresses could easily be effectively stronger though absolutely weaker.

MSICBI

THE PRESIDENT winding up the debate said, that this question of *kankar* lime *surkhi* mortar was important, since *crores* of rupees would be involved. They might not use this mortar for every high dams, but for moderate dams up to 200 feet it could be used and since its cost was about half that of cement there would be much saving in its use. Since, however, there were no standards of strength for this kind of mortar it could not be used with the same confidence as cement mortar. Without standardization weak points might develop in the dam though the dam might stand as a whole. So he urged for the standardization of the strengths of mortars and limes.

THE PRESIDENT asked Mr. Govinda Rao to introduce the next item.

MR. GOVINDA RAO introduced item (3) and said that Krishnarajasagar Dam was completed in 1929. The data of sweatings were collected from that date. The conclusion was that the sweatings were definitely decreasing.

MR. DILDAR HUSSAIN wanted to know whether similar results had been collected with reference to Vanivilas and other dams and whether they had been compared with each other.

In reply MR. GOVINDA RAO stated that sweating did not occur in all cases because the front face had been pointed with cement to a depth of about nine inches. Where there was deep cement pointing, there was no sweat at all. In the Vanivilas dam, there was practically no sweat at all. He was told that sweating existed in the first nine or ten years but now there was no sweating. The present practice was to point the front face of the dams to a depth of 10 to 12 inches.

RAO BAHADUR JOGLEKAR asked what was the objection to sweating?

MR. GOVINDA RAO said that continuous sweating would cause increased pores in the masonry and thus weaken the dam. In that case, a certain amount of lime might also be carried away with the water as in the case of Mettur Dam.

THE PRESIDENT asked if any information was available about Bombay dams?

RAO BAHADUR MAYDEO said that in some dams in Bombay there was sweating. The sweating had not been measured. It was insignificant.

KHAN BAHADUR KARIM said that they had a dam (earthen) which had almost silted about 50 per cent., but the percolation was on the increase. He would like the research workers to enlighten him on the cause of this phenomenon.

RAO BAHADUR VENKATACHARYA speaking about Mettur Dam said that in the initial stages some sweating might have taken place in Mettur Dam. There was no sweating now. They had got figures regarding the amount of lime leached, chemical analysis of the water *etc.* which had been published and a copy of which was available in the Central Board of Irrigation. In the Mettur Dam construction too they had used 20 per cent. *surkhi*, along with cement. The proportion of sand varied from two parts in the front face and four parts in the rear face.

MR. COCHRANE asked whether the *surkhi* was added in the factory or locally. Was it used for saving cement?

THE PRESIDENT asked if it was added for making cement slow setting?

DEWAN BAHADUR GOVINDARAJA AYYANGAR said no doubt *surkhi* as made contained some amount of sand, allumina etc. The main idea of its addition was as a puzzolona. So much so, that the final proportion of puzzolona to cement and sand used might work out to some required proportion. *Surkhi* was added primarily to reduce the heat of hydration and the proportion was worked out by trial and error. Compared to the local rates, at the dam site *surkhi* did not work much cheaper.

THE PRESIDENT asked Mr. Govinda Rao how nine inches cement pointing was done in dams.

MR. GOVINDA RAO said that the lime mortar on the face of the dam in the front joints of the front course of the stones would be raked to a depth of nine inches while the mortar was still green and cement mortar was then put in.

THE PRESIDENT remarked that it was virtually building the front face with cement mortar.

MR. GOVINDA RAO replied it would be more or less so. Regarding cost he said it depended on the cost of labour, distance of *kankar* etc. from the work site and other conditions prevailing in each place.

(iv) CONSTRUCTION

There was no discussion under this sub-head.

CONSIDERATION BY THE BOARD

THE SECRETARY said that the former subject "Dams" had been split into two subject heads viz. "Dams and Weirs" and "Dam appurtenances". The former comprised four sub-heads.

(i) INVESTIGATIONS

THE SECRETARY reported that there was no contribution on the subject at the Research Committee Meeting.

RAO BAHADUR VENKATACHARYA asked if a model could be constructed on Bligh's theory and then tested.

THE PRESIDENT remarked that Bligh's theories were now antiquated. The design should now be tested against the latest theories.

It was decided that the subject should continue on the Agenda.

(ii) DESIGN

The Board noted the discussions held at the Research Committee Meeting.

(iii) MATERIALS

It was decided that the subject should continue on the Agenda.

(iv) CONSTRUCTION

No discussion.

5C-Dam Appurtenances

- (i) Inlet and outlet works
- (ii) Tunnels
- (iii) Conduits and penstocks
- (iv) Fish passes
- (v) Other works

The Year's Work

There was no contribution under sub-heads (i) to (iii) and (v) at the 1947 Research Committee Meeting.

(iv) FISH PASSES

PRELIMINARY NOTE

In 1941, Rai Bahadur Dr. S. L. Hora of the Zoological Survey of India sent a note on "Dams and the problem of the Indian migratory fishes" to the Secretary Central Board of Irrigation with the request that the note be circulated to the members of the Board for eliciting their opinion on the subject. The note was put up to the Research Committee and it was decided that the matter should be placed before the Board. At the Board Meeting "it was resolved that the letter (along with the note) be circulated to Board members for opinion".

The Secretary was later directed to enquire from Rai Bahadur Dr. S. L. Hora the requirements of fish passes so far as these could be given in general terms, so that it may assist irrigation engineers in the design of fish passes, though it may be necessary for them to get in touch with the Zoological Survey Department in individual cases. The following is some of the information which was supplied by Dr. Hora.

GENERAL REQUIREMENTS OF FISH PASSES

1. "Fish ways, ladders or passes should aim at permitting *at all times* as free an exchange of the fish population on either side of the dam, weir or other obstacle, as would naturally take place if there were no obstruction at all.

The volume of water in different seasons, the idiosyncracies of fish, the height of the dam, the nature of the soil, *etc.*, are bound to necessitate local modifications in design.

2. For the reasons stated in paragraph 1 above no single design will suit all the rivers.

3. The slope of the fish ladder should not be steeper than one foot vertical to ten feet horizontal, so as to ensure a current of velocity of not exceeding seven feet per second in any portion of the fish-way. The flow of the water should be gentle and without deep falls.

4. The compartments of bays of the pass must be of such dimensions that the fish do not risk collision with the sides and upper end of each bay when ascending.

* Introduced by the Research Committee 1947.

5. Plenty of light should be admitted in a fish way, both from maintaining therein the natural conditions of the water, and in order that the interior may easily be inspected and any foreign matter removed. There must be nothing in the formation of the pass to suggest the existence of a trap.

6. It should in all its parts, by action of the current of water passing through it, be as nearly as possible self cleaning of all sand gravel and rubbish.

7. The water supply should be ample and there should always be water in the ladders in the season when the fish are making ascent.

8. The top and side of a fishway should be above ordinary highwater.

9. The pass must be situated where it will be self advertising *i.e.* it must form a current impinging into a certain place below the obstacle so strongly that it becomes the chief or predominant current of the stream, where the fish will be led to that point for attempting the ascent. One may have the best possible kind of pass, but if fish do not get into it one may as well have no pass at all. The position of the entrance is of paramount importance no matter what type of pass is selected. The ideal position for the entrance to the pass is close to where the fish lie, so that there will be an attractive flow of water from the pass, when the river at the level at which they chiefly run. The entrance should be located in a pool at the bottom of the ladder where fish would naturally collect before ascending the river and these pools must be kept clear of all silt and other accumulations and deposits.

10. The Cail fish pass is a combination of the "inclined plain system" with the pool and fall or step system. It consists of series of compartments arranged in steps, and separated by number of cross partitions, which are provided with suitable orifices at the bottom alternating successively from side to side so as to allow the fish according to their individual habits to ascend the fish-way by either leaping over the small waterfalls over the cross partitions or by darting through the orifices, at the same time enabling them to rest in the compartments in comparatively still water. Such a pass may prove satisfactory for most of the fish that are found in the Indian rivers."

In 1944 the Central Board of Irrigation informed the Imperial Council of Agricultural Research for the consideration of their Fish Committee that the investigations in fish passes may be divided into the following three parts:—

- "(a) A study of the habits of various kinds of fish.
- (b) The capacity and strength of various kinds of fish to progress through different velocities and falls *etc.*
- (c) The design of the fish pass to obtain conditions required vide (a) and (b) above.

The investigations under (a) may be carried out by zoologists but the investigations under (c) is purely a matter for engineers. The investigations under (b) are such where it is necessary for both the engineers and the zoologists to work together."

A scheme financed by the Imperial Council of Agricultural Research for an investigation into the bionomics of Indian Migratory fishes was instituted with a view to standardising designs for fish passes on the 1st May, 1946.

The following resolution was moved by Dr. S. L. Hora at the Imperial Council of Agricultural Research Fish Committee Meeting 1946.

“The Fish Committee of the Imperial Council of Agricultural Research strongly urges the Government of India to take adequate steps to safeguard fish productivity of the various provinces in which power and irrigation projects are to be undertaken and that before starting construction of any dam or other type of structure obstructing movements of fish, surveys of fisheries resources be made by qualified experts.”

On this resolution the Fish Committee recommended that Dr. Hora's suggestion was premature and so the suggestion was not adopted.

The question being of immense importance has now been brought on the agenda for the Research Committee.

Recent Literature,

1. Pryce-Tennant, T. E. Fish passes,
2. Report of the Committee on Fish-Passes—Institution of Civil Engineer (London) 1942.
3. Fish passes for weirs and dams. Full scale test of Designs—Central Board of Irrigation Journal January, 1944, page 10.
4. Central Board of Irrigation Bulletin No. 38, June 1943, page 1.
5. Hora, S. L. : Construction of Dams and River Fisheries—Central Board of Irrigation Journal, April 1947.
6. Dams and Fisheries—The Central Board of Irrigation Journal, April 1947 (Editorial) page 77.
7. A. M. Mcleod : An Investigation of Fish-ways 1942.

The Year's Work

The following item was discussed at the 1947 Research Committee Meeting :

(1) Problem of fish cultivation in Irrigation and Hydro-electric reservoirs by S. L. Hora.

(1) PROBLEM OF FISH CULTIVATION IN IRRIGATION AND HYDROELECTRIC RESERVOIRS

by

Sunder Lal Hora, D.Sc., F.R.S.E., F.Z.S., F.R.A.S.B. F.N.I.

Director, Zoological Survey of India, Banaras.

ABSTRACT

Enunciates the main problems of fish culture in large reservoirs and shows their complexity and varied nature. Suggests means to tackle them on comprehensive scale.

Like agriculture, aquiculture or more popularly called Fish Culture is an ancient industry and implies conscious human effort directed towards increasing food supplies through the cultivation of desirable species of fish in impounded

waters. From the above definition, it will be clear that for initiating a programme of fish culture, the following fundamental conditions must be satisfied :—

1. Availability of the varieties of fish suitable for cultural operations ;
2. Suitability of impounded waters for the culture of desirable species of fish ; and
3. Conscious efforts on the part of man to utilise impounded water-resources for raising fish crops. This effort will not only entail human labour, knowledge and skill but will also imply the production and use of fishing implements. Like agriculture again, the fish cropping has three aspects—sowing, tending and harvesting.

This is not an occasion to give details of fish cultural operations, which are capable of considerable improvement through experimentation and research, but to correlate the three fundamentals of Fish Culture with vast sheets of impounded waters dammed up to serve as reservoirs for irrigation and/or hydro-electric purposes. I shall, therefore, discuss the subject under four headings, *viz.*, Fish, Reservoir, Man and Fishing Implements.

FISH

Irrigation or hydro-electric reservoirs are usually constructed by damming a channel of run-away water which becomes flooded at certain times of the year and runs almost dry in other seasons. The water thus wasted annually is indeed a great loss and its storage is definitely a national asset. In such channels, there is usually an indigenous fish fauna, which through centuries has become acclimatised to the variable ecological conditions prevailing in its habitat and may have evolved structural features which enable it to live in harmony with its environment. Let us take, for instance, a hydro-electric reservoir. Such reservoirs are usually constructed by damming up a hill-stream, the main physico-biological features of which are :—

- (i) Rocky bottom, strewn with pebbles and shingle at certain places and rocks and boulders at others.
- (ii) Clear, fast-flowing water cut up into a series of pools and rapids or cascades or waterfalls.
- (iii) Highly oxygenated water in view of (ii) above.
- (iv) Absence of higher aquatic plants which could serve as food for fishes.
- (v) Animal or vegetable matter which could serve as food for fishes lies encrusted on rocks and stones or hidden among pebbles or shingle.

The indigenous hill-stream fish fauna generally comprises of either strong, powerful swimmers, such as Mahseer, Indian Trout, *etc.*, which can negotiate through muscular effort cascades between rock pools ; small loaches or loach-like fishes which live among pebbles and shingle or of species provided with organs of adhesion for sticking to rock and stones and rasping jaws for feeding on encrusted matter. All of them are lovers of highly oxygenated waters and

in quite a number of them gills are greatly reduced in response to this factor in their environment. The feeding habits and the consequent modifications of lips and jaws are also very characteristic of hill stream fishes.

If such a highly specialised fauna is subjected to a changed environment, such as will be produced by the construction of a reservoir, it is obvious that most of the specialised forms will die out and only generalised species may survive. For cultural operations, therefore, we shall have to determine experimentally the species that are likely to survive and their suitability for cultivation rate of growth, feeding habits, breeding habits, palatability, boniness, etc.. The temperature of water in the reservoir and the new physico-biological conditions produced by the construction of the reservoir will have to be studied in detail so as to find out indigenous or exotic species which could be cultivated therein.

Even when suitable indigenous or exotic species have been discovered, it will be necessary to find out a source of fish seed supply. If the cultivable fish becomes established in the reservoir and breeds freely in it, then the seed supply problem will not be so acute, otherwise hatcheries and nurseries may have to be set up and a regular fish farm established near the reservoir. Usually several species of fish are cultured together so as to make use of all food zones in their environment.

In short, for stocking a reservoir of the nature described above with fish, considerable amount of objective applied research and experimentation will be necessary. The nature and extent of research will naturally depend on the type and construction of the reservoir and also on the availability of suitable local varieties of fish but in no case it will be possible to undertake fish culture on a commercial basis without undertaking the investigations referred to above.

RESERVOIR

In discussing the availability of the varieties of fish suitable for cultural operations, references have already been made to the nature of the reservoir. Besides depth and perennial nature of the water, both of which are unsuitable for fish cultural operations, the ruggedness of the bed of the reservoir is an important consideration for harvesting the crop. I shall have occasion to refer to this aspect of fish culture when dealing with fishing implements.

With the construction of a reservoir, one may find the following varieties of environmental conditions:—

- (i) Feeder hill-streams of the reservoir which will continue to afford suitable ecological conditions to the existing torrential fauna.
- (ii) Shore waters up to a depth of ten to 15 feet; warmer and more suitable for the young fish and growth of vegetation.
- (iii) Clear surface waters lying off shore in depths of 15 to 50 feet permitting growth of aquatic vegetation.
- (iv) Deeper waters, over 50 feet in depth, where light does not penetrate and which are not suitable for the growth of plant life. The bottom will be covered with silt and detritus.
- (v) Shallow, muddy bays with the bottom covered with silt.

Ecology of each habitat will have to be investigated experimentally for the determination of varieties of fish suitable for each. It would mean continuous observations over a number of years after the construction of the reservoir and a watch on the changes that might occur in the habitats of fishes from time to time.

MAN

Both the fish and the reservoir can achieve little in the way of fish culture if left to nature alone. There is no doubt that subject to the considerations detailed above there may be a small crop of natural fish, but no regular fishery worth the name can be established without human effort. Besides research and experimentation, which should be directed to attain the objective in view, there will also be need for workers who could interpret and help to apply in practice the new knowledge, the new creations and the new ideas coming out from experimental farms and laboratories. Besides a large number of practical fish farmers, skilled and unskilled labour will be needed to make fish cultures a success after the reservoir is found suitable and the desirable varieties of fish have been discovered. In fact, the success of the whole venture will be in proportion to the knowledge, industry and earnestness of the persons entrusted with the job. It should be hardly necessary in the modern times to argue the case for an increased rate of progress in the technique of food production which is the ultimate objective of fish culture in reservoirs.

FISHING IMPLEMENTS

Human efforts without the aid of tools of a particular industry can achieve little and in tackling the problems of fish culture in reservoirs new methods of sowing, tending and harvesting may have to be found. For instance, in a deep reservoir most of the fishing appliances now used in fresh water fisheries may not be of much practical value. Even deep-sea fishing methods cannot be used on account of the nature of the bottom of the reservoir which may contain tree trunks, immersed villages, rocks and boulders, etc. Experiments in fishing methods carried out at the Mettur Reservoir have shown that 'Rangoon Net' used by the fishermen of the Godavari for catching *Hilsa* is fairly effective but much research and experimentation is still needed to evolve suitable methods for harvesting the crops.

It will thus be seen that a considerable amount of investigation will have to be undertaken in the case of each reservoir before we can think of heavy profits from fish cultural operations. It must also be borne in mind that each piece of water has its individuality and specific problems, so no rules of thumb can be formulated for all reservoirs.

SUGGESTIONS FOR THE COMMITTEE

In this brief account, I have enunciated the main problems of Fish Culture in large reservoirs and have shown their complexity and varied nature. The only way to tackle them on any comprehensive scale is to set up investigations through fishery scientists, simultaneously with their investigations considered essential for undertaking a reservoir-construction project. For each reservoir, an experimental fish farm must be constructed and a staff of one senior and two junior technical officers with the necessary complement of subordinate staff engaged. The same staff can carry out the fish

and fisheries survey of the river and work out the bionomics of commercially important migratory fishes with a view to advise on the type of fish passes that may be required in that particular dam or weir.

In other countries such investigations are invariably included in any project of dam construction and it is high time that India also thought of conserving the fishery resources of the rivers and of augmenting fish supply through the proper utilisation of large areas of impounded water.

DISCUSSION BY THE RESEARCH COMMITTEE

DR. HORA introduced his paper and said that he was very grateful to the Committee for having afforded him an opportunity to speak on the subject of fishery problems connected with the construction of protective works in waterways and how engineers could be helpful in the solution of those problems. He had been inviting attention to the study of fish passes in relation to the migratory movements of Indian fishes concerned since a long time and was happy to state that he had now been afforded an opportunity to survey the fish and fisheries of the Kosi River in connection with the multi-purposes dam.

The waterway engineers, he said, were mainly concerned with the water resources of a country and their proper utilisation for human welfare and in this connection they attached importance to irrigation, navigation, and hydro-electric power. He wanted the audience to go back to the period of the primitive man when there were no engineers in the modern sense. Even then waterways played a very important part in the economy of man as is evidenced by the fact that all old, prehistoric cities were built either on the banks of rivers and lakes or on the sea shore. To those primitive people, water resources meant fisheries and navigation intermittent or all the year round. Dr. Hora, therefore, wanted engineers to regard fisheries, nature's most valuable and prolific gift, as one of the water resources deserving of their consideration. It was necessary, he said, that surveys, such as geological, economic, *etc.* pertaining to a project should also include a survey of the fisheries resources of that waterway.

For fish culture in reservoirs, it was necessary, he said, to know the types of fish locally available, varieties of fish that could adapt themselves to changed conditions, the physical and biological conditions affecting fish life produced by the construction of dams and the general ecology, bionomics and life-history of the species to be cultured therein. He recommended, therefore, that Research Fish Farms on an experimental scale should be instituted at dam sites to investigate the fishery problems likely to arise with the construction of dams in the country.

With regard to the existing fish passes in various dams, Dr. Hora characterised them as fish traps and stated that they had not been built with any regard to the life-history and bionomics of the fishes likely to use them in their migratory movements.

Referring to some earlier subjects discussed by the Research Committee and with regard to acidity or alkalinity of tube-well waters, he referred to a tube-well irrigated fish farm at Batala, Gurdaspur District, Punjab, and stated that its water was definitely acid and found unsuitable for the culture of fish. Several measures had to be taken to make it suitable for fishery purposes. It was well known that a p_H value of 8.0 to 8.8 was suitable for fish growth and

he thought the same p_H value was good for plant growth. It was necessary, therefore, that the Irrigation Engineers should check the p_H of tube-well waters and, if necessary, correct them for irrigation purposes.

For the reclamation of low lands, DR. HORA suggested that much improvement could be made by digging fishery tanks and by filling up the neighbouring lands with the earth excavated therefrom. He further pointed out that fish slime had precipitating action on the mud suspended in the water and fish culture could, therefore, help in silting up low lands gradually.

By showing as above how problems of engineers were inter-connected with those of fishery biologists, the speaker pleaded for co-ordination of their activities in the best interest of the nation. He also emphasised the need of establishing a Fisheries Research Section in the Central Waterways Irrigation and Navigation Commission, so that proper fishery surveys and fishery biological investigations could be instituted on a comprehensive scale in connection with the various dam projects. Financial burden of running such a section could perhaps be shared with Indian Council of Agricultural Research. Preliminary fishery work connected with any dam project could perhaps be undertaken by the Provincial or State Fisheries Department, for such departments now existed everywhere. The riverine research staff of the Central Inland Fisheries Research Station could also help in these investigations and the Zoological Survey of India could be helpful in carrying out faunistic surveys. In view of these considerations, he presented the following draft resolution for consideration of the meeting, which was discussed and passed.

“Resolved that the Research Committee recommends to the Board that in the construction of protective river works, due consideration should be paid to the fishery resources of the waterways by conducting faunistic and economic surveys of the fisheries. In view of the proposals to construct a large number of dams in the country the Committee desires that Irrigation Research Stations should undertake work on fish passes by associating fishery scientists with them in this work.”

THE CHAIRMAN (SYED DILDAR HUSSAIN) thanked the speaker for his instructive speech.

CONSIDERATION BY THE BOARD

THE SECRETARY said that this was a new subject head introduced by the Research Committee with the following sub-heads: (i) Inlet and outlet works (ii) Tunnels (iii) Conduits and penstocks (iv) Fish passes and (v) Other works.

There was no discussion at the Research Committee Meeting under this head except on Fish passes, under which Dr. Hora had contributed a note of general interest. The resolution passed by the Research Committee might be considered by the Board.

After some discussion, the following Resolution was adopted by the Board.

Resolved that the Board is of the considered opinion that in the construction of protective river works due consideration should be paid to the fishery resources of the waterways by conducting faunistic and economic surveys of the fisheries, and in view of the proposals to construct a large number of dams in the country, the Board desires that Irrigation Research Stations should undertake work on fish passes by associating fishery scientists with them in this work.

6C—Spillways

PRELIMINARY NOTE

This subject has been introduced as a separate item for the first time. Previously, discussion on the subject has taken place under "Dams". The following items discussed at the 1946 Research Committee Meeting, under "Dams" could be classified under the head "Spillways".

- (1) Mysore type of spillway siphon, by V. Ganesh Iyer, Esq.
- (2) Siphon spillways: Ganesh Iyer's volute siphon: Factors affecting the co-efficient of discharge of.
- (3) Determination of the co-efficient of discharge of automatic gates spillways for Tansa Dam.
- (4) Spillway siphon for Rasul Hydro-electric Power Station.
- (5) Volute type siphon models.
- (6) Spillway dams.
- (7) Siphon spillways.
- (8) Siphon spillways.

The Board at their last meeting accepted the recommendations of the Research Committee that Mr. Ganesh Iyer's paper on Spillway Siphons be published as a Board Publication after the author has made such additions and alterations that he considers necessary. Mr. Ganesh Iyer has not sent in the completed copy of the draft publication as yet.

There are three main types of siphon spillways. One is the overflow spillway, the second is the inclined tunnels spillways and the third may be called Shaft spillways. This includes what the Americans call the "Glory Hole" spillways and the Ganesh Iyer's Volute Siphon.

The following are some of the features which require investigation:

- (1) Overflow spillways
 - (a) Slope of crest for maximum co-efficient of discharge
 - (b) Slope of glacis and cistern for minimum erosion below the spillway
- (2) and (3) Inclined Tunnel and Shaft spillway
 - (a) Mutual interference
 - (b) Priming head
 - (c) Co-efficient of discharge
 - (d) Cavitation
 - (e) Measures against erosion on the downstream side

Bibliography

1. Stickney, G.F. :—Siphon Spillways, Transactions American Society of Civil Engineers, Paper No. 1485, page 1098, 1922.
2. Ganesh Iyer, V and Narasimha'ya, M :—Note on Siphons and Siphon Spillways, typewritten Mysore State, P.W.D. 1941.
3. Stuart, J.M.B. —Siphon Spillways, Proceeding of Burma Engineering Congress, Volume VI, paper No. 45, pages 64-84, 1923.

4. Naylor, A. H.—Siphon spillways, Edward Arnold and Co., London, 1935.
5. Rock, E.—Design of a high head siphon spillway, Proceedings of the American Society of Civil Engineers, April 1939, pages 607-618. Discussions in April 1940 issue, pages 666-670, June 1939 issue, pages 1098-1100, November 1939, pages 1641-1646.
6. Co-efficient of discharge of automatic siphon spillways, Journal of the Institute of Civil Engineers, December 1937, page 318.
7. Inglis, C. C.—Copy of letter No. 682 dated 1st June 1938 from Mr. C. C. Inglis, C.I.E., Director, Central Irrigation and Hydro-dynamic Research Station, Poona, to the Superintending Engineer, Deccan Irrigation Circle, regarding design of a syphon for the waste weir at Bhandardara.
8. Stevens, J. C.—Siphons as water level regulators, Proceedings of the American Society of Civil Engineers, October 1938, pages 1627-1641.
9. Varadarajan, M. R.—Siphon spillway for the Vashvant Sagar dam in Holkar state, Mysore Engineers Association Bulletin Volume XI No. 2, page 8, 1933.
10. Nelidov, I. N.—Determination of pressure within a siphon spillway, Proceedings of the American Society of Civil Engineers, March 1945, page 297.
11. Brown, G. A. M.—Note dated 10th March 1945 on the discharge coefficient of certain types of spillway siphons (Typewritten) March 1945.
12. Siphon Spillway experiments, Report of Central Irrigation and Hydro-dynamic Research Station, Poona, 1942-43, page 24, 1943.
13. Bonnet, L, Blockman, J. and Lamoen J.—Experimental investigation of various types of self priming siphons using scale models, International Association for Hydraulic structure Research, Summary in English, German and French, 1937.
14. Hegley.—Automatic Siphon Spillways, Civil Engineering and Public Works Review, November 1937, page 394.
15. Binnie, G. M.—Model Experiments on bell mouth and siphon bell mouth overflow spillways, Journal of the Institute of Civil Engineers, paper No. 5116, pages 65-90, November 1938.
16. Experiments with siphon spillways (Typewritten) Report on Research work in Irrigation, Research Division Poona, 1933-34 page 27.
17. Ahmed Shukry.—Experimental investigation of the behaviour of flow through siphon spillways. Fuad University Giza, Egypt.
18. Siphon Spillways, United Provinces, Public Works Department, Annual Report, Irrigation Research Section, Technical Memo. No. 16. page 24, 1945.
19. Iyer, V. G.—Volute Siphons, 1946, (stencilled).
20. Creager, W. P. Justin, J. D. and Hinds, J.—Engineering for dams, 3 volumes, 1945, pages 871, 909.

21. Hanna, F. W. and Kennedy, R. C. —Design of dams 1931, page 196.
22. Justin, J. D.—Earth Dam Projects, 1932.
23. Lewitt, E. H.—Hydraulics and the mechanics of fluids, 1937, page 111.
24. O'Brien, M. and Hickox, G. H.—Applied fluid mechanics, 1937, page 139.
25. Jameson, A. H.—Introduction to fluid mechanics, 1937, pages 133, 234.
26. King, H. W.—Handbook of Hydraulics, 1939, page 61.
27. Kuttiammu, T. P.—Irrigation Research, Division Poondi, Madras Annual Report for 1944, page 2.
28. Franzius, O.—Waterway Engineering, 1936.
29. Davis, C. V.— Handbook of applied hydraulics, 1942.
30. Soucek, E. and Zelnick, E. W., Lockmanifold Experiments, Dock and Harbour Authority, Vol. 26, No. 303, January 1946.
31. Hill, R.A. and McNaughy, D.C.—Design of Spiral Spillway Chutes Civil Engineering Vol. No. 15 No. 11, November. 1945.
32. Roberts, K.C.—Spillway gates and equipment on T.V.A. Projects Civil Engineering Vol. 16, n. 1, January 1946.
33. Vibration damage to Spillway gates, Engineering News-Record, Vol. 136, No. 12, March 21, 1946.
34. Spillway and drainage siphons, Annual Report. Technical Punjab Irrigation Research Institute, Lahore 1935.

The Year's Work

The following items were discussed at the 1947 Research Committee Meeting :

- (1) Scour downstream of a five-vented spillway dam for various combinations of shutter openings.
- (2) Pressure distribution on ogee spillways—Tungabhadra Spillway Dam.
- (3) Pressure distribution on the Ramapadasagar Spillway Dam.
- (4) Co-efficient of discharge for the Ramapadasagar Spillway Dam.
- (5) Stilling-basins for overflow dams.
- (6) Roller bucket for the Ramapadasagar overflow dam.
- (7) Siphon spillways for Pahuj weir.
- (8) Preventing scour below Ganesh Iyer Volute Siphon Spillways by mutual impact of jets issuing from adjacent siphon outlets.
- (9) Experiments on the effect of altering the shape and number of volutes in a volute siphon.
- (10) Experiments on dissipating the energy of flow from volute siphons
- (11) Experiments on variation of co-efficients of discharge under varying degrees of submersion of siphon outlets.
- (12) Experiments on Shaft Spillways for Tanks.

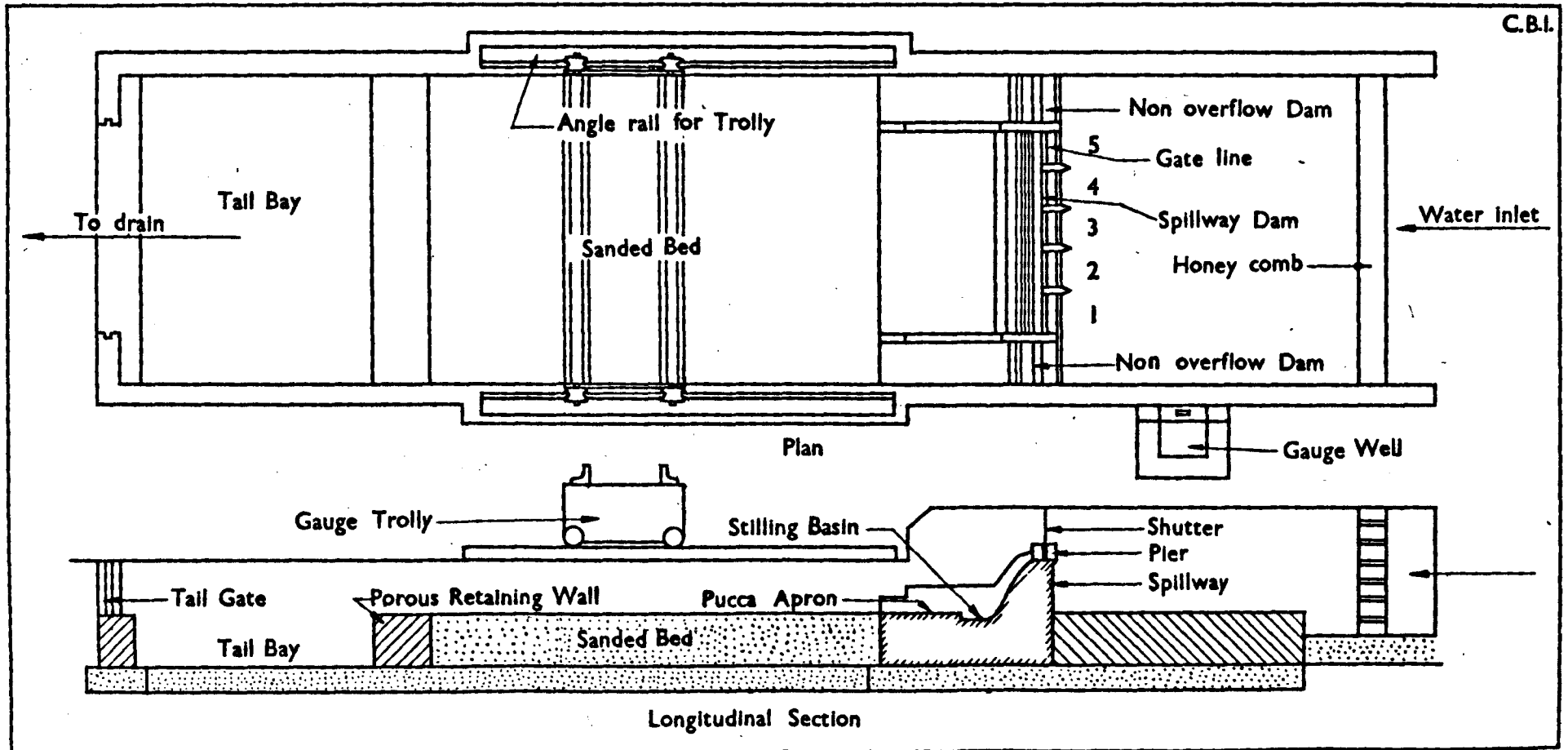


Figure 6 C.1:- Showing Model Set-up of a Five Vented Spillway

Scale 2 1 0 2 1 6 Feet

- (13) Model investigations regarding the design of Nagwa Dam,
- (14) Rasul Hydel Scheme Spillway Siphon.
- (15) Bhakra Damp Spillways.

(1) SCOUR DOWNSTREAM OF A FIVE-VENTED SPILLWAY DAM FOR VARIOUS COMBINATIONS OF SHUTTER OPENINGS (1)

ABSTRACT

In the case of a regulator or a gated spillway, the scours developed downstream of the structure during regulation are very much influenced by the way in which the shutters are opened. To study the nature and magnitude of this influence, a series of experiments were carried out on a model of a spillway dam consisting of five gated spans, which is briefly described in this report.

DESCRIPTION OF THE MODEL

For this study, the design proposed for the Tungabhadra Dam was adopted in some of the details and the model was constructed to a scale of 1/100. A plan and section of the layout is given in Figure 6 C.1 The model of the spillway was fitted up across a parallel sided flume 26 feet \times 5.83 feet \times 2.5 feet taking off from one side of the experimental tank 20 feet \times 20 feet \times 2.5 feet. Water supply was controlled and maintained steady by means of inlet and overflow sluice valves and the water levels downstream of the model were adjusted by means of a tail gate. The model itself consisted of an ogee profile overflow section of five spans each 60 feet wide (reduced to scale 1/100) separated by piers ten feet wide and two non-overflow sections of 120 feet length, each constructed in continuation of the spillway on either side. The spillway was 94 feet high and maximum depth of flow over it 22.5 feet. It had a stilling basin nine feet deep and 26 feet long from the bucket toe and a *pucca* apron 120 feet long beyond. The top of the apron was taken as + 100.00 and this datum was adopted for calculating scour contours. The stilling basin and apron downstream of it were separated from the river bed immediately in rear of the non-overflow dams by means of river walls 12 feet thick and of sufficient height. The bed downstream of the masonry floor was moulded level with screened sand passing through 24 mesh and retained on 30 mesh per inch sieves. All measurements were made by pointer gauges reading to a thousand of a foot.

DESCRIPTION OF THE EXPERIMENTS

Excluding a few test runs the following combinations of shutter openings were experimented up on—

- (1) All shutters fully open ; upstream water level 22.5 feet over crest ; downstream water level 25 feet over apron.
- (2) All shutters open half ; upstream water level same as above ; downstream water level 17.8 feet over apron.
- (3) All shutters open one-third ; upstream water level same as above ; downstream water level 13 feet over apron.

(1) Irrigation Research Station, Poondi, Madras, Annual Report 1946, pages 29 to 41, etc.

(4-6) Four vents fully open in three different combinations ; upstream water level same ; downstream water level 21.8 feet above apron.

(7-11) Five combinations of three vents fully open ; upstream water level same : as above ; downstream water level 18.5 feet above apron.

(12-17) Six combinations of two vents fully open ; upstream water level same ; downstream water level 14.5 feet over apron.

(18-20) One vent open full—three separate cases ; upstream water level same as above ; downstream water level 12 feet over apron.

The period of run in each case was one hour.

DISCUSSION OF RESULTS

(i) *Flow pattern.*—Observing the flow during the several runs the following characteristics were noticed :—

(a) When all the shutters were open either partly or fully there was steady forward flow away from the structure except for some return flow and eddying on the wings, i.e. opposite to the non-overflow sections.

(b) When only some of the shutters were open the flow invariably got unbalanced and return flow obtained. The resulting heaving and eddying were severe when the shutter openings were concentrated and unsymmetrical about the centre line. By opening the shutters symmetrically and by distributing them evenly the flow could be more or less steadied. All these characteristics of flow had their effect on the location, development and depth of scours.

(ii) *Scours.*—A comparative study of the scour contours yielded many interesting inferences.

Though the discharges with all vents half or one third open were much less than with all vents fully open the depth of scour remained the same because in the former two cases the downstream water levels were proportionately less.

The benefit of opening all shutters partly in preference to a few of them was fully clear.

These tests indicated the advantage of opening the vents in a symmetrical manner and of spreading them evenly along the full length of the spillway.

The unbalanced flow observed in the cases where shutters were opened unsymmetrically will have considerable adverse effect on marginal banks and will lead to side erosion and bellying.

Another inference evident from these studies relates to the technique of model experimentation for the design of energy dissipators. Two dimensional models usually adopted for such experiments can give a solution for dissipating the excess energy which gives rise to eddies and surging flow, but cannot indicate the effects of lateral flow, return eddies, etc.. It is, therefore, advisable to check the design finally on a three dimensioned model.

CONCLUSION

Scours downstream of a regulator are influenced not only by the discharge and tail water depth but by the way in which the shutters are operated. With

a level bed in rear it is best to open all shutters partly for allowing the desired discharge rather than a few of them. In case only a few shutters are to be opened it is better to distribute them evenly along the full length of the regulator and to prefer a symmetrical arrangement of opening. The basic idea should be to eliminate altogether or to minimise to the extent possible all return flow and lateral eddies. These cause erosion of margins and accentuate bedscours.

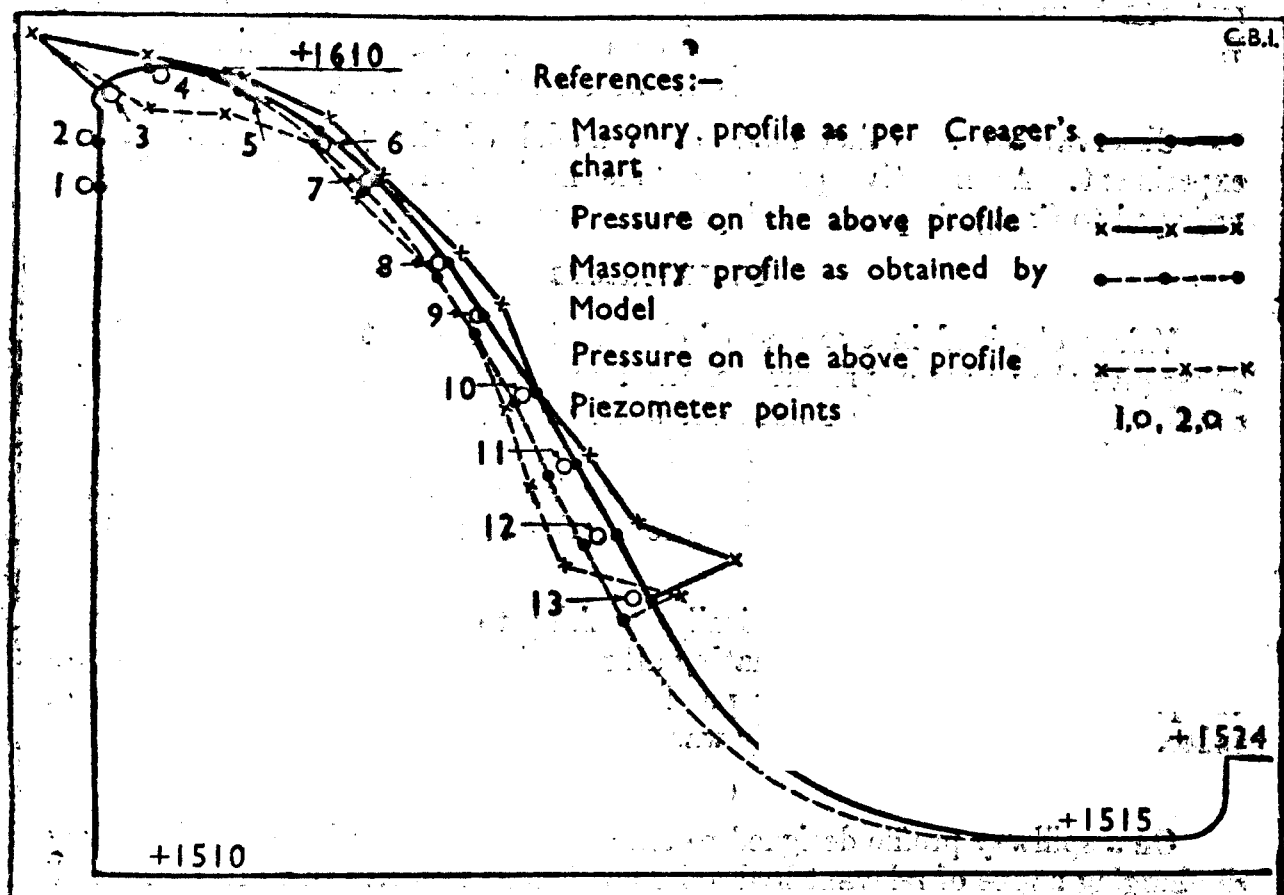


Figure 6 C.2: Pressure observation on alternative designs of profile of Tungabhadra spillway dam

(2) PRESSURE DISTRIBUTION ON OGEE SPILLWAYS—TUNGABHADRA SPILLWAY DAM ⁽²⁾

Two designs are shown for the profile of the Tungabhadra spillway dam in Figure 6 C.2. The profile indicated by thick line is based on the chart given by Creager ⁽³⁾.

The other profiles indicated by dotted lines, followed the underside of a freely discharging nappe satisfying relevant hydraulic particulars, as obtained from a hydraulic model. The latter contained slightly less masonry than the former and also gave a slightly higher co-efficient of discharge. Before it could be adopted, however, tests had to be made with regard to cavitation stress and stability. A start was made with a model incorporating piezometers to evaluate the nature and magnitude of pressures on the surface of the spillway for the full designed overflow.

⁽²⁾ Irrigation Research Station, Poondi, Madras Annual Report, 1946, pages 59 to 60

⁽³⁾ Creager, Design of Masonry Dam.

THE MODEL

A masonry flume with parallel walls two feet apart (clear) was constructed and the models of the spillway to scale 1/36 were fitted in the flume. Suitable arrangements were provided for the supply and withdrawal of water, for adjusting and measuring the levels upstream and downstream, and for maintaining these levels constant and steady. A number of piezometer holes were left along the centre line of the profile and these were connected to a piezometer set up outside the flume by means of suitable tubings and nipples. Water was let into the flume and pressures were noted and recorded for various conditions of flow. Seven experiments were carried out in this study.

Observations were first made on a model of the profile obtained by experiment. As negative pressures were noticed on this, the model was reshaped to Creager's profile and pressures noted again.

OBSERVATIONS

Values obtained in two sets of observations are plotted in Figure 6C.2. These give the values of pressures exerted on the surface of the spillway by the overflowing water for the maximum flood discharge conditions. It may be noticed that negative pressures were indicated at several points of the profile based on model experiments and that Creager's profile did not show any negative pressure at all. The minimum pressure in the former was six feet and in the latter it was almost zero.

It would be possible theoretically speaking to obtain a new profile intermediate between the two eliminating all negative pressures and keeping down the pressures as near to zero as possible. But the practical difficulties were indeed considerable and the study was not pursued.

CONCLUSIONS

On a spillway profile designed to follow closely the underside of a freely discharging nappe of relevant hydraulic particulars negative pressures of small magnitude were detected. The profile based on Creager's chart did not disclose any such negative pressures. The positive pressures obtained on the latter were of small magnitude. For the Tungabhadra spillway the original design, based on Creager's profile could, therefore, be adopted. This, it was noted, gave a co-efficient of discharge of 3.75 in the formula $Q = CLH^{3/2}$ for the designed maximum head.

(3) PRESSURE DISTRIBUTION ON THE RAMAPADASAGAR SPILLWAY DAM (4)

Tests on a "Minimum Section Profile for the Ramapadasagar spill way similar to those carried out on a model of Tungabhadra spillway dam also indicated negative pressures. A "Practical Profile" satisfying considerations of strength and stability and constructional convenience was also tested on the piezometer set up. This design is shown in Figure 6C.3 experiment made of this is explained below.

(4) Irrigation Research Station, Poondi, Madras Annual Report, 1946 pages 60 to 63.

indicating a highly efficient shape from the point of view of discharge. The design indicated no negative pressures along the profile. Observations of pressures for the bucket portion will have to be taken when the design for it is finalized.

(4) CO-EFFICIENT OF DISCHARGE FOR THE RAMAPADASAGAR SPILLWAY DAM (5)

INTRODUCTION

The profile of the Ramapadasagar Spillway Dam as designed is shown in Figure 6C.3. A series of experiments were made on a 1/60 scale model of this design for obtaining values for the co-efficient of discharge. A few preliminary experiments were made for perfecting the model. After the model was perfected two sets of experiments were carried out.

Further experiments are in progress to find out the effect of cantilevering out the pier cut waters on the upstream side of the spillway on the coefficient of discharge.

THE MODEL

The model was a two dimensional one and was fitted up between the parallel sides of a masonry flume 2.5 feet wide \times 4 feet deep \times 20 feet long taking off at one end of a stilling cistern 20 feet \times 10 feet \times 5 feet. The model which as already stated was to scale 1/60 was constructed in masonry and plastered perfectly smooth. The heads over the crest of the model were measured in a gauge well five feet upstream of the model and the discharge was measured by means of a precalibrated weir over which the water was let into the stilling cistern. All measurements were made by vernier gauges, reading correct to thousandth of a foot. Arrangements for maintaining constant heads during the experiments were also provided.

RESULTS

The values obtained for C in the formula $Q = C L.H. \frac{3}{2}$ after applying necessary corrections for the velocities of approach are furnished in Table 6C.1. The heads over the spillway noted refer to the prototype. The co-efficient could be seen to rise from 3.45 for ten feet overflow to 3.8 for the full depth of 30 feet. In the prototype the value might be actually a little higher. For design purposes, however, the value of 3.8 was recommended to be adopted.

(5) Irrigation Research Station, Poondi Madras, Annual Report, 1946, page 63.

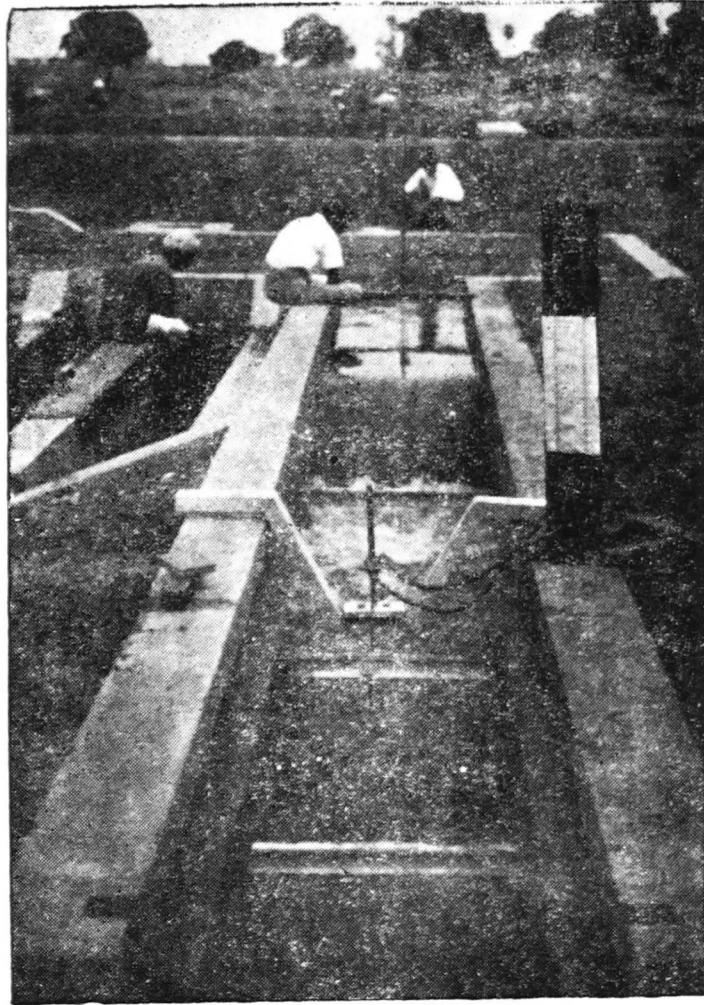


Figure 6C.4 : *Part model of the Tungabhadra Spillway Dam.*

TABLE 6C.1
Ramapadasagar Spillway Dam—Model 1/60:co-efficient of discharge.

Experiment No. 1		Experiment No. 2	
Head over spillway (feet)	Co-efficient of discharge	Head over spillway (feet)	Co-efficient of discharge
11.2	3.53	10.4	3.20
12.0	3.47	14.9	3.56
15.7	3.64	15.2	3.61
16.1	3.64	17.0	3.62
20.2	3.77	17.4	3.59
21.2	3.82	22.3	3.83
23.1	3.81	22.4	3.74
27.0	3.82	23.6	3.81
28.1	3.86	24.8	3.73
30.5	3.77	26.6	3.46
		27.5	3.74
		29.2	3.77

(5) STILLING BASINS FOR OVERFLOW DAMS (6)

ABSTRACT

Experiments conducted on two dimensional models of the Tungabhadra spillway dam in 1945 were continued in 1946 in order to see if an improved type of stilling basin could be evolved. The studies became particularly important as similar spillways were proposed in several of our Post-war Projects, for example, the Gandikota (Pennar), the Vagai, the Lower Bhavani, the Malampuzha Projects, etc. A very large number of experiments, mostly of an exploratory and qualitative type were carried out and a brief description of the more important of these studies is furnished here. The studies did not yield an entirely satisfactory solution and the research is, therefore, in progress.

DESCRIPTION OF THE MODEL

Most of the experiments discussed here were conducted on a 1/48 scale model of the Tungabhadra spillway already available at the station, Figure 6C.4. Its salient dimensions were as given below:—

Length—2 feet 10 inches \equiv 156 feet.

Height—28 inches \equiv 100 feet.

Profile—based on Creager's chart.

Bucket radius—6.25 inches \equiv 25 feet.

Depth of stilling basin—2.25 inches \equiv 9 feet.

Length of apron in rear—24.5 inches \equiv 98 feet.

Maximum depth of flow overcrest—3.94 inches \equiv 15.75 feet.

Maximum tail water depth over apron—5.8 inches \equiv 23.2 feet.

(6) Irrigation Research Station Poondi, Madras, Annual Report 1946 page 63.

The model was constructed in smooth plastered masonry except for the sill which was in teakwood. Some of the types of sills tested are shown in Figure 6C. 5(A to L). They were fixed one after the other at the end of the stilling basin and the nature of flow with varying discharge and tail water conditions were observed. In typical cases the water surface elevations were traced. In a few cases a continuous run for one hour each was maintained with maximum water level conditions upstream and downstream and the scour formed in rear traced. For these scour experiments the bed material used was screened river sand of 14/18 mesh per inch.

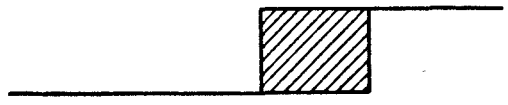
Some of the observations were repeated quantitatively on other models available at the station including one in the laboratory glass flume.

STANDING WAVE CHARACTERISTICS

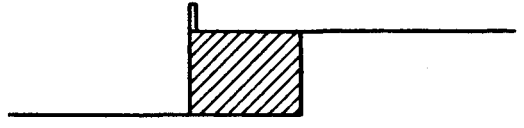
These spillway dams are all designed to have crest shutters and the conditions of flow that may obtain will vary considerably from practically nothing to the maximum depth over crest on the one hand and from the maximum depth to almost nothing in the river downstream on the other. With such varying conditions the design of a perfectly satisfactory stilling basin becomes almost impossible and a selection has to be made of some device which will be satisfactory for as wide a range as possible. In these designs a standing wave is made to form on the solid apron downstream of the dam by means of some type of sill or baffle. Let us consider a simple stilling pool with no tail water, dissipation of energy is caused by actual impact of the shooting nappe against the baffle. When there is some tail water, a standing wave is formed. With high tail water levels the standing wave may be submerged and may form very near the dam face. If in this case the tail water is kept steady and the discharge increased, the standing wave will move farther and farther down till for a certain condition a perfect standing wave with a large roller is obtained just over the baffle. If the discharge is further increased the standing wave rises higher and dives down to meet the tail water and another standing wave is formed here. The rollers in such case become less effective in dissipating energy. Increase the discharge still further: the surface roller disappears and the jet shoots forward with hypercritical velocity and strikes the river bed downstream producing a deep scour. The studies made were intended to find out how best to get a design which would not cause such deflection of the nappe for the worst condition of discharge and at the same time would give the best possible standing wave. The possibilities of using various kinds of sills were gone through in the experiments. Some of the types of sills treated are shown in Figure 6C.5 (A to L). Observations made and inferences gathered are described below.

SIMPLE VERTICAL SILL

The profile of standing wave obtained with an ordinary stilling basin having a horizontal floor and a simple vertical sill for conditions corresponding to maximum water level upstream and downstream is shown in Figure. 6C.6, curve (1). It was not a perfect standing wave; the wave had a crest rising above the tail water and diving jet which created a second standing wave and a series of waves downstream. When the tail water was gradually lowered



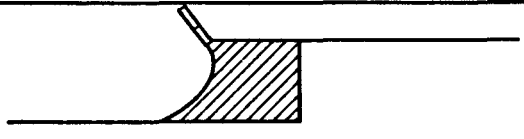
(A) Simple Vertical Sill



(B) Simple Vertical Baffle



(C) Re-entrant Sill



(D) Re-entrant Perforated Baffle

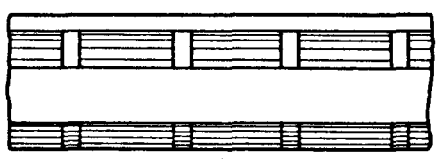


Section



Elevation

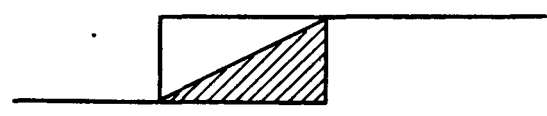
Plan at Top



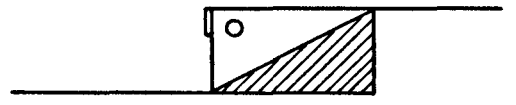
(E) Re-entrant Perforated Sill



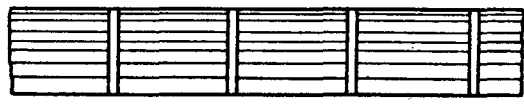
(F) Simple Deflector



Section

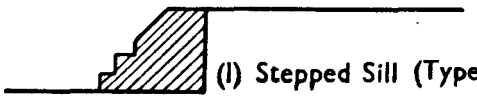


(H) Toothed and barred Deflector



Elevation

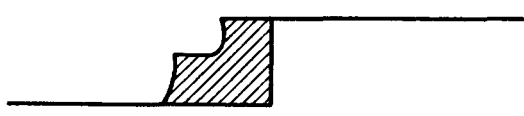
(G) Toothed Deflector



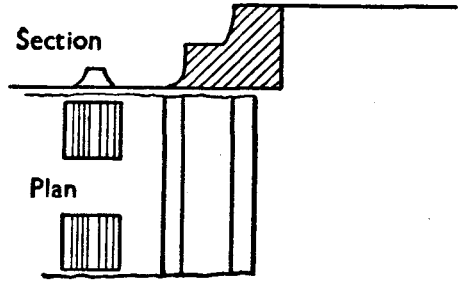
(I) Stepped Sill (Type 1)



(J) Stepped Sill (Type 2)



(K) Stepped Sill (Type 3)



(L) Stepped Sill with Spray Blocks in front

Figure 6C.5:- Showing various types of Tested Sills of Stilling Basin for Spillway Dam

the dive became more powerful and the disturbances more pronounced. When the tail water was reduced to the minimum the standing wave was still formed, but it was practically due to the impact on the sill rather than any impact on the tail water. On the other hand when the tail water was kept high and the discharge reduced the standing wave showed signs of improvement and at a certain stage a perfect standing wave was formed. When the discharge was further lowered gradually the standing wave got nearer and nearer to the dam face. Considered as a whole the design was just satisfactory in that there was no bodily deflection of the high velocity jet for the worst condition.

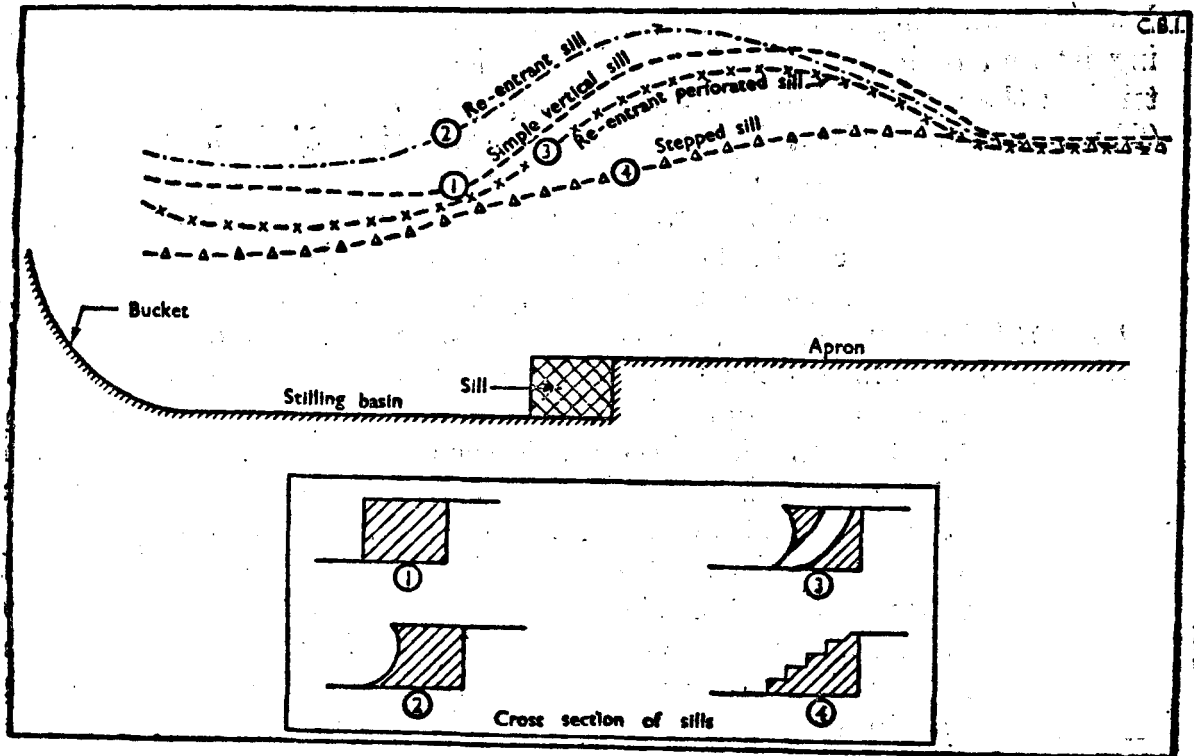


Figure 6 C.6 *Standing wave profiles for different types of Sills.*

Now the depth of the stilling basin was reduced and the observations repeated. The standing wave for most of the conditions improved, but when the tail water was low the sill acted as a deflector and the hypercritical jet was bodily deflected in a violent shooting sheet. Reducing the length of the pool also had the same effect. The sill was then changed to a baffle wall rising above the apron Figure 6C.5 (B). The general characteristics remained the same as if the cistern were deepened.

RE-ENTRANT SILL

Next the sill was replaced by a re-entrant sill Figure 6C.5-(C) of the same overall dimensions. The sill rolled back the shooting nappe towards itself and hastened the formation of a standing wave on the one hand and obviated deflection downstream on the other. The standing wave obtained for the same maximum water level conditions upstream and downstream is shown in Figure 6C.6, curve (2). It was higher than the one obtained with the vertical sill. When the re-entrant sill was brought near towards the bucket the standing

wave rose higher. Deepening of the pool did not improve the standing wave. A better standing wave was got by increasing the length of the pool or by decreasing its depth. Here again when the sill was low enough or when it was far enough from the bucket a shooting jet was produced. On the whole the curving of the sill has more or less the same effect as putting a baffle to increase the effective depth of the basin. The relative advantage may have, therefore, to be judged by comparison of cost of construction and maintenance and by structural considerations.

RE-ENTRANT PERFORATED SILL

The re-entrant sill was next provided with perforations with a view to lower the standing wave crest, to relieve the sill from direct impact of the shooting jet and to improve its dissipating efficiency. The first sill of this kind tried on the model is shown in Figure 6C.5(D). Here the re-entrant sill shown in (C) was surmounted by a sloping baffle and perforations $3/8$ inches \times $3/8$ inches \equiv 1.5 feet \times 1.5 feet spaced $3/8$ inches \equiv 1.5 feet apart were made in this baffle. This gave a high standing wave. The baffle was then removed and perforations were given in the sill itself as shown in (E). The larger the perforation the lower was the standing wave. Perforations 2.5 inches \times 1 inch \equiv 10 feet \times 4 feet spaced $1/2$ inch \equiv 2 feet apart was found quite good. The standing wave profile obtained with this sill for identical conditions of discharge and tail water levels as adopted in the previous experiments is shown in Figure 6C.6, curve 3. The standing wave was lesser in height than in the previous two cases. The functioning of this sill for various discharges and tail water conditions was studied and was found almost similar to what was noticed in the case of the re-entrant sill with the one difference that the standing wave had a lesser height due to the relief effected by the perforations. Against the advantages mentioned already we may have to consider the structural and financial aspects of construction and maintenance of the sill. Also the jets escaping through the perforations might carry residual energy to be stilled lower down

SIMPLE DEFLECTOR

In every one of the types of sills described above the tip of the sill was either vertical or sloping inwards. This was responsible for the heaving up of the standing wave. To see how far this could be eliminated by introducing modifications to a deflector sill was the next approach made. A simple deflector as shown in Figure 6C.5 (F) was examined. Except when the tail water level was sufficiently high compared with the depth of flow over crest, no standing wave was formed.

TOOTHED AND BARRED DEFLECTOR

The deflector was then provided with teeth as shown in Figures 6C.5(G.) These consisted of simple vane walls $1/2$ inch \equiv one foot thick spaced 3 inches \equiv twelve feet apart and having their upstream faces vertical and tops horizontal. This was not effective. Increasing the thickness of the teeth and thus reducing the gaps between them had the effect of reducing to a small extent the range within which deflection occurred. This would then be a modification of Rehbock's dentated sill. Rehbock's dentated sill has this effect that for high heads and low tail water depths it acts as a deflector.

In order to break the nappe and so prevent deflection the teeth were all pierced and a rod $\frac{3}{8}$ inches diameter \approx 1.5 feet was inserted through them. This was found ineffective as for a wide range of discharge conditions the shooting jet could easily shoot through beneath the bar. To reduce this space the rod was replaced by a reeper $\frac{3}{4}$ inch \approx 3 feet deep. This again was in operative till the overflow became large. When the overflow was large the shooting jet got split up into two parts, the bottom part passing through the gap beneath the obstruction in a shooting jet with undiminished velocity and the top part which got intercepted by the obstruction rising very high in a spray caused by the impact. This did not appear satisfactory and the attempt to modify a deflector in this manner was not pursued.

STEPPED SILL

Three types of stepped sill are shown in Figure 6C.5 (I, J and K) and were tested on the model. In type (I) there were two steps and a deflector top. In type (J) the same total depth was covered by five steps: there was no deflector top and the front faces of the steps were curved inwards as in the re-entrant sill. In type (K) there were only two steps both having re-entrant faces and the steps being steeper than the previous types. Figure 6C.6 curve 4 shows the standing wave profile obtained with type (J) for maximum water level condition. The standing wave was better than what the other sills gave. Not only this: this standing wave was of a uniformly perfect type for a very wide range of flow conditions. The sill had the merit of adjusting itself in depth to suit varying tail water levels up to a limit. But when the tail water was below this limit for a certain discharge it failed to produce a standing wave and functioned as a mere deflector. By making the steps steeper the range of usefulness of the sill without its becoming a deflector could be increased. A sill which would be effective for a wide range of tail levels would be very deep indeed.

SPRAY BLOCKS IN CISTERN

To see if the action of a sill could be improved by providing some kind of blocks in the cistern the design indicated in Figure 6C.5 (L) was tried. The usefulness of this addition was practically insignificant. For certain ranges of flow the spray blocks would alone function and for certain other ranges these would be redundant. For the ranges for which both were simultaneously functioning the blocks appeared to be making some contribution towards dissipation of energy, which was, however, not appreciable.

SLOPING FLOOR FOR BASIN

In all the above studies the floor of the stilling basin was horizontal. A few of these tests were repeated with the floor sloping down one in eight to the foot of the sill. The standing wave appeared to show some improvement in certain cases, the sloping floor apparently having the merit of adjusting its depths to suit a wide range of tail water elevations. This study is proposed to be pursued.

SCOUR AND ITS PREVENTION

In a number of experiments scours were traced and in Figure 6C.7 a few typical ones are plotted. These may now be discussed.

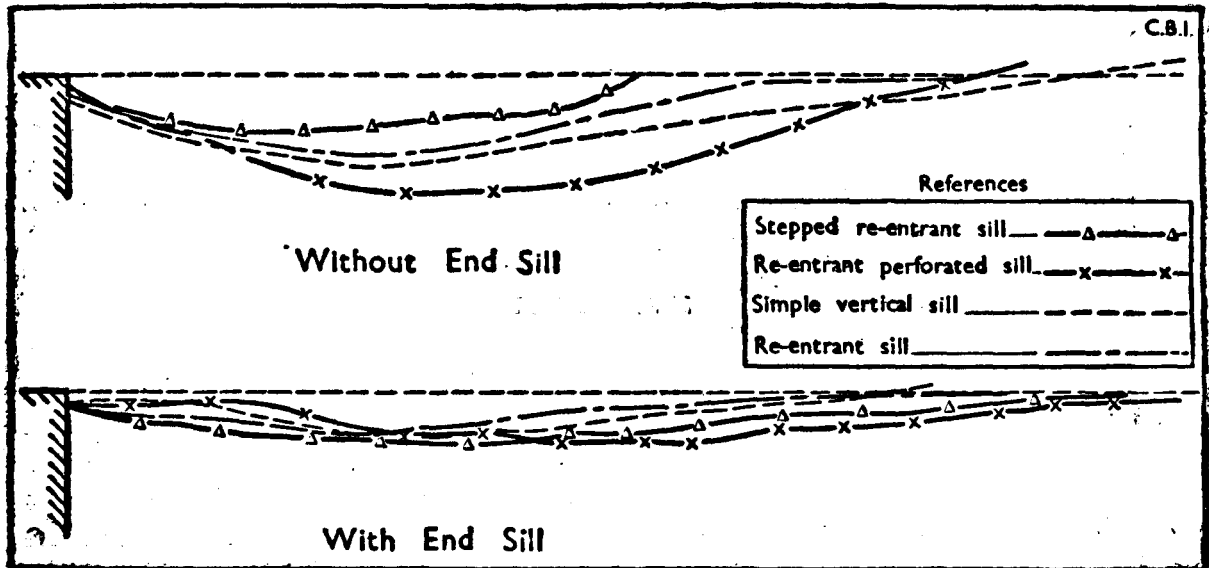


Figure 6 C. 7 : Longitudinal sections of scours with different type of sills

The longitudinal sections of the scours are shown superimposed in two sets. In the first set the scours obtained with (1) a simple vertical sill at the end of the basin, (2) a re-entrant sill, (3) a re-entrant perforated sill and (4) a stepped sill are shown and in the second set the scours obtained when an end sill was added to each of the above are shown. Considering the first set, *i.e.*, without an end sill the scour was least with a stepped sill. This could be anticipated from the fact that the standing wave with this sill was of a perfect type. The scours with a vertical sill and a re-entrant sill respectively were almost the same, the latter being less deep. By altering the dimensions of either we may get practically the same scour and the same standing wave profile with both types of sills. The scours with the re-entrant perforated sill was deeper than the above. One plausible explanation would be that the jets escaping through the perforations were not as efficiently stilled as the rest. Here again it would be possible to alter the dimensions and get more or less the same scour as obtained with a simple vertical or a re-entrant sill.

In all cases the addition of an end sill was a definite improvement. Adding an end sill to the apron having a stepped sill at commencement did not show marked improvement, because there was not much of a scour to reduce. In all the other three cases where the standing waves over the first sill were not perfect and had given rise to secondary standing waves the improvement effected by the end sill was marked.

CONCLUSIONS

The inferences discussed above may now be summarized—

(i) In designing stilling basins for spillway dams careful consideration should be given to the varying conditions of discharge and downstream water level that may obtain at the structures. A stilling basin which may ensure a

perfect standing wave being formed on the apron for a certain set of conditions may not be so effective under certain other conditions and may altogether fail in still other conditions. The length and depth of the basin and the type of sill to be provided at the end of the basin should be so designed for each particular case as will be satisfactory for all ordinary conditions of flow and for most of the conditions that may obtain extraordinarily.

(ii) A stilling basin having a vertical sill at its end is the simplest design. By making the basin sufficiently deep or by raising the sill sufficiently high above the apron a standing wave may be forced to form on the apron for the worst possible condition, *viz.*, maximum discharge and minimum tail water level. With a sill of this type a perfect standing wave will be obtained for a few conditions of flow. For most other conditions the standing wave will rise above the tail water level and dive down to meet the latter, forming a second standing wave. One objectionable feature about this sill is that it has to bear the direct impact of the shooting jet for conditions of low tail water level.

(iii) Curving the vertical sill and giving it a slope towards upstream minimises such direct impact. In the matter of forming the standing wave it has the same effect of a high baffle. In other words when a re-entrant sill is used instead of a plain vertical sill the depth of the basin can be reduced. The weak point about this design is the tip of the sill whose construction and maintenance will require very special care and expense.

(iv) By providing large perforations in a re-entrant sill the height of standing wave can be lowered and impact on the sill relieved. The water issuing through the perforations, however, seem to retain a part of their kinetic energy undissipated. Structurally the design is complicated and may be costly.

(v) When the sill is kept with a batter towards the downstream side it acts as a deflector and no standing wave is formed except under very few conditions of tail water levels. Efforts to modify a deflector by providing it with teeth bars, *etc.*, with a view to making it effective in creating standing waves for a wide range of flow conditions did not prove successful.

(vi) A stepped sill is a great improvement on a deflector and when correctly designed will create a perfect standing wave for a wide range of flow conditions. But the range is not half as wide as obtained with a vertical sill and with low tail waters a stepped sill will be no better than a deflector.

(vii) Addition of spray blocks upstream of stepped sill did not indicate any marked improvement in the matter of preventing deflection at the stepped sill or of energy dissipation.

(viii) The scour formed downstream of the structure was the least when the standing wave was of the perfect type as obtained with the stepped sill. The scours obtained with the other sills were more or less same or could be made more or less same by modifying the dimensions. By adding a second sill at the end of the apron further reduction of scour could be effected.

(6) ROLLER BUCKET FOR THE RAMAPADASAGAR
OVERFLOW DAM (7)

ABSTRACT

The spillway section of the Ramapadasagar Dam is designed to have a height of about 150 feet above the river bed at site and to discharge a total of nearly 2.4 million cusecs with 30 feet depth of flow over crest. The design is shown in Figure 6C.3. The river bed is sandy for great depths and the foundation of the dam will extend nearly 150 feet below bed before meeting firm rock. For the dissipation of energy below the spillway the usual devices of aprons and sills cannot be adopted as these will not have proper foundations and will be prohibitively costly. The only suitable type of dissipator appears to be the "Roller bucket" which can be formed on the face of the dam itself, as a part of it and supported by it. The elevation, radius, and shape of the bucket for efficient disposal of floods, minimum scour below the structure and security of the bucket itself against abrasion, cavitation, etc., had to be determined and this was taken up for investigation on hydraulic models. The present report embodies the results obtained in the series of experiments carried out so far on a number of models. There were two sets of models, the first set consisting of two dimensional models to scale 1/60 fitted up in an out door flume and the other set consisting of similar models to scale 1/200 fitted up in the glass sided hydraulic flume inside the laboratory.

DESCRIPTION OF THE 1/60 MODEL

The model of the spillway was constructed in masonry and plastered smooth between the parallel walls of a masonry flume 25 feet long two feet wide and five feet deep. At the head of the flume was a stilling chamber into which water at constant head was supplied and maintained by means of inlet sluice valves and overflow arrangements. At the end of the flume was a tail bay with gate which enabled the correct downstream levels to be maintained. The portion of the flume downstream of the model bucket was packed with sieved sand of 14—18 mesh to a depth of $1\frac{3}{4}$ feet with its top levelled off to correspond to +30.0. The model was run for a period of 45 minutes in each experiment, the profile of the water surface was taken during the run and the longitudinal section of the scour was traced at the close of the run.

Two designs, namely, (1) a deep bucket with invert at ± 0 and radius 50 feet and (2) a shooting bucket with invert at +24.0 and radius, 80 feet. shown in Figure 6C.8 (A) and (B) respectively, were tested in these models. The effect of cutting down the tip of the shooting bucket to various inclinations on the size and type of the standing wave was qualitatively observed in a separate experiment by replacing the tip with a hinged wooden plank.

DISCUSSION OF RESULTS

The standing wave profiles and scours observed on the above two models for conditions corresponding to maximum water level upstream (+194.0) and the downstream (+90) are exhibited in Figure 6C.9. With the deep bucket the standing wave was higher and the scour was deeper than with the shooting

(7) Irrigation, Research Station Poondi, Madras, Annual Report, 1946, pages 70 to 79.

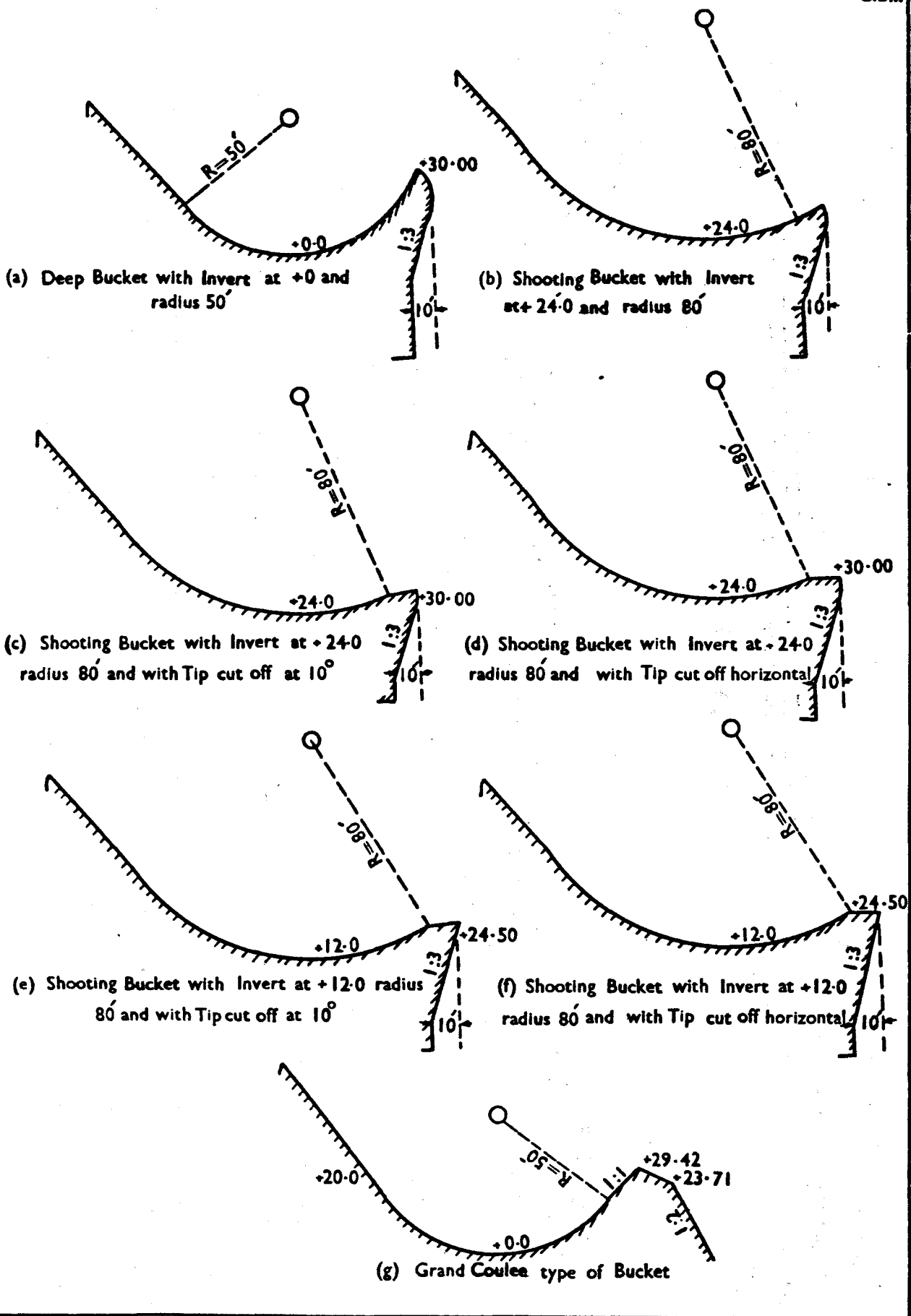


Figure 6C.8:- Showing Design of Buckets (Ramapadasagar Spillway Dam)



bucket. The standing wave in the former case was found oscillating about the longitudinal axis of the flume presumably due to the excessive depth of tail water (especially before the full development of the scour) which had to be balanced by the hypercritical stream to maintain the standing wave. With the shooting bucket no such condition was noticed. However, the standing wave appeared to be rather high.

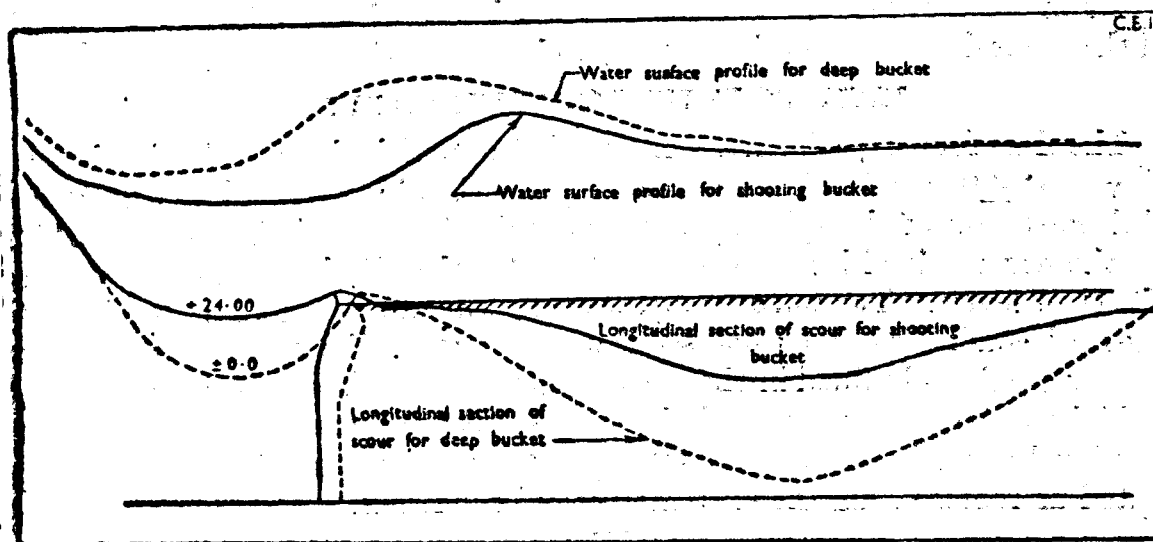


Figure 6C.9: *Longitudinal section of standing wave profiles and scours*

Observations made with the hinged board forming the tip of the bucket indicated that by cutting down the tip, the height and efficiency of the standing wave could be improved. Within limits, the smaller the angle which the tip made with the horizontal the better the result. Cutting down the tip too much, however, resulted in the standing wave being thrown farther down to the sandy bed, which, of course, could not be contemplated.

Thus on the whole, it was inferred that a bucket of the shooting type with its tip cut down to nearly horizontal would prove suitable. Further studies were then taken up on the smaller model fitted inside the glass flume.

DESCRIPTION OF THE $\frac{1}{100}$ MODEL

The glass sided hydraulic flume of the laboratory which was used for these experiments has an overall length of 40 feet and uniform width of two feet. For the first 12 feet length, it is three feet deep and this includes the eight inches diameter inlet pipe, the two feet wide, sharp edged precalibrated gauging weir and necessary stilling arrangements. For the remaining length the flume has a uniform depth of two feet and this section includes in addition to the model the tail gate and outlet pipe. A trolley traversing this portion of the flume is provided and the trolley and its rails are equipped with vernier scales and gauges and this equipment is used for tracing water surface profile, and scour contours correct to 1/1,000 feet. Gauge points are also provided for measuring water levels upstream of the gauging weir, and upstream and downstream of the model. Water supply to the flume is drawn from an overhead tank 20 feet \times 20 feet \times 5 feet which is supplied by continuous pumping.

The models of the spillway and bucket were made in well seasoned teak-wood, polished, smooth and fitted up between the glass sides in the center of the flume. The portion of the flume in rear of the model bucket was packed with sand of 24—30 mesh per inch grade, to a depth of about $7\frac{1}{2}$ inches and top of the sand bed was levelled off to correspond to + 20.0. The level of the bottom of the flume was thus about -100.0. The time adopted for each experimental run was four hours which was found sufficient to develop the maximum scour below the bucket. For each run, the standing wave profile and the scour details were taken and flow characteristics recorded.

PRELIMINARY TESTS ON A DEEP BUCKET

The first four experiments were carried out with the deep bucket having its invert at ± 0.0 and radius 50 feet Figure 6C.8 (A). Water levels upstream and downstream of the spillway were maintained as given in Table 6C.2 which were the normal flood levels corresponding to the discharges noted against each set of levels. The purpose of these experiments was to see for what set of normal conditions, the worst scour was produced. The standing waves and the scour profiles obtained in these experiments are shown in Figure 6C.10. The highest standing wave and the deepest scour occurred under maximum water level conditions. For all subsequent experiments this was, therefore, adopted as representing the extreme normal condition.

TABLE 6C.2

Ramapadasagar Spillway—Bucket studies—Statement of water levels for various discharges

Discharge	Front water level	Rear water level
2.25 million cusecs	+194.00	+90.00
1.85 " "	+190.00	+88.00
1.36 " "	+185.00	+85.50
0.91 " "	+180.00	+83.50
0.53 " "	+175.00	+73.00
0.22 " "	+170.00	+62.50

The model was run in the fifth experiment with maximum water level upstream and with a low water level (+53.0) downstream. The scour extended down to the flume bottom. This of course was an exceptionally abnormal condition. However, the various designs tried were tested for this condition also representing an exceptional case.

GRAND COULEE TYPE BUCKET

This is shown in Figure 6C.8 (G). The invert and radius were the same as those in the case of the deep bucket discussed above; the only difference was in

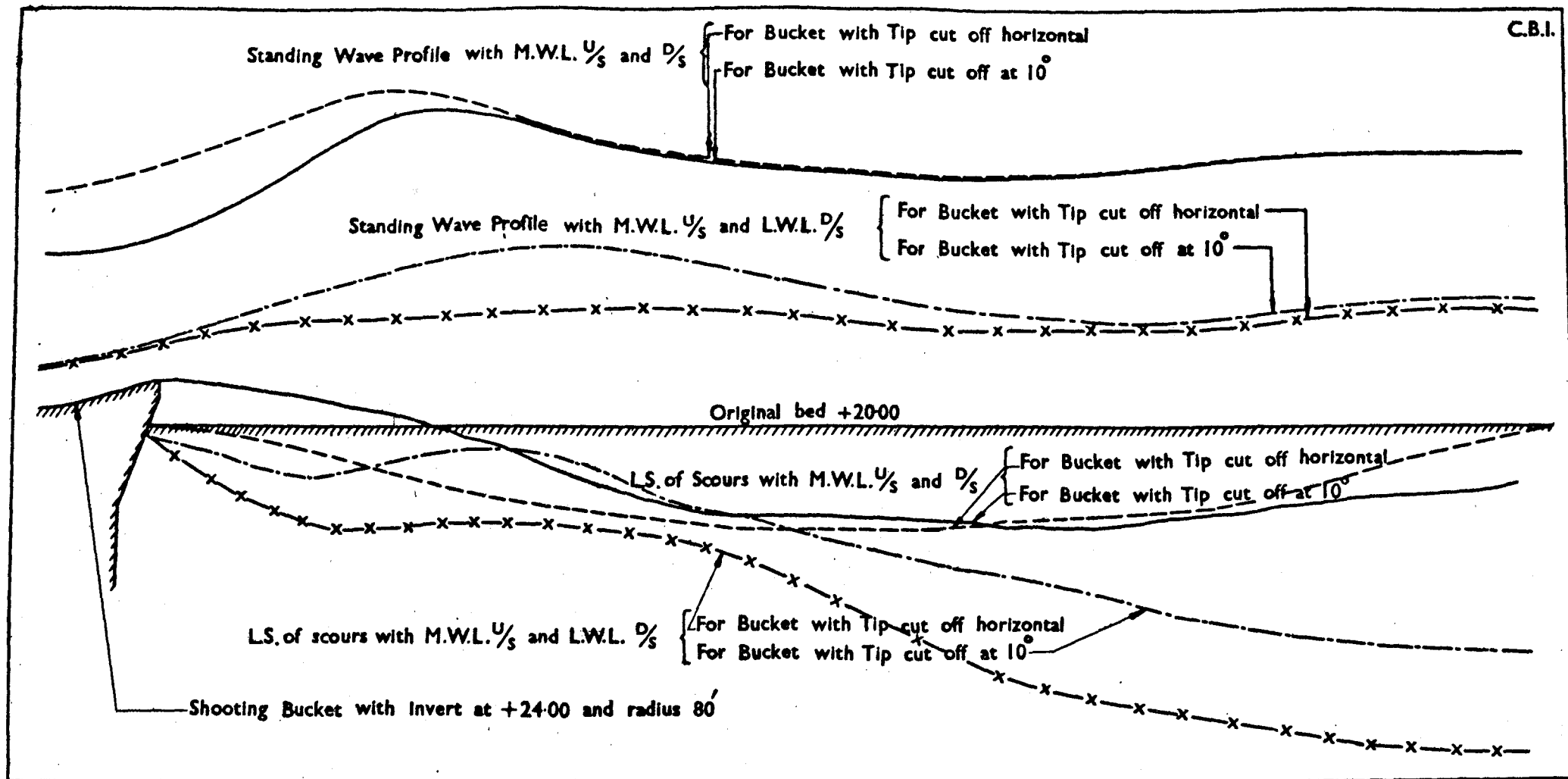


Figure 6C.11:- Showing Standing Wave Profiles and Scour Sections
[Ramapadasagar Spillway model $\frac{1}{200}$]

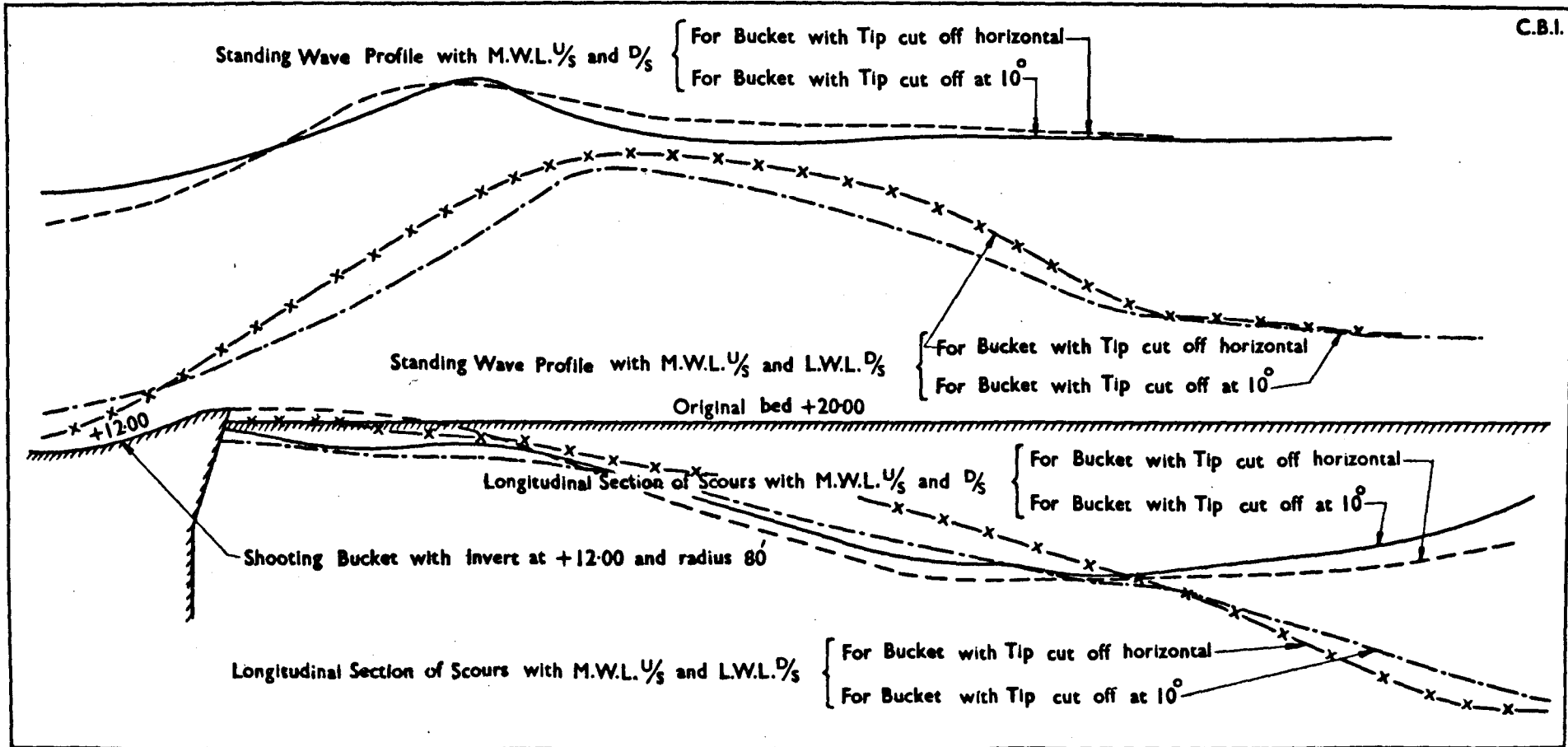


Figure 6 C.12:- Showing Standing Wave Profiles and Scour Sections
[Ramapadasagar Spillway model $\frac{1}{100}$]

the shape of the bucket beyond the tip. Trying the design on the model no difference was noticed in the standing wave profile, depth of scour or in the general flow characteristics between this and the deep bucket. The remarks made above regarding the oscillations of the standing wave were observed to apply to this design also. The design might be rejected.

SHOOTING BUCKET WITH TIPS CUT OFF

Next came the following four designs :—

- (i) Bucket with invert at $+24.0$ radius 80 feet and ten feet length of the tip cut off to have a slope of ten degree with the horizontal Figure 6C.8 (C).
- (ii) Same bucket with ten feet length of the tip cut off horizontal Figure 6C.8(D).
- (iii) Bucket with invert at $+12.0$ radius, 80 feet and ten feet length of the tip cut off to a slope ten degree with the horizontal Figure 6C.8 (E).
- (iv) Same bucket with ten feet length of the tip cut off horizontal Figure 6C.8 (F).

These four designs were tried for conditions corresponding to maximum water level upstream and downstream and also to maximum water level upstream and lowest water level downstream. The standing wave profiles and the longitudinal sections of the scours obtained in each experiment are shown in Figure 6C. 11 and 6C. 12. Items (i) and (iii) were generally satisfactory, but the former appeared to be better of the two. Salient features observed on these models are discussed below.

STANDING WAVES AND ROLLERS

The general characteristics of the standing waves, surface and ground rollers and the consequent development of scours were similar for items (i) and (iii) and the only differences, noticed consisted in the heights of standing waves and the depths of scour.

Under maximum water level conditions upstream and downstream a fairly good standing wave formed just over the tip of the bucket, giving rise to efficient surface rollers inside the bucket. The standing wave rose a little above the tail water level and dived down causing a secondary standing wave. Scour started where the diving jet struck the bed and developed towards upstream and downstream. The scoured sand carried towards the bucket by the ground roller action travelled up the slope of the scoured bed to the tip of the bucket. This action might cause wear on the bucket tip. Excepting this, no abrasive action was noticed. No sand entered the bucket proper. The downstream face of the bucket did not get exposed. The secondary wave formed

downstream of the main standing wave gave rise to a surging flow which persisted till the bed had scoured to some extent after which the flow got steady and smooth.

With maximum water level upstream and lowest water level downstream no proper standing wave was formed. The jet shot beyond the bucket and touched the bed about 200 feet downstream in a dive starting a deep but flat scour. There was considerable agitation and consequent surging flow. Scour developed rapidly. Sand carried by the ground rollers moved up to the tip of the bucket and was then thrown back. No sand entered the bucket and the downstream face remained unexposed to abrasive action.

The conditions observed with the designs (ii) and (iv) were generally similar to the above. Under normal conditions upstream and downstream, proper standing waves and good rollers were obtained. When the tail water was low shooting flow occurred and the hypercritical jets dived down to the bed scouring it deep. With regard to the movement of the sand carried up by the ground rollers it was noticed that the sand travelled up to the tip of the bucket [in both cases (ii) and (iv)] and then along the horizontal portion and then alone it was shot back away from the bucket. This might cause abrasion of the flat tip.

Comparing the heights of the standing waves for the various designs tried for maximum water level conditions upstream and downstream shown in Figures 6C.10, 6C.11 and 6C.12, it may be noticed that the higher the invert of the bucket the lower the maximum height of standing wave. Clearly the depth of water obtaining at Polavaram in rear of the spillway for high floods appeared considerable and did not call for any deepening of the bucket. This explained why the scours obtained with the shallower buckets were not so deep as with the deep ones.

SCOURS

Under normal maximum water level conditions the nature and depth of scour obtained were more or less the same for items (i) and (ii). Depth for items (i) and (ii) were less than for items (iii) and (iv). With maximum water level upstream and lowest water level downstream scours were very deep for all the four items and went down to the bottom of the flume corresponding to -100.0 . In these scours the slope of the bed downstream of the bucket might be taken as the criterion for comparison. Viewed thus the scours for items (i) and (ii) were similar and more favourably shaped than scours for items (iii) and (iv) which latter were again similar between themselves. Table 6C.3 gives details of scours obtained on the several models in a convenient form. Considering scours alone as the criterion for selection, the choice would thus be in favour of the bucket with invert at $+24.0$. The cutting down of the tip from ten degrees inclination to horizontal did not materially influence the scour.

TABLE 6C.3
Ramapadasagar spillway dam—Model studies on bucket design—
Statement giving details of scours and other particulars

Serial No.	Number and description of bucket	Water level— Reduced levels		Scour		Remarks
		Up-stream	Down-stream	Reduced level of deepest part	Distance of the deepest part beyond bucket tip in face (feet)	
1		2	3	4	5	6
ON 1/60 SCALE MODEL						
1	Deep buckets with radius 50 feet and invert at +0.0.	+194.0	+90.0	-43.3	180	Standing waves high and unsteady—Scour deep and close to structure. Abrasion at tip only.
2	Shooting bucket with radius 80 feet and invert at +24.0.	+194.0	+90.0	±0.0	180	Standing waves better—Scourless deep and more flat—Abrasion at tip.
3	Shooting bucket as above but with a hinged tip.	Lowering of the tip improved the Standing wave.
ON 1/200 SCALE MODEL						
4	Deep bucket with radius 50 feet and invert at ±0.0.	+194.0	+90.0	-52.0	234	Out of the four normal conditions of upstream and downstream water levels tried in items 4—7 the first, viz., upstream and downstream having maximum water level gave the worst combination.
5	Do.	+190.0	+88.0	-40.8	184	
6	Do.	+185.0	+85.0	-5.4	182	
7	Do.	+180.0	+83.0	+14.2	220	
8	Shooting bucket with radius 80 feet and invert at +24.0 and tip of bucket cut off to ten degrees with horizontal.	+194.0	+90.0	-6.8	234	Recommended as the best design—Low Standing wave and low scour—Abrasion at tip only.
9	Do.	+194.0	+53.0	Scours extends down to flume bottom—Slope 1 vertical: 6 horizontal.

TABLE 6 C.3—*contd.*

Serial No.	Number and description of bucket	Water level— Reduced level		Scour		Remarks
		Up stream	Down stream	Reduced level of deepest part	Distance of the deepest part beyond bucket tip in face (feet)	
1	2	3	4	5	6	
10	Shooting bucket with radius 80 feet and invert at +24.0 and with tip cut off to horizontal.	+194.0	+90.0	-8.4	184
11	Do.	+194.0	+53.0	Scour extended to flume bottom slope 1 vertical: 4.5 horizontal.
12	Shooting bucket with radius 80 feet and invert at +12 with tip cut off to ten degrees with horizontal.	+194.0	+90.0	-23.4	234
13	Do.	+194.0	+53.0	Scour extended to flume bottom slope 1 vertical: 6 horizontal.
14	Shooting bucket with radius 80 feet and invert at + 12 with tip cut off to horizontal.	+194.0	+90.0	-20.8	234
15	Do.	+194.0	+53.0	Scour extended to flume bottom slope 1 vertical: 6 horizontal.

CONCLUSIONS

The inferences may be summed up as follows :—

- (1) The depths of tail water for all normal conditions of flood discharge appeared sufficiently large to create a proper standing wave with a shallow bucket. A deep bucket with invert at +0.0 and radius 50 feet gave an excessively high standing wave causing a deep scour near the structure. Lateral oscillation of the standing wave was noticed inside the flume under certain conditions. The design appeared definitely unsuitable.
- (2) With the other types it was noticed that the higher the invert the lower the height of the standing wave and the less deep the scour. A bucket having a radius of 80 feet and invert at + 24.0 was generally satisfactory
- (3) The standing wave in the above design could be improved further by cutting down the tip or the bucket. A design with ten feet length of the tip cut down in a slope of ten degrees with the horizontal was found to be the best. Cutting down the tip further to horizontal did not materially improve the standing wave and this also indicated the likelihood of abrasion of the horizontal bit.

- (4) Thus a bucket of 80 feet radius with invert at +24.0 and having ten feet length of its tip cut down to slope at ten degrees above horizontal Figure 6C. 8(C) was found best. With this a proper standing wave with good rollers was formed for all normal conditions of flow. The maximum scour with maximum water level upstream and downstream was 27 feet and formed 234 feet downstream of bucket tip. With maximum water level upstream and lowest water level downstream no standing wave was formed and a shooting jet obtained. The scour in this case was very deep, but it was also far away from the structure. The slope of the scoured bed below bucket tip was about one vertical to six horizontal.

(7) SIPHON SPILLWAYS FOR PAHUJ WEIR (8)

A battery of saddle siphons with a volute priming siphon was designed for Pahuj weir at Lucknow Research Station. (9). Since the project is still under consideration, model experiments were continued to test the possibilities of further improvements.

(A) SADDLE TYPE SIPHON

The three inches lip was reduced to $1\frac{1}{2}$ inches. This minimised the region of turbulence and cavitation in the vicinity without materially affecting the priming depth and co-efficient of discharge. The full details are shown in Figure 6C.13.

Table 6C.4 gives the priming depth and co-efficient of discharge of saddle siphon under four different conditions :—

TABLE 6C.4

Serial No.	Detail of Experiments	Priming Depth (inches)	Co-efficient of discharge
I	Saddle siphon with $1\frac{1}{2}$ inches lip and with volute priming siphon in operation	7	0.66
II	Saddle siphon with $1\frac{1}{2}$ inches lip and without volute priming siphon	11	0.66
III	Saddle siphon without $1\frac{1}{2}$ inches lip and with volute priming siphon in operation	11	0.70
IV	Saddle siphon without $1\frac{1}{2}$ inches lip and without volute priming siphon.. .. .	19	0.70

NOTE :—The outlet bend is provided with a sealing cistern in all cases.

From the above table, it is apparent that a lip improves the priming qualities considerably. The co-efficient of discharge is affected but slightly. The provision of $1\frac{1}{2}$ inches lip is, therefore, desirable for Pahuj siphons in view of close regulation and consideration of safety of the weir.

(8) United Provinces, Public Works Department, Irrigation Research Station, Annual Report on Research Progress, 1946, Technical Memorandum No. 17, pages 46 to 64.

(9) United Provinces, Public Works Department, Irrigation Research Station, Annual Report on Research Progress, 1945, Technical Memorandum, No. 16 pages 24 to 26.

Observation of Pressures

Pressures were next observed at several points throughout the siphon conduit to study the nature of flow. A large number of pressure pipes were inserted in the side wall at sections AA, BB, etc., *vide* Figure 6C.14 and in the inner and the outer shells of the siphon model (scale 1/18) *vide* Figure 6C. 14. The pipes were made absolutely leak proof, the inside edges being thoroughly smoothed and flush with concrete surface to ensure accurate results. The actual pressure head was determined by measuring the suction head in a U-tube, one end of which was connected to the point concerned, the other being open to the atmosphere. The observation from the model and the results derived therefrom have all been converted for the prototype to afford an easy means of comparison with those in the actual siphon installation.

Tables, 6C.5 and 6C.7 give vertical distances of outer and inner shells and the side wall pressure pipes respectively above or below crest. Tables 6C.6 and 6C.8 show the actual observed pressure at these points in the saddle siphon. Figure 6C.15 indicates the distribution of pressure in the outer and inner shells. Figure 6C.16 depicts the pressure contours within the siphon barrel at 4.0 feet difference of head.

TABLE 6C.5

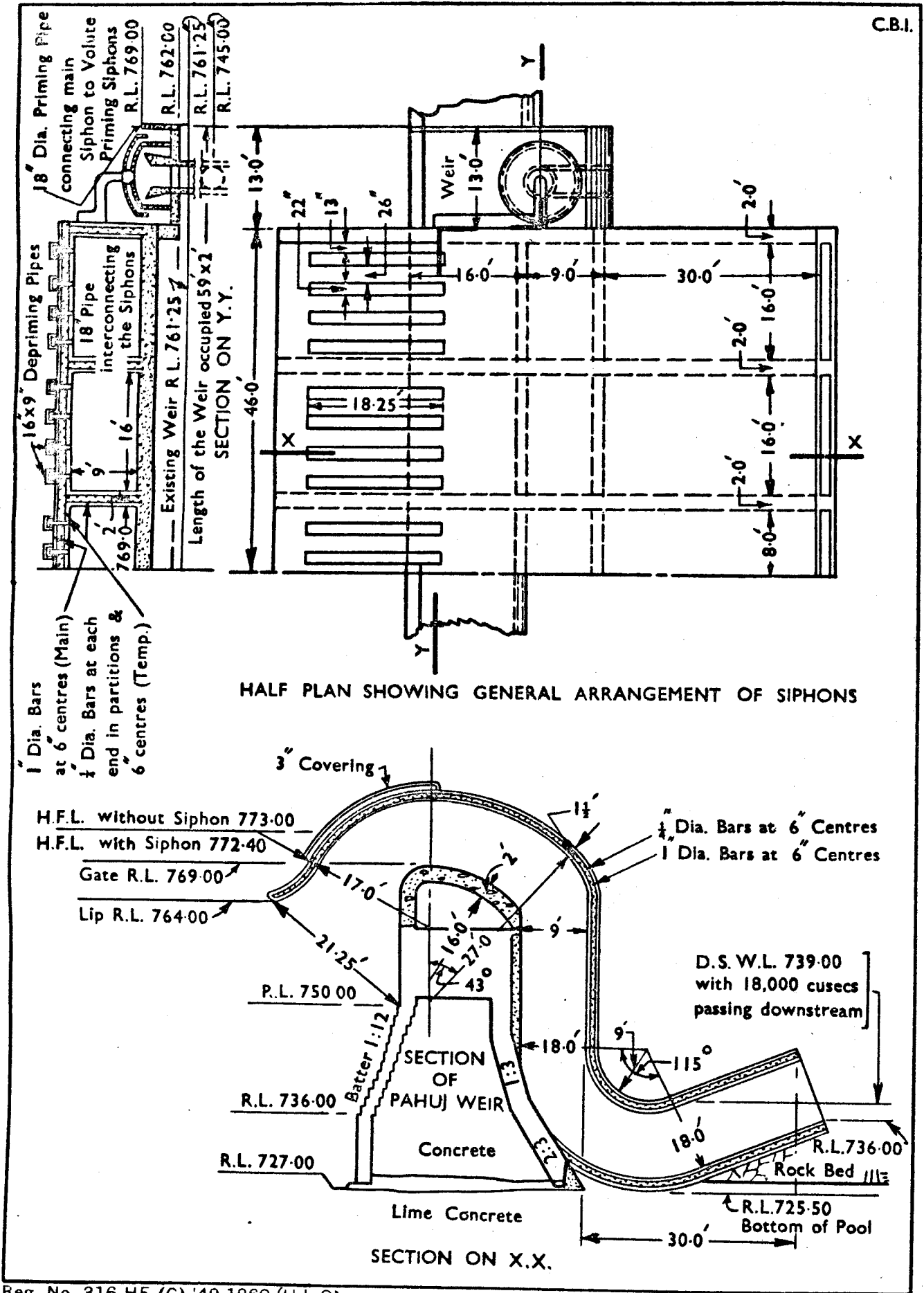
*Vertical distances of Outer and Inner shell pressure pipes above or below crest
(-) sign denotes above crest, and (+) sign denotes below crest*

Place of Observation in the siphon conduit	Pressure pipes No.										
	1	2	3	4	5	6	7	8	9	10	11
Outer shell	9.00	-9.0	-6.0	5.0	10.0	15.0	20.0	24.0	31.0	32.0	27.0
Inner shell	0.00	1.50	5.0	10.0	15.0	20.0	24.0	39.0	40.0

TABLE 6C.6

Pressure observations in the Outer and Inner shell of Saddle siphon conduit

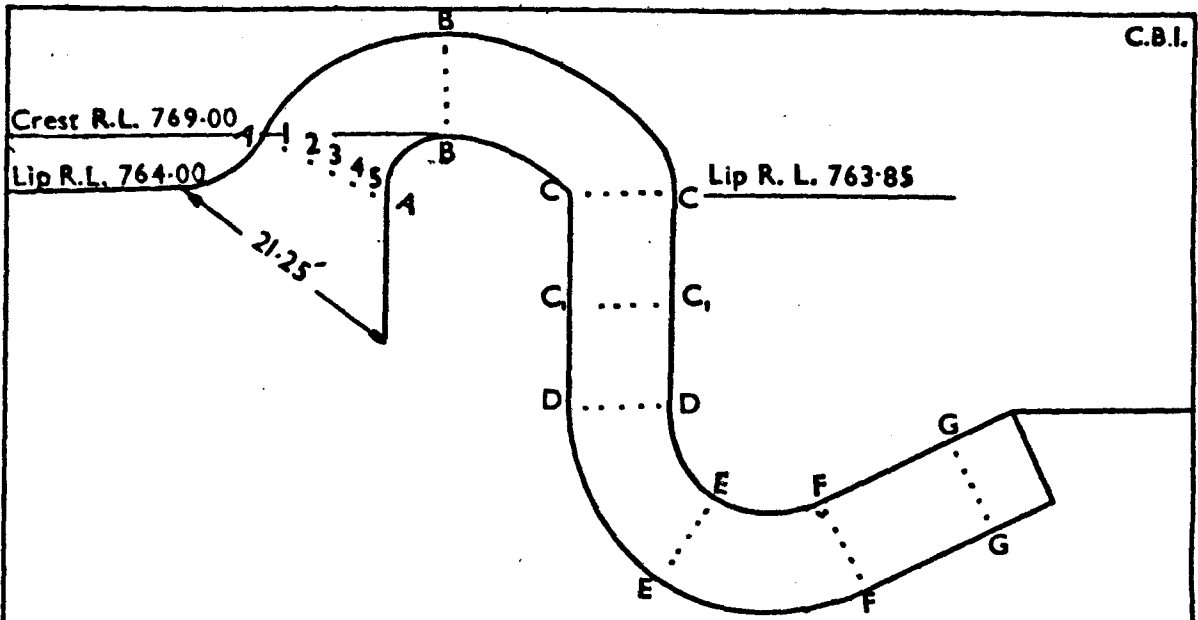
Place of Observation	Pressure in pipe No.										
	1	2	3	4	5	6	7	8	9	10	11
Outer shell	-5.09	-17.83	-17.19	-5.99	-8.55	-8.65	-9.0	-11.79	1.35	3.29	1.62
Inner shell	-26.64	-15.39	-28.8	-24.00	-18.0	-7.20	2.0	23.04	22.24



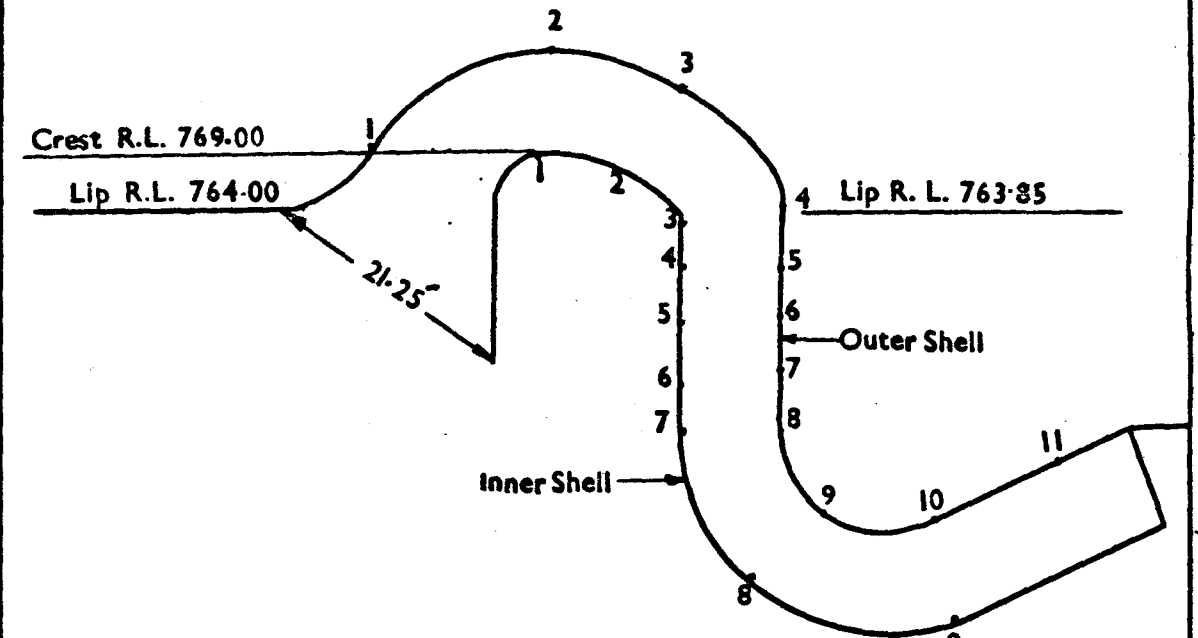
Reg. No. 316 HE (C) '49-1260 (H.L.O)

Figure 6 C. 13:— Design of Saddle type Siphon for Pahuj Weir





(a) Showing position of Pressure Pipes in right side wall



(b) Showing position of Pressure Pipes in Inner and Outer Shells

Figure 6C.14: Showing Saddle Type Siphon for Pahuj Weir



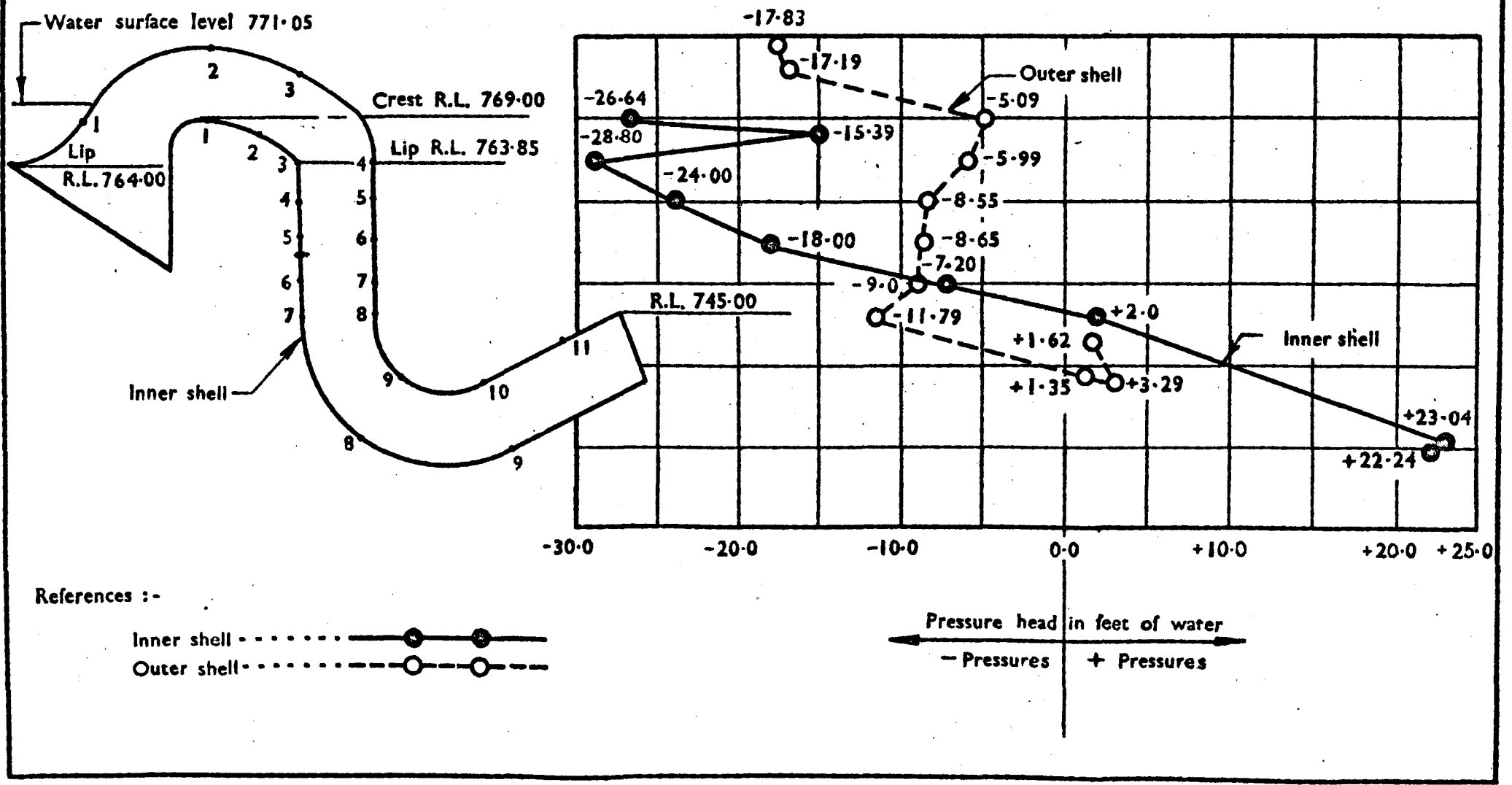
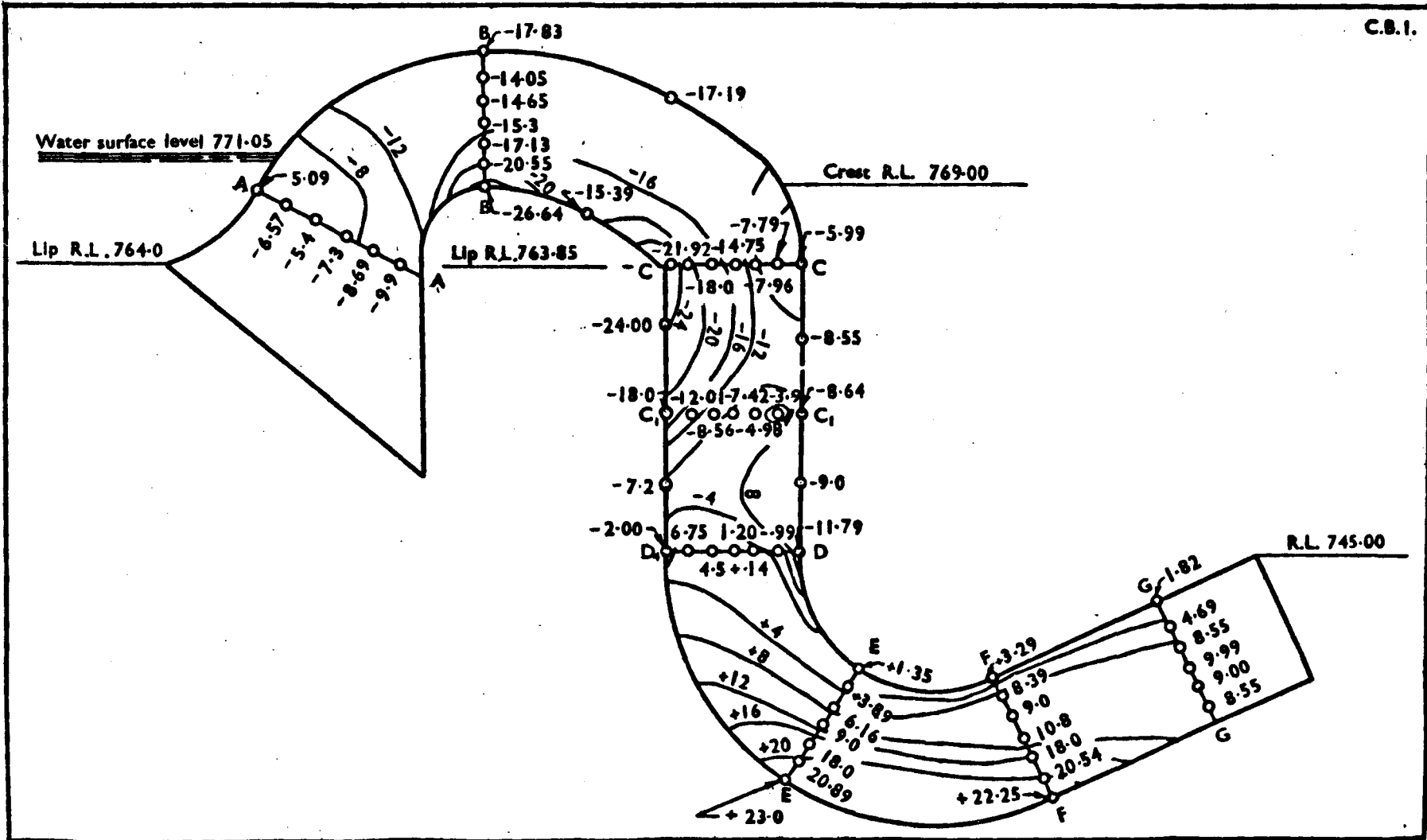


Figure 6C.15 :- Showing Distribution of Pressure In Inner and Outer Shell
 [Saddle type Siphon for Pahuj weir]



C.B.I.

Figure 6C.16 :- Saddle Type Siphon for Pahuj Weir showing Pressure Contours within Siphon Barrel
 [Drawn at 4.0 Change of Head, Atmospheric Pressure 0.0]

TABLE 6C.7

Vertical distances of side-wall pressure pipes above or below crest (—) sign denotes above crest, (+) sign denotes below crest

Section where observation were taken.	Pipe No.				
	1	2	3	4	5
AA	1.0	2.0	3.0	4.0	5.0
BB	—7.5	—6.0	—4.5	—3.0	—1.5
CC	5.0	5.0	5.0	5.0	5.0
C ₁ C ₁	15.0	15.0	15.0	15.0	15.0
DD	24.0	24.0	24.0	24.0	24.0
EE	33.0	34.20	35.50	36.70	38.0
FF	33.40	34.80	36.20	37.60	39.0
GG	28.0	29.20	30.50	31.80	33.20

TABLE 6C.8

Pressure observations in the right wall of saddle siphon barrel

Section where pressures were observed	Pressure in pipe No.				
	1	2	3	4	5
AA	—6.57	—5.40	—7.30	—8.69	—9.90
BB	—14.05	—14.55	—15.30	—17.19	—20.56
CC	—7.79	—7.96	—14.75	—18.0	—21.92
C ₁ C ₁	3.91	—4.94	—7.42	—9.55	—12.01
DD	—0.99	0.14	1.94	4.50	6.75
EE	3.89	6.16	9.00	18.00	20.5
FF	3.39	9.00	10.80	18.00	20.54
GG	4.69	8.55	9.99	9.00	8.55

The following conclusions have been derived :—

(i) The maximum negative pressure at the crest and lip of the siphon are of the order of—26.04 feet and —28.80 feet respectively. Thus there seems possibility of slight cavitation in the neighbourhood of the lip.

(ii) The pressure in the siphon leg changes its character from suction to positive pressure, almost at the section DD, in line with outlet R.L. 745.0, where the upper filaments of the issuing jet of water first make contact with the atmosphere. This shows that the head

TABLE 6C.9

Loss of pressure head (in feet of water) at various section of Saddle siphon conduit. Water depth over crest—2.05 feet

Sections	Loss of pressure head in pipe No.					Mean loss of pressure head
	I	II	III	IV	V	
AA	9.62	9.45	12.35	14.74	16.95	12.82
BB	8.60	10.60	12.85	16.24	21.11	16.45
CC	14.84	15.01	21.80	25.05	28.97	21.12
C ₁ C ₁	20.96	21.99	24.47	26.60	29.06	24.61
DD	27.04	25.91	24.11	21.55	19.30	23.58
EE	31.16	30.09	28.55	20.75	19.16	25.93
FF	27.06	27.85	27.45	21.65	20.51	25.59
GG	25.36	22.70	22.56	24.85	26.70	24.62

producing flow should not be taken as the difference between upstream water surface and the top of the issuing stream, but the difference between the upstream water surface and a point in this particular case, 1.04 feet below the surface of the issuing jet.

Conclusion (ii) above is in conformity with the findings of Davies ⁽¹⁰⁾ and it was considered desirable to test other fundamental laws of siphon flow

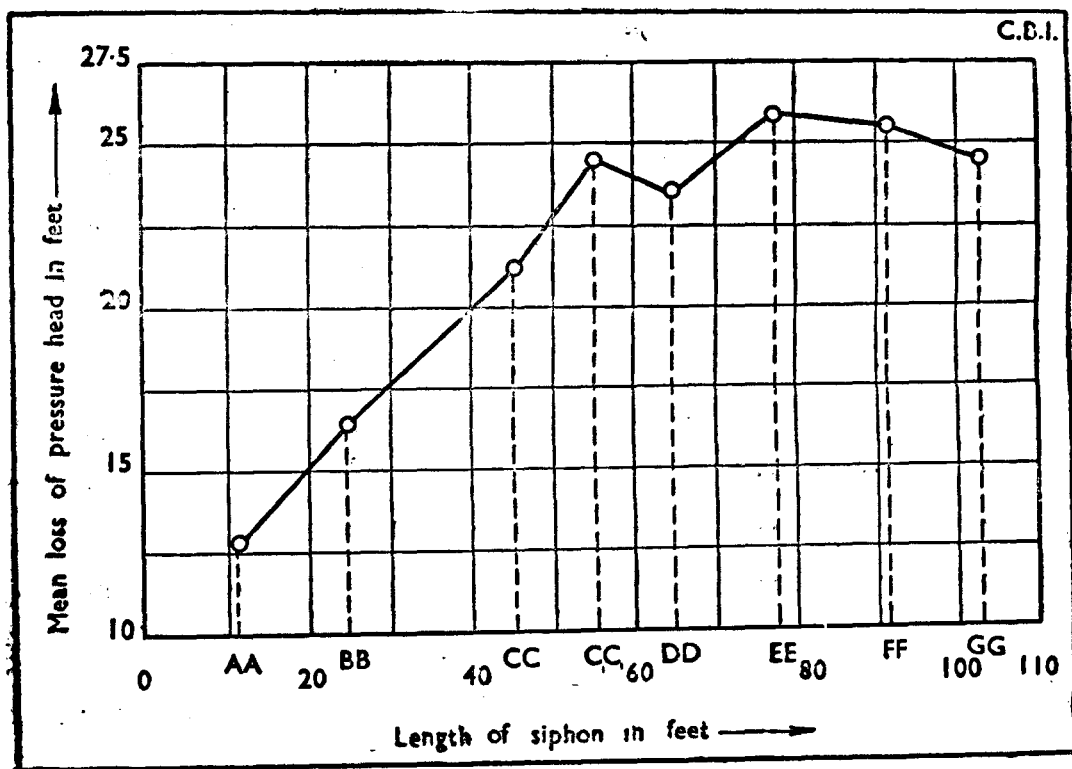


Figure 6C.17 : Showing variation in the mean loss of pressure head within siphon barrel

¹⁰⁾ Davies :—The laws of siphon flow Minutes of Proceedings of the Institution of Civil Eng (ineers, Vol. 235, 1934, pages 352 and 361.

enunciated by him. His first law is self evident and needs no verification. His second law is borne out from the pressure contours *vide* Figure 6C.16, clearly indicating the free vortex flow. To verify his third law losses of pressure head were worked out with respect to upstream water level for pressure pipes at various sections AA, BB, etc., of the side wall, and the mean loss of pressure head Table 6C.9 plotted *vide* Figure 6C.17 for the whole length of siphon barrel. It is clear from the above graph that:—

- (i) The mean loss of pressure head up to $C_1 C_1$ is greater than the loss at DD. This increase could only be attributed to the surging action that is likely to take place owing to the slight contraction of the stream, partly due to the existence of the lip and partly due to the bend being a sharp one. As a result of this the actual speed of the water particles is greater than that required for axial flow with consequent local increase in kinetic energy.
- (ii) At the section DD, where the pressure in the siphon barrel changes from negative to positive, the mean loss of pressure head being 23.58 feet while the static head up to water level upstream is 26.05 feet. Also net loss downstream of section DD is (24.62 feet—23.58 feet) 1.04 feet which is supposed to represent the static head up to the assumed mean position of the partially submerged outlet. These results substantially confirm the third principal law of siphon flow enunciated by Davies.

It will be noted, however, that the above pressure head loss at section DD, is about ten per cent. lower as compared to the static head of water upstream. This discrepancy can either be due to the incorrect determination of the pressures, or to a difference between the side wall pressures and the actual mean pressures, obtaining in the stream or to both.

Co-efficient of loss in bends.—

Davies conducted a series of experiments to determine the mean co-efficients of the loss in pipe bends of square section ⁽¹¹⁾. He has claimed that such co-efficient were equally applicable to similar bends of a siphon conduit. Table 6C.10 gives the probable co-efficient of loss in the crown and outlet bends of the saddle siphon for Pauhuj weir as ascertained from Davies co-efficient.

TABLE 6C.10

Detail of Siphon Bend	Angle of Bend	$\frac{R}{D}$	K	Remarks
Crown Bend	47°	2.0	0.14	100 per cent. extra loss is assumed due to lip
	60°	1.5	9.19	
	43°	2.5	0.13	
	47°	0.5	0.20	
	Lip	..	0.20	
Outlet Bend	115°—22½'	1.5	0.27	

Where—

R is the radius of the bend up to the centre of the siphon barrel.

D is the depth of siphon barrel.

K is the probable co-efficient of loss.

⁽¹¹⁾ Davies :—The Flow of Water through rectangular pipe bends : Journal Institution of Civil Engineers, February 1936, page 90.

The co-efficient of loss in the crown and outlet bends can also be determined from the actual observed mean loss of pressure head with the help of the formula :—

$$K = \frac{2gL}{V^2}$$

Where K is the co-efficient of loss.

L is the observed loss of pressure head in the bend.

V is the observed mean velocity of flow.

Table 6C.11 gives K as obtained from the observed loss of pressure head.

TABLE 6C.11

Detail of Siphon Bend	Mean loss of pressure head up to	Mean loss of pressure head up to	Loss of pressure head in the bend L	$\frac{V}{2g}$	$K = \frac{2gL}{V^2}$
	Section CC	Section AA			
Crown Bend	21.13	12.82	8.31	10.33	0.86
	Section FF	Section DD			
Outlet Bend	25.59	23.58	2.01	10.33	0.20

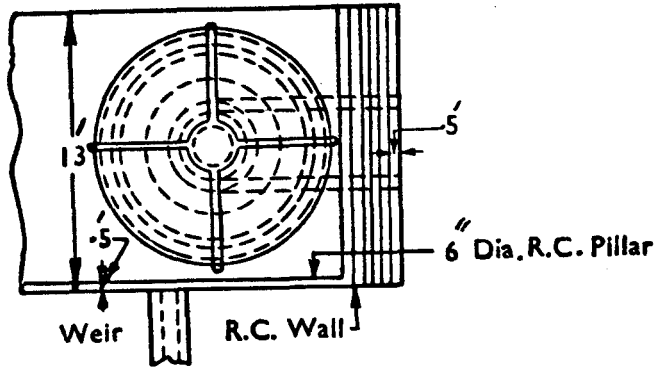
From the study of the Tables, 6C.10 and 6C.11 above, it will be apparent that the values of ' K ' as ascertained from Davies' results compare well with the values obtained from the actual observed head losses.

(B) Volute Siphon

Work on volute siphons for Pahuj was also continued this year and an improved design was evolved as per Figure 6C.18. This gave a priming depth of three inches and co-efficient of discharge 0.6. Circular and parabolic volutes were tried and ultimately the shape of volute as per Figure 6C.19 was adopted which gave optimum results.

Variation of co-efficient of discharge and priming depth with different outlet lengths.

A 1/8 model of siphon as per Figure 6C.18 was constructed to study the nature of flow in greater detail. In the first instance only the length of outlet was varied. The results of these experiments are given in Table 6C.12.



Proposed Battery of Saddle Type Siphon

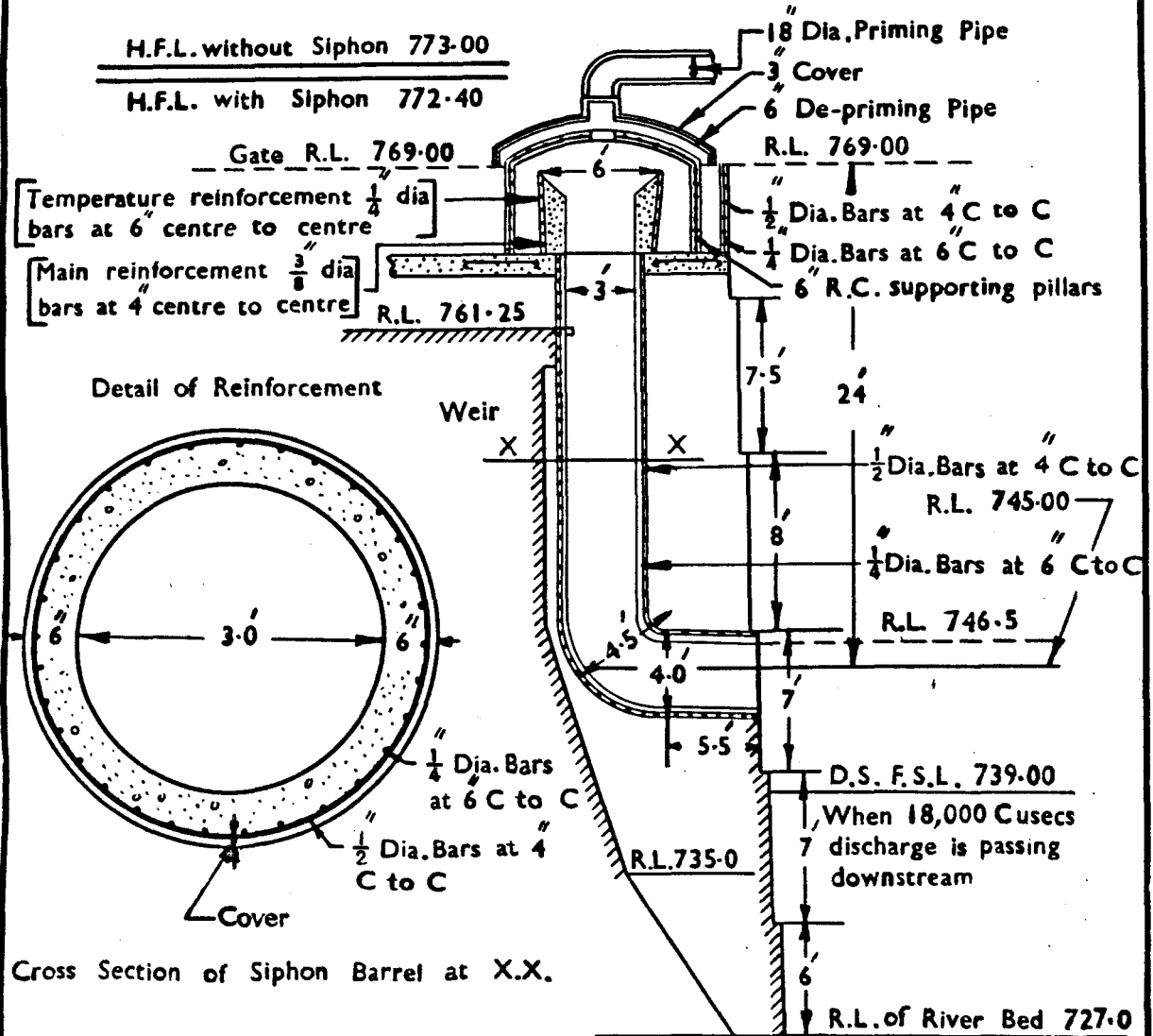


Figure 6C.18 :- Showing design of Volute Priming Siphon for Pahuj Weir

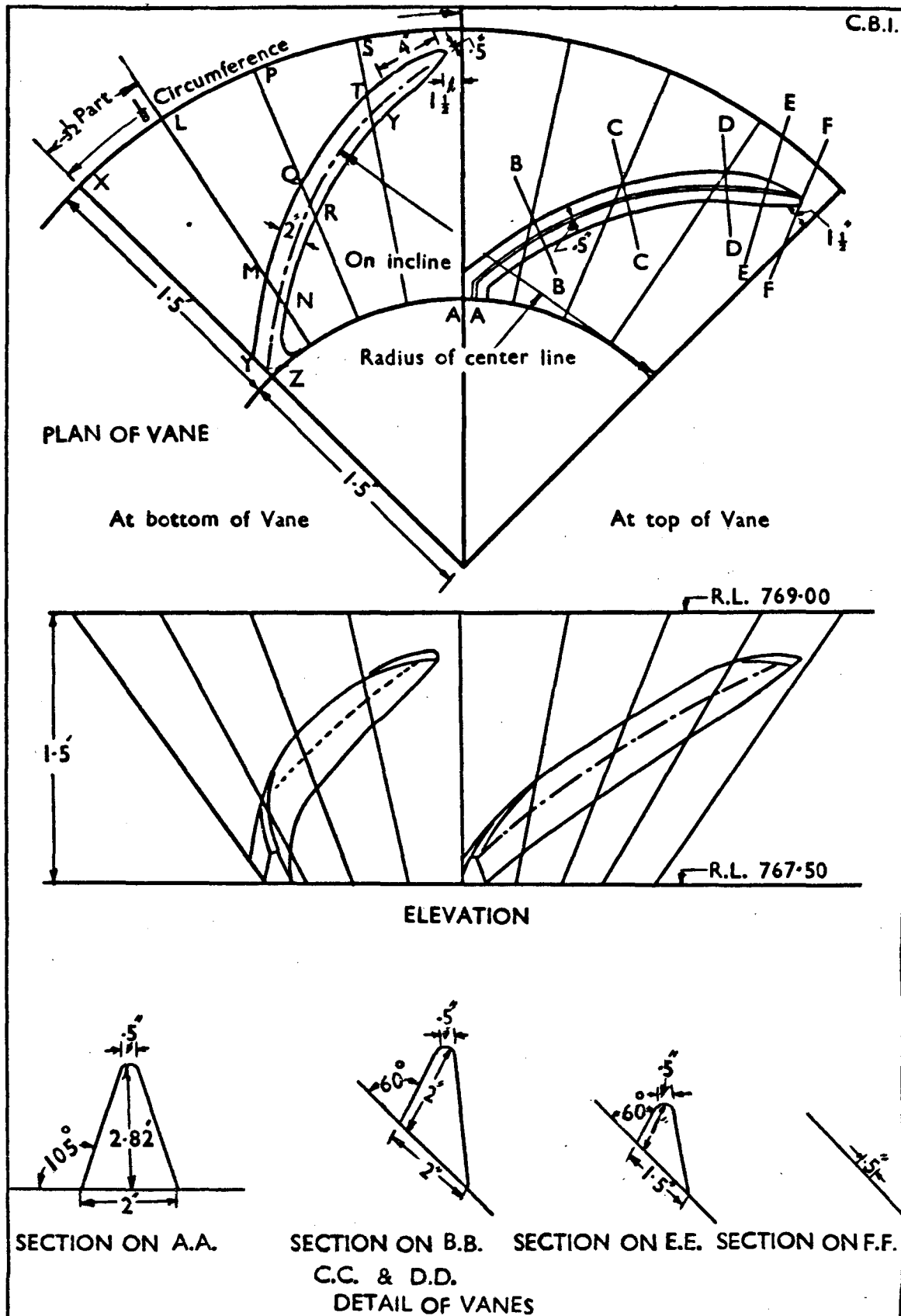
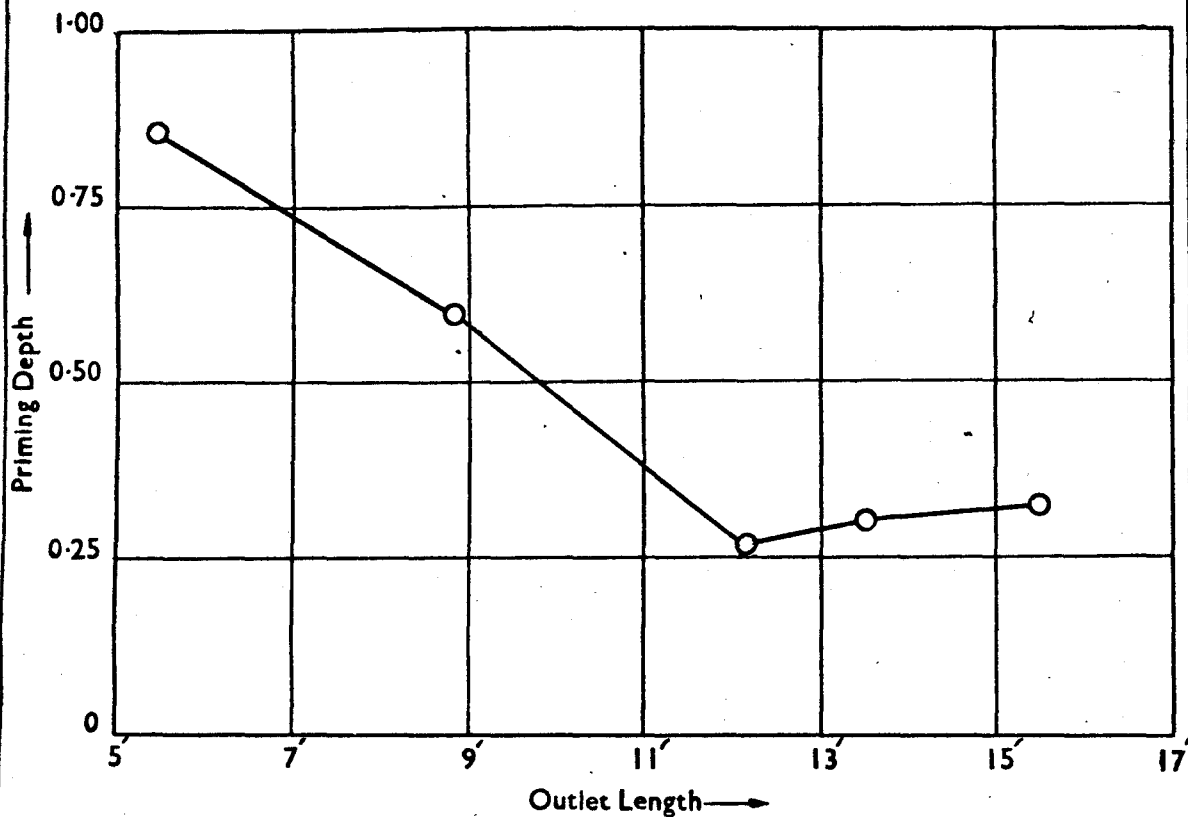
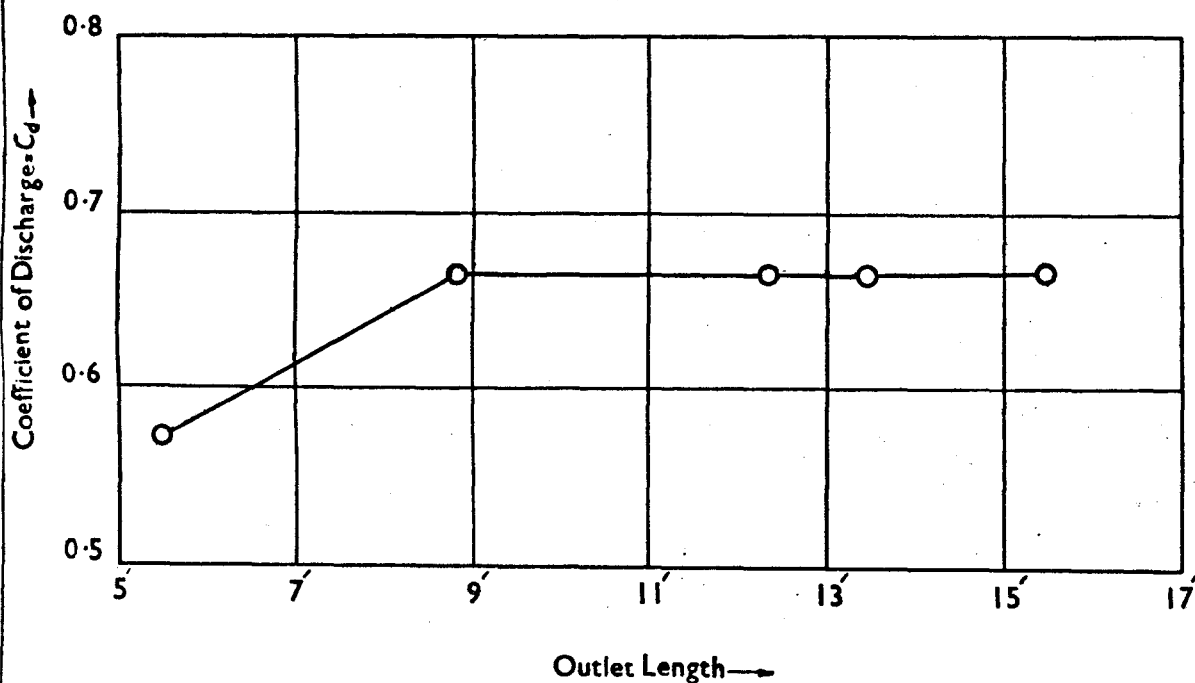


Figure 6C.19:- Showing detail of Vanes of the Volute Siphon on Pahuj Weir



(a) Variation in Priming Depth



(b) Variation in Coefficient of Discharge

Figure 6C.20:- Showing Variation in Priming Depth & Coefficient of Discharge for various Outlet Lengths

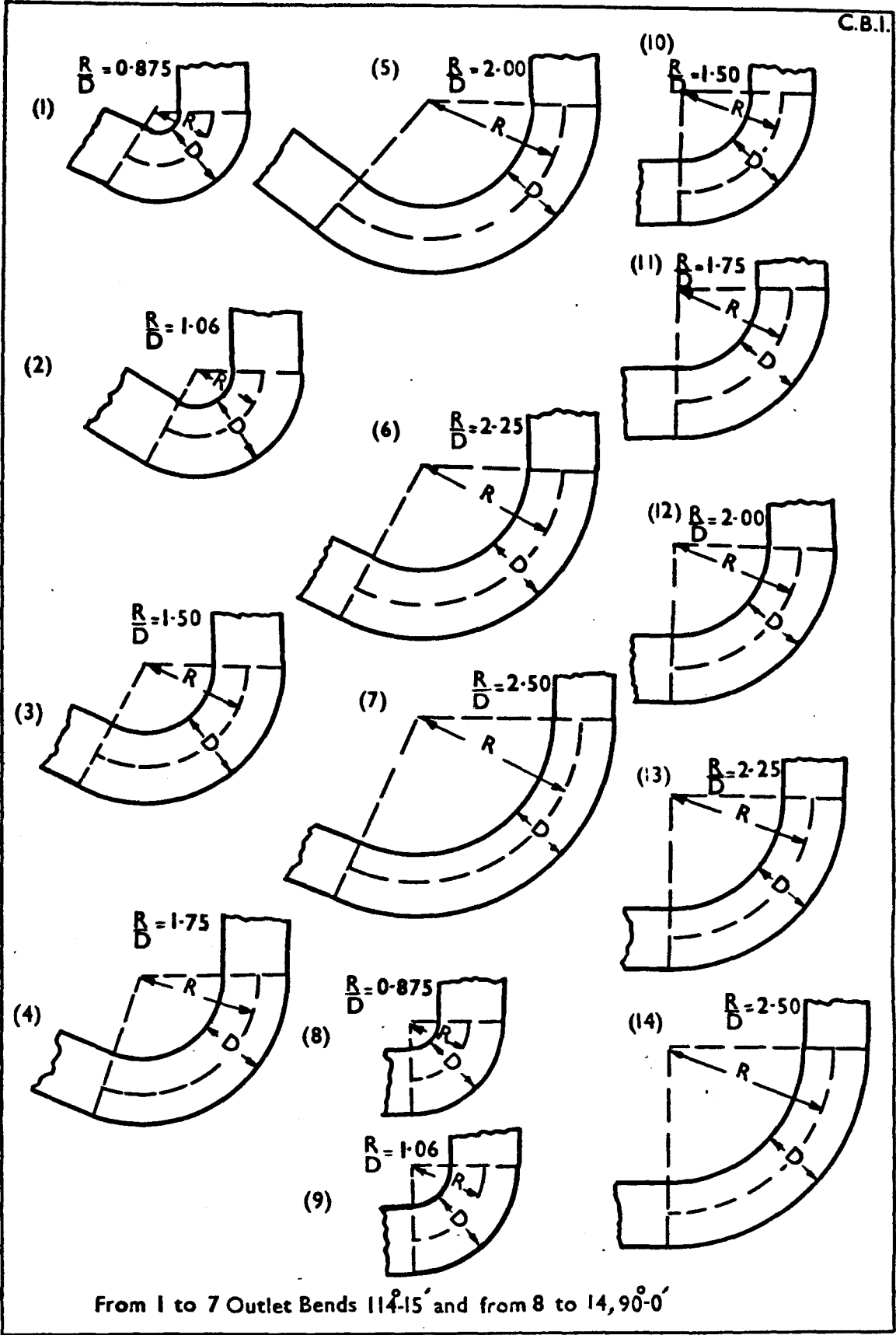


Figure 6C.21:- Showing Volute Priming Siphon for Pahuj Weir



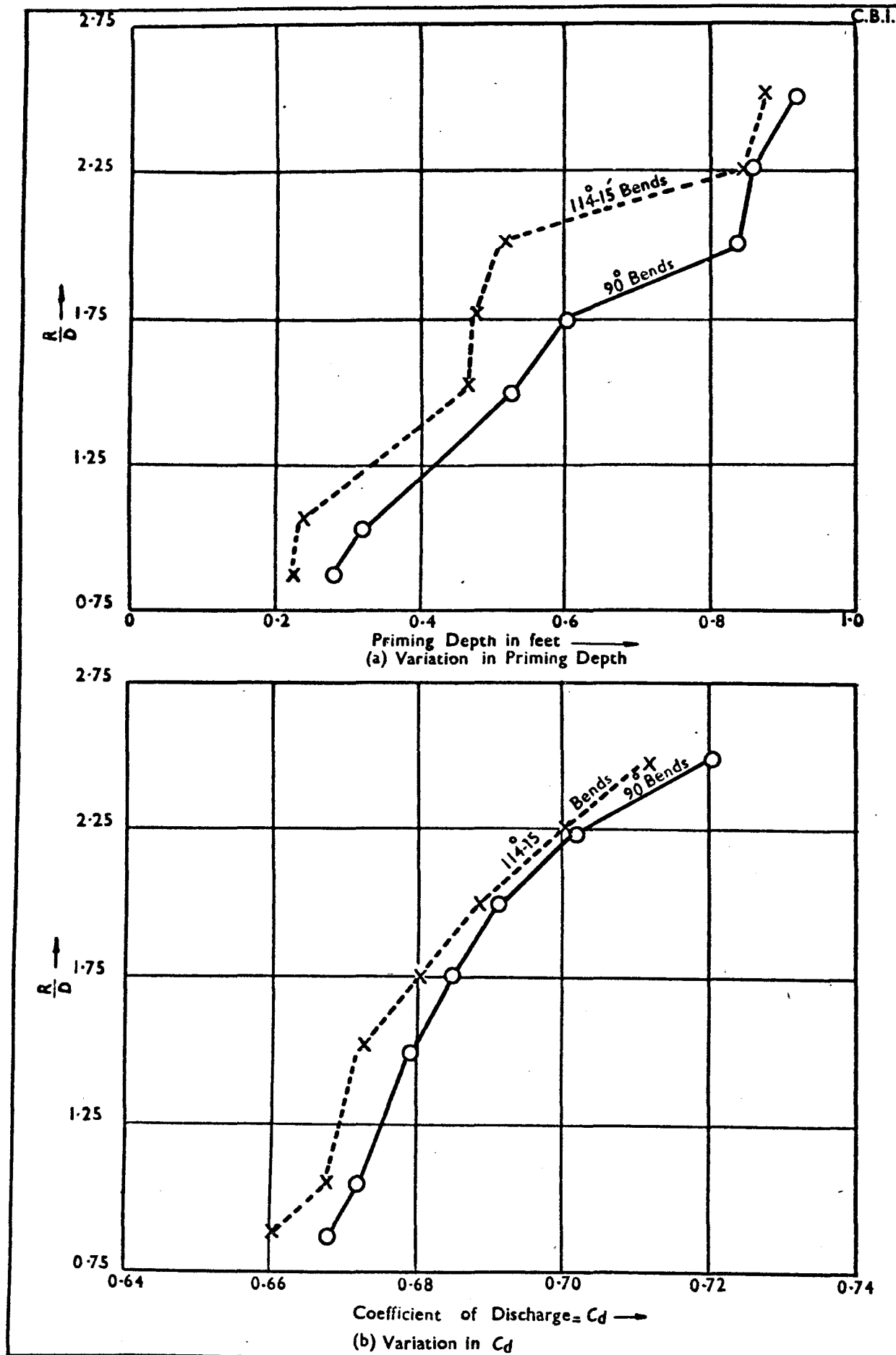


Figure 6C.22:- Showing Variation in Priming Depth & Coefficient of Discharge for different Values of $\frac{R}{D}$
(Volute Priming Siphon for Pahuj Weir)

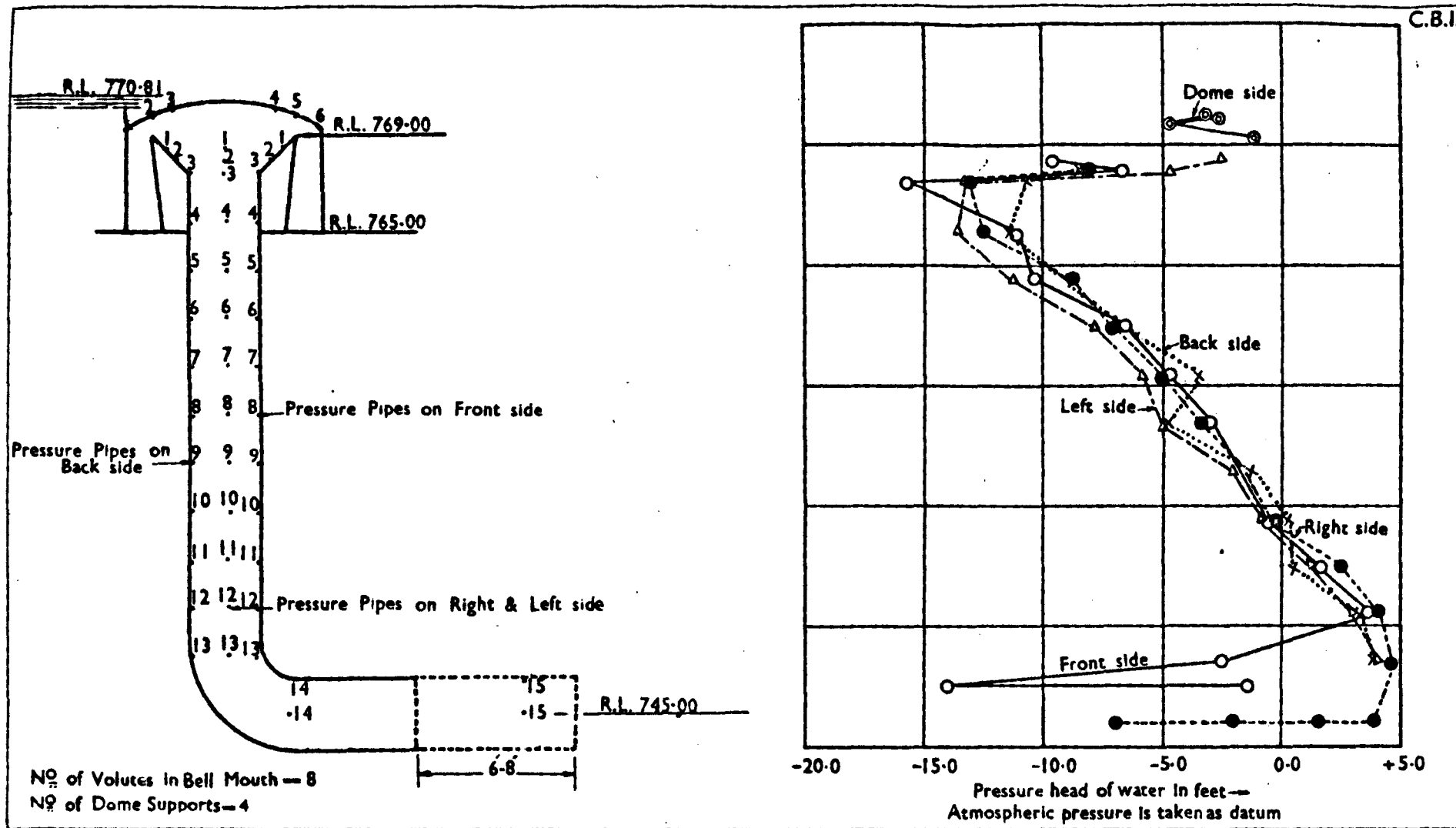


Figure 6C.23:- Showing Distribution of Pressure within the Siphon Barrel
(Volute Priming Siphon for Pahuj Weir)

TABLE 6C.12

Serial No	Length of outlet (feet)	Priming depth (feet)	Co-efficient of discharge
1	5.5	0.864	0.576
2	8.86	0.608	0.668
3	12.16	0.256	0.670
4	13.50	0.304	0.665
5	15.50	0.320	0.670

Figure 6C.20 shows how priming depth and co-efficient of discharge vary with varying lengths of outlet. It would be clear from the above graph that the minimum priming depth and maximum co-efficient of discharge is obtained with 12.16 feet long outlet. This suggests a basic study of outlet length for different diameters and heads which is under investigation.

Variation of priming depth and co-efficient of discharge with different outlet bends.

Model investigations were next taken up to determine the priming depth and the co-efficient of discharge for 90° and 114°—15' outlet bends generally adopted in practice for different ratios of $\frac{R}{D}$. The line diagrams of these bends are shown in Figure 6C. 21. The observations are given in Table 6C.13 and results plotted graphically in Figure 6C.22.

TABLE 6C.13

Showing variation in priming depth and co-efficient of discharge for different values of $\frac{R}{D}$ in Volute Priming Siphon

Serial No.	$\frac{R}{D}$	For 90° Bends		For 114°-15' Bends		Remarks
		Priming depth (feet)	Co-efficient of discharge	Priming depth (feet)	Co-efficient of discharge	
1	0.875	0.272	0.663	0.224	0.660	Sharp (v) ¹¹
2	1.06	0.320	0.672	0.232	0.667	
3	1.50	0.536	0.679	0.464	0.672	
4	1.75	0.600	0.685	0.472	0.680	
5	2.00	0.832	0.691	0.516	0.688	Easy (v) ¹¹
6	2.25	0.856	0.702	0.846	0.700	
7	2.50	0.912	0.721	0.872	0.712	

CONCLUSIONS

(i) The priming depth is minimum with sharp bends, while the co-efficient of discharge is greater in the case of easy bends in both 90° and 114°--15' bends. With very easy bends siphons may not prime.

(ii) The priming qualities are better in the case of 114°--15' bends while the discharging capacity is higher in the case of 90 degrees bends.

Pressure Observations :—

Pressures in dome, bell mouth and barrel of the same model were observed. The position of the pressure pipes 1, 2, 3 etc., both above and below crest within the siphon conduit, are shown in Figure 6C.23. The pressure observations at various sections are given in Table 6C.14 and results plotted graphically in Figure 6C. 23. From the study of the above graph it is evident that —

TABLE 6C.14

Showing pressure observations at various sections of Volute Priming Siphon
(a) **Below Crest.**

Serial No.	Distance of pressure pipe below crest	Pressure head of water in feet with respect to atmospheric pressure			
		Front side	Back side	Right side	Left side
1	0.5	-9.69	-9.89	-9.29	-2.59
2	1.0	-6.73	-7.59	-7.99	-4.99
3	1.5	-15.55	-10.75	-13.03	-13.29
4	3.5	-10.95	-11.29	-12.49	-13.69
5	5.5	-10.23	-9.09	-8.89	-11.29
6	7.5	-6.43	-6.89	-6.89	-7.75
7	9.5	-4.89	-3.39	-5.09	-5.75
8	11.5	-2.95	-4.75	-3.43	-5.09
9	13.5	-1.75	-1.55	-1.55	-2.03
10	15.5	-1.29	0.25	-0.69	-0.89
11	17.5	1.77	0.45	2.51	1.31
12	19.5	3.71	3.37	4.11	3.11
13	21.5	-2.43	4.31	4.77	4.25
14	22.5 24.0*	-14.00	..	4.07	-7.33
15	22.5 24.0*	-1.63	..	-2.05	1.67

*For side pipes only.

(b) Above Crest.

TABLE 6C. 14—Contd.

Serial No.	Distance of Pressure pipes above crest	Pressure head of water in feet with respect to atmosphere pressure
1	0.50	-1.35
2	1.00	-4.85
3	1.25	-3.10
4	1.25	-2.64
5	1.00	-4.85
6	0.50	-1.35

(1) The trend of pressure is approximately the same at four sections within the straight length of the siphon, though some variation is indicated at the bell mouth and the outlet. This is probably due to the free vortex flow being changed to straight line flow and *vice versa* at these two places.

(2) The maximum negative pressure in the volute siphon is only 16 feet compared to 26.8 feet in the saddle for the same head. The volute siphon can, therefore, be improved to give larger discharges by exploiting the suction head up to 26 feet.

(8) PREVENTING SCOUR BELOW GANESH IYER VOLUTE SIPHON SPILLWAYS BY MUTUAL IMPACT OF JETS ISSUING FROM ADJACENT SIPHON OUTLETS ⁽¹²⁾

ABSTRACT

In the high head dams, proposed to be constructed in various post-war schemes, Ganesh Iyer Volute Siphons will suggest themselves for adoption for

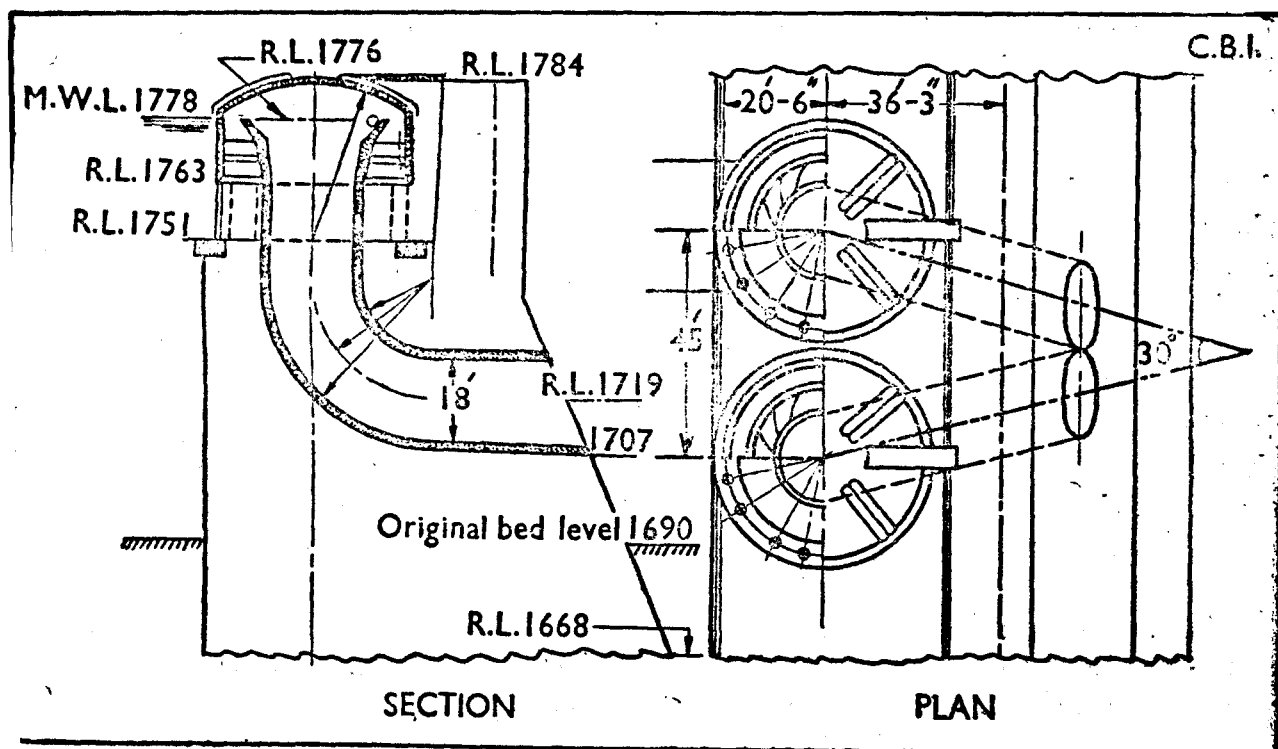


Figure 6C.24: Ganesh Iyer Volute Siphon (Hirabhasagar Dam, Mysore) with outlet angle modified for model experiments.

⁽¹²⁾ Indian Waterways Experiment Station, Poona, Annual Report, (Technical), 1946, Pages 91 to 94.

excess flood disposal. They have a better discharging capacity and a lower priming depth as compared to ordinary saddle type siphon spillway. Also, it has been claimed that, in their design, heads exceeding the atmospheric pressure head could be effectively utilised for producing flow through the siphon, thus requiring less spillway capacity in high dam design, if this type is used.

Where rock or inercdile formation is at a considerable depth below the normal bed level of the river downstream of the spillway, jets issuing from the outlets are likely to create new problems concerning the protection of the river bed against heavy scour ; and unless the kinetic energy of the jets is sufficiently destroyed before it hits the bed, and/or the river bed in the vicinity is adequately protected, conditions endangering the safety of the dam are likely to develop.

Experiments were commenced to study the dissipation of energy of the jets, by making them hit each other at different angles immediately after they leave the outlets.

MODEL

Two 1/24 scale geometrically-similar models of volute siphon spillways at Hirebhasagar dam (Jog Power Scheme, Mysore State), already constructed at the Station to study mutual interference of siphons, were utilised for this study. The volutes in the bellmouth inlet were made geometrically-similar and the distance between the siphons was equivalent to that of the Hirebhasagar siphons. The only change made was in the alignment of the horizontal outlets ; thus, in experiment 1, an angle of 18° was made and in Experiment No. 2 of 30°, Figure 6C.24.

PROCEDURE

The bed downstream of the spillway was laid in shingle at level equivalent to R. L. 1690 to a depth of 2.55 feet. in the model. Discharge was run through the siphons till the maximum depth of scour observed in the bed remained unaltered in consecutive runs.

Water level in the reservoir upstream was maintained equivalent to two feet above the crest of bellmouth maximum flood level, while water level in the river downstream was maintained at R. L., equivalent to 1707, or 17 feet above the original ground level. Thus, the drop of level from the centre of the outlet to the water level in the river was equivalent to 12 feet.

EXPERIMENTS

Experiment No. 1.—(Jet angle=18° and bed material=17.31 mm.) In this experiment, the jets were made to hit at an angle of 18°. Coarse shingle (mean diameter=17.31 mm.) was used for laying the river bed downstream of the spillway. It was observed that the two jets affected each other insignificantly, so far as energy dissipation was concerned ; but dropped down vertically as a single jet, practically producing the effect of the sum total of the two, as was judged by the depth of scour-hole. The depth of scour was considerable, the cement plastered bottom of the channel over which shingle was laid to a depth of 2.55 feet becoming exposed ; indicating that depth of scour if not restricted by the floor would have been more. The depth of scour-hole obtained with a single jets was 1.55 feet only.

Experiment No. 2 : (Jet angle=30° and bed material=17.31 mm.) In this experiment, the angle made by the adjacent jets was increased from 18° to 30°. The material used for simulating river bed was the same as in

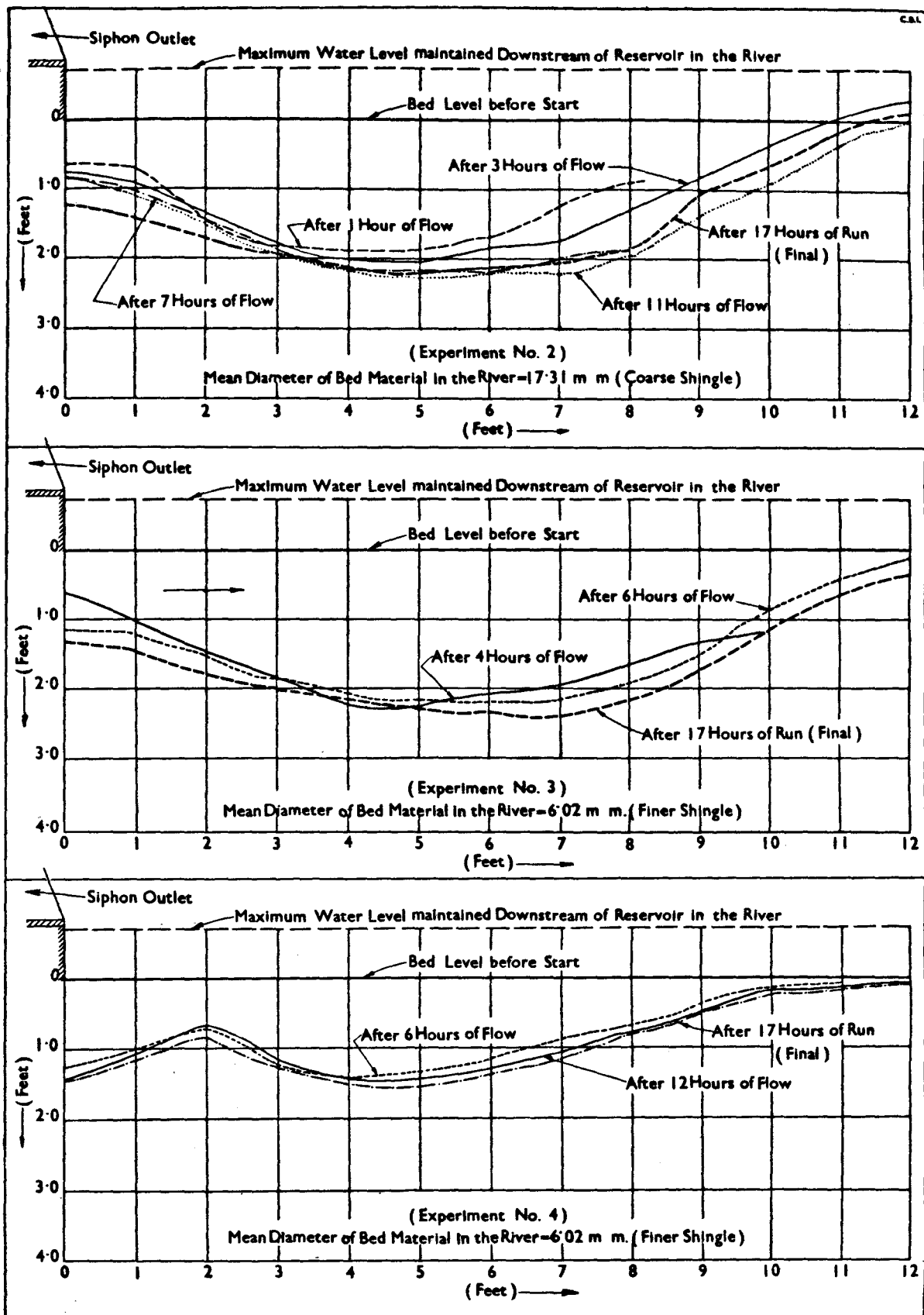


Figure 6C.25, 6C.26 & 6C.27:— Showing Profiles of Scour Hole along the Center Line with Jets of Adjacent Siphons hitting at an Angel of 30° [Scale Volute Siphon Model]

experiment No 1, viz. $m=17.31$ mm. Figure 6C.25 shows the longitudinal section of the scour-hole after stability was reached. The maximum depth of scour was 2.3 feet $\equiv 55.2$ feet giving slightly better results than with 18° jets.

Experiment No. 3 : (Jet angle = 30° and bed material = 6.02 mm.) In this experiment, the bed material was changed from 17.31 mm diameter to 6.20 mm., to see the effect of the grade of bed material on scour. It has been found that scour-holes in such cases dependent on the geometry of flow and the ultimate form of score hole is, therefore, independent of the grade (provided the bed material is incoherent), ⁽¹³⁾ The depth of scour was 2.4 feet, as compared with 2.3 feet in experiment No. 2 with $m=17.31$ mm. This point requires further investigation. Figure 6C. 26 shows the longitudinal profile of scour hole.

Experiment No. 4 : (Only one siphon running and bed material = 6.02 mm.) In this experiment, the depth of scour was observed with only one siphon discharging so that this could be compared with scour-hole obtained with jet hitting at angle of 30° in experiment No. 3, the bed material being the same. Figure 6C.27 shows the profile of scour-hole. The maximum depth of scour was only 1.55 feet $\equiv 37.2$ feet. Thus, with jets hitting at an angle of 30° , the maximum depth of scour-hole was 1.55 times that obtained with only one siphon.

CONCLUSIONS

The jets hitting at an angle of 30° were not effective in reducing the maximum depth below that obtained with only one siphon. The angle tested was the maximum that could be constructed with the existing design of the Jog Volute Siphons. If further increase in angle of incidence has to be tested, it will be necessary to increase the distance between two adjacent siphons (present distance 45 feet) which will, however, increase the length of the spillway and thus increase the cost.

(9) EXPERIMENTS ON THE EFFECT OF ALTERING THE SHAPE AND NUMBER OF VOLUTES IN A VOLUTE SIPHON ⁽¹⁴⁾

The volutes and the funnel are two of the important parts in the volute siphon which determines its performance.

Experiments were conducted firstly to determine the conditions under which maximum co-efficient of discharge could be obtained irrespective of the priming depth; secondly to determine the conditions under which priming depth was a minimum and finally to determine the optimum conditions under which a siphon could function with maximum co-efficient of discharge with minimum priming depth. All the other conditions in the models that is, the shape and curvature of the domes, vertical pipes, bends, outlets etc., were kept constant throughout the experiments.

Experiments were made with a nine inch model operating under, 3.25 feet head.

⁽¹³⁾ Burns, R. V. and White, C. M. "Protection of dams, weirs and sluices against scour," Institution of Civil Engineers, Paper No. 5127 (1938-39).

⁽¹⁴⁾ Hydraulic Research Station, Krishnarajasagar, Mysore, Annual Report, 1946, pages 47 to 49.

The effect of increasing the capacity of the funnel on the co-efficient of discharge was first studied. To effect this increase in area the volutes were chipped off and the siphon was allowed to function. The co-efficient of discharge increased from 0.75 to 0.85. But priming took place only after the dome was completely submerged giving no access to air from outside, *i.e.*, a depth of about eight inches which was very high. The experiment clearly showed that increase in area of the funnel was necessary for getting increased co-efficients and that volutes were equally necessary to reduce priming depth. This increase in area of the funnel with the volutes on it could be accomplished either by increasing the slope or the diameter of the siphon.

The effect of varying the slope was first tried. The slope was reduced from 2 to 1 to $1\frac{1}{4}$ to 1. Priming depth was found to get reduced. But a further reduction in slope from $1\frac{1}{2}$ to 1 to $1\frac{1}{4}$ to 1 did not very much affect the priming depth. While it reduced the co-efficient of discharge. A slope of $1\frac{1}{2}$ to 1 was considered the optimum and was adopted. The width of the funnel at the top is another factor which affects the discharge. Greater the ratio between the diameter at top of the funnel and the diameter of the vertical pipe, better the discharging capacity of the siphon. The ratio has no appreciable effect on the priming depth. Since increased diameters of funnel resulted in fewer number of siphons and lowered the discharge that could be passed per foot length of spillway, it was not desirable to give wide widths to the funnel. The ratio 1.5 between the diameters of the funnel at top and bottom gave satisfactory results.

The effect of altering the number, height, shape and curvature of the volutes was next studied. It was found that the greater the number of volutes, the more powerful would be the vortex and consequently lesser would be the priming depth. Since they reduced the area of flow in the funnel, the discharge also got reduced. Table 6C. 15 gives the results of the experiments.

TABLE 6C-15

Experiment No.	1	2	3	4	5	6	7	8	9	10	11
No. of volutes	6	6	6	9	9	9	9	12	12	12	12
Height of volutes in inches .. .	$\frac{3}{4}$	1	$1\frac{1}{2}$	$\frac{3}{4}$	1	$1\frac{1}{2}$	$1\frac{1}{2}$	$\frac{3}{4}$	1	$1\frac{1}{4}$	$1\frac{1}{2}$
Priming depth in inches .. .	6	6	5	$5\frac{1}{2}$	5	5	$4\frac{1}{2}$	$4\frac{1}{2}$	4	$3\frac{1}{2}$	3
Co-efficient of discharge .. .	0.85	0.85	0.83	0.81	0.81	0.80	0.80	0.78	0.78	0.76	0.75

Twelve was considered the minimum number of volutes necessary, for forming a strong vortex so as to ensure minimum priming depth. Keeping the number of twelve volutes constant, experiments were next conducted to study the effect of increasing their heights. Table 6C.16 gives the results.

TABLE 6C-16

Heights of volutes above funnel surface In inches	0	$\frac{1}{2}$	$\frac{3}{4}$	1	$1\frac{1}{2}$	2	$2\frac{1}{2}$	5
Priming depths in inches	Did not prime till the water level was $\frac{1}{2}$ inch above dome (i.e. about 8 inchs)	5.5	5	4.5	3.5	3.5	3.5	3.5

It was thus seen that the heights of the volutes influenced the priming depth only to a certain limit. Any increase of height beyond $1\frac{1}{2}$ inches had no influence on priming.

Regarding the shape, it was found that for the same height of volute, a smooth planar shape, that is, the cross section at any point being made trapezoidal was the best. Cup shapes, and shapes imitating the blades of a centrifugal pump, and turbines, etc., were tried and found not very advantageous.

Having thus fixed the number, height and curvature of volutes for minimum priming depth and shape of the funnel, for maximum discharge, experiments were continued further to evolve the optimum combination of both. It was found that the number of volutes necessary for low priming reduced the capacity of the funnel. This difficulty was overcome by slightly increasing the diameter of the funnel at the top from 18 inches to $19\frac{1}{2}$ inches, keeping the same slope of $1\frac{1}{2}$ to 1. The junction between the funnel and the vertical pipe was made more gradual by decreasing uniformly the diameter of the funnel below the volute portion from 14 inches to 12 inches in a height of 8 inches.

This resulted in a siphon which gives a co-efficient of 0.79 with a priming depth of three inches in the nine inch model operating under 3.25 feet head.

(10) EXPERIMENTS ON DISSIPATING THE ENERGY OF FLOW FROM VOLUTE SIPHONS ⁽¹⁵⁾

ABSTRACT

Where siphons have been installed in dams with their outlets above the highest flood level of the river in rear of them, their discharges shoot out in a clear overfall and impinge with further momentum added during their descent on the river bed below. Various methods have been employed to dissipate their energy so as to cause least damage to the protective works in the rear.

One of the methods employed is to give a tilt upwards so that the jets shoot up high into the air dissipating their energy in their upward movement. Sometimes they are further broken into small spray by fitting jet dispersers to the mouths of the outlets. Such devices cause considerable resistance to flow reducing their efficiency considerably. Other methods were tried to dissipate the energy of flow.

Experiments were tried with an elliptical outlet, the transition from the circle to the ellipse being made from the bend. The area of the ellipse was kept same as the circle, the width of the ellipse being increased proportionately.

⁽¹⁵⁾ Hydraulic Research Station, Krishnarajasagar, Mysore, Annual Report, 1946, pages 50 to 51.

Two sets of experiments were made. In one of them the transition from circular to elliptical section was made giving a linear divergence, *i.e.*, in plan the line joining tangentially the circle and the ellipse was straight. In the other series the transition was made giving a parabolic divergence, that is the flare between the circle and the ellipse was made a parabola. In the 12 inches siphon model working under an operating head of 3 feet used for the purpose, the transition was made from 12 inches circular to 10 inches by 8 inches elliptical.

With linear divergence, the flow did not spread itself out very much. The co-efficient of discharge which was 0.79 in a circular outlet got reduced to 0.71 due to energy lost in change of section.

With parabolic, divergence, the flow spreads out better. The co-efficient of discharge got reduced from 0.79 to 0.74.

As mere divergence of flow did not effectively dissipate the energy of the flow, experiments were next conducted with a mouth piece attached to the outlet. It had a guide in the form of a spiral traced along the linear circumference of the mouth piece. Its pitch was equal to the diameter of the outlet piece. Experiments were conducted with various lengths of mouth pieces attached to a 12 inches siphon operating under a head of 5.75 feet. It was found that even with a spiral guide which had been twisted only through 180° , the resistance offered to the flow was so great that the co-efficient of discharge fell down to 0.575 from 0.815 which was the co-efficient obtained without the mouth piece.

Next a mouth piece with a screw vane was tried. The vane was helical in shape equal to the diameter of the outlet in width and had a pitch equal to the peripheral length of the outlet section. The length of the mouth piece was two feet. Though it was more effective in dispersing the jet, the reduction in discharge co-efficient was almost similar to that of the screw guide *i.e.*, from 0.815 to 0.58, The siphon used was the same as in the former experiment.

A suggestion was given by Rai Bahadur A. N. Khosla, Consulting Engineer to Government of India, that siphons may be placed in pairs with outlets askew to each other so that the jet from one impinged on the other, the energies of both getting dissipated during impact. The angle of the skew, that is, the divergence from the axis perpendicular to the face of the dam could not be made big as then the space occupied by any pair of siphons would increase, lessening the discharging capacity of the siphon per foot length of the spillway length available in a dam. If the skew angle was kept small, the jets could be made to impinge against each other only after they had travelled a considerable distance away from the exit surface, with the result they would have gained considerable momentum due to gravity before the impinge. Impinging would take place only at a low level above the bed of the river. As this would not be effective, short mouth pieces with bends facing each other at 45° were fitted up. The impinging took place, quite high just a little below the exit level and is fairly effective. The reduction in co-efficient was from 0.815 to 0.66.

So far no satisfactory method has been evolved. The alternative of keeping the siphons with their outlets just at the bed of the river itself thus avoiding all impinging action on it seems to be preferable.

(ii) **EXPERIMENTS ON VARIATION OF CO-EFFICIENT OF DISCHARGE UNDER VARYING DEGREES OF SUBMERSION OF SIPHON OUTLETS** ⁽¹⁶⁾

ABSTRACT

Several experiments were tried to dissipate the energy of the discharge before it issued from the outlet exit of a volute siphon. Since all of them resulted in reduction of co-efficient of discharge, fresh methods had to be tried.

In the experiments tried, the siphons discharged free with a clear overfall on the tail water over the river bed in rear of the dam. The severity of the impact could be mitigated by keeping the outlet submerged under the tail water just on the river bed itself. Experiments were conducted to find out the reduction in the discharge that may result thereby. Two cisterns were constructed with a dividing wall between them one cistern to represent a reservoir, the dividing wall to represent a dam and the other cistern to represent a river bed.

A battery of six siphons each of six inches were installed so as to operate under a head of 3.75 feet. They were all placed close together in a row with two inches between the outer edge of one dome and the other. The siphon outlets were kept with their centres above the bed of the floor of a masonry cistern which represented the bed of the river. The water level on the upstream was kept constant of each set of experiments, while the depth of water over the cistern floor on the downstream side was raised each time to get the required depth of submersion of the outlet. The levels of water upstream and downstream were taken with reference to the lip of the funnel.

For purposes of calculating the co-efficient of discharge, when the discharge from the outlet, was a clear over fall, the operating head is taken as the difference between the water level upstream and the centre of the outlet exit. In all other cases, the head is taken as the difference of water levels upstream and downstream. In this method of calculation, the operating head is the same whether the outlet exit is just below or several feet below the downstream water level.

Results of the experiments show that the co-efficient of discharge is a maximum being 0.83 for a clear overfall discharge. Even when the issuing jet is just supported on the downstream water, the co-efficient of discharge falls from 0.83 to 0.782. The co-efficient falls gradually as the depth of submersion of the outlet increases as shown in Table 6C.17. Table 6C.17 shows the relation between the co-efficient of discharge and submersion ratio which has been taken as the ratio between the depth of downstream water level above the invert of the outlet exit and the diameter of the outlet. The co-efficient of discharge is a minimum when the outlet is fully submerged being 0.67.

⁽¹⁶⁾ Hydraulic Research Station, Krishnarajasagar, Mysore, Annual Report, 1946, pages 52 to 53.

It is thus seen that the co-efficient of discharge falls down from 0.83 to 0.67 when the discharge changes from that of a clear overfall to one of complete submersion. The siphon outlet if placed at the bed of the river while it may be effective in preventing the damages caused by impinging of discharge on the river bed, reduces the efficiency of siphon considerably. To overcome this disadvantage, it is proposed to conduct experiment to create standing wave conditions in rear of the outlet keeping it at the bed of the river. This may increase the operating head and the discharge through the siphon and also dissipate the energy of the flow more efficiently.

TABLE 6C.17

Effect of submersion of siphon outlet on the co-efficient of discharge

Submer-sion	Upstream water level (feet)	Down-stream water level (feet)	Head applied (feet)	Dis-charge (cusecs)	Co-efficient	Remarks
Free	-4.00	3.625	2.666	0.83	
0.00 ..	0.125	-3.625	3.750	2.318	0.782	Submersion ratio
0.42 ..	0.225	-3.415	3.640	2.266	0.76	
0.58 ..	0.167	-3.333	3.500	2.205	0.75	depth over invert of outlet = diameter of outlet
0.85 ..	0.227	-3.223	3.450	2.101	0.72	
0.97 ..	0.281	-3.139	3.420	2.063	0.71	
1.00 ..	0.345	-3.125	3.470	2.018	0.67	

Diameter of pipe=6 inches

Datum=Lip level of siphon

(12) EXPERIMENTS ON SHAFT SPILLWAYS FOR TANKS (17)

ABSTRACT

Almost all tanks in the Mysore State have open weirs for discharging surplus floods. If the weirs are sharp crested and admit of a clear overfall discharge they spill about 3.1 cusecs per foot length per foot depth of overflow. If by any means it is possible to increase the discharge per foot length, the length of a weir required for passing the surplus floods reduced and consequently the width of the draft channel taking off from it is also reduced, saving in cost. Where it is not costly to have long lengths of weirs, the length may be kept the same when the depth of over flow required to pass the floods may be reduced. The height of tank bund and the area of land to be acquired between full tank level and maximum water level are decreased. Eitherway, a spillway which can discharge more per foot length than ordinary weirs results in considerable economy.

(17) Hydraulic Research Station, Krishnarajasagar Mysore, Annual Report, 1946, pages 54 to 59.

A profile of a high co-efficient weir was developed ⁽¹⁸⁾ which gave a co-efficient of 4.69 in the formula.

$$Q = CL H^{\frac{3}{2}}$$

Where Q = Discharge in cusecs

C = Co-efficient of discharge

H = Length of weir in feet

L = Depth of flow over the weir in feet

The co-efficient was for an overflow depth of 12 inches in a model of three feet length and 1.45 feet in height.

The discharge in this spillway was proposed to be increased by having a number of holes in the body of the weir. It was expected that each one of the holes would act as a circular weir and the floods would pass partly through these holes and partly over the weir.

The holes were given at the entry a funnel shape with volutes and were flared at the outlet. They were exactly at the centre of the top of the body wall of the weir. When water was allowed it was found water would jump over the circular holes and the volutes never came into action. Since the volutes were likely to be damaged when left exposed in tank weir in out of the way places a design without the volutes was considered more suitable.

The design of the various component parts of the weir were taken up separately. The efficiency of each part individually and in relation to the whole has been taken up. The factors entering the design are :—

1. The front and rear batters to the body wall in relation to its stability and co-efficient of discharge.
2. The shape at the top of the body wall.
3. The junction of the toe of the body wall with the apron below.
4. Diameter and number of wells.
5. The shape of wells and the shape at the top edge to regulate entry of water into the well.
6. Effect of interference of flow between the wells.
7. Interference of flow between well and body wall of weir.
8. Shape of outlet.
9. Shape of the outlet at the junction and the well.
10. The method of flaring and shape of exit.
11. Position of outlet.
12. Interference of flow between discharge over the weir and that from the outlet.
13. Rate of increase of co-efficient with depth of discharge during rising stage and falling stage.
14. Pressures below the nappe and in the wells.
15. Effect of end contraction on discharges in models.
16. Effect of altering the position of the weir in plan, making it circular or parabolic.

⁽¹⁸⁾ Indian Waterways Experiment Station, Poona, Annual Report (Technical), 1939-40.

17. Effect of silting up in rear of the weir.
18. Protective works to the apron in rear of the weir.
19. Relation between flow in the model and proto-type.

Experiments were started with new design. The body wall of the weir as it was designed was two third of a right angled triangle with its angle at the apex equal to 30° . The top surface was rounded off to a semicircular shape. The model designed was 12 inches high and $2\frac{1}{2}$ feet in length. Four semicircular wells each two inches in diameter, equally spaced apart were attached to the weir. Outlet were taken from the wells, at the bed level through weir. The outlet was three inches in diameter at the entry to the well and flared to $3\frac{1}{2}$ inches of the exit.

The front batter was kept vertical as it was presumed any change in its shape did not affect the efficiency of the weir. Rear batter was kept such that the section of the weir was stable under various conditions of over flow without leaving any air gap between the body wall and the nappe. Experiments were made by gradually cutting the section of the weir evolved at Poona, and making the rear batter as steep as possible. After a few trials, it was found that the rear batter $1 : \sqrt{3}$ gave good results.

Regarding the shape at the top of the body wall, a lip was added at the upstream edge. This did not appreciably increase the discharge co-efficient. This was given up and a semicircular and semi-elliptical tops were next tried successfully. The semi-elliptical top gave better co-efficients.

The junction of toe of the body wall with the apron below is generally covered over with an up turned bucket so as to facilitate formation of a standing wave away from the weir. The problem in this case was solved by constructing a platform with a slight reversal slope over the top of all the outlets so that the flow as it descended on the rear slope of the body passed over the platform and jumped clear of the flow from the outlets taking off from the wells. Interference of flow between the outlet discharges and that from the body wall of the weir was prevented. The outlets have also been given a reverse slope so that the discharge from them impinges on those descending from the platform, thus dissipating energy during their impact.

Experiments were next conducted regarding the optimum number and diameter of wells that can be added on to each weir. Increasing the number of wells interfered with the discharge over the portions of the weir between them. The ineffective length of the straight weir at its junctions with the walls of the well got increased, thus decreasing the flow over the weir. Too small a number of wells were not also useful as to pass the same discharge through the wells, their diameter had to be increased which meant a corresponding increase in the diameter of the outlets. A disproportionate increase might endanger the stability of the weir. For $2\frac{1}{2}$ feet length of weir experiments were tried with five wells each $2\frac{1}{2}$ inches in diameter when it was reduced to four wells each three inches in diameter the co-efficient of discharge increased from 4.50 to 5.06. The wells were made semicircular in shape as they could then be built economically as a semicircular arch. To prevent the thrust from wells being communicated to the straight weir, it might be better to design them as independent shells placed close to the straight weir. The diameter

of the well was kept such that the discharge over its peripheral length at the top was at least twice as much as could pass through the outlets at the maximum designed depth of flow over the weir. The outlets had been kept circular in section with a bell mouth entrance at the junction of the outlet with the well. The outlet was flared at the exit from three inches to $3\frac{1}{2}$ inches in a distance of eight inches. The exit was changed from circular to elliptical shape, the transition from circular at the start to the elliptical section at the exit was made gradual without changing the cross sectional area. The divergence was straight from circular to the elliptical section.

The effect of keeping the outlet above the bed was not advantageous, as it lessened the discharging capacity of the outlets and also caused damage by impact of water on the apron below.

Table 6C. 18 gives the results of the experiments :—

TABLE 6C. 18

Head in feet				Co-efficient 'C' in the formula $Q=CLH^{\frac{3}{2}}$
0.080	12.130
0.199	6.331
0.279	5.278
0.320	5.135
0.403	5.996
0.459	4.979
0.643	3.746
0.791	3.636

Strictly the formula $Q=CLH^{\frac{3}{2}}$ is not applicable to this type of spillway as the portion of the discharge passing through the wells varies as the square root of the head of water above the outlet. Still a table of co-efficient is given just to indicate the increase in the discharging capacity of these types of spillways compared with the usual type of weirs.

The wells were all closed with clay and water was allowed to pass over the weir. The co-efficient of discharge varied from three to 36 for depths of flow varying from 0.3 to 0.72 feet. Water was allowed to pass only through the wells without allowing any spillage over the weir. The co-efficient of discharge in the outlets worked out to 0.75 in the formula $Q=CA\sqrt{2gh}$ where Q is the discharge in cusecs, A = area of the outlet, and h = is the operating head measured from water surface upstream to centre of the outlet :

The effect of silting up to the crest of the weir on the co-efficient of discharge was also investigated. In the small model the effect was not perceptible the increase in the co-efficient being within five per cent.

Further experiments were conducted on only the weir profile without the wells. They were confined only to the investigation of variation of co-efficient of discharge due to merely changing the shape at the top, other factors remaining the same. The model was only ten inches high and $2\frac{1}{2}$ feet in width. For a circular top, the maximum co-efficient of discharge realised was 3.889 for a depth of 0.62 foot above the crest of the weir. For a semi-elliptical top :

It has been customary to base the curve of a spillway crest on the profile obtained with the under-surface of the ventilated nappe of a sharp-crested weir at a corresponding head. This has been founded on the belief that the presence of a solid structure below the nappe and in immediate contact with it will not appreciably affect the course of the freely falling particles of water. Consequently a sharp crested weir corresponding to the proposed height of Nagwa Weir (60.0 feet height was changed to 65.0 feet later on) was constructed in 1.5 feet flume to scale 1/40 with one side of glass plate to trace the under-surface of the nappe, and ordinary brick-wall on the other side, with a hole to aerate the nappe. Figure 6C.29 shows the under-surface of the ventilated nappe under a head corresponding to the maximum flood discharge. A weir profile was made 1.5 feet wide, traced as above as per No. 1 of Figure 6.C.30. The co-efficient of discharge was found to be 3.587 only. This co-efficient of discharge again appeared to be rather low in view of the Poona experimental results⁽²⁰⁾, where a maximum co-efficient of 4.83 had been obtained on high co-efficient weir models.

Model investigations were taken up with various profiles as per Figure 6C.30 to determine the shape for the maximum co-efficient of discharge. The common feature of all these profiles tested is that the downstream slope of 2:3 was invariably maintained in all, as was indicated by the under-surface of the ventilated nappe of a corresponding sharp-crested weir, *vide* Figure 6C.29. Only the upper portion of the crest was modified to find out the variation in the co-efficient of discharge. Profile No. 11 Figure 6C.30 with a protruding lip upstream gave the maximum co-efficient of 4.049. This profile has been incorporated in Figure 6C.29 and it would be interesting to note the difference in the shape of the high co-efficient type profile and the one made in conformity with the under-surface of the ventilated nappe of a sharp crested weir. It will be appreciated that $C_d=4.049$ is for $H/D=0.135$; where H is the head over weir crest, and D is total upstream water depth. The value of C_d increases as ratio H/D increases. At Poona C_d was obtained as high as 4.83 but for $H/D=0.57$. The second best profile in Figure 6C-30 without the projecting lip is No.6 which gives a co-efficient of discharge of 4.012. Details of these two profiles can be seen in Figures 6C.31 and 6C.32.

The nature of rock at Nagwa site being sand stone, it was feared that the high velocity jet discharging freely over the weir, might undermine the rock foundation and thus endanger the safety of the whole structure. Experiments were, therefore, conducted with the best profile to find out devices for maximum dissipation of energy and minimum protection downstream. Cistern depths ranging from three feet to eight feet and lengths ranging from 60 feet to 100 feet were tried. Four feet deep and 90 feet wide cistern gave optimum conditions. Figure 6C.33 (a) shows a stepped cistern with one foot high and ten feet wide steps and a row of staggered blocks 8 feet \times 4 feet \times 4 feet. Figure 6C.33 (b) shows sloping cistern with two rows of staggered blocks 8 feet \times 4 feet \times 4 feet. The performance of the former was slightly better than the latter.

It was decided later on, to provide at least six feet of flat surface at the top of crest for walking across for inspection purposes. Such profiles designed with and without lip, Figures 6C.34 and 6C.35 gave co-efficients of 3.648 and 3.548 respectively with the maximum anticipated discharge.

⁽²⁰⁾ The Central Irrigation and Hydrodynamic Research Station, Poona, Research Publication No. 4, pages 46 to 47.

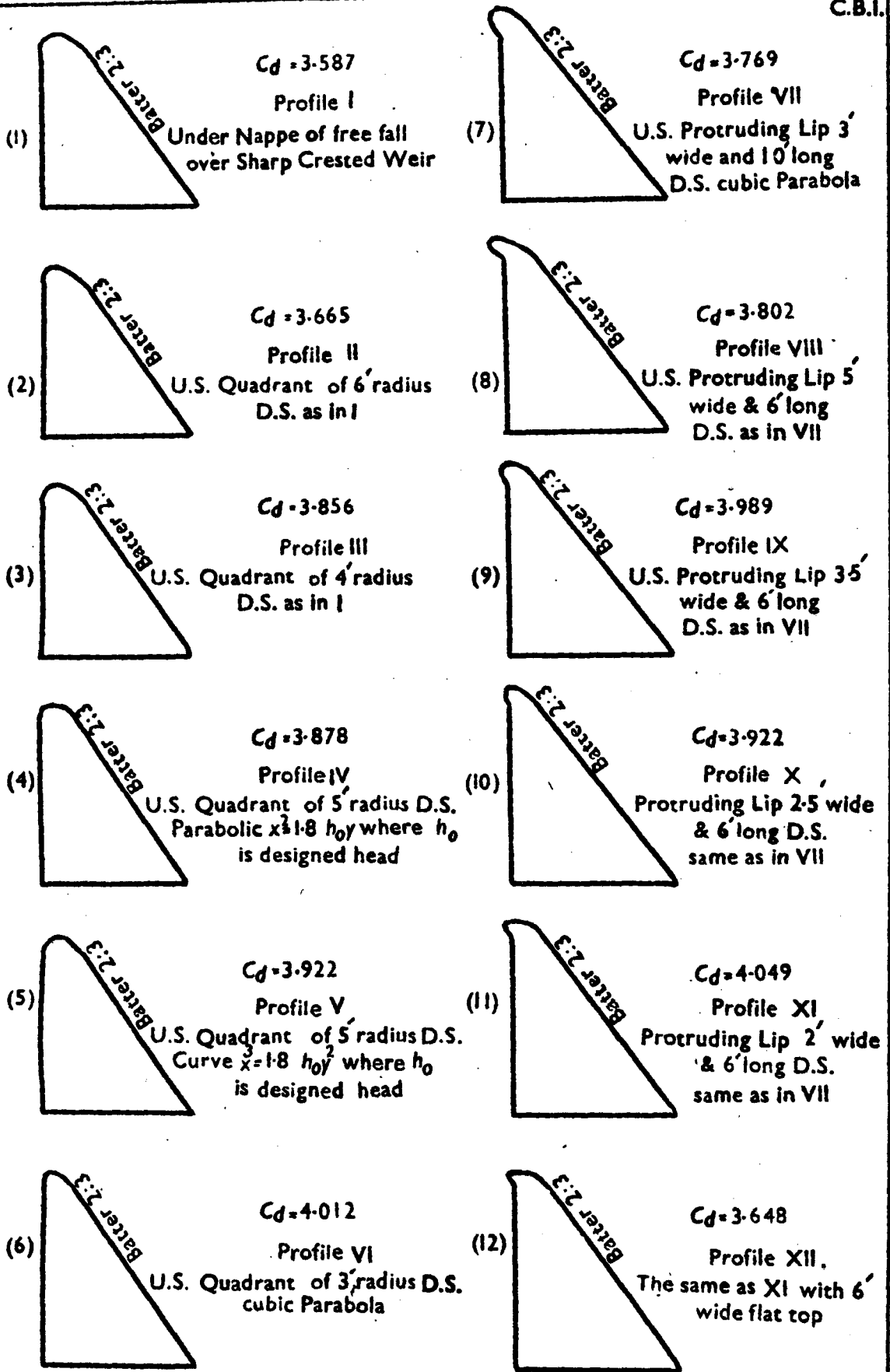
Model of the above four different profiles were now constructed to scales 1/20 and 1/30 and the corresponding co-efficients of discharge observed. These investigations would indicate how the co-efficient of discharge varied with the scale ratio. Table 6C.19 gives the co-efficient of discharge for various profiles to different scales.

TABLE 6C.19

Figure No.	Profile No.	Scale	Observed co-efficient of discharge	Extrapolated Co-efficient of discharge in prototype
1	2	3	4	5
6C-31	A (with lip) ..	1/40	4.049	4.098
		1/30	4.061	
		1/20	4.075	
6C-32	B (without lip) ..	1/40	4.012	4.070
		1/30	4.026	
		1/20	4.042	
6C-34	C (with lip) ..	1/40	3.648	3.704
		1/30	3.665	
		1/20	3.675	
6C-35	D (without lip) ..	1/40	3.548	3.600
		1/30	3.558	
		1/20	3.575	

These values of C_d were extrapolated *vide* Figure 6C.36 and the co-efficient of discharge for prototype found out *vide* column 5, of Table 6C.19 thereon.

Pressure observations were next taken at different ratios of H/D i.e., (Head over weir crest/Depth of flow) for all the four profiles constructed to scale 1/40. The details of observations are given in Tables 6C.20 to 6C.23 and curves showing pressure variations have been plotted in Figures 6C.31, 6C.32, 6C.34 and 6C.35 as also corresponding values of C tabulated.



Maxium flood Discharge 79,000 Cusces

Figure 6C.30:- Showing Sections of Various Profiles of high Coefficient Weir for Nagwa Reservoir

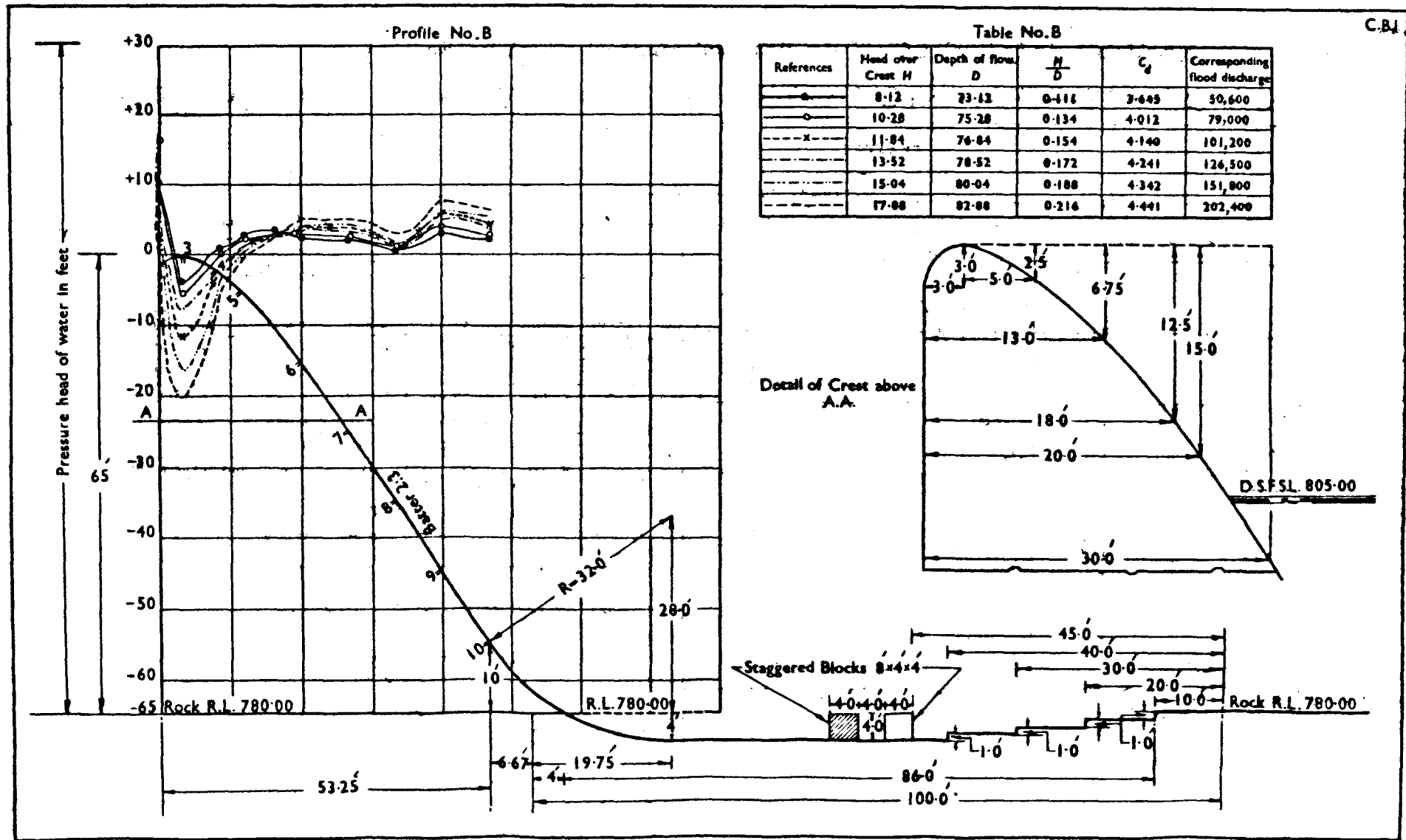


Figure 6C.32 :— Showing Section of high Coefficient Weir for Nagwa Reservoir [Curves showing Pressure variations for Discharges other than maximum flood Discharges]

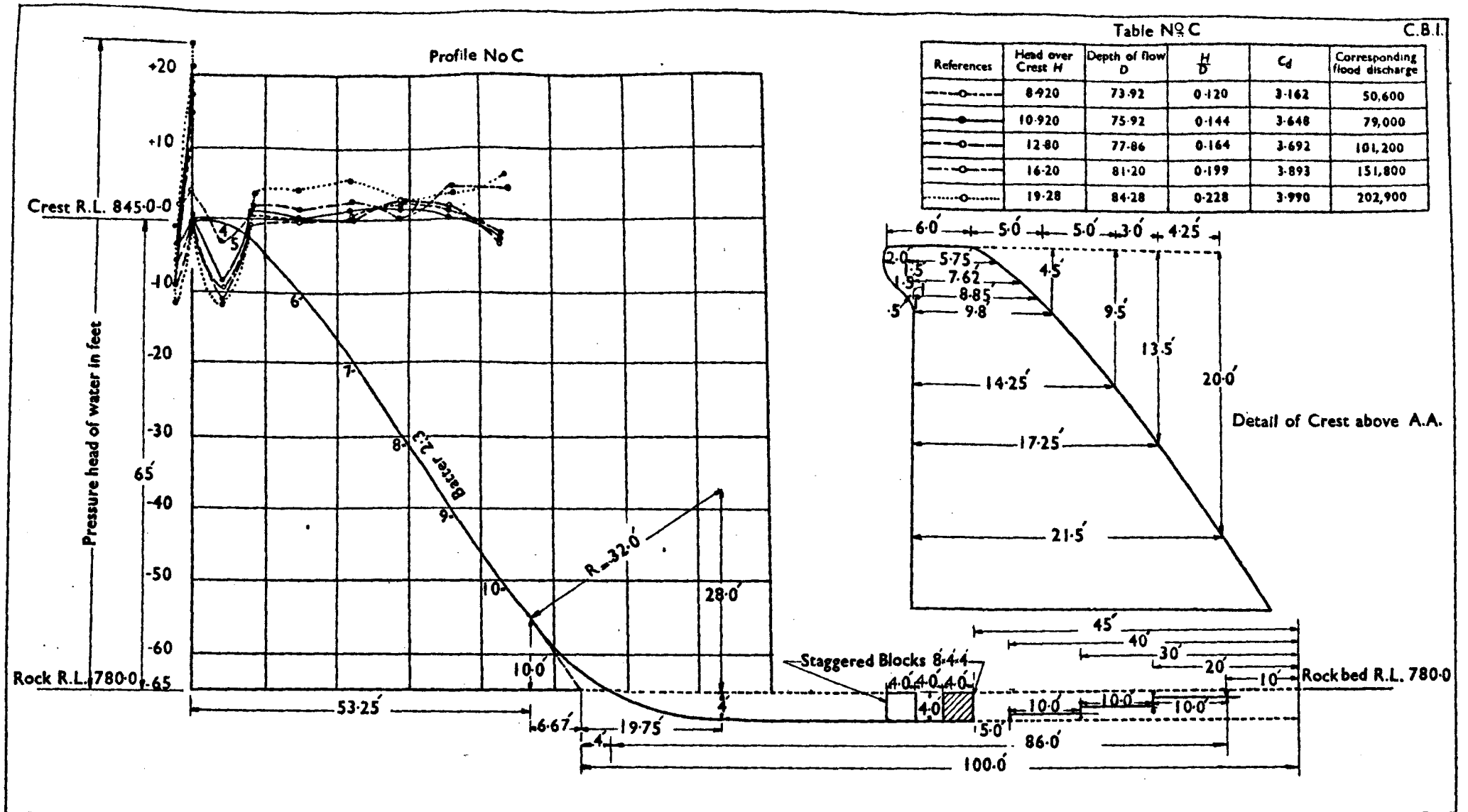


Figure 6C.34:—Showing Section of a high Coefficient Weir for Nagwa Reservoir [Curves showing Pressure Variations for Discharges other than Maximum Flood Discharge]

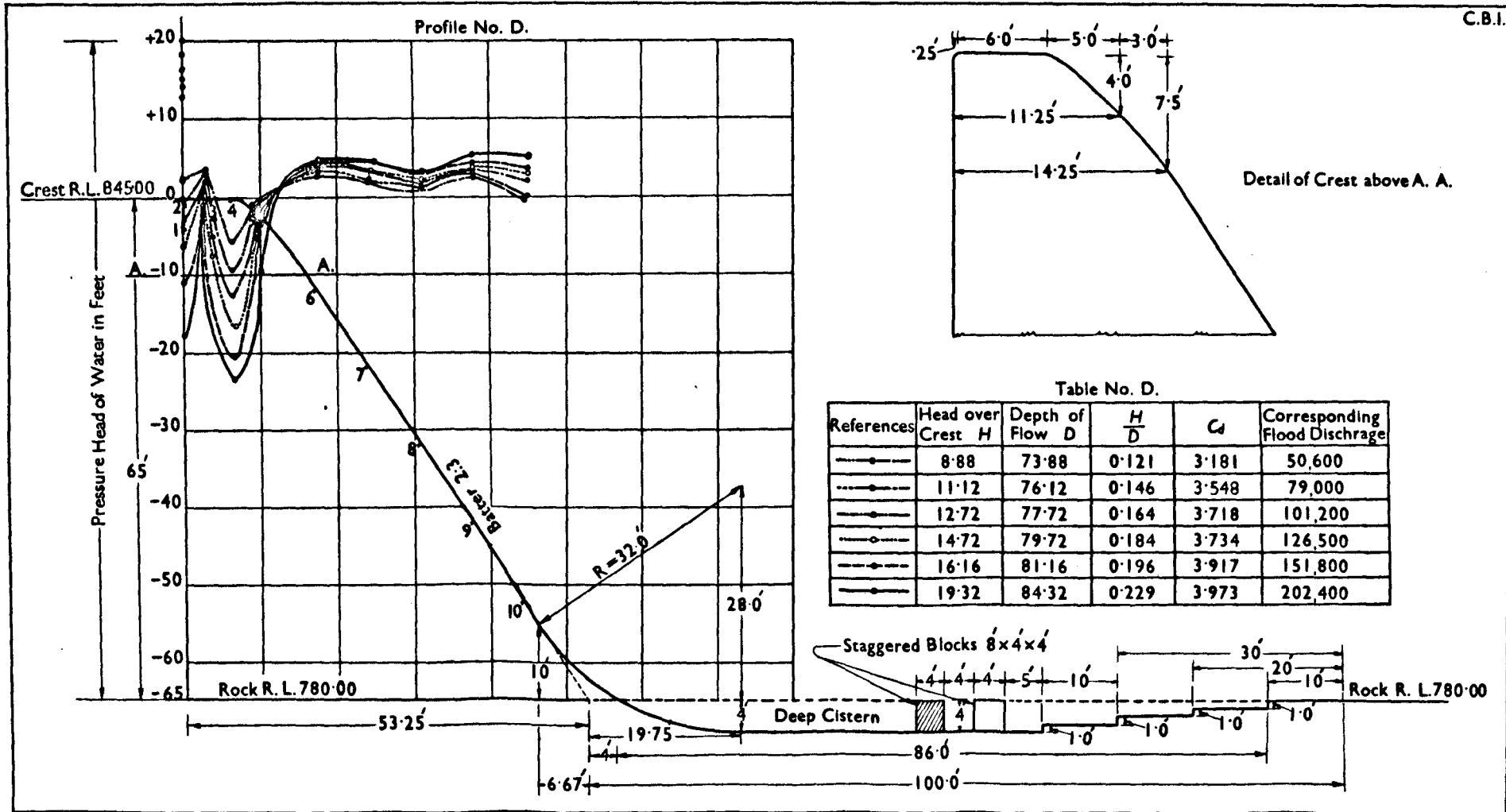


Figure 6C.35:- Showing Section of a High Coefficient Weir for Nagwa Reservoir [Curves Showing Pressure Variations for Discharges other than Maximum Flood Discharges]

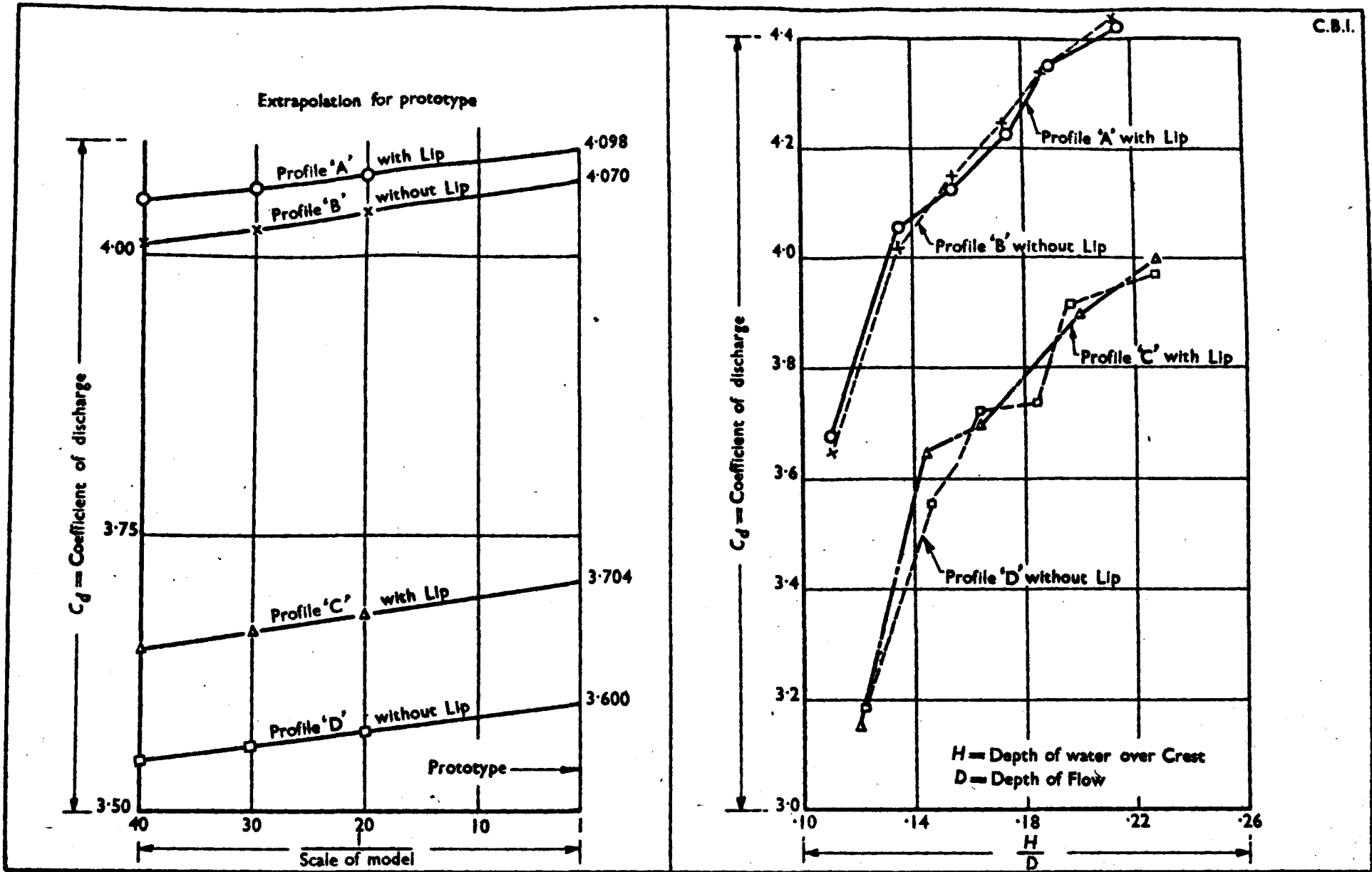


Figure 6C.36 & 37:- Showing High Coefficient Weir for Nagwa Reservoir

TABLE 6C. 20

Pressure observations at various points in weir profile No. A. Figure 6 C. 31.

Pipe No.	Depth of pipes below crest of weir (feet)	Pressure head in feet of water, atmospheric pressure being taken as datum					
		Q-50,600	Q-79,000	Q-101,200	Q-126,500	Q-151,800	Q-202,400
1	6.00	14.08	16.53	18.33	19.89	21.50	24.62
2	2.50	6.58	6.04	4.55	1.73	1.96	1.46
3	0.00	-0.92	-3.63	-6.12	-10.10	-13.66	-19.04
4	1.50	-0.42	-0.96	-1.95	-3.60	-5.50	-7.54
5	4.00	1.08	1.37	1.38	0.90	1.50	1.30
6	11.50	0.58	1.34	1.38	1.73	2.17	4.13
7	21.50	0.58	0.70	1.05	1.06	2.17	3.46
8	31.50	2.08	-1.96	-1.62	1.06	2.50	1.80
9	41.50	1.58	1.37	2.72	2.63	4.17	6.46
10	51.50	-2.92	0.03	0.38	3.06	6.84	8.80

Q : Corresponding flood discharge in cusecs

TABLE 6C. 21.

Pressure observations at various points in weir profile No. B. Figure 6C. 32

Pipe No.	Depth of pipes below crest of weir in feet	Pressure head in feet of water, atmospheric pressure being taken as datum					
		Q-50,600	Q-79,000	Q-101,200	Q-126,500	Q-151,800	Q-202,400
1	6.0	13.33	15.83	17.33	18.66	20.00	22.50
2	3.0	9.17	10.00	10.50	10.83	10.83	11.00
3	0.0	-4.17	-5.33	-8.00	-12.00	-16.33	-20.50
4	3.5	0.83	0.00	-0.50	-1.66	-2.66	-5.83
5	5.0	2.50	2.33	2.00	1.66	0.83	-0.33
6	15.0	2.33	2.67	3.00	3.33	3.66	4.17
7	25.2	2.00	2.83	3.33	3.67	4.17	5.50
8	35.0	0.33	1.33	1.67	1.67	2.33	3.00
9	45.0	4.00	4.00	5.33	5.50	5.50	7.50
10	55.0	2.00	2.67	3.33	4.66	5.00	6.10

Q : Corresponding flood discharge in cusecs

TABLE 6C. 22

Pressure observations at various points in weir profile No. C. Figure 6 C. 34

Pipe No.	Depth of pipes below crest of weir in feet	Pressure head in feet of water, atmospheric pressure being taken as datum				
		Q-50,600	Q-79,000	Q-101,200	Q-151,800	Q-202,400
1	4.5	13.58	16.08	17.63	21.20	24.44
2	1.0	1.59	-3.74	-6.36	-10.13	-12.05
3	0.0	4.26	0.59	0.03	0.37	0.05
4	0.0	-3.74	-8.41	-9.66	-11.13	-11.39
5	2.5	0.92	1.75	-0.20	1.87	3.78
6	10.0	0.09	0.74	0.47	1.54	4.95
7	20.0	0.08	1.75	0.03	2.03	5.28
8	30.0	2.91	1.92	2.70	0.20	2.62
9	40.0	0.09	0.92	1.80	4.87	4.62
10	50.0	-1.58	-1.63	-1.70	5.54	6.23

Q : Corresponding flood discharge in cusecs

TABLE 6C.23

Pressure observations at various points in weir profile No. D. Figure 6 C. 35

Pipe No.	Depth of pipes below crest of weir in feet	Pressure head in feet of water, atmospheric pressure being taken as datum					
		Q-50,600	Q-79,000	Q-101,200	Q-126,500	Q-151,800	Q-202,400
1	4.0	12.33	14.33	15.33	16.33	18.66	20.33
2	1.0	2.66	0.00	-2.66	-5.83	-11.00	-18.67
3	0.0	3.67	3.33	2.67	1.33	0.66	1.33
4	0.0	-5.33	-9.33	-12.67	-16.67	-21.00	-23.67
5	2.0	-0.67	-1.33	-2.33	-4.00	-5.33	-9.33
6	12.0	2.67	3.00	3.67	3.83	4.00	4.67
7	22.0	2.00	2.50	3.00	3.33	4.00	4.67
8	32.0	1.00	1.33	1.67	2.33	2.67	3.33
9	42.0	2.67	3.00	3.33	4.00	4.67	5.67
10	52.0	-0.33	0.00	2.00	3.00	3.67	5.00

Q : Corresponding flood discharge in cusecs

CONCLUSIONS

1. From the comparison of the values of C_d with or without lip in the two sets, *vide* 6C.37 it is evident that C_d increases with increase in H/D ; values being slightly higher, only at certain discharges, for profiles with the projecting lip than without it. It will be realised, therefore, that a lip on the upstream side does not always appreciably influence the discharge as the beneficial effect of the slight reversal of flow is offset to a certain extent by the increased curvature of the filaments.

2. For the observed range of discharges, negative pressure in the region of crest increases as H/D increases, the increase is below cavitation pressure even for the highest discharge. The nappe was adherent, and there seemed to be no danger of its separation.

3. From the curves of pressures it is evident that increase in negative pressure is accompanied by an increase in discharge. Apparently there seems to be no danger of pulsation in the region of crest as a result of pressure variations in profiles *vide* Figures 6C.31 and 6C.32. It is, however, markedly significant or profiles *vide* Figures 6C.34 and 6C.35. Some extra reinforcement is therefore necessary to safeguard against pulsation in the case of flat-topped crests.

It is generally accepted that, although experiments on small scale models as have been discussed above, cannot be relied upon to give accurate quantitative results for determining the behaviour of much larger prototypes, qualitatively their indications are of direct importance in actual design. It is proposed to take up a series of experiments at Bahadradad for further study of this structure at various bigger scales to know the operations of large scale spill-ways with greater accuracy.

(14) RASUL HYDEL SCHEME SPILLWAY SIPHON ⁽²¹⁾

A number of designs were tested and their priming depths and priming times were observed.

Different designs of lips were also examined on the model. The tests showed that a lip depressed by 1.5 feet below the full supply level and with horizontal projection was the best for priming. A depriming pipe placed four feet away from the lip was found suitable.

The work on the Rasul Hydel Siphon model was continued and the following investigations were completed.

- (a) Level and shape of the end of the air intake pipe
- (b) The angle and level of the exit nozzle
- (c) Pressures at different points in the syphon

The conclusions were :—

- (1) A sharp edged bell-mouth end of the air intake pipe with an area of the end 1.5 times the area of the pipe, and fitted at R.L. 788.25, gave the best results.
- (2) The exit nozzle when fitted at an angle of 35° with the horizontal threw the jet to the maximum distance of 87.6 feet from the nozzle.
- (3) The most suitable level of the exit nozzle was at R.L. 733.5.

⁽²¹⁾ The Punjab Irrigation Research Institute, Lahore, Annual Report, 1946 pages 5 and 6.

- (4) The pressure at the crest of the siphon was negative with an average value of 16.2 feet of water.

After completion of the above tests a fresh design of the siphon spillway was received. The total fall was split up into a siphon and three falls.

The conclusions were :—

- (1) A straight lip might be adopted.
- (2) The crest of the siphon should be joined to the bed of the approach channel through a vertical slope and a transition curve of 1.5 feet radius.
- (3) The area of the mouth should be 1.7 to 1.9 times the throat area,
- (4) The maximum negative pressures on the crest and crown equalled in magnitude, 22.0 feet and 8.4 feet of water respectively.

(15) BHAKRA DAM SPILLWAYS (22)

A closed tunnel and an overflow spillway with deep curved bucket, and with bucket shooting into the air were examined. By changing the vertical transition curve in the closed tunnel, negative pressures were reduced but not eliminated. The bucket shooting into the air has the advantage that it is open to inspection and repair.

DISCUSSION BY THE RESEARCH COMMITTEE

MR. T. P. KUTTIAMMU introduced items (1) to (6). Referring to item (2)-(4) first he said that the profile of an ogee spillway should be designed to coincide with the underside of the freely discharging nappe of water over a sharp crested weir of the relevant hydraulic particulars. The most efficient profile for the spillway was obtained. If the profile was made just coinciding with the underside of the nappe the coefficient of discharge would then be 3.96. In all the tests made with such profiles it was found that negative pressures were obtained in the region below the crest, but these were of small magnitude, say less than five feet of water and could be ignored. If the profile were slightly in excess of this minimum even then negative pressures were eliminated. The masonry profile given by Creager in his book on Masonry Dams gave such a design which was quite sound and economic. No negative pressures were produced, but then the coefficient of discharge was reduced to 3.8. In Madras this design was adopted for most of the spillways.

MR. KUTTIAMMU continued to say that in the United Provinces Report Mr. Chaturvedi had referred to high coefficient profiles for a 65 feet high dam giving negative pressures as high as 20 feet of water. Such designs were likely to develop vibrations in the structure and may not be quite sound. A high coefficient weir was all right where the height was not great. But in the case of dams it was advisable to avoid negative pressures of high magnitude developing near the crest.

(22) The Punjab Irrigation Research Institute, Lahore, Annual Report, 1946 page 6

Referring to items (5) and (6) MR. KUTTIAMMU explained an important feature of the stilling basin design. The simplest design of a stilling basin consisted of one formed by a bucket, a horizontal floor sunk below average bed and a vertical sill. Dissipation of energy in this case was effected by a standing wave. But the formation of a standing wave was influenced (1) by the discharge over the dam (2) by the downstream or tail water depth and (3) by the dimensions of floor sill *etc.* For a particular discharge and tail water depth a simple stilling basin could be designed as would give a perfect standing wave. But if either the discharge or the tail water depth was varied the nature and efficiency of the standing wave was affected. He took the example of a stilling basin giving a perfect standing wave having a large elliptical back roller for a particular set of water levels upstream and downstream. If now the tail water was gradually lowered or the discharge was gradually increased, the shape of the hydraulic jump changed. Its crest rose above the tail water level forming a "boil" as it was called. There was then a dive and a secondary standing wave. This caused scour downstream of the sill, the scour being almost a reflection of the boil. The higher the boil, the deeper was the scour. The boil was an indication and a measure of undissipated residual energy. If now the tail water level was still further lowered the standing wave is washed clear out of the basin, the hypercritical jet shot over in a powerful sheet of water and struck the river bed downstream with undiminished energy. This would certainly cause deep scours in the bed and should be avoided for ordinary designs.

In the case of spillways with crest shutters it was quite possible to have wide variations in discharge and tail water conditions and the stilling basin should be so designed as would avoid the shooting flow and will yield a standing wave for the entire range of variations. With a simple sill this meant that we might have to put up with a boil and there might be some amount of residual energy beyond the standing wave. A strong apron in continuation of the sill and perhaps an end sill or deflector at the end of the apron might then become necessary.

MR. KUTTIAMMU stated that in 1947 he continued the studies and found that by providing two rows of staggered blocks inside the basin just upstream of the sill called floor blocks for convenience the efficiency and range of usefulness of the standing wave could be very much increased. With such design a strong apron downstream of the sill might be dispensed with.

In some cases, as in the Ramapadasagar Dam, it is very costly to have an apron because of foundation difficulties, the depth of sand in the river bed is as great as 150 feet (and any apron would be very costly). In such situations a roller bucket was the best design. Experiments for obtaining a suitable design of bucket of the Ramapadasagar were described in the Report. The height and shape of standing wave and the depth and position of scours were all affected by flow conditions and by radius and invert level of buckets. By cutting off the edge of the bucket at different angles the height of the standing wave could be lowered. Another factor that had to be examined in these tests was regarding the possibilities of abrasion of the concrete by the sharp and carried on to it by the rollers.

In the studies carried out in 1947 in Madras a real shooting bucket with invert at +74 was tested and found suitable in many respects.

Referring to item (1) Mr. KUTTIAMMU said that scours downstream of a gated spillway having a large number of spans depended also on the way in which these gates were operated. Experiments were made on a model with five vents and contours of scours obtained with various combinations of gate openings were plotted. A study of these indicated how very important it was to have proper regulation of shutters during floods. With a uniform bed downstream it was best to open all shutters uniformly to the required height for the disposal of the surplus, instead of opening only a few to greater heights. In case only a few shutters could be opened, it was better to distribute them evenly over the full length of the regulator and to prefer a symmetrical arrangement of opening. The idea was to prevent or minimise the effect of return flow and eddies.

Mr. S. N. GUPTA introduced items (7) and (13) and said the experiments conducted in the United Provinces on siphons showed that in the case of a big saddle siphon of cross-sectional area 16 feet \times 9 feet or so as in Pahuj, two things viz. (1) the provision of a suitable lip and (2) a sealing cistern at the outlet were essential for attaining the minimum priming depth. It would be seen from Table 6C.4 that the provision of 1½ inches lip in Pahuj saddle siphon reduced the priming depth from 12 inches to 7 inches whereas the co-efficient of discharge is affected but slightly.

It was learnt from Mr. Ganesh Iyer during his visit to the United Provinces in March last that in similar actual saddle siphons installed at Kanva in Mysore State, which came into operation, for the first time after their construction, during the last monsoons, the priming depth was observed to be two feet and siphons were not running full. That it would be so was self explanatory in the absence of a suitable lip and a sealing cistern. Increasing the capacity of the priming volute siphons would help matters, but not very much.

Experiments on volute siphons indicated that the length of outlet played a significant part both for minimum priming depth and maximum coefficient of discharge. Figure 6C.20 showed that 12.0 feet long outlet for a head of 25.0 feet gave optimum results. In other words, the length of the outlet should be a certain minimum and according to these experiments it was nearly half the working head. Mr. Ganesh Iyer agreed with the above conclusion and he was of the opinion that 22.0 feet long outlet of volute siphons at Hirabhasagar Dam might not be adequate to give the desired results. Further experiments with different diameters and heads to determine the optimum length of outlet were in progress at Bahadurabad.

Mr. GOVINDA RAO introduced items (9) to (12) and briefly described the experiments carried out. He said item (9) related to the effect of altering the shape and number of volutes in a volute siphon. The volutes and the funnels are two important parts in a siphon which determine its performance. Experiments were made with a nine inches model operating under 3.25 feet. A slope of 1½ to 1 was considered the optimum and was adopted for the funnel. Any steeper slope decreased both the priming depth and the coefficient of discharge. Any flatter slope had the opposite effect. The effect of altering the number, height, shape and curvature of the volutes was next studied. It was found that twelve volutes of 1½ inches height gave the best results.

MR. GOVINDA RAO also described briefly the experiments and conclusions obtained *vide* items (10) to (12).

RAO BAHADUR JOGLEKAR introduced item (8) and said that in high dams it was necessary to see that the jets issuing from siphon outlets did not cause damage to the river bed by impact. The volute siphon designed by Ganesh Iyer was unique in design, in that it had got a very high discharging capacity and a low priming depth. He had already got two siphon models of the Hirebhasgar design. These were used in the experiments to find the effect of adjacent jets hitting each other at various angles, on the depth of scour downstream. These experiments could not unfortunately be started earlier as information regarding volutes was obtained from Mysore only by the end of December 1946. Scour was observed on the bed downstream, which consisted of 17.3 mm. single and 6 mm. coarse sand. The drop from the centre of the outlet to the downstream, water level was equivalent to 12 feet. With jets hitting at an angle of 18° , energy was not dissipated effectively because the two jets combined to drop down vertically as a single jet, practically producing the effect of the sum total of the two. When the angle was changed to 30° , slightly better results were obtained; but the maximum depth of scour was 1.55 times that obtained with only one siphon. In these experiments the coefficient of discharge remained unaffected as there was no back pressure. Further experiments with outlets tilted upwards so that the jets travel upward before hitting each other and experiments where the outgoing jet was fanned out before incidence of the jets and other devices would be carried out.

DR. N. N. BHANDARI in a written discussion referring to item (5) said that he had an opportunity of studying the same problem in the University of London in 1941. The line of attack was also almost identical to that followed by Mr. Kuttiammu. With a constant head upstream of the crest of an overflow dam, as the downstream depth was gradually varied the type of flow, changed considerably and abruptly at certain depths of tail water. Figure 6C.38 gave an idea of the nature of four different flow patterns.

Continuing DR. BHANDARI said that with sufficient depth of tailwater a perfect standing wave was formed, but if the depth of the tail water exceeded a certain value the effect of the sill was not felt at all and the water moved along the downstream bed just as it would do in the absence of a sill *i.e.* the high velocity jet, after having been thrown up slightly, once again moved along the bed. As a result, any movable material that came in the way, would be carried away by the current. Figure 6C.38(a) illustrated this type of flow. As this flow was mainly along the bottom, it was named by DR. BHANDARI as "Bottom Flow". It might be pointed out that "Bottom Flow" moved along the bed "un-separated" from the boundary.

As the downstream water level was lowered a definite critical point was observed where the flow suddenly "separated" or "broke-away" from the sharp edge of the sill. The high velocity jet was thrown up to the surface and thereafter moved along the top. This type of flow was named by him as "Top Flow", and was represented by Figure 6C.38(b). The flow along the bed was reversed in direction as was characteristic of the phenomenon of "separation" or "break-away" as it was sometimes called. This type of flow was favourable where the prevention of erosion was desired.

If the tail water was further lowered, the downstream water became less than what was required to meet the momentum of the high velocity jet deflected by the sill. As a result as shown in Figure 6C.38(c), the jet formed another

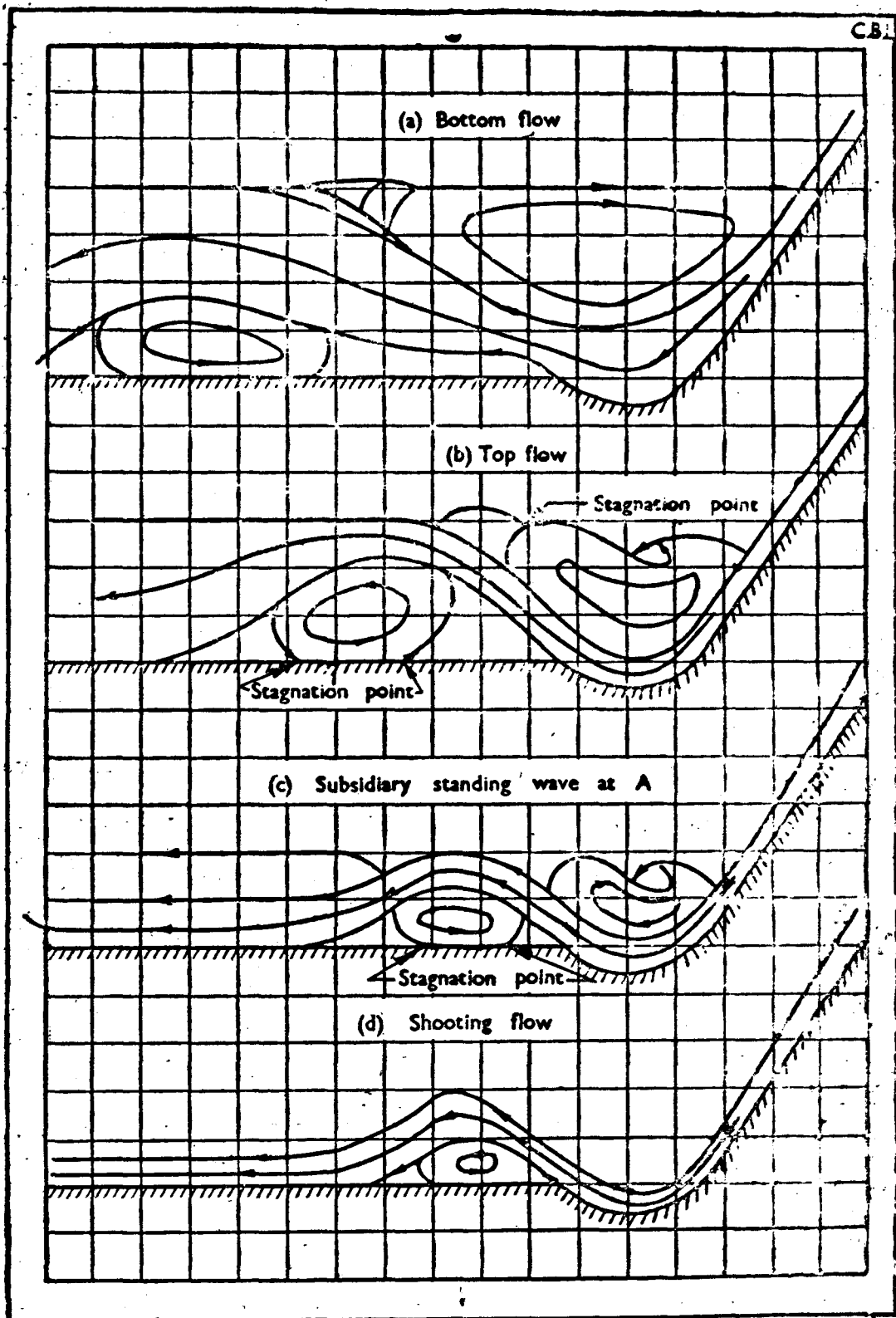


Figure 6C. 38 Showing the nature of four different flow patterns.

hydraulic jump, downstream of the sill, which might be called a subsidiary hydraulic jump. The formation of this subsidiary hydraulic jump was not desirable because the high velocity jet shot through the downstream depth of water, on to the erodible bed causing scour, some distance downstream of the sill so that this flow placed a limitation on the "Top Flow" discussed in the previous paragraph.

Still further lowering of the downstream water level caused the main standing wave to shoot clear of the sill. The whole of the discharge came down in a jet on to the bed causing great havoc if the bed was erodible. This form of flow was shown in Figure 6C.38(d).

Reversing the process, that is, increasing the downstream water level only raised the critical point where the "Top Flow" changed to "Bottom Flow".

The two limits of the "Top Flow" gave the range of the utility of any energy dissipation arrangement.

DR. BHANDARI had also carried out a series of tests with various sizes of sills and dimensions of cisterns with different discharges, and had observed a marked effect on the critical points where "Top Flow" changed into "Bottom Flow" and *vice versa*. It was also noted that at a particular discharge a certain size of sill became useless because there was no top flow taking place at all. It was either bottom flow, with high downstream level or shooting flow with a low level downstream. In some other cases it was found that the discharge available in the laboratory flume was not enough to cause bottom flow at all. It was always top flow which changed to shooting flow with a low downstream level.

DR. BHANDARI had come to the following definite conclusions :

- (1) The efficiency of a sill increases with its height.
- (2) The quantity of discharge that can safely be passed over a structure in the presence of a sill increases with the height of the sill.
- (3) The angle of sill varied with the slope of the glacis Figure 6C.39. The flatter the glacis the steeper is the sill that is required. For horizontal flow it is the rectangular type of sill that is most useful. Therefore, if the sill is placed just at the toe of glacis a triangular sill is required and if the sill is placed at the end of a horizontal floor it is a rectangular sill that is wanted.

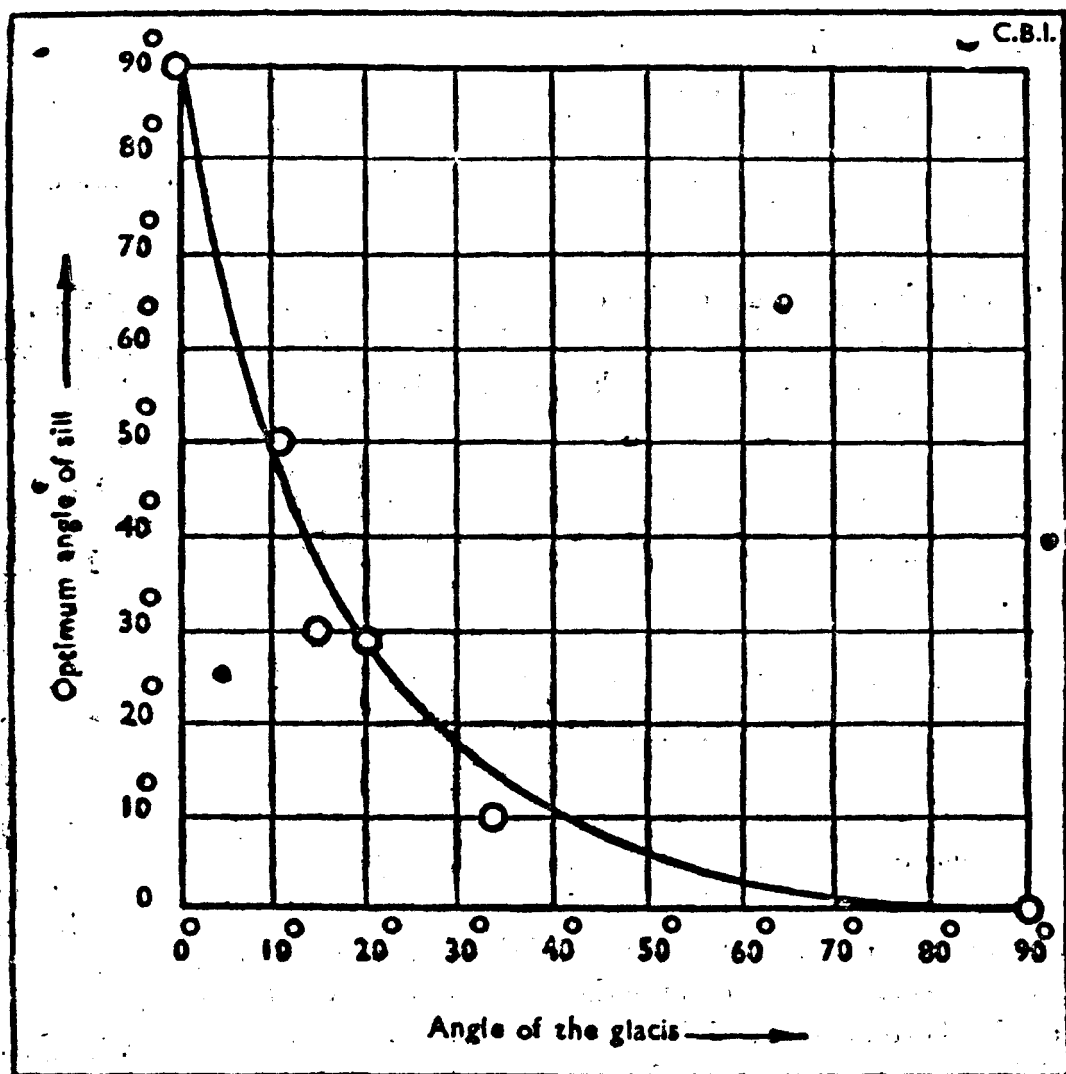


Figure 6C.39

- (4) If there is a roller bucket installed in between the glacis and the sill then the height of sill required decreases with the increase in the radius of curvature of the bucket and *vice versa*.
- (5) Results obtained from the experiments confirmed those obtained by Rehbock and Schocklitsch regarding the depth and length of cisterns.

Referring to item (13) DR. N. N. BHANDARI said that a series of experiments had been carried out on profiles given in Figure 6C.30. It was interesting to note that for profiles No. 1 to 6 in this diagram the coefficient of discharge went on increasing as the crest of the weir was made more and more sharp. This evidently conformed to the general idea that the coefficient of discharge was more in the case of a sharp crested weir than in a broad crested weir.

The downstream slope had very little effect if any on the coefficient of discharge of weirs (23).

(23.) Bhandari N. N. "Weir Coefficient," Reports and papers for the postponed meeting, 1939, of the International Association for Hydraulic Structure Research (Liege) page 115.

In Figure 6C.37 the value of C with and without lip were compared. But it was not indicated anywhere if the rest of the profile in both cases was the same. Coefficient of discharge was affected by the shape of the lip to a considerable extent as was shown by White and Burns⁽²⁴⁾ Rehbock⁽²⁵⁾ and others. The result would have been very useful if the coefficient of discharge was plotted against the radii of curvature of the lip and of the portion of the crest downstream of the lip.

In the opinion of DR. BHANDARI extrapolation of a curve passing through only three given points, as done in Figure 6C.36, was not justified. The minimum number of points should be four.

MR. KUTTIAMMU in reply to the discussion by Dr. Bhandari said that the remarks by Dr. Bhandari were very interesting and contained a clear exposition of the nature of flow downstream of spillways. Dr. Bhandari's conclusions were generally acceptable, of course with certain limitations which MR. KUTTIAMMU said he had already explained. For example, MR. KUTTIAMMU said that DR. Bhandari had stated that the efficiency of a sill increased with its height, and MR. KUTTIAMMU had already explained how the shape and efficiency of a standing wave and hence depth and position of scours varied with height of the sill. There was then always an optimum position which gave the best overall result.

MR. FRAMJI said that in the Mysore experiments the jets just met each other and fell conjointly. They should first strike each other, dissipate the energy and then fall on the ground.

THE PRESIDENT agreed with MR. FRAMJI.

THE PRESIDENT asked if any body had tried to go into the mathematics of volute siphon and requested Mr. Blench to speak on the subject.

MR. BLENCH said he had not given the subject sufficient thought to justify an extemporaneous comment on the mathematics, which would be three dimensional. He pointed out that, without whirl, such a siphon was unsuitable for priming; with whirl the conditions for sucking air became present, but the coefficient might drop.

MR. CHATURVEDI said that further progress in the design of volute siphon had been made in the United Provinces as a result of the initiative of Mr. Hutchinson. They had been able to combine four volute siphons placed in one line at right angles to the dam, with only one outlet tunnel, saving lot of money and space. The area of the outlet tunnel was increased gradually from inside to outside.

In reply to the President MR. CHATURVEDI said that the diameter of the shaft was kept at 12 feet.

THE PRESIDENT remarked that one could have a siphon of much larger diameter in place of having four shafts.

MR. CHATURVEDI replied that there was a limit to the diameter that could be used.

(24). Burns R.V. and White C.M. "Protection of Dams, Weirs and Sluices against Scour" Journal of the Institution of Civil Engineers, November 1938 page 28.

(25). Rehbock, Th. Handbuch der Ingenieur-Wissenschaften, Vol. 2 (Part 3, Section 1), page 54, Leipzig, 1912.

THE PRESIDENT said that siphons upto 30 feet diameter had already been designed.

MR. CHATURVEDI said that in that case the siphon would not prime.

THE PRESIDENT said that there was no reason why they should not.

MR. CHATURVEDI said that in any case 30 feet diameter was not practicable in every case.

MR. SAXENA said that experiments were carried out to provide one single cover over all the siphons but that did not work.

MR. FRAMJI pointed out that with the arrangement as described by Mr. Chaturvedi there would be difficulty in the dissipation of energy too.

SYED DILDAR HUSSAIN asked Mr. Chaturvedi how would such an arrangement be constructed.

MR. SAXENA described the method of construction. He said that coefficient of discharge of 0.71 had been reached. Formerly it was only 0.50.

THE PRESIDENT asked Dr. Bose to say whether he had been able to carry out any mathematical investigations on spillways.

DR. BOSE said that he had been trying to get a physical picture of the whole process. It was not clear to him as to what happened inside the barrel of the siphon after the water passed through the volutes into the barrel. Was it still rotating? He would like to get some sort of insight on the following questions from workers in this field :

- (a) What happens to the water as it passes through the volutes and enters the barrel. Is it still rotating?
- (b) Is there an empty space in the centre of the mass of water going down the barrel?
- (c) Does the outside of the rotating volume of water touch the barrel walls. If not, what is the pressure in this space."

DR. BOSE said that if the barrel could be given a shape following the outside of the rotating liquid the coefficient of discharge could be made very high. He requested Mr. Govinda Rao, to let him have a reply on these points. DR. BOSE said that unless the physics of the problem was clearly understood, mathematics could not be worked out.

MR. N. S. GOVINDA RAO said he would reply Dr. Bose on the basis of the model experiments he had conducted " Regarding question (1) the water is seen still to be rotating in the barrel after skirting round the volutes. This is clearly seen before priming. After the siphon primes, the entire dome is filled with water full to the crown. Water as it issues from the outlet is seen to be discharging with a twist (2) There is no empty space in the centre of the mass of the water going down the barrel. The volutes are so shaped that the flow from all of them impinge at the centre of the vertical pipe. (3) It is not possible to observe whether the outside of the rotating volume of water touches the

barrelwalls or not. All the water filaments issuing from the funnel are made to meet at a point from where they may diverge and fill the barrel as in Figure 6C.40. The space ABCDE may be empty."

DR. BOSE said that in such a model, one could only see the top of the flow.

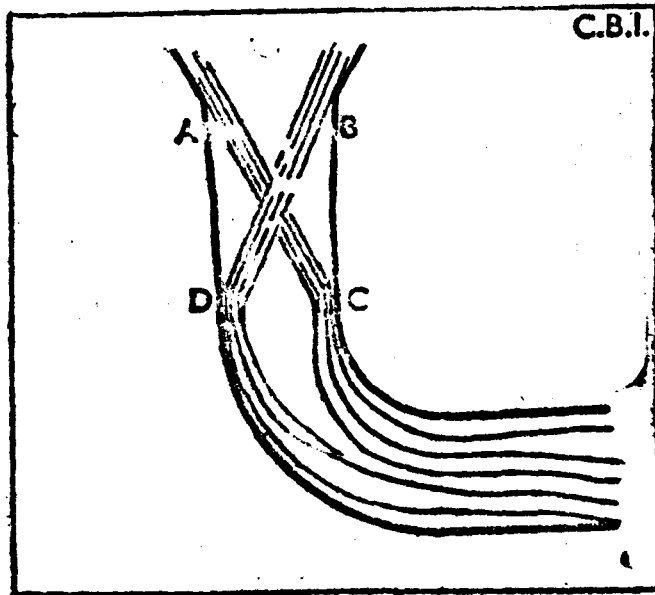


Figure 6C.40

MR. FRAMJI said that the action described by Dr. Bose took place only up to the time when priming took place. After that the flow through the siphon was full bore.

MR. GOVINDA RAO corroborated Mr. Framji's statement and explained the action that took place. It was full bore because of the outlet tunnel.

DR. BOSE said that if there were no outlet tunnels the flow would be as he had described above.

MR. GOVINDA RAO said that in the absence of the horizontal outlet tunnel, the flow would be like that through a vertical pipe.

MR. FRAMJI corroborated Mr. Govinda Rao's statement.

THE PRESIDENT asked Rao Bahadur Joglekar if he had taken any pressure observation inside the siphon.

MR. CHATURVEDI referred to his report where pressures inside the siphon had been reported.

RAO BAHADUR JOGLEKAR explained the position of negative pressure in the siphon. He said that the outlet ran full bore only if the length of the outlet was more than a certain minimum otherwise air was sucked in from the downstream end and the flow was not full bore.

MR. CHATURVEDI stated that pressure had been observed for such a length.

RAO BAHADUR JOGLEKAR said that maximum negative pressures obtained just below the bellmouth of the volute siphons and on the inside of the outlet bend ⁽²⁵⁾. These indicated the places where cavitation is likely to occur. He pointed out that though 30% increase in discharge was obtained in 1/8th scale

⁽²⁵⁾ Central Irrigation Hydrodynamic Research Station, Poona, Annual Report (Technical), 1942-43, figure 37 and page 26.

model when its outlet was flared; the percentage increases in full scale 500 cusecs; saddle siphon was only three per cent. and cavitation took place as the large siphon was working near the limiting negative pressure (26).

THE PRESIDENT suggested that the profile of the siphon should be changed whenever cavitation was found to exist.

RAO BAHADUR JOGLEKAR said that that could be done. He emphasised the danger of assuming the coefficient of discharge in the prototype from model experiments under such condition.

THE PRESIDENT remarked that that was exactly why he was particularly anxious about the mathematical analysis of flow through siphons.

MR. GOVINDA RAO reported that measures had been taken to observe pressures and discharges on the Hirebhasgar Dam.

THE PRESIDENT suggested that work should be done on siphons on a co-operative basis by deputing research staff to Mysore.

DR. BOSE asked if any one had tried to introduce air into the places where negative pressure occurred to avoid cavitation.

RAO BAHADUR JOGLEKAR said that he had tried that. It broke the cavitation but reduced the coefficient of discharge.

DR. BOSE said that this was the method followed in the prevention of separation of the boundary layer from airfoils. R. B. Joglekar's experiments showed that the coefficient of discharge and cavitation were inter-connected.

MR. FRAMJI referred to items (2) and (3) and said that the pressure depending on the velocity of flow was affected by scale, so models of more than one scale were required and the pressures extrapolated for the prototype. With larger weirs negative pressures were considerably greater. It should also be remembered that a small error in the profile might have a big effect on pressure, hence a considerable margin of safety was desirable.

Referring to item (4) he said that the experiments had been done with only one scale model.

Due to interchange of momentum between filaments of different velocities by viscous shear, and as the viscous effect reduced with an increase in scales higher coefficients were obtained in geometrically—similar weirs of greater scale. Another dissimilar feature in the boundary layer, was the relative thickness and surface drag which were also dependent on viscosity and hence on scale. It was therefore, necessary to carry out experiments with three or four models of different scales and to extrapolate the value of the coefficient for the full scale weir. Experiments of this type were carried out at Poona during 1937-38, 1939-40 and 1941 and were reported in the Annual Reports of those years. An extrapolated values of about $C=5$ was obtained for the most efficient section evolved at Poona while the Madras Report figures are much lower probably due to the small scale of the model.

(26) Ibid.

Regarding item (6) Mr. FRAMJI continued to say that a very exhaustive series of hydraulic laboratory studies on dam spillways were performed by the U. S. Bureau of Reclamation and an analysis of these results had been ably presented in an article on "Protection against Scour below overfall Dams" by Lane and Bingham in the Engineering News—Record of March 14, 1935.

The ideal condition was to have a tail-water at such a height above the river bed for each discharge so as to form a perfect hydraulic jump for the depth and velocity which would occur in the overfalling stream at the toe of the dam for that discharge. But as the tail water elevations were controlled, by the river conditions below the dam this ideal was never attained. Frequently the tail water rating curve and the curve of jump heights lay above or below or crossed each other. The design of the stilling device must depend on the relation of these curves and Lane and Bingham's analysis followed this natural classification. The devices tested should also, accordingly, be classified as useful for one or other of the four classes of conditions.

The flumes used for the models appeared too small in length to give still entry conditions and the depth of sand over the rigid bed quite insufficient for the worst conditions to be tested for maximum water level upstream and low water level downstream.

The Ramapadasagar bucket experiments with $\frac{1}{60}$ model were obviously run for too short a period (45 minutes) to give maximum scour, if the smaller model $\frac{1}{200}$ required four hours for stabilizing.

An intermediate condition of tail-water level might bring a deeper scour nearer to the work than with minimum downstream water level and should have been tested.

MR. GOVINDA RAO referring to the experiments made for preventing scour below Ganesh Iyer's volute siphons at the Poona Research Station stated that turning the position of the horizontal outlets through 18° or 30° did not materially help in dissipating the energy of the flow. Since the angle of skew was very small, the point at which the two jets could meet was very far from the rear face of the dam. In the distance the jets from the outlets had to travel in the horizontal plane before impinging, they would be dragged down in the vertical plane considerably due to the action of gravity. This resulted in the two jets meeting only slightly above the tail race affording little scope for the water to fan out after the impact and fall in the form of a spray.

MR. GOVINDA RAO congratulated Mr. Chaturvedi on the valuable experiments he had made in designing the siphons for the Pahuj weir. He had brought in an additional item of outlets in the design of volute siphons. This had been recognised in our designs though not with the prominence, Mr. Chaturvedi felt they should have.

In the design of the saddle type siphon, the following well known three methods of effecting water seal had been adopted :—

- (1) Insertion of a lip
- (2) Water seal formation at the bend and
- (3) Auxiliary volute siphon.

It has been stated that the three inches lip has been reduced to $1\frac{1}{2}$ inches apparently in its width without materially effecting the coefficient of discharge and priming depth. If the lip width had been increased from three inches to say six inches or even 12 inches, its effect on the priming depth and discharge coefficient would have been more pronounced. While a greater width of lip may give lesser priming depth by shooting the jet to the outer shell more effectively, the discharge coefficient may correspondingly decrease due to the high loss of energy in the jet impacting on the outer wall, and thus lessening its velocity.

It is seen from Table 6C.4 that the saddle siphon with only the volute siphon 11 inches priming depth and 0.7 discharge coefficient with the addition of the lip—both the priming depth and the coefficient of discharge have decreased. This decrease in priming depth without sacrificing the discharge coefficient could have been obtained by merely increasing the capacity of the auxiliary volute siphon.

Water seal formation.—This has been achieved by tilting the outlet pipe upwards. This has naturally reduced discharge coefficient. Instead the water seal could have been formed a little away from the outer leg of the siphon by constructing a baffle wall to form a cistern with water level—first a little below the end of the vertical leg as Figure 6C.41

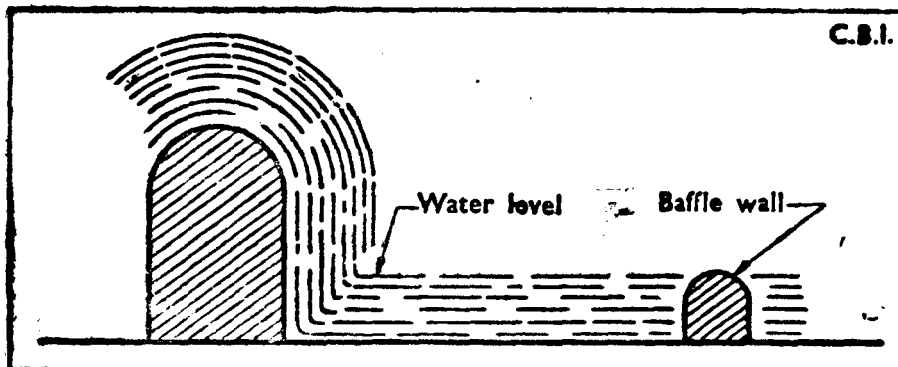


Figure 6C.41

Pressures.—Regarding observation of pressures and pressure contours these are evidently at the periphery only, as it is obviously impossible to measure pressures in the body of the siphon barrel without disturbing the few conditions. The section D in Figure 6C.16 is not the controlling section determining the operating head of the siphon. It is merely the point where the characteristics of the flow change. The controlling section is at BB where the discharge is proportional to the vacuum developed there.

Volute Siphons.—The experiments on outlet lengths necessary for minimum priming head and full bore discharge are very interesting. It has also been his experience that where the outlet length has not been sufficient it has not been possible to get full bore discharge due to air rushing in. The flow in an insufficient length of outlet will be merely that due through an orifice of diameter equal to the neck the tunnel and the operating head being equal to the head of water plus vacuum pressures about it. Where the outlet length is sufficient, the flow through the barrel will be similar to the flow through a

pipe under pressure and the operating head producing the discharge will be the difference of water level upstream and downstream of the reservoir or the centre for the outlet pipe according as the discharge is submerged or free.

RAO BAHADUR JOGLEKAR referred to item (7) and said that in the experiments in which the radius of curvature of outlet bends (R) was varied, it was not clear whether the head was kept the same by removing a part of the vertical leg to accommodate the extra height of outlet bends with increased radius of curvature.

In Figure 6C.15 sudden decrease in pressure at pressure point 2 on the inner shell at the crest indicated that the curvature of the crest was too flat and could have been improved by a smooth curve, between points 1 and 3. The Poona experiments had shown that a lip at the downstream end of the crest (meant to lower the priming depth) was detrimental for the discharge due to distorted curvature of flow and such a design was not recommended. Instead one could get low priming depths by providing a priming siphon.

Continuing, RAO BAHADUR JOGLEKAR said that in Table 6C.12, the effect of increasing the length of the siphon outlet pipe on priming depth and coefficient was shown. It appeared that the siphon did not run full-bore till the length of the outlet pipe was made 12.15 feet, when the priming depth was minimum and the coefficient maximum. This showed that with a shorter length, the curved flow round the bend did not properly redistribute in the horizontal portion of the outlet when the outlet length was less than 12 feet, thus admitting air from the downstream end, which delayed priming and reduced coefficient. Pressures in the outlet legs should be observed to elucidate this point. It was stated by Mr. Chaturvedi that the optimum length of the siphon outlets was probably half the height of siphons. More experiments were required to verify this point.

Regarding item (9), RAO BAHADUR JOGLEKAR said that the experimental results showed that the priming depth was reduced by increasing the height of volutes but the coefficient of discharge was adversely affected. So the design of volutes had necessarily to be modified to obtain a compromise between these two conflicting factors as regards coefficient and priming depth. A low priming depth was associated with a low coefficient. Similarly, there was an optimum height of volutes, which when exceeded did not reduce the priming depth; besides the coefficient was affected.

So far as the priming depth was concerned, the Poona experiments had shown that the results were not geometrically similar. The model always gave a higher priming depth than the prototype. So in a particular design of a volute siphon one would expect a lower priming depth for a full-scale siphon; but lower priming depth required volutes of relatively less height. In order to obtain a minimum priming depth in the prototype, it would be necessary to carry out experiments with models of three or more different scales and in each case the minimum priming depth should be determined by increasing the height of volutes. This would give a very useful relation of the height of volutes and priming depth with model scale ratios.

Further he referred to item (13) and said that experiments were carried out at Poona with a full scale high coefficient weir profile (27). A maximum coefficient of 4.84 was obtained with energy head over weir crest ($H=H_1+h_v$) of 2.741 feet and the ratio of energy head to total depth of river and velocity head was

$$\frac{H}{D+H} = \frac{2.741}{6.5+2.74} = 0.297$$

and not 0.57 as stated in the United Provinces Report.

In the United Provinces weir profile, the extrapolated value of C for prototype was 4.10 with $\frac{H}{D}+H=0.35$; whereas in the Poona profile of full scale weir, value of $C=4.40$ was obtained with the same ratio.

With increase in depth of water over crest or increase in the ratio of $\frac{H}{D+H}$ coefficient increased, which was due to the increase in curvature of flow over the crest, which increased negative pressures; but there was a limit beyond which, if the depth of water was increased the coefficient reduced due to separation.

When choosing a particular profile, one had to be careful not to choose the absolute maximum C but a slightly lower value of C on the rising $C : H$ curve otherwise there was a danger of separation occurring in case the maximum estimated discharge is exceeded. As regards the lip of the upstream face of the weir, with a properly designed lip, the coefficient increased; but one might have to omit it due to constructional difficulties and the likelihood of damage.

Flattening the weir crest to six feet to provide a foot path, the coefficient reduced from 4.1 to 3.64 which meant a reduction of storage depth of about seven per cent. To overcome this, a foot bridge with piers could be constructed over the weir and before the design was finalised, the relative merits of gain in storage capacity and the extra cost of a foot bridge should be considered.

Regarding item (15) in the Punjab Report on Bhakra Dam Spillways RAO BAHADUR JOGLEKAR said the Figures 6C.31 and 6C.32 of overflow spillway showed that the approached channel to the spillway was at right angles to the spillway which was quite incorrect for reproducing prototype conditions. The entry conditions would all be incorrect causing incorrect distribution of velocities and discharge concentrations. The results would, therefore, be undependable.

RAO BAHADUR VENKATACHARYA said that priming depth should be as low as possible. In Mysore they had tried removable vanes for the volutes so that once the priming had taken place the volutes could disappear. Those disappearing vanes improved the coefficient of discharge. For the dissipation of energy they had tried the method of impinging jets. They had also tried to divide the bore of the siphons. Relative distance between the siphons had yet to be worked out. These steps might solve the problem of coefficient of discharge and the dissipation of energy.

(27) Central Irrigation and Hydrodynamic Research Station Poona, Annual Report (Technical), 1940-41

MR. CHATURVEDI replied to the criticisms. Regarding the lip provided in the siphon as remarked by Rao Bahadur Joglekar, MR. CHATURVEDI said that every case required individual study. He said that the lip was provided to get the minimum priming depth. The circumstances were such in that particular case that they were prepared to do anything to reduce the priming length. As regarded the smooth curve suggested by Mr. Joglekar, he said he did not go in for that as several crests were tried just to know which was going to give the maximum discharge at low heads.

Regarding the suggestion of Mr. Govinda Rao for reduction of the co-efficient, several methods were tried and the present one was perhaps the best to satisfy the local conditions. The pressure had not reduced because of that. The pressure could only be measured at sides and there was likelihood of some discrepancy which he admitted. Coming to the point raised by Rao Bahadur Joglekar that the design might have been different to eliminate flood, MR. CHATURVEDI said that it was only a local consideration. He said that they were providing six feet shutters and that it was very helpful. He said that a lip should not have been added. He further said that some basic work had to be done in this respect. This, he said, had been discussed with Mr. Ganesh Iyer. As the subject was quite new this could be discussed still further and experiments had got to be done.

MR. KUTTIAMMU replying to the criticism of Mr. Framji said that Mr. Framji wanted the co-efficient of discharge experiments to be conducted on several models and prototype values to be obtained by extrapolation. Co-efficient of discharge over a dam was influenced by gravity, viscosity friction and atmospheric pressure. In the design of spillways he had already explained that no negative pressures were allowed near the crest. Hence the effect on atmospheric pressure could be neglected. The effect of viscosity was also ignored in these experiments. The model was worked on Froude's Law and if the scale was large enough to eliminate surface tension effects and if the friction was correctly adjusted one model should be quite sufficient to give satisfactory results. The scales were sufficiently large ($\frac{1}{48}$ or $\frac{1}{60}$). Friction in the model was not reduced sufficiently. It might, therefore be assumed that the coefficient in the prototype would be slightly higher. If there were negative pressure, the scale effect would be, considerable. MR. KUTTIAMMU said that he had studied the Khadakvasla experiments on high coefficient weirs. The scale effect in those models was due to the reduction of pressure created at the crest. As such reduction of pressure was not allowed in the Madras experiments there was no necessity for repeating the studies on other models.

As regards pressure observations there were limitations due to space discharge etc. The models were of scales $\frac{1}{36}$ or $\frac{1}{48}$. Smaller models would not be sufficiently accurate. In larger models the fluctuations would be large. There might be some scale effect in these, but these had not been examined.

Regarding the designing of a stilling basin to suit discharge and rating curve, MR. KUTTIAMMU said he had already explained how it was essential to have the design suitable not only for normal conditions of flow, but also for a wide range of extraordinary conditions as well. If this was not borne in mind the sill would fail to serve its purpose under certain very likely conditions.

MR. FRAMJI said that the tail water depth and rating curve might have to be designed for different conditions. That allows for various factors which might come up under different conditions.

THE PRESIDENT said that there had been the most interesting discussion on the subject. The general feeling was that these researches should continue before one could make final decision as to what was the best solution in each particular case. So far as the volute siphon was concerned, its mathematics should be studied in greater detail. Just at present the operating head was taken as the difference between the upstream and downstream water levels. That was very debatable point and it was necessary to bring to bear a mathematics study from point to point. One had to be careful in developing the subjects from small to big scales and it was essential, therefore, to study the mathematics of the problem. It would help to get an idea of the whole situation and at the same time it would save valuable time of research workers in going into details which could be worked out from mathematical analysis. If the mathematician and the experimenter worked together, it would save considerable time. The President made personal appeal to the mathematicians to give the experimenters a helping hand. Quite a number of interesting remarks had been made with respect to the experiments made at different stations. He was sure the research workers had taken note of these and it would be of great advantage to the profession as well as to the country if these remarks were acted upon and the results presented in the next year's meeting. The question of spillways was very important in the design of new dams throughout the country so that it formed a part of the future development plans.

MR. CHATURVEDI in a written reply to Dr. Bhandari's remarks said that Dr. Bhandari's conclusion about the general idea was correct.

He agreed with Dr. Bhandari that the co-efficient of discharge of the weirs was not much affected by their downstreams slope. Downstream slope of Nagwa weir was changed from 2 : 3 to 4 : 5 in order to ensure better stability conditions and it was found from model studies that the co-efficient of discharge remained practically unchanged. Figures 6C.31, 6C.32, 6C.34 and 6C.35 gave details of the profiles A to D. It was evident from these drawings that the profiles A and B and C and D were similar except on the upstream side. It would be apparent from Figure 6C.30 that the shape of the upstream lip in profiles No. 7 to 11 were derived experimentally without conforming to any radius of curvature. Since the experiments were still in progress, Dr. Bhandari's suggestion about the graph showing the co-efficient of discharge plotted against the radii of curvation of the lip and portion of the crest downstream of the lip was welcome.

The extrapolation through three points could not be avoided as the scale variation was limited owing to very little discharge available at Lucknow Research Station. Further investigations were in hand in this connection at Bahadradabad.

DISCUSSION BY THE BOARD

THE SECRETARY described the discussion held at the Research Committee Meeting.

MR. CHATURVEDI said that certain experiments on the Ganesh Iyer's Volute Siphon were carried out in connection with the shape and design of outlet. They had not met with much success. He requested Mr. Narasimhaiya to advise.

MR. NARASIMHAIYA said that the research officers had got the details of the volute siphon as constructed at Hirebhasgar. This type was a modification of that adopted at Marconhalli and was based on model tests carried out at Krishnarajasagar. The model results were satisfactory, but on actual working it was found that the siphons did not function with one foot depth over the top of the siphon. It was designed for nine inches priming depth. A priming depth beyond one foot could not be attained as with depths higher than that the land in the Madras Presidency would come under submersion. Later on the level of the lip was reduced by about six inches so that there was a margin of about two feet and even then the siphons did not function. Afterwards the vertical portion was constructed. In that case the siphon started functioning at about one foot $5\frac{1}{2}$ inches priming depth. But the discharge was not quite full. For the reason that there was not a perfect vacuum. A note had been prepared on this and he trusted Mr. Ganesh Iyer would place it before the next meeting of the Research Committee.

THE PRESIDENT said that the report would be awaited with interest.

MR. NARASIMHAIYA said that the capacity of the sluices was more than ample for the surplusing. There was no difficulty in disposing off the floods.

MR. FRAMJI said that the modification made must have caused a reduction in the discharge.

MR. NARASIMHAIYA agreed that the discharge was very much less and one of the reasons was the constriction. The other reason was that there was a lot of leakage of air into the siphon at the joints. Even the pressures did not indicate full vacuum, but all that could not be perfected in the short time available between other floods.

THE PRESIDENT asked Mr. Narasimhaiya if the leakage was from the joint only.

MR. NARASIMHAIYA said that in the last experiment it was not from any where else. In the first instance the air was getting in from the air pipe. Even though the rear end was exposed the siphon action was going on. There was no sudden break in the siphonic action, but the discharges were becoming less.

MR. FRAMJI said that he found Rao Bahadur Venkatacharya's suggestion a good one. The suggestion was regarding the dissipation of energy in a high co-efficient weir by the use of friction blocks to a depth of $\frac{1}{2}$ the cistern as in

Figure 6C.42 This design was based on the design of the St. Antony Falls in America. These blocks were tied on to the cistern so that they could not get uprooted.

THE PRESIDENT asked Mr. Framji as to what the new point in the design was.

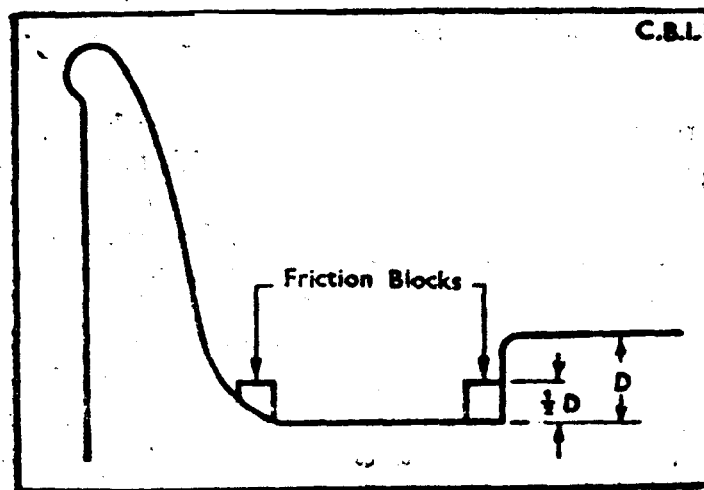


Figure 6C. 42

MR. FRAMJI said that the Madras was experimenting on the dissipation of energy in spillways. He was simply reporting the progress that had been made so that others who were doing similar work on the spillways may try this design too.

DR. BOSE liked to know the height of the St. Antony Falls on which these blocks had been tried. He said that these blocks did not stand the impact of the water for long and were washed away in some other falls. That is why buckets had been advocated.

MR. FRAMJI said that probably the height of the Antony Falls was 60 feet. The blocks had been functioning in the St. Antony Falls so far.

It was decided that the subject be kept on the agenda.

7C--Hydraulic Structures on Permeable Foundations

PRELIMINARY NOTE

This subject was on the agenda of the Research Committee from 1939 till the time of issue of the Board Publication No. 12 ; Design of Weirs on Permeable Foundations.

The design of pavements of regulators on clay and sand-clay foundations was also on the agenda of the Research Committee. This subject was removed from the agenda in 1941 with the following resolution passed by the Board :

“(a) Resolved that this subject be taken off the Agenda, but in order to complete the information published by the Board, the two notes by Rai Bahadur Khosla be published as a Supplement to Board publication No. 12 (with any modifications which the Author may consider necessary).

“(b) The Board is of the opinion that basic research on flow under dams, where the subsoil is not homogeneous, is not likely to be useful, in view of the diversity of formation which may possibly occur and the difficulty of reproducing these conditions in models.

“(c) In cases where effectively impervious strata of soil occur, *ad hoc* experiments might be carried out to determine the effect on sub-soil flow in each particular case.

“(d) In view of this, the subject “the design of pavements of regulator on clay and sand-clay foundation” should be removed from the Agenda.”

No action was taken on part (a) of this resolution. Central Board of Irrigation, Publication No. 12 is now to be revised, and it has been decided that all developments that have been achieved since its publication should be included in the revised edition. The Board decided in 1946 to re-introduce the subject this year on the agenda of the Research Committee.

Recent Literature.

1. Waugh, W. R.—Structural Behaviour of Fontana Dam revealed by dual purpose instruments,—Engineering News Record Volume 135, No. 20, November 15, 1945.
2. Taylor, Glover F. N.—Method of constructing deep land foundation—Institution of Civil Engineers, London 1945.
3. Ferguson, P. M.—Balancing Footing on clay—a Mirage ; Proceedings of 6th Texas Conference on Soil Mechanics and Foundation Engineering August 13, 14, 1943.
4. Legro, H. W.—Physical properties of earth materials—Structural foundation soils, *etc.*, 43rd Annual Conference Rly., Engineering Association vol. 42, 1941, and vol. 43, 1942.
5. Ramaswami, N.S.—Design of foundations from stand point of soil mechanic.—Ind. Con. Journal, October, 1946.
6. Leliavsky, S.—Percolation under Aprons of Irrigation work—Engineer dated April 12, 19 and 26, 1946.

7. Elsmny, E. A. —Heading up works on Pervious soils ; Fuad University, Giza, Egypt.

8. A theory for the design of control works on deep permeable foundations, Sudan Irrigation Department, Departmental Technical Note 1947.

The Year's Work

The following items were discussed at the 1947 Research Committee Meeting.

- (1) Pressures on Lloyd Barrage Floor.
- (2) The Poondi Regulator.

(1) PRESSURES ON LLOYD BARRAGE FLOOR (1)

A study of uplift pressures on weir floors built on permeable foundations is important from the point of view of stability of structures. With this purpose in view observation pipes upstream and downstream of each pile line on the Barrage floor have been fixed at pier No. 32 and observations made every fortnight and the data analysed to study the situation.

Figure 7C.1 gives a section of the floor along pier No. 32 with proper dimensions :—

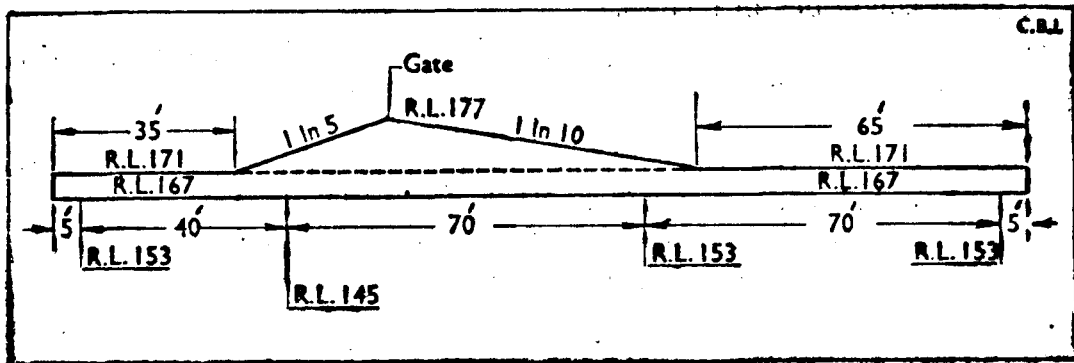


Figure 7C.1 : Section of the Lloyd Barrage floor along pier No. 32

The points numbered (1) to (8) show the position of pressure pipes.

For such a floor it is possible to find the theoretical pressures at points 1 to 8 according to Khosla's theory of pressures on permeable foundations, which is being used extensively ever since its origin. The theoretical pressures expressed as a percentage of working head have been worked out (2) and are given in Table 7C.1.

TABLE 7 C. 1

Pipe	% Pressure	Pipe	% Pressure
1	90.9	5	45.5
2	77.1	6	39.9
3	72.5	7	24.2
4	60.3	8	9.1

(1) Irrigation Research Division, Sind, Annual Report, 1946, pages 20 to 22.

(2) Central Board of Irrigation, Publication No. 12, page 128.

A study of the pressures as actually observed show that (1) as the water is headed up after the river starts falling the pressure in pipe No. 1 upstream of pile line 1 goes on decreasing as the duration of ponding increases; (2) when the pond is raised again after it was lowered during December canal closures the pressure in pipe No. 1 shows higher pressure than it was showing before lowering the pond; (3) till the difference between upstream and downstream water level at the Barrage is less than three feet the pressure observation in the pipes give erratic results; (4) the difference in pressures observed in pipes No. 3 and 4 *i.e.* in pipes upstream and downstream of the second pile line is more than theoretical; (5) the difference in pressures observed in pipes No. 7 and 8 *i.e.* in pipes upstream and downstream of last pile line is less than theoretical.

Item (1) can be explained as due to silt deposit occurring on the upstream thereby decreasing the pressures and item (2) as due to scouring out the silt deposited at the time of heading up by water flowing freely when the Barrage gates are lifted out of water during canal closure. Item (3) can be explained as due to inertia, the four inches diameter observation pipe may not respond to smaller head.

Item No. (4) can be explained as due to the penetration of pile No. 2 in less pervious strata, pressure reduction is more than theoretical.

In the past, to study the pressures, it was customary to correct the observed pressures for temperature and *pro rata* increase them in such a manner that the pressure in pipe No. 1 was made to correspond to the theoretical pressure of 90.9 per cent. This showed that not only the difference in pressure between pipes No. 7 and 8 was less than theoretical but the *pro rata* pressures were higher than theoretical.

This made people conclude that the last pile line was leaking and that the Barrage floor was unsafe.

In Central Board of Irrigation Publication No. 12 on page 45, it is said 'a deposit of silt is equivalent to an increase in the length of sheet pile'. An attempt was, therefore, made to find out what depth of sheet pile would be required in the position of pile No. 1 such that it would give the observed pressures in pipe line No. 2 just downstream of the first pipe line, and with this increased depth of piling, the pressures all along the floor were tested. This did not give any solution. Then it was proposed to consider an equivalent increase in length of floor upstream such that it would give the observed pressure with the same length of sheet pile No. 1 as that observed in pressure pipe No. 2. This gave somewhat better results compared to those observed in other pipes but yet there was a considerable drop downstream of line No. 2 as compared to the theoretical drop.

Calculations were then made assuming what length of increased pavement would be required to give the observed pressures downstream of pile No. 2 and with this increased length, the pressures upstream and downstream of the third and fourth sheet piles were calculated. These calculated pressures so arrived, more or less tally with the observed pressures.

The matter was then referred to the Secretary, Central Board of Irrigation to give his opinion in consultation with Rai Bahadur Khosla, if necessary, whether the method adopted was correct. As Rai Bahadur Khosla was busy,

the Secretary suggested that Dr. Bose may be consulted. Dr. Bose suggested that the actual difference between pressure pipes No. 4 and 8 should be made equivalent to 100, and then the pressures in all the pipes No. 4 to 8 increased *pro rata* and compared with the theoretical pressures. This method also gives a much better fit, than normally increasing the pressures *pro rata* to what it would be downstream of first pile line as was done originally.

Even by Dr. Bose's method, it can be said that the last pile line is not leaking and there is no danger to the Barrage pavement.

To correctly work out the pressures without the variations in water level upstream and downstream of the Barrage, special set of observations were taken three times a day for a week during the month of January 1947, and they have been analysed and given in Table 7 C.2, 7 C.3 and 7 C.4.

Table 7. C.2.—compares the theoretical pressures with those actually observed but increased *pro rata* such that the pressure in pipe line No. 1 corresponds to the theoretical pressure.

Table 7 C. 3 compares the theoretical pressures with those obtained by extending pavement upstream such that the pressure in downstream of pipe line No. 2 in pipe No. 4 corresponds to the one actually observed.

Table 7 C.4—compares the differences in pipes Nos. 5 and 6, 7 and 8, with the theoretical ones where the actual observed pressures have been increased in such a manner as if the difference in observed pressures in pipes 4 and 8 was 100.

TABLE 7 C.2

Percentage pressure in pipes on the basis of pro-rata increase to pressure in pipe No. 1 equal to theoretical

Date	Time	Head	Percentage pressure in pipes							
			1	2	3	4	5	6	7	8
1947										
January.										
29	8 a.m.	14.1	90.9	54.0	51.1	34.1	27.7	22.7	15.6	9.9
	12 noon	14.2	90.9	54.0	51.1	34.1	27.0	22.7	15.6	9.9
	4 p.m.	14.4	90.9	55.4	54.0	32.7	28.4	22.7	15.6	11.4
30	8 a.m.	14.6	90.9	56.8	54.0	34.1	28.4	22.7	17.9	11.4
	12 noon	14.6	90.9	56.3	53.4	33.2	27.4	23.1	15.9	11.4
	4 p.m.	14.8	90.9	56.3	51.9	31.7	27.4	21.6	14.4	10.1
31	8 a.m.	15.1	90.9	54.0	52.6	32.7	27.0	21.3	14.2	9.9
	12 noon	15.1	90.9	54.0	52.6	32.7	27.0	21.3	14.2	9.9
	4 p.m.	15.3	90.9	54.0	52.6	31.2	27.0	21.3	14.2	9.9
Februa-										
ry.										
1	8 a.m.	15.8	90.9	55.4	54.0	34.1	28.4	22.7	15.6	11.4
	12 noon	15.8	90.9	55.4	54.0	34.1	28.4	21.3	15.6	11.4
	4 p.m.	15.8	90.9	56.3	53.4	33.2	27.4	21.6	15.9	10.1
2	8 a.m.	15.8	90.9	56.3	53.4	33.2	27.4	21.6	15.9	10.1
	12 noon	15.8	90.9	56.3	53.4	33.4	27.4	21.6	14.4	8.7
	4 p.m.	15.8	90.9	54.0	52.6	32.7	27.0	22.7	15.6	11.4
3	8 a.m.	15.8	90.9	54.0	52.6	34.1	27.0	22.7	15.6	11.4
	12 noon	15.8	90.9	54.0	52.6	34.1	27.0	22.7	15.6	9.9
	4 p.m.	15.6	90.9	54.0	52.6	34.1	27.0	22.7	15.6	11.4
4	8 a.m.	16.2	90.9	55.4	52.6	34.1	28.4	22.7	15.6	11.4
	12 noon	16.2	90.9	54.5	53.1	35.0	28.0	22.4	15.4	11.2
	4 p.m.	16.2	90.9	55.4	54.0	34.1	28.4	22.7	15.6	11.4
The theoretical			90.9	77.1	72.5	60.3	45.5	39.9	24.2	9.1

TABLE 7 C.3

Percentage pressure calculation according to principle of extended pavement methods

Date	Head	%Pressure in pipes					
		1	2	3	4	5	6
January 1947—							
29	14.1	24	19	16	11	7	190
		24	20	18	12	7	1,094
30	14.1	24	20	16	12	8	190
		24	20	19	13	8	1,106
31	15.1	23	19	15	10	7	190
		23	20	18	12	7	1,105
February 1947—							
1	15.8	24	20	16	11	8	190
		24	20	19	13	8	1,150
2	15.8	23	19	15	11	7	190
		23	20	18	12	7	1,209
3	15.8	24	19	16	11	8	190
		24	20	19	13	8	1,106
4	16.2	24	20	16	11	8	190
		24	20	19	13	8	1,110

TABLE 7 C. 4

Percentage pressure calculated according to Dr. Bose's method

Date	Time	Water level		Head	pressure in pipes				
		Up-stream	Down-stream		1	2	3	4	5
January 1947									
29 ..	8 a. m.	192.8	178.7	14.1	100.0	72.5	52.5	23.1	0.0
30 ..	8 a.m. ..	192.7	178.1	14.6	100.0	76.4	50.7	23.7	0.0
31 ..	8 a. m. ..	192.7	177.6	15.1	100.0	73.1	54.1	22.3	0.0
February 1947--									
1 ..	8 a. m. ..	192.8	177.0	15.8	100.0	77.7	50.4	21.9	0.0
2 ..	8 a.m. ..	192.8	177.0	15.8	100.0	72.0	48.8	21.6	0.0
3 ..	8 a.m. ..	192.7	176.9	15.8	100.0	68.8	48.8	18.0	0.0
4 ..	8 a.m. ..	193.1	176.9	16.2	100.0	72.0	47.5	20.9	0.0
					100.0	71.1	60.1	29.3	0.0

(2) THE POONDI REGULATOR⁽³⁾

During December 1946 the regulator had to pass an unprecedented high flood and observations were made on the working of the regulator.

The dissipation of energy below this structure is effected by a baffle wall five feet high located 25 feet downstream of the toe of the glacis and three rows of staggered blocks near the end of the masonry apron 75 feet downstream of the baffle. As all the shutters were not uniformly opened a comparative study of the standing wave profile for a more or less constant rear water level and varying gate opening was made. For a shutter opening corresponding to the average of all vents put together, the standing wave had its commencement within the downstream third of the glacis. Starting from this line it rose steadily to a maximum height just over the baffle and then dived down to meet the rear water and form a secondary standing wave. When the shutter opening was less, the standing wave was located higher and was of a more regular type. At the close of the flood regulation one of the shutters could not be closed properly and had to be taken out for repair. This gave rise to an unusual condition *viz.*, maximum level upstream, practically no tail water downstream and the shutter fully open. In this state no standing wave was formed, but the shooting jet was bodily deflected by the baffle wall to form a parabola which dived down striking the apron in front of the staggered blocks. Knocking against these blocks, the jet rose high in large sprays and fell down on the loose apron in rear causing a scour crater 40 feet × 40 feet and having a maximum depth of about five feet. Except for this there were no harmful scours in rear of the aprons. The baffle and blocks functioned satisfactorily.

Whenever the shutter openings were irregular, there was considerable unbalanced flow accompanied by return eddies. In the stream downstream there were heavy waves directed towards the right margin and the violence of these waves was accentuated when ventway openings in the left half were greater than in the right half. Another undesirable effect of irregular shutter opening was that it gave rise to several vortices near the cut waters upstream. These vortices caused more disturbance in the case of pointed cutwaters than in the case of semi-circular cutwaters.

DISCUSSION BY THE RESEARCH COMMITTEE

MR. SETHNA in introducing item (1) explained the behaviour of pressure pipes on the barrage floors and the method of accounting for their departure from Khosla's method. The latter method tended one to conclude that the last pile line was leaking and the floor was unsafe from the point of exit gradient.

As it was stated that in the case of small structures the floor was three dimensional, and since there were a large number of *bund* sluices which had to be redesigned from considerations of pressures on floors, the President and Dr. Bose were requested to suggest some solution for this case, as it would effect a large economy in the remodelling of the pavements.

From a model of sluices, it appeared that in case of a structure which was narrow in comparison to its length, there was no pressure on the floor.

⁽³⁾ Irrigation Research Station, Poondi, Madras, Annual Report, 1946, pages 5 and 6.

Another point expressed was, where the water level was higher than the pavement, whether the sum of the cut off and water level above pavement should be taken for working out the pressures or the bigger of the two.

Also in the design of pavement wherein cistern was constructed, whether there was any benefit and whether any saving of thickness of pavement was affected.

He also enquired how calculations should be made when the sub-soil water level was below the pavement level and, therefore, the sub-soil was not being saturated.

MR. T. P. KUTTIAMMU introduced item (2).

MR. BLENCH said that comparable experiments had been done in the Punjab. He pointed out that a theory relating to two-dimensions and homogeneous porous material could not be applied to real conditions that were greatly different. His Institute was now trying to develop a three-dimensional electric analogue.

MR. M. P. MATHRANI said that there were cases where sub-soil water level on the side of the canal was higher than the full supply level. He knew this was an extra factor which should be considered when a subsoil water level was higher.

THE PRESIDENT said that he was sorry he had no time to look into the particular points mentioned by Mr. Sethna. He had examined a few other cases and had found that there was no leakage in the piles. Sometimes observations were wrong. In one or two cases on the Trimmu barrage, he got similar alarming reports but found that all the observations were completely wrong when he actually got to the site, he found it was all due to wrong measurements. He said that he would discuss the matter in detail with Mr. Sethna later in the day.

MR. M. P. MATHRANI said that it was doubtful whether it was worth while spending more money over this. His opinion was that the design should be for the full pressure, but considering the cost, he doubted whether it was worth while going in for it. In this case, he mentioned that there were hundreds of smaller barrages throughout Sind which required reconsideration.

DISCUSSION BY THE BOARD

THE SECRETARY said that there were only two items for discussion at the Research Committee and that there was no resolution for the consideration of the Board. He said that Mr. Sethna wanted to clear up certain points regarding the President's remarks made at the Research Committee Meeting.

MR. SETHNA referred to the President's remarks where he had said "He had examined a few other cases and found that there was no leakage in the piles". MR. Sethna thought that the President was referring to the data that MR. Sethna had supplied to him.

THE PRESIDENT regretted that he had had no time to look into that data.

DR. BOSE said that Mr. Sethna's case was not solitary. In another case also some difficulty had been experienced in explaining or interpreting the results of these pressure pipes. When DR. BOSE was in the Punjab it was thought that the pressure pipes were not indicating the actual condition. The trouble was that they tapped only one point and it was not possible to know what was happening underneath a structure by observation from a few points only. In view of this he liked to know whether it was worthwhile spending money for putting these pipes on a new barrage. Pressure pipes were going to be installed in the Mor Project, but he would like to know from the President whether it was worthwhile putting them.

THE PRESIDENT said that it was everytime worthwhile putting in the pressure pipes. He said he could quote a number of cases where the warning given by the pipes was right. In any case the cost involved was insignificant, but information obtained may be very useful. No doubt the pressure pipes did require some attention. Pressure pipes should be put in wherever possible.

SARDAR BAHADUR SARUP SINGH said that the information collected from the Islam and Ferozepore weirs used to be sent to the Director of Irrigation Research Punjab for examination. It was found after this procedure had been adopted that the predictions obtained were very useful. He added that unless silt observations were also made, *i.e.* the observation of the level of the downstream silt along with the pressure observations, the result were not of much use.

It was decided to keep the subject on the agenda.

8 C- Canal Falls

PRELIMINARY NOTE

This subject came on the agenda in 1931. A Board Publication was issued in 1935 which is now being revised. The progress made in this respect is rather slow for lack of staff in the Board's office.

At the last Board meeting it was decided to request the Chief Engineers Punjab to send in their final reports so that the Secretary could proceed with the proposed publication.

The contribution made at the 1946 Research Committee Meeting, it was felt, had advanced the Research on this subject and Research Officers were requested to forward further contributions as early as possible.

The following items were discussed at the 1946 Research Committee Meeting

(1) Basic research on different elements of straight and curved glacis type falls.

(2) Design of glacis profile in canal falls.

(3) Sarda type fall.

(4) The curvature of flow in the throat of a standing wave flume.

(5) Formula for weir discharge with free fall conditions.

(6) Laboratory experiments on models of existing falls at R. D. 217,000 Dipalpur canal, and R. D. 60, 510 Lower Jhelum Canal.

(7) Observations, at site, on actual falls at R. D. 217,000 Dipalpur Canal, and R. D. 184, 000 Mailsi Canal.

(8) Model experiments in connection with proposed addition and alteration to Tando Mastikhan Fall at R. D. 117,999 of Rohri Canal,

(9) Model investigation for preventing scour below falls and bridges in United Provinces.

(10) Investigation to evolve a design for canal drops utilising, counter-acting jets for dissipation of energy.

The Year's Work

The following items were discussed at the 1947 Research Committee Meeting :—

(1) Experiments on one span (part model) of siphon in connection with proposed additions and alterations to the existing Tando Mastikhan fall at R. D. 119, 117 of Rohri canal.

(2) Canal drops with counteracting jets for the Neivasal Tenpathi Channel.

(3) To find a device for controlling side erosion at foot bridge at mile 8-4-232, Marihan Branch—Mirzapur—canal.

(4) Rasul Hydel Scheme Falls.

(5) Fall at R. D. 129, 500 Pakpattan Canal.

(6) Design of canal falls.

(1) **EXPERIMENTS ON ONE SPAN (PART MODEL) OF SIPHON IN CONNECTION WITH PROPOSED ADDITIONS AND ALTERATIONS TO THE EXISTING TANDO MASTIKHAN FALL AT R. D. 119, 117 OF ROHRI CANAL⁽¹⁾**

ABSTRACT

The model experiments converting the existing Tando Mastikhan fall into a siphon, to serve as a control point in the transition stage during the lowering of the bed of Rohri Canal, were carried out during 1945⁽²⁾. The experiments showed that a minimum loss of 5.89 feet was necessary for the siphon to work under reasonable priming conditions, though a loss of 4.2 feet was obtained but with an inordinately high priming level. It was then decided to carry out further experiments with a part model of one span, which would be provided with a glass side to study the nature of flow and losses. This experiment was done during the year under report.

OBJECT

The object of the experiments was to find out the best profile and design of the siphon which would give least loss under reasonable priming conditions, making use of the existing structure (Tando Mastikhan Fall) to the maximum advantage possible.

MODEL

A geometrically similar part model 1/12th full size i.e., only one central span was reproduced as shown in Figure 8 C. 1. The discharge of the model was 2.125 cusecs corresponding to average discharge of 1050 cusecs on prototype through one span.

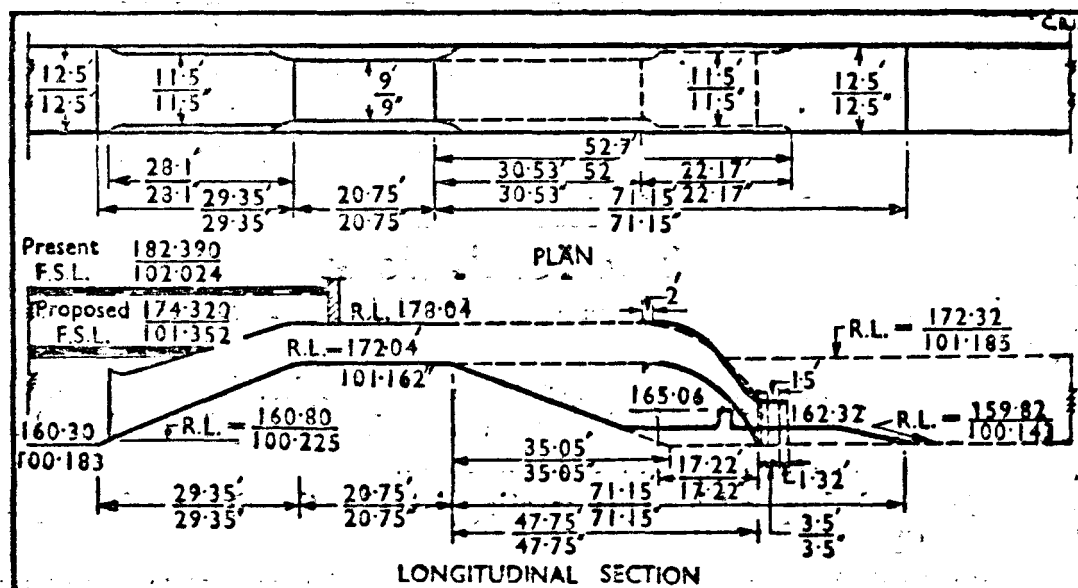


Figure 8C. 1: *Experiments in one span siphon model.*

(1) Irrigation Research Division, Sind, Annual Report, 1946, page 28 to 29.

(2) Irrigation Research Division Sind, Annual Report. 1945, pages 67 to 70.

Experiments.

The existing crest of the Tando Mastikhan Fall was extended in a length of 30.53 feet (proto) with a view to avoiding dismantling of the R. C. C. platform below the toe of the existing glacis. The length of the piers was also extended in the same length so as to keep a uniform width of throat over crest.

The profile of the downstream glacis and roof was so designed as to keep the barrel full and thereby obtain the lowest priming level without the use of a baffle, as the experiments carried out last year showed that though the baffle wall helped in early priming it added to the loss of head.

The first design tried out consisted of a gravity parabola of the form $X^2 = 23.9 Y$ (where X is the horizontal distance from the end of the crest and Y the ordinate of the profile) for the glacis and ogee curve for the roof.

The first experiment was without a baffle and the downstream water level at full supply level. The loss of head recorded was 4.7 feet. Table 8C. 1 gives the summary of all the experiments. It will be seen that this experiment gives the lowest loss. The initial priming level and the upstream water level were the same in all the six experiments for design I.

TABLE 8 C. 1

Experiments on one span (Central) siphon model
 $Q = 1059 = 2.123$ (gates fully open)
 Size of barrel at entry = 8 feet and at crest = 6 feet

Design	Expt. No.	Profile of		Water level up-stream	Water level down-stream	Loss of Head	General description and Remarks
		Glacis	Roof				
I	1	Gravity Parabola	Research (Max.) Ogee curve	177.12	172.42	4.7	Crest extended in a length of 30.53 feet so as to avoid dismantling in the original R.C.C. work, no baffle; downstream controlled to full supply depth an air lock was observed at the top of downstream glacis; initial priming level same as upstream water level.
	2			181.68	162.59		Same as above except the downstream left uncontrolled siphon did not prime at all.
	3			181.57	163.56		Expt. 2. repeated after providing a baffle two feet high at 155 feet from the toe of glacis. siphon did not prime at all.
	4			177.36	172.49	4.87	Expt. 1. Repeated after providing a baffle two feet high at 15.5 feet from the toe of glacis. Priming level same as upstream water level.
	5			177.23	163.87		Expt. 2. Repeated after providing a baffle five feet high at 15.5 feet from the toe of glacis.
	6			178.31	172.46	5.85	Expt. 1. repeated after providing baffle five feet high at 15.5 feet from the toe of glacis, priming level same as upstream water level.
II	1	Gravity Parabola	Gravity Parabola	183.16	163.27		No. baffle; downstream uncontrolled. Initial priming level was 183.48.
	2			183.97	172.28	11.69	Expt. II (1) repeated after controlling. Downstream loss of head was too high therefore unsuitable.

It was observed from the glass side that there was an air-lock downstream of the glacis just above the end of the crest. A baffle wall was therefore provided to see if it would improve matters.

Experiments 4 and 6 were therefore performed by providing a baffle wall of two feet to five feet height, respectively at a distance of 15.5 feet (prototype) from the toe of the glacis. The condition did not improve and the losses recorded were 4.87 feet and 5.85 feet respectively which were higher than those given by Experiment No. 1. The air-lock still persisted in the above experiments.

The presence of the air-lock in the above design led to design II, wherein gravity parabola was provided both in the downstream glacis and the roof, so as to follow the shape of the jet. This has been shown in Figure 8C.1. This design instead of improving matters gave a very high value of the loss of head, *vide* Item II 2, hence no further experiments were carried out with this design.

Conclusions

From the experiments performed this year, a further reduction of 1.19 feet in loss of head was affected against a loss of 5.89 feet (prototype) obtained last year in the full model (to smaller scale) *i.e.* the siphon could pass the full supply discharge with a loss of 4.7 feet by providing a gravity parabola for the downstream glacis and an ogee curve for the roof, after extending the crest and piers in a length of 30.53 feet (prototype).

(2) CANAL DROP WITH COUNTERACTING JETS FOR THE NEIVASAL TENPATHI CHANNEL ⁽³⁾

The design was approved for adoption in the reconstruction of the collapsed drop at R. D. O./1600 of the Neivasal Tenpathi Channel (Grand Anicut Canal System) and the drop was constructed accordingly. Immediately higher up in the channel at R. D. O./0600 there is another drop having the same discharge and fall and it was decided to compare the working of these two drops for a few years. The reports received from the local officers (Executive Engineers, Grand Anicut Canal Division) indicated that the drop had been working fairly satisfactorily. Observations made by them in 1945 showed that bed scour downstream of the new drop was roughly 16 per cent less than the bed scour downstream of the other. Observations made in 1946 indicated less scour and side erosion in rear of this drop than in rear of the old one. Flow in rear of the new drop got smooth within 50 feet downstream while in the case of the ordinary drop it was nearly 70 feet. Two rows of friction blocks were provided in rear of the old drop. The pipe conduits of the new drop were provided with gratings to prevent accidents to persons getting into the channel. Floating debris caught on these gratings had to be removed periodically to prevent their choking the pipes. Further observations will be made before drawing final conclusions.

(3) TO FIND A DEVICE FOR CONTROLLING SIDE EROSION AT FOOT BRIDGE AT MILE 8-4-232 MARIHAN BRANCH-MIRZAPUR CANALS ⁽⁴⁾

A 1.6 feet raised crest-type fall was inserted at this site sometimes back after the foot bridge had been constructed. Owing to the increased velocity of flow below the fall and the contracted section of the channel, both banks of the Marihan Branch were falling in a length of 100 feet.

⁽³⁾ Irrigation Research Station, Poondi, Madras Annual Report, 1946, page 6

⁽⁴⁾ The United Provinces, Public Works Department, Irrigation Research Station Annual Report of Research Progress 1946, Technical Memorandum No 17 pages, 102 and 103

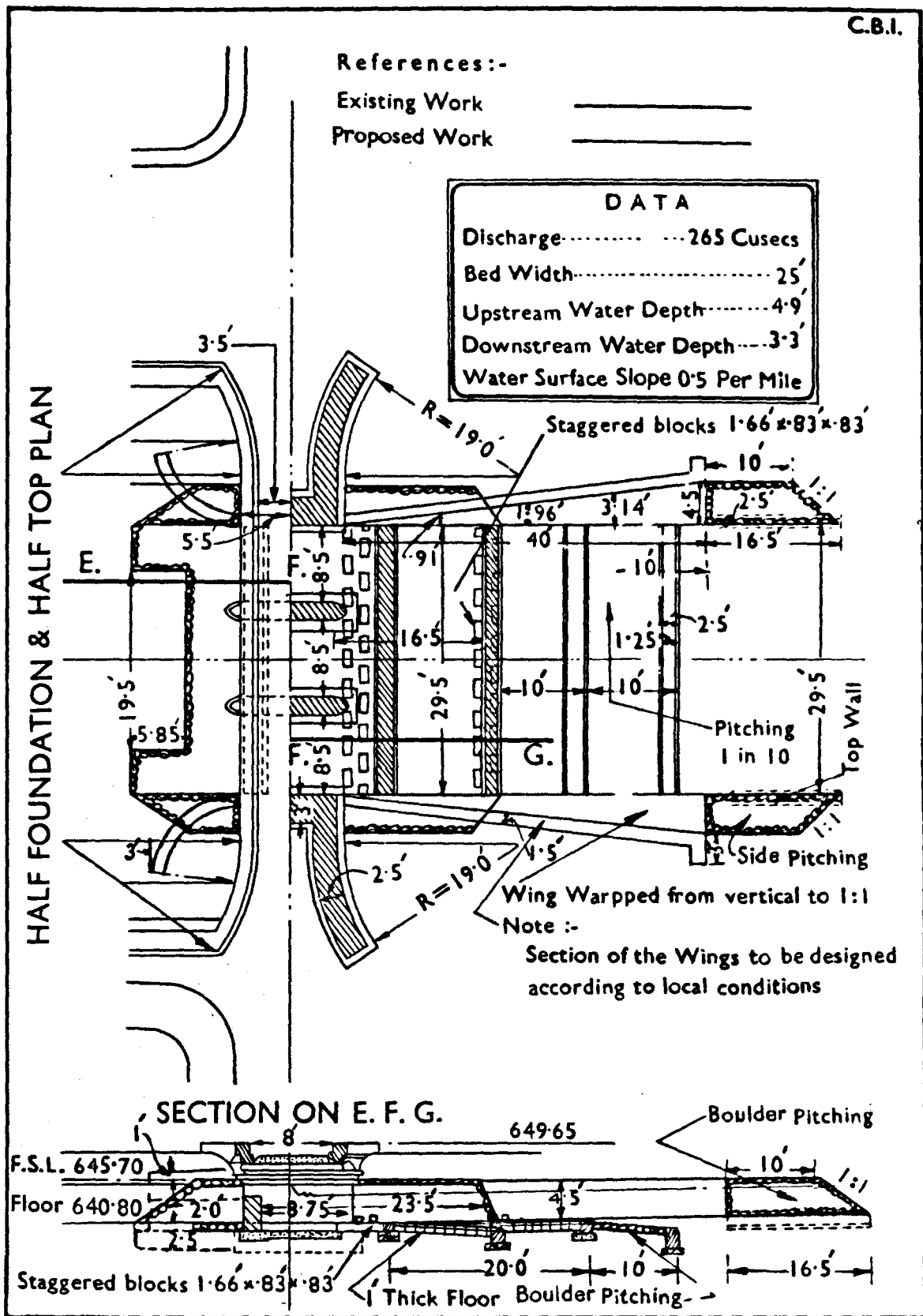
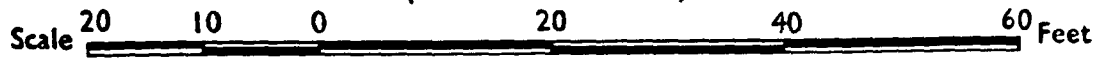


Figure 8 C.2:- Foot Bridge at M.8-4-238 Showing new proposal
 (Marihan Branch)



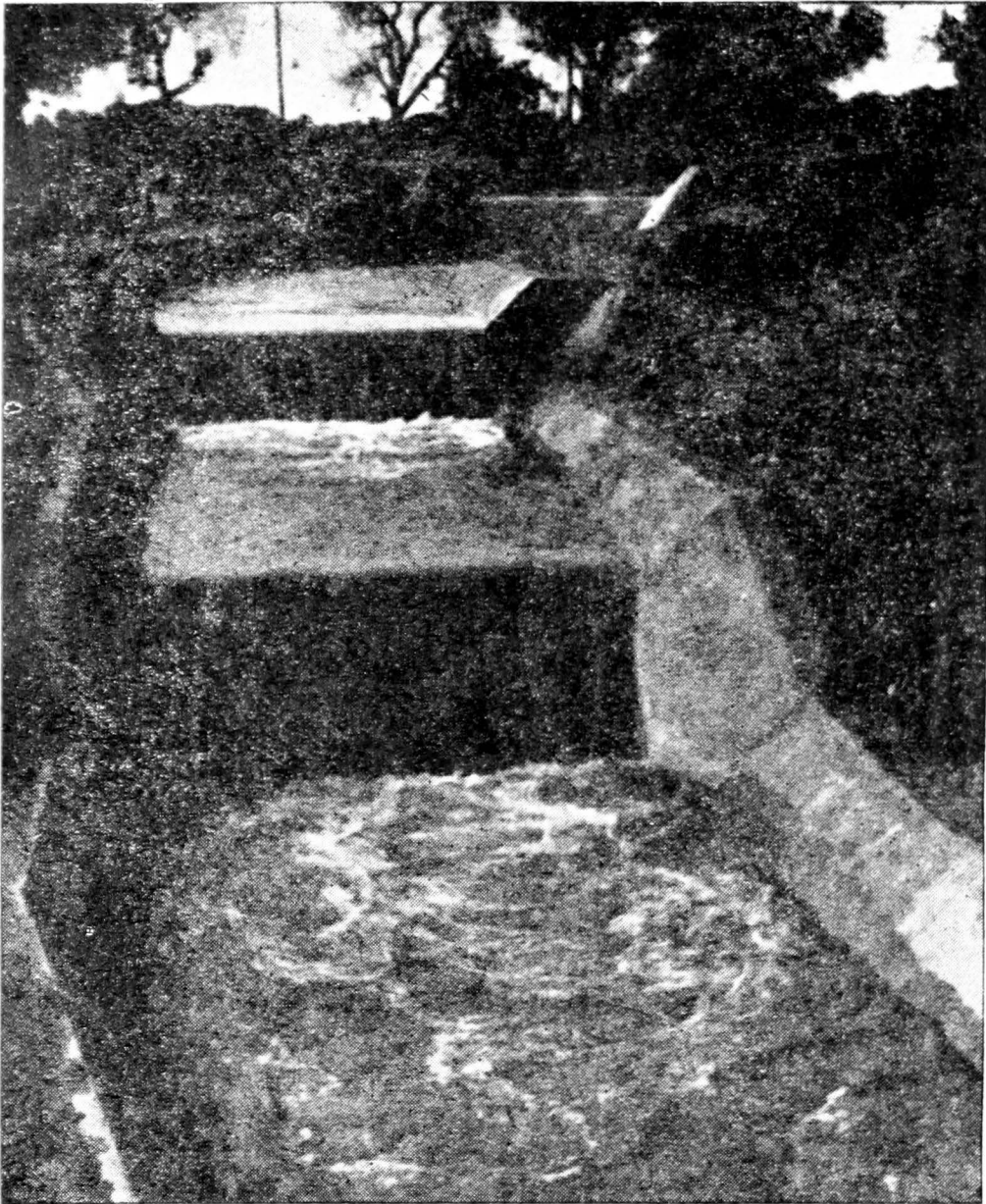


Figure 8C.3: *The vertical fall and the original parabolic fall in operation. Notice that flow conditions downstream of vertical fall are very satisfactory.*

A model of this bridge and fall was constructed to scale $1/8$ vide Figure 8C.2. It reproduced prototype conditions when run with a full supply discharge of 265 cusecs. The theoretical bed width was increased to 29.5 feet equal to the water way of the bridge including piers. Upstream approaches were streamlined by providing segmental wing of ten feet radius. Downstream wings were warped from vertical to 1 : 1 side slope of the channel in a length of 40 feet. The flow was now smoother, side erosion being reduced to 70 per cent. The existing floor was extended by 20.0 feet and bowed 1 in 60; ten feet pitching in 1 in 10 slope was added to it. This greatly relieved the action on the sides and with two rows of staggered blocks 1.66 feet \times 1.83 \times 0.83 feet at a distance of 8.75 feet and 23.5 feet respectively from the crest of the fall, the bed and side erosion were reduced almost to nil.

CONCLUSIONS

The following recommendations have been made for modification of prototype.

1. Upstream approaches should be provided with segmental wings of radius ten feet.
2. Downstream wings to be warped from vertical to 1 : 1 side slope in a length of 40 feet, 20.0 feet floor length should be provided downstream of existing floor and bowed 1 in 60 ; in ten feet pitching in slope 1 in 10, to be added down-stream.
3. Two rows of staggered blocks 1.66 feet \times 0.83 feet \times 0.83 feet to be provided at a distance of 8.75 feet and 23.5 feet respectively from the crest of the fall. (vide Figure 8C.2.)

(4) RASUL HYDEL SCHEME FALLS ⁽⁵⁾

An examination of a proposed design showed that it distorted the flow. A vertical fall as an alternative was also examined. Figure 8C-3 shows the two types of falls in operation. It will be seen that there is much less turbulence below the vertical fall than below the originally proposed type.

(5) FALL AT R. D. 129,500 PAKPATTAN CANAL ⁽⁵⁾

On a fall at R. D. 129,500 considerable bed scour and bank erosion existed. Model tests showed that it was due to uneven distribution of flow caused by piers constructed on the downstream floor for supporting the Village Road Bridge. Distribution was controlled by constructing piers on the crest. Bank erosion was reduced. Bed scour was further controlled by the introduction of three rows of staggered blocks.

(6) DESIGN OF CANAL FALLS ⁽⁶⁾

The summarized results are :—

Optimum depth and length of cistern flow

- (a) With a plain floor, the scourepth D_s continued to decrease with increase in floor length up to $L_f=12$, whereas with staggered blocks

⁽⁵⁾ Irrigation Research Institute, Punjab, Lahore, Annual Report 1946 page 6.

⁽⁶⁾ Irrigation Research Institute, Punjab, Lahore, Annual Report 1946, page 8.

in two rows no further decrease in scour occurred for L_f greater than $7D$. (D is channel depth, L_f is the length of the floor, D_s is scour below designed channel depth).

- (b) With a plain floor, side erosion decreased as L_f increased up to $L_f = 9D$ and became approximately constant for L_f greater than $9D$. With staggered blocks—however no further decrease in scour was experienced for values of L_f higher than $7D$. Keeping cistern length constant and varying cistern depth below the designed downstream channel bed, it was found that for all the fluming ratios cistern depths greater than $0.4D$, did not help in further reduction of scour and while with a plain floor the extent of side erosion was not affected by cistern depth; with staggered blocks it was more or less steady for values of D_f greater than $0.4D$.

As staggered block protection is usually provided it was concluded that from the point of view of bed scour and side erosion the optimum cistern depth is $0.4D$, always assuring a hydraulic jump to form.

(2) Relation between Scour Depth and Discharge Intervals

A sectional model of a hypothetical fall in different Lacey channels with discharges 300, 600, 1,000, 1,500, 2,000, 3,000 and 5,000 was tested on 1/12 scale. The following relations between scour depth D_s and discharge intensity q were determined.

$$D_s = 0.759 q^{0.67} \quad (\text{Without blocks})$$

$$D_s = 0.388 q^{0.67} \quad (\text{With blocks})$$

The relation is of the general form.

$$D_s = Kq^{2/3}$$

The eroded width denoted by W_s —the stable width of water surface at the section of maximum side erosion was related with q by the relations :

$$W_s = 2.47 q^{3.35} \quad (\text{Without blocks})$$

$$W_s = 2.52 q^{2.82} \quad (\text{With blocks})$$

(3) Relation between glacis slope and scour depth.

Glacis slopes of 0 : 1, 1 : 1, 2 : 1, 3 : 1, 4 : 1, 5 : 1 and 7 : 1 were investigated for a fall in which q , H_1 , D_f , L_f , D were kept constant. The important results were :

- (i) The change in glacis slope does not affect the scour depth when the floor length L_f is kept constant.
- (ii) The ratio of length of standing wave, to downstream depth, is constant at about five for all the tests.

The hydraulic jump was located at the toe of the glacis.

DISCUSSION BY THE RESEARCH COMMITTEE

MR. D'ABREO introduced item (1). He explained the necessity of the conversion of the existing Tando Mastikhan Fall into a siphon in the interim stage of lowering of the Kotri Canal from mile No. 2 to mile No. 24. He said that these experiments were an extension of the experiments carried out during

the previous year where a loss of head of 5.89 feet was obtained as against 4.7 feet in the present experiments, but the losses were still high as a loss of three feet was aimed at. Further experiments were in progress.

MR. KUTTIAMMU introduced item (2).

MR. CHATURVEDI introduced item (3) and said it represented a basic study for eliminating wave wash. In the United Provinces falls were built after detailed model experiments. In 1928, vertical drop falls gave the best results. It had now been confirmed that the selection was justified. Experiments had been carried out to find out the optimum depth of cistern so as to eliminate wave wash.

DR. UPPAL explained the work done at the Punjab Irrigation Research Institute. He said that they had carried out basic experiments on the length of cisterns and found that staggered blocks should be an essential feature of the design. His contention was that side erosion was quite different from wave wash. He said that there was no advantage in lengthening the floor beyond an optimum length. The dimensions $0.4D$ for depth of cistern and $6D$ for length worked out very satisfactorily where D is the downstream depth of water in the channel.

MR. BLENCH described some erroneous designs, and stated that his Institute was developing the idea of a "Spreading-floor" to fan the flow in the hypercritical stage, so that hydraulic jump would occur at about regime channel breadth and the fanning would continue even after the jump. The principle was merely that fanning would continue once it was started; it could not be induced clearly or readily by acting on only the sides of a jet. He had found the TVA hydraulic laboratories using the principle for convergence as well. There the problem was to persuade water to enter the shaft of a tunnel spillway, of some 40 feet diameter, and 300 feet drop, and refrain from pulsation that would cause it to foul the roof. The engineers' design, shown on a model was a failure; but the laboratory's design, providing a curved weir crest near the tunnel entry, compelled the flow to take the right course at entry, so that it continued in the right course thereafter.

MR. BLENCH said that falls would continue to be useful as meters, but would have to be calibrated. Only in the long-crested weir, properly designed, could the theoretical discharge table be relied on to a fine degree of accuracy.

DR. UPPAL said that the three experimental falls on the Montgomery—Pakpattan link were working nicely.

MR. FRAMJI said that instead of the flat crest, length design, exceeding 55 feet, (crest R.L. 172) provided in the Sind design he would suggest that the crest be curved (proposed crest level R. L. 176.0)—say to a radius of 24 feet—to give advantages of curved flow to the siphon in order to reduce the inordinately excessive priming depth which still persisted with the Sind design; the depth of entrance be made ten feet and depth at crown six feet. The outlet should be flared at 1 in 10 to improve the discharging capacity. The proposed siphon involved no modification to the existing structure upstream of the crest and only a small amount of reshaping of the existing downstream galls, if possible. The width

of siphon (excluding the partition walls) would be 100 feet. The total losses in upper arm, lower arm and in friction were estimated to be 3.89 feet, probably somewhat less in the prototype say 3.75 feet. The siphon would pass the designed discharge with upstream water level of 175.75 and was expected to prime in the prototype with a less than one foot rise over crest, or below R. L. 177.0. The position and height of the baffle on the downstream of the outlets would have to be tested in the model. The co-efficient of such a design was expected to be appreciably higher than 0.5 of the proposed design. This might also be tested in the model.

MR. H. P. MATHRANI said there was a difference of 1.7 between the two and that there was no objection in lowering the crest by 1.6. The two ideas could be combined.

MR. FRAMJI asked Dr. Uppal what were the depth and extent of scour before and after the blocks in the model at the Railway Bridge? Was the work done on the geometrically similar model with both rigid and mobile portions to a geometrical scale?

The height of blocks provided appeared to be excessive. They would create distortion of curvature and block up the waterway just where nature required greater waterways on the outside of the curve, causing distortion of flow and accentuating conditions downstream. In a similar case on the Tapti at Hope Bridge where the spans on the concave sides were blocked with loose stone with the idea of centralizing flow, conditions downstream worsened.

Referring to the Rasul Hydel Scheme Falls, MR. FRAMJI said that the results appeared to be judged merely visually by turbulence whereas bed scour should have been compared.

Regarding the Design of Canal falls MR. FRAMJI asked Dr. Uppal if the figures given for the reduced length of floor with blocks were obtained with retrograded conditions downstream; because a fall must not only be designed for the designed water level downstream but for water level after anticipated retrogression.

DR. UPPAL said that the Punjab practice was to allow for 25 per cent. retrogression in all cases.

MR. FRAMJI further asked Dr. Uppal if there was some type error in the formula given at the foot of page nine of the Punjab Report!

DR. UPPAL said that there was no typing error except that the words 'with blocks' and 'without blocks' given in the formula correlating W_s and q had to be interchanged.

RAO BAHADUR JOGLEKAR pointed out that if this was so, the formula was wrong; because for discharge of 5,000 cusecs up to which experiments were carried out, the surface width according to Lacey formula on which all the experiments were based works out to 190 feet; whereas according to the formula given by Dr. Uppal, the maximum width including embayment would only be 59 feet.

DR. UPPAL stated that the formula was applicable only upto 300 cusecs.

MR. FRAMJI said that for the discharge of 300 cusecs, the Lacey surface width works out to 47 feet; whereas the formula gave the width including bellying, only 15 feet.

MR. BLENCH said that there was an error which he would have corrected.

RAO BAHADUR JOGLEKAR referred to the item (2) and said that in this fall, gratings were provided "to prevent accidents to persons getting into the channels; floating debris caught on these grating had to be removed periodically to prevent their choking the pipes." This showed serious defects in the design: in case floating debris was not removed through negligence, flooding and breaches of the canal bank upstream may result. The design would be suitable only where there was no debris in the water.

MR. FRAMJI said that if the hydraulic jump was located at the toe of the glacis for the designed conditions in this type of fall, it had been observed that with any retrogression of levels, the jump moved downstream and then was unstable.

DR. UPPAL replied that it was an important point and he would give his best consideration to it.

RAO BAHADUR VENKATACHARYA replying to Rao Bahadur Joglekar stated that the canal referred to was only an artificial controlled canal and the grating was provided to catch anything that might flow down occasionally and not as a normal feature. He said that it was very suitable for these conditions and not for rivers and canals.

He further said that some sort of a device should be found out to reduce the wave wash in canals.

MR. CHATURVEDI said that vertical falls had worked satisfactorily as full width falls.

MR. FRAMJI said it was no good comparing falls of different fluming ratios. They must have the same fluming ratios and discharges in order that costs are compared for equal performance.

MR. BLENCH agreed with Mr. Framji.

THE SECRETARY said that the cost should be the over all criterion.

RAO BAHADUR VENKATACHARYA asked if staggered blocks had been tried on the slopes to prevent wave wash.

DR. UPPAL said that staggered blocks had proved successful on slopes too.

THE SECRETARY said that the old practice of projecting bricks from the side pitching served the same purpose.

DEWAN BAHADUR GOVINDARAJA AYYANGAR said that wave breakers had been used in tanks in Madras.

MR. COCHRANE referred to the wave breaking berms used on the Suez Canal.

DEWAN BAHADUR GOVINDARAJA AYYANGAR said that they had been tried on rivers also.

CONSIDERATION BY THE BOARD

The work done at the Research Committee Meeting was noted by the Board.

C—Distributary Heads

PRELIMINARY NOTE

This subject has been on the agenda since 1940, and has been considered from three aspects, viz., regulation, metering of supply and silt control.

At the 1945 meeting of the Board, "it was resolved that a Central Board of Irrigation Publication should be issued on the "Design of Distributary Heads" and with this end in view research officers be asked to contribute their drafts on the subject by May, 31 1946."

Punjab has sent in its contribution. Indian Waterways Experiment Station have suggested certain publications on the subject which may be included.

Replies from Sind, the United Provinces, Madras and Ceylon are still awaited. The rest of the Provinces and States have nothing to contribute.

The following items were discussed at the 1946 Research Committee Meeting :—

- (1) Basic experiments to investigate the effect on silt distribution of (a) angle of offtake and (b) varying discharges.
- (2) Model Experiments in connection with silting of Dhoro Naro Distributary.
- (3) Further Model Experiments on the head regulator on Reservoir Branch.
- (4) Basic experiments for determining co-efficient of radial gates.

Recent Literature

The location of Head Regulators in the Gash Delta,
Sudan Irrigation Department, Departmental Technical Note 1947.

The Year's Work

The following items were discussed at the 1947 Research Committee Meeting :—

- (1) Basic experiments to investigate the effect on silt distribution of (a) angle of offtake. (b) varying discharges.
- (2) To find a suitable device for eliminating bed scour below Ghaggar Canal Head.
- (3) Basic experiments for determining co-efficients for radial gates.
- (4) Model experiments in connection with scour below cross regulator over Sind Canal at Rustom.
- (5) Model experiments in connection with silting of Saidkhan Distributary ex. Hyderabad Branch.
- (6) Model experiments in connection with silting of Dalore and Patoi distributaries ex Jamrao Canal.
- (7) Model experiments in connection with silting of Jamrao Canal.

(1) BASIC EXPERIMENTS TO INVESTIGATE THE EFFECT ON SILT DISTRIBUTION OF (a) ANGLE OF OFFTAKE AND (b) VARYING DISCHARGES⁽¹⁾

INTRODUCTION

Some experiments had been carried out at the Hydrodynamic Research Station, Karachi in connection with proportionate silt distribution at the heads of offtakes, the results of which showed that the angles of offtake did not seem to control the entry of silt in the offtaking channel. As this conclusion was based on insufficient data, it was considered necessary to conduct basic experiments to study the problem in detail.

Experiments to study this problem were started in the year 1943 and two sets of experiments *i.e.*, 10 per cent. and 20 per cent. discharges in the offtaking channel and angles varying from 30° to 120° in 30° stages were completed during the year, the pivotal point about which the offtaking channel was made to rotate with respect to the parent channel being the intersection of the centre lines. In this method as the relative position of the cross regulator and the nose of the offtaking channel changed with the angle, it was felt the results would be defective.

In the year 1944, the experiments were reorientated, the pivotal point for rotating the offtaking channel being the downstream intersection point of the full supply lines of the two channels and (ii) a shorter length of thread (indicator) being used *i.e.*, three inches length instead of six inches used previously. In the year 1944 only one set of experiments with 20 per cent discharge in offtake was completed.

In the year 1945, in addition to the indicator method, the injection method with liquid was adopted⁽²⁾. Experiments with 10 per cent discharge in offtake were conducted in 1945. The conclusions drawn from the experiments are:—

- (a) The thread method does not show a consistent behaviour though in general, there is a tendency of increasing percentage of filament entry as one moves from bed to surface. In the liquid injection method the feature is definitely marked as the bed filament percentage is generally the least.
- (b) In respect of the variation of the percentage with the angle of offtake no consistent behaviour is obtained either with the thread or liquid method.
- (c) The increase of bed filament with increase in the percentage discharge of the offtaking channel compared to the total discharge entering the parent channel is approximately proportional to the increase in percentage discharge for all angles.
- (d) The average filaments entering the offtaking channel approximate to the discharge entering the offtaking channel with the liquid method while the thread method gives smaller values, thereby indicating superiority of liquid method.

It was decided to give up the thread method and use only the liquid injection method, which was followed in the experiments of 1946.

⁽¹⁾ Irrigation Research Division Sind, Annual Report, 1946, pages 23 and 24.

⁽²⁾ C. B. I. Publication No. 38. "Annual Report (Technical) 1946".

TABLE 9C.1
Hydraulic Data Model Scale Ratio 1/12th.

Serial No.	Description	Parent Channel															
		1st set		2nd set		3rd set		4th set		5th set		1st set					
		Upstream	Downstream	Downstream	Downstream	Downstream	Downstream	Downstream	Downstream	Downstream	Downstream	Downstream	Downstream	Downstream	Downstream	Downstream	
		Pro.	Mod.	Pro.	Mod.	Pro.	Mod.	Pro.	Mod.	Pro.	Mod.	Pro.	Mod.	Pro.	Mod.	Pro.	Mod.
1	Discharge in cusecs	1,000	2.00	900	1.80	800	1.60	700	1.40	600	1.20	500	1.00	100	.20	100	.20
2	Full supply level	105.50	100.458	105.40	100.450	105.40	100.450	105.40	100.450	105.40	100.450	105.40	100.450	105.50	100.458	105.40	100.450
3	Bed level	100.00	100.000	100.10	100.008	100.30	100.025	100.050	100.042	100.70	100.058	101.00	100.083	102.70	100.225	102.60	100.217
4	Full supply depth in feet	5.5	0.458	5.3	0.442	5.1	0.423	4.9	0.408	4.7	0.392	4.40	0.367	2.8	0.233	2.8	.233
5	Bed width in feet.	72.00	72.00	68.00	64.0	64.0	64.0	59.50	59.50	54.50	54.50	50.00	50.00	20.80	20.80	20.80	20.80
6	Sida slopes	$\frac{1}{2} : 1$	1	$\frac{1}{2} : 1$	1	$\frac{1}{2} : 1$	1	$\frac{1}{2} : 1$	1	$\frac{1}{2} : 1$	1	$\frac{1}{2} : 1$	1	$\frac{1}{2} : 1$	1	$\frac{1}{2} : 1$	1
7	Mean velo in Ft/Sec.	2.433	0.7022	2.403	0.6937	2.357	0.6805	2.305	0.6656	2.246	0.6483	2.177	0.6284	1.608	0.4643	1.608	0.4643
8	Pavement level	100.00	100.000	100.10	100.008	100.30	100.025	100.20	101.042	100.70	100.058	101.00	100.083	102.70	100.235	102.60	100.217
9	Crest level	100.58	100.048	100.95	100.079	101.04	100.087	101.15	100.096	101.49	100.123	102.91	100.243
10	Reynold's No.	31,340	..	29,500	..	27,630	..	25,660	..	23,240	10,280
11	Width of Meter flume in inches	28.68	..	27.00	..	25.19	..	22.88	..	21.11	9.31
12	Length of Meter flume in inches	11.94	..	11.50	..	11.02	..	10.6	..	9.9	6.3
13	Crest level in inches	100.052	..	100.069	..	100.083	..	100.037	..	100.120	100.240
14	Length of Glacis 1 in 5 in inches	2.64	..	2.52	..	2.46	..	2.34	..	2.22	1.38
15	Radius of curvature in inches	25.8	25.8	..	7.0	..	6.75	..	6.25	..	6.0	..	5.5	9.37	9.37	..	2.75

TABLE 9C.1—contd.

Serial No.	Description.	Offtaking Channels															
		2nd Set				3rd Set				4th Set				5th Set			
		Upstream		Downstream		Upstream		Downstream		Downstream		Downstream		Upstream		Downstream	
		Pro.	Mod.	Pro.	Mod.	Pro.	Mod.	Pro.	Mod.	Pro.	Mod.	Pro.	Mod.	Pro.	Mod.	Pro.	Mod.
1	Discharge in cusecs	200	0.40	200	0.40	300	0.60	300	0.60	400	0.80	400	0.80	500	1.00	500	1.00
2	Full supply level	105.50	100.458	105.40	100.450	105.50	100.458	105.40	100.450	105.50	100.458	105.50	100.450	105.50	100.458	105.40	100.456
3	Bed level	102.10	100.175	102.00	100.167	101.70	100.141	101.60	100.133	101.40	100.116	101.30	100.108	101.10	100.091	101.00	100.083
4	Full supply depth in feet	3.4	0.283	3.4	0.283	3.8	0.317	3.8	0.317	4.1	0.342	4.1	0.342	4.4	0.367	4.4	0.367
5	Bed width in feet	30.8	30.5	30.5	30.5	38.0	38.0	38.0	38.0	44.5	44.5	44.5	44.5	50.0	50.0	50.0	50.0
6	Side slopes	$\frac{1}{2} : 1$	$\frac{1}{2} : 1$	$\frac{1}{2} : 1$	$\frac{1}{2} : 1$	$\frac{1}{2} : 1$	$\frac{1}{2} : 1$	$\frac{1}{2} : 1$	$\frac{1}{2} : 1$	$\frac{1}{2} : 1$	$\frac{1}{2} : 1$	$\frac{1}{2} : 1$	$\frac{1}{2} : 1$	$\frac{1}{2} : 1$	$\frac{1}{2} : 1$	$\frac{1}{2} : 1$	$\frac{1}{2} : 1$
7	Mean velo. Ft/Sec.	1.827	0.5272	1.827	0.5272	1.978	0.5711	1.978	0.5711	2.095	0.6049	2.095	0.6049	2.177	0.6284	2.177	0.6284
8	Pavement level	102.10	100.175	102.00	100.167	101.70	100.141	101.60	100.133	101.40	100.116	101.30	100.108	101.10	100.091	101.00	100.83
9	Crest Level	102.37	100.198	102.12	100.177	101.72	100.143	101.242	100.118
10	Reynold's No.	14,600	17,240	20,720	23,240
11	Width of Meter flume in inches	12.43	15.78	18.72	21.11
12	Length of Meter flume in inches	7.65	8.56	9.24	9.9
13	Crest level in inches	100.195	100.165	100.142	100.120
14	Length of Glacis 1 in 5 in inches	1.68	1.92	2.04	2.22
15	Radius of curvature in inches	3.75	14.82	14.82	..	4.25	16.61	5.0	18.46	18.46	..	5.50

DISTRIBUTARY HEADS

OBJECT

As the full series of experiments as originally envisaged were not completed i.e., with the discharge varying in offtake from 10 per cent to 50 per cent in stages of 10 per cent. the experiments were continued this year.

MODEL

The geometrically similar model 1/12th the full size which had been used in the previous experiments was made use of with the necessary modifications for change from 20 per cent. to 30 per cent. Figure 9C.1 shows the general lay out of the experiment. The hydraulic data were same as shown in the previous experiments and have been shown in Table 9C.1.

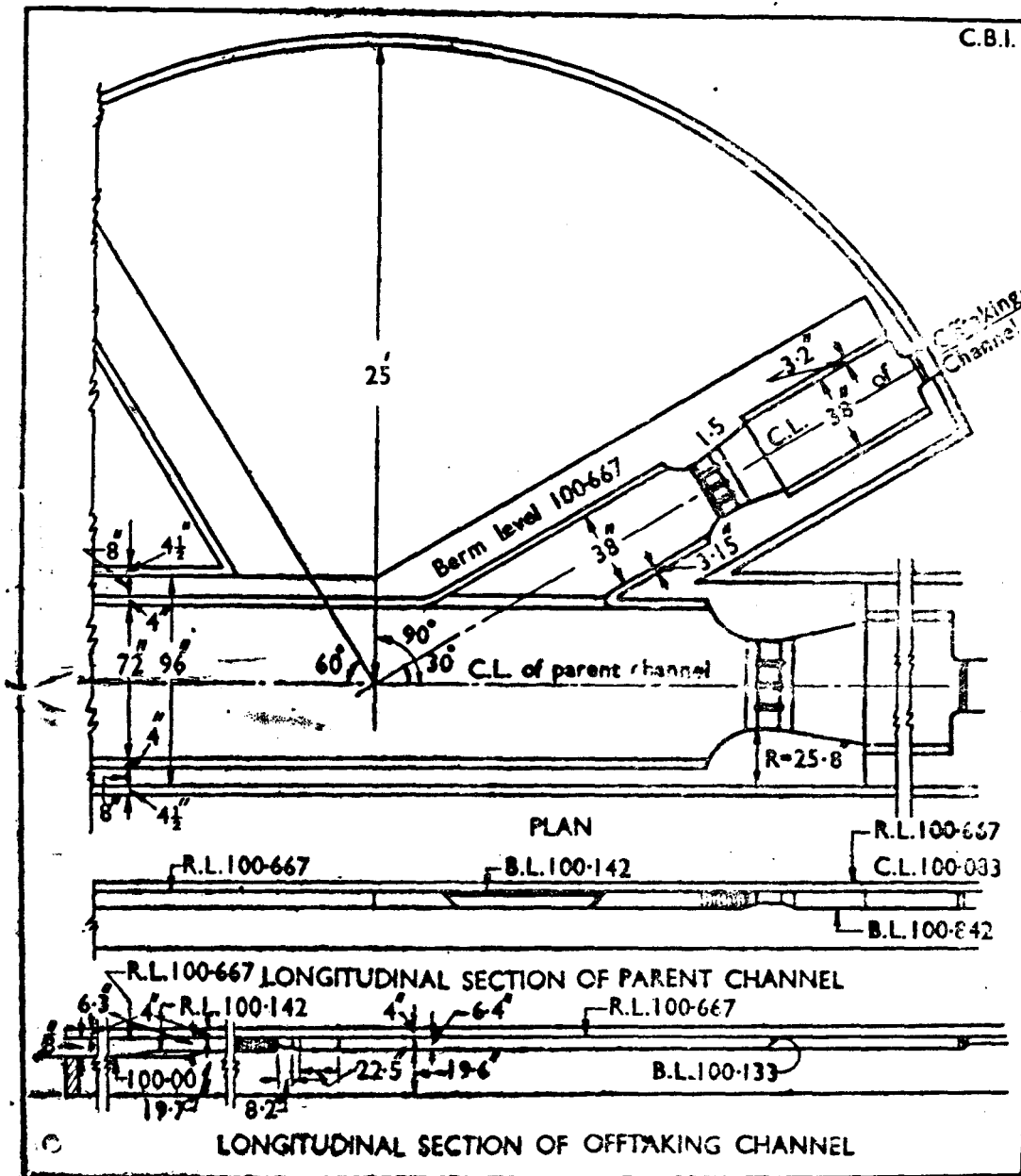


Figure 9 C.1:—Model or basis experiments to determine the effect on silt distribution of different angles of offtakes and varying discharges.

EXPERIMENTS

In all 12 experiments were carried out, three for each angle i.e., 30°, 60°, 90° and 120°. For each angle, full supply discharge $\frac{2}{3}$ discharge and $\frac{1}{3}$ full-supply discharge were admitted in turn into the parent channel, the upstream water level being the full supply level in the parent channel for all cases. The experiments were conducted with 30 per cent. of the full supply discharge of the parent channel in the offtaking channel, the remaining being passed down the cross regulator. The experiments were done with the liquid injection method. The filament distribution is shown in Table 9C-2 and Figure 9C-2.

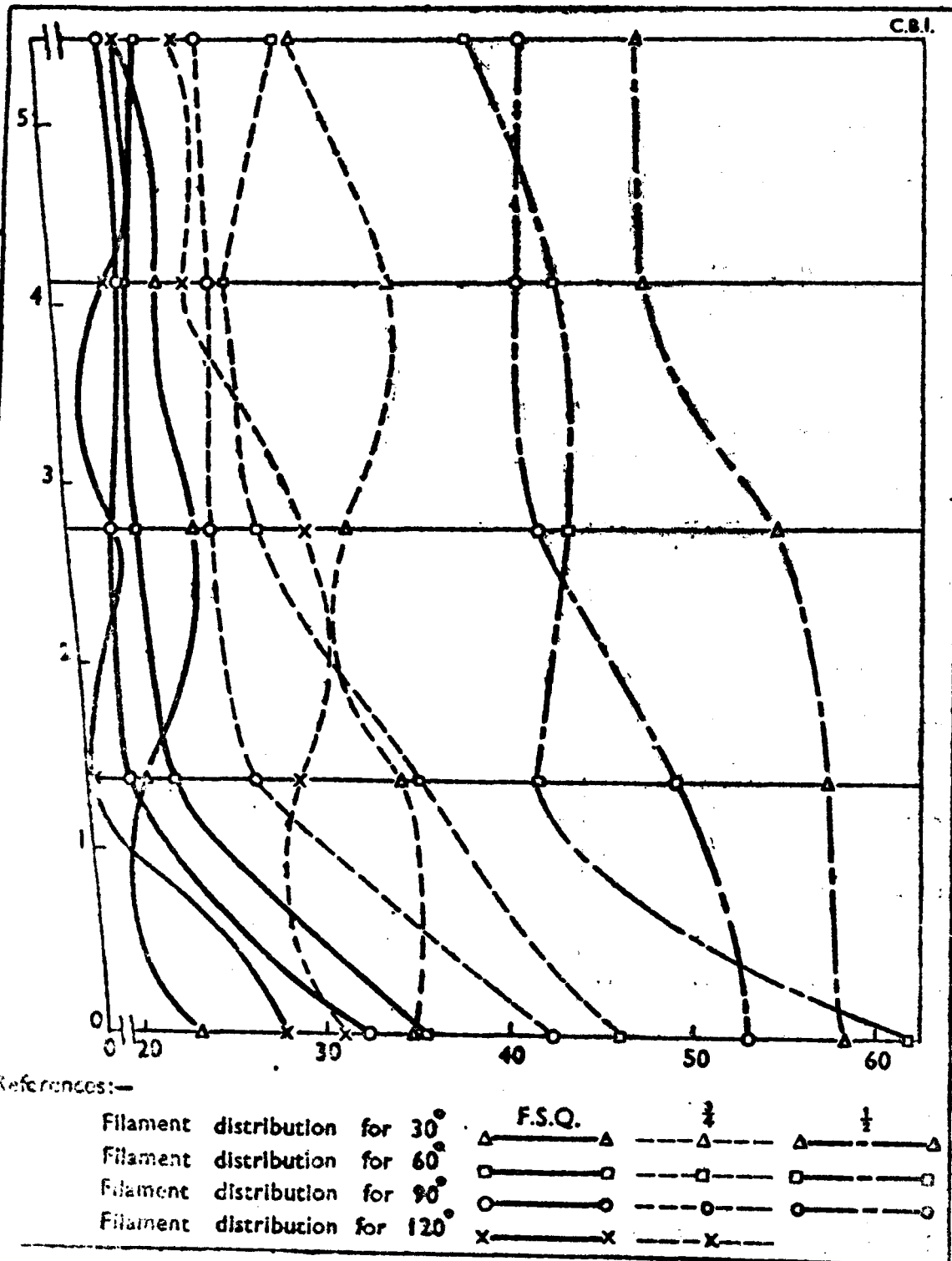


Figure 9 C. 2 : Filament distribution for basic experiments to determine the effect on silt distribution of different angles of offtake and varying discharges.

TABLE 9C. 2

Showing percentage filament distribution for 30 per cent. of total discharge entering offtake

By liquid method

Description				Percentage of total discharge entering offtake	Percentage of filaments entering offtake.			
					30°	60°	90°	120°
Bed	Filaments	Full supply discharge	31.9	49.3	44.4	38.9
$\frac{1}{4}$ Depth	(30)	28.2	30.2	26.8	24.2
$\frac{1}{2}$		31.9	27.9	25.3	25.9
$\frac{3}{4}$		29.0	27.0	26.4	25.0
Surface		26.1	27.4	24.8	26.1
Average					29.42	32.36	29.54	28.0
Bed	Filaments	$\frac{1}{4}$ Full supply discharge	48.5	64.6	59.9	51.4
$\frac{1}{4}$ Depth	(40)	47.3	48.6	36.4	32.3
$\frac{1}{2}$		43.4	36.6	33.3	34.0
$\frac{3}{4}$		46.1	34.2	32.9	24.5
Surface		38.4	37.7	31.9	37.30
Average					44.74	44.34	38.70	35.9
Bed	Filaments	$\frac{1}{4}$ full supply discharge	81.3	86.1	72.0	Not concluded
$\frac{1}{4}$ depth	(60)	79.3	57.5	67.7	
$\frac{1}{2}$		74.7	59.4	57.4	
$\frac{3}{4}$		64.5	57.9	55.2	
Surface		63.50	52.6	55.2	
Average					72.66	62.70	61.5	

CONCLUSIONS

1. The bed filaments for the 30 per cent. discharge show that the maximum bed filament entry is at 60°.

2. The bed filament entry increases with the increase in the percentage discharge in the offtake, but not in direct proportion in all cases.

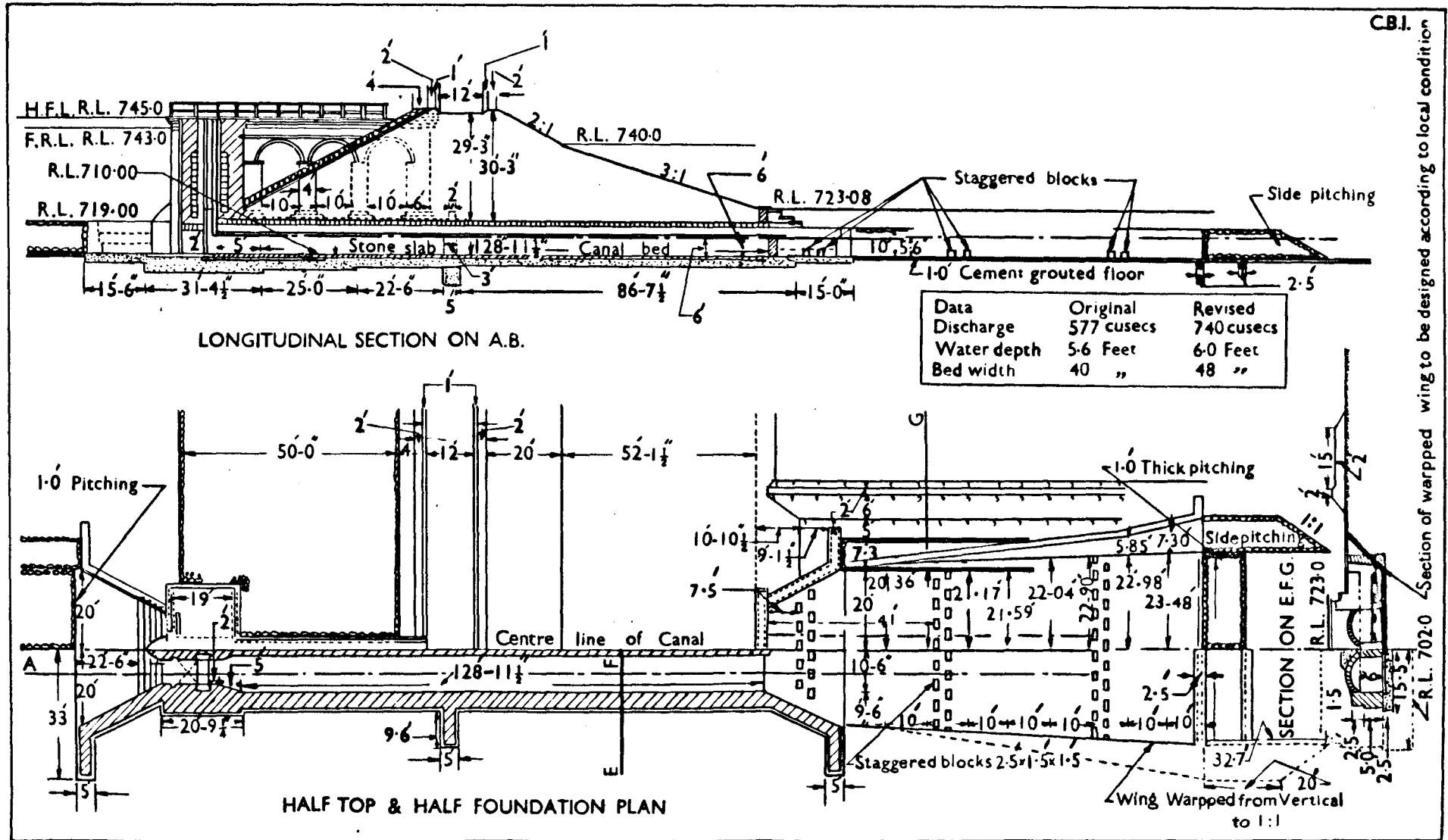


Figure 9C.3:- Ghagar Canal Head showing new proposals in the Downstream

3. The average of the filaments entering the offtake correspond to proportionate discharge in the offtake more or less.

4. The experiments are in progress and 40 per cent. and 50 per cent. of discharge in the offtake are yet to be tried, before final conclusions are reached.

(2) TO FIND A SUITABLE DEVICE FOR ELIMINATING BED SCOUR BELOW GHAGGAR CANAL HEAD⁽²⁾

The pitched floor below Ghaggar canal head is being repeatedly ripped off in a length of 35.0 feet as a result of which 3.0 feet deep bed scour is also developed. Subsequent filling of the scour with stones was of no avail, as it got washed off when the canal was run.

A model of this head was constructed to scale 1/12 *vide* Figure 9C.-3. When this was run with a full supply discharge of 577 cusecs, the model reproduced prototype conditions. It will be noted that owing to 33.0 feet head of water on the upstream side, the pitched floor could not stand the terrific velocity of the issuing stream. The model also indicated side erosion of the order of 8.0 feet on either bank. On inspection at site, it was found that side erosion did exist due to wave wash and the side pitching was eroded at places.

Since Nagwa Dam is under construction on Karamnasa river, the discharge of Ghaggar canal would increase to about 740 cusecs, the corresponding designed bed width being 48.0 feet. Accordingly the model was changed to the proposed dimensions. Hyperbolic expansion of different lengths were tried and 1 in 22.5 expansion gave minimum bed and side erosion. The model was tested with one, two and three rows of staggered blocks 2.5 feet × 1.5 feet × 1.5 feet. Three rows of blocks provided at a distance of 7.5 feet, 41.0 feet and 79.0 feet respectively from the edge of the existing floor reduced the bed scour to almost nil.

The floor was further bowed 1 in 80 and ten feet boulder pitching added to it, which made the side erosion almost negligible.

CONCLUSION

The following recommendations have been made for the modification of the prototype.

1. The downstream wings should be flumed with hyperbolic expansion 1 in 22.5 and 90.0 feet floor length to be bowed 1 in 80 with ten feet boulder pitching added to it.

2. Three rows of staggered blocks 2.5 feet × 1.5 feet × 1.5 feet to be provided at a distance of 7.5 feet, 41.0 feet, and 79.0 feet respectively from the edge of the existing floor where the tunnels end.

⁽²⁾ The United Provinces, Public Works Department, Irrigation Research Division Annual Report of Research Progress, 1946, Technical Memorandum No. 17, pages 101 and 102.

TABLE 9C. 3

Statement showing the value of Co-efficient in the formula $q=Cx\sqrt{2gH}$ in two span Radial Gates model of Nasrat Branch R.D. 99,160 under Standing Wave conditions (Nine feet Head Radial Gates).

Water Level up-stream	Model ratio 1/9				Model ratio 1/10				Model ratio 1/11				Remarks
	Gate opening x	Depth over Reg. Crest D	$\frac{H-D}{3x-4}$	C	Gate opening x	Depth over Reg. Crest D	$\frac{H-D}{3x-4}$	C	Gate opening x	Depth over Reg. Crest D	$\frac{H-D}{3x-4}$	C	
1	2	3	4	5	6	7	8	9	10	11	12	13	14
Full supply Level	1.0	0.908	0.825	0.87	1.0	0.825	0.750	0.87	1.0	0.748	0.680	*0.94	*value omitted due to experimental error
0.5 below full supply level	0.859	0.776	..	0.773	0.698	0.85	..	0.735	0.667	0.86	
1.0	0.803	0.720	..	0.721	0.646	0.87	..	0.664	0.596	0.85	
1.5	0.709	0.676	..	0.692	0.617	0.87	..	0.664	0.547	0.84	
2.0	0.693	0.610	..	0.618	0.543	0.89	..	0.575	0.507	0.87	
2.5	0.632	0.549	..	0.572	0.497	0.88	..	0.525	0.457	0.85	
3.0	0.582	0.499	0.84 0.864	..	0.521	0.446	0.80 0.866	0.480	0.412	0.88 0.858	
Full supply level	2.0	0.888	0.721	0.76 0.845	2.0	0.827	0.677	0.74 0.845	2.0	0.746	0.610	0.74 0.845	
0.5 below full supply level	0.859	0.692	0.76	..	0.765	0.615	0.73	..	0.703	0.577	0.74
1.0	0.804	0.637	0.76	..	0.727	0.577	0.71	..	0.645	0.509	0.74
1.5	0.752	0.585	0.75	..	0.672	0.522	0.73	..	0.600	0.464	0.75
2.0	0.689	0.522	0.78	..	0.637	0.487	0.73	..	0.569	0.433	0.74
2.5	0.628	0.461	0.78 0.765	..	0.577	0.427	0.75 0.732	..	0.513	0.377	0.75 0.743
				0.767				0.757				0.757	

TABLE 9C.3—(continued)

1				2	3	4	5	6	7	8	9	10	11	12	13	14
Full supply level	3.0	0.933	0.683	0.73	3.0	0.817	0.592	0.71	3.0	0.754	0.549	0.73	
0.5 below full supply level	"	0.855	0.615	0.73	"	0.766	0.541	0.69	"	0.705	0.500	0.74	
1.0 "	"	"	"	"	0.786	0.536	0.73	"	0.726	0.501	0.70	"	0.647	0.442	0.75	
1.5 "	"	"	"	"	0.754	0.504	0.73	"	0.677	0.452	0.70	"	0.604	0.399	0.75	
2.0 "	"	"	"	"	0.688	0.483	0.72 0.728	"	0.634	0.409	0.72 0.704	"	0.568	0.363	0.74 0.742	
Full supply level	4.0	0.918	0.585	0.68 0.718	4.0	0.847	0.547	0.69 0.718	4.0	0.748	0.475	0.69 0.718	
0.5 below full supply level	"	0.856	0.523	0.69	"	0.776	0.476	0.69	"	0.702	0.429	0.71	
1.0 "	"	"	"	"	0.807	0.474	0.70	"	0.717	0.417	0.69	"	0.669	0.396	0.71	
1.5 "	"	"	"	"	0.748	0.415	0.71	"	0.663	0.363	0.69	"	0.629	0.356	0.72	
2.0 "	"	"	"	"	0.712	0.379	0.70 0.696	"	0.637	0.337	0.70 0.692	"	0.573	0.300	0.72 0.710	
Full supply level	4.5	0.905	0.530	0.68 0.693	4.5	0.819	0.481	0.68 0.693	4.5	0.740	0.433	0.72 0.693	
0.5 below full supply level	"	0.854	0.479	0.71	"	0.756	0.418	0.68	"	0.733	0.426	0.71	
1.0 "	"	"	"	"	0.806	0.431	0.69	"	0.717	0.379	0.68	"	0.665	0.358	0.71	
1.5 "	"	"	"	"	0.750	0.375	0.72 0.700	"	0.676	0.338	0.69 0.683	"	0.634	0.327	0.72 0.715	
Full supply level	5.0	0.918	0.501	0.69 0.686	5.0	0.830	0.455	0.69 0.686	5.0	0.741	0.400	0.68 0.686	
0.5 below full supply level	"	0.856	0.439	0.69	"	0.776	0.401	0.69	"	0.714	0.373	0.72	
1.0 "	"	"	"	"	0.785	0.368	0.65 0.677	"	0.726	0.351	0.69 0.690	"	0.666	0.324	0.74 0.713	
Full supply level	5.2	0.918	0.485	0.70 0.676	5.2	0.827	0.390	0.73 0.676	5.2	0.745	0.390	0.67 0.676	
Full supply level	5.4	0.914	0.464	0.70 0.672	5.4	0.820	0.405	0.70 0.672	5.4	0.719	0.351	0.72 0.672	
Full supply level	5.6	0.922	0.455	0.72 0.668	5.6	0.832	0.420	0.69 0.668	5.6	0.734	0.352	0.73 0.668	

DISTRIBUTARY HEADS

(3) BASIC EXPERIMENTS FOR DETERMINING COEFFICIENTS FOR RADIAL GATES⁽⁴⁾

INTRODUCTION

The charts showing co-efficients for radial gates arrived at as the result of experiments carried out during the year 1932-33, were reported to give values of discharges which were different from the values actually observed on the field, and therefore needed verification. This was also confirmed by taking field observations during the year 1935-36. Briefly the defects from which the previous experiments seemed to have suffered were, (1) the results of one scale had been translated into prototype without the investigation of effect of scale, (2) the experiments did not take the effect of span and piers into account, and (3) the model was built with three spans, but in carrying out the experiments the two end spans were closed, and no change was introduced in the downstream expansion, though a change had been made on the upstream on account of not operating the two spans⁽⁵⁾.

Fresh experiments were therefore started in 1944, to arrive at the correct values of the co-efficients to save time for canal flow measurement and obviate the necessity of actual observation of discharges. The observations of the last two years showed that the value of the new co-efficients were higher than those obtained previously for standing wave conditions specially for the higher gate openings. The values of the coefficients for drowned conditions were also found to be higher. The experiments were continued during the year under report to collect further data so that final conclusions may be drawn.

OBJECT

The object of the experiments as already given in the ' Report of this Division for the year ending December 1945 ', is to find out the correct co-efficients for adoption.

EXPERIMENTS

Of the five regulators selected for investigation, four regulators to three different scales have been completed so far. The results of two regulators, namely (1) Fall Regulator over Nasrat Branch at R. D. 98,160 and (2) Cross Regulator over Warah Branch carried out during the year under report, are given in Tables 9C.-3 to 9C.-6 and are described below :—

Experiments on model of Fall Regulator over Nasrat Branch R. D. 98,160 (two span model).

⁽⁴⁾ Irrigation Research Division, Sind, Annual Report, 1946, pages 5 and 6.

⁽⁵⁾ Irrigation Research Division, Sind, Annual Report, 1944, Chapter IVa.

TABLE 9 C. 4

Statement showing the value of Coefficient in the formula $q=Cx\sqrt{2gH}$ in two span Radial Gates model of Nasrat Branch R.D. 99,160 under drowned conditions

Water Level upstream.	Model Ratio 1/9			Model Ratio 1/10			Model Ratio 1/11		
	Gate opening x	$H = D_1 - D_3$	C	Gate opening x	$H = D_1 - D_3$	C	Gate opening x	$H = D_1 - X_3$	C
Fall Supply Level	5.0	0.121	0.84	5.0	0.190	0.83	5.0	0.115	0.85
" ..	4.5	0.120	0.87	4.5	0.204	0.83	4.5	0.134	0.83
" ..	4.0	0.143	0.84	4.0	0.220	0.82	4.0	0.150	0.83
" ..	3.5	0.181	0.85	3.5	0.260	0.82	3.5	0.170	0.85
" ..	3.0	0.205	0.83	3.0	0.323	0.85	3.0	0.188	0.74
{ 1.0/below									
{ F. S. L. ..	4.0	0.118	0.81	4.0	0.188	0.86	4.0	0.105	0.83
" ..	3.5	0.151	0.86	3.5	0.213	0.83	3.5	0.095	0.88
" ..	3.0	0.222	0.83	3.0	0.260	0.83	3.0	0.117	0.84
" ..	2.5	0.290	0.84	2.5	0.342	0.84	2.5	0.141	0.86
{ 2.0/below									
{ F. S. L. ..	3.0	0.170	0.82	3.0	0.220	0.81	3.0	0.106	0.79
" ..	2.5	0.177	0.81	2.5	0.240	0.84	2.5	0.134	0.82
" ..	2.0	0.250	0.82	2.0	2.0	0.151	0.86
			0.835			0.833			0.831
			0.720			0.720			0.720

TABLE 9 C. 5

Statement showing the value of Co-efficient in the formula $q=Cx\sqrt{2g}H$ in 3 span Radial Gates model of Warah Br. R. D. 68,902 under Standing Waves conditions (9 feet Head Radial Gates)

Water level up-stream	Model Ratio 1/9				Model Ratio 1/10				Model Ratio 1/11				Remarks
	Gate opening x	Depth over Reg. crest D	$H=D-\frac{3x}{4}$	C	Gate opening x	Depth over Reg. crest D	$H=D-\frac{3x}{4}$	C	Gate opening x	Depth over Reg. crest D	$H=D-\frac{3x}{4}$	C	
1	2	3	4	5	6	7	8	9	10	11	12	13	14
Full supply level ..	1.0	0.921	0.838	0.73	1.0	0.841	0.766	0.72	1.0	0.863	0.795	*0.87	*Value omitted due to Experimental error
0.5 below Full supply level	..	0.873	0.790	0.75	..	0.795	0.720	0.73	..	0.825	0.757	0.83	
1.0 " "	..	0.828	0.745	0.75	..	0.740	0.665	0.74	..	0.771	0.703	0.77	
1.5 " "	..	0.779	0.696	0.79	..	0.692	0.617	0.74	..	0.723	0.655	0.79	
2.0 " "	..	0.723	0.640	0.76	..	0.645	0.570	0.75	..	0.682	0.614	0.79	
2.5 " "	..	0.648	0.565	0.75	..	0.595	0.520	0.77	..	0.628	0.560	0.78	
3.0 " "	..	0.593	0.510	0.76	..	0.545	0.470	0.77	..	0.592	0.524	0.76	
				0.756				0.746				0.787	
				0.845				0.845				0.845	
Full supply level ..	2.0	0.898	0.732	0.77	2.0	0.843	0.693	0.75	2.0	0.863	0.726	0.75	
0.5 below Full supply level	..	0.882	0.715	0.75	..	0.794	0.644	0.75	..	0.828	0.691	0.73	
1.0 " "	..	0.818	0.651	0.76	..	0.748	0.598	0.75	..	0.773	0.636	0.72	
1.5 " "	..	0.761	0.594	0.75	..	0.687	0.537	0.75	..	0.733	0.596	0.73	
2.0 " "	..	0.702	0.535	0.74	..	0.642	0.492	0.74	..	0.685	0.548	0.72	
2.5 " "	..	0.653	0.486	0.73	..	0.595	0.445	0.74	..	0.638	0.501	0.70	
				0.750				0.747				0.725	
				0.757				0.757				0.757	
Full supply level ..	3.0	0.924	0.674	0.75	3.0	0.845	0.620	0.71	3.0	0.858	0.653	0.72	
0.5 below Full supply level	..	0.878	0.628	0.74	..	0.795	0.570	0.72	..	0.815	0.610	0.71	
1.0 " "	..	0.819	0.569	0.73	..	0.744	0.519	0.73	..	0.763	0.558	0.72	
1.5 " "	..	0.753	0.503	0.73	..	0.695	0.470	0.73	..	0.730	0.525	0.72	
2.0 " "	..	0.703	0.453	0.74	..	0.645	0.420	0.73	..	0.682	0.477	0.71	
				0.734				0.724				0.716	
				0.718				0.718				0.718	

TABLE 9 C. 5—continued

1	2	3	4	5	6	7	8	9	10	11	12	13	14
Full supply level	4.0	0.919	0.586	0.74	4.0	0.841	0.541	0.71	4.0	0.863	0.590	0.73	
0.5 below Full supply level	"	0.862	0.529	0.75	"	0.795	0.495	0.72	"	0.818	0.545	0.72	
1.0 " "	"	0.815	0.482	0.75	"	0.748	0.448	0.72	"	0.769	0.496	0.72	
1.5 " "	"	0.760	0.427	0.75	"	0.698	0.393	0.72	"	0.733	0.460	0.72	
2.0 " "	"	0.729	0.396	0.74	"	0.642	0.342	0.73	"	0.688	0.415	0.72	
				<u>0.746</u>				<u>0.720</u>				<u>0.722</u>	
				<u>0.693</u>				<u>0.693</u>				<u>0.693</u>	
Full supply level	4.5	0.923	0.548	0.72	4.5	0.803	0.465	0.73	4.5	0.863	0.556	0.71	
0.5 below Full supply level	"	0.873	0.498	0.72	"	0.760	0.422	0.73	"	0.828	0.521	0.71	
1.0 " "	"	0.825	0.450	0.72	"	0.725	0.387	0.70	"	0.773	0.466	0.70	
1.5 " "	"	0.772	0.397	0.73	"	0.684	0.346	0.71	"	0.723	0.416	0.71	
				<u>0.723</u>				<u>0.718</u>				<u>0.708</u>	
				<u>0.686</u>				<u>0.686</u>				<u>0.686</u>	
Full supply level	5.0	0.917	0.500	0.73	5.0	0.825	0.450	0.70	5.0	0.863	0.522	0.70	
0.5 below Full supply level	"	0.876	0.459	0.73	"	0.785	0.410	0.71	"	0.818	0.477	0.71	
1.0 " "	"	0.823	0.406	0.74	"	0.745	0.370	0.72	"	0.775	0.434	0.72	
				<u>0.733</u>				<u>0.710</u>				<u>0.710</u>	
				<u>0.680</u>				<u>0.680</u>				<u>0.680</u>	
Full supply level	5.2	0.925	0.492	0.72	5.2	0.820	0.430	0.69	5.2	0.868	0.514	0.69	
				<u>0.676</u>				<u>0.676</u>				<u>0.676</u>	
Full supply level	5.4	0.923	0.473	0.72	5.4	0.819	0.414	0.69	5.4	0.863	0.495	0.69	
				<u>0.672</u>				<u>0.672</u>				<u>0.672</u>	
Full supply level	5.6	0.919	0.452	0.73	5.6	0.801	0.381	0.72	5.6	0.864	0.482	0.68	
				<u>0.668</u>				<u>0.668</u>				<u>0.668</u>	

DISTRIBUTARY HEADS

TABLE 9 C. 6

Statement showing the value of Co-efficient in the formula $q=Cx\sqrt{2gH}$ in three span Radial Gates model of Warah Branch R.D. 68,902 under drowned conditions

Water Level upstream	Model Ratio 1/9			Model Ratio 1/10			Model Ratio 1/11			Remarks
	Gate opening x	$H = D_1 - D_2$	C x	Gate opening x	$H = D_1 - D_2$	C x	Gate opening x	$H = D_1 - D_2$	C	
Full Supply Level	5.0	0.142	0.86	5.0	0.082	0.83	5.0	0.094	0.86	
" ..	4.5	0.161	0.81	4.5	0.090	0.82	4.5	0.112	0.87	
" ..	4.0	0.176	0.81	4.0	0.096	0.83	4.0	0.132	0.85	
" ..	3.5	0.203	*0.91	3.5	0.113	0.84	3.5	0.160	0.84	*Value omitted due to experimental error
" ..	3.0	0.214	0.83	3.0	0.139	0.82	3.0	0.181	0.85	
{ 1.0' below Full Supply Level	4.0	0.126	0.83	4.0	0.048	0.83	4.0	0.066	0.85	
" ..	3.5	0.119	0.83	3.5	0.064	0.80	3.5	0.085	0.84	
" ..	3.0	0.143	0.83	3.0	0.078	0.83	3.0	0.104	0.86	
" ..	2.5	0.187	0.84	2.5	0.104	0.85	2.5	0.140	0.86	
{ 2.0' below Full Supply Level	3.0	0.092	0.82	3.0	0.057	0.84	3.0	0.067	0.94	
" ..	2.5	0.096	0.84	2.5	0.079	0.81	2.5	0.094	0.88	
" ..	2.0	0.130	0.83	2.0	0.096	0.83	2.0	0.122	0.90	
			0.830 0.720			0.828 0.720			0.867 0.720	

STANDING WAVE CONDITIONS

Observations were taken on three models of scales 1/9, 1/10 and 1/11. Figure 9 C. 4 shows the details of the model 1/9 scale with the pertinent hydraulic data. The results show a higher value of the coefficient for nearly all range of gate openings, *vide* Table 9 C. 3. The figures in *italics* show the values of coefficient in the original experiments. This confirms the results of the experiments carried out last year.

DROWNED CONDITIONS

Observations for drowned conditions were taken on all the three models. The results obtained gave higher coefficients than those obtained in the experiments of 1932-33, and confirm the results of the experiments performed in 1944 and 1945. The results are given in Table 9 C. 4.

Experiments on model of Cross Regulator over Warah Branch R. D. 68,902 (three span model)

STANDING WAVE CONDITIONS

Figure 9 C. 5 shows the model of the regulator constructed to a scale of 1/9. Observations were seen on all the three models to 1/9, 1/10 and 1/11 scales. The value of the coefficient obtained was higher for the larger gate openings *vide* Table 9 C. 5. This confirms the results on the two span model in respect of higher gate openings.

DROWNED CONDITION

Observations were taken on the three above mentioned models, *vide* Table 9 C. 6. The value of the coefficient is higher than the one obtained previously in 1932-33 and confirms the results obtained during the last two years.

CONCLUSIONS

The experiments are in progress and one set remains to be completed before final conclusions are drawn in respect of coefficients to be adopted, the effect of scale, piers and spans.

(4) MODEL EXPERIMENTS IN CONNECTION WITH SCOUR BELOW CROSS REGULATOR OVER SIND CANAL AT RUSTOM (6)**INTRODUCTION**

The Rustom cross regulator is at mile 5/4 of Sind Canal. It is an open type regulator, without a raised crest and the regulation is done by needles. It was constructed in the year 1938.

On the downstream side below the pitching a belly forms with the result that the bank erodes and the groyne on both sides collapse, and have to be repaired annually. The reverse flow is considerable on the left side but slight on the right side.

In the year 1945 cement concrete blocks were put in below the end of the downstream pitching in the bed.

OBJECT

Model experiments were desired to be carried out with the object of stopping the side and bed scour and bringing about the silting up of the existing scour hole.

MODEL

A geometrically similar model 1/17th the full size was constructed with hydraulic data as per Table 9 C. 7. The regulator and the channel were reproduced from details supplied by the Shikarpur Canals Division.

TABLE 9 C. 7*Hydraulic data for Cross Regulator at Rustom model scale ratio 1/17*

Serial No.	Particulars	Rustom				Regulator			
		Designed				Working			
		Upstream		Downstream		Upstream		Downstream	
		Proto	Model	Proto	Model	Proto	Model	Proto	Model
1	F.S. Discharge in cusecs	1,907	..	1,907	..	2,381	2.0	2,381	2.0
2	F.S. Level ..	203.3	101.063	202.3	101.004	203.8	101.092	203.0	101.045
3	Bed level ..	192.1	100.404	191.1	100.345	189.9	100.275	189.9	100.275
4	F. S. Depth in feet ..	11.2	0.659	11.2	0.659	13.9	0.818	13.1	0.771
5	Bed width in feet ..	44	2.41	44	2.41	62	3.65	62	3.65
6	Hydraulic gradient	(1	in.	7,500)	(1	in	7,500)	..
7	Side Slopes	1/2 : 1	..	1 : 1
8	Crest level	191.5 /	100.369	191.5 /	100.369

(6) Irrigation Research Division, Sind, Annual Report 1946, pages 36 to 38.

EXPERIMENTS

The principal experiments have been described below. The velocity distributions referred to in the experiments are bed velocity distributions and are shown in Table 9 C. 8.

TABLE 9 C. 8

Statement showing velocity distribution below Cross Regulator over Ruston

Expt. No.	Particulars	Left			Right				Velocity	
		13"	12"	6"	0	6"	12"	19"	Max.	Mean
1	Original conditions as existing at site with Full Supply Discharge = 2.0 cusecs baffle at 0.71"/12" at the end of the <i>pucca</i> pavement. Distribution taken at cross section 25 feet away from downstream end.	1.65	3.06	3.20	3.10	2.27	2.47	2.27	3.20	2.57
3	Same as Experiment 1 except that baffle placed at 1.71/28".	0	2.16	3.05	2.56	2.06	1.92	2.64	3.05	2.18
4	Same as experiment No. 1. except that an additional baffle 0.71"/12" high placed at the end of downstream end the original baffle wall also reduced to 0.71"/12".	0	1.65	2.68	2.64	1.05	2.56	2.93	2.93	3.06
8	Same as in experiment 4 Except the height of both baffles was kept 1"/7" instead of 0.71"/12" two baffles. One at the end of <i>pucca</i> pavement second at downstream end were provided.	0	2.24	2.64	3.39	3.61	2.97	2.31	3.61	2.45
9	Same as experiment 8 except line of observation shifted 32 feet away from downstream or 7 feet from 1st cross section.	0	2.10	2.56	2.10	2.68	2.51	1.80	2.68	1.95
10	Same as experiment 4 except the line of observation has been changed as for experiment 9.	0	1.57	..	2.27	2.31	2.31	1.73	2.31	1.70
11	Same as experiment 4 also groynes were introduced to get regular section.	1.73	2.64	2.31	1.54	2.31	2.16	1.92	2.64	2.09
14	Same as Experiment 4 except downstream Water Level was raised to get a cutoff of 5.8' 0.409'	0.66	2.56	2.60	1.32	1.73	2.04	1.98	2.60	1.84
	Water Level upstream 202.5									
	downstream 196.76 101.016									
	100.675									
20	The existing deflector removed and the curtain wall raised by 9 inches in a width of one foot six inches to act as deflector with cut-off of 5.5' and discharge 290 cusecs.	1.85	1.32	0.66	0.66	0.66	0.66	1.32	1.85	1.02
21	Same as experiment 20 with a cut-off 5.5 feet and discharge 250	1.56	1.32	0.66	0.66	1.56	1.85	1.85	1.85	1.35

Experiment No. 1

The model was run with full supply discharge of 2381 cusecs, (prototype) corresponding to a model discharge of two cusecs, under original conditions with the one foot high baffle provided at the end of the *pucca* pavement. The velocity distribution was taken 25 feet (prototype) below the end of the downstream pitching. The maximum and minimum (prototype) bed velocities were 3.2 feet per second and 2.57 feet per second respectively.

Experiment No. 3

The baffle was raised to 1.31 feet and the velocities observed. The maximum and mean velocities were 3.05 feet per second and 2.28 feet per second respectively.

Experiment No. 4.

The model was then run with the baffle wall one foot high just at the downstream end of the ease waters in addition to the existing 1.31 feet high baffle wall at the end of *pucca* pavement, and the distribution taken, the maximum and minimum velocities being 2.93 feet per second and 2.06 feet per second respectively.

Experiment No. 8

Both baffles were then raised to 17 inches height and the distribution taken. The maximum and mean velocities were found to be 3.6 feet per second and 2.45 feet per second respectively.

Experiment No. 9

With the conditions as in experiment No. 8, the velocity distribution was taken 32 feet away (prototype) from the end of the pitching. The maximum and mean velocities were 2.68 feet per second and 1.95 feet per second respectively.

Experiment No. 10

Under conditions as per experiment No. 4, the velocity distribution was taken at the cross section as per experiment No. 9. The maximum and mean velocities were 2.31 feet per second and 1.7 feet per second respectively.

Experiment No. 11

Groynes were then introduced in the bellies and the model run as for experiment No. 4. There was no reverse flow and the velocity distribution gave a maximum and mean velocities of 2.64 feet per second and 2.09 feet per second respectively.

Experiment No. 14

From theoretical considerations it was seen that the regulator was safe for a maximum cut off of 5.5 feet only.

The model was run with a cut off of 5.8 feet (prototype) with a discharge of 250 cusecs as actually obtained in the field, the other conditions being the same as for experiment No. 4. The maximum and mean velocities were 2.60 feet per second and 1.84 feet per second respectively.

Experiments No. 20 and 21

In these experiments both the baffles were removed and the end curtain wall raised by nine inches to serve as a deflector, the groynes being retained and the model run with a discharge of 290 cusecs and a cut off of 5.5 feet. In this experiment the silted bed below the scour hole was also cleared.

In experiment No. 21 a discharge of 250 cusecs was run with a cut off of 5.5 feet other conditions being as per experiment No. 20. The velocity distributions for these are shown in Table 9 C. 8. Though the velocities are low the distribution is not good.

It was suggested that the upstream approach channel be eased, and the downstream channel splayed 1 in 10 from the downstream and of the pitching to meet the normal channel, the junction points being rounded off, a nine inches deflector at the end of the pavement be provided and groynes be provided in these reaches as found necessary to bring channel to proper section.

RECOMMENDATION

As a result of the experiments the following recommendations were made.

Upstream side

The channel upstream to be eased and groynes provided, the spacing of the groynes being three times the average length. The earth obtained from the cutting on the right side to be used in filling the scour hole and in between the groynes on the downstream.

Downstream side

The channel to be splayed from the exit to 1 in 10 to join the channel further downstream and the junction points rounded off as shown ; groynes to be spaced $1\frac{1}{2}$ times the average length.

A deflector at the end of the pavement to be provided.

(5) MODEL EXPERIMENTS IN CONNECTION WITH SILTING OF SAIDKHAN DISTRIBUTARY EX. HYDERABAD BRANCH(?)

INTRODUCTION

Saidkhan distributary takes off from Hyderabad Branch Ex. Rohri Canal at Mile 11. It has been giving silting trouble since long. A silt survey performed at the head of the distributary in the *abkalani* * of 1944 showed that it was carrying a large excess of harmful silt, *vide* column 11 of Table 9 C. 9, which appears to be responsible in a great measure for the silting trouble experienced. The hydraulic gradient is also deficient being 1 in 8700 for a discharge of 86 cusecs. The distribution of the silt charge and grade across the channel along three verticals upstream of the head regulator over Saidkhan distributary is shown in Figure 9 C. 6. It will be seen the coarse and heavier silt charge is in the lower levels.

* Irrigation Research Division, Sind, Annual Report, 1946, pages 25 to 27.

* Flood season.

TABLE 9 C. 9

Showing the results of analysis of silt data in connection with the silt surveys of Saidkhan Distributary 1944

Serial No.	Channel	Discharge in cusecs observed	Total silt charge in Gms/Litre	Percentage of fine silt charge i.e. upto 0.075 mm diameter	Percentage of harmful silt charge i.e. above 0.075 mm diameter	Harmful silt charge in Gms/Litre	Total silt load in lb/sec. passing the X-Section	Harmful silt load in lb/Sec. passing the X-Section	Percentage of discharge as compared to the discharge of item 1 in each case	Percentage of harmful silt load as compared to the harmful silt load of item 1	Percentage of Total silt load as compared to silt load of item 1	Average weighted mean diameter of suspended silt	Average weighted mean diameter of bed silt	Percentage of harmful bed silt i.e. with diameter of silt above 0.075 mm
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
1	Upstream Hyderabad Branch	214.3	2.782	98.62	1.38	0.0384	37.20	0.5134	100.00	100.00	100.00	0.0187	0.0438	12.70
2	Downstream Hyderabad Branch	143.5	2.743	98.85	1.15	0.0315	24.57	0.2826	66.99	55.04	66.05	0.0193	0.0534	21.05
3	Saidkhan Distributary	62.56	3.036	97.02	2.98	0.0905	11.85	0.3531	29.20	88.77	31.85	0.0201	0.0572	24.15
4	Masu Minor ..	9.434	2.968	99.09	0.914	0.0271	1.74	0.0159	4.40	3.10	4.68	0.0172	0.0373	2.10

DISTRIBUTARY HEADS

On account of the silting trouble in the distributary, certain modifications to the head regulator were proposed with the idea of keeping off more of the bad water and taking in more of the top water. The modifications comprised removal of the upstream right side winding pitching of the head regulator from where it is vertical to where it is 1 : 1 slope, and substitution of 1 : 1 pitching all through except where it is vertical, where a circular winding pitching for the entry was provided. A cut water at the downstream end of the upstream curtain wall of the Head Regulator was also provided.

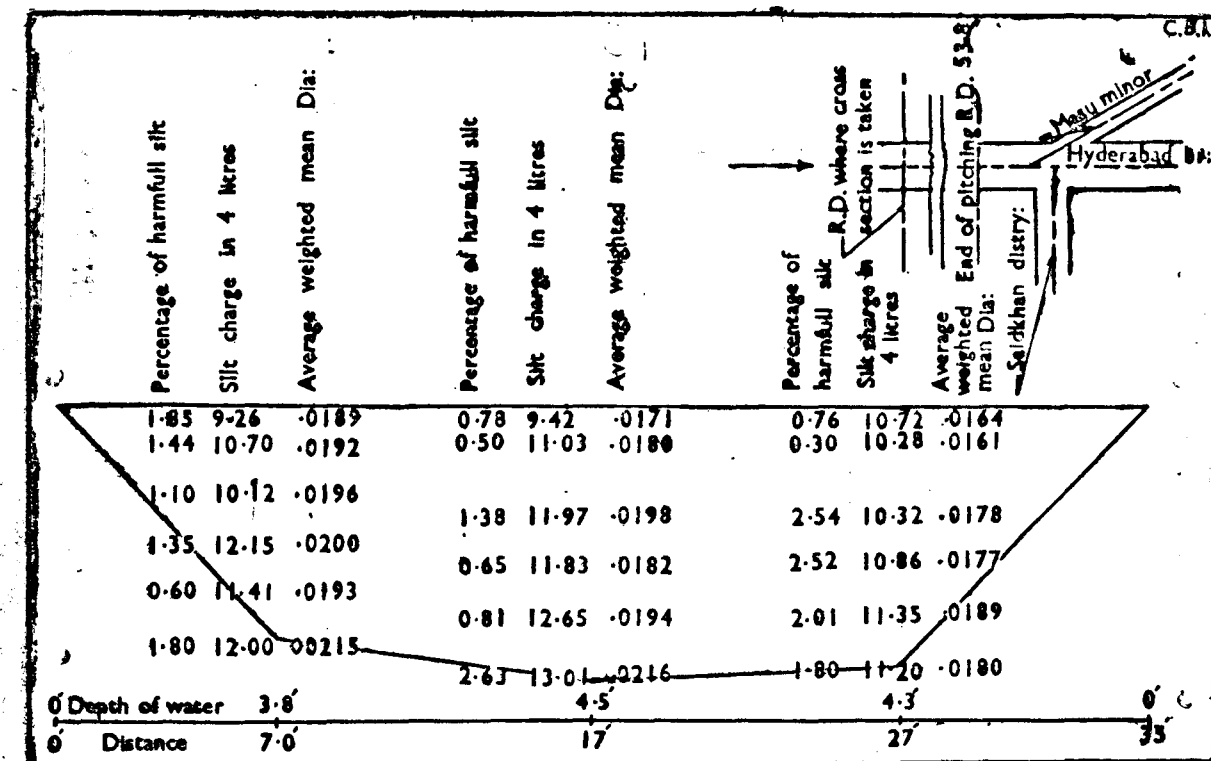


Figure 9 C. 6 : Cross section of Hyderabad Branch at R. D. 53 upstream of Saidkhan Distributary showing distribution of silt.

MODEL

A geometrically similar model $\frac{1}{4}$ th the full size was constructed with hydraulic data as given in Table 9 C. 10. Figure 9 C. 7 shows the upstream view of the head regulator over Saidkhan Distributary under original conditions.

EXPERIMENTS

The model was run under working full supply level conditions, the discharge in the model being 1.96 cusecs. In all 13 experiments were performed before final conclusions were drawn. The salient features of the principal experiments are given below.

Experiment No. 1

In this experiment the model was run without any upstream curtain wall. It was found that bed filaments upto $\frac{1}{4}$ th the bed width were all entering the offtake.

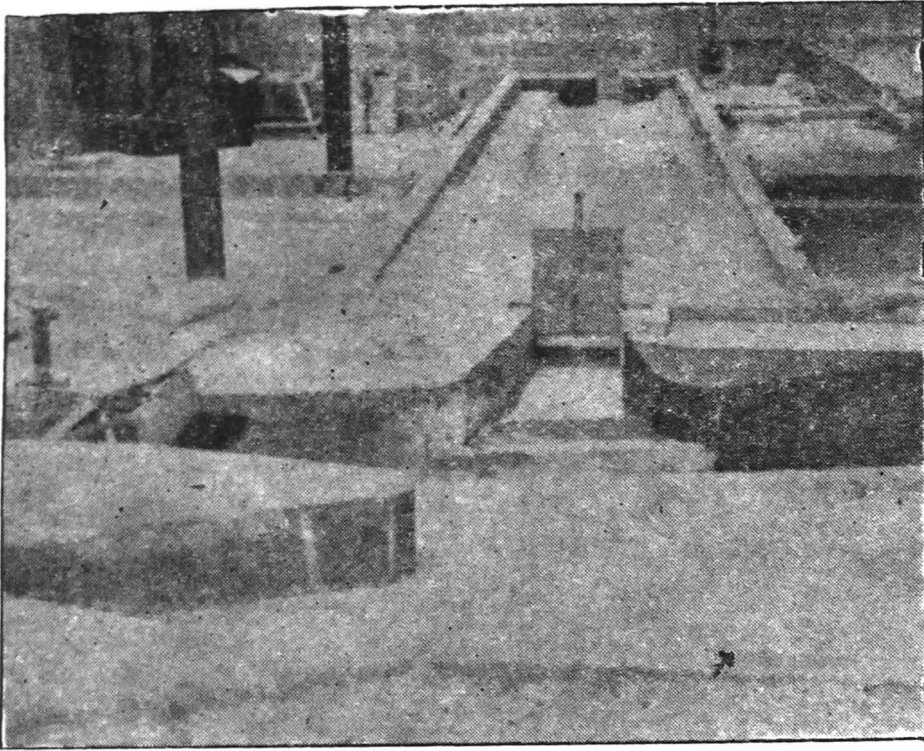


Figure 9C. 7 : *Showing upstream view of Head Regulator over Saidkhan Distributary under original conditions.*

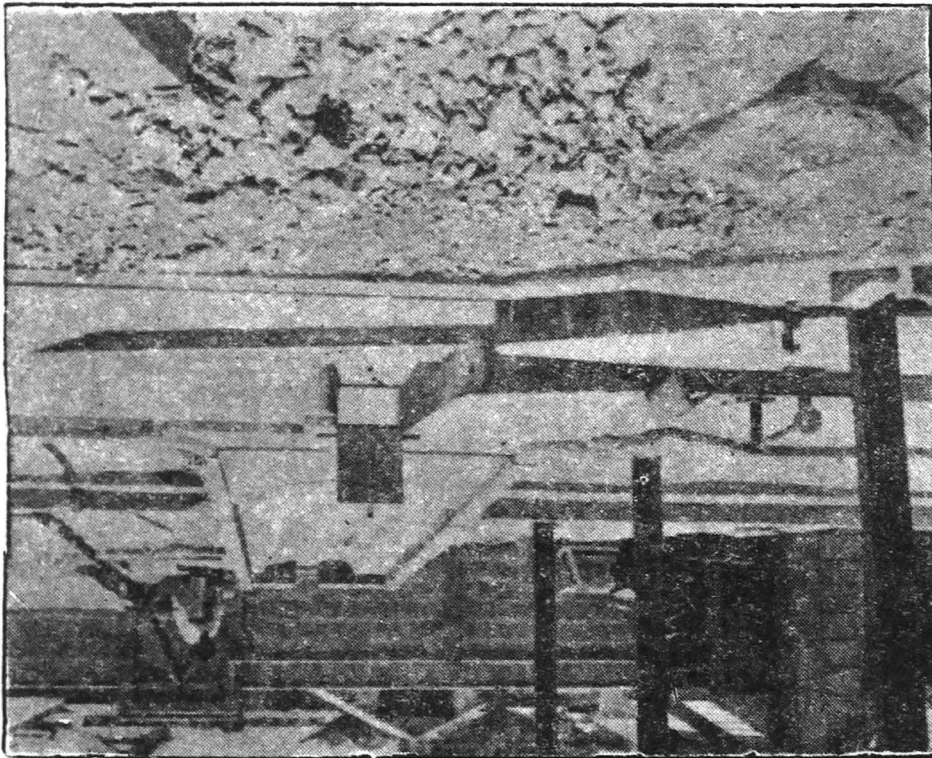


Figure 9C. 8 : *Showing upstream view of Head Regulator over Saidkhan Distributary after final modifications.*

738(a)

TABLE 9 C.10
Hydraulic Data Sheet

Serial No.	Particulars	Hyderabad Branch Upstream				Hyderabad Branch Downstream				Saidkhan Distributary Downstream				Masu Minor Downstream			
		Designed		Working		Designed		Working		Designed		Working		Designed		Working	
		Pro.	Model	Pro.	Model	Pro.	Model	Pro.	Model	Pro.	Model	Pro.	Model	Pro.	Model	Pro.	Model
1	Full Supply Discharge	267.76	2.065	254.5	1.963	161.52	1.246	156.0	1.203	92.46	0.713	86.0	0.063	13.78	0.106	12.50	0.0564
2	Full Supply Level ..	75.54	100.877	75.54	100.877	75.44	100.863	75.10	100.814	75.44	100.863	75.44	100.863	75.44	100.863	75.44	100.863
3	Bed Level	70.54	100.163	70.64	100.177	71.24	100.263	70.50	100.157	72.24	100.406	72.24	100.406	73.84	100.634	74.14	100.677
4	F. S. Depth in feet ..	5.0	0.714	4.9	0.700	4.2	0.600	4.6	0.657	3.2	0.457	3.2	0.457	1.6	0.229	1.3	0.186
5	Bed width in feet ..	27.5	3.93	31	4.43	20.25	2.89	24	3.43	19.5	2.79	19.5	2.64	8	1.4	8	1.14
6	Side slopes		$\frac{1}{2}:1$				$\frac{1}{2}:1$				$\frac{1}{2}:1$				$\frac{1}{2}:1$		
7	Hydraulic gradient parts per thousand	0.107	..	0.091	..	0.122	..	0.109	..	0.116	..	0.116	..	0.166	..	0.166	..
8	Mean velocity in feet per second	70.94	72.17	73.35
9	Crest Level ..						100.220				100.396				100.564		
10	Width of Meter Flume in inches	17.73	12.33	5.01
11	Length of Meter Flume in inches	10.59	10.07	5.64
12	C.L. of meter Flume	100.223	100.452	100.696
13	Length of Glacis 1 in 5 in inches	3.96	2.76	1.14
14	Radius of curvature in inches	12.78	7.4	1.93

DISTRIBUTARY HEADS

Experiment No. 3

Modifications as suggested by the Superintending Engineer Southern Sind Circle were introduced, except that a narrow upstream curtain wall was put in as against a wide curtain wall actually existing. By these modifications the bed filaments were practically cut off.

Experiments Nos. 4 and 5

Conditions for these experiments were the same as for Experiment No. 3 except that the upstream curtain wall was raised by six inches each time in two stages. The results were nearly the same as in Experiment No. 3

Experiments Nos. 6 & 7

The conditions for these experiments were the same as for Experiment No. 5, except in the matter of the length of cut water which was extended by 21 inches (prototype) in Experiment No. 6 and shortened by 21 inches (prototype) in case of Experiment No. 7. The latter gave better results than the former.

Experiment No. 8

The conditions were kept the same as in Experiment No. 1 except that a wide curtain wall as existing at site was provided. It was observed that upto 1/6 of the bed width the bed filaments were entering the offtake.

Experiment No. 10

The conditions were the same as in Experiment No. 3, except that the curtain wall was wide and a short cut water as in Experiment No. 7, was provided. The effect of the cut water, it was observed, was to introduce a roller near the nose which drew the bed filaments into the offtake at times.

Experiment No. 11

The conditions were the same as in Experiment No. 10, except that there was no cut water provided. This showed an improvement over Experiment No. 10 in that there was no suction effect due to absence of the cut-water.

Experiment No. 12

The conditions were the same as in Experiment No. 11, except that a narrow curtain wall 1.5 feet wide (prototype) was provided. Its behaviour in respect of bed filaments was erratic, and on the whole not as good as the wide curtain wall.

Experiment No. 13

In this experiment the conditions were the same as in Experiment No. 12 except that a short cut water was introduced. This did not show any benefit from a short cut water.

CONCLUSIONS

As a result of model experiments, the results of Experiment No. 11 were found to be satisfactory and were recommended for adoption. The modifications recommended have been shown in Figures 9C.8 and 9C.9.

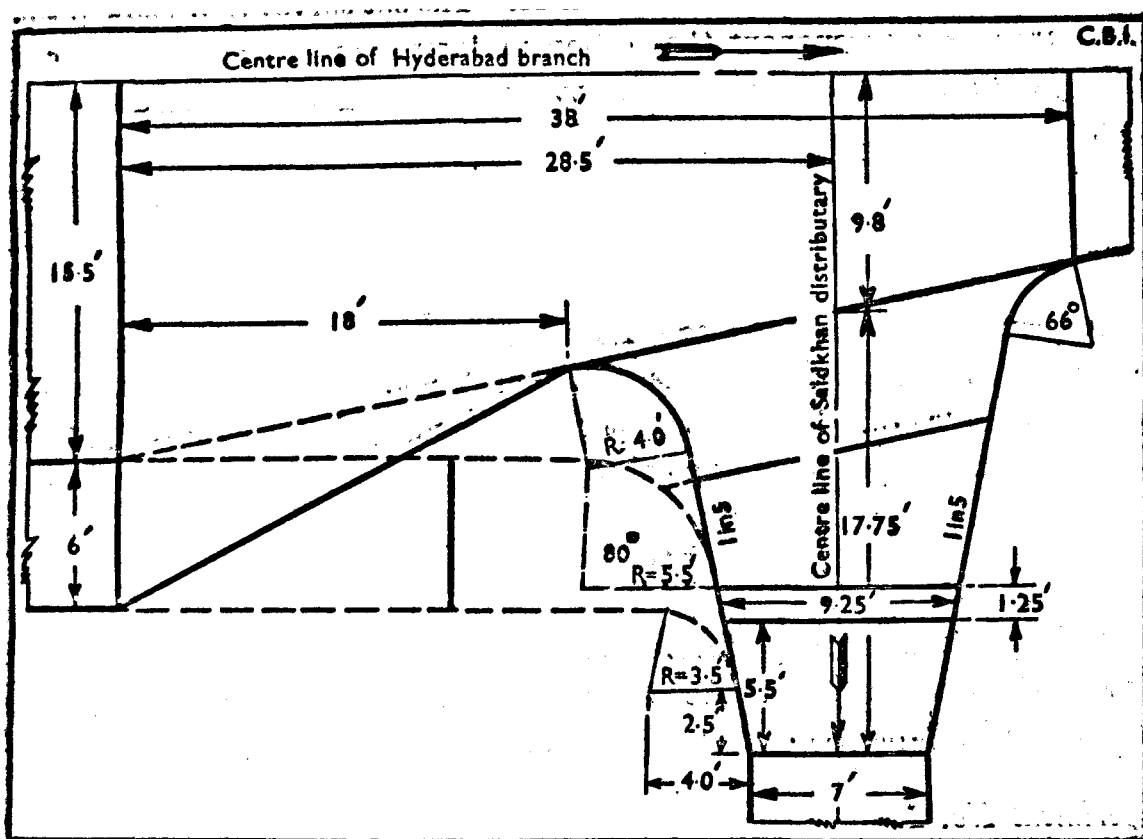


Figure 9C.9: Plan showing proposed additions and alterations to Head Regulator over Saidkhan Distributary Ex. Hyderabad Branch.

(8) MODEL EXPERIMENTS IN CONNECTION WITH SILTING OF DALORE AND PATOI DISTRIBUTARIES EX. JAMRAO CANAL (8).

INTRODUCTION

Dalore and Patoi distributaries take off from a common approach channel from Jamrao Canal at its 48th Mile. Both these channels silted very badly in their head reaches. The channels take off from a common approach channel from a group of regulators.

To bring about better distribution of silt and prevent entry of heavy silt from Jamrao canal a divide wall in front of these regulators and a raised curtain wall at the entry of the approach channel were constructed in December 1938. These modifications did not give any relief.

Before model experiments were taken in hand in 1946, the curtain wall of the approach channel had been raised from R. L. 71.20 to R. L. 72.88 as a silt excluding measure.

Prior to model experiments a silt survey at the head of Dalore and Patoi Distributaries was carried out in the *abkalanis* of 1943. The silt survey showed that the entry of the approach channel appeared to be defective in that both the off-taking channels carried a disproportionate share of the heavier silt, though

(8) Irrigation Research Division Sind, Annual Report 1946, pages 33 to 35.

the total silt load carried was their fair share. The silt survey also showed that the nature of silt throughout the section in Jamrao canal was mainly fine, showing the limitations under which any silt excluding device would have to work.

MODEL

A geometrically similar model 1/16th the full size was constructed. The working cross sections of the parent, approach and offtaking channels are given in Figure 9C.10. Figure 9C.11 shows the empty view of the approach channel at its entry under original conditions.

EXPERIMENT

17 Experiments were performed, the principal ones of which are described below. The filament distribution for each of the experiments described has been shown in Table 9C.12 and Figure 9C.12.

Experiment No. 1

The model was run under original conditions, the top of the curtain wall being at R. L. 71.20 (prototype) and the filament distribution taken. The observations showed that filaments upto 1/4 depth were entering the approach channel. Back flow was also observed in the pocket formed by the projecting nose.

Experiment No. 2

In this experiment, the conditions of working were the same as for Experiment No. 1, except that the top of the curtain wall was raised to R. L. 72.88, as existing at present. The observations showed for this case that 1/4 depth filaments were cut off, but the back flow was still there.

Experiment No. 4

In order to stop the back flow caused by the projecting nose, the nose was removed and entry taken straight. The silt deposit between the curtain walls of the approach channel was also removed. The filament distribution for this condition has been shown. The observations showed that the distribution was more or less the same as for Experiment No. 2 and at the same time the flow was regular.

DISTRIBUTARY HEADS

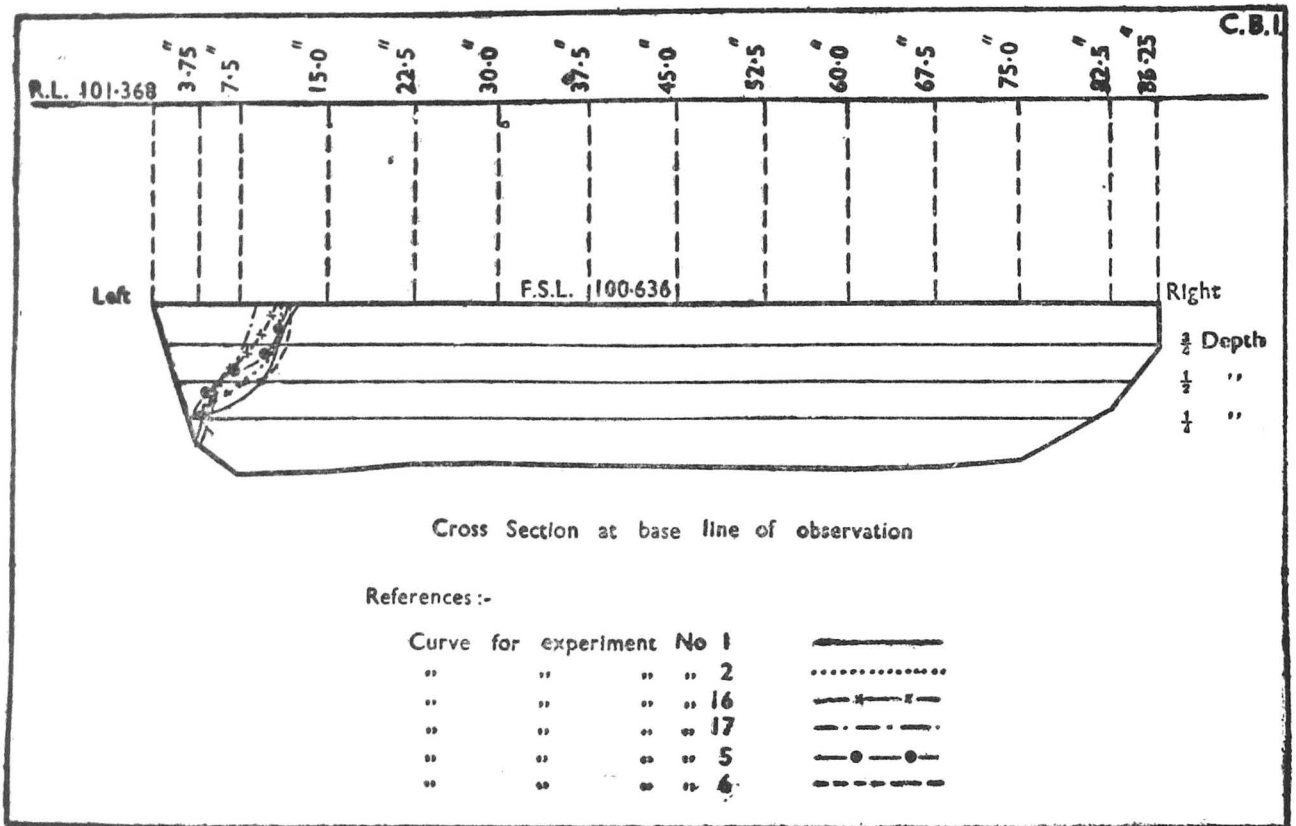


Figure 9C.12 : Model experiment in connection with silting of Dalore and Patoi Distributaries Ex-Jamrao Canal.

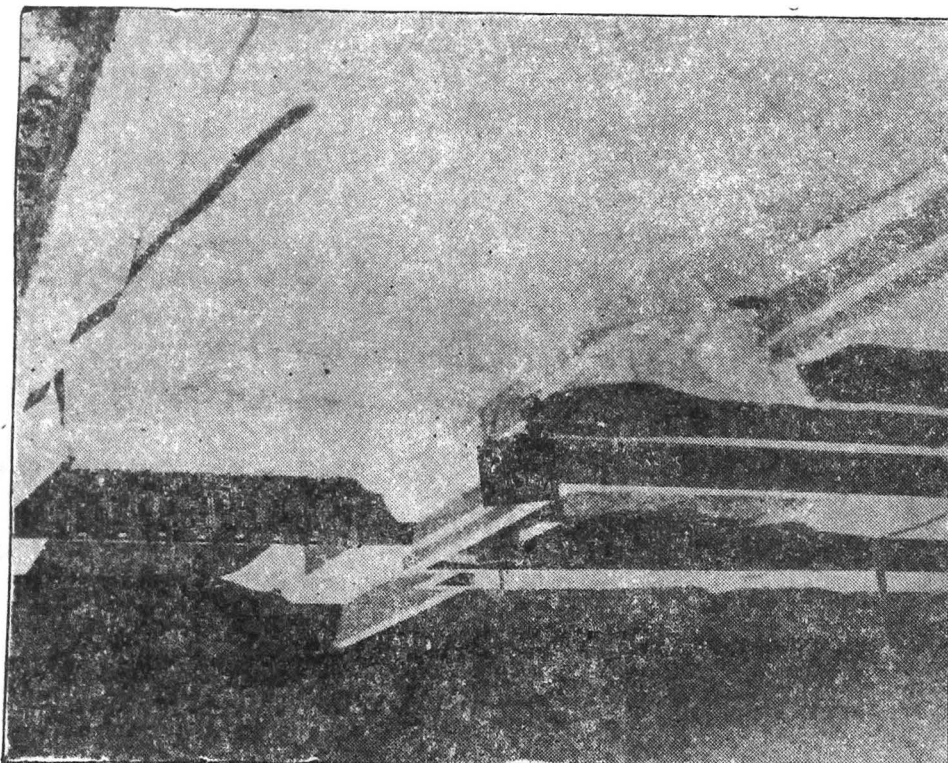


Figure 9C.13 : Showing entry to approach channel after final modifications.

TABLE 9C. 11

Hydraulic data for Dalore and Patoi Distributaries. Model scale ratio = $\frac{1}{8}$ R.L. 65.21 on Prototype = 100.0 on Model

Serial No.	Particulars	Jamrao Canal								Dalore Distributary				Patoi Distributary			
		Designed Upstream		Working Upstream		Designed Downstream		Working Downstream		Designed		Working		Designed		Working	
		Pro.	Model	Pro.	Model	Pro.	Model	Pro.	Model	Pro.	Model	Pro.	Model	Pro.	Model	Pro.	Model
1	Full Supply Discharge	23.96	2.3405	24.96	2.4385	21.53	2.1025	22.79	2.2255	164.0	0.1602	144.0	0.14065	79.0	0.07715	73.0	0.0713
2	Full Supply Level ..	73.20	100.499	75.38	100.636	70.30	100.318	71.00	100.362	74.96	100.609	75.21	100.625	74.91	100.606	75.21	100.625
3	Bed Level ..	64.40	99.949	66.88	100.104	61.5	99.768	62.50	99.831	71.26	100.378	72.31	100.444	71.98	100.432	72.91	100.481
4	Full Supply Depth in feet	8.8	0.550	8.5	0.531	8.8	0.550	8.5	0.531	3.71	0.231	2.9	0.181	2.93	0.183	2.3	0.144
5	Bed width in feet ..	93.5	5.845	100	6.25	76	4.75	90	5.625	25.51	1.594	26.0	1.625	17.2	1.075	20.0	1.25
6	Side slopes ..	1 : 1		$\frac{1}{2}$: 1		1 : 1		$\frac{1}{2}$: 1		$\frac{1}{2}$: 1		$\frac{1}{2}$: 1		$\frac{1}{2}$: 1		$\frac{1}{2}$: 1	
7	Hydraulic Gradient ..	1 in 13,706		1 in 20,000		1 in 11,377		1 in 16,000		1 in 8,560		1 in 8,400		1 in 6,860		1 in 6,800	
8	Mean Velocity in feet per second	2.781	0.6953	2.817	0.7043	3.042	0.7605	2.844	0.7110	1.620	0.4050	1.809	0.4523	1.444	0.3610	1.501	0.3753
9	Crest Level	63.78	69.16	69.86
10	Width of Meter Flume in inches	99.911	..	24.14	..	24.14	..	8.57	..	8.57	..	6.06	..	6.06
11	Length of Meter Flume in inches	14.85	..	15.42	..	5.31	..	4.89	..	4.11	..	3.90
12	Crest level of Meter Flume	99.823	..	99.848	..	100.432	..	100.462	..	100.469	..	100.495
13	Length of Glacis 1 in 5 in inches	3.30	..	1.02	..	3.24	..	1.08	..	2.76	..	0.84
14	Radius of curvature in inches	10	..	10	..	3	..	3	..	2	..	2

DISTRIBUTARY HEADS

TABLE 9C. 12

Statement showing Filament Distribution between Approach Channel and Jamrao Canal

Expt. No.	Particulars	Percentage of Filaments entering Approach Channel							
		Bed	$\frac{1}{4}$ Depth	$\frac{1}{2}$ Depth	$\frac{3}{4}$ Depth	Sur-face	Ave- rage	Dis- charge	Remarks
1	Original working conditions as per plan with top of curtain wall at R. L. 71.20 i.e. tapping 4.18 depth of water	0	1.28	9.10	11.70	15.03	7.42	8.68	
2	Original conditions as existing now i.e. with top of curtain wall at R. L. 72.88 so as to tap 2.5 depth of water	0	0	7.88	11.70	12.71	6.46	8.68	
4	Same as per Exp. 2 except that the entry to approach channel was carried straight and the space in between two curtain walls was cleared of all silt	0	0	8.49	11.12	12.71	6.46	8.68	
5	Same as per Expt. 4 except that the width of curtain wall at entry was increased from 20 to 6.25 and then allowed to meet 2nd curtain wall in a slope	0	0	5.48	11.70	13.30	6.09	8.68	
6	Same as per Exp. 5, except that the actual X-sections were replaced by designed section upto Dalore and Patoi Head Regulators	0	0	7.88	12.29	13.30	6.69	8.68	
12	Smooth curved entry of R. 65 along centre line of approach channel was provided, other conditions remaining same as in Expt. 6	0	3.2	7.9	11.1	12.7	6.98	8.68	
13	Smooth curved entry of R. 65 along C.L. of approach channel curtain wall with top at R.L. 72.88 and 6.25 wide the curtain wall was shifted out and the Upstream edge kept vertically above the left side bed edge of Jamrac Canal	0	1.28	5.26	9.94	12.14	5.72	8.68	
16	Do. Do. Do. Except R. 78 along centre line	0	1.92	5.26	9.94	12.14	5.85	8.68	
17	Same as per Expt. 1, except the winding pitching Upstream of entry was removed and curtain wall with its top at R.L. 72.0 provided as shown in plate	0	2.56	5.26	8.77	9.84	5.29	8.68	

Experiment No. 5

In this experiment the width of the curtain wall was increased from two feet to 6.25 feet *i.e.* 25 times the depth of water over it at full supply level (working). The observations showed that the wide curtain wall was effective in bringing down the 1/2 depth filaments entering the approach channel from percentage 8.49 to 5.46. The flow was regular.

Experiment No. 6

The approach channel was at this stage cleared of all silt and designed section provided to see how it worked after clearance. The observations showed that the filament distribution was more or less the same as in Experiments Nos. 2 to 5, but reverse flow was observed on left side in the approach channel in a length of 150 feet (prototype) starting from the entry.

Experiment No. 12

In this experiment the model was run after providing a curved entry to the approach channel of radius 65 feet (prototype), the other conditions remaining the same as in Experiment No. 6. The observations showed that 1/4 depth filaments were also entering the approach channel, and the reverse flow decreased considerably.

Experiment No. 13

The curtain wall was then shifted out and the upstream edge kept vertically above the left side bed edge of the Jamrao Canal as shown in Figure 9C.10. The observations showed that the percentage of filaments 1/4th depth and 1/2 depth entering the approach channel were reduced from 3.2 and 7.9 to 1.28 and 5.26, respectively. The nature of the flow was the same as in Experiment No. 12.

Experiment No. 16

Here again the curved entry was made easier, the radius of curvature being 78 feet along centre line, all other conditions remaining the same as in Experiment No. 13. The filament distribution for this condition was practically the same. The flow for this case was also regular except for a little back flow along the side slope.

Experiment No. 17

It was suggested that the original winding pitching on the upstream at the entry should be removed and the 1 : 1 pitching continued, meeting the left side pitching with a small circular entry, the top of the curtain wall being at the working bed level (R. L. 72) of the approach channel and the other conditions being as for original conditions. This was tried, giving results which were nearly the same as for Experiment No. 16 and it was more economical.

As the original conditions so far as bed filament exclusion were already good, it was felt that much further benefit could not be given by changing entrance conditions.

In looking into the design of the channels it was seen that both the channels were functioning with 15 per cent. deficient slopes and redesign was necessary to improve the performance of the channel.

CONCLUSIONS

As a result of the model experiments, the following suggestions were made :-

1. Removal of the winding pitching upstream to be replaced by 1 : 1 sloped pitching to join the approach channel in a small radius of five feet at top.

2. The existing curtain wall to be widened to 6·25 feet with its top at R. L. 72·00 (prototype) and its junction with the sloped pitching to be made as shown in Figure 9C.12. Figure 9C.13 shows the final modifications proposed.
3. The channels to be redesigned for deficient slopes on the modified Reid-Lacey Formulæ.
4. From a study of the existing widths of the channel it was seen that there was restriction at several places which appeared to be a contributory factor in silting, which were recommended to be removed.

(7) MODEL EXPERIMENTS IN CONNECTION WITH SILTING OF JAMRAO CANAL (9).

INTRODUCTION

Jamrao canal is an old canal taking off from the Eastern Nara Canal at Mile 116. This canal was completed in 1889—1900 and was designed as a perennial canal. It was remodelled to the Barrage duties and intensities, the designed discharge being 3,400 cusecs at the head as against 3,200 cusecs under pre-barrage conditions. In remodelling it a considerable amount of raising of full supply level was done to convert lift land under pre-barrage conditions into flow under Barrage conditions.

It appears that in pre-barrage times between 1900—1920, there was considerable rise in water level for the same discharge in Jamrao, and the fall at mile 7 of Jamrao canal got obliterated. Between 1920—1932, Jamrao had attained some stability and the rise in gauge for the working full supply discharge was one foot in *kharif*, which went down in *rabi*. There was no residual rise left in the head reach.

After the functioning of the Barrage, the rise in gauge came to notice but not till 1934, when the head had risen by about 0·85 foot. The normal residual rate of rise of gauge under post-barrage conditions was of the order of 0·3 foot per year.

There was excessive silting in 1940, and this after special measures were taken to reduce draw-off of bed silt during the year 1939, as a result of model experiments. The special measures comprised of the extension of the nose by about 400 feet at the bifurcation. Notwithstanding these measures the silting still continued.

In the model experiments carried out in 1939, it was seen that a new approach channel taking off much above the present approach channel showed promising results, but the proposal was not adopted as the then Superintending Engineer was of the opinion that it might result in silting of the lower offtakes.

MODEL

A distorted model with an exaggeration about 1·5 foot (actually 31/21) was constructed at the Hydrodynamic Research Station, Karachi, from a survey, plans and cross sections of the structures and channels involved. The total length of the model corresponded to one mile on the prototype, of which 2,000 feet was upstream of the bifurcation and 3,000 feet downstream. Figure 9C.14. shows the general lay out of the model. The hydraulic data sheet for the model is shown, in Table 9C.13.

(9) Irrigation Research Division Sind, Annual Report, 1946, pages 30 to 32.

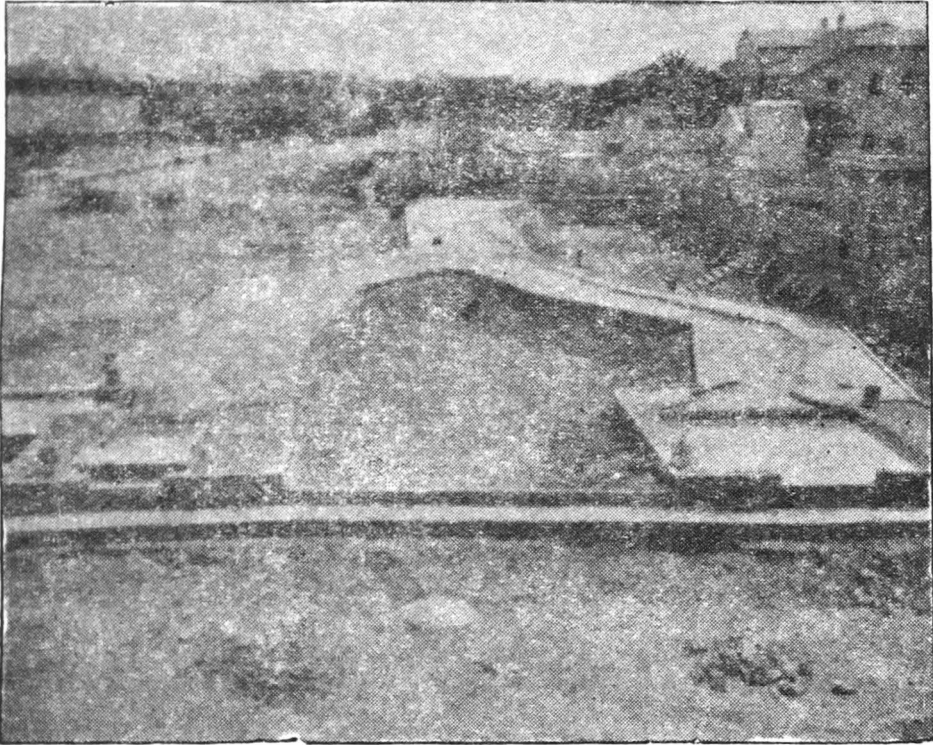


Figure 9C.14: *Showing general layout of Jamrao Canal*

TABLE 9C.13

Hydraulic data for Jamrao Canal Ex. Nara

Model scale ratio $\left\{ \begin{array}{l} \text{Hor. } \frac{1}{1} \\ \text{Ver. } \frac{1}{1} \end{array} \right.$ R. L. 93.50 on proto type = 100.00 on Model.

Serial No.	Particulars	Nara Upstream						Nara Downstream						Jamrao Canal			
		Designed		Working				Designed		Working				Designed		Working	
		Pro.	Model	Kharif		Rabi		Pro.	Model	Kharif		Rabi		Pro.	Model	Pro.	Model
				Pro.	Model	Pro.	Model			Pro.	Model	Pro.	Model				
1	Full Supply Discharge	11,300	3,786	10,000	3,352	7,000	2,346	7,900	2,646	71.00	2,380	4,100	1,374	3,400	1,140	2,900	0,972
2	Full Supply Depth in feet	7.7	0.37	6.0	0.29
3	Full Supply Level	110.5	100.810	110.5	100.810	106.0	100.595	103.0	100.452	109.2	100.748	109.2	100.648
4	Bed level	101.5	100.381	103.2	100.462
5	Bed width in feet	138	4.45	160	5.16
6	Side Slope	1/2 : 1	Irregular
7	Hydraulic Gradient	1 in 7,500 Nara	1 in 8,600 Jamrao	Approach Channel	1 in 5,600	1 in 4,400

Design not prepared—water level at weir simply raised to suit conditions from time to time

Design not prepared—water level at weir simply raised to suit condition from time to time

DISTRIBUTARY HEADS

EXPERIMENTS

In all 41 experiments were performed before final conclusions were drawn. The principal ones have been described below and tabulated in Table 9C.14. In all these experiments the dividing line for the bed load going into Jamrao approach channel to Nara was determined from a base line upstream of the bifurcation, the bed width at which place was 114 inches on the model, by injection of a coloured liquid slightly heavier than water which developed a stream of globules, the path along the bed, which could be traced.

Experiment No. 1

Under original conditions (the discharge on the model being 3.28 cusecs corresponding to 9475 cusecs on proto) it was seen that the normal distribution of the bed load in proportion to the discharges drawn by Jamrao and Eastern Nara for the *kharif* discharge, should have been 33 inches on the base line, but the actual dividing line was 60 inches, indicating that an excessive bed load was drawn by Jamrao canal.

Experiments Nos. 2a to 5d

An extended nose at the bifurcation of lengths, 233 feet, 295 feet, 200 feet and 186 feet (prototype) was tried in various positions under *kharif* conditions of discharge. The best result was given by Experiment No. 2a with the extended nose of length 233 feet with the nose 171 feet from the left side berm from a selected cross sectional base line. The dividing point was 36 inches as against 60 inches obtained for original conditions, showing the immense improvement given by the extended nose.

Experiments Nos. 6 and 7

In these experiments the model was run under *rabi* conditions of discharge without a nose and with an extended nose as in Experiment No. 2a above. The dividing point for *rabi* discharge with an extended nose was 54 inches on base line as against 78 inches for a proportionate distribution. This shows the efficacy of the extended nose under *rabi* conditions of working.

Experiments Nos. 8 and 9

These were same as Experiments Nos. 6 and 7 except that the discharge was boosted by 25 per cent. in Nara, the discharge in Jamrao being full supply discharge. The dividing point on base line was 27 inches as against the value of 30 inches for proportionate distribution, showing the efficacy of the modifications under boosted conditions of working.

TABLE 9C. 14

*Experiments in connection with silting of Jamrao Canal**Statement showing bed filament distribution between Jamrao approach Channel and Nara.*

Item No.	Expt. No.	Conditions of working	Point from where it just enters Nara	Proportionate dividing point
1	1	Original conditions without extended nose and carrying full supply discharge both in Nara and Jamrao-Kharif Discharge.	60	33
2	2a	Original conditions with extended nose 233 feet long and 171 feet from a fixed point in left side berm edge.	36	33
3	6	Same as in Expt. No. 2a i.e. 233 feet long extended nose at 171 feet from left side berm edge but run under <i>rabi</i> conditions of discharge.	54	54
4	7	Same as in Expt. No. 6 except the extended nose was totally removed.	78	54
5	8	Same as in Expt. No. 2a i.e. 233 feet long extended nose at 171 feet from left side berm edge but run with boosted discharge of (1625 + 6475) cusecs in Nara, the gauges were also maintained accordingly.	27	30
6	9	Same as in Expt. No. 8 except the extended nose was totally removed.	57	30
7	11b	Right side berm cut as per S.E.D.C's instructions 156 feet long extended nose kept at 213 feet from left side berm edge and model run with boosted discharge.	18	30
8	12b	Right side berm cut as per S.E.D.C's instructions. 156 feet long extended nose kept 197 feet from left side berm and model run with <i>kharif</i> discharge.	33	33
9	12c	Same as Expt. No. 12b except extended nose kept at 205 feet from left side berm edge.	36	33
10	15	Expt. 12c repeated i.e. 156 feet long extended nose at 205 feet from left side berm after putting in the groynes along left bank.	33	33
11	17b	Expt. No. 12b repeated with groynes put along left bank and model run with <i>Rabi</i> discharge.	51	54
12	18a	Model run with boosted discharge, right side berm cut and groynes provided along left bank as per Expt. No. 15, 156 feet long extended nose kept at 205 feet from left side berm edge.	21	20
13	18b	Do. Do. At 197 feet from left side berm edge. . .	15	30
14	19	156 feet long extended nose kept at 201 feet from left side berm edge, right side berm cut as per S.E.D.C's instructions and groynes put along left bank model run under <i>kharif</i> conditions.	30	33
15	20	Expt. No. 19 repeated with <i>rabi</i> conditions	54	54
16	21	Do. Do. Do. Boosted conditions	21	30

Experiments Nos. 10a to 11b

In these and subsequent experiments a change was made in the shape of the berm on the right side, giving a reverse curve roughly opposite the extended nose as shown in Figure 9C.15. The shape of the right side berm before and after cutting is shown in Figures 9C.16 and 9C.17 respectively.

The model under experiments 10a to 11b was run with an extended nose of lengths 233 feet and 156 feet (prototype) in various positions under boosted conditions of discharge. The best results among these experiments were given by the extended nose 156 feet long and at 213 feet from left side berm edge. The dividing point for this was 18 inches on the base line, as against a value of 30 inches for proportionate distribution.

Experiments Nos. 12a to 12c

The model was then switched on to *kharij* conditions of discharge and extended nose 156 feet long was tried out in various positions. The results for all the positions of nose from 192 feet to 205 feet from left side berm edge were satisfactory, the dividing line being 33 inches to 36 inches on the base line.

Experiments Nos. 13 and 14

In these experiments Experiments 12a and 12c were repeated with the nose extended to 200 feet and 233 feet length respectively, which gave no improvement, and showed that a length of 146 feet was sufficient.

Experiment No. 15

The extended nose of 156 feet length at a distance of 205 feet (prototype) from left side berm edge was further repeated after introducing groynes on the left side berm in a regular curve. The dividing point for this was 33 inches on the base line.

Experiments Nos. 16a to 17b

In these experiments the extended nose of 156 feet length was tried in various positions under *rabi* conditions of discharge, both with and without groynes on the left bank. The presence of groynes gave improved distribution. The dividing point was 51 inches and 60 inches for with and without groynes respectively, the value for proportionate distribution being 54 inches.

Experiments Nos. 18a to 18b

These experiments were under boosted conditions of discharge, the other conditions being as for Experiment No. 17 above. The results were satisfactory

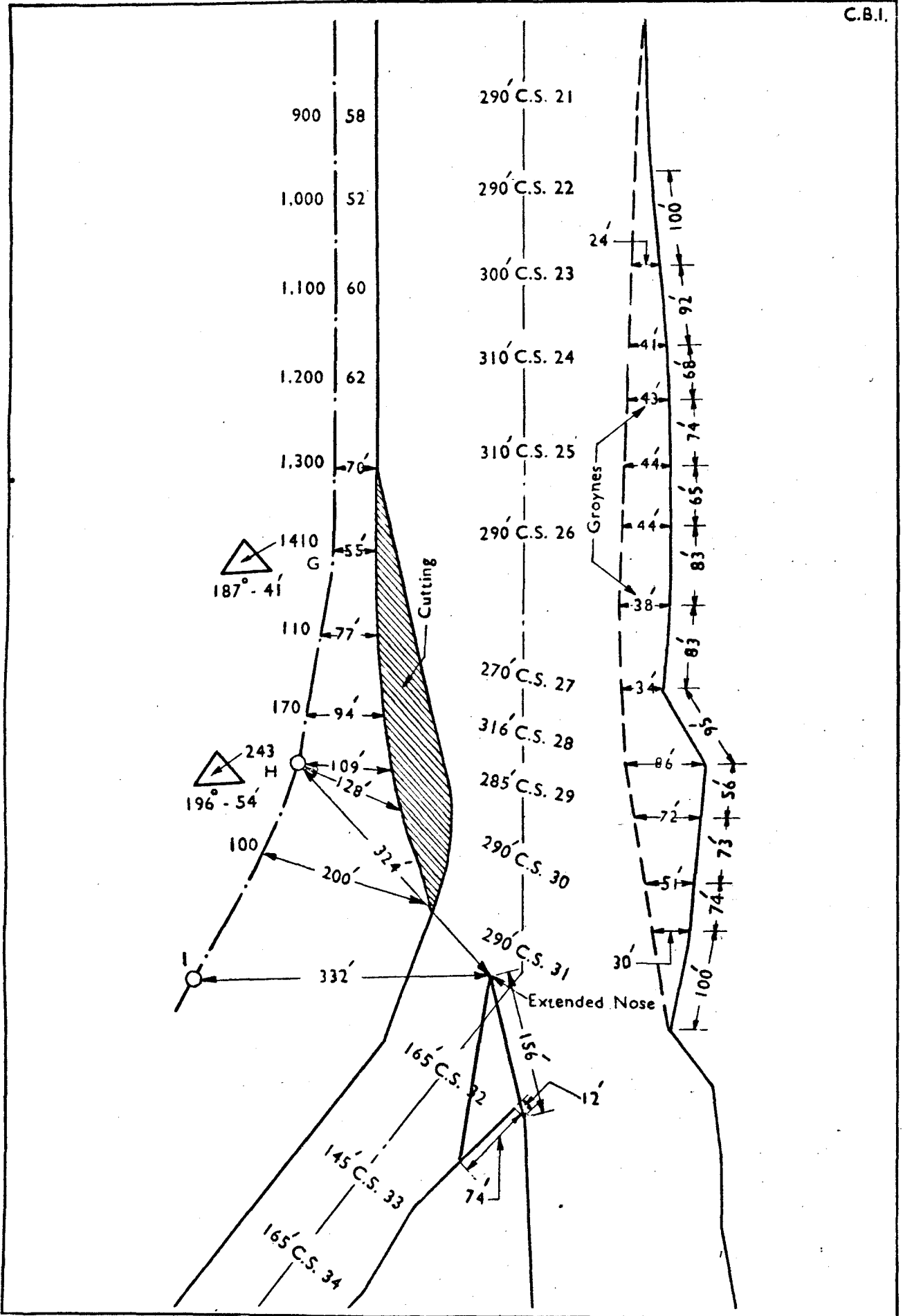


Figure 9C.15 :- Showing Proposed Modifications at the Site of Bifurcation
[Silting of Jamrao Canal]

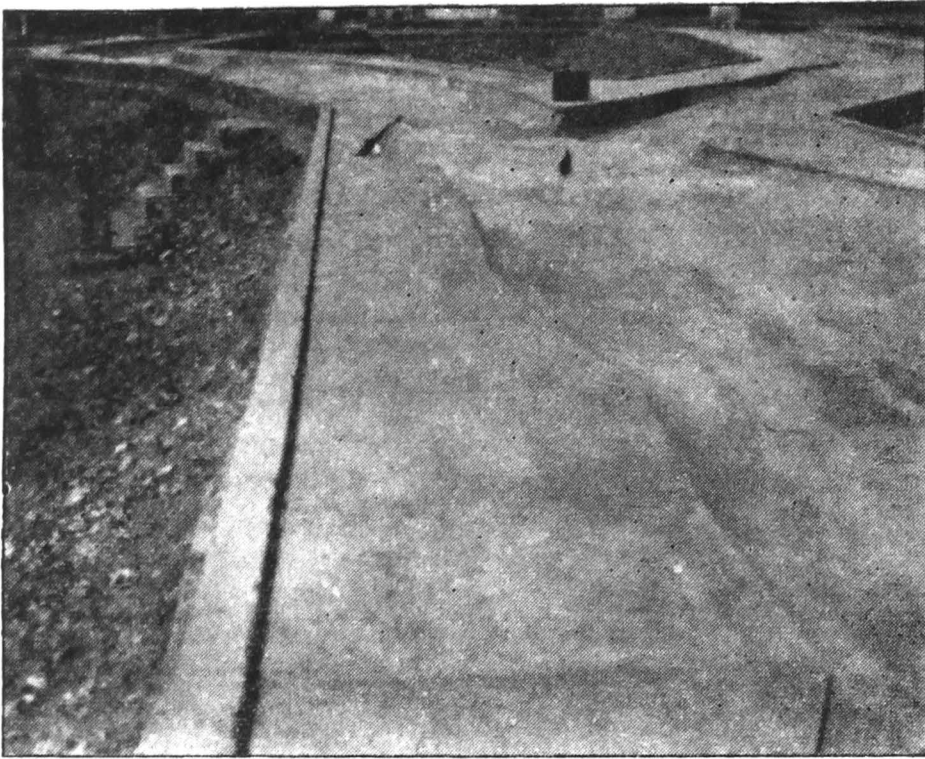


Figure 9C.16 : *Showing the site at bifurcation under original conditions.*

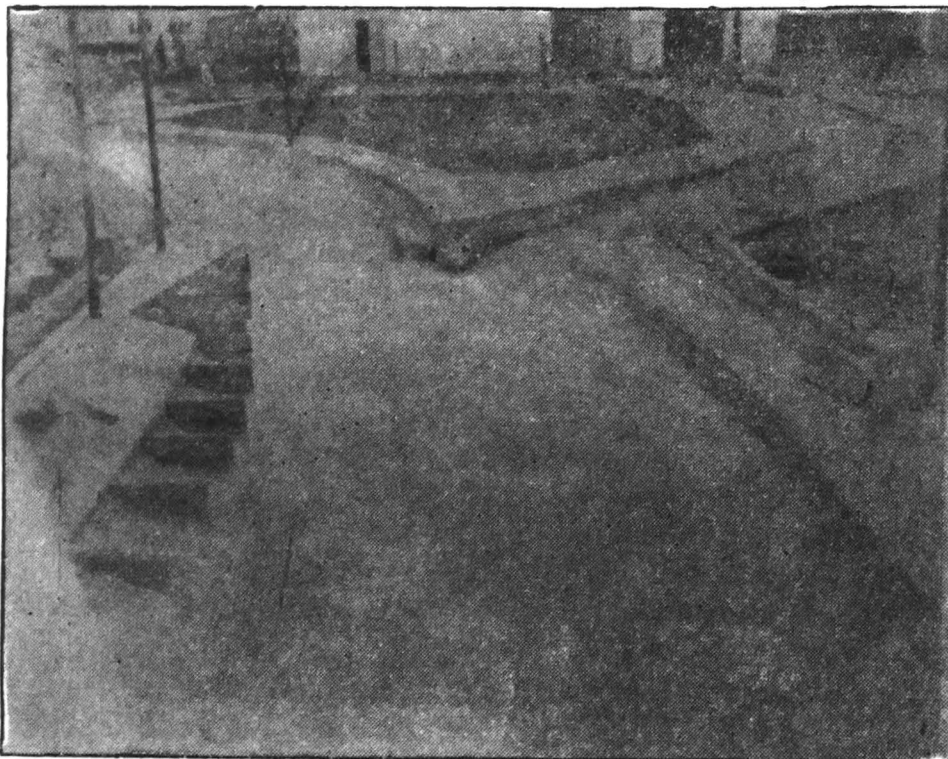


Figure 9C.17 : *Showing the site at bifurcation after final modifications.*

The dividing point varying from 21 inches to 15 inches against 30 inches for proportionate distribution.

Experiments No. 19 to 21

From the previous experiments, it was observed that an extended nose of length 156 feet at 197 feet and 205 feet respectively from left side berm gave the best results and not appreciably different from each other. The final position selected was with the nose at 291 feet (mean value) from left side berm from considerations of alignment of the nose, and experiment No. 19 was for the *kharif* conditions of discharge. The experiment was also repeated for *rabi* conditions (Experiment No. 20) and boosted conditions (Experiment No. 21) of discharge and satisfactory results were obtained. The dividing points for the *kharif*, *rabi* and boosted conditions of discharge were 30 inches, 54 inches and 21 inches as against 33 inches, 54 inches and 30 inches for proportionate distribution.

CONCLUSIONS

As a result of the experiments, the following modifications were suggested and were executed in the December closure of 1946.

(1) Extended nose 156 feet long at 201 feet from the left side berm, the position of which was defined by offsets of 324 feet and 332 feet from the survey points H and I, Table 9C.14 cutting in right side berm as per Table 9C.14 groyning of left bank as per Table 9C.14. All the final modifications suggested are visible in Figure 9C. 17.

DISCUSSION BY THE RESEARCH COMMITTEE

MR. D' ABREG introduced items (1) and (3) to (7). Referring to item (1) he said that these were basic experiments to find out the effect of angle of off-take and discharge on silt entry. He explained that no actual self injection was done but the filament distribution was obtained by the liquid injection method. The angles of offtake were varied from 30° to 120° and the discharge in the offtake from ten per cent. to 50 per cent. of the parent channel. During the year under report experiments with 30 per cent. discharge in offtake were done and further experiments were in progress. The conclusions for 30 per cent. discharge were that 60° offtake gave the worst bed filament entry and increased with the percentage discharge in offtake though not in direct proportion.

Regarding item (3) Basic experiments for determining coefficient for radial gates, he said that these experiments were carried out to find the coefficients to be adopted in the formulae for radial gates for both standing wave and drowned conditions as the previous coefficients derived from experiments done in 1930-32 did not give correct discharges when checked with observed discharges as complained by Irrigation Division. There were defects in the original experiments in the matter of technique and the fact that only one scale and one span model experiments were tried out. The new experiments showed that the coefficients were higher than those previously obtained and these were tried out to three scales and one, two spans, three span models were in progress in 1947 and experiments have just been completed.

Referring to item (4) Model Experiments in connection with scour below crest regulator over Sind Canal MR. D' ABREO said that this was a specific experiment for dealing with side and bed scour downstream of Rustom Regulator on Sind Canal. Baffles were tried in the first instance. It was seen that the regulator had been worked for greater cut off than it was safe. It was therefore restricted to 5.5 feet. On the suggestion of the Superintending Engineer, Drainage and Research Circle the recommendations made to the Executive Engineer Shikarpur Canals Division were that the upstream channel should be straightened and the downstream channel will be splayed 1 in 10 with groynes, provided in the bellies and a deflector nine inches at the end of the pavement.

Continuing Mr. D' ABREO introduced item (5) Model experiments in connection with silting of Saidkhan distributary ex Hyderabad branch and said that this was a specific experiment dealing with the silting of Saidkhan distributary ex Hyderabad branch. The Superintending Engineer, S.S. Circle suggested certain modifications which were to be tried out on a model to see the effect. These consisted of removing the upstream winding pitching on the right side of the parent channel, provision of a cut water on the downstream end of the upstream curtain wall. These were tried out and found effective except that the cut water was not found beneficial and was therefore omitted.

Further he said that item (6) Model experiments in connection with silting of Dalore and Patoi distributaries ex Janrao Canal, was a specific experiment in which silting of Dalore and Patoi distributaries was dealt with. These two channels take off from a common approach channel. After various experiments tried as shown in the report the recommendation made was that the upstream binding pitching should be removed and 1 to 1 slope pitching provided and the width of curtain wall at Reduced Level of the working bed level to be $2\frac{1}{2}$ times the depth of flow over it at full supply level.

As regards item (7) Model experiments in connection with silting of Jamrao Canal he said that this was a specific experiments for dealing with the silting of Jamrao Canal. Experiments had been done in 1939 and certain measures suggested, which though adopted the canal still persisted in silting. Further experiments were therefore done as a result of which an extended nose was provided at the bifurcation cutting done on right side to a curve such that proportion distribution would result, groynes were provided on the left side of the parent canal.

MR. CHATURVEDI introduced item (2) and briefly described the experiments carried out.

MR. BLENCH asked Mr. Chaturvedi how the scour was affected.

MR. CHATURVEDI replied that the scour was very severe. The blocks created the standing wave and reduced the scour.

MR. FRAMJI referred to item (1) and said that the experiments were carried out in 1/12 scale geometrically-similar model. Geometrically-similar models were not correct for this type of enquiry, as the whole problem depended on the curvature of flow, and on the ratio of the discharge of parent channel. The

distribution of bed water and surface water drawn by offtake could only be obtained with correct bed configuration of the parent channel. These could only be obtained in vertically-exaggerated models. It was also not clear whether there was silt in the bed, as there should be, or the model was rigid.

Regarding items (5) and (6) he said that a geometrically-similar rigid model had been used, whereas a vertically-exaggerated mobile model was required for simulating correct curvature of flow.

MR. FRAMJI also enquired what was the modified Reid-Lacey formula.

MR. D' ABREO said that the modified Reid-Lacey formula was given in Sir Claude Inglis' paper published in 1942-43.

MR. DILDAR HUSSAIN asked Mr. D' Abreo if he had tested scour on the model of radial gates.

MR. D' ABREO replied that the radial gates model was rigid and there the scour could not be tested. It was only the coefficient of discharge that was studied on this model.

CONSIDERATION BY THE BOARD

THE SECRETARY described the work done at the Research Committee Meeting.

It was decided to keep the subject on the Agenda.

10 C-Excluders and Ejectors

PRELIMINARY NOTE

This subject has been on the agenda since 1938, and papers have been contributed describing the design and results of excluders and ejectors in the Punjab, dealing with various aspects of design and giving experimental results.

At the 1945 meeting of the Board it was resolved that a Central Board of Irrigation publication should be issued on the subject summarising the position upto 1945-46, and that the Punjab Irrigation Research Institute be requested to draft the proposed publication.

The Chief Engineer, Punjab Irrigation, issued instructions to the Director, Irrigation Research Punjab to prepare a draft of the proposed publication. The draft is still being awaited.

The following items were discussed at the 1946 Research Committee Meeting :—

- (1) Design of the Lower Sind Barrage : Experiments to test the design for scour and sand exclusion from canals
- (2) Flume experiments on silt extraction
- (3) Excluding trash and silt from the feeder channel of the Renala Power House
- (4) Determining a suitable design of silt extractor on a model of the Upper Bari Doab canal
- (5) Exclusion of detritus from Upper Bari Doab canal
- (6) Model Experiments on exclusion of shingle and silt from the Western Jumna Canal.

Recent Literature

- (1) Sand traps designed for streams with heavy bed-loads, by H.A. Burt, Civil Engineering, August 1946, vol. 16, No. 8, page 357.

The year's work

The following items were presented for discussion at the 1947 Research Committee Meeting :—

- (1) Model experiments for exclusion of silt from Damodar Canal
- (2) Analysis of observations on silt exclusion at the headworks of Damodar Canal at Rhondia
- (3) Silt exclusion works at Lloyd Barrage at Sukkur by T.R. Sethna.
- (4) Design of Lower Sind Barrage at Kotri for effective sand exclusion from canals
- (5) Optimum shape of the nose of Keti Island at Kotri for the Lower Sind Barrage
- (6) Model experiments on the exclusion of sand from a feeder channel taking off from Pennar river
- (7) Sediment investigation—river Sutlej at Hussainiwala Headworks
- (8) Sediment investigation—river Jumna at Tajewala Headworks
- (9) Sediment investigation—river Ravi at Dhanna Weir.

(1) MODEL EXPERIMENTS FOR EXCLUSION OF SILT FROM DAMODAR CANAL (1) HISTORY

A canal called the Damodar canal was taken out of the left bank of the Damodar river just above the Anderson Weir at Rhondia drawing a maximum discharge of 2,000 cusecs from the river for irrigation purposes. There is no canal on the right bank. The undersluice on the left end of the weir at the canal head regulator consists of three bays each 60 feet wide with gates falling from above on a sill at level 156.0 feet (R. L.). The weir which is 3,750 feet long from the divide-wall to the right guide-bank has its crest at level 164.0 feet (R. L.). The level of the *pucca* floor in the undersluice pocket between the divide-wall and the canal head regulator is 156.0 feet (R. L.). The canal head regulator consists of five openings each 20 feet wide and 4.5 feet deep. The lower sill of each opening is at level 160.0 feet (R. L.) and the upper sill at level 164.5 feet (R. L.). The gates operating in these openings rise from below, so as to take water from the upper layer into the canal. After the construction of the weir and the canal system in 1933, the canal was opened and it has been experienced since then that considerable volume of coarse sand gets into the canal through the head regulator each year during the flood season, which is also the irrigation season and deposits within the first five miles of the canal from the head regulator. The width of the canal within this reach is 100 feet and the section was designed to carry a maximum discharge of 2,000 cusecs. The Irrigation Department has to spend a considerable amount of money each year for removal of this sand. Further when silting is heavy, difficulty is experienced in drawing the full discharge of 2,000 cusecs into the canal towards the latter part of the irrigation season. The Irrigation Department wants to save this annual expenditure by adopting some method of silt exclusion at the head regulator and the problem was referred to the River Research Institute West Bengal for investigation and remedy.

From about two miles above the weir the main channel of the river is along the left bank (see Figure 10C. 1.) which is practically normal to the weir line

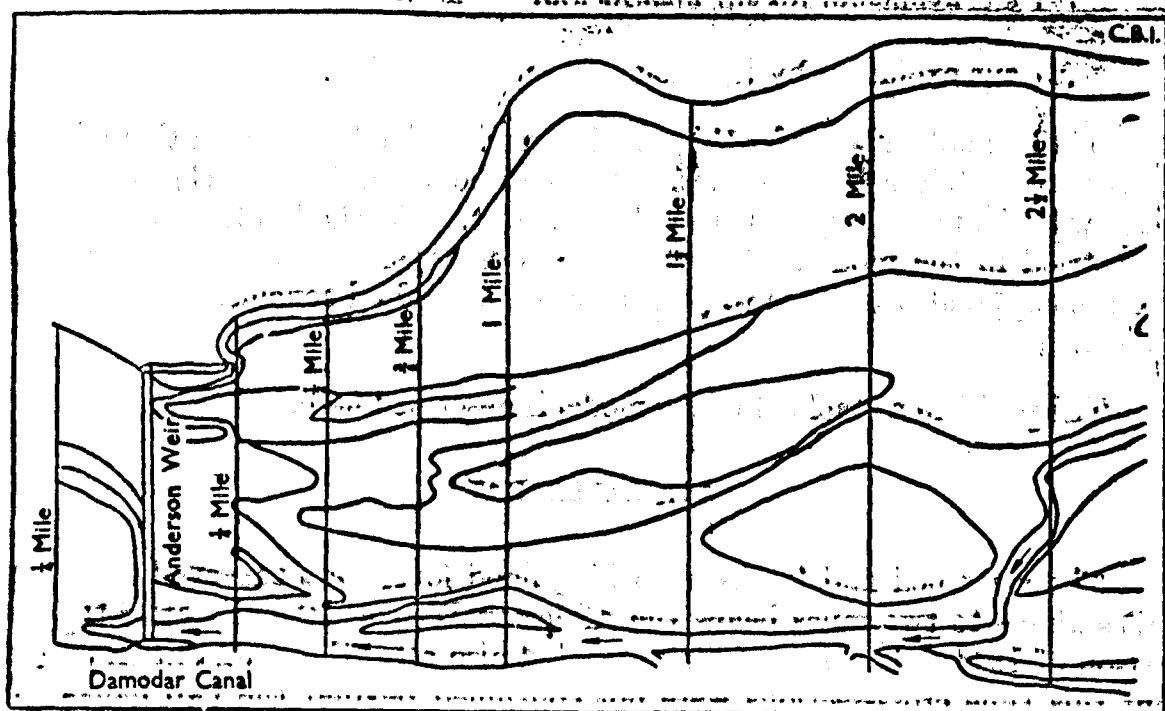


Figure 10C. 1: Plan of the Damodar river at Anderson Weir-survey after 1944 Flood Season,

(1) River Research Institute, Bengal, Annual Report, 1946, pages 81 to 85.

within this portion. This being a rain-fed flashy river, the whole discharge during the winter season comes along this deep channel, and, if undersluices are fully open, flows through the undersluices. This channel along the left bank has, therefore, been maintained without any difficulty. The flow in the undersluice portion is practically normal to the weir line. In the flood season the major portion of the discharge flows over the left 1,000 feet of the weir, which has, therefore, been specially protected by construction of friction blocks on the downstream floor. There is no shingle on the river bed here and the mean diameter of bed sand of this reach would be about 0.8 mm. It has also been experienced at the head regulator that unless the water level in the undersluice pocket is maintained above 164.80 feet (R. L.), the designed maximum discharge of 2,000 cusecs cannot be fed into the canal. The crest level of the weir being 164.0 feet (R. L.) and there being no crest shutters it is evident, therefore, that a part of the river discharge must be allowed to flow over the weir crest in order to push the full supply discharge into the canal. It has been found that if the river discharge is below 6,000 cusecs, the canal cannot be fed with its full supply and difficulty is experienced sometimes during the latter part of the irrigation season to feed the canal with full supply for want of required discharge in the river. It has been mentioned before that almost all the silt enters the canal drops in the first five miles of the head reach. This is due to the fact that the canal in this reach runs through deep cutting and high spoil banks are on both sides. During rains these spoil banks slide down and obstruct the passage of water in the canal. This heads up supply and the head available at the head regulator is considerably reduced.

There are some well known methods of dealing with this problem. They are :—

- (1) silt exclusion at the head by silt excluder
- (2) silt exclusion by silt ejector and
- (3) silt exclusion by judicious gate manipulation.

Method (2) is not suitable for this canal as during the flood season there is practically no stretch of the river where this can be done. Method (1) is expensive and will be taken up as the last resort. Method (3) was, therefore, tried in the model. A similar series of experiments were designed for the prototype also but due to paucity of data no final conclusion could be obtained.

The problem as tackled in the model is described here.

AVAILABLE DATA

The river bed and banks upto high flood level were surveyed from eight miles above to two miles below the Anderson Weir at Rhondia after the flood season of 1944. This survey was repeated also after the flood season of 1945, but this time cross-sections were not continued beyond the two banks. The cross-sections of a small channel called *Bodai Nala* which takes off the right bank

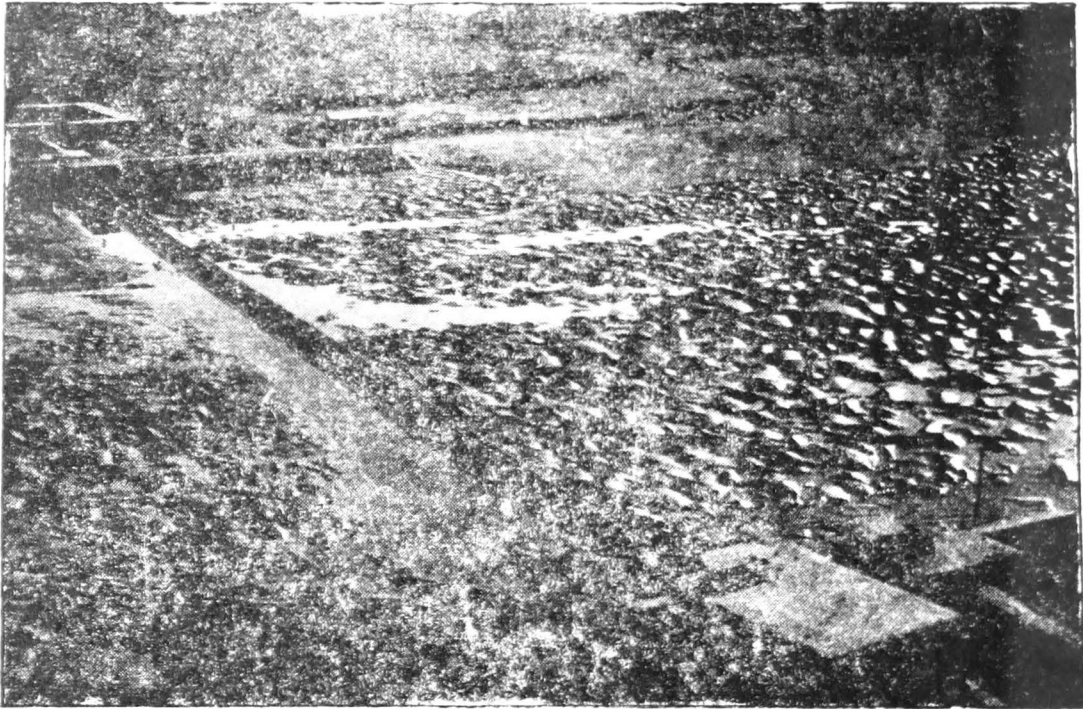


Figure 10C. 2: *Damodar River Model—Showing the weir, undersluice, etc., looking upstream.*

of the Damodar at Rangamati, about $3\frac{1}{2}$ miles above the weir and joins the present river again at the upstream nose of the right guide bank at the weir, were also left out in this survey. The model was, therefore, prepared on the survey done after 1944 flood season.

Gauge readings observed during the flood seasons of 1945 and 1946 at the following gauge sites were available for reference. They are as follows :—

- (1) Right and left banks, upstream and downstream of the weir at the headworks
- (2) Saldanga on the left bank of the river, one mile above the weir
- (3) Bharatpur on the left bank of the river about three miles above the weir
- (4) Rangamati on the right bank of the river about $3\frac{3}{4}$ miles above the weir.

It was also pointed out that the *Bodai Nala* mentioned above, had made a big embayment behind the upstream end of the right guide bank of the weir and was attacking its nose heavily. Emerging from this nose it has made not her encroachment into the river forming a big *char* in front of the right guide bank. This *char* is about 600 feet wide and extends from this upstream nose up to the weir crest. This attack on the upstream nose of the right guide bank and the *char* formation in its front have become another problem to be tackled immediately. This is increasing the concentration of the river flow on the left portion of the weir. It was also intended to study this problem in this model, but the latest data have not yet been received.

It will be proper here to mention that the head regulator gates are fully closed to stop the entry of river water into the canal when the water level in the pond goes above 170.0 feet (R. L.). The river discharge at this stage is about 200,000 cusecs and the river water is highly charged with silt. The divide-wall extends up to 153 feet above the weir line and its top level is 169.0 feet (R. L.), so that when the water level goes above 169.0 feet (R. L.) the river spills over the divide-wall.

THE MODEL AND THE EXPERIMENTS

The portion of the river from $7\frac{1}{2}$ miles above to two miles below the Anderson Weir at Rhondia was incorporated in the model, Figure 10 C. 2. The model was laid according to the data of the survey made after 1944 flood season. The banks were contoured up to high flood level as surveyed. Horizontal and vertical scale ratios were chosen to be 1/200 and 1/80, respectively, and sand having a mean diameter of 0.33 mm. was used in the river bed. The model of the weir, undersluice and head regulators was constructed in the same scale as above and put in proper position in the model.

Experiments were started from June 1946 when water-supply to the models was available from the Damodar Canal at Galsi after a period of closure for the annual silt clearance in the canal bed. The canal water was turbid with fine suspended silt which was found to settle on the model river bed after a day's run and prevent further silt movement on the bed. Moreover due to turbidity of water bed movement could not be seen from above. It soon, however, became clear that very little bed movement occurred in the model within one mile above the weir, unless the discharge passed in the model was more than that equivalent to 200,000 cusecs in the prototype, a stage at which the head-regulator gates in the prototype are completely closed to prevent excessive silt-entry into the canal. The prospect of studying bed-movement and silt entry into the canal in this model for discharges below 200,000 cusecs was not encouraging, though it was decided to try a lighter bed material for the purpose. Small coloured balls made of wax and chalk powder mixed in such proportion as to make the balls just heavier than water, were prepared and dropped on the model river bed at different points to see how they move along the bed and whether they enter the canal through the head-regulator opening. It was found that in such a small model of a flashy river, small balls of even one mm. diameter would not enter the canal through the head-regulator opening when the proper discharge was allowed into the canal. It was anticipated that it may be necessary to study the problem in a bigger model where there would be sand movement even at a lower stage. It was, therefore, decided to study the flow pattern in the reach within one mile above the weir, in this model and simulate these conditions in a bigger model (a part-model, if necessary) made for studying the problem of bed-movement and silt-entry into the canal.

Four additional gauges were fixed at distances $\frac{1}{5}$, $3\frac{1}{5}$, $6\frac{1}{5}$ and $9\frac{1}{5}$ feet from the left bank at the cross-section $\frac{1}{2}$ mile above the weir line and readings were observed at these gauges for different stages of flow. These readings would enable only this $\frac{1}{2}$ mile reach to be incorporated in a bigger part model and flow pattern simulated properly. Current directions were observed within this $\frac{1}{2}$ mile reach by dropping small paper floats above this cross-section at distances, $\frac{1}{5}$, $1\frac{1}{5}$, $2\frac{1}{5}$, $3\frac{1}{5}$, $4\frac{1}{5}$, $5\frac{1}{5}$, $6\frac{1}{5}$, $7\frac{1}{5}$, $8\frac{1}{5}$ and $9\frac{1}{5}$ feet, from the left bank noting the points at which they cross the weir line. These current lines were then drawn in plan for different stages of flow. Current directions were observed and drawn in the following cases [see Figures 10C-3, 10C-4, 10C-5 and 10C-6] :—

(1) With discharges 0.35 cusec in the model (equivalent to 50,000 cusecs in the prototype) and undersluice gate opening equal to 0 inch, $\frac{1}{2}$ inch, $\frac{3}{4}$ inch, $\frac{7}{8}$ inch and 1.0 inch, respectively.

(2) With discharge 0.70 cusecs in the model (equivalent to 100,000 cusecs in the prototype) and undersluice gate opening equal to 0 inch, $\frac{1}{2}$ inch, $\frac{3}{4}$ inch, 1.0 inch and $1\frac{1}{4}$ inch, respectively.

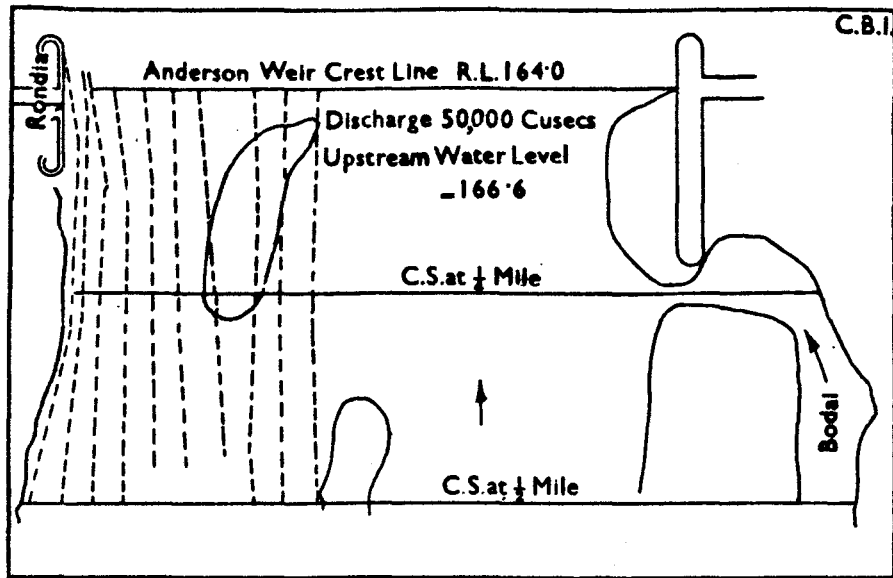


Figure IOC.3:- Showing Current Direction of Damodar

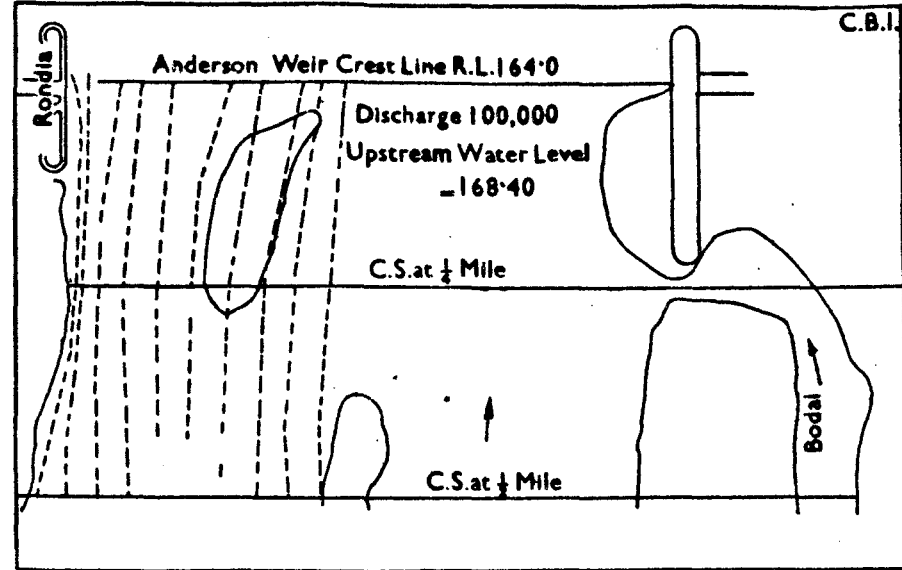


Figure IOC.4:- Showing Current Direction of Damodar

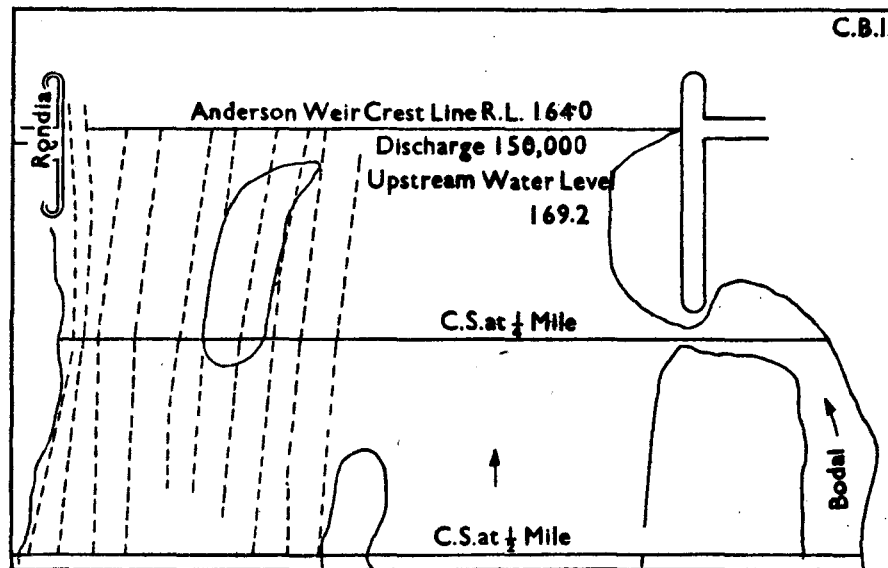


Figure IOC.5:- Showing Current Direction of Damodar

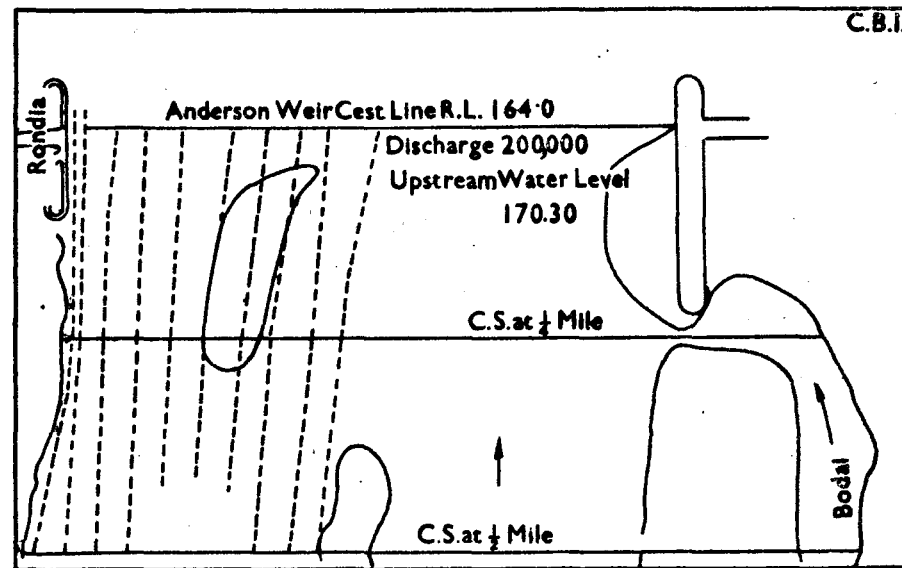


Figure IOC.6:- Showing Current Direction of Damodar

(3) With discharge 1.05 cusecs in the model (equivalent to 150,000 cusecs in the prototype) and under sluice gate opening equal to 0 inch, $\frac{1}{4}$ inch, $\frac{1}{2}$ inch, $\frac{3}{4}$ inch, 1.0 inch and $1\frac{1}{4}$ inches, respectively.

(4) With discharge 1.40 cusecs in the model (corresponding to 200,000 cusecs in the prototype) and under sluice gate opening equal to 0 inch, $\frac{1}{4}$ inch, $\frac{1}{2}$ inch, $\frac{3}{4}$ inch, 1.0 inch, $1\frac{1}{4}$ inches, respectively.

At the beginning of November 1946 the canal water became clear and when this water was passed over the model, it was possible to see clearly whether bed movement occurred or not. It was found that bed silt did not move in the reach within one mile above the left undersluice when the river discharge was equivalent to 100,000 cusecs in the prototype. Practically no silt was found to enter into the canal through the head-regulator. A finer sand of mean diameter 0.23 mm. was dropped at the $\frac{1}{2}$ mile cross-section upstream of the undersluice in the main river channel but even this sand was not found to move along the bed when a steady discharge of 0.7 cusecs (equivalent to 100,000 cusecs in the prototype) was passed over the model. A flood cycle rising from 30,000 cusecs to 200,000 cusecs within 15 hours and falling back to 30,000 cusecs within 45 hours in the prototype was next passed over the model with a time-scale ratio of 1/22 (whole cycle taking about three hours in the model) again and again for four times, but this also could not produce any sand movement; sand entering the canal during this period being practically negligible.

Next idea was to see in this model at what stage of steady discharge sand (mean diameter 0.33 mm.) begins to move in the bed of the model. When a discharge equivalent to 250,000 cusecs in the prototype was passed over the model, the bed movement within this reach was found to start. With discharge equivalent to 300,000 cusecs in the prototype appreciable bed movement was observed within this reach. The surface velocity observed at this stage within this reach was 1.25 feet per second.

Tractive force for these two stages of flow within this reach of this model was calculated and is given below:—

Tractive force = Depth of water \times Surface slope \times Weight of one cubic foot of water.

For discharge equivalent to 250,000 cusecs Tractive force = 0.0072 lb

For discharge equivalent to 300,000 cusecs Tractive force = 0.0098 lb

It seemed, therefore, that unless the value of the tractive force went above 0.007 lb. bed movement within this reach would not start in this model for this particular type of sand (mean diameter = 0.33 mm.). It was, therefore, decided to distort the model further by changing the vertical scale ratio to 1 : 50. With vertical scale ratio 1 : 50, and assuming similar flow in the changed model also (that is, surface slope and depth of water in the model proportionately increased), the tractive force works out as follows:—

For discharge equivalent to 100,000 cusecs, Tractive force = 0.007 lb.

For discharge equivalent to 200,000 cusecs, Tractive force = 0.01 lb.

For discharge equivalent to 300,000 cusecs, Tractive force = 0.025 lb.

It was expected, therefore, that with the vertical scale ratio changed to 1 : 50, bed movement within this reach would start with a discharge even lower than 100,000 cusecs in the prototype.

The model was reconstructed with horizontal and vertical scale ratios 1/200 and 1/50, respectively. The models of the weir, undersluice and head regulator, etc. were prepared in the same scale and inserted properly. The whole work took about a month and the reconstructed model was ready for run on December 21, 1946. The water-supply from the Damodar Canal was again stopped on December 24, 1946 and was resumed on January 22, 1947. It has not yet been possible to do much work on the changed model and the work is in progress.

It has been found in this model that movement within this reach starts at discharges corresponding to 50,000 cusecs in the prototype and the tractive force for this stage of flow within this reach works out to 0.0076 lb. This has been calculated from the gauge readings observed in the model. It is known from observations on the river at Rhondia that sand movement on the bed starts for discharges between 30 to 50,000 cusecs (see Table 10C.1).

Work on this model is now in progress and it is hoped that some indication will be available from the results, as to the best means of reducing silt entry into the Damodar Canal at Rhondia Headworks.

(2) ANALYSIS OF OBSERVATIONS ON SILT EXCLUSION AT THE HEADWORKS OF DAMODAR CANAL AT RHONDIA⁽²⁾

There had been a report of excessive silting in the head reaches of the Damodar Canal. This canal takes off the Damodar river at Rhondia upstream of the Anderson Weir. Figure 10C.1 shows the river, the weir, the undersluice and the canal. There are three gates of the undersluice each 60 feet wide and five gates of the canal regulator. The gates of the undersluice close from above and sit on the floor of the pocket while the gates of the canal regulator rise from the sill—there being a breast wall at a height of 4.5 feet above the sill. At present the practice is to operate both the canal and undersluice gate uniformly.

These experiments were designed to study the effect of varying the openings of different undersluice and head regulator gates on the suspended silt in the canal. Observations of silt contents in the river and in the canal were taken along with discharge in the canal, first with uniform gate openings and then by varying the openings. The silt intensity in the river water was observed just upstream of the weir and the observation in the canal was taken near about the first mile from the headworks. A consolidated statement of the silt data is given in Table 10C.1.

In these observations the gates were (1) uniformly opened and silt samples collected in the pocket and the canal, (2) the undersluice gate openings were increased from the left to the right hand bays and the canal regulator gates were raised more and more from the upstream end.

⁽²⁾ River Research Institute Bengal, Annual Report, 1946 pages 85 and 86.

TABLE 10C.1

Statement of suspended silt content and discharge in the Damodar River
and Damodar Canal

Date 1948	River discharge	Canal discharge	Upstream gate opening	Canal silt contents in grams per litre	River silt contents in grams per litre	Expected river silt in grams per litre	Canal Silt
							$\frac{S_c}{R}$
August							
19 ..	53,587	1,709	Uniform ..	0.7475	0.4275	0.3056	2.45
20 ..	35,382	1,696	Do. ..	0.4680	0.2630	0.1732	2.64
21 ..	27,743	1,896	Do. ..	0.3855	0.1430	0.1239	3.11
22 ..	21,706	1,890	Do. ..	0.2210	0.0965	0.0885	2.50
23 ..	22,652	1,917	Do. ..	0.2110	0.1455	0.0994	2.12
26 ..	40,746	1,800	Do. ..	0.3660	0.3775	0.2095	1.75
27 ..	26,184	1,802	Do. ..	0.1920	0.1210	0.1144	1.68
28 ..	18,835	1,800	Do. ..	0.1505	0.4200	0.0734	2.05
29 ..	57,368	1,800	Do. ..	0.5715	0.4245	0.2885	1.98
30 ..	76,903	1,803	Do. ..	0.7070	0.7290	0.5009	1.41
31 ..	63,176	1,807	Do. ..	0.5240	0.4025	0.3918	1.34
September							
1 ..	141,119	1,501	Do. ..	0.8795	1.2480	1.1503	0.76
2 ..	149,963	1,808	Do. ..	0.5140	0.4310	1.2486	0.41
3 ..	92,663	1,811	Do. ..	0.4695	0.4340	0.6466	0.72
4 ..	63,547	1,503	Do. ..	0.3085	0.2735	0.3858	0.79
5 ..	46,997	1,504	Do. ..	0.2725	0.1545	0.2554	1.06
6 ..	40,670	1,551	Do. ..	0.2425	0.1220	0.2096	1.15
7 ..	54,934	1,505	Do. ..	0.4260	0.1935	0.3162	1.34
9 ..	40,661	1,699	Different ..	0.3380	0.1585	0.2101	1.60
10 ..	26,744	1,692	Do. ..	0.3125	0.1135	0.1182	2.64
11 ..	21,312	1,712	Do. ..	0.2200	0.1125	0.0866	2.54
12 ..	21,724	1,712	Do. ..	0.2855	0.1010	0.0873	3.91
13 ..	31,288	1,699	Do. ..	0.3835	0.2080	0.1467	2.61
14 ..	34,321	1,704	Do. ..	0.4980	0.2525	0.0943	5.28
16 ..	18,346	1,700	Do. ..	0.2640	0.0875	0.0706	3.74
17 ..	105,675	393	Uniform ..	0.6435	0.8040	0.7736	0.83
21 ..	57,339	499	Do. ..	0.4280	0.1560	0.3352	1.28
23 ..	29,804	1,504	Different ..	0.3450	0.0765	0.1370	2.52
24 ..	23,819	1,505	Do. ..	0.2850	0.0685	0.1009	2.82
26 ..	17,450	1,505	Uniform ..	0.1105	0.0290	0.0706	1.57
27 ..	13,549	1,497	Do. ..	0.0740	0.0220	0.0470	1.57
28 ..	11,809	1,517	Do. ..	0.0835	0.0155	0.0385	2.17
30 ..	8,510	1,461	Do. ..	0.0510	0.0590	0.0247	2.06
October							
1 ..	7,045	1,461	Do. ..	0.0815	0.0240	0.0191	4.27
7 ..	13,571	1,519	Do. ..	0.1195	0.0380	0.0467	2.56
8 ..	22,251	1,856	Different ..	0.2200	0.0915	0.0919	2.39
9 ..	15,796	1,856	Uniform ..	0.1085	0.0255	0.0575	1.89
10 ..	11,811	1,519	Do. ..	0.0835	0.0320	0.0386	2.16
12 ..	29,604	1,856	Different ..	0.4655	0.2555	0.1361	3.42
14 ..	15,796	1,856	Uniform ..	0.0965	0.0325	0.0575	1.68
15 ..	11,811	1,519	Do. ..	0.0635	0.0190	0.0386	1.65
16 ..	8,500	1,457	Do. ..	0.0600	0.1700	0.0246	2.44
17 ..	6,939	1,417	Do. ..	0.0540	0.0105	0.0187	2.89
18 ..	6,939	1,417	Do. ..	0.0475	0.0050	0.0187	2.54
19 ..	11,800	1,519	Do. ..	0.0940	0.0335	0.0386	2.46
21 ..	111,134	1,785	Different ..	0.9985	0.7395	0.8285	1.21
22 ..	62,189	1,809	Do. ..	0.5870	0.3350	0.3746	1.57
23 ..	39,555	1,997	Do. ..	0.3945	0.1340	0.2018	1.95

It is seen from the Table 10C.1 that the river silt content is almost invariably less than the canal silt content which is contrary to expectation. This is most probably due to defective water sampling in the river in as much as the sampling had to be done at a point where the flow is obstructed and consequently the coarser suspended particles drop down. It was not possible to get more representative samples from any other place in the river. Assuming that the error in the river silt content is a constant multiplying factor the ratio of the canal silt to river silt has been subjected to analysis for uniform and varying gate openings.

The observations were taken when the canal discharges were roughly of the same order, but as would be found from the Table 10C.1 the river discharge varied within a wide range for different observations. As the river discharge directly affects the suspended silt content, the effect of these varying river discharges has been taken into account by making the river silt content to be a function of the river discharge. The two factors have been correlated logarithmically. The correlation coefficient is 85 per cent. and the logarithmic regression line is—

$$Si_R = 1.04 \times 10^{-7} \times Q_R^{1.38}$$

where Si_R is the silt content and Q_R the discharge in cusecs in the river. The expected river silt content has been worked out from this formula and in the subsequent analysis instead of taking the actual river silt content the expected value of the river silt content has been used.

The ratio between the observed canal silt content and the expected river silt as given by the above formula has been studied. Of course, the constant multiplying factor remains in this ratio also but this factor, being a constant quantity, has been omitted in the following analysis without affecting the results. The values of this ratio (canal silt/river silt) have been divided into two samples according as the gate-openings were uniform or varying. It is found that on an average the ratio is higher for varying gate-openings than for uniform openings. The means of the ratio in the two samples are (the suffix 1 refers to uniform openings and the suffix 2 refers to different openings):

$$\bar{x}_1 = 1.91 \text{ and } \bar{x}_2 = 2.69.$$

The significance of the difference between them has been tested by Fisher's *t*-test. The value of *t* is

$$t = \frac{\bar{x}_1 - \bar{x}_2}{\sqrt{\frac{(n_1-1)s_1^2 + (n_2-1)s_2^2}{n_1 + n_2 - 2}}} = \sqrt{\frac{1}{n_1} + \frac{1}{n_2}} = 2.879$$

and the number of degrees of freedom is $v = n_1 + n_2 - 2 = 45$.

The probability of this value of *t* being exceeded in a random sampling is less than one per cent. The difference is therefore highly significant.

The above leads to the belief that instead of more silt being excluded by the varying gate-openings more silt actually enters the canal through the headworks by this operation and probably this method is not suitable for silt exclusion.

An interesting feature to be noted is that the ratio worked out above (S_{iO}/S_{iR}) is a decreasing function of the river discharge, that is, the ratio decreases as the discharge increases. Figure 10C. 7 shows the ratio plotted against

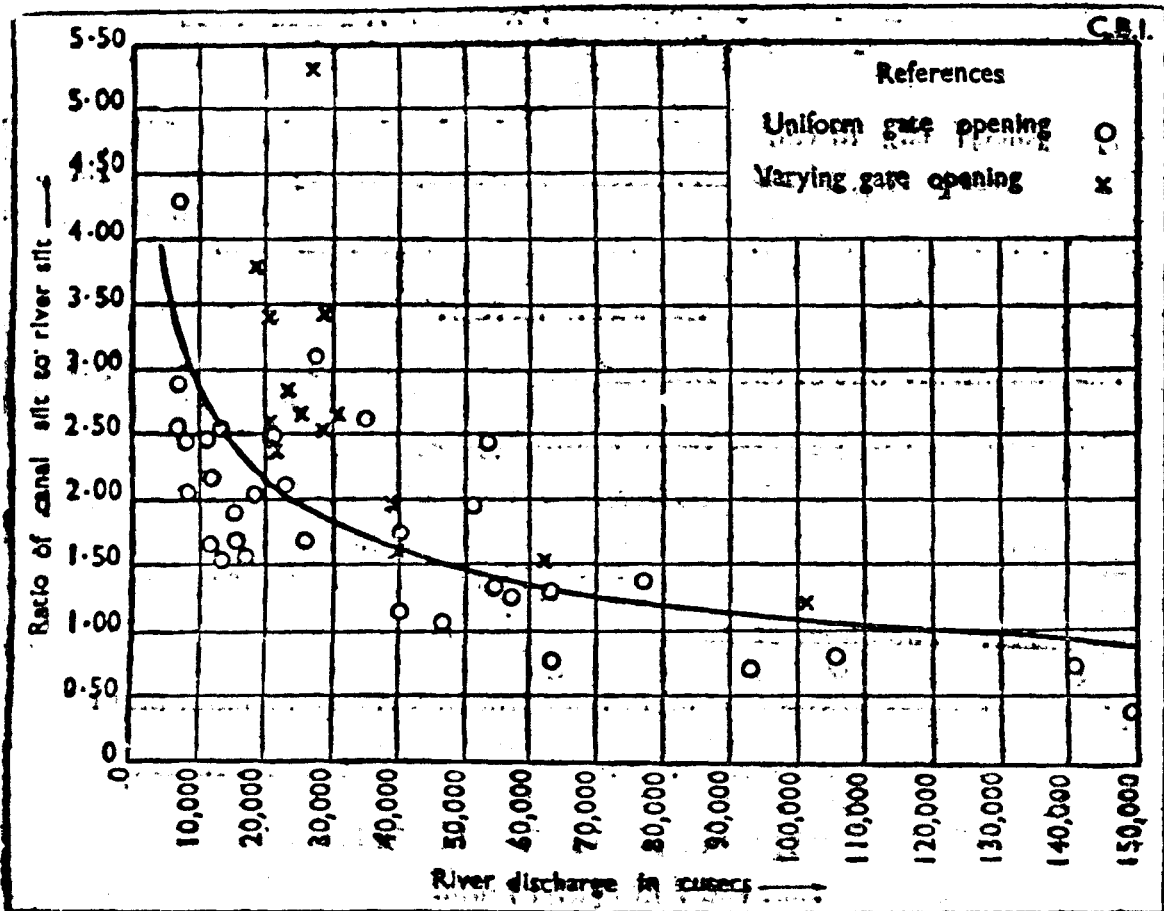


Figure 10C.7 : Relation between the weir discharge and the rate of river silt content for the Damodar river and Damodar canal at Rhondia

the river discharge. The points marked by cross refer to varying gate-openings and those marked by circles refer to uniform openings. The logarithmic correlation coefficient is negative and of the order of 66 per cent. and the logarithmic regression line is given by

$$\frac{S_{iO}}{S_{iR}} = 124.5 \times Q^{-0.41}$$

This has been shown in the figure.

These observations will be continued next year also.

(3) SILT EXCLUSION WORKS AT LLOYD BARRAGE AT SUKKUR

by

T. R. Sethna*Executive Engineer, Irrigation and Research Division, Sind.***BRIEF HISTORY**

The construction of the Lloyd Barrage, the head regulators of the canals and the greater part of the canal systems was completed by the end of 1931.

Canals were opened for wetting and testing of the banks shortly afterwards and irrigation from the canals commenced in May 1932.

GENERAL LAYOUT

The general lay out of the Barrage and the canals and the relative position of the Barrage canals, head regulators, Sadhbelo Island, Bukkur Island, the gorge *etc.* will readily be seen from Figure 10C.8.

CURVATURE OF FLOW

It will be noticed, from Figure 10C.8, that the river from the gorge to the Barrage is slightly curved in plan and that the concave side of the curve is on the left Bank. The effect of this curvature is that surface water tends to flow towards the left bank and bed water, from the lower stratum of the river which contains much more heavy silt, flows over towards the right bank and it is for this reason that the right bank canals have given considerable trouble since the commencement of irrigation by drawing an excessive silt charge.

METHOD OF REGULATION 'OPEN FLOW'

Figure 10C.8 shows that there are two divide-walls which shut off a portion of the river adjacent to the canal head regulators on each bank. The spaces so shut off are called 'pockets' and the discharge passing through these pockets can be controlled by the Barrage gates situated within them. In the original project for the Barrage it was laid down that the system of control of the water at the Barrage should be what is called 'open flow' *i.e.* the Barrage gates situated within the pockets were intended to be kept opened as far as possible and as big a discharge as would be available from the river was to be run through each of the pockets, from which the canals would be fed.

CHANGES IN DESIGN

'*Still Pond*'.—During the construction of the project, it was realized that it was not right to design for one system of regulation only and steps were accordingly taken to modify certain features of the design in such a way as to permit of the operation of the pockets on the 'still pond' system. In the 'still pond' system of control the water taken into the pocket is only sufficient for the requirements of the canals plus a very small quantity allowed to pass under the undersluice or pocket gates to prevent the gates becoming stuck by silt being deposited against them.

It is, of course possible to regulate at the Barrage by any method lying between the extremes of 'still pond' and 'open flow' by varying the opening of the Barrage gates in the respective pockets.

HEAVY SILT DRAWN IN 1932

During the first irrigation season in 1932 the Right Bank Pocket was worked semi-open flow and silt was drawn into the canals in large quantities. This silt draw became apparent towards the end of July 1932, and the system of regulation on the right bank was immediately changed to 'still pond' conditions.

Any scouring operation which is now carried out to free the pocket of the deposited silt is done with the right bank canals closed.

EXPERIENCE WITH WORKING OF BARRAGE

Silting in right bank canals.—Experience in the working of the Barrage has shown that the working of the left bank Pocket by 'still pond' or 'semi-open flow' makes little difference to the conditions as regards silt entry into the canals. On the right bank however, it has been found from the beginning that silt has entered the North Western and Rice Canals during the *abkalani* months in spite of the fact that the right bank pocket has always been worked 'still pond' since July 1932.

In the North Western Canal the silt drawn in to the canal was deposited in the head reach in the earlier years and during the successive *rabi* seasons, when clean water is taken into the canals, the silt deposit has been moved on and spread out, each successive intake of silt thus resulting in a longer length of the canal being covered with silt and a rise in the general depth of the silt.

In 1937 *abkalani* it was found that the full supply discharge of the North Western canal could not be taken into the channel at the proper full supply gauge and in order to get full supply in canal it was necessary to raise the pond level in the river above the Barrage. The reason for this reduction in discharge was that the silt drawn into the canal had reached the Ruk Cross Regulator and had accumulated to such a depth that the gradient from Ruk to Sukkur had become flattened with a consequent reduction in the discharge which could pass down the channel.

At Ruk Cross Regulator there are two offtaking channels—the Warah Branch which is designed to draw 1951 cusecs, and a small minor. The silt draw of the Warah Branch was entirely disproportionate and in 1937, it was found to have silted badly.

In 1938 *abkalani* very great difficulty was experienced with the Warah Branch and it was necessary to help irrigation on this system extensively from the Rice Canal.

The same process had occurred in the Rice Canal but the movement of silt down the channel had not occurred to the same extent as was the case in the North Western because the Rice Canal was being closed for the *Rabi* season. However, it was found during the 1939 *abkalani* season that for lower discharges, the discharge of the Rice Canal for a given gauge was about 800 cusecs less than it was in 1938 showing it was obvious that, if the silt draw of the Rice Canal was allowed to go on, it would be necessary to remove silt from the bed of the Rice Canal since a reduction in discharge would be a very serious matter. A discharge of 800 cusecs is equivalent to some 35,000 acres of rice.

The Dadu Canal, which is situated nearest the Barrage, draws relatively less silt than the other two canals, and the first 16 miles of the channel made use of the old Sukkur Canal which was much deeper than the present Dadu Canal is designed to be.

Trouble was not experienced in regard to reduction in discharge in the Dadu Canal because, presumably, the silt which was going into this channel had merely been filling up the surplus section in the old Sukkur Canal below designed bed.

EXPERIMENTS ON CONTROL OF SILT

It has already been stated that it was early realized that the Right Bank canals would silt and experiments were carried out at the Hydrodynamic Research Station in Poona by Sir C. C. Inglis in order to evolve measures for the control of such silt. Sir C. C. Inglis proposed a curved vane situated above the present right bank pocket which was to operate by deflecting bed water from the entrance of the pocket. Sir C. C. Inglis advocated the construction of a similar vane on the left bank of the river also because he considered that the existence of the right bank vane only might adversely affect the conditions on the left bank. Figure 10C.8 shows the approximate position of these two vanes in the river.

In the *Kharif* of 1933 conditions regarding silt entry into the canals on the right bank apparently improved and it was decided by the Chief Engineer that these vanes should not be constructed and that the canals should be run for a time to see what conditions would develop.

By 1937, it had become abundantly clear that the silting of the right bank canal was serious and would become still more serious with the passage of time and the Director, Central Irrigation and Hydrodynamic Research Station Poona, was accordingly requested to carry out model experiments and suggest a remedy.

The Director made further experiments with the vanes mentioned in the previous paragraphs but decided that conditions had changed so much in the river that the vanes would not be effective.

Other methods for the control of the silt were, therefore, tried and the experiments were continued in 1937 and 1938. The proposals made are shown in Figure 10C.9. They consist very briefly of a stone guide bank about 5,000 feet long built from pier No. 23 towards the upstream, the bank being suitably curved in plan. The remainder of the work consists in extending the right bank pocket in an upstream direction and curving away from the river bank. The whole object of this lay out is to produce correct curvature of flow so that surface water will be directed into the new approach channel and pocket and conditions thereby set up which will exclude bed water from the approach channel and from the new entrance to the pocket.

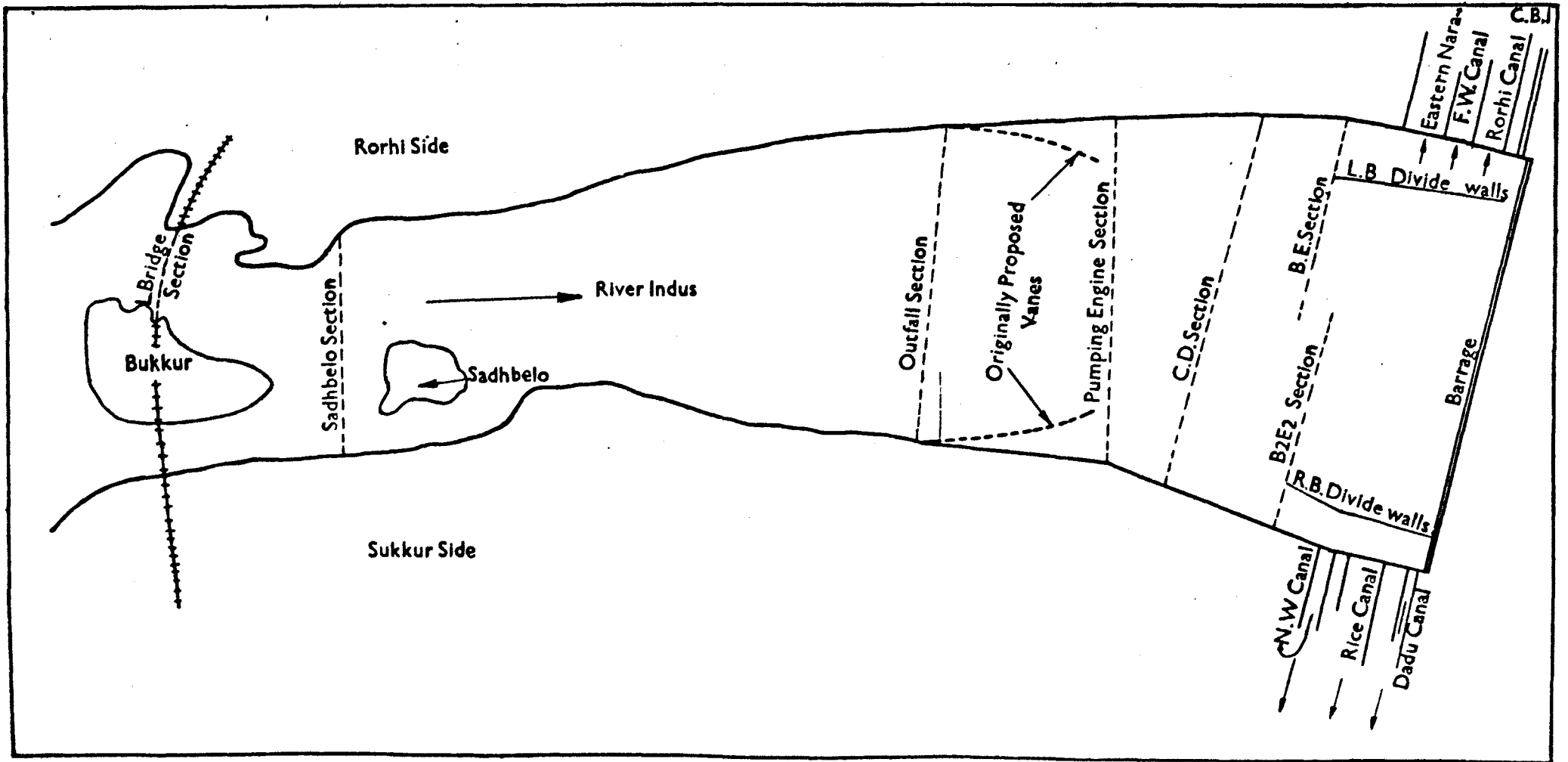
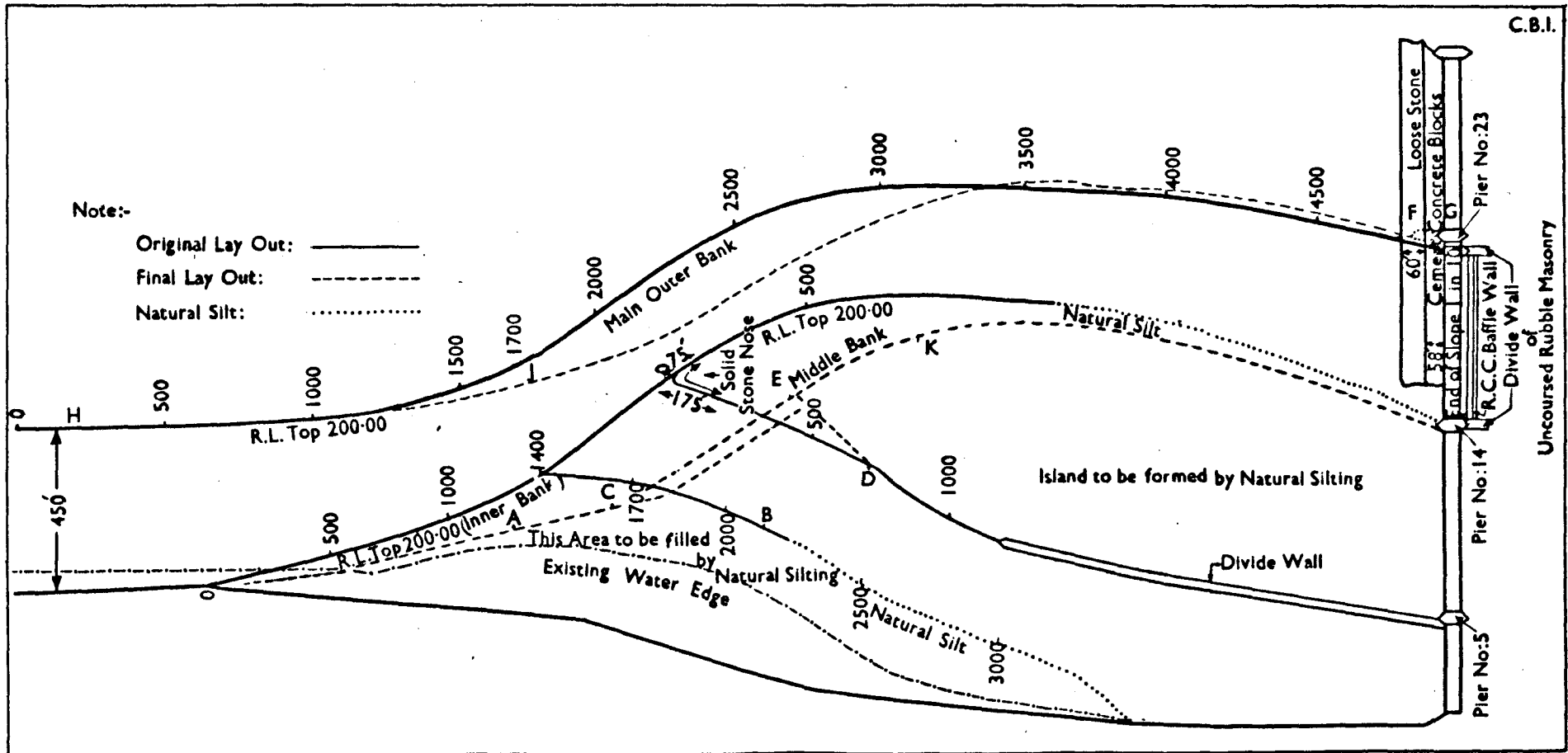


Figure 10C.8:- Showing Right and Left Bank Vanes as Originally Proposed
[River Indus at Sukkur]



The proposals involved entirely putting out of use nine of the Barrage gates (Nos. 6 to 14) and the use of gates (Nos. 15 to 23) as undersluice gates for the new approach channel. It will not be possible to pass more than 65,000 cusecs into the new approach channel without setting up dangerous scour conditions in the approach channel itself and the closing of eighteen of the Barrage gates in this way might be a serious matter if it should ever happen that the maximum flood of $1\frac{1}{2}$ million cusecs, for which the Barrage was originally designed, should ever occur.

It is seen from the past records that the maximum recorded discharge at Sukkur is less than $7\frac{1}{2}$ lakhs cusecs and a provision for a discharge of nine lakhs cusecs as the maximum to be passed has now been considered reasonable.

The discharge per foot width of the river opening with nine lakhs cusecs as maximum discharge works out to 348 cusecs.

It was fortunate that one of the Dredgers used for the construction of the Barrage had been retained at Sukkur together with certain pile driving plant, a tug, barges etc. The methods of construction proposed for the various parts of the new work had to be based chiefly on the possibilities of carrying out the work with the existing plant.

COST OF WORK

Figure 10C.9 shows the lay out as originally proposed in the dotted line and in full line the final lay out to which the works were constructed. In the original lay-out portions were to be done with vertical faces using steel piling and portion FG was to be of loose concrete blocks. The remaining portion was to be of stone pitched banks covering dredged material inside. The cost of the work was nearly 21 lakhs of rupees.

In addition to the heavy cost there was difficulty of getting sheet piling to provide the vertical faces.

The Director, Indian Waterways Experiment Station was again requested to see if he could make further experiments with pitched banks similar to those suggested for the outer bank as that would decrease the cost. He was also requested whether the dredging and constructing the apron to the slope to which the scour was expected to develop could be done away with and aprons constructed on the existing bed of the river and allowed to launch themselves as the scour developed.

The Director made necessary experiments and suggested a lay-out as shown in full lines in Figure 10C.9. The cost of the work according to revised lay-out worked out to roughly $10\frac{1}{2}$ lakhs of rupees. In this only the raised sill was to be of vertical sheet piling.

PROGRESS OF WORK

Work was started in the beginning of January 1940. As all the work could not be completed in one season, portion HJ i.e. first 1,700 feet of the outer bank was completed before May 1940. The remaining outer bank, whole of inner bank, middle bank, the raised crest downstream deflector, side walls between piers No. 14 to 23 and the baffle were all started in October 1940 and completed by May 1941.

Table 10C.2 gives the year and page No. of the Central Board of Irrigation Annual Report (Technical) with brief details of experiments done by the Director Indian Waterways Experiment Station, Poona, which may be referred to for further details.

The work has been completed at less than 8.25 lakhs of rupees.

TABLE 10C.2

Statement giving reference to publication where results of experiments are given.
Central Board of Irrigation Publication—Annual Reports (Technical)

Pub- lica- tion No.	Year	Pages	Brief details of experiments
22	1938-39	37—38	It gives the history of the silting of Right Bank canals since the Sukkur Barrage first came in operation in 1931 and describes the conclusions drawn from experiments carried out with vertically exaggerated models during 1930-33.
24	1939-40	10—13	Specific experiments on silt control in Right Bank approach channel were carried out for :— (a) Probable scour along the banks of the approach channel. (b) Optimum design of approach channel and of offtake to the Right pocket for silt exclusion. (c) Scour round the nose of outer bank under various conditions of flow. (d) Protection of Apron downstream of approach channel sluices. (e) Discharges available for scouring the Right Bank approach channel at Sukkur. (f) Alternatives for constructing the approach in two working seasons. With the note are shown various alternative designs for Right Bank Approach channel to exclude silt from Right Bank canals.
27	1941	11—12	It deals with the Specific experiments carried out for :— (a) Scour at nose of outer bank of the new approach-channel at Sukkur in prototype and model. (b) Alignment of inner bank of the tail channel and the junction of the outer bank with the Barrage ($\frac{1}{20}$ and $\frac{1}{10}$ vertically exaggerated model). (c) Tail channel scouring and regulation. (d) Protection of Island nose by laying a stone apron along its face.
29	1942	9—10 and 46	Gives the final note on experiments on new approach channel at Sukkur to control sand entering the Right Bank Canals describing the various alternatives designed to reduce the quantity of sand as also improving curvature of flow in the approach channel along inner right bank and the tests made there for on page 46. Figure 2 shows plan of Indus above Sukkur Barrage with the new approach channel designed to control the quantity of silt entering Right Bank Canals.
33	1944	8—9	Describes the improvements of the Rice Canal due to Right Bank approach channel by illustrating on a longitudinal section the condition of silt before and after the constriction of approach channel.

METHOD OF WORKING THE APPROACH CHANNEL

For working the new approach channel efficiently, the Director had said that a ratio of discharge of tail channel to pocket, should be a minimum of $1\frac{1}{2} : 1$ and if it was increased, it could work better for silt exclusion. The Director, however, when he visited Sukkur after the *abkalani* of 1943 said that the ratio should be kept as 1.5 to 1.0 and not increased as it was not advisable to bring in more silt and then redistribute it between the pocket and the tail channel. The designed discharge of the canals on the right bank is 18,000 cusecs. Taking $1\frac{1}{2}$ times the discharge through tail channel (*i.e.* 27,000 cusecs) and total discharge that should enter the approach channel works out to 45,000 cusecs.

The Director also suggested that the discharge entering the approach channel, should not be increased quickly or fluctuated as that would disturb the bed and bring up more silt in suspension. If the canal discharge was less than 18,000 the remaining discharge should be bye-passed first through span No. 5. If the discharge be more, then span No. 4 could be opened and if still further discharge had to be passed, then span No. 3 could also be opened, but spans 1 and 2 should always be kept closed.

During cold weather when the river discharge goes down below 1,35,000, only 1/3rd the river discharge is taken through the approach channel and redistributed in the ratio of 1.5 to 1.0 so long as enough discharge is available but when the discharge goes down considerably, the ratio cannot be stuck to. When the discharge is less there is very little silt in movement and need for silt exclusion does not exist.

The instructions of the Director are being carefully followed and so far as the working of the canals is concerned, the new approach channel has been found very useful.

Longitudinal sections showing the bed levels of the right bank canals observed immediately after the *abkalani* season of 1939, 1941, 1943, 1945 and 1946 revealed that considerable amount of silt has been scoured out from the canals and the working of canals has considerably improved.

However, when the river is low and the Barrage fully ponded up, the gradient between the extreme nose of the outer bank and the Barrage is flatter on the river side, than on the approach channel and pocket side, with the result that higher pond level has to be maintained at such times.

Also when the river is in floods, the discharge passing per foot width from spans No. 24 to 59 is much more than what passes from spans No. 15 to 22 with the result that heavy silt bars are formed downstream of piers No. 15 to 22 and the tail channel is unable to carry the required discharge. To enable the tail channel to function properly, the main Barrage gates from spans No. 24 to 59 have to be kept slightly submerged in water so as to artificially create a cut off (afflux) of from 0.2 to 0.5 of a foot.

The above drawbacks are not much when compared to the improvement caused to the working of the canals.

(4) DESIGN OF LOWER SIND BARRAGE AT KOTRI FOR EFFECTIVE SAND EXCLUSION FROM CANALS (3)

WHY WAS IT CONSIDERED NECESSARY TO SHIFT THE BARRAGE UPSTREAM OF KOTRI ?

As the question of heading up water above the natural river levels—the main factor responsible for accretion upstream of a barrage—transcends in importance all other considerations in the selection of the optimum site for the Barrage in Sind, it was strongly pressed that the Lower Sind Barrage should be sited upstream of Kotri town instead of at Hajipur site B, 23 miles downstream of it, the site which had been previously selected. After due consideration this recommendation has been at last accepted.

Figure 10C.10 shows the site of the barrage, tentatively proposed by the Project Division, $\frac{1}{2}$ mile below the head of the existing Fuleli Canal at Jamshoro.

The following advantages have been claimed for this site :—

- (a) Concave curvature, favourable to sand exclusion by natural means, exists on both banks of the river with a natural island in between, which will enable and ensure, when armoured, complete control over the barrage.
- (b) The course of the river, upstream and downstream of the site, has been stable for all the period for which surveys are available.
- (c) The existing head of the Fuleli Head Regulator can be possibly used with a little remodelling.
- (d) The river downstream does not fan out, as in the case of the Sukkur Barrage, but, on the contrary, narrows considerably ; hence there would be less tendency for accretion downstream ; and so less tendency for accretion upstream.

MODEL SCALE RATIOS

SCALE RATIOS OF LOWER SIND BARRAGE MODEL

Length scale.—It was decided to adopt a length scale of 1/250 for the model with a view :—

- (a) to accommodating the model in the area available ;
- (b) to getting the maximum discharge of the prototype within the range of seven to ten cusecs in the model—so far found, from experience, to be the best workable range, yielding optimum results in the model in respect of bed movement *etc.* within a reasonable period.

Discharge scale.—As this is model of a barrage, with curvature characteristics as a predominant factor—

$$Q \text{ scale} = L^2 ; \text{ as } M_L \propto \sqrt{Q},$$

where M_L = meander length.

∴ One cusec in the model = 62,500 cusecs in the river.

The maximum discharge of the river Indus at Kotri for the year 1943 was 5,30,000 cusecs.

∴ The maximum discharge in the model worked out to 8.48 cusecs—*i.e.* within the optimum range.

The bankful discharge is estimated to be 2,50,000 cusecs = 4.0 cusecs in the model.

(3) Indian Waterways Experiment Station, Poona, Annual Report (Technical) 1946, pages 98 to 106.

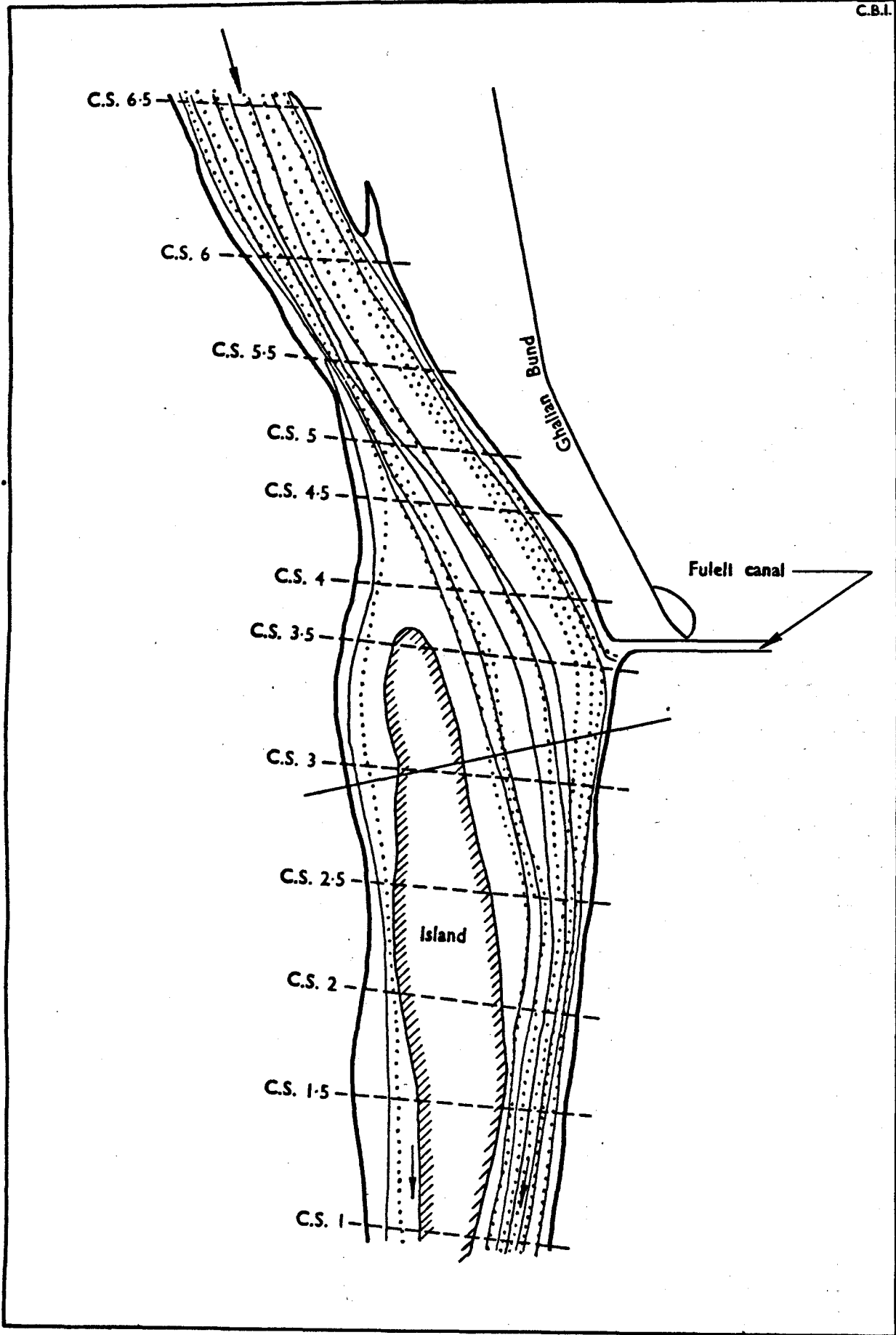


Figure 10C.10:- Kotri Barrage Model showing Lines of Flow for $Q \approx 200,000$ Cusecs [Without Barrage]

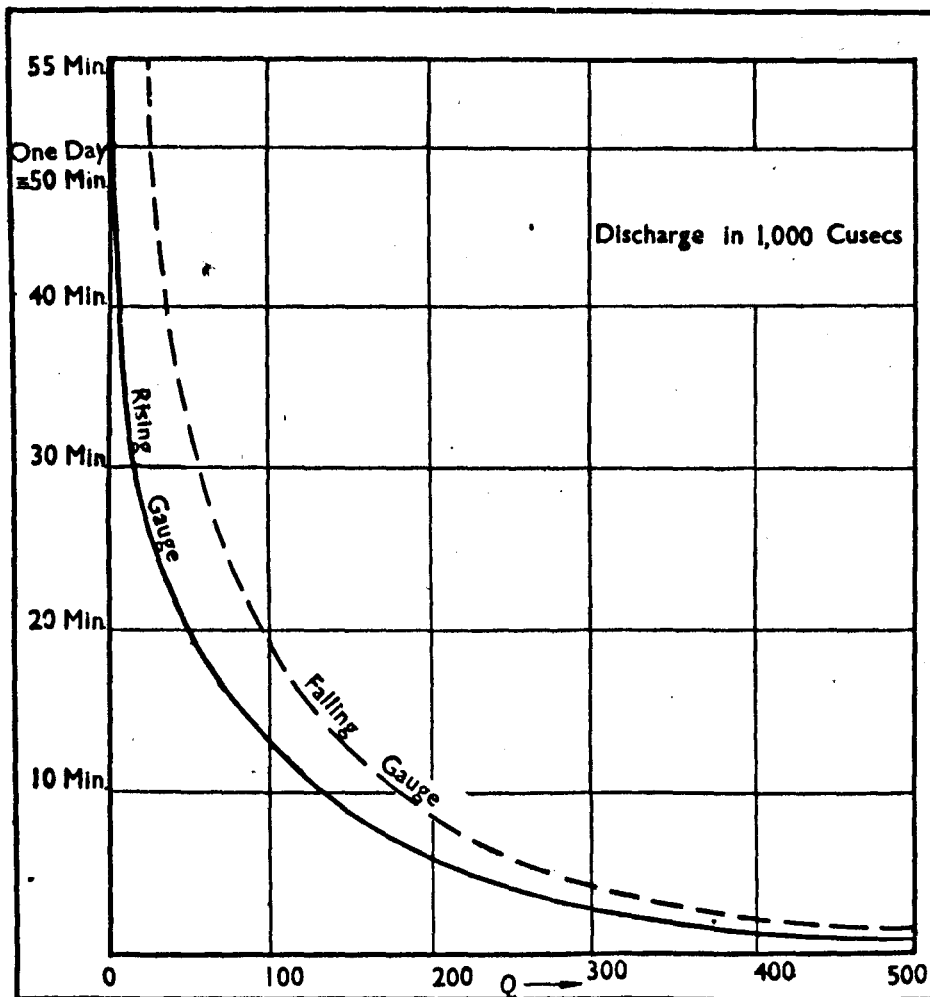


Figure 10 C.11:-Showing Relation between Discharge & Time in $[\frac{1}{250} : \frac{1}{50}]$ Lower Sind Barrage Model]

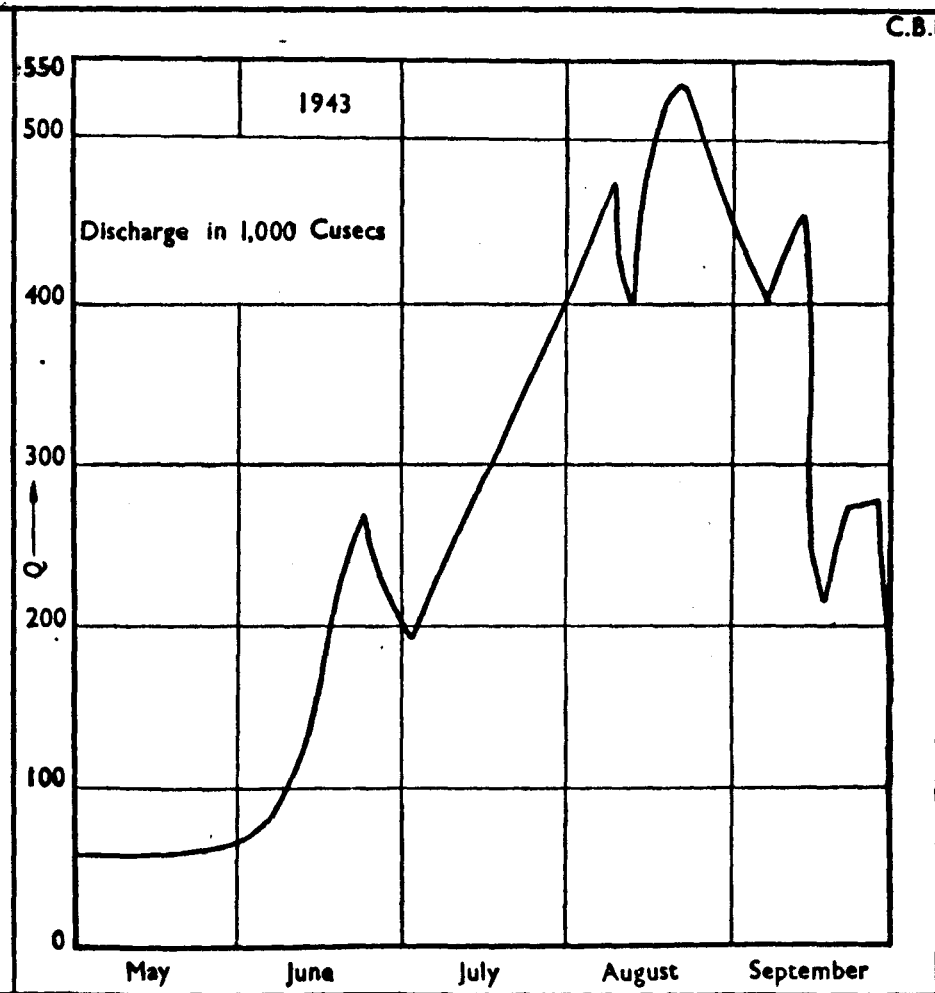


Figure 10 C.12:- Showing Relation between Discharge & Time at Kotri 17 Miles Upstream of Proposed Barrage Site 'B'

It was observed that 1/62,500 discharge scale simulated conditions satisfactorily ; hence this scale was adopted.

Depth scale.—(a) According to Manning's formula for regime conditions—

$$D=L^3=250^3=63, \text{ i.e. } V. E.=4.0 \text{ (where } L=\text{Horizontal scale and } D=\text{Depth scale).}$$

(b) Lacey's formula (for no difference in sand)—

$$D=L^3=250^3=40 ; \therefore V. E.=6.25.$$

The model was initially laid in the Sangam sand with $m=0.18$ m m., i.e. same as the prototype grade. It has been now observed that finer particles have been washed out and coarser sand with $m=0.30$ mm. has remained in the model.

Hence allowing for difference in sand we have according to Lacey :—

$$L_p/L_m=(m_p/m_m)^{3/2} (d_p/d_m)^{3/2}$$

$$\therefore L_p/L_m=(0.18/0.30)^{3/2} (d_p/d_m)^{3/2}.$$

$\therefore d_p/d_m=45$ against 40 with same grade of material.

$\therefore V. E.=250/45=5.5$ against 6.25 with same grade of material.

Slope scale.—According to Lacey :—

$$(a) S \propto f^{3/2}/Q^{3/2}.$$

$$\therefore S. E.=\frac{(m_m/m_p)^{3/2} (L_p/L_m)^{3/2}}{(V. E.)^3}.$$

$$=6.33, \text{ with } V. E.=6.25 \text{ and } m_m=m_p=0.18 \text{ mm.}$$

$$(b) S. E.=\frac{(250)^{3/2} (0.30/0.18)^{3/2}}{(V. E.)^3}.$$

$$=6.33, \text{ with } V. E.=5.5; m_m=0.30 \text{ mm. and } m_p=0.18 \text{ mm.}$$

Thus due, to coarsening of bed material, while the $V. E.$ (calculated) decreases from 6.25 to 5.5, the $S. E.$ (slope exaggeration) increases from 6.33 to 8.30 ; that is, the tilt has to be increased, which is as must be expected with coarser material.

When the Kotri barrage model was run without barrage, it was observed that the slope exaggeration varied from 7.5 to 8.5 for low discharge of 3.2 cusecs \equiv 2,00,000 cusecs to high discharge of 8.48 cusecs \equiv 5,30,000 cusecs, and the vertical exaggeration varied between 5.25 and 5.95. Thus, in this particular case, the actual scale ratios nearly agreed with those calculated by the Lacey formulas.

Time scale for the Lower Sind Barrage model.—The time scale was worked out according to tractive force formula :—

$$\frac{T_p}{T_m} = \frac{L_p}{L_m} \cdot \left(\frac{d_p}{d_m}\right)^{3/2} \cdot \frac{S_p}{S_m}.$$

For the Lower Sind Barrage—

$$L_p/L_m=250 ; d_p/d_m=50 \text{ and } S_p/S_m=1/5$$

$$\frac{T_p}{T_m} = \frac{250 \times (50)^{3/2}}{5} = 350$$

i.e. one second in model \equiv 350 seconds in the prototype

i.e. four minutes in the model \equiv one day in prototype

This has been assumed for the dominant discharge of $Q=250,000$ cusecs

As low river discharges have got to be run very much longer than high ones due to the narrow range of model velocities and its limitations—

one day \equiv 8 minutes has been assumed for $Q \equiv 150,000$ cusecs.

one day \equiv one minute for $Q \equiv 500,000$ cusecs.

and a smooth asymptotic curve Figure 10C.11, is drawn through these three points for the time scale of other discharges.

Silt charge scale.—Data of silt charge of the river Indus at Kotri, for the year 1943, according to which a rising and a falling cycle, Figure 10C.12, is run in the model, have been analysed and show charge in parts per 1,000 for various discharges in the river as given in Table 10C.3.

TABLE 10C.3

Discharge (cusecs)	Charge in parts per 1,000 in the river	Charge in parts per 1,000 injected in the model
200,000	2.30	0.115
250,000	2.90	0.145
300,000	4.80	0.190
350,000	3.00	0.200
400,000	4.30	0.215
450,000	4.75	0.238
500,000	4.75	0.238

It is assumed that 90 per cent. of the charge in the river is in suspension and only ten per cent. moves along the bed.

As bed sand injected in the model is coarser and there is relatively less bed movement in the model due to narrow range of velocities, only half of the natural bed charge of the river was injected in the model as shown above. The model proved that this charge was satisfactory.

EXPERIMENTS WITHOUT THE BARRAGE

Four complete rising and falling cycles were run till the model was stabilised. The model showed considerable erosion along the right bank between Cross Section No. 5.5 to 7 which indicated either incorrect survey in this reach or the existence of hard, inerodible banks, which would need to be simulated on the model for correct reproduction of prototype conditions.

On inspecting the site it was observed that the right bank in this reach is made up of inerodible clay which practically precludes erosion—as the model had shown. In view of this, the experiments had to be repeated after holding the banks in the model.

The following data were observed during the experiments :—

(i) Lines of flow were observed for $Q \equiv 2, 3, 4$ and 5 lakhs cusecs, of

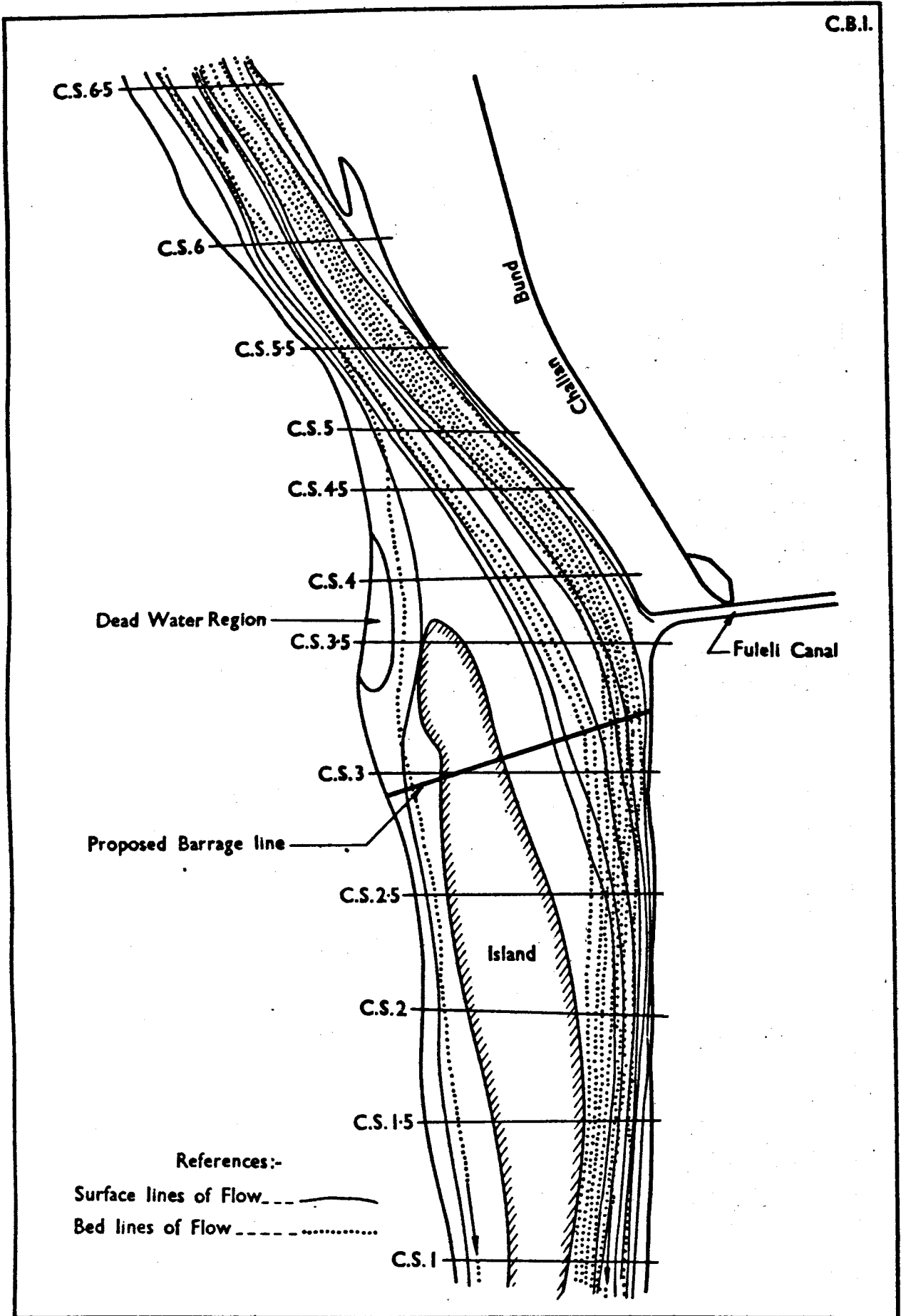


Figure 10 C.13:- Showing Kotri Barrage Model, Surface and Bed Lines of Flow with $Q=6.4$ Cusecs $\approx 400,000$ Cusecs. [without Barrage]

which Figures 10C.10 and 10C.13 show the lines of flow for $Q=2,00,000$ and $4,00,000$ cusecs.

- (ii) Figures 10C.14 (a, b, c and d) show isovels for 2, 3, 4 and 5 lakh cusecs at the proposed barrage site.
- (iii) Figure 10C.15 is a graph showing the percentage of the total river discharge carried by the right channel, as observed in the model and the river.

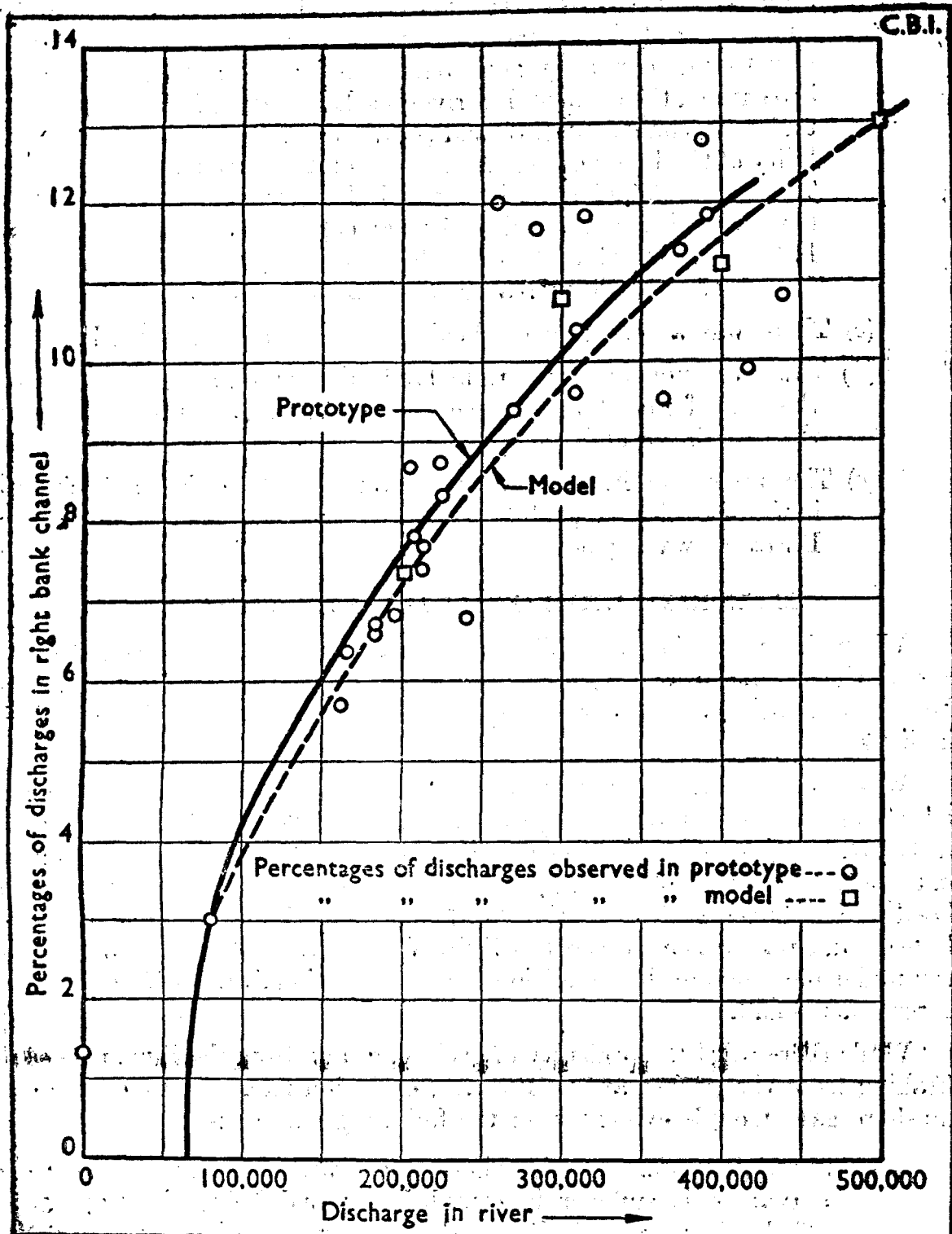


Figure 10C.15. Showing percentage of discharge in the Right Bank Channels

These data indicate the following :—

- (a) The lines of flow showed that the proposed barrage line was some what upstream of the reach where the curvature effect has become fully established.
- (b) Bed seeds—used in the model as one way of tracing the bed flow—dropped at Cross-section No. 7 did not enter the Fuleli canal for any of the discharges between 2 to 5 lakh cusecs with Fuleli draw-off alternatively at (i) 12,000 cusecs—*i.e.* the present maximum discharge—, and (ii) 19,500 cusecs, the maximum discharge under the post-barrage conditions : but some seeds dropped at Cross-section No. 5·5 entered the canal for low river discharges (2 lakhs cusecs and less) under the conditions of (i) and (ii) above, thus showing the unsuitability of the Fuleli offtake from point of its position along the curve. The entry of bed sand into the Fuleli appreciably increased, at low river discharges, when 19,500 cusecs was passed down the Fuleli, showing that the difficulties of efficient exclusion would be considerable with increased post-barrage draw-off.
- (c) There was a tendency of shoaling along the right bank.
- (d) Water did not follow the right bank, but struck against the nose of the island for discharges above, 2,00,000 cusecs, and then bifurcated, thus coming round the island.
- (e) The agreement in the discharge drawn by the right bank channel in the model and prototype—under existing conditions without Barrage—was good.

EXPERIMENTS WITH BARRAGE AT THE PROPOSED SITE

When the first rising and falling cycle was run, it was observed that the flow at the barrage was too oblique, resulting in severe attack downstream of the barrage, indicating the necessity for very heavy protection all along the bank.

When the discharge = 200,000 cusecs (the minimum discharge at which bed movement occurs in the river) was run in the model—with upstream pond level at R. L. 68·0 (maximum) the downstream water level 61·80 (maximum)—*i.e.* the worst conditions for testing sand exclusion from the canals—80 per cent. of bed seeds dropped at Cross-section No. 7 entered the Fuleli Canal. Considerable bed sand was drawn by the Fuleli Canal, even at higher discharges, *i.e.* even under more favourable conditions when the ratio of draw-off to river discharge was small.

While there was little difficulty of excluding sand from the Pinyari Feeder, offtaking from the pocket at the Barrage, exclusion from the Fuleli head at Jamshoro gave trouble evidently for the following reasons :—

- (a) The Fuleli offtake at Jamshoro is not within the reach where the curvature is sufficiently established ; and being the uppermost offtake draws practically all the bed-sand entering the pocket ;
- (b) this insufficient curvature effect is further reduced by the barrage ;

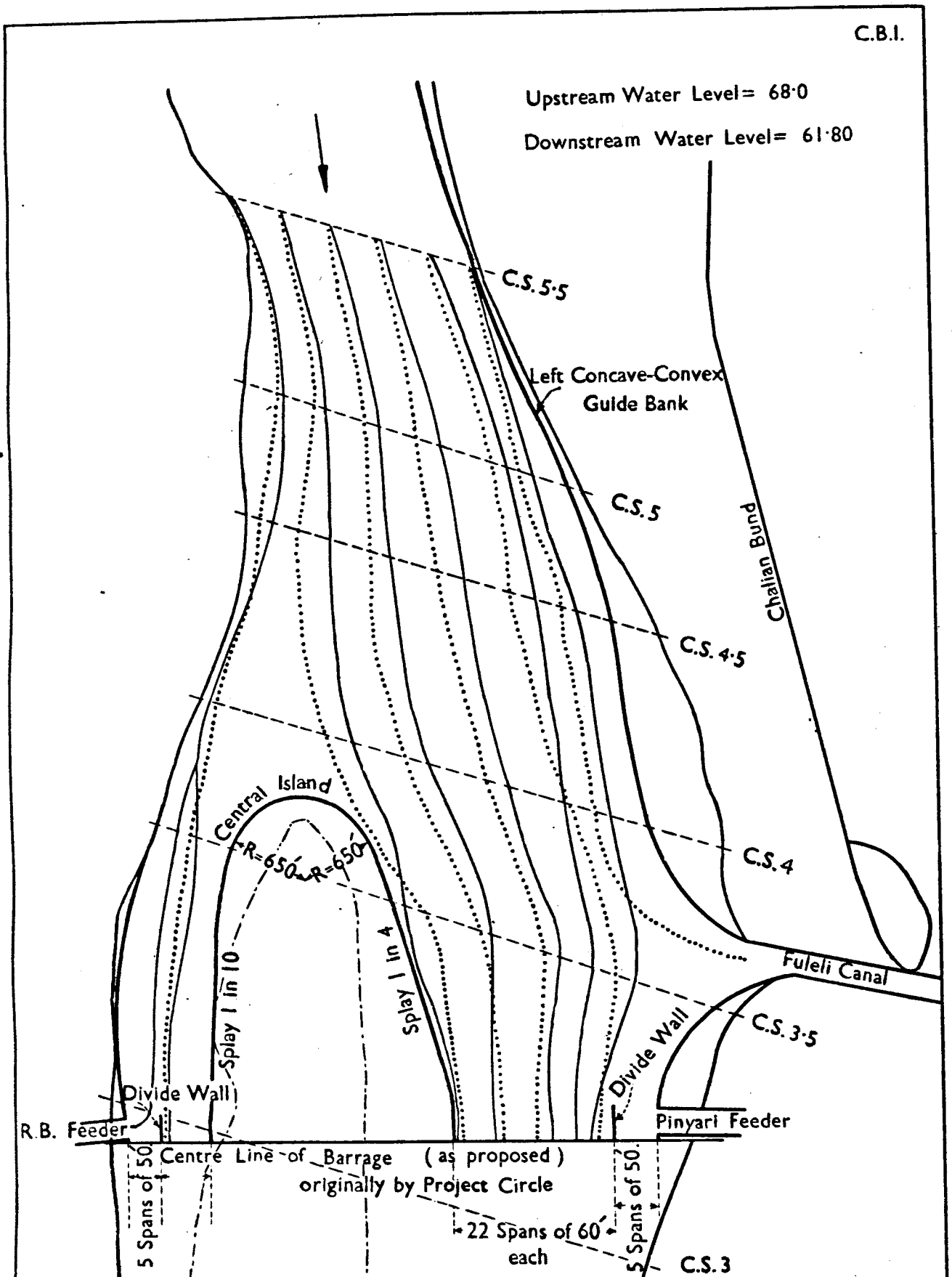


Figure 10C.16 :- Showing Lines of Flow for $Q \approx 200,000$ Cusecs with Concave-Convex Left Guide Bank. [Kotri Barrage Model]

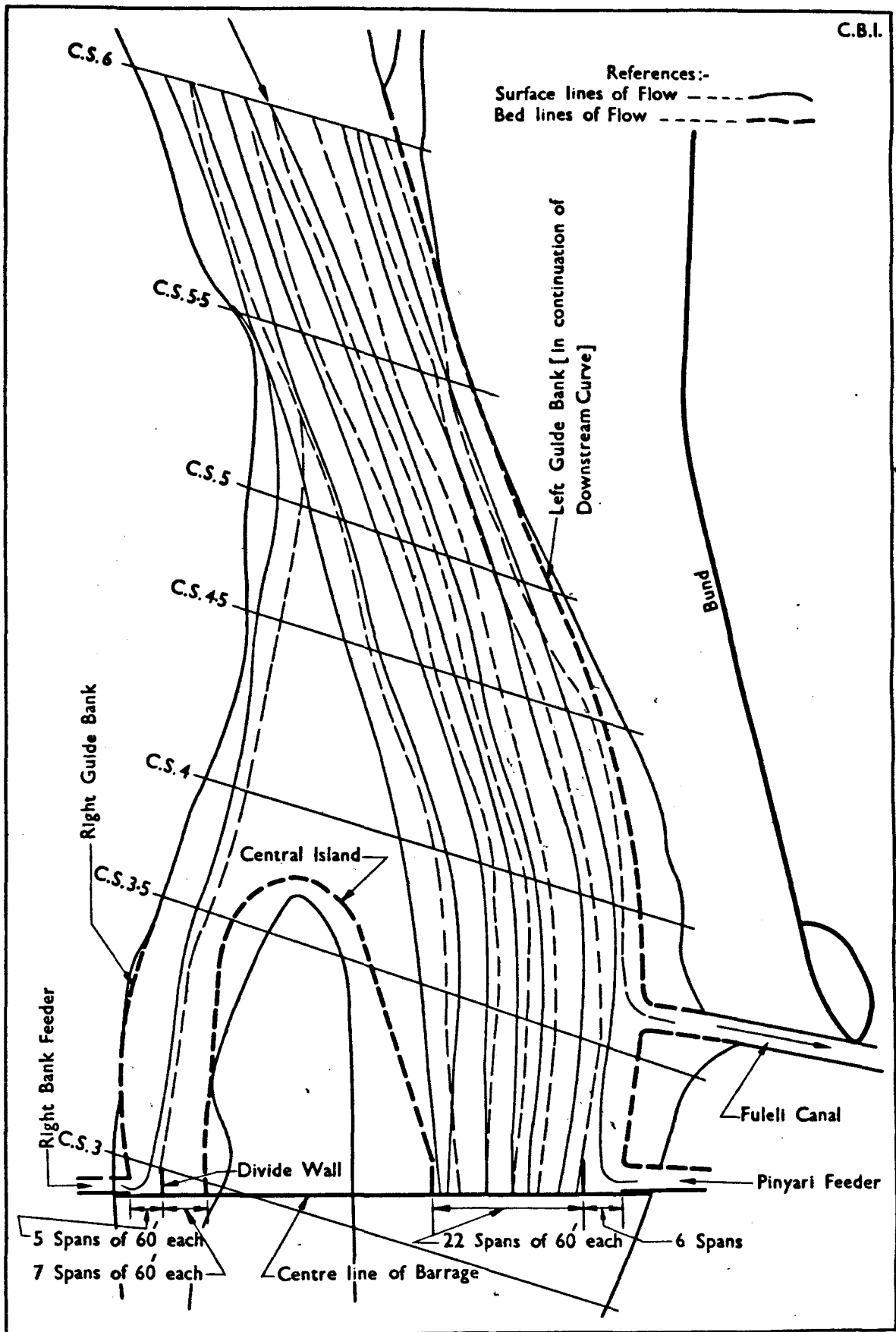


Figure 10C.17 :- Showing Lines of Flow for $Q \approx 200,000$ and Left Guide Bank in continuation of Downstream Curve (Kotri Barrage Model)

- (c) the relatively excessive draw-off of the left bank canals at low river discharges is accentuated by 75 : 25 per cent. distribution of discharges in the left and right arms respectively ; and
- (d) attraction of flow by the island results in reduced flow along the left bank.

MEASURES TESTED FOR IMPROVING SAND EXCLUSION OUT OF FULELI

In order to restore and increase the curvature reduced by the barrage, a suitably aligned guide bank appeared the first necessity.

In order to get sufficient curvature established upstream of the offtake a concave followed by convex curvature was imposed on the left bank by a guide bank so shaped upstream of the canal—see Figure 10C.16 in which lines of flow with this guide bank are shown. Though apparently promising, this measure was completely unfruitful : while its upper concave portion helped to quicken, to a slight extent, the slack and feeble flow along the left bank and keep the bed-sand away from the bank, the sand was pulled back into the canal, partly by the convex curvature at the entry at Jamshoro, but mainly by the considerable draw-off of the Fuleli.

It was then considered that a wholly concave alignment of the guide bank, in line with the curvature of the barrage, so that the whole made one curve Figure 10C.17 would eliminate at least the avoidable of the two drawbacks. Though partially successful, and better than Figure 10C.16 exclusion was still unsatisfactory, indicating that an artificial increase of local curvature would not in itself be sufficient.

A raised crest at the entrance of the approach channel to the Fuleli, Figure 10C.18, with its top at 2/3rd below the pond level, was next tested in the hope that only mid-depth and top water would be taken into the canal, thus excluding bed-sand.

The bed material, however, rolled up the sill into the canal, being thrown into suspension by the turbulence created by the raised crest.

The raised crest appeared to do more harm than good.

As the effect of the existing curvature, even when assisted by the additional curvature imposed by the guide-bank, was still insufficient for exclusion of sand from the Fuleli offtake at Jamshoro, the effects of different shapes and sizes of the central island on curvature were tested :—

- (a) The splay of the island in the left arm was changed from 1 in 4 to 1 in 10 with a view to making it symmetrical and pulling the main flow towards the left bank, thus boosting up velocities along that bank. But this did not remove the trouble from the Fuleli.
- (b) The left arm of the island was still further brought in (towards the left bank), this time by imposing convex curvature on it, [see Figure 10C.18 island (b)] ; but this also failed to bring about any appreciable improvement in the Fuleli.
- (c) In order to set back the “ pull ” of the island till after the Fuleli takes off, the length of the island was reduced by about 500 feet [see Figure 10C.18 island (c)]. But this had results contrary to expectations. A larger island would not materially, improve ex-

clusion but would cause greater difference of river levels in the two arms and besides be more expensive ; and so was not tried.

This showed that modification of the shape or size of the central island had little beneficial effect on the sand exclusion.

A long divide wall at right angles to the barrage and extending right up to the upstream abutment of the Fuleli Canal at Jamshoro was also tested. It was found ineffective because—

- (i) the ratio V_{river}/V_{pocket} is very small, especially at Jamshoro where the width of the river is excessive ;
- (ii) the divide-wall was too long to cause a sufficiently strong twist of bed flow away from the pocket into the river, by regulation at the barrage ;
- (iii) As the rate of scour is, theoretically, proportional to the (length of pocket, L^3) and the time required for scour the pocket to L^4 —while actual experiments (4) have shown the theoretical approximation of proportionality to L^4 to be on the low side—it would be quite impracticable to scour such an enormous pocket within the period available for scouring operations ; and (iv) not only it did not help the Fuleli but also worsened the exclusion from Pinyari, which had been free from sand-draw in the previous tests.

In order to improve the V_r/V ratio, a divide-wall splayed at 1 in 10 was tried but was also unsuccessful.

As the costs of such a long pocket would be prohibitive, even if it was advantageous, which it is not, this idea was summarily dropped after testing its effects.

At this stage, the possibility of constructing a groyne to attract a fair share of top water towards the left bank was considered. But the unpromising prospects of the groyne for varying river discharges in the post-barrage period; the inability of a model to indicate the effectiveness of such a groyne for many years ahead ; the possible deterioration of the right bank canal which was so far working quite satisfactorily ; and the experience that the more you train a river upstream of the barrage by groynes and pitched islands, the more likely it is that you will lose control upon the barrage in time, resulting in difficulties and conditions much more adverse than before ;—all these considerations ruled out the advisability of depending on such expedients as groynes and pitched islands as an all-time measure.

It has been observed that the most severe conditions for exclusion were obtained with low river discharges running under ' still pond '—the lowest river discharge at which appreciable bed movement occurs at Kotri is known to be 200,000 cusecs—and that at higher discharges even the Fuleli did not show signs of excessive sand-draw. It remained to see, therefore, whether by suitable regulation at the Barrage for the low discharges—200,000 and 250,000 cusecs—sand exclusion from Fuleli was possible. The following four types of regulation were, therefore, tested with results as indicated :—

- (i) Wedge shaped opening of the river gates with maximum opening of gates adjacent to the divide wall (*i.e.* the openings of the gates

(4) " Silt exclusion from canals, still pond *versus* open flow " Bombay Public Works Department, Technical paper No. 45 page 2, para 9.

progressively decreased from the divide-wall outwards) did not yield sufficiently satisfactory results;

- (ii) nine spans just outside the divide wall with \equiv four more gate opening than the remaining river gates equally opened, had little effect.
- (iii) The central nine spans kept with larger gate openings than the rest gave worse exclusion than either (i) or (ii).
- (iv) The end nine spans (*i.e.* nine spans adjacent to the island) opened more than the rest gave the worst results of the four types of regulation tested.

Of all the above regulation tests, (i) gave the best results but was yet not sufficiently satisfactory.

The effects of a concave, submerged vane pointing downstream, located at different points, upstream of the Fuleli canal, with the top of the vane at $\frac{2}{3}$ rd the depth with ultimate pond level (R. L. 68), Figure 10C.18, were also tested.

It was observed that, as the position of the vane was shifted upstream, the length of the vane had to be increased to get satisfactory sand exclusion out of Fuleli. The most promising position for the vane was in the gorge, at 'A', Figure 10C.18, sufficiently upstream to ensure that all the bed sand thrown up into suspension would return to the river bed at a point well above the Fuleli offtake. Its optimum alignment is shown in Figure 10C.18, its length being 1,250 feet, and radius 2,687 feet.

A vane has the following obvious merits :—

- (a) it would be easy to construct ;
- (b) it would be easy to maintain ; and
- (c) there would be very little obstruction to the natural waterway of the river.

A submerged vane is, therefore, in the correct place and at the proper time, very useful excluding device. But, though to all appearances effective for sand exclusion, it may not be desirable in certain cases, as in the present one, where in the balance it is disadvantageous ; for this purpose it is wise to weigh carefully all its not-so-obvious demerits against its obvious merits. The possible demerits are, therefore, listed *in extenso* :—

- (i) The effect of a vane imposing curvature and excluding efficiently depends upon a deep channel occurring at high river discharges and being maintained through the season, along or in the proximity of the bank. If in years of poor supply—especially when the Punjab Projects come into operation—this deep channel does not form or may not be maintained as a result of this vane, the vane would be substantially ineffective.
- (ii) A single vane on the left bank is liable to draw an unfair share of the top water to feed the canal and, under certain favourable circumstances, it may even pull the whole river across to it, necessitating subsequent measures to counteract the adverse conditions it would thus create on the right bank.

- (iii) Unless constructed sufficiently upstream, to preclude the possibility of the bed sediment thrown up in suspension finding its way into the canals, sand may enter the canal because some suspended and saltating sand travels further than theoretically expected. Silt in suspension cannot be satisfactorily simulated in a model; nor, when simulated, does it act identically as suspended silt does in the prototype. Hence, we are forced back to using our judgment and experience.
- (iv) Experiments have shown that there is a possibility of the vane going completely or partially out of action when accretion starts taking place in the river downstream of it. Such accretion is likely to take place rapidly in certain cases, such as in the present case, downstream of the vane between the vane and the Fuleli offtake, due to the persistent slack and feeble flow along the bank. Such accretion—as shown by the model—causes hurdling of the bed sand over the vane instead of the sand following it to mid-stream, and ultimately results in decrease of the efficiency of exclusion caused by the vane.

When accretion was reproduced in the model—to a slightly exaggerated scale, because siltation in a model is relatively slower than in the prototype—the model revealed unpromising ultimate results from a vane because most of the bed seeds hurdling over the vane found their way into the canal, instead of being deflected away from the left bank towards midstream, following the back of the vane, as they did in the experiments without accretion reproduced in the model. Thus, though a vane may salutarily accentuate curvature, the effect may be purely temporary and is undependable as a permanent measure in such a case as this.

- (v) The river stage for which the vane should be designed is another uncertain and difficult factor: a design highly effective for 2,00,000 cusecs is less efficient than another suited to $Q \approx 500,000$ cusecs; and neither of these designs is the optimum for discharges in between the two. Moreover, experiments conducted with a vane showed that it was less effective at high stages.
- (vi) On the downstream side of the vane, an apron protection is required to prevent scour occurring as a result of rotary flow and to allow the bed flow to follow the downstream face of the vane towards midstream. To save cost, a sharp slope is desirable; but a sloping apron not steeper than 1 in 5, with a flat apron extending to a distance of 125 feet all along parallel to the vane, appeared desirable, for preventing scour as well as deflecting bed sand away from the approach channel. This tendency of deflecting sand towards midstream, however, disappeared—
- (vii) when the apron level was changed from R. L. 42.0 to R. L. 47.0, due to the fact that the lower the apron is laid, the less is the turbulence responsible for throwing the bed material into suspension;

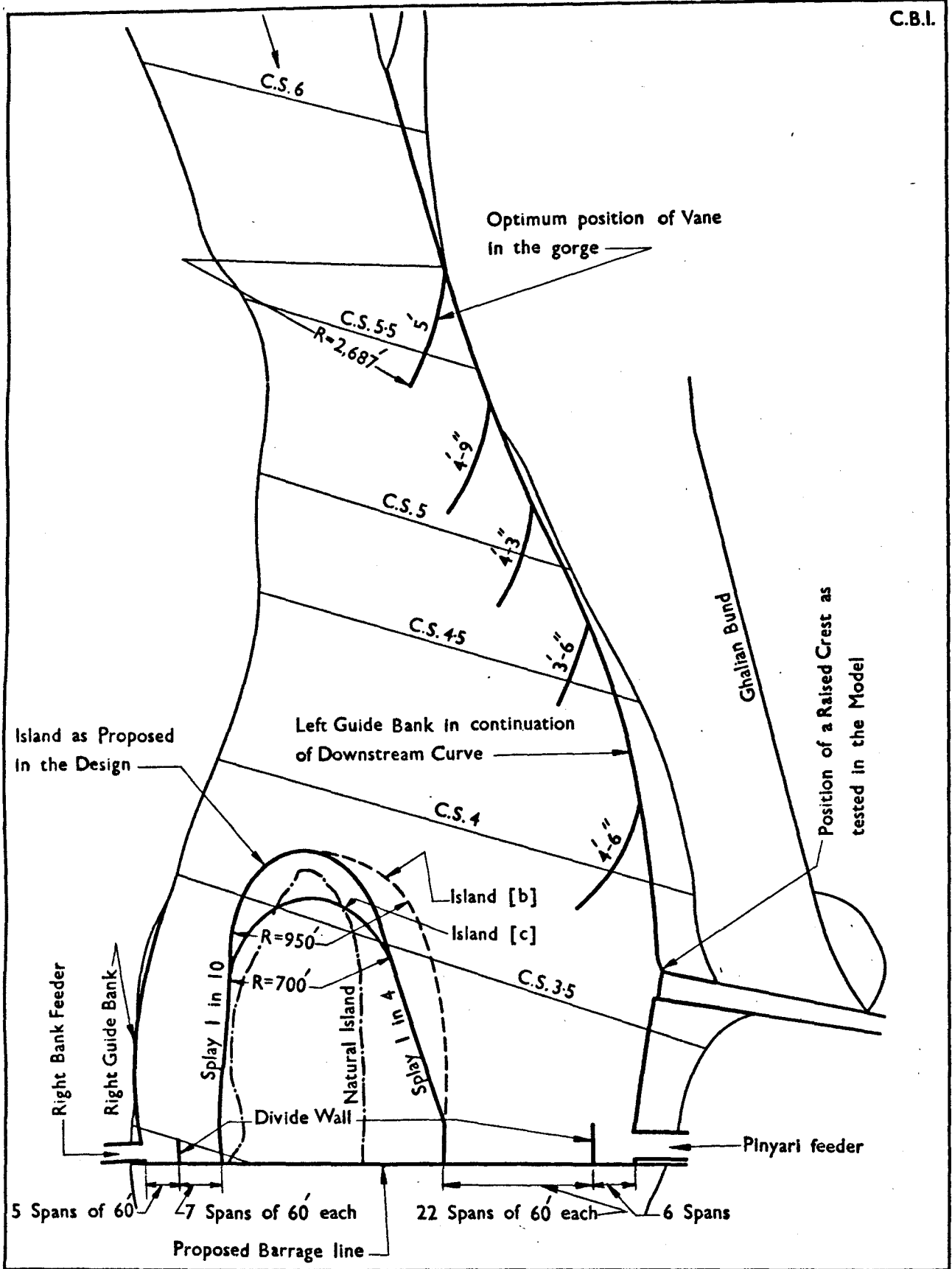


Figure 10 C.18 :- Showing various positions of vane and different shapes of central Island and position of raised sill at the entrance of approach to the Fuleli

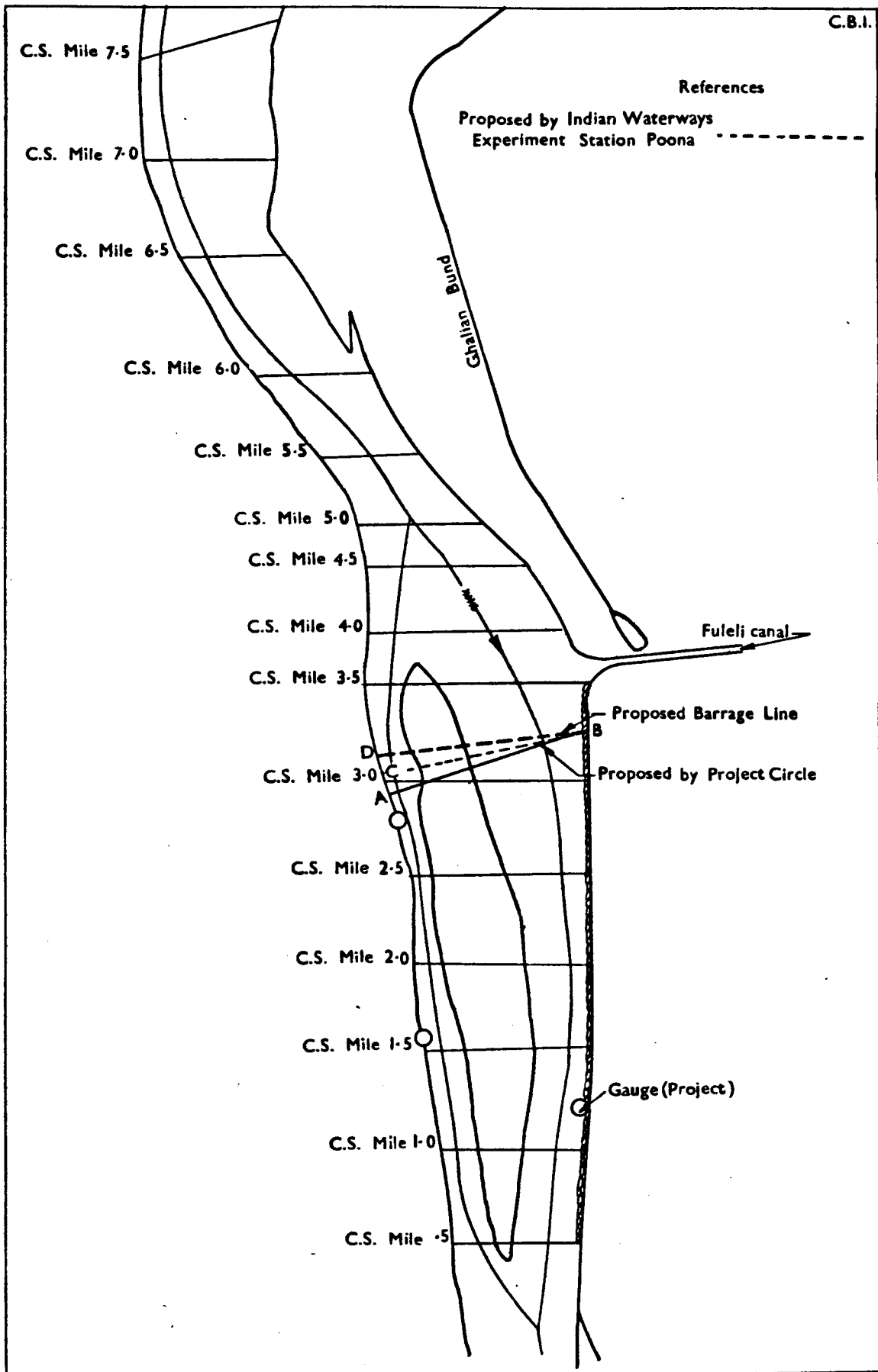


Figure 10C.19 :- Showing Proposed Alignments of [Kotri Barrage Model]

(b) when the width of apron was extended from 125 feet to 175 feet and (c) accretion downstream of the vane up to the barrage was reproduced in the model.

No appreciable scour is observed to occur along the upstream face of the vane ; hence a 20 feet apron would be sufficient there.

Thus, in this particular case, a vane could not be regarded as an all-time satisfactory solution.

WHY IT BECAME NECESSARY TO RECONSIDER THE ALIGNMENT OF THE BARRAGE LINE ?

While the experiments were in full swing, a discrepancy in the plotting of the *Mukam* area (Burial ground) was reported. As encroachment on *Mukam* areas has to be avoided, a change in the alignment of the barrage line was necessary. Accordingly, the Project Division suggested that the Barrage line might be shifted 200 feet upstream, parallel to the previous alignment, and the abutment be moved 600 feet inwards (riverwards)—instead of 300 feet as contemplated in the previous alignment.

The Director, however, proposed an investigation on the site of the feasibility of an alignment between BC and BD, Figure 10C.19, where B, the left abutment, would be taken 500 feet riverwards from the *pucca* edge to obviate the necessity of crossing the *Mukam* area and D about 1,125 feet from the original portion of the right bank abutment. This change was suggested with a view to (a) making the barrage normal to the general direction of flow ; and (b) reducing the length of the island with a consequent reduction in (i) the difference between the pond levels in the left and right arms and (ii) the cost of the island.

The investigations on site showed no obstacles to either of these two alignments or, thus, to any intermediate alignment.

The model experiments were, therefore, repeated with the alignment shown in Figure 10C.20.

RESULTS OF THE EXPERIMENTS WITH THE NEW ALIGNMENT VIDE FIGURE 10C-20

When the different measures were retested with this changed alignment of the barrage, there was no perceptible improvement in the sand exclusion from the Fuleli offtake.

An interesting feature in regard to positioning of the Barrage abutment is worthy of note. The model showed that shifting of the abutment inside the river channel tended to reduce the curvature effect and consequently adversely affected exclusion, probably because it increased the still water area bounded by the divide wall and the high bank. It was possible to counteract this in the following ways :—

- (a) by shifting the guide bank riverwards to restore curvature ;
- (b) by rotating the barrage centre line in a clockwise direction about the left abutment to the full extent possible without affecting the curvature on the right bank, while keeping the direction of current normal to the barrage ; and
- (c) by giving a 1 in 10 splay to the face of the left bank regulators.

When the worst discharge for exclusion, of 2,00,000 cusecs was run in the model, it was observed that the exclusion at Fuleli cftake showed no improvement ; but there was an appreciable reduction in the water levels between the two arms with equal gate openings.

The percentage of river discharges naturally drawn by the right channel, as observed at various stages, with and without barrage, and with downstream levels at the mean level corresponding to the discharge and at a level showing four feet accretion above the mean level, are given in Table 10C. 4.

TABLE 10C. 4

Serial No.	Total discharge in the river (cusecs)	Percentage in the right arm			Remarks
		Without barrage	With barrage (mean level downstream)	With barrage (four feet accretion downstream)	
1	200,000	7.35	23.0	19.50	} With no difference of levels in the two arms
2	300,000	10.80	21.0	16.0	
3	400,000	11.20	18.50	14.0	
4	500,000	13.00	17	13.50	

[Note.—Sand exclusion was satisfactory from Pinyari and Right Bank Feeder in all cases].

It will be observed that, without forcing flow by regulation, the percentage discharges in the right arm decreased with accreted downstream water level, or with increase in the river discharge ; under natural conditions there was no difference of levels in the two arms ; and sand exclusion was satisfactory except from the Fuleli canal.

However, forcing the same discharge under accreted conditions as obtained with mean level downstream, *i.e.* at 200,000 cusecs forcing 23 per cent. *etc.*, resulted in six inches difference in water levels between the two arms, at all the discharge stages. It, moreover, improved exclusion in the right arm.

NECESSITY OF SHIFTING THE FULELI HEAD INTO THE POCKET

All the observations so far made had shown that the Fuleli Head at Jamshoro, located remote from the immediate control of the barrage and at a point along the curve where the curve had not sufficiently established itself, was unsatisfactory for sand-exclusion.

Accordingly, the design was modified to bring the Fuleli Head Regulator into the pocket alongside the Pinyari Head Regulator Figure 10C.20 ; at the same time the arrangement of spans as well as their crest levels were altered to simplify the design. Table 10C.5 sets out the salient features of the proposed and the revised designs, the latter as suggested.

A few quick tests were made with this revised design before recommending it.

MERITS OF THE SIMPLIFIED DESIGN

The following are the main advantages arising out of the proposed changes in design :—

(a) the design is uniform and simple with consequent advantages in construction, maintenance and operation ;

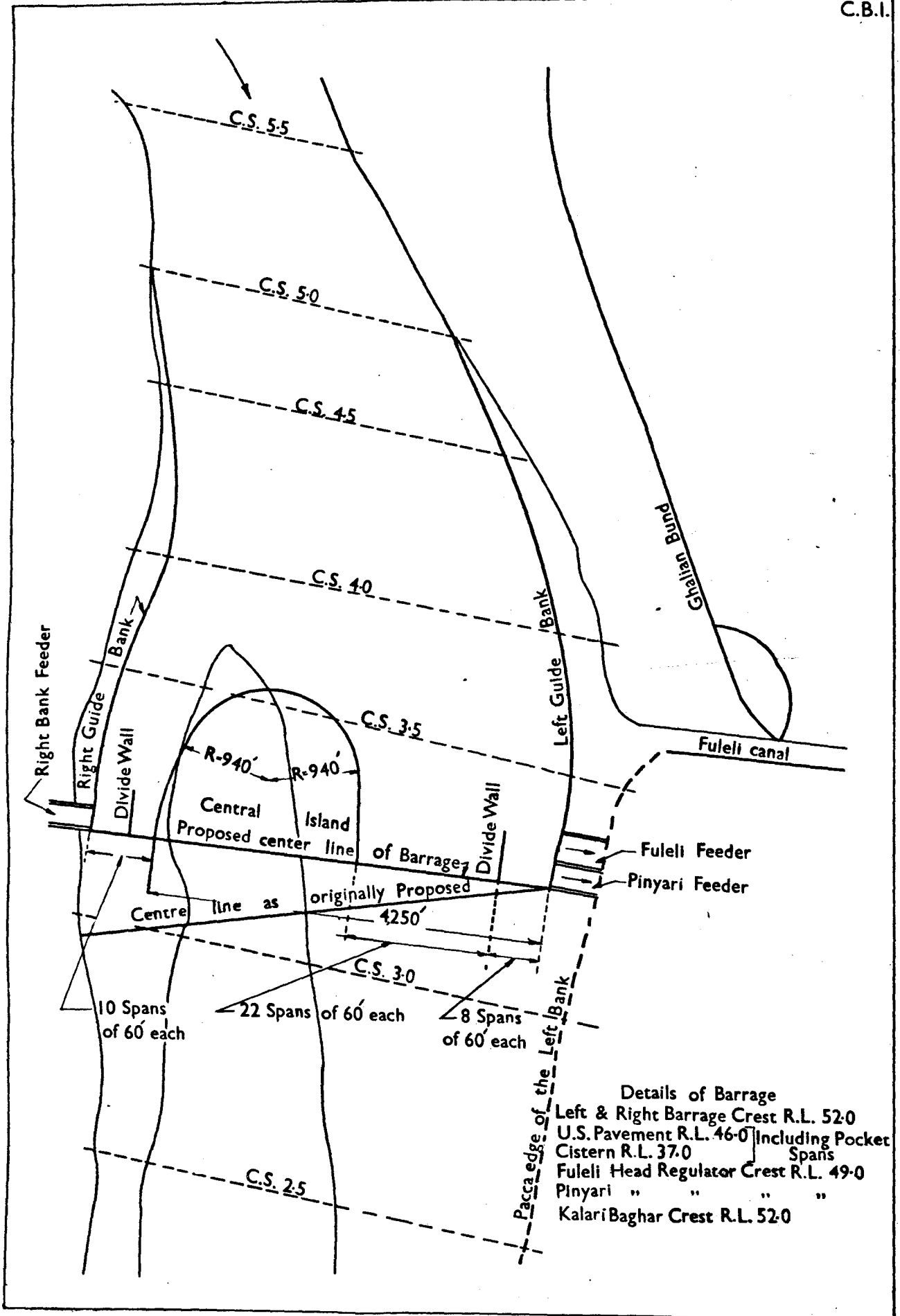


Figure 10C.20:- Showing Alignment of Barrage (Kotri Barrage Model)

TABLE 10C.5.

Serial No.	Item	Project Division design of Double Barrage							Proposed revised design of Double Barrage							Remarks
		Bed width (feet)	Depth (feet)	<i>f</i>	Slope	Full Supply Level	C.B.L.	Velocity feet per second	Bed width (feet)	Depth (feet)	<i>f</i>	Slope	Full Supply Level	C.B.L.	Velocity feet per second	
1	Design of Feeders— Fuleli-cum-Guni ..	254	19.7	0.67	0.0537	65.7	46.0	3.7	280	18.0	0.67	0.0537	66.0	48.0	3.7	Not absolutely essential to the revised design.
		Offtake	Discharge (cusecs)	Spans	Crest level	L.H.	Full Supply Level Up-stream	Offtake	Spans	Crest level	L.H.	Full Supply Level Up-stream	Down stream	Remarks		
2	Design of Head Regulator— Fuleli-cum-Guni ..	Jamshoro	19,215	Existing	46.0	0.9	66.6 65.7	Barrage	10×24'	49.0	0.5	66.5 66.0		Two spans saved.		
	Pinyari ..	Barrage	16,300	10×24'	52.0	0.6	66.5 64.5	Barrage	8×24'	49.0	0.5	66.5 64.5				
	Kalri-Baghar ..	„	12,415	8×24'	52.0	0.5	66.5 66.0	Barrage	8×24'	52.0	0.5	66.5 66.0				
		Left pocket	Left River	Right River	Right pocket	Left pocket	Left River	Right River	River pocket	Remarks						
3	Number of spans ..	6	22	7	5	8	22	5	5	Total number same; spans redistributed to accord more closely with existing river conditions.						
		Nos.	Upstream pavement level	Crest level	Cistern level	Nos.	Upstream pavement level	Crest level	Cistern level	Remarks						
4	Left Pocket spans	6	52.0	52.0	37.0	8	46.0	52.0	37.0	Raised crest instead of flat regulator crest six feet higher than upstream pavement.						
6	Left River spans	22	46.0	52.0	37.0	22	46.0	52.0	37.0	All spans have same upstream crest and downstream cistern levels in proposed design.						
	Right River spans	7	49.0	55.0	40.0	5	46.0	52.0	37.0							
7	Right Pocket spans	5	52.0	55.0	40.0	5	46.0	52.0	37.0	Raised crest of normal profile and regulator crests six feet higher than upstream pavement level.						

- (b) there will be two more spans in the left pocket without adding to the total number of river and pocket spans ; so that the ' still pond ' limit will be increased, assuring a longer period of *ungated* flow with consequent diminution in, and postponement of accretion.
- (c) the total linear waterway in each arm of the river will more closely follow the existing channel widths, thus least interfering with the natural conditions with least risk of serious river changes ;
- (d) it is thus possible to curtail the length of the pitched island with resulting decrease in (i) cost of pitching and apron, and (ii) the difference of pond levels in the left and right arms with equal gate openings (actually, with the short island, the model showed no difference in water levels in the two arms) ;
- (e) afflux over natural river levels caused by the barrage will be reduced ; because the raised crest in the pocket will have a correspondingly higher coefficient than the flat crests of the original design ;
- (f) it is thus possible to reduce two spans of Pinyari Head Regulator by lowering its crest level, the lower crest level helping to maintain low pond levels ;
- (g) the regulator crests are at a higher level than the upstream barrage pavement level ; the design thus allows a margin of three feet on the left and six feet on the right bank for bed sand and silt to settle in the pockets without passing into the canals. (This point is not considered as a material advantage by the Station, although local officers put a value on it, because the high regulator crest does not by itself prevent the bed silt from rolling up and into the canals.)

CENTRAL ISLAND FOR CONTROL AND EFFICIENT EXCLUSION ON BOTH THE BANKS

The Central Island of the Kotri Barrage design must be distinguished from the pitched " attracting island " used to centre a river, as at Suleimanki because it is essentially different in principle and action. While the success of an " attracting island " depends wholly on its capacity to attract flow to itself and this, in turn, depends on its correct positioning in respect of the meander characteristics, the Central Island at Kotri is *not* to attract flow—in fact a series of experiments is done with a view to evolve the best design of the island which will not unduly attract flow.⁽⁵⁾ but to distribute it between the two river arms as desired and to create favourable curvature on both banks ; so that while an " attracting island " may cease to be efficient in course of time with changes in the river course higher upstream, there is little likelihood of the Central Island becoming ever less efficient in its designed functions.

The model has substantiated the elasticity of the control which the Central Island helps to achieve ; even when shoaling of the right arm of the river was greatly exaggerated to counteract model limitation, the island enabled that channel to be kept alive, and the exclusion from the Right Bank Canal did not deteriorate. In fact, it is to this Central Island, artificially enforcing favourable curvature under all conditions, that the Right Bank Feeder, although otherwise unfavourably disposed in respect of river curvature, owes its property of consistently excluding bed sand ; it is this island which helps to maintain a deep channel near the right pocket which swings into the river spans by the favourable twist created by the divide wall action.

On the left bank, too, in spite of the very high relative draw-off of the left Bank Canals—*i.e.* a high Q_p/Q_r ratio at the worst stage of $Q = 200,000$ cusecs—when the Fuleli Canal was silted into the pocket, and ^{br} brought within the

(5) See item (5) of this chapter.

influence of the island the design gave fully satisfactory results even without regulation at the Barrage. The Island, however, affords additional control and assistance in exclusion, because it enables any desired redistribution of discharge in the two arms so as to adjust Q_p/Q_r as necessary for efficient exclusion on both banks.

Because there has existed at this site a natural central island for over forty years, the introduction of the armoured central island in its place is not likely to cause any adverse disturbance in the river regime; the attraction by the pitching more than offsets any tendency of the Barrage to cause deterioration of the right arm channel—see Table ICC.-4. But, even should this unlikely eventuality ever come to pass, the Right Bank Canal can be extended, at little cost, to have a favourable mouth from the concave bend upstream.

FACTORS AFFECTING THE EXCLUSION OF SAND FROM THE CANALS

Model experiments, made for the Kotri and Hajipur Barrage sites, have focussed attention on the following factors as affecting the exclusion of sand from the off-taking canals of a properly aligned and well designed Barrage :—

- (a) The Barrage should be located at a place where the desired curvature of flow is sufficiently established and will persist, even after the construction and operation of the Barrage—because a Barrage tends to diminish to some extent the existing curvature of flow at a site.
- (b) The proper positioning of the off-takes of the canals in relation to the curve; e.g. the Fuleli Canal off-taking at Jamshoro away from the direct control exercised by the Kotri Barrage and at a point where the curvature effect is not sufficiently established, gave serious trouble in regard to sand-exclusion at its head.
- (c) The position of the abutment relative to the *pacca* edge of the river; if the abutment is moved considerably inwards into the river channel the area bounded by the *pacca* bank and the divide wall increases, resulting in an increase in the area of dead water and slack flow along the bank, which is responsible for silting along the bank and, therefore, easier entry of bed sand into the canals.
- (d) Orientation of the barrage line with respect to the main flow; a re-orientation in the right direction will help to restore partially the diminution in the curvature effect caused by the Barrage and the river ward shifting of the abutment, *vide* (a) and (c).
- (e) The relative Pocket draw-off Q_p with respect to Q_r , the discharge at which appreciable bed sand movement begins in the river, i.e. the ratio Q_p/Q_r . The higher this ratio is, the greater the difficulty of exclusion at the worst stage, Q_r ; hence, where Q_p/Q_r is already high, excluders and ejectors will increase the difficulties of exclusion.
- (f) The 'still pond' limit of the barrage. This affects accretion as the lower the 'still pond' limit, the earlier the gates are out of water, and the less accretion there would be; and the less the accretion, the less the ultimate draw of sand into canals.

- (g) The number of pocket spans relative to the barrage spans. In the original design of the Kotri barrage, the pocket spans were designed with flat crests—necessitating a larger number of these spans ; so the ' still pond ' limit was reduced—an advantage—while the area bound by the divide wall and the bank was increased, leading to greater draw into the offtake. There should be a compromise between these two conflicting considerations.
- (h) Suitable regulation of the barrage assists in creating a favourable twist of bed flow away from the pockets, particularly at lower discharges ;—wedge shaped openings with least opening away from the pocket and greatest openings adjacent to the Divide Wall gave the best results, but was not enough to correct fully for defective curvature.
- (i) Splay to the head regulator faces gives an upstream orientation to the canals, enabling water to flow smoothly into the canal without creating turbulence in the pocket, which may otherwise throw sand into suspension under semi-open flow conditions.
- (j) Division of the discharge into two arm by a Double Barrage reduce the Q_p/Q_r ratio in each arm, and admittedly, at low river discharges this is a draw back as it increases difficulties of sand exclusion from the canals ; but the other advantages more than offset this one disadvantage.
- (k) The length of the divide wall ; when the pocket draw is considerable relative to the river discharge, *i.e.* Q_p/Q_r is high, and the draw of the first offtake is high, it was observed that the divide wall should extend at least to the upstream abutment of the canal, otherwise the first (uppermost) canal drew most of the bed sand ; where Q_p/Q_r and the discharge of the uppermost canal are relatively small a dividewall of two-thirds this length is generally sufficient.
- (l) The relative difference between pocket pavement level and the crest of the head regulators has little or no effect on exclusion, any bed material entering the pockets rolling up into the canals.
- (m) Intelligent regulation of the Pocket for all discharges —especially under semi-open flow conditions. It was observed that the Pocket gates nearer the face of the head regulator should be opened less than those away from the face to check the sand entry into canals.

(5) OPTIMUM SHAPE OF THE NOSE OF KETI ISLAND AT KOTRI FOR THE LOWER SIND BARRAGE⁽⁶⁾

ABSTRACT

It is proposed to construct the Kotri Barrage in two parts, roughly one-third on the right and two-thirds on the left bank, corresponding approximately to the present right and left channel sizes respectively, with the existing " *Keti* " natural island) permanently left in between as a pitched, dividing island. Thus both the right and left bank canals would have the desired takeoff from the outside (concave side) of that curve provided by nature without the necessity

⁽⁶⁾ Indian Waterways Experiment Station Poona, Annual Report(Technical) 1946 page 11.

elaborate sand exclusion devices. The natural conditions at the site, for the last 40 years or perhaps even longer, are that two channels exist with the intermediate 'Keti' in practically the same position except that the Keti appears to have grown a little in size with age. Thus, the proposed construction is likely to interfere as little as possible with the stable, natural river conditions.

With the utilisation of the existing natural island, the natural meander length is not disturbed and an unnatural distorted curvature is not imposed at the very point where it is essential to have sweet natural curvature for easy exclusion.

A possible objection is that if the nose is held by pitching, the island may act as an "attracting" spur by virtue of the scour-hole around it, changing the favourable curvature on the two banks to an unfavourable one on one of them. A deep scour on the right side, as a consequence of intense turbulence, is also objectionable in that it may throw the bed material into suspension to be drawn in by the right bank canal—the Kalri Baghar Feeder.

So, model experiments were undertaken to evolve the best shape of the Central Island which would give the minimum depth and extent of scour, particularly in the right arm and would thus cause less concentration of flow and turbulence at the nose.

MODEL

A 1/500 : 1/83.3 scale model of the pitched island was constructed in the 16 feet parallel-sided Left Bank Channel, the island being centrally located,

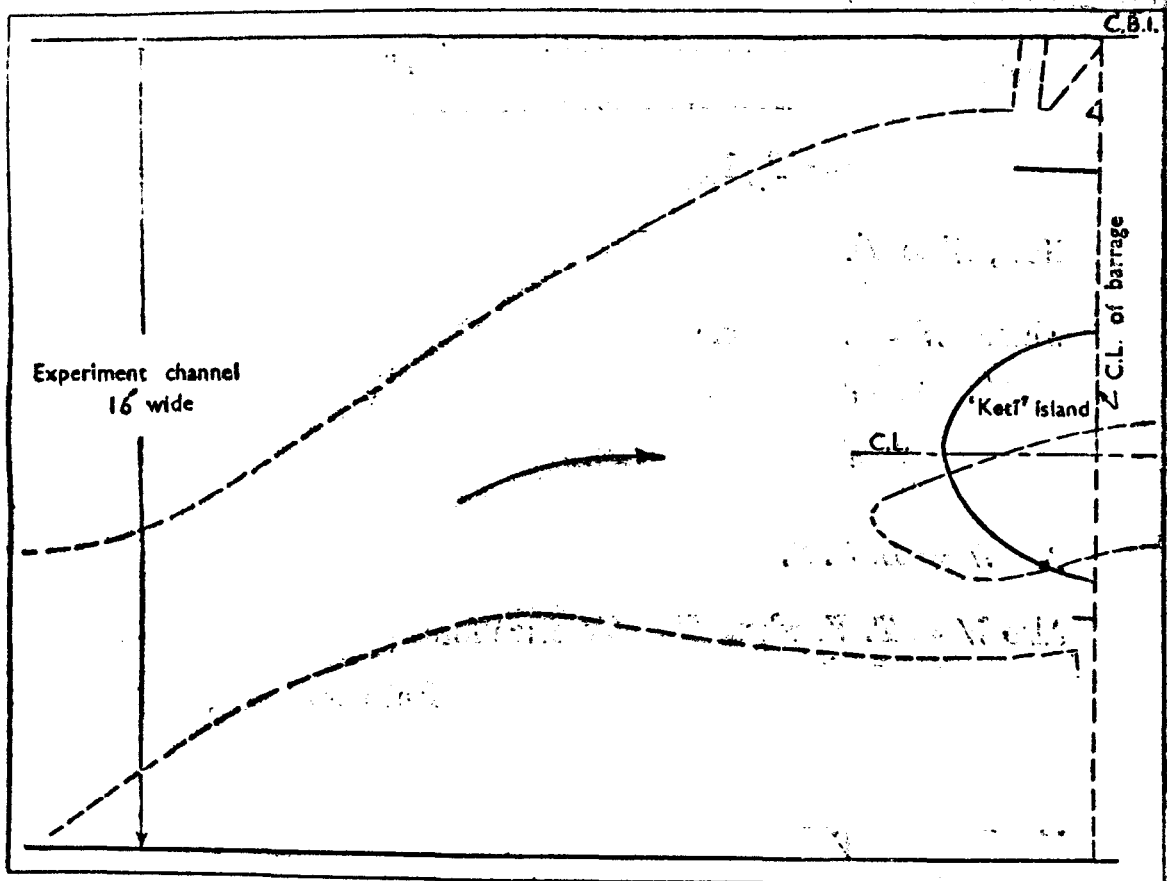


Figure 10C.21 : Plan showing the river Indus and the location of the 'Keti' Island

- Note : (1) The channel in which the island was located has been shown in full lines.
 (2) Proposed Kotri Barrage is shown in dotted lines.

Figure 10C.21. The intensity of discharge in the right and left arms in the model was = 346 cusecs, whereas actually in the river the intensities in the two arms are 250 and 333 cusecs respectively. So far as the Right Bank Canal is concerned—which is the crux of the problem—the model conditions of equal intensity were thus clearly more severe than would occur in the prototype. The curvature of flow upstream was not reproduced, because it being favourable to the Right Bank Canal in the prototype the straight model would give results erring on the safe side.

The model was laid in Londha sand (0.3 mm.) and the top of the island was maintained at R. L. 84.00

SCALES

(a) *The length scale* was tentatively fixed at 1/500 and, though as little vertical exaggeration as possible was desirable, a vertical exaggeration of 6.0 had to be adopted, (a) owing to the very small depth of island over the river bed as compared with its length and width, and (b) because 1 : 3 side slopes in the prototype could be conveniently simulated by (1/2 : 1) slopes in the model. The vertical scale was thus fixed as 1/83.3.

(b) *Time scale.*—The time scale was worked out from the tractive force formula as follows :—

$$\begin{aligned} \text{Time scale} &= \frac{\text{Tractive force in prototype}}{\text{Tractive force in model}} \\ &= T_p/T_m \end{aligned}$$

Now, $W \propto D.S.$

where W = work done

D = depth

S = slope

$$\therefore W = L. D. S.$$

Also $W = T. V.$ where T = Tractive force

V = Velocity in feet per second.

$$\therefore W = L. D. S. = T. V.$$

$$\therefore T = \frac{L. D. S.}{V}$$

$$\Delta \text{ Time scale} = T_p/T_m = L_p/L_m \cdot D_p/D_m \cdot S_p/S_m \cdot (D_m/D_p)^{1/2}$$

as $V \propto D^{1/2}$ (approx).

$$\text{Now } \frac{L}{L_m} = 500$$

$$\frac{D_p}{D_m} = 83.3$$

$$\text{and } \frac{S_p}{S_m} = 1/6$$

$$\therefore \text{ Time scale} = \frac{00 \times \sqrt{83.3}}{6} = 760$$

i.e. one day in prototype = 1.9 minutes (approx).

This time scale applies to the dominant discharge but has to be adjusted for other discharges, by trial and error, the higher discharges being run for a corresponding smaller time and discharges lower than dominant discharges for a longer time.

(c) *Discharge in model*

$$L = 1/500 \quad D = 1/83.3$$

$$\therefore \text{ Discharge scale} = LD \sqrt{D}$$

$$= 500 \times 500/6 \sqrt{83.3} = 380,000$$

$$= \frac{500 \times 16 \times 346}{380,000}$$

$$= 7.22 \text{ cusecs}$$

where 16 feet = width of flume in the model 346 = Intensity in cusecs in prototype ($\equiv 750,000$ cusecs)

PROCEDURE

It was first necessary to design an approximately sound and economical shape of the island. This being ascertained, the side slopes of the island were altered in the various tests and the resulting scour-hole observed. A constant superflood discharge of 346 cusecs $\equiv 750,000$ cusecs (approximately), with upstream water level at R. L. 74, was run in each case, and the scour-hole was allowed to stabilize. The final stable scour-hole was compared in respect of depth, extent and shape. Stable conditions were generally reached after about 25 hours of constant run in all cases. Some experiments were finally repeated under rising and falling flood conditions.

EXPERIMENTS AND DISCUSSION OF RESULTS

(a) *Profiles I, II and III.*—A constant side slope of 1/2 : 1 was maintained in all these three cases (see Figure 10C.22) and Figures 10C.23 to 10C.25 show the configuration of the stable scour-hole obtained in each case.

From Table 10C.6, it is seen that profile III gave more uniform and less extensive scour at the nose as well as at the sides. Profile III, however, indicated higher depth of scour. But in view of less afflux and the large economy possible in the case of a short island (shorter by about 300 feet) it was decided to adopt it and modify it suitably by correcting the side slopes so that the depth of scour is well within the permissible limits.

TABLE 10C.6

Showing comparative merits of various profiles in $\frac{1}{100} : \frac{1}{81.5}$ Scale Model of " Central " Island (Kotri Barrage).

Profile	Scour in feet and corresponding R. L. of scour at Nose	Scour in feet and corresponding Lowest R. L. of scour on the Left Flank	Scour in feet and Corresponding Lowest R. L. of scour on the Right Flank	Extent of scour at the nose (feet)	Extent of scour on the Left Flank (feet)	Extent of scour on the Right Flank (feet)	q_p in cusecs
I Elliptical and long	1.22 \equiv R.L.—17.5	1.17 \equiv R.L.—13.5	1.20 \equiv R.L.—15.96	1.8 \equiv 900	3.3 \equiv 1,650	3.6 \equiv 1,800	346
II Pointed nose and long	1.20 \equiv R.L.—15.96	1.25 \equiv R.L.—20.0	1.20 \equiv R.L.—15.96	1.8 \equiv 900	2.8 \equiv 1,400	3.8 \equiv 1,900	Do.
III Elliptical and short	1.25 \equiv R.L.—20.0	1.20 \equiv R.L.—20.0	1.30 \equiv R.L.—24.0	1.8 \equiv 900	3.5 \equiv 1,750	3.3 \equiv 1,650	Do.
IV Short	1.03 \equiv R.L.—1.0	1.17 \equiv R.L.—13.5	1.18 \equiv R.L.—14.4	1.6 \equiv 800	3.1 \equiv 1,550	2.9 \equiv 1,450	Do.
V Do.	1.12 \equiv R.L.—3.4	1.30 \equiv R.L.—24.0	0.95 \equiv R.L. +5.0	2.4 \equiv 1,200	3.4 \equiv 1,700	2.9 \equiv 1,450	Do.
VI Do.	1.08 \equiv R.L.—6.0	1.50 \equiv R.L.—41.0	0.90 \equiv R.L. +9.03	2.4 \equiv 1,200	3.1 \equiv 1,550	2.6 \equiv 1,300	Do.
VI Test (A) Do.	0.52 \equiv R.L. +40.7	0.76 \equiv R.L. +20.7	0.62 \equiv R.L. +32.4	0.3 \equiv 150	2.8 \equiv 1,400	1.8 \equiv 900	Flood cycle
VI Test (B) Do.	No scour	No scour	Dominant discharge
VII Do.	1.00 \equiv R.L. +0.07	1.07 \equiv R.L.—5.13	1.04 \equiv R.L.—2.63	1.9 \equiv 950	3.0 \equiv 1,500	3.2 \equiv 1,600	346
VIII Do.	1.16 \equiv R.L.—12.63	1.37 \equiv R.L.—30.12	1.12 \equiv R.L.—9.3	2.5 \equiv 1,250	3.0 \equiv 1,500	346

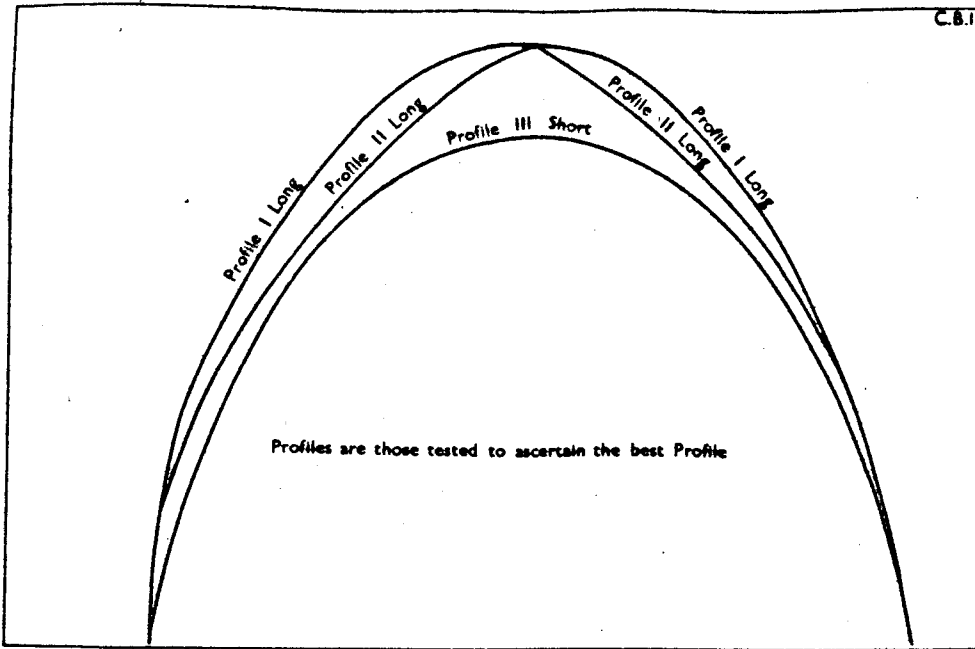


Figure IOC.22- Showing Various Profiles of Keti Island $\left[\frac{1}{500} : \frac{1}{83.3} \right]$ Scale Model

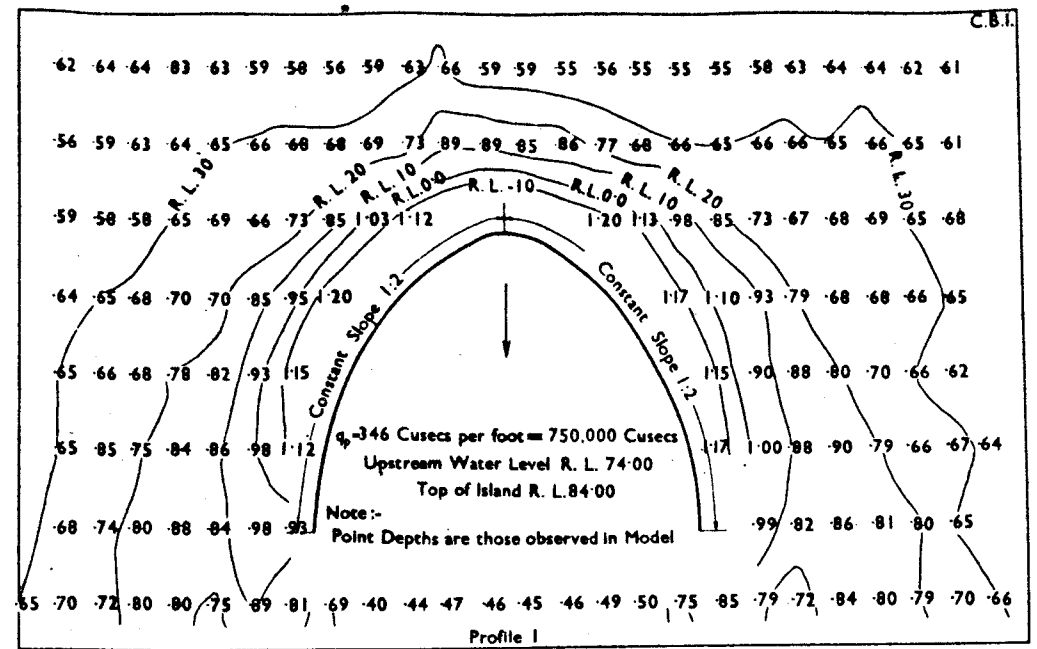


Figure IOC.23- Showing Kotri Barrage Island $\left[\frac{1}{500} : \frac{1}{83.3} \right]$ Scale Model

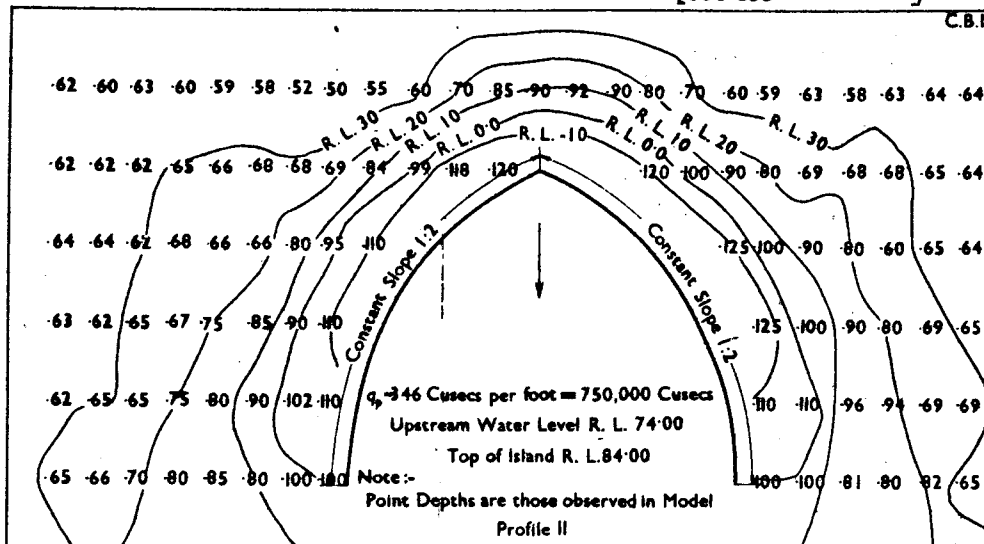


Figure IOC.24- Showing Kotri Barrage Island $\left[\frac{1}{500} : \frac{1}{83.3} \right]$ Scale Model

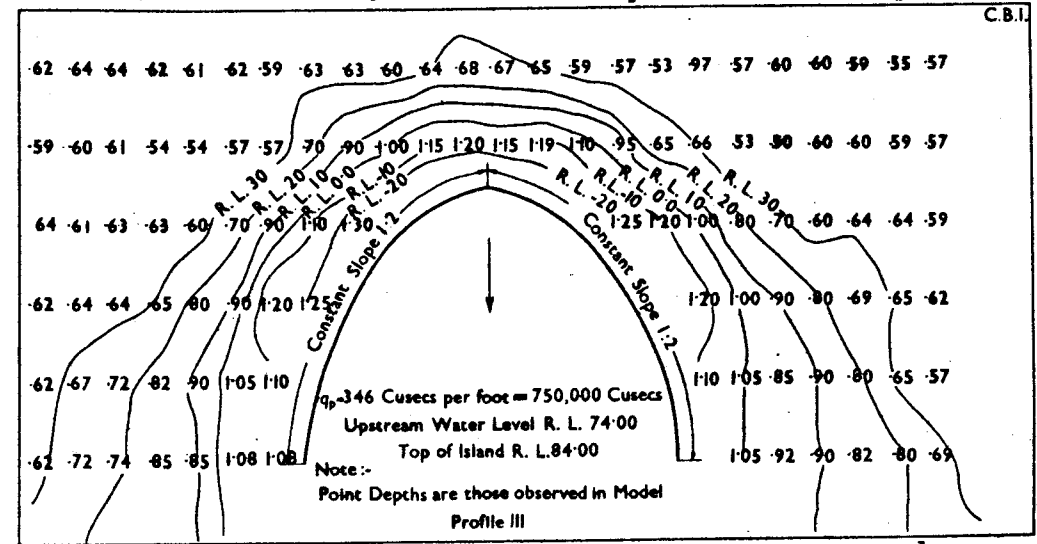


Figure IOC.25- Showing Kotri Barrage Island $\left[\frac{1}{500} : \frac{1}{83.3} \right]$ Scale Model

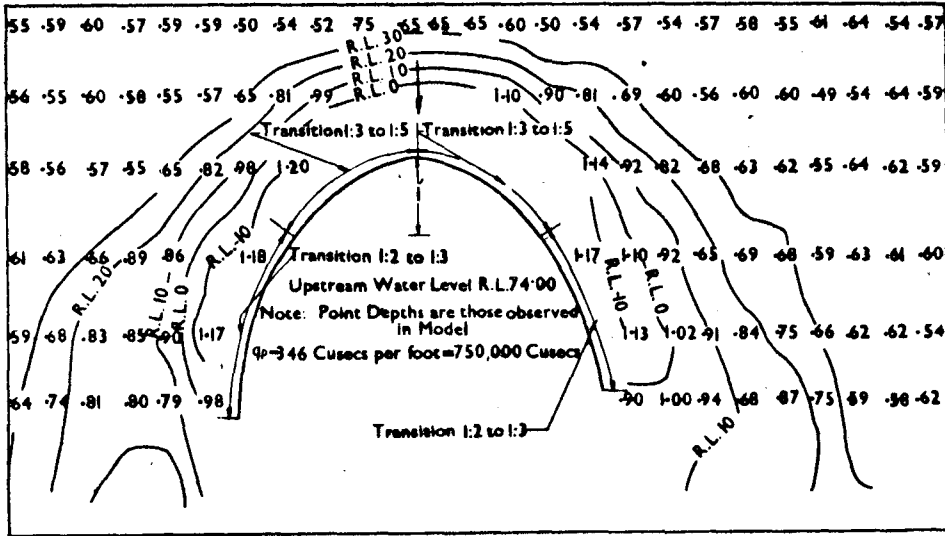


Figure 10C.28- Showing Kotri Barrage Island Profile $V \sqrt{\frac{1}{500} \frac{1}{833}}$ Scale Model

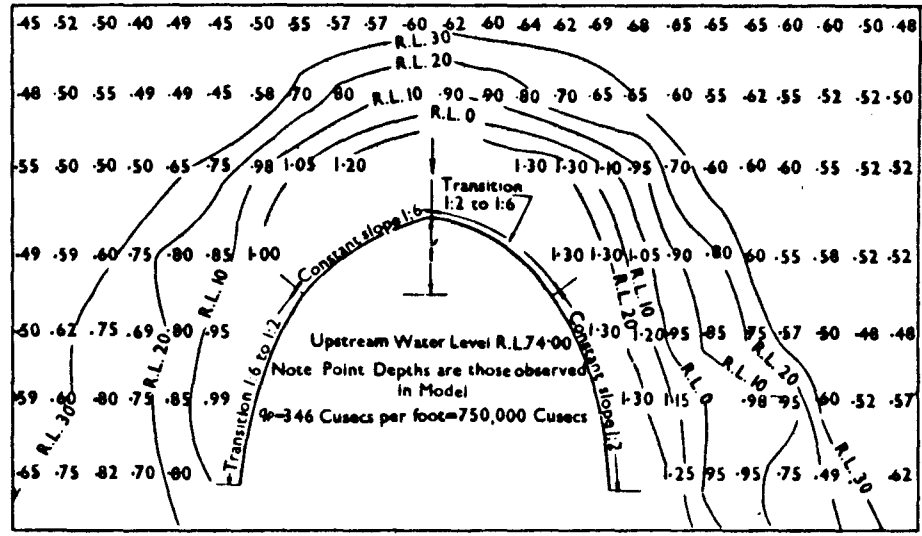


Figure 10C.29- Showing Kotri Barrage Island Profile $V \sqrt{\frac{1}{500} \frac{1}{833}}$ Scale Model

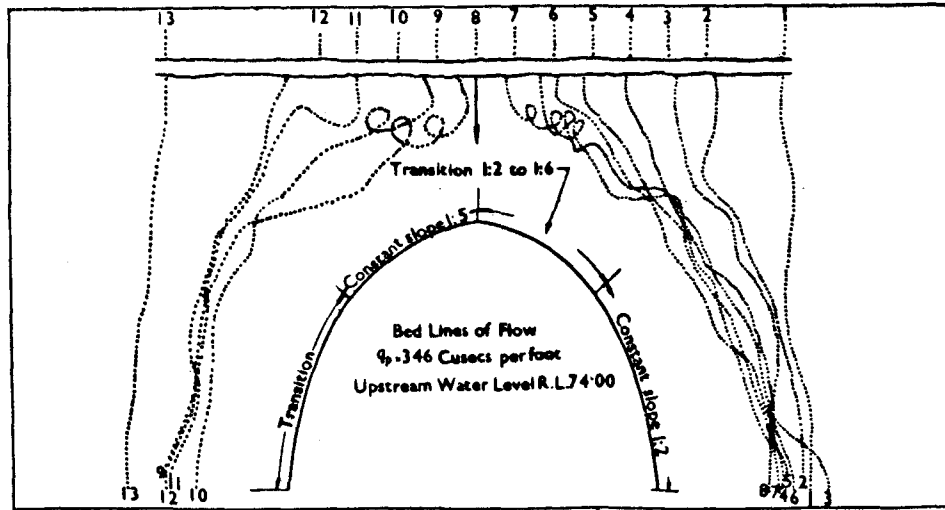


Figure 10C.30- Showing Keti Island Profile $V \sqrt{\frac{1}{500} \frac{1}{833}}$ Scale Model

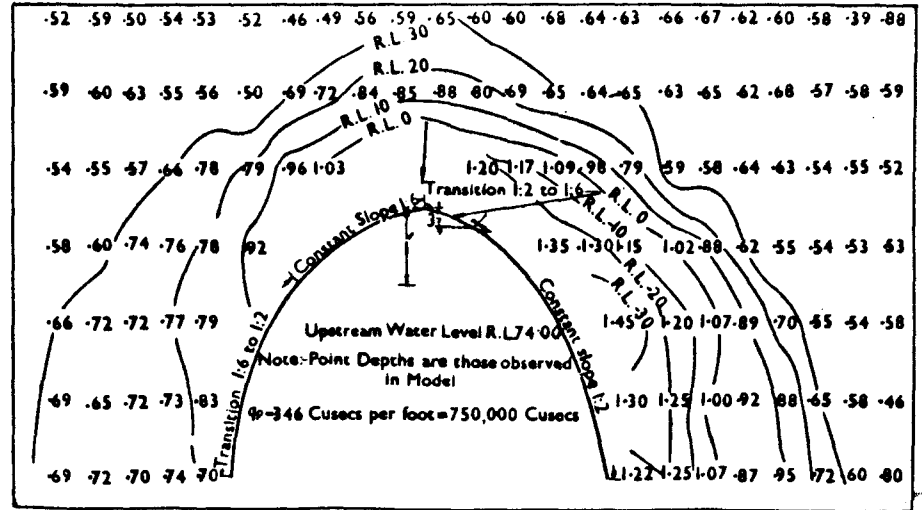
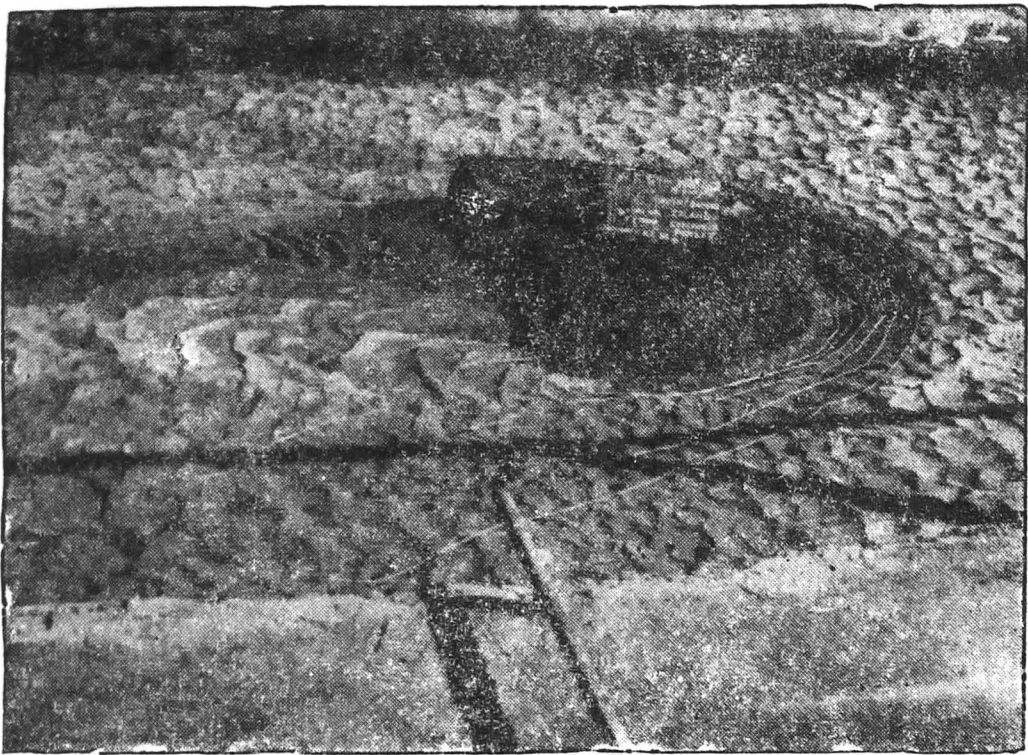
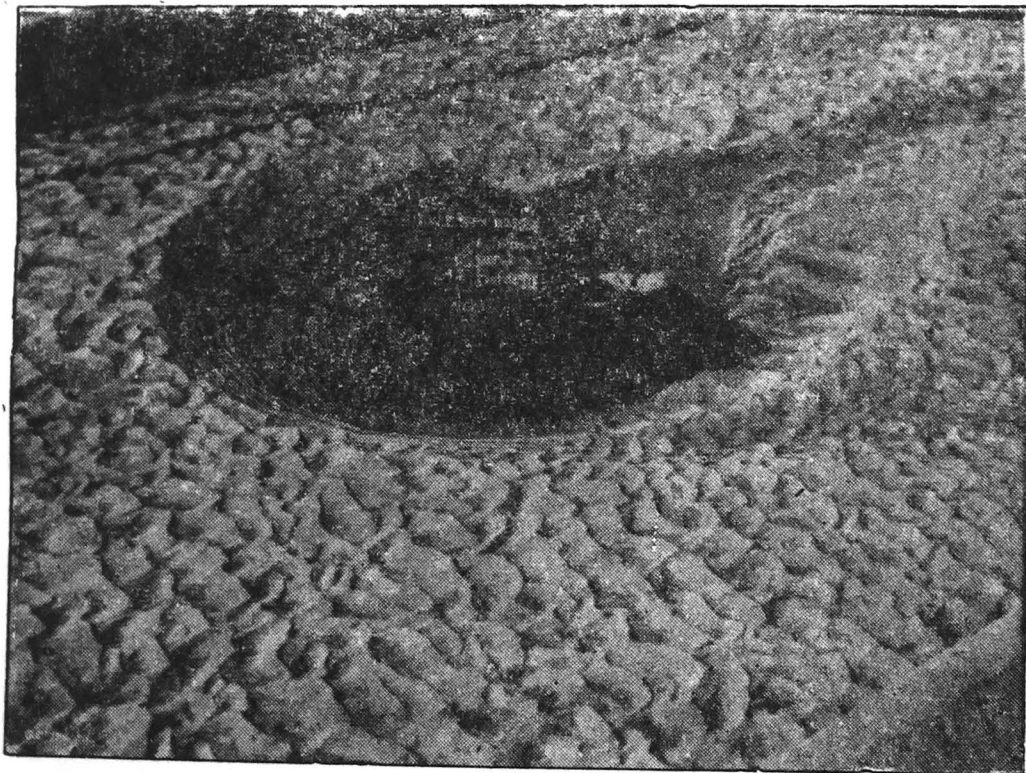


Figure 10C.31- Showing Kotri Barrage Island Profile $V \sqrt{\frac{1}{500} \frac{1}{833}}$ Scale Model



Profile I

Figure 10C.26 : *Looking sideways upstream—showing the shape of the scour hole obtained*



Profile III

Figure 10C.27 : *Looking sideways downstream—showing the shape of the scour hole obtained*

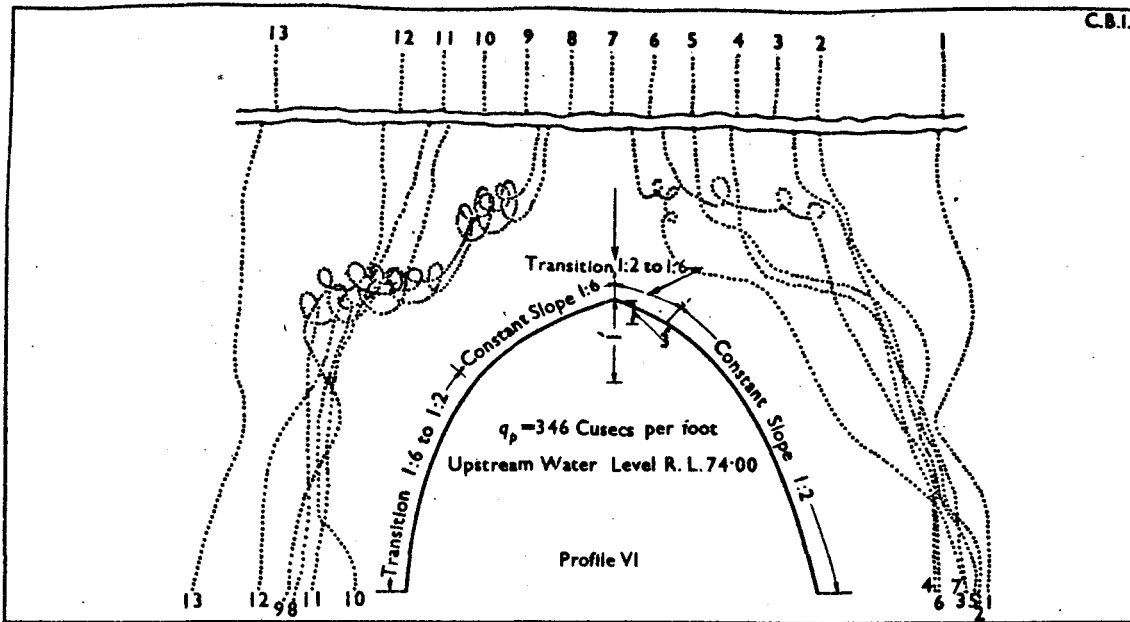


Figure IOC.32 - Showing Bed Flow $\left[\frac{1}{500} : \frac{1}{83.3}\right]$ Scale Model of Keti Island

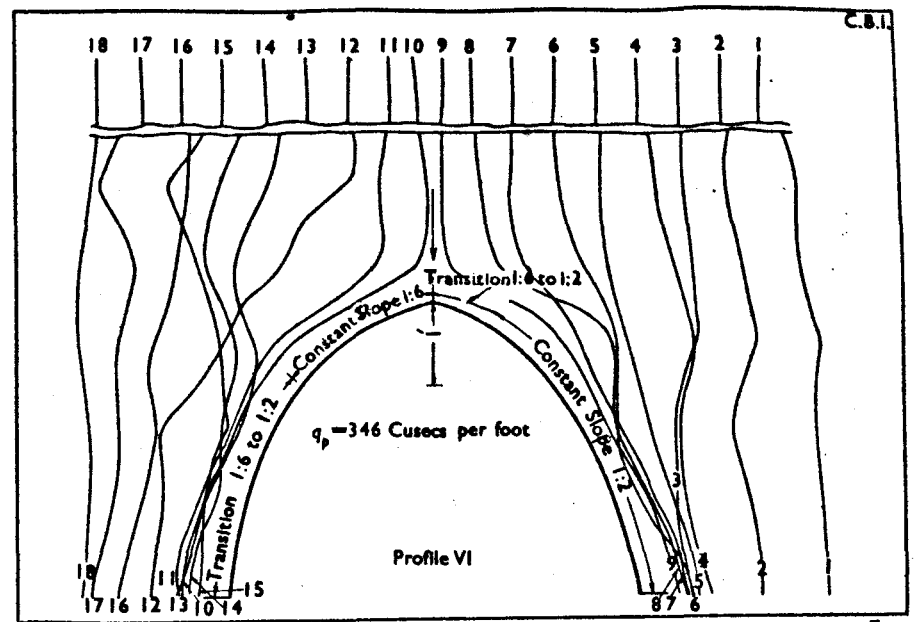


Figure IOC.33 - Showing Surface Flow $\left[\frac{1}{500} : \frac{1}{83.3}\right]$ Scale Model of Keti Island

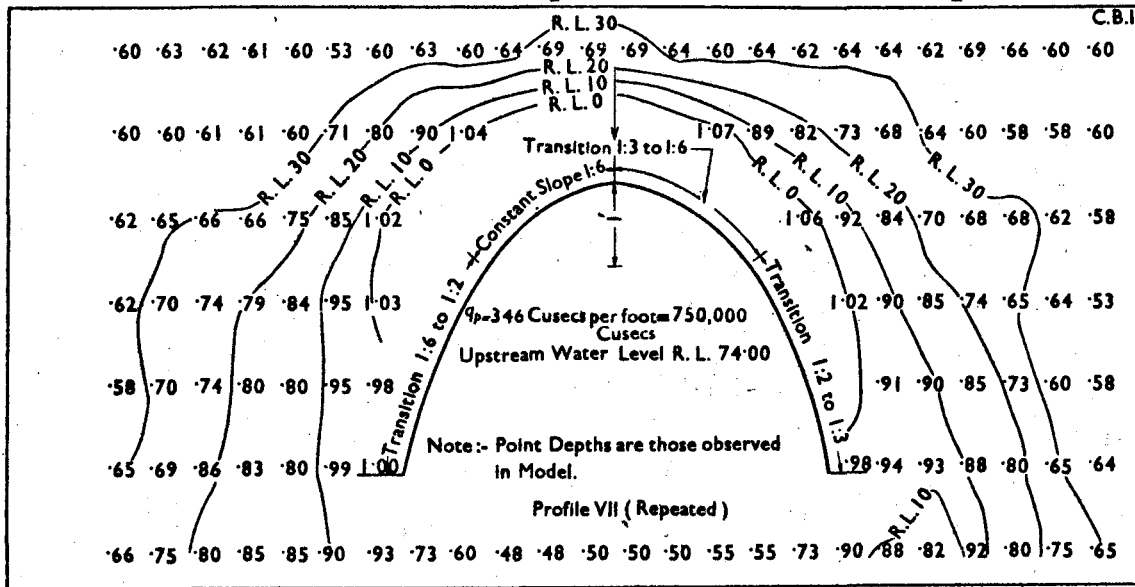


Figure IOC.34:- Showing Bed Lines of Flow $\left[\frac{1}{500} : \frac{1}{83.3}\right]$ Scale Model of Kotri Barrage Island

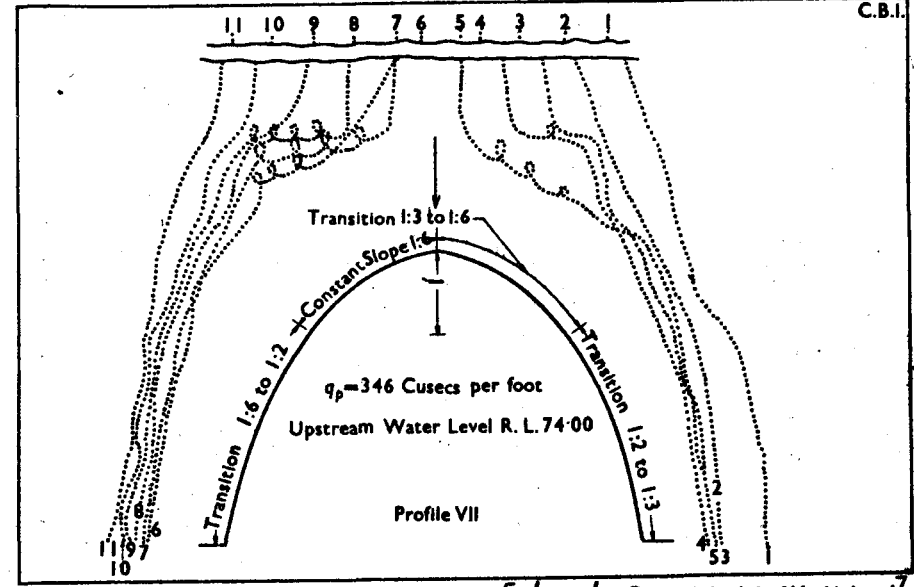


Figure IOC.35 - Showing Bed Lines of Flow $\left[\frac{1}{500} : \frac{1}{83.3}\right]$ Scale Model of Keti Island

Figures 10C.26 and 10C.27 show the shape of the scour hole obtained with profiles I and III.

(b) *Profile IV.*—The natural slope of the island at the nose in nature approximates to 1 : 6 (*i.e.* 1 vertical : 6 horizontal) and hence it was thought feasible to retain this slope in the dividing island as far as possible, Figure 10C.28. The side slopes were, however, gradually steepened, but were maintained at all points flatter than those at the corresponding points in profile III. This was accomplished by providing transition reaches in two stages, *viz.* from 1 : 6 to 1 : 3 and from 1 : 3 to 1 : 2. The flatter slope had the desired effect of reducing scour, on account of the increased effective surface area, thus reducing the intensity of attack all around. But this island would not be so effective in excluding sand from the right bank due to the symmetrical scour-hole on either side.

(c) *Profile V.*—To offset as far as possible the inherent defect of profile IV differential scour—*i.e.* more scour in the left arm than in the right—had to be induced by providing suitable different slopes to the sides Figure 10C.29. As further flattening of slopes on the right side was not feasible—due to possible gulletting—differential scour was proposed to be obtained by keeping the same side slopes on the right side as in profile IV and altering only the left side slopes and the lengths of transition reaches on that side.

The steeper slope of profile V increased the severity of attack on the left flank and accentuated scour to as low a depth as R. L.—24, while the depth of scour on the right flank was only up to R. L. + 5.0.

Figure 10C.30 illustrates the bed lines of flow and indicates the probable exclusion. The sand exclusion was judged by the extent and the number of central bed seeds that were pulled to one side or the other, as the case may be.

(d) *Profile VI.*—The effective slope on the left side was made still steeper by reducing the length of the transition reach, Figure 10C.31. This resulted in the largest depth of scour obtained in the series, *viz.* to R.L.—41, Figure 10C.32. This was obviously too deep a scour-hole and consequently this profile was given up in spite of satisfactory exclusion.

It was decided to adopt a profile which limits the scour on the left side to L. R. = 13.5, obtained with profiles I and IV. Figure 10C.33 shows the surface lines of flow with profile VI.

(e) *Profile VII.*—To meet the above objection, profile VII was evolved. Two stage transitions were provided for the left side, *viz.* from 1 : 6 to 1 : 3 and from 1 : 3 to 1 : 2, Figure 10C.34. The maximum depth of scour was limited to R. L. —5.13 and owing to the differential scour, the exclusion was satisfactory. Figures 10C.35 and 10C.36 show bed and surface lines of flow. The excluding action at the nose by twisting the bed lines is clearly visible. This was the profile finally adopted.

(f) *Profile VIII.*—This was tried to see if the lengths of the transition reaches on the left side could be curtailed still further, but proved unsuccessful as it again developed a deep scour-hole, as deep as R. L. —30.12, Figure 10C.38. The profile of the island was clearly not improved by the shorter lengths of transition reaches.

(g) *Profile VI (Test A)*.—As a matter of interest, a flood cycle was run ; but both the depth and extent of scour were smaller as compared with the depth obtained by running a constant superflood discharge, Figure 10C.39.

Test B. To see whether scour consequent on a cycle is nearer that obtained with the dominant discharge than the maximum discharge, a dominant discharge of 250,000 cusecs was run ; scour with the dominant discharge was negligible.

CONCLUSIONS

Profile VII gave the best results and hence, it was recommended. This profile is adopted in the full-width model of the Kotri Barrage and is being tested under the curvature inherent in the site. Owing to the many disturbing factors, no absolute numerical accuracy is claimed for the values of depth and extent of scour given in Table 10.C6. The relative values are however, useful for assessing the best shape.

(6) MODEL EXPERIMENTS ON THE EXCLUSION OF SAND FROM A FEEDER CHANNEL TAKING OFF FROM PENNAR RIVER⁽⁷⁾

A project has been prepared for taking off a channel with full supply level one foot below the bed of the Pennar river. The river at the point of offtake is 115 feet in width. It has an average fall of about 12 feet per mile and the bed is all sand overlying rock. The channel proposed to be taken off from it has a carrying capacity of 100 cusecs and is 12 feet wide at bed with side slopes of 1 to 1.

The problem was both to draw off as much water as possible from the river and to prevent entry of sand into the channel. Due to other reasons, the full supply level of the channel had to be kept one foot below the bed of the river.

In order to draw off water, it was considered desirable to have a pitched island to attract the flow to the channel. Incidentally the river by-pass between the pitched island and the offtake weir of the channel would get scoured out, thus rising the offtake weir above river bed level. This would automatically prevent sand from overflowing into the channel. To exclude sand further the offtake weir was made concave in shape. The side of the pitched island facing the offtake weir was to be convex in shape.—When the flow carrying sand passes in the river between the weir and the island, the water of the river would hug the weir concave in shape and the sand would get deposited at the convex side of the pitched island. The entry to the draft is made smooth by giving a bell mouth shape to it.

The solution suggested above was tried on a model of the river project put to 1/12th scale. All the dimensions of both the channel and the river were made scalar except the width which was restricted to only six feet. Further increasing the width of the river would not have had any consequence as only such portion of the sand which is deflected by the pitched island was all that could get into the channel. Sand similar to that obtained in the river was laid on the model.

⁽⁷⁾ Hydraulic Research Station, Krishnarajasagar Mysore Annual Report 1946, pages 62 to 64.

The model was first laid without any pitched island or giving any curvature to the offtake weir. When water was allowed into the river, the channel got choked up with sand very quickly. A pitched island was next put up. Its position in the river in relation to the offtake weir, its shape, length width and height were determined by a series of experiments. It was found with a velocity of flow of two feet per second that out of the total quantity of sand getting into the river by-pass, 25 per cent. of it got into the channel.

To minimise this further, the bank of the river which was straight at this reach was cut into a concave shape and the offtake weir rebuilt to follow the curve of the bank. A bell mouth entry was also given (*vide* Figure 10C.40). Only 12½ per cent. of the sand getting into the river by pass entered the channel. Most of the sand used to enter at the end of the offtake weir left side where the flow gets bifurcated finally, a portion entering the channel and the remaining getting back to the river.

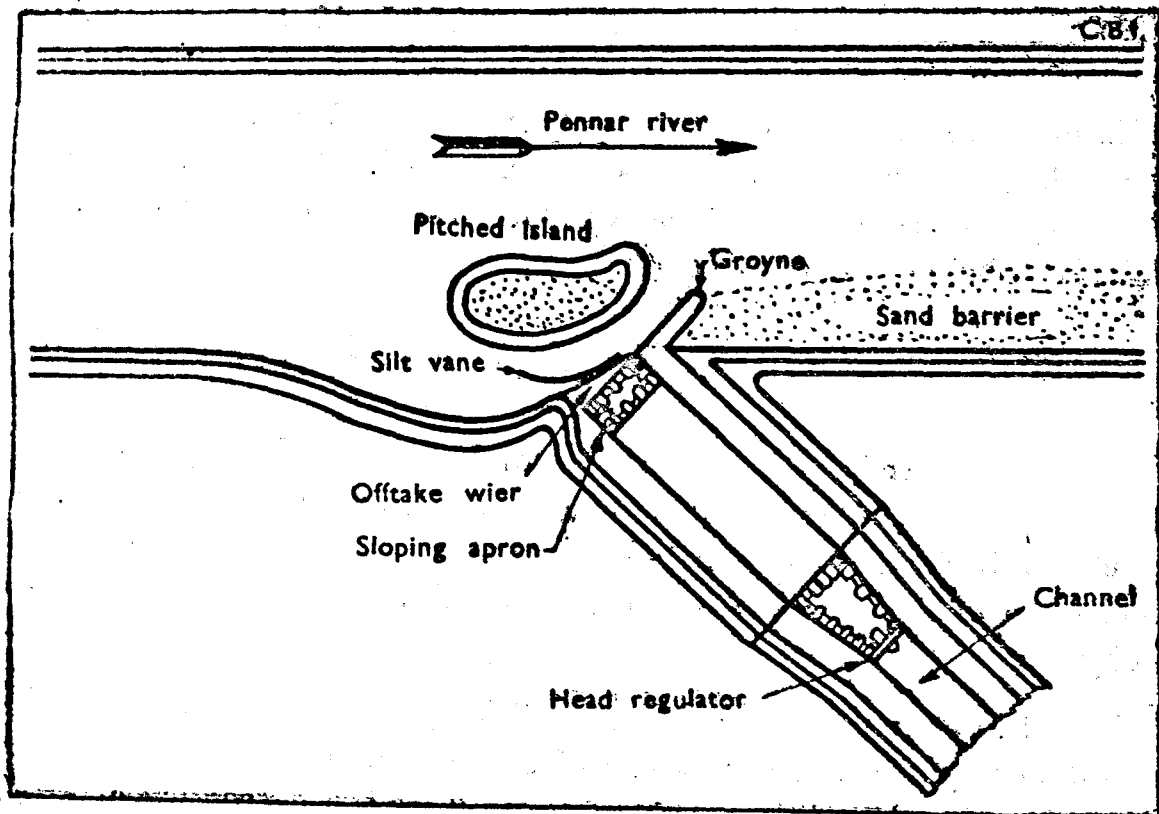


Figure 10 C.40 *Experiments for sand exclusion*

To prevent entry of the sand at the end, another small vane was constructed. A groyne wall was also built in continuation of the offtake weir. The results were very successful. Only 8.3 per cent. of the sand entering the by pass entered the channel. As a silt trap has also been given in the channel

before the head works, so that any sand that passed over the weir got deposited in it and the channel was practically free from sand.

In this connection a basic experiment was made to find out whether the rate of exclusion of sand depends on the amount of sand carried by the river. For this purpose sand was added to the flow at a uniform rate of six cubic feet per hour, eight cubic feet per hour, and 12 cubic feet per hour, the depth of flow in the river being maintained constant at three inches and velocity of flow uniform at two feet per second. The results are given in Table 10C.7.

TABLE 10C.7

Rate of sanding	6 cubic feet per hour	8 cubic feet per hour	12 cubic feet per hour
Percentage of exclusion	92.5 per cent.	91.7 per cent.	91.3 per cent.

(7) SEDIMENT INVESTIGATION—RIVER SUTLEJ AT HUSSAINIWALA HEADWORKS⁽⁸⁾

Excessive sediment enters the Dipalpur Canal at Head. To reduce it different designs of sediment excluders were examined. A design which gives an efficiency of 60 per cent. has been evolved. Work is being continued to secure higher efficiency.

(8) SEDIMENT INVESTIGATION—RIVER JUMNA AT TAJEWALA HEADWORKS⁽⁸⁾

The sediment excluder of two tunnels developed at the Malikpur Station and constructed at the head regulator of Tajewala Headworks in 1942, worked very efficiently for the given conditions. It was now desired to run the canal with higher river discharges than previously, and, therefore, to increase excluder capacity.

Three tunnels were examined. The efficiency over that of two tunnels increased only by a small percentage. In fact the efficiency of the two tunnels design under original conditions is 95 per cent. Work is being continued with high discharges in the pocket.

(9) SEDIMENT INVESTIGATION—RIVER RAVI AT DHANNA WEIR⁽⁸⁾

Heavy detritus is brought down by the river Ravi into the Upper Bari

(8) The Punjab Irrigation Research Institute, Lahore, Annual Report 1946, page 4 and 5.

Doab Canal. It was desired to control the sediment flow in the river upstream of the upper training works at the Dhanna Weir four miles upstream of the Madhopur Headworks. At this site the river divides into two channels one leading to the Madhopur pocket and the other to the waste weir. A series of shingle deflectors was constructed on the model at the Dhanna Weir such that comparatively sediment free water went to the Madhopur pocket channel. Further work is in progress to improve the design.

DISCUSSION BY THE RESEARCH COMMITTEE

MR. PRAMANIK in introducing item (1) said that these experiments were undertaken to find out a device by which exclusion of silt from the existing canal from the Rhondia Headworks on the Damodar, could be effected. To incorporate the reach from $7\frac{1}{2}$ miles above to two miles below the canal headworks, the model scale was chosen as 1/200 horizontal. The vertical scale was taken as 1/80. But it was found in this model that bed movement within one mile above the canal headworks did not start unless the discharge passed in the model was equivalent to 250,000 cusecs in the prototype. As the canal supply was stopped when the river discharge went above 2000,000 cusecs it was not possible in this model to try silt-entry conditions as in the prototype. This model was, therefore, utilised to study the nature of flow within one mile above the canal headworks, so that a bigger part model might be built for studying silt-exclusion problems, if necessary.

The vertical scale was later increased to 1/50 and bed-movement in this model had been found to start corresponding to 25,000 cusecs in the prototype. This model was now being run for studying silt-exclusion problems.

DR. BOSE in introducing item (2) said that these observations were started in conjunction with the model experiment on the silt exclusion from Damodar Canal whose results had been presented by Mr. Pramanik. While starting these observations on the headworks, it was found that the only place from where any sample of river water could be collected was at a point immediately upstream of the canal head regulator from the abutment wall. This was done by means of bottle sampler lowered to the desired depth in the pocket almost against the abutment of the head regulator. During the progress of these observations it was found that the quantity of silt in the sample obtained from the river water was always less than that obtained from the canal water. Though the result was, on the face of it, inconsistent, there was no other way of collection the water samples from the river and the present method was continued. Efforts had been made while analysing the data to rectify this drawback.

This had been done in the following way. It was well known that the silt intensity in the river water varied with the river discharge. This had been proved in many cases in other parts of the country. The silt content as actually obtained from the river by the present method was correlated with the river discharge. It was found that the correlation was very high and significant. Regression curve was also drawn. In further analysis, instead of taking the actual silt content of the river water the silt content as obtained from this regression curve for different river discharges was taken into consideration. In the investigations it had been assumed that there was a multiplying factor to the ratio between the silt intensity in the river water and the canal water though this multiplying factor was not known. Proceeding on this basis, the analysis had shown that by operating the undersluice gates in such a way that there was an increase in the opening from the canal regulator side away there was very much more silt entry into the canal as compared to the other method of opening the head regulator less and less from the canal regulator side. The results appeared to be contrary to what had been observed at other headworks. The same sort of result was also obtained from the model. As the results from the model as well as from the prototype appeared to contradict the present conception of silt exclusion these observations were being continued.

MR. FRAMJI said that it was stated that the canal was drawing excessive sand; it was also stated that the high spoil banks in the first five miles of the head reach slide down during rain and obstruct the free passage of water in canal. It was not clear, therefore, on what grounds it was believed that there was excessive sand-draw from the river. MR. FRAMJI asked if the hydraulic characteristics of the head reach, i.e. its slope, shape, section and divergence from regime, had been studied to see whether the silting was not a consequence of a defective canal design being unable to carry forward the material which had slid down from the spoil banks.

According to MR. FRAMJI the model must be such that there was correct curvature of flow and correct bed movement in all its reaches at the lowest discharge stage (30,000 cusecs) at which bed movement commenced in the river, because it was at this crucial stage that the ratio of q (offtake) : q (river) would be maximum and, therefore, at this stage exclusion would be most difficult.

If this experiment had to be done at Poona, even as a first trial scales with a vertical exaggeration greater than 2.5 would have been adopted, as the problem was essentially one of studying bed movements which depend on curvature of flow and required a higher exaggeration, say of the order of five or six. With the scales adopted, presuming the discharge scale was taken as $\frac{1}{LD\sqrt{D}} = 1/1, 43,000$, for the highest discharge of three *laks* tested on the model, the model discharge was of the order of 2.1 cusecs and the length of the weir being 3,750 feet the intensity q per foot run would be 0.112 cusec per foot, giving a Reynold's Number of about 11,000; for a discharge of one *lakh* cusecs in the river, the q per foot run in the model was only 0.037 giving Reynold's Number of 3,740. Although theoretically Reynold's Number of 2,000 was sufficient for giving turbulent flow it was their experience that Reynold's Number of 30,000 to 40,000 for the maximum discharge conditions was essential. But more increase of the depth scale to the extent proposed—i.e. increase in vertical

exaggeration from 2.5 to 4.0—without giving a slope exaggeration would not produce the necessary bed movements and a slope exaggeration was also essential for these tests.

The rigid portion of the weir, under sluices and head regulators were constructed to the same scale as the river portion. The discharge scale for the river was probably $1/LD \sqrt{D}$. This would not hold for masonry structures when they were vertically exaggerated and so the depth scale for rigid portion would be different from that for the river portion. The scales of the rigid portion should be determined as described in the note on 'Model Technique' in the 1945 Annual Report of the Indian Waterways Experiment Station.

MR. FRAMJI further asked if still pond regulation had been tried on the model or the prototype and with what results. The divide wall should be raised to R. L. 171 in order to enable still pond at the highest river discharge at which the canal was kept open.

With regard to the experiments on the prototype the observations were stated to have been done

- (1) with gates uniformly opened ;
- (2) with the undersluice gate opening increasing from the left to the right and the canal head regulator gates raised more and more from the upstream end.

Under (1) presumably both undersluice and regulator gates were equally opened so that the change made in (2) involved the simultaneous change of two variables. More conclusive results would be obtained by changing one variable at a time, say, first keeping the head regulator gates uniformly opened and the undersluice gates with wedge opening, i.e. the gates nearest to the head regulator having least opening ; and then raising the regulator gates more and more from the upstream and keeping uniform opening of the undersluice gates. Why was it not possible, asked MR. FRAMJI, also to try "still pond" regulation on the prototype, alternatively with canal gates uniformly opened and with varying openings.

DR. BOSE referred Mr. Framji to his report.

KHAN BAHADUR KARIM said that it was not clear at what stage the entry of silt into the canal was the maximum.

DR. BOSE suggested in reply to K. B. Karim's query that the movement of silt depended not only on the discharge but also on the rise of discharge in the river.

KHAN BAHADUR KARIM said that there must have been some closing level with the canal head regulators.

DR. BOSE in reply to this invited attention to the Table 10C. 1.

DR. BOSE further went on to say that it was not the quantity of silt that was giving the trouble. The reason for the silting was that during the rains the banks, which were rather very sloppy got dumped into the canal and the Government of Bengal had to spend something like Rs. 70,000 on this account. This kind of bank was for a length of five miles of the head reach. The soil there was peculiar. It cracked and during the rains the water got into the cracks. The only solution, according to DR. BOSE was the removal of these spoil banks.

RAI SAHIB GUHA asked as to whether the difficulty was due to the banks or the silt. He further asked where the dredged earth was disposed off.

MR. GOVINDARAO introduced item (6) and invited attention to Figure 10C.40 and added that small scale model experiments had been done since then.

MR. H. P. MATHRANI wanted to know that was the pavement level of the canal.

MR. GOVINDARAO explained that the pavement level was six feet below the bed of the river.

MR. H. P. MATHRANI said that in that case the canal would suck all the bed material.

MR. GOVINDARAO said that was the purpose of these experiments.

RAO BAHADUR JOGLEKAR said that it had been suggested in the paper that the model experiments were done in geometrically similar models. He contended that this experiment as explained could not be done in geometrically similar model.

MR. GOVINDARAO replied that because of the peculiar features of the river it had been possible to do the same.

DR. UPPAL described the double decantation method followed in Punjab. After working on these experiments for some years he said they had been able to find out suitable sites on canals three to four miles upstream of the head works, where there was a bifurcation. A deflector was put at the site. The deflector deflected most of the sand. The second decantation was done at the head works.

On account of this design there had been a change from the old designs which were followed in the Punjab in the previous years.

MR. FRAMJI referred to item (7) and stated that the design is claimed to give an efficiency of exclusion of 60 per cent. from Dipalpur Canal. No details of the design were furnished. It would be interesting to know how efficiency of exclusion was measured ?

Regarding item (8) he asked, what was the present discharge of the Western Jumna Canal and what was the proposed capacity ?

What were the present and proposed excluder capacity and how was it proposed to increase the excluder capacity ?

Referring to item (9) MR. FRAMJI stated that no plan had been furnished so one could only infer from the photograph the salient features of the proposal. In this paragraph a curved vane (or effectively a curved raised sill in this case) had been misnamed 'deflector'. It was stated that the comparatively sediment-free water went to the Madhopur pocket.

What was the efficiency of exclusion shown by the model, he asked, and whether the same efficiency was expected in the prototype ? Such a raised sill could only be partly successful with its favourable curvature upstream of it as model experiments have shown that it created turbulence and coarse bed material rolls over it. MR. FRAMJI asked what was the height above bed of the so-called deflector compared to the normal depth of the river during medium flood ?

MR. GOLE asked Dr. Uppal what the height of the deflector was.

MR. CHATURVEDI asked Dr. Uppal what the distribution was.

DR. UPPAL replied that he preferred large scale models to distorted ones. He said that the height of the deflector was $d/4$ where d was the depth of water at bankful stage.

Items (3), (4) and (5) were left over for discussion at the next Board Meeting.

DISCUSSION BY THE BOARD

THE SECRETARY said that there were nine items for discussion at the Research Committee Meeting out of which items (3), (4) and (5) could not be taken up. He suggested that these might be discussed by the Board.

MR. SETHNA introduced item (3) and said that at the time of the construction of the barrage it was proposed that it would be working on open flow conditions. Afterwards it was found that free flow within the right bank canals was not possible. Upto 1939 the right bank canals silted badly. The Director, Poona Research Station (Sir Claude Inglis) was requested to perform model experiments to determine means for rectifying these errors. When the work was under construction, the Director had performed some experiments, and he suggested two submerged vanes to prevent the silt entering the right canals and to make the flow more or less central. This was not acted upon. Afterwards the conditions grew worse and finally the Director suggested the construction of an island in the right half of the weir as shown in Figure 10C.9.

After construction of the island, the canal on the right bank started to scour out and so far as the canals were concerned the arrangement was functioning all right.

He said that the dis-advantages were not so many as compared to the advantages.

In reply to a question MR. SETHNA said that 18 spans of the barrage were out of action on account of the island.

SARDAR BAHADUR SARUP SINGH said that that was an inefficient way of silt exclusion.

In reply it was pointed out that the weir had been designed for a flood discharge of 1.5 million cusecs but was not likely to get even a million cusecs. The blocking of 18 spans was therefore, no real loss.

MR. FRAMJI congratulated Mr. Sethna on a fair and balanced note. In Table 10C.2 he had given brief details of experiments carried out at Poona. The results of these experiments had been confirmed by actual observations on the prototype not only in respect of the main purpose of the works—sand exclusion—but also in respect of other important features. For example, as had been predicted, experience had shown that while 'still-pond' or 'semi-open' regulation made little difference to sand exclusion from the Left Bank Pocket on account of the existing favourable curvature, on the Right Bank Pocket, with its adverse curvature, still-pond regulation was essential.

MR. FRAMJI continued to say that the immense good that the river works had done must not be lost sight of. Conditions in respect of silting had got so bad by the end of 1938 that a large expenditure had to be incurred on silt clearance of the Right Bank Canals and the Right Pocket. Not even the worst critic of these works now denied that so far as sand exclusion was concerned the new Approach Channel had been completely successful. The total cost of the works was $8\frac{1}{4}$ lakhs of rupees which compared favourably with the heavy recurring cost of clearance of the Right Bank Canals and Pockets which had been saved, as judged by the cost incurred in one year 1938-39 with so little lasting good.

MR. FRAMJI answered some of the criticisms made against the exclusion works—

Though this design had reduced active river spans from 54 to 36 and had masked or particularly reduced a few others, there was still ample capacity left to pass the maximum discharge ; the intensity required to pass the maximum recorded discharge would be about 290 cusecs per foot, with C in the formula $q=CD^{1.5}$ only 2.7, which was much lower than that adopted in the latest barrage designs.

Secondly, it was stated that when the river was *low*, on account of the flatter slope on the river side than on the Approach Channel side, a higher pond level had to be maintained. This was only necessary at river stages of the order of two lakhs cusecs and under, when there was little or no bed movement in the river and so no conceivable harm could come from higher ponding at such times.

Thirdly, it was stated that an afflux of 0.2 foot to 0.5 foot was necessary during floods to clear away sand deposits downstream of the tail channel to enable it to pass the required discharge.

An afflux of 0.5 foot for only a few hours of flow was sufficient to restore the discharging capacity of the tail channel. There was no evidence that in these few hours any irreparable harm was done. But even this feature could be substantially reduced, if not eliminated, by the provision of a downstream divide wall carried above high Flood Level with openings and a submerged curved vane in extension thereof, as found efficient in experiments of the Kotri Barrage.

Fourthly, it had been stated that though the bed of the Rice Canal had now scoured down to, or near the designed bed level, yet the slope in this canal was still high. This was because coarse sand, the residue of several feet of scoured bed material, still remained exposed on the bed. This concentration of coarse sand must be removed and the velocity reduced before the canal could be run with the new flatter slope now possible on account of the exclusion of the bed sand.

Lastly, the Approach Channel had been charged with accelerating the accretion. This was a very controversial issue over which MR. FRAMJI would rather reserve his opinion for the proper occasion.

MR. FRAMJI introduced items (4) and (5) and said that as Executive Engineer, in charge of the Lower Sind Barrage Project Division, he had strongly

pressed for the shifting of the site from Hajipur to Kotri, 23 miles upstream of the former which suggestion had been accepted after much discussion and consideration. This item dealt with experiments with the Barrage at Kotri in the 1/250 : 1/50 vertically-exaggerated model to test various features of design, particularly in their bearing on sand exclusion.

A long series of experiments were carried out first of all without a Barrage to prove the model and the stability of the site. These showed that the proposed Barrage line was somewhat upstream of the reach where the curvature effect became fully established and the existing offtake of the Fuleli at Jamshero was unsuitable in respect of its position along the curve, particularly for high discharges of Fuleli at low river stages.

Experiments carried out with a Barrage (at the site at Kotri proposed in the Project) showed that—

(1) sand would be excluded from the Pinyari Feeder offtaking from the pocket at the Barrage ; but

(2) exclusion from the Fuleli Head at Jamshero would be very unsatisfactory because of—

(a) its unsuitable position along the curve ;

(b) partial counter-action of natural curvature by the Barrage ;

(c) high canal discharge in proportion to river discharge ; particularly at the low river stages (*e.g.* two *lakhs* cusecs, the lowest stage at which the bed movement occurs in the river) ; and

(d) attraction of flow by the island resulting in reduced flow along the left bank.

In order to restore curvature and improve sand exclusion from the Fuleli, various measures were tested but none was sufficiently satisfactory as an all-time solution for exclusion of sand from Fuleli.

It was only when the Fuleli Head Regulator was shifted along side of the Pinyari Head Regulator into the Left Pocket that there was sufficient promise of satisfactory sand exclusion from the canal. The Project design was accordingly modified and a similar design recommended to Sind which had been accepted.

The Central Island was a special feature of this design. This island—to be distinguished from the pitched 'attracting island' for training a river such as at Suleimanki—was found eminently successful for its designed purposes of creating favourable curvature on both banks, and securing the desired control and distribution of flow in the two arms of the river. It also precluded the possibility of the deterioration of the right arm channel of the river ; as experiments with exaggerated shoaling of the right bank showed, the central island would keep open the requisite channel even without artificial assistance by regulation.

Model experiments were undertaken to compare qualitatively, different shapes of this Central Island and its nose to give minimum depth and extent of scour and cause less "attraction" and turbulence at the nose. Profile VII was found to be the best. In this profile, two stage transitions were provided, viz. from 6 : 1 at the nose to 3 : 1 at an intermediate point and from 3 : 1 to 2 : 1 near the abutment for the left side, and 6 : 1 constant for a longer length from the nose and then transition from 2 : 1 at the abutment for the right side.

This profile was adopted in the full-width model of the Kotri Barrage and was being tested under the curvature inherent in the site.

DR. BOSE wanted to know as to what would happen to the channel on the right of the island when a sudden flood occurred.

MR. FRAMJI replied that the right hand channel opened out. The danger was only when the water level went down.

DR. BOSE wanted to know the rate of rise of the river.

MR. FRAMJI referred Dr. Bose to Figure 10C.12 where a cycle was given.

It was decided to keep the subject on the agenda.

11C—Outlets o - Sluices

PRELIMINARY NOTE

This subject has been introduced on the agenda for the first time.

The subject has been dealt with comprehensively in the Punjab Engineering Congress paper No. 264 by Messrs. Mahbub and Gulhati. It is understood that the authors propose to re-publish this work in book form after making considerable additions and modifications to their Punjab Engineering Congress paper.

The Year's Work

DISCUSSION BY THE RESEARCH COMMITTEE

MR. BLENCH stated that experiments on A. P. M. flumes, their coefficient and their adaptability for bigger heads, were being carried out for the last two years, but there were at present no special features to report.

CONSIDERATION BY THE BOARD

THE SECRETARY said that there was no contribution under this subject head.

It was decided to keep the subject on the agenda.

12 C—Other Works

PRELIMINARY NOTE

This subject has been introduced on the agenda to facilitate discussion of the design of canal works other than those which are included on the agenda as separate items.

The Year's Work

The following items were discussed at the 1947 Research Committee Meeting:—

(1) Fluming the Tungabhadra Low Level Canal between miles 27/4 and 28/4.

(2) Reduction of afflux at sluices, culverts and inverted siphons.

(3) Siphoning of the Cumbum Tank sluice.

(1) FLUMING THE TUNGABHADRA LOW LEVEL CANAL BETWEEN MILES 27/4 AND 28/4⁽¹⁾

ABSTRACT

The Tungabhadra low level canal is proposed to be flumed in the reach 27/4 to 28/4 where it has to pass through a cutting in soft rock. The hydraulic particulars for the flumed section were obtained by analytical calculations. These are furnished. In order to examine how far reliance could be placed on such calculations model experiments were carried out on the model of the flumed canal to a natural scale of 1/20 with the difference that instead of having the full length of 5,308 feet reproduced in the model only 800 feet was reproduced.

The procedure adopted was as follows:—

(a) Construct the model to the proper dimensions, allow the discharge corresponding to the full supply of the canal and adjust the tail water to the correct level; (b) for this condition calculate theoretically the elevations of water surface and depths at the four critical positions, viz., (1) approach flume, (2) head of throat, (3) end of throat and (4) departure flume; (c) observe the corresponding values actually obtaining in the model; (d) compare the two sets of values and see how far the computed values tally with the observed ones.

A series of experiment were carried out and in all these the flow was observed to be quite smooth and steady and the characteristic fall in level between (1) and (2) and between (2) and (3) and rise between (3) and (4) were noticed. Computed and observed values for one experiment are furnished. Close agreement was obtained between the two sets of values which showed that reliance could be placed on computed values in fluming long lengths of canals.

DESIGN FOR THE FLUMED SECTION

Calculations for the flumed section of the prototype are furnished below:

Data—

$$n=0.020$$

Discharge = 1,512 cusecs

Depth of flow in normal channel = 8½ feet

Bed width = 66 feet

(1) Irrigation Research Station, Pondi, Madras. Annual Report 1946 Pages 44 to 52.

The section is proposed to be rectangular at the entrance to the approach transition.

Froude number.—The main consideration in this design is to see that no standing wave will be formed.

Hence, Froude number should be less than 1

$$\frac{V^2}{gL} \text{ should be } < 1$$

Let the throat be 24 feet and depth of flow 10 feet

$$V = \frac{1,512}{24 \times 10} = 6.302 \text{ feet per second}$$

$$\frac{V^2}{gL} = \frac{(6.301)^2}{32 \times 10} = 0.1241$$

Hence the proportions suggested are suitable. The approach transition is proposed to converge at 1 to 1 slope and departure transition diverge at 1 to 5 slope.

Figure 12.C. 1 illustrates the plan and section of the flume.

It is convenient to start the computations from the tail end of the flume and let us assume for convenience the level at section (4) as 100.00. This value can finally be modified to correspond to the reduced level at the site.

Bed level = 100.00
 Water surface level = 100.00 Bed level+depth
 8.50

108.50

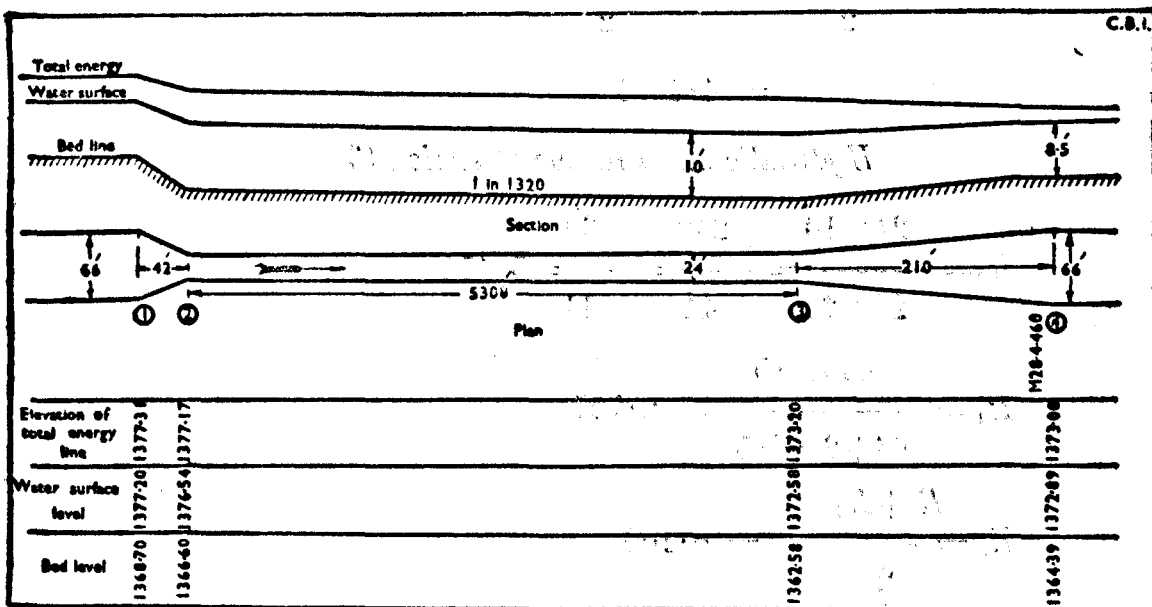


Figure 12.C.1 : Fluming of Tungabhadra Low-Level Canal between M 27-4-180 and M. 28-4-460 (Prototype).

(Sketch not to scale)

[N.B.—Actual dimensions of canal shown in plan]

$$\text{Velocity at section (4)} \quad V_4 = \frac{1,512}{66 \times 8.5} = 2.696$$

$$\frac{V_4^2}{2g} = 0.1136$$

$$\begin{aligned} \text{Elevation of total energy line at (4)} &= 108.5 + 0.1136 \\ &= 108.6136 \end{aligned}$$

Losses in departure transition

Loss of head due to impact and friction

$$\frac{0.25 (V_3^2 - V_4^2)}{2g} + \frac{V_m^2 L}{C_m^2 R_m} \quad \text{where } V_m \text{ is the mean velocity, } L \text{ is the length, } C_m \text{ is mean coefficient and } R_m \text{ is the hydraulic radius.}$$

$$\text{Here } V_3 = \frac{1,512}{24 \times 10} = 6.301 \text{ feet per second}$$

The depth in the throat $d_3 = 10$ feet as assumed in the beginning
 $V_4 = 2.696$ feet per second.

$$\begin{aligned} V_m &= \frac{V_3 + V_4}{2} = \frac{6.301 + 2.696}{2} \\ &= 4.4985 \end{aligned}$$

Hydraulic mean radius at section (3)

$$R_3 = \frac{24 \times 10}{24 + 20} = \frac{240}{44} = \frac{60}{11} = 5.454$$

$$R_4 = \frac{66 \times 8.5}{66 + 2 \times 8.5} = 6.757$$

$$R_m = \frac{R_3 + R_4}{2} = 6.106$$

From Manning's formula :

$$\begin{aligned} C_3 &= \frac{1.486}{n} (R_3)^{\frac{1}{6}} = \frac{1.486}{0.020} \times (5.454)^{\frac{1}{6}} \\ &= 98.60 \end{aligned}$$

Similarly

$$C_4 = \frac{1.486}{0.020} \times (6.754)^{\frac{1}{6}}$$

$$= 102.2$$

$$C_m = \frac{C_3 + C_4}{2} = 100.4$$

Loss in departure transition :

$$\frac{0.25 (V_3^{\frac{2}{3}} - V_4^{\frac{2}{3}})}{2g} + \frac{(V_m^2 L)}{C_m R_m} \quad \text{Here } L = \text{length of departure transition 210 feet}$$

$$\frac{0.25 (6.301)^2 - (2.696)^2}{2g} + \frac{(4.499)^2 \times 210}{(100.4)^2 \times 6.106}$$

$$= 0.1958 \text{ or } 0.20 \text{ foot}$$

Elevation of total energy line at (3)

Elevation at (4) + loss in transition (3) to (4) —

$$108.6136 + 0.20$$

$$= 108.81$$

Specific energy at (3) —

$$d_3 + \frac{V_3^2}{2g}$$

$$= 10 + \frac{(6.301)^2}{2g}$$

$$= 10.6203$$

Bed level at (3) —

$$108.81 - 10.62 \quad (\text{Elevation of total energy line — specific energy})$$

$$= 98.19$$

Elevation of water surface at (3) —

$$98.19 + 10 = 108.19$$

Loss of head in the throat :

$$\text{Friction loss in throat from Chezy's equation} = \frac{V^2 L}{C^2 R}$$

Assuming uniform flow $V_2 = V_3$ and $d_2 = d_3$

$$\text{Loss of head in throat, i.e. from (2) to (3) } \dots \frac{V_3^2 L}{C_3^2 R_3}$$

$$= \frac{(6.301)^2 \times 5,308}{(98.6)^2 \times 5.454}$$

$$= 3.974$$

Hence the bed fall in throat should be 3.974 feet in 5,308 feet or assuming 1 in 1320 as proposed in the original design the bed fall works out to $\frac{5,308}{1,320} = 4.02$

feet against 3.974 required. Hence the assumed bed fall of 1/1320 might be accepted.

Elevation of total energy line at (2)

Elevation at (3) + losses in throat

$$108.81 + 3.974$$

$$= 112.78$$

$$\text{Bed level at (2)} = \text{Bed level at (3)} + \text{Bed fall } 98.19 + 4.02$$

$$= 102.21$$

$$\text{Specific energy at (2)} = 10.57$$

Hence the depth at (2) has to be obtained by trial so that

$$\frac{d_2 + V_2^2}{2g} = 10.57$$

$$\text{Let } d_2 = 9.93$$

$$V_2 = \frac{1,512}{24 \times 9.93} = 6.346$$

$$\text{and } \frac{V_2^2}{2g} = 0.63$$

Hence the depth d_2 might be taken as 9.94 in which case $9.94 + 0.63 = 10.57$ as required.

Elevation of water surface at (2)

$$\text{Bed level} + \text{depth of water} = 112.15$$

$$102.21$$

$$9.94$$

$$112.15$$

Loss of head in the approach transition

$$0.25 \times \left\{ \frac{V_2^2 - V_1^2}{2g} \right\} \frac{V_m^2 \times L}{C_m^2 \times R_m}$$

$$L = 42 \text{ feet}$$

$$V_1 = V_4$$

$$\frac{V_2^2}{2g} = 0.63$$

$$0.25[(0.63) - 0.1136] + \frac{0.06907 \times 42}{210}$$

$$= 0.14 \text{ feet}$$

Elevation of total energy line at (1)

Elevation at (2) + loss of head in approach transition :

$$112.78 + 0.14$$

$$= 112.92$$

Specific energy at (1) = that at (4). (Since $d_1 = d_4$ and widths are same)

$$= 8.61$$

$$\text{Bed level at (1)} = 112.92 - 8.61$$

$$= 104.31$$

$$\text{Water surface elevation at (1)} = 104.31 + 8.5$$

$$= 112.81$$

at the bed level at M. 27—4—180 or section (1) is given as B.L. 1368.70 against 104.31 obtained above. Hence the datum of all levels is to be raised by 1,264.39.

Table 12C.1 shows the levels at various sections as calculated and also the corrected reduced levels :—

TABLE 12C. 1

With datum of B.L. Section (4) as 100.00				Reduced levels corrected				Remarks
Sections				Sections				
(1)	(2)	(3)	(4)	(1)	(2)	(3)	(4)	
104.31	102.21	98.19	100.00	1,368.70	1,366.60	1,362.58	1,363.39	Bed level
112.81	112.15	108.19	108.5	1,377.20	1,376.54	1,372.58	1,372.89	Water surface elevation.
112.92	112.78	108.81	108.61	1,377.31	1,377.17	1,373.20	1,373.00	Total energy line elevation.

DESCRIPTION OF THE MODEL

A model of the flumed canal to a scale of $\frac{1}{20}$ was constructed as shown in the Figure 12C.2 . But in the model the throat was only 49 feet long corresponding to 800 feet in prototype whereas the actual length of throat in the prototype is 5,308 feet. It was neither convenient to provide the complete length in the model nor was it necessary for the present purpose which was mainly to find how far observed and computed values tallied in fluming of this kind.

* This value can be written down from the frictional losses in the departure transition as all quantities are same except length.

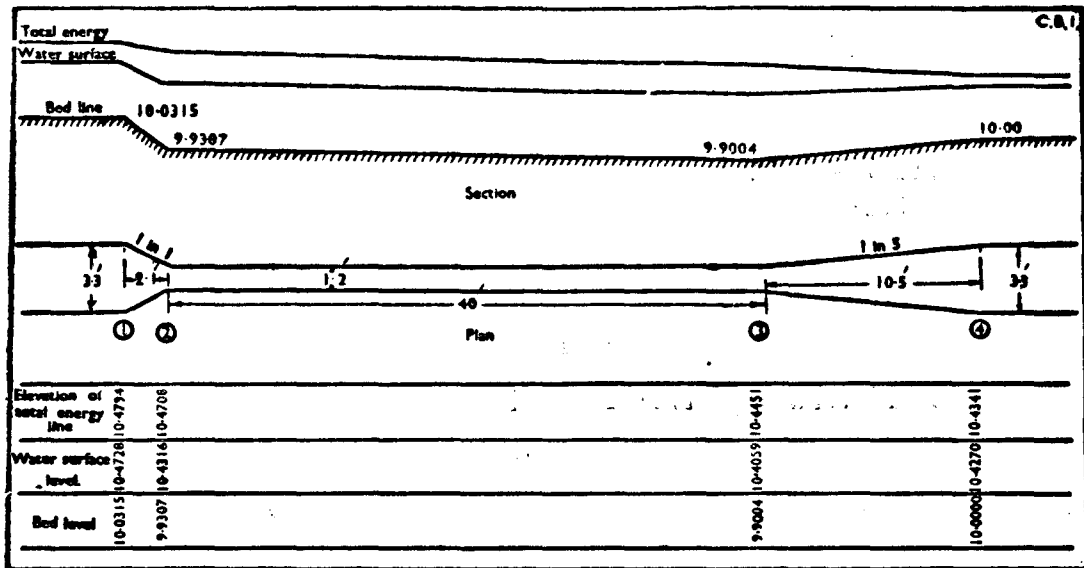


Figure 12C.2 : Model Experiments on Fluming of Tungabhadra

Low-Level Canal between M. 27-4-180 and 28-4-460 (model)
(not to scale)

N.B.—Actual dimensions of model shown in plan

The flume was constructed in masonry plastered very smooth with cement mortar. Its normal width was 3.3 feet which got reduced to 1.2 feet in the throat. The approach transition converged at 1 to 1 slope in plan and the departure transition diverged at 1 : 5 slope. The invert of the flume was constructed with a fall of five per cent. in the approach transition and a rise of 1 per cent. in the departure transition. These approximately corresponded to the prototype values deduced above. The actual levels of the bed after construction of the model were finally measured and the exact values recorded. The supply to the flume was gauged over a measuring weir and there was a tail gate to adjust the necessary depths of flow for any discharge. In the final experiment discharge of 0.95 cusecs was passed corresponding to 5,373 cusecs in prototype and the tail water level was adjusted to 0.427 feet corresponding to 8.54 feet in prototype. The sections (1), (2), (3) and (4) were the critical sections for the study. In locating the piezometers in the side walls of the flume at the sections marked it was found advisable to locate the piezometers (1) and (3) one foot upstream and the piezometers (2) and (4) one foot downstream of the respective sections. This ensured steady conditions of flow at the points of observation. The piezometer points were connected by pressure rubber tubing and communicated the pressures to a set of brass cans all assembled in a central pit. A single pointer gauge travelling on a level rail from which the rise and the drop of water surface between the respective sections were computed. The pointer gauge was also placed at the various sections and the actual depths of water noted separately.

Computation of theoretical water surface elevations for the discharge allowed is given below:—

Let the bed level at section (4) be 10.00.

The bed levels at other sections as obtaining in the model are shown in Figure 12.C.2.

Bed level section (3) 9.9004

Do. (2) 9.9307

Do. (1) 10.0315

Discharge 0.95 cusec

Depth at section (4) = 0.427 foot

Velocity at section (4) = $V_4 = \frac{0.95}{3.3 \times 0.427} = 0.6742$ foot per second

Specific energy at section (4) =

$$\begin{aligned} d_4 + \frac{V_4^2}{2g} &= 0.427 + \frac{(0.6742)^2}{2g} \\ &= 0.427 + 0.007103 \\ &= 0.4341 \end{aligned}$$

Elevation of total energy line = 10.4341

Elevation of water surface of section (4) = 10.427 (bed level + depth) = 10.427

Loss of head in the departure transition

Loss of head due to impact and friction :

$$0.25 = \frac{(V_3^2 - V_4^2)}{2g} + \frac{V_m^2 \cdot L}{C_m^2 R_m}$$

Let us assume the depth $d_3 = 0.5$ foot or (10 feet in prototype). The exact depth obtaining in the model will be checked in the end.

$$V_3 = \frac{0.95}{1.2 \times 0.5} = 1.583 \text{ feet per second}$$

$$V_m = \frac{V_3 + V_4}{2} = \frac{1.583 + 0.6742}{2} = 1.1288 \text{ feet per second}$$

$$R_3 = \frac{1.2 \times 0.5}{1.2 + 2 \times 0.5} = \frac{0.6}{2.2} = 0.2727$$

$$R_2 = \frac{3.3 + 0.427}{3.3 + (2 \times 0.427)} = 0.3398$$

$$R_m = \frac{0.6119}{2} = 0.3059$$

$$C_3 = \frac{1.486}{n} (R_3)^{\frac{1}{2}} \quad \dots \quad \text{Let } n = 0.01 \text{ for the cement plastered smooth surface}$$

$$= \frac{1.486}{0.01} \times (0.2727)^{\frac{1}{2}}$$

$$= 119.7$$

$$C_4 = \frac{1.486}{n} (R_4)^{\frac{1}{2}}$$

$$= \frac{1.486}{0.01} \times (0.3392)^{\frac{1}{2}}$$

$$= 124.1$$

$$C_m = \frac{243.8}{2} = 121.9$$

Loss in departure transition :

$$= \frac{0.25}{2g} (1.583^2 + 0.6742^2) + \frac{(1.1386)^2 \times 10.5}{(121.9)^2 \times 0.3059}$$

$$= 0.008013 + 0.00301$$

$$= 0.0110 \text{ feet}$$

Elevation of total energy at section (3)

T.E. at section (4) + Loss of head

$$10.4341 + 0.0110$$

$$= 10.4451$$

Specific energy 10.4451 — Elevation of bed (9.9004)

$$= 0.5447$$

For the assumed depth of 0.5 foot the specific energy separately works out :

$$d_3 + \frac{V_3^2}{2g}$$

$$0.5 + \frac{(1.583)^2}{2g}$$

$$0.5392 \text{ against } 0.5447 \text{ above}$$

Let depth be (0.5055)

$$\begin{aligned} & \text{Specific energy} \\ & \quad (1.583)^2 \\ & 0.5055 + \frac{\quad}{2g} \\ & \underline{=0.5447} \end{aligned}$$

It might be noted that due to the altered depth, *viz.*, 0.5055 instead of 0.5000 it is not necessary to compute the velocity separately as the variation will not be appreciable.

Hence depth $d_2 = 0.5055$ foot

$$\begin{aligned} \text{Elevation of water line at (3)} & = \text{Bed level} + \text{depth} \\ & \quad 9.9004 + \\ & \quad \underline{0.5055} \\ & \underline{=10.4059} \end{aligned}$$

$$\begin{aligned} \text{Hence rise in water surface between sections (3) and (4)} \\ & \quad 10.4270 - 10.4059 \\ & \quad = 0.0211 \text{ feet} \end{aligned}$$

against 0.026 feet as observed in the model
Loss of head in the throat :

$$\begin{aligned} \text{Frictional loss} & \frac{V_3^2 L}{C_3 \cdot R_3} \text{ assuming uniform flow} \\ & \frac{(1.583)^2 \times 40}{(119.7)^2 \times (0.2727)} \quad (L=38 \text{ feet between pressure observations points ; but for calculation purposes the theoretical length between critical section, } \textit{viz.}, 40 \text{ feet is taken).} \\ & = 0.02565 \end{aligned}$$

$$\begin{aligned} \text{Elevation of total energy line at section (2)} \\ & \quad 10.4451 + 0.02565 \\ & \quad = 10.4708 \end{aligned}$$

$$\begin{aligned} \text{Specific energy at section (2)} & 10.4708 - \text{Bed level} \\ & \quad = 10.4708 - 9.9307 \\ & \quad = 0.5401 \end{aligned}$$

$$\begin{aligned} \text{Depth at section (2) (specific energy)} & \frac{V_2^2}{2g} \\ & = 0.5401 - 0.0392 ; (V_2 = V_3 \text{ nearly}) \\ & \underline{= 0.5009} \end{aligned}$$

$$\begin{aligned} \text{Elevation of water surface at (2)} & = 9.9307 + 0.5009 \text{ (Bed level} + \text{depth)} \\ & \underline{= 10.4316} \end{aligned}$$

Fall in water surface between (2) and (3) 0.0257 against
0.022 observed

Loss of head in the approach transition

$$\left[\frac{0.25V_2^2 - V_1^2}{2g} \right] + \frac{V_m^2 L}{C_m^2 R_m}; L=2.1$$

$$0.008013 + \left(\frac{0.00301 \times 2.1}{10.5} \right)$$

N.B.—These values are taken from the calculations of loss of head in departure transition.

$$0.008013 + 0.000602 = 0.008615$$

Elevation of total energy line at (1) :

$$10.4794$$

$$\left\{ \begin{array}{l} \text{loss} + \text{Elevation at (1)} \\ 0.0086 \\ 10.4708 \\ \hline 10.4794 \end{array} \right.$$

Specific energy at (1) $10.4794 - \text{Bed level}$
 $10.4794 - 10.0315$
 $= 0.4479$

Let depth at section (1) 0.4413 (This was of course obtained by previous trials.)

$$d_1 + \frac{V_1^2}{2g} = 0.4413 + 0.0066$$

$$= 0.4479$$

$$\left\{ \begin{array}{l} V_1^{0.95} \\ 3.3 \times 0.4413 \\ 0.6509 \end{array} \right.$$

Hence depth $d_1 = 0.4413$

Elevation of water surface at (2) $10.0315 + \text{W.L. at (1)} = 10.4728$
 $0.4413 \text{ do. at (2)} = 10.4316$
 $= 10.4728 \qquad \qquad \qquad 0.0412$

Surface fall in (1)—(4) 0.0412 against
0.033 observed

TABLE 12 C 2

Water surface elevations with bed level at (4) as + 10.00

Description	Sections			
	1	2	3	4
Bed levels	10.0315	9.9307	9.9004	10.00
Total energy line elevation	10.4794	10.4708	10.4451	10.4341
Water surface elevation	10.4723	10.4316	10.4059	10.4270

The water surface elevations obtained by calculation are abstracted in Table 12.C.2 and a comparison of the differences in elevations between critical points as obtained by calculations and as observed on model are furnished in Table 12.C.3. The agreement between the computed and observed difference is close and is within the limits of experimental error.

TABLE 12 C 3
Differences in water surface elevations

Values in feet									
Sections									
Computed									
Observed									
Remarks									
(1)—(2)	0.412	0.033	Fall
(2)—(3)	0.0257	0.022	Fall
(3)—(4)	0.0211	0.026	Rise

The least count of the gauges used in the tests was 0.001 foot. If more precise instruments were used and the exact value of 'n' for the surface is determined from preliminary experiments a closer agreement could be expected. It is important to note that the water surface had a drop in the approach transition and in the throat and had a rise in the departure transition. Both the computed and observed values denote this characteristic. It can, therefore, be concluded that reliance can be placed on the computed values in the fluming of canals involving long lengths.

(2) REDUCTION OF AFFLUX AT SLUICES, CULVERTS AND INVERTED SIPHONS (2)

ABSTRACT

Possibilities of reducing afflux caused at culverts, siphons, aqueducts, sluices *etc.*, working under pressure by simple devices such as streamlining the entry and exit were investigated on a model. The device contemplated had to be simple and easily adaptable to existing structures without involving radical modifications to their components.

THE PROBLEM

When the normal waterway of a stream is restricted by any control, afflux is caused in order to enable the coming discharge to pass through the structure with increased velocity. The afflux noted consists of the loss of head (1) at entrance to the barrel, (2) at the outlet, and (3) frictional losses through the barrel. Since no modification to the main structure was contemplated no attempt to minimize item (3) was made. A bell-mouthed sill may minimize the losses at entrance due to sudden contraction and smooth diverging walls at the outlet may diminish the loss of head due to sudden expansion at the outlet. The reduction effected will of course be small; but even this will have practical application in many cases. In remodelling channels and drains to carry extra discharges than originally contemplated we do come across existing culverts

(*) Irrigation Research Station, Poondi, Madras, Annual Report, 1946, pages 79 to 80.

where a simple device for reducing afflux ma considerable economy. In the case of long barrelled sluices these devibe adopted for increasing the discharge to a small extent. These are also for increasing the velocity of flow in inverted siphons which may be des for several reasons.

MODEL DESIGN AND CONSTRUCTION

A model culvert with a rectangular opening 1.0 foot \times 0.55 foot and 3.0 feet long was constructed in a masonry flume 2.0 feet wide having a level bed plastered over smooth with cement plaster. The model was supposed to be of scale 1/10 of an hypothetical slab culvert having a rectangular barrel 10 feet \times 5.5 feet and 30 feet long. The water-supply to the flume was gauged over a precalibrated, rectangular weir 2.0 feet wide before admission to the model. The water was stilled by screens on approach to the model and its levels downstream were adjusted by means of a tilting tail gate in the rear end of the flume. The water levels were read in gauge wells located $3\frac{1}{2}$ feet upstream and $3\frac{1}{2}$ feet downstream of the models by pointer gauges reading to 1/1,000 foot.

OPERATION OF THE MODEL

Every time an experiment was taken up the tail gate was raised and water admitted to the model till the barrel was all submerged. The supply was then cut off and water allowed to find its level under hydrostatic conditions, leaks being sealed. A pointer gauge P was installed across the flume. The water surface elevation was read on the upstream and downstream gauges and on the gauge P thus connecting the three gauge readings. The bed level of the flume was then read on gauge P.

Discharges varying from 1.2 to 2.2 cusecs corresponding to 380 to 700 cusecs in prototype were passed and for each discharge about half a dozen conditions of submersion corresponding to tail water levels varying from six feet to ten feet were tried.

The first series of experiments was conducted on the normal culvert. Modifications were then introduced in stages to notice the individual efficiency of the devices. Lastly their performance in combination was observed. An abstract of the observations made tabulated for easy reference is furnished in Table 12.C.4.

DISCUSSION

In any experiment for a single discharge there were different conditions of submergence tried. It was noticed that as long as the submerged condition was obtained the actual depths upstream and downstream had no marked effect on the value of afflux. Hence a mean of the afflux values for any discharge was taken for purpose of comparison. These are tabulated in Table 12.C.4. Referring to this statement it may be noticed, that for any particular discharge the following was the order of efficiency of the devices tried, viz., (1) bell mouth on three sides and rear expansion of 1 in 5 downstream, (2) bell-mouthed sill on three sides on (3) rearly, expansion only and (4) bell mouthed sill on top only.

TABLE 12.C 4

Statement showing affluxes at the culvert model with various devices tried

Discharge (cusecs)	Plain vent		Top bell mouth only		Rear expansion 1 : 5		Bell mouth on three sides		Bell mouth on three sides with rear expansion	
	Afflux (feet)	Reduction percent- age	Afflux (feet)	Reduction percent- age	Afflux (feet)	Reduction percent- age	Afflux (feet)	Reduction percent- age	Afflux (feet)	Reduction percent- age
700	2.9	..	2.75	5.1	2.6	10.2	2.45	15.5	2.35	19
600	2.16	..	2.07	4.1	1.95	9.73	1.88	13	1.79	12.5
500	1.45	..	1.4	3.4	1.3	10.3	1.27	12.4	1.2	17.2

Note : all values relate to prototype

In adopting the devices however it should be noted that they are not uniformly applicable to all situations. In the case of single vent culverts all the devices above stated are adaptable. Where there are more than one span the rear expansion can be had only at the abutments. The bell-mouthed sill on top is adaptable to all situations. But the bell-mouthed sill on three sides is applicable only when there are single vents. The principle could however be applied where there are multiple spans by stream lining the cut waters of piers. The figures given in the table enable us only to have a general idea of the performance of each device, but for any particular design the reduction in afflux will be a function of discharge and other factors and generalization is not possible. The same discharge can be passed with a lower upstream level by fitting any of the above devices. But when the upstream water level rises to the old value the discharge does not proportionately increase because with an increase in discharge on the downstream the tail water also rises and reduces the afflux obtaining at the structure. Thus the discharge improvement due to the adoption of any device could be computed only be a process of successive approximations.

Bearing all these limitations in mind it may be assumed that the reduction in afflux possible by the devices explained may be about 3 to 30 per cent. the actual value in each case being dependent on several factors influencing the same.

(3) SIPHONING OF THE CUMBUM TANK SLUICE⁽³⁾

INTRODUCTION

The Nakhalagundi Sluice of the Cumbum tank has a barrel 2.5 feet square and 120 feet long. It is necessary to increase the discharging capacity of the sluice under ordinary conditions of supply and to enable the sluice to draw supplies even when the water in the tank goes down below the sill by about four feet. Lowering the sill and increasing the vent way dimensions could not be approved as it was thought inadvisable to cut open the huge bund formed of earth and shale which had got consolidated in the course of centuries. Model experiments were taken up to examine the possibilities and benefits of converting the sluice into a kind of siphon. The rear channel being about eight feet below the sill the head available was considered sufficient.

⁽³⁾Irrigation Research Station, Poondi, Madras, Annual Report, 1946, pages 80 to 81.

THE MODEL

A model made of teak wood could not be made air-tight and was discarded. Another model was then made in masonry and concrete to a scale 1/10 geometrically similar. The barrel was three inches square and 12 feet long and took off from a tank 30 feet \times 10 feet \times 3 feet. After a few trial tests inlet and outlet limbs of the shape and dimensions shown in Figure 12.C.3 were added. At the outlet end a box was fitted up to ensure complete submersion of the outlet mouth when the model was to work as a siphon. Two brass cocks were fitted on top of the barrel as shown. Arrangements were also provided for measuring the water-supply let into the tank and the depth of water maintained in the tank.

PRIMING

Qualitative studies were first made for effecting the priming of the "siphon" under different conditions of water levels in the tank—

(a) With water level in the tank above the sluice barrel and with the tail limb submerged the sluice was made to run under pressure. All air was removed by utilizing the cocks. The water level in the tank was then gradually lowered below the sill of the sluice to see if the siphon continued to function. After a good deal of touching up of the model to eliminate possibilities of air entering the barrel the model got primed and continued to operate as a siphon.

(b) With water level in the tank initially at a level below sill priming was attempted to be effected by exhausting the air inside the barrel. The following device obtained after a series of trials was found effective in priming the siphon : An air tight chamber (*vide* Figure 12.C.3) 1.25 feet \times 1.25 feet \times 3.0 feet inside was constructed near the sluice head and was provided with arrangements for (1) filling it with water at top, (2) for connecting one of the nipples fitted on the sluice barrel with a similar nipple fitted on top of the box and (3) for draining away the water at the bottom by means of a pipe dipping into a pond. By connecting the barrel to the chamber when the inlet and outlet mouths were sealed by waters upstream and downstream respectively and draining away the water from the box a partial vacuum could be induced and air sucked out of the barrel gradually. This led to the priming of the sluice.

DISCUSSION BY THE RESEARCH COMMITTEE

Mr. KUTTIAMMU introducing item (1) said that the Tungabhadra Canal was proposed to be flumed where it passed through a cutting in soft rock. The hydraulic particulars for the flumed section, as obtained by analytical calculations, were checked by means of model experiments and the results on the model agreed with those obtained from theoretical considerations.

RAO BAHADUR JOGLEKAR referred to the considerable amount of work done at the Indian Waterways Experiment Station on fluming of aqueducts with the object of reducing head lost and the cost of construction, and specially invited reference to page 122 of the annual report (1946) of his station, to Bombay technical paper No. 31⁽⁴⁾ and to Bombay technical paper No. 29.

Introducing item (2), Mr. KUTTIAMMU remarked that the possibilities of reducing afflux caused at culverts, siphons, aqueducts, sluices *etc.* working under

(*) Reports of experiments carried out at the Karachi Model Testing Station on a model of flumed regulator by C.G. Hawes and A.S. Kahi with an introductory note by W.A. Evershed—(1929).

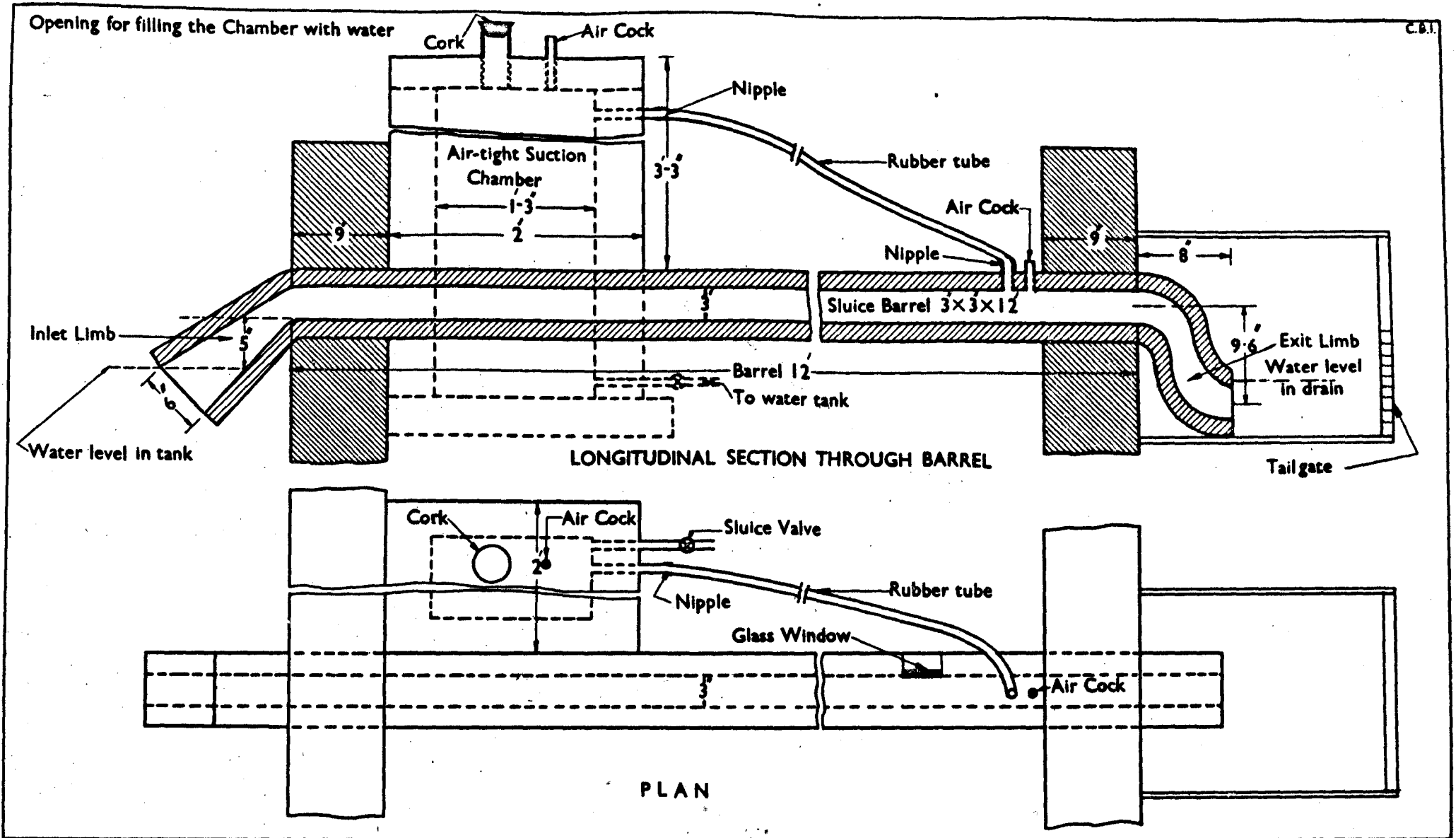


Figure 12C.3:- Showing Nakkalagundy Sluice of Cumbam Tank

pressure by simple devices such as streamlining the entry and exit, were investigated on a model. The reduction in afflux possible by such devices might be about three to 30 per cent., the actual value in each case depending on several factors.

Referring to Bombay technical paper No. 54, 1935 RAI BAHADUR JOGLEKAR stated that experiments of this type had already been carried out at the Indian Waterways Experiment Station, for increasing discharge of the head regulator of the Nira Left Bank Canal by providing a bellmouth upstream and expansion downstream. The head regulator consisted of nine spans, each span four feet wide with upstream gates four feet \times four feet. The sluices were nine feet long and had arched tops. Model experiments showed that the coefficient of the sluices could be increased from 0.83 to 1.93 by adding a bellmouth upstream and expanding the barrels downstream. These recommendations were carried out and 720 cusecs could now be passed with a loss of head of 0.17 foot as against the previous loss of head of 0.38 foot when only 428 cusecs were flowing, *i.e.* head required was reduced by 55 per cent. despite 50 per cent. increase in discharge. The cost of remodelling was only Rs. 5,800 as against Rs. 20,000 estimated for additional expansion had this been possible. The head regulators on the Godavari Right Bank and Left Bank Canals were similarly treated with satisfactory results ⁽⁵⁾.

MR. KUTTIAMMU introduced item (3) and said that the Nakhlagundi sluice of the Cumbum tank had a barrel two feet six inches square and 120 feet long. Model experiments were taken up to examine the possibilities and benefits of converting the sluice into a kind of siphon, in order to increase the discharging capacity of the sluice. He said that the model studies were being continued.

CONSIDERATION BY THE BOARD

THE SECRETARY described the work done at the Research Committee Meeting.

It was decided that the subject should continue on the agenda.

⁽⁵⁾ Central Irrigation and Hydrodynamic Research Station Poona, Annual Report 1937-38.

13C— Stauching of Canals and Embankments*

PRELIMINARY NOTE

In 1945 it was decided to issue a Board Publication on the subject and a request was made to various Research Organisations to review the work done to date at their institutions in their annual reports for the year 1945.

In response to this request the Irrigation Research Division Sind gave a summary of the work done by that Division on the subject. The Indian Water ways Experiment Station, Poona and the River Research Institute Bengal reported that they had nothing to contribute.

A request was again made to the rest of the Research Organisations to include all the data available in their Institutes in their reports for the year 1946.

At its 1946 meeting the Board accepted the recommendations of the Research Committee and changed the title of the subject from "Stauching of Canals" to "Seepage losses from canals and their prevention", with the following sub-heads.

- (i) Seepage losses from canals,
- (ii) Stauching of canals and,
- (iii) Economics of lining

The following items were discussed at the 1946 Research Committee Meeting under the sub-head "Design and Construction":—

- (1) Usar lining
- (2) Tests of lining materials
- (3) Tests on the joints of Thal type lining
- (4) Effect of compaction on absorption losses in water courses
- (5) Deterioration of compacted soils
- (6) Summary of work done in Sind
- (7) A note on Stauching of Canals by J. W. Russell Esqr., I.S.E.,
- (8) Losses from channels
- (9) Stauching of canals
- (10) Lime *Surkhi* Concrete Mortar as a stauching material

*The subject "Seepage losses from canals and their prevention" has been divided into "Stauching of canals and embankments with the two sub-heads (i) Materials and (ii) Economics and "Seepage losses which has been included under Section A-HYDROLOGY—item 5 Regeneration and losses".

Recent Literature.

- (1) Repair lining of Rora Irrigation Canal, Engineering News Record, July 11, 1946, page 76.
- (2) Byrne, W. S. Canal lined with stabilised earth, Indian Concrete Journal October 15th, 1946, page 245.
- (3) Tyldsley, Thomas B. Improved method of grouting leaks in foundation of dams, Water Works Engineering January 23, 1946,
- (4) Sanford, Hollis. Asphalt lining Oregon Canal, Western Construction News, April, 1945.
- (5) Asbeck, W.F. Baron Von. The use of bitumen in Hydraulic Works, Central Board of Irrigation Journal, April, 1945.
- (6) Tomlinson, George E. Leakage corrections at the Great Falls Reservoir, Engineering March 14, 1947, pages 181-183.
- (7) Tyler, C.I. Concrete lining on the Yakima Project, (on. and Const. Engineer Vol. XIX, No. 12, December, 1945).

The Year's Work

(i) MATERIALS

The following items were discussed at the 1947 Research Committee Meeting:—

- (1) Staunching of canals (the use of *usar* soil).
- (2) Experiments on staunching of canals with *surkhi* mortar.

(1) STAUNCHING OF CANALS—(THE USE OF USAR SOIL)⁽¹⁾

To study the use of *usar* soil as canal lining material to reduce losses, experiments have been in progress since March 1943. A six furlong reach of Bastauli minor in the Lucknow Division was divided into two portions, one of which was lined with 1½ inches thick layer of *usar* soil while the other was left un-lined to enable a comparison of the losses to be made. Both the portions were sub-divided into compartments and losses observed in each periodically. Figure 13C.1 shows the relation between 'reduction in seepage' and time.

⁽¹⁾ The United Provinces, Public Works Department, Irrigation Research Station, Annual Report on Research Progress, 1946, Technical Memorandum No. 17, page 2.

It will be observed :—

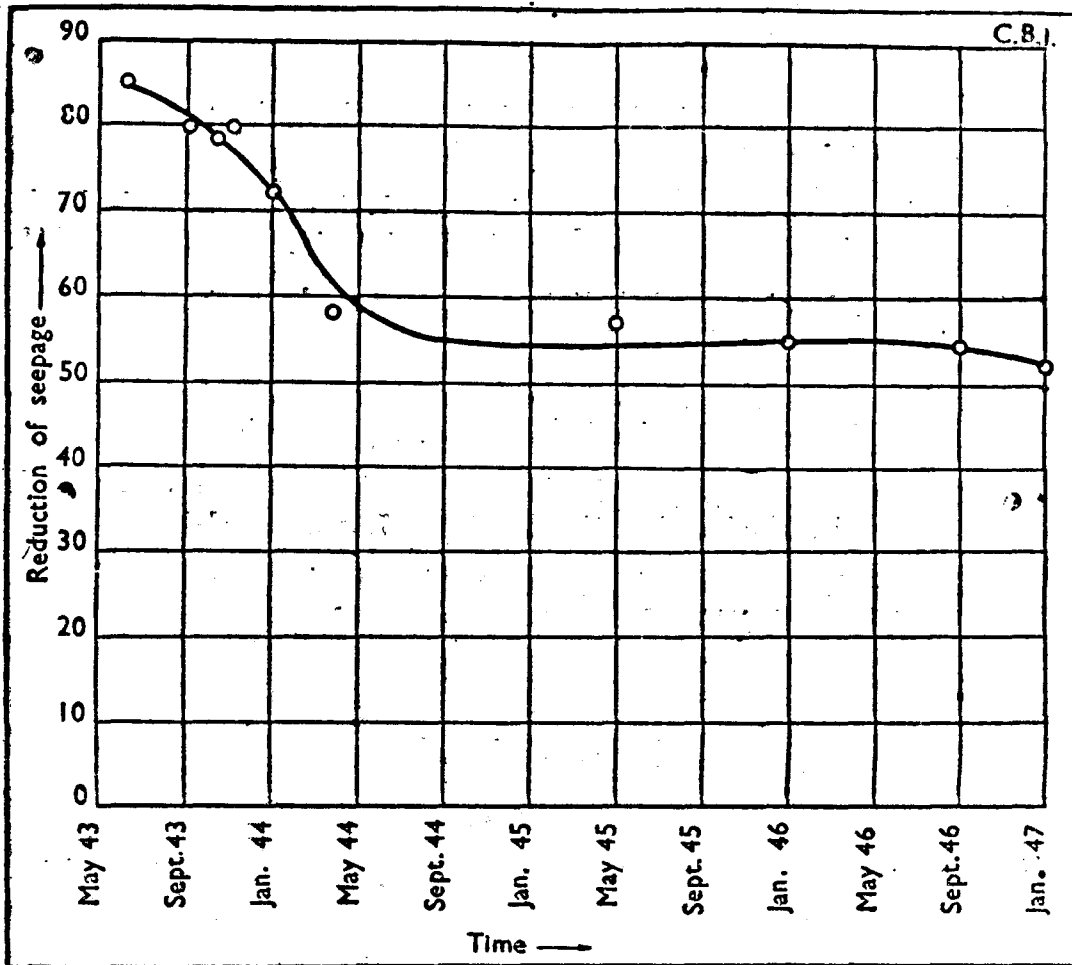


Figure 13 C. 1 : Showing relation between reduction in seepage and time.

(1) That initially seepage reduced by 85 per cent. due to *usar* soil lining. The lined portion becomes less permeable because sodium makes the soil particles swell.

(2) That there was a continuous decrease of seepage reductions up to the end of one year, when from 85 per cent. it came down to 55 per cent. The sodium soil due to continuous leaching action of water gets hydrolised with the result that hydrogen soil, which is more permeable than sodium soil, forms gradually.

(3) Up to January 1947 the reduction in seepage is above 50 per cent.

Observations of losses in the channel will be continued to ascertain its full life.

(2) EXPERIMENTS ON STAUNCHING OF CANALS WITH SURKHI MORTAR⁽²⁾

As a result of gaugings it was found that the transmission losses in the Irwin Canal worked out to ten cusecs per million square feet of wetted area. Various methods of preventing the losses were tried.

⁽²⁾ Hydraulic Research Station, Krishnarajasagar, Mysore, Annual Report, 1946, pages 20 and 21.

In the Maddur Branch of the canal, which is in cutting in gravelly soil, puddle lining six inches thick was done for a grade length. The puddle lining though it lessened the percolation at the start could not stand for more than two years.

In the first mile of the Irwin Canal, a short length of 270 feet of the canal in partial cutting was selected and its down stream slope and berm width was lined with cement concrete four inches thick. The proportion used for the concrete was one of cement two of sand and four of burnt stone jelly, $1\frac{1}{2}$ inches in size. The discharge in the canal is 2,200 cusecs and flows with a full supply depth of 12 feet. The sides of the canal are of fissured rock. The seepage has practically stopped in these two years.

The cost works out to Rs. 40 per 100 square feet. As this is prohibitively costly, experiments are being conducted at the Research Station with *surkhi* mortar concrete (two of jelly to one of *surkhi* mortar.) lining four inches thick subjected to a head of 16 feet clear water. The seepage losses through the concrete lining is decreasing year after year as per Table 13 C.1.

TABLE 13C.1

Date	Rate of seepage
October 1945 ..	1.03 cusecs per million square feet of wetted area
November 1945 ..	1.09 Do.
December 1945 ..	1.09 Do.
March 1946 ..	0.73 Do.
June 1946 ..	0.63 Do.
September 1946 ..	0.65 Do.
January 1947 ..	0.59 Do.

The cost works out to Rs. 15 per 100 square feet as per current rates.

DISCUSSION BY THE RESEARCH COMMITTEE

MR. CHATURVEDI introduced item (1) and said that a six furlong reach of Bastauli minor in the Lucknow division was divided into two portions, one of which was lined with $1\frac{1}{2}$ inch thick layer of *usar* soil while the other was left unlined to compare results with. He observed that the lining was successful to start with and the seepage was reduced by 85 per cent. but gradually this percentage fell to 50. He hoped to continue the experiments to ascertain the full life of *usar* lining.

MR. GOVINDA RAO introducing item (2), presented a resume of the work done in this connection at his research station. He said that in the Maddur branch of the Irwin Canal, a six inch thick puddle clay lining considerably reduced the percolation, but could not stand for more than two years. He observed further that a short length of 270 feet in the first mile of the Irwin Canal was lined with four inch thick cement concrete lining. Though the seepage in this reach had practically disappeared the cost of Rs. 40/- per 100 square feet proved prohibitive. Further experiments with *surkhi* mortar concrete (2 aggregate : 1 *surkhi* mortar) lining four inch thick subjected to a head of 16 feet clear water, reduced the seepage losses to 1.03 cusecs per million square feet of wetted surface. After a short period of 15 months, the seepage losses were reduced further to 0.59 cusec. The cost worked out to Rs. 15/- per 100 square feet.

MR. BLENCH enquired how the seepage losses were isolated and measured in such a short length as 270 feet.

MR. GOVINDA RAO replied that the advantage of a drain crossing the canal was availed of and the difference of discharge in the drain before and after the lining, immediately below the point of crossing, gave the seepage losses.

DR. S. P. RAJU stated the Nizamsagar Main Canal in Hyderabad was perennial excepting for about 20 days in the year and enquired if any information was available for staunching such canals without stopping flow.

MR. GUHA referred to the use of P.B.S. (Prebitumised sheets) in his province (Bihar).

KHAN BAHADUR KARIM said that such linings (with P.B.S.) had already been tried in Bihar, but were found useless in some places.

RAO BAHADUR BIJAWAT said that hessian cloth sheets impregnated with bitumen, were also tried in *guls** in the United Provinces a few years back but did not prove successful as the cattle used to cross over such *guls** and spoiled the lining. He said that theoretical considerations apart, practical difficulties (as suggested above) also made these linings useless.

MR. BLENCH said that bitumenised hessian (P.B.S.) used on air-fields and roads during the war was durable because traffic kept the bitumen 'ironed-out'. Pieces of P.B.S. left about his experiments station soon formed cracks and let the elements rot the cloth. For this reason PBS of present specification would hardly be suitable for canal lining; a further defect was that the bitumen flowed through the fabric under prolonged water-pressure.

(ii) ECONOMICS

There was no contribution under this head.

CONSIDERATION BY THE BOARD

(i) MATERIALS

The work done at the Research Committee Meeting was noted by the Board.

(ii) ECONOMICS

No discussion.

*Field Water Course

14C—Grouting

- (i) Materials
- (ii) Of foundation
- (iii) For staunching

PRELIMINARY NOTE

A new subject recommended by the Research Committee for introduction.

CONSIDERATION BY THE BOARD

THE SECRETARY said that this was a new subject head recommended by the Research Committee and that there was no contribution on the subject. It was decided that the subject should continue on the Agenda.

15C—Tubewells*

PRELIMINARY NOTE

At the 1945 Research Committee Meeting it was decided that Mr. Sanwal be requested to write up an appendix to his hand book on tubewells incorporating the experience gained by him in America. Mr. Sanwal has not been able to write up the appendix as yet as he intends to incorporate the results of experiments on rotary-rigs as well in the appendix. It will take some years for these experiments to be completed.

Recent Literature.

- (1) Carlstan, C.W. Note on the Early History of Water Well Drilling in the United States. Reprinted from Economic Geology, March-April 1943.
- (2) Sanwal, K.D. Handbook on tube wells, 1945. Printing and Stationery, U.P., Allahabad.
- (3) Mehta, B.C. Eccentricity in tube wells. Magazine of the Baroda Engineering Association Vol. V, No. 3, July 1945.
- (4) Machis, A. Experimental observations on grouting sands and gravels. Proceedings of the American Society of Civil Engineers, Vol. 72, No. 9, November 1945, page 1203.
- (5) Johnston C. N. Irrigation wells and well drilling methods in California. University of California, Circular 361, March 1945.
- (6) Well drilling (TM5—297) 1943. U.S. War Department.
- (7) Ground water supply for military operations (T.M.5—296) 1944 U.S. War Department.

(i) TYPES

The following items were discussed at the 1946 Research Committee Meeting.

- (1) Model experiments on radial, cavity and shrouded wells.
- (2) Electrical Analogy experiments to study the relative yields of a single well and of various systems of wells, with different strainer lengths, the source being a cylindrical boundary.
- (3) Electrical analogy experiments to study the relative yields of a group of wells placed parallel to a canal as a line source.

(ii) YIELD IN RELATION TO SUB-SOIL

(iii) DESIGN

(iv) QUALITY OF YIELD

(v) CONSTRUCTION

(vi) EXTRACTION OF TUBES

(vii) OPERATING MACHINERY AND THEIR TESTS

There were no contributions and no discussions on the above at the 1946 Research Committee Meeting.

(viii) EFFECT OF TUBEWELL PUMPING ON SUBSOIL WATER LEVEL

The following items were discussed at the 1946 Research Committee Meeting :—

*The subject of "Tubewells" has been divided into "Sub-terranean Waters" and "Tubewells". The sub-heads falling under the former have been taken under Section A—Hydrology—item 7.

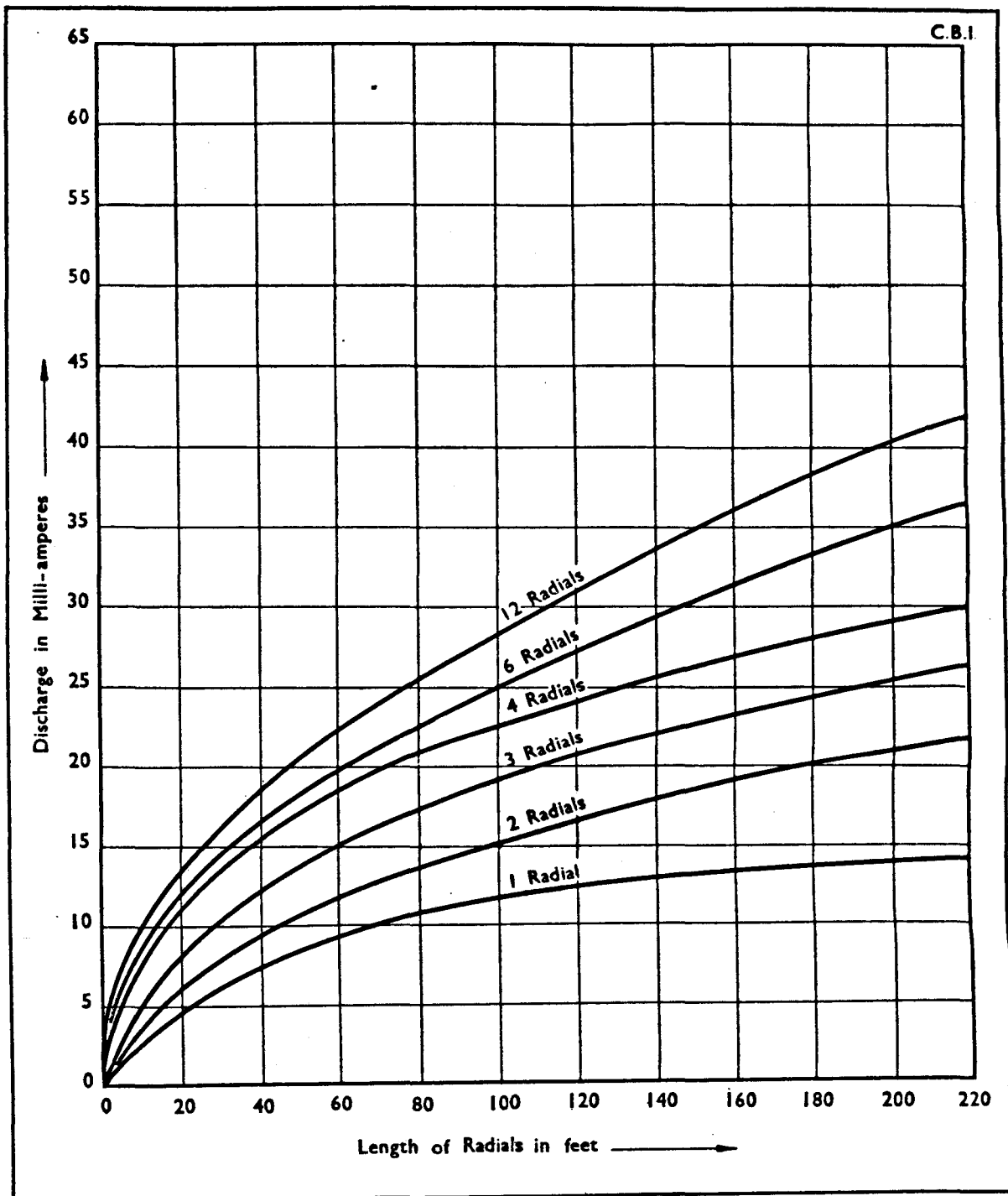


Figure 15 C.1--Showing Relation between Discharge
and Length of Radial [Radial Well
Experiment]

- (1) Field observations on the effect of tube-well pumping on the subsoil water table near the Upper Gugera Branch of the Lower Chenab Canal.
- (2) Electrical analogy experiments to study the suitability of the tube-well arrangements proposed for unwatering the Rasul Power Station Site to certain specific depths.
- (3) Well irrigation and its effect on the subsoil water level.

(ix) **RECHARGING THE UNDERGROUND BASIN FOR FUTURE PUMPING**

The following items were discussed at the 1946 Research Committee Meeting :—

- (1) Penetration of rain water to water table.
- (2) Investigation on possibilities of additional development in the Doab West of Ganges.

(x) **DISTRIBUTION OF SUPPLIES**

There were no contributions and no discussions on this subject at the 1946 Research Committee Meeting.

(xi) **METHOD OF ASSESSMENT**

The following items was discussed at the 1946 Research Committee Meeting :
Tubewell method of assessment in United Provinces by B. S. Mathur.

(xii) **ORGANISATION**

(xiii) **DEFINITION OF TERMS IN COMMON USE**

There were no contributions on the above at the 1946 Research Committee Meeting.

(xiv) **TUBEWELLS—THEIR CAUSES OF FAILURE AND REMEDIES**

This subhead is introduced for the first time.

The Year's Work

(i) TYPES

The following item was discussed at the 1947 Research Committee Meeting :— (1) Radial Tubewell

(1) **RADIAL TUBEWELL** ⁽¹⁾

Basic study of radial wells was taken up last year on a hydraulic model ⁽²⁾ to scale 1/12. This year an attempt has been made to study the radial tubewell by electrical analogy method. A model of the radial well to scale 1 foot = 0.5 mm. was constructed. The radials were 200 feet long and the central sump 16 feet in diameter.

The following problems have been tackled :—

1. The effect of length and number of radials on discharge.
2. The optimum position of radials with respect to strata.
1. *Effect of length and number of radials on discharge.*

Discharge for varying number and lengths of the radials were observed and plotted *vide* Figure 15C.1. It will be noticed that as the number of the radials increases the discharge does not proportionately increase. Similarly with increasing lengths of the radials the increase in discharge is not in proportion to the length of the radial.

CONCLUSION

(a) A combination of three radials seems most efficient and it is not worthwhile putting more than six radials.

⁽¹⁾ The United Provinces, Public Works Department, Irrigation Research Station, Annual Report on Research Progress, 1946, Technical Memorandum No. 17, pages 97 and 98.

⁽²⁾ The United Provinces, Public Works Department, Irrigation Research Station, Annual Report on Research Progress, 1945, Technical Memorandum No. 16, page 16.

(b) The rate of increase of discharge with length is pronounced up to a length of 80 feet. It is doubtful whether longer radials will be economical.

2. *The optimum position of radials with respect to strata.*

In order to find the place where the radials shall be located in a stratum, four different depths viz., 50 feet, 100 feet, 150 feet, and 200 feet of strata were used. The radials were kept at different places and the discharge observed and plotted as on Figures 15C.2 and 15C.3. Results are summarised below vide Table 15C.1.

TABLE 15C.1

No.	Strata depth	Position for Maximum discharge from Figure 15C.4	Percentage of the depth of radial to depth of strata (col. $\frac{3}{4} \times 100$)
1	2	3	4
1	50	27 feet. below the top	54
2	100	55 feet. " " "	55
3	150	90 feet. " " "	60
4	200	120 feet. " " "	60

CONCLUSIONS

The best place for the radials is at 60 per cent of the strata depth

DISCUSSION BY THE RESEARCH COMMITTEE

Introducing the subject MR. CHATURVEDI explained that in the case of a radial tube well the slotted tubes were placed horizontally instead of vertically and that discharge of the order of ten cusecs from one well could be expected. The experiments were conducted in order to find the length of the slotted tube to be placed, the number of radials to be put in and the position in the strata where the radials could be located. Proceeding he said, that electrical analogy experiments indicated that about 80 feet of radial length was the maximum that could be placed economically and the maximum number was four. As regards the place it has been observed that the maximum discharge would be obtained by placing the radials at a depth equal to 60 per cent. of the strata depth.

THE CHAIRMAN enquired how the radials were fixed and what type of strainers were used.

MR. CHATURVEDI replied that the radials were driven by means of jacks and that ordinary slotted pipes were used.

MR. BLENCH enquired the conditions under which the radial assemblage described became more effective than a plain masonry well subjected to the same pumping effort.

MR. CHATURVEDI said that the aim was to increase the infiltration into the well.

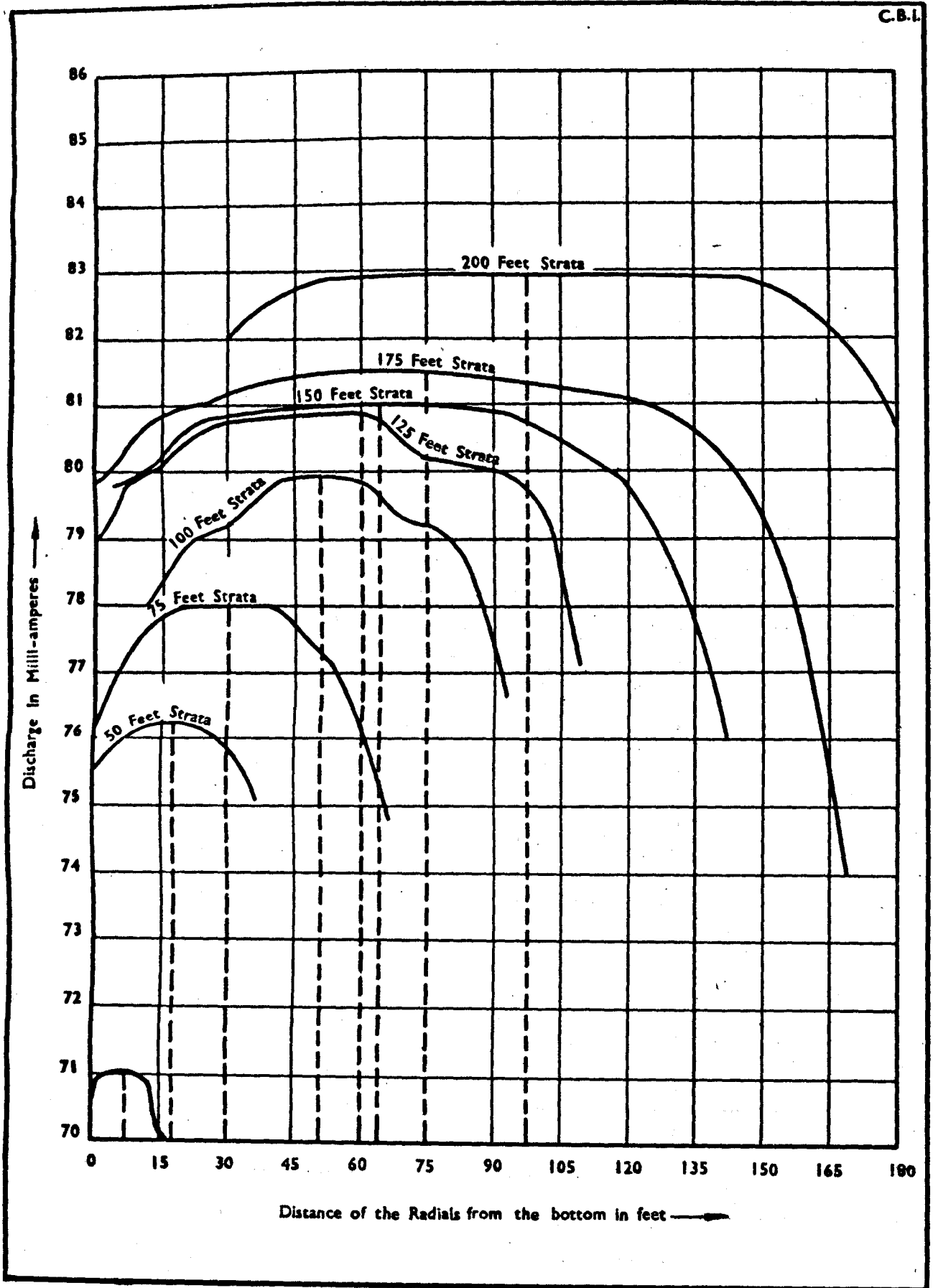


Figure 15C.2 :- Showing relation between Discharge & Position of Radials with respect to Strata Depth

[Radial Well Experiment]

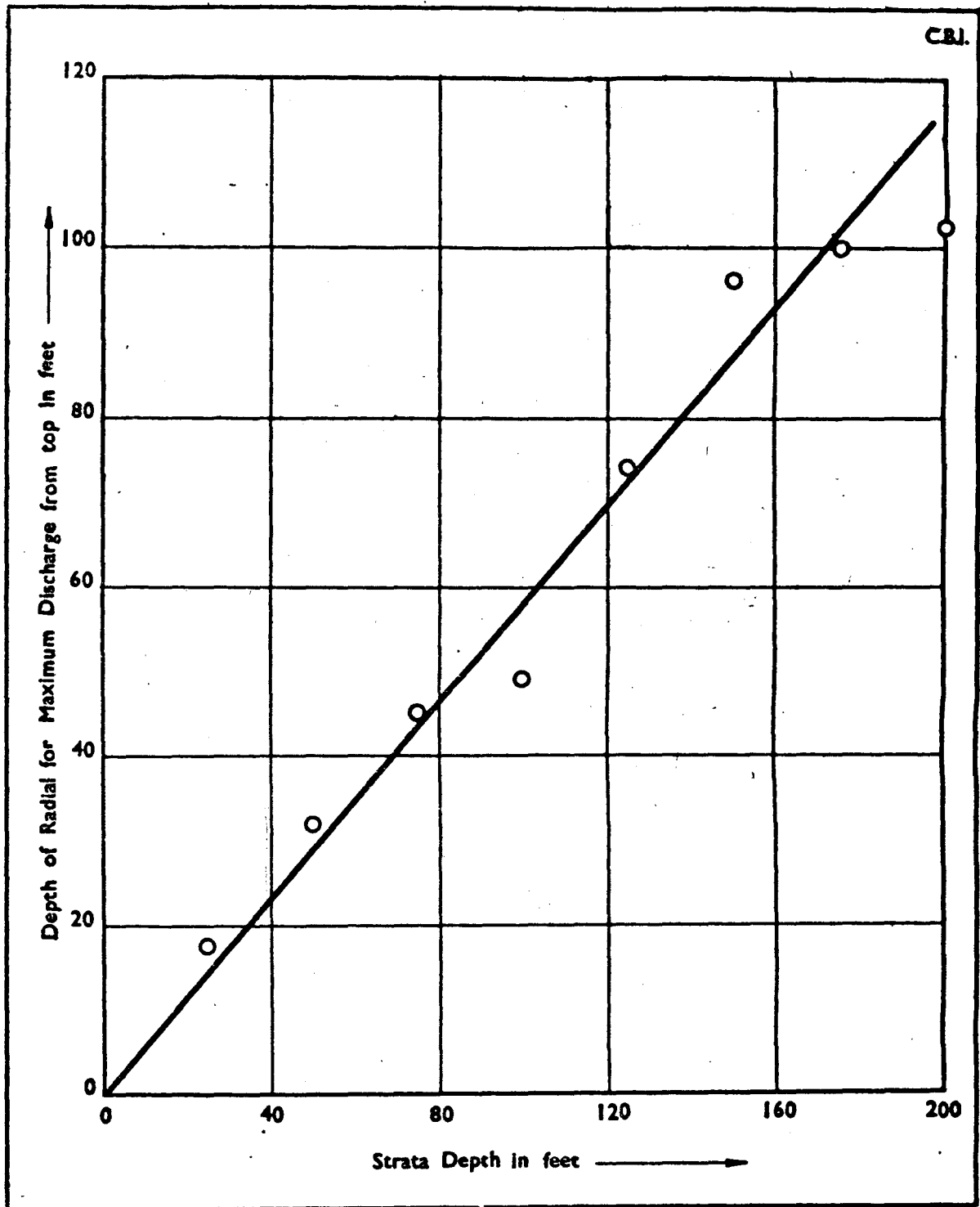


Figure 15C3:- Showing Relation between Depth of Radials for Maximum Discharge and the Strata Depth [Radial Well Experiment]

DR. VAIDHIANATHAN doubted if it was possible to get so much discharge from such a well. He also said that it would be very expensive and mechanically difficult to place the radials to a length of 80 feet. He suggested that it would be more economical to make a well with a big cavity or a well with a large shrouding.

MR. CHATURVEDI replied that it was not possible to make a cavity well at all places and that unless a clay layer was there on top to support, a cavity well could not be made. He also said that so far cavity wells made in the United Provinces had not yielded more than a cusec and a half at the most. So far as the shrouding was concerned, there was a limit to the size of shrouding. He also said that the strata in which such wells could succeed were of a very coarse sand capable of yielding large quantities of water. Only one such site had so far been found in the United Provinces after considerable prospecting.

THE SECRETARY reminded that the conditions in United Provinces and the Punjab (for tube-wells) were very different.

KHAN BAHADUR KARIM said that ordinarily it appeared that radial wells would give better results of yield.

RAI BAHADUR BIJAWAT replied that there were two or three such wells in Bihar, where the discharge of each well with four to five radials, was of the order of two cusecs.

KHAN BAHADUR KARIM replied that such wells (in Bihar) were referred to as 'infiltration galleries'.

In reply to Mr. Guha MR. SAXENA said that these radials were suited most for the particular strata of sand and shingle.

R. S. GUHA enquired about the diameter of strainers.

MR. CHATURVEDI replied that generally nine inches was the diameter.

In reply to Khan Bahadur Karim, MR. SAXENA said that such radial wells were under construction near Pillibhit in the United Provinces.

R. S. GUHA referring to the type of strainers (specially metallic) said that in Bihar during the last world war, the general scarcity of metallic strainers, necessitated the use of wooden strainers. Exhibiting a few wooden strainers in the meeting, he explained how these strainers had worked.

In reply to an enquiry from Mr. Framji, he said that they had not tried cement strainers in Bihar so far.

MR. COCHRANE asked about the type of pumps used and what was the depression head.

R. S. GUHA replied that electrically driven pumps were used and that the depression head was about 14 feet.

RAI BAHADUR BIJAWAT stated that radial tubewells were yet in an experimental stage and their future could not be predicted with certainty. These wells are not suitable in all situations and under all circumstances. The success of radial wells depends on the following two conditions :—

- (1) sufficiently thick permeable water bearing strata of coarse sand;
and

- (2) the strata should not be very deep down below the ground surface otherwise the sump becomes a difficult and costly affair.

As it is, the construction costs of these wells are heavy and mechanical difficulties greater as compared with vertical tubewells. The scope for such wells will therefore always be limited. In the United Provinces two experimental radial wells have been sanctioned recently at a cost of about Rs. 1½ lacs each. The cost of water pumped up from ordinary tubewells amounts roughly to about Rs. 10,000 per cusec. At this rate the radial wells should give a discharge of about 15 cusecs. It is doubtful whether such a big discharge will be available from them.

Continuing, he said, that during the war period wooden strainers were used in Bihar because it was difficult to procure metal strainers at reasonable cost. This was alright as a temporary measure but the wooden strainers had been tried in the Punjab and the United Provinces and had to be discarded in favour of metal strainers. In the United Provinces after trying various types of strainers the agricultural type was standardized. Experience in America, however, shows that gravel packed slotted tube type strainer such as recommended by Mr. Roscoe Moss an American Expert in tubewell drilling gave better results as regards yield of water. They were also cheaper in first cost and required less depression head thus reducing pumping costs and maintenance. Such wells have been recently constructed in Bihar with the help of mechanical percussion rigs. They have yielded a discharge of about 40 to 47 thousand gallons per hour at a depression head of four to six feet. Even in cheapness and life wooden strainers will have no advantage over such gravel packed wells. He quoted figures of cost to prove that the gravel packed type of strainer will be cheaper in initial cost, will yield greater supplies with lesser depression heads and will also have longer life than that of the ordinary types of strainers.

RAI BAHADUR BIJAWAT said that about 150 feet of slotted tubes were used.

THE CHAIRMAN (RAO BAHADUR VENKATACHARYA) enquired if wooden strainers had the advantage over metal strainers when the underground water contained salts.

RAI BAHADUR BIJAWAT replied that the over-all advantages of metal strainers were very much greater as compared with the wooden strainer which could not be recommended as a general rule.

KHAN BAHADUR KARIM remarked that (in any case) the life of metal strainers was rather limited.

RAI BAHADUR BIJAWAT replied that the advantages were greater in the case of metal strainers and that wooden strainers could not be recommended as a general rule.

DR. VAIDHINATHAN said that wooden strainers after being tried for three to four years in the Punjab had been finally discarded. He believed that acidity had more pronounced effects on wooden strainers than on metal strainers. He suggested the use of brass strainers.

RAI SAHIB GUHA said that in Bihar, wooden strainers were yet in experimental stage and had to be used when no metal strainers were easily procurable (during last world war). He said that as regards the slotted pipes, these were being slotted instead of punching.

RAI BAHADUR BIJAWAT remarked that the cutting of slots *in situ* was a difficult and laborious process and therefore slots were punched in plates which were then rolled into pipes and soldered. Referring to the point of Dr. Vaidhiana- than that the slits should vary according to the size of sand strata in different layers, he stated that straining was done by the gravel shrouding which has proved satisfactory in America, and there is no reason why it should not prove successful in India also.

(ii) DESIGN

There was no contribution under this sub-head.

(iii) CONSTRUCTION

The following item was discussed at the 1947 Research Committee Meeting:—

- (1) Tube well borings with 18 inches un-reinforced cement concrete pipes by Rameshwar Saran.

(1) TUBEWELL BORINGS WITH 18 INCHES UN-REINFORCED CEMENT CONCRETE PIPES

by

Rameshwar Saran

Executive Engineer, Tubewell Division, Aligarh.

INTRODUCTION

The construction of bore-hole-wells raised an important problem due to want of some suitable design such that almost any size bore hole pump could be inserted without difficulty. Wrought iron pipes of large diameters, *e.g.*, 12 inches to 15 inches were either not available at all, or their prices were exorbitant. Efforts were made in the past to construct this part of the well of cement concrete pipes of 18 inches diameter. These pipes could not be sunk properly upto the required depth as the pipes became dog-legged, and would not go deeper.

A suitable design has now been prepared for 18 inches diameter unreinforced cement concrete pipes. These pipes are manufactured in four feet lengths. A trial boring was done with these pipes upto 80 feet below ground level (which is generally the depth required for inserting bore hole pumps for spring level upto 35 feet). The experiment proved quite successful, inspite of the fact that the strata encountered was exceptionally unfavourable, being hard sticky clay and *kankar*. The first attempt encouraged further tests and three more borings were tried in similar strata which were all successful. It has now been established that 18 inches diameter cement concrete pipes can be used with certainty for boring upto about 80 to 100 feet below ground level in all types of strata generally met with in the United Provinces.

DESCRIPTION OF THE PIPES

The pipes for this purpose should be strong enough to withstand rough handling during transit and boring. These are unreinforced cement concrete pipes 18 inches internal and 23 inches external diameter. One inch semi-circular tongue has been provided in the middle of the thickness at one end, and semi-circular groove at the other end. These pipes also have 1½ inches semi-circular vertical grooves along the whole length, at 120 degrees apart and 1½ inches

semicircular grooves $2\frac{1}{2}$ inches in length at both ends in between the above grooves, as shown in Figure 15C.4.

One pipe out of every three is slightly different from the others in that it has a notch $\frac{1}{2}$ inch deep and $\frac{3}{4}$ inch wide on the outer edge at one end, in addition to the groove, as shown in the Figure 15C.4. The other end of this pipe

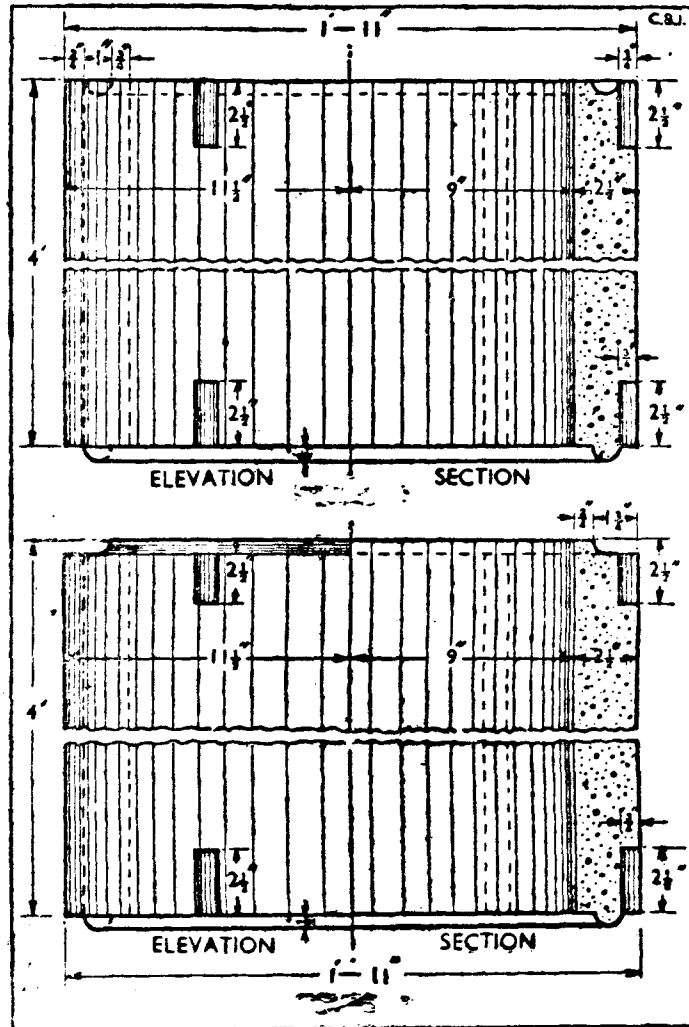


Figure 15C.4. : Details of pipe.

has a tongue similar to that in the other pipes. For boring upto about 80 feet fourteen pipes are required as shown in Figure 15C.4 top and 7 pipes as shown in Figure 15C.4 bottom. The cement concrete used in the manufacture of these pipes is of the proportion 1 : 2 : 2, i.e., one part of cement to two parts of sand (comprising $1\frac{1}{4}$ part of coarse Najibabad or Badarpur sand and $\frac{3}{4}$ part of clean local fine sand) and 2 parts peagravel of $\frac{1}{8}$ inch to $\frac{1}{2}$ inch size.

The pipes are manufactured with moulds (described in the next para.) working on a spinning machine shown in Figure 15C.5. The moulded pipes are cured under water for 15 days in order to allow them to attain full strength.

Details of cost of the pipes per foot are given in Table 15C.2 and 15C.3

TABLE 15C.2

Details of cost of 8 feet length of 18 inches unreinforced cement concrete pipes

Quantity	Labour	Rate	Per	Amount
1	2	3	4	5
4½ Nos.	Labour employed in moulding pipe (<i>vide</i> Table 2)	0 8 0	Each	5 1 0
Job	Watering for curing	L. S.	Job	0 4 0
Job	Screws and bolts'	L. S.	"	0 4 0
Job	Depreciation of machine and mould	L. S.	"	0 8 0
Job	Local sand	L. S.	"	0 2 0
Job	Digging of tank <i>etc.</i>	L. S.	"	0 6 0
Job	Miscellaneous expenses and breakage	L. S.	"	0 8 0
	Total			7 1 0
2½ Nos.	Cement bags	2 8 0	Each	5 10 0
5 cft.	Peagravel	80 0 0	%cft.	4 0 0
3 cft.	Najibabad sand	67 0 0	%cft.	2 0 0
Job	Crude oil	L. S.	Job	0 6 0
	Total			12 0 0
	Total			19 1 0

Say Rs. Ninteen only
 Cost of manufacturing 8 ft. of pipe=Rs. 19 Rate per foot=Rs. 2-6-0.

TABLE 15C.3

Details of labour for making pipes of four feet length

2 Beldars	Scratching mould for two hours	4 Beldars hours
1 Beldar	Cleaning the mould one hour	1 Beldar hour
1 Beldar	Oiling the mould one hour	1 Beldar hour
3 Beldars	Fitting the mould two hours	6 Beldars hours
7 "	Manufacturing the pipe one hour	7 " "
4 "	Opening and taking the pipe to the tank two hours	8 " "
7 "	Taking out the pipe from the tank one hour..	7 " "
	Total	34 Beldars for two pipes or 4½ beldars @ 7½ working hours per day.

DESCRIPTION OF MOULD

The design of a mould is shown in Figure 15C.5. A mould is made of ordinary $1/16$ inch thick iron sheet, 8 feet long, with cast iron flanges at both the ends, and one separator in the middle which enables the manufacture of two four feet long pipes with one mould. One inch semi-circular grooves are made in the flanges in order to mould the tongues at one end of the pipes. In order to manufacture pipe as shown in Figure 15C.4 top, one inch semi-circular tongues are provided on each face of the separator (Figure 15C.6), which mould the grooves at the other ends of the pipes. In order to mould the pipe as shown in Figure 15C.4 bottom, a different separator is used as shown in Figure 15C.7.

CUTTING SHOE

A cast iron cutting shoe (Figure 15C.8) is attached at the bottom of the first pipe (Figure 15C.4) top to help in boring. It is $3\frac{1}{2}$ inches to 4 inches long and has 18 inches internal and $24\frac{1}{2}$ inches external diameter. The cutting shoe provides a clearance of $\frac{3}{4}$ inch on each side, of the pipe. The cutting edge itself is about $3/8$ inch thick and is bevelled upwards at 45 degrees. On the top of the shoe a one-inch semi-circular groove is provided, so that the tongue of the first pipe may fit into this groove. Three $7/16$ inch holes are drilled at 120 degrees in the cutting shoe as shown in Figure 15C.8.

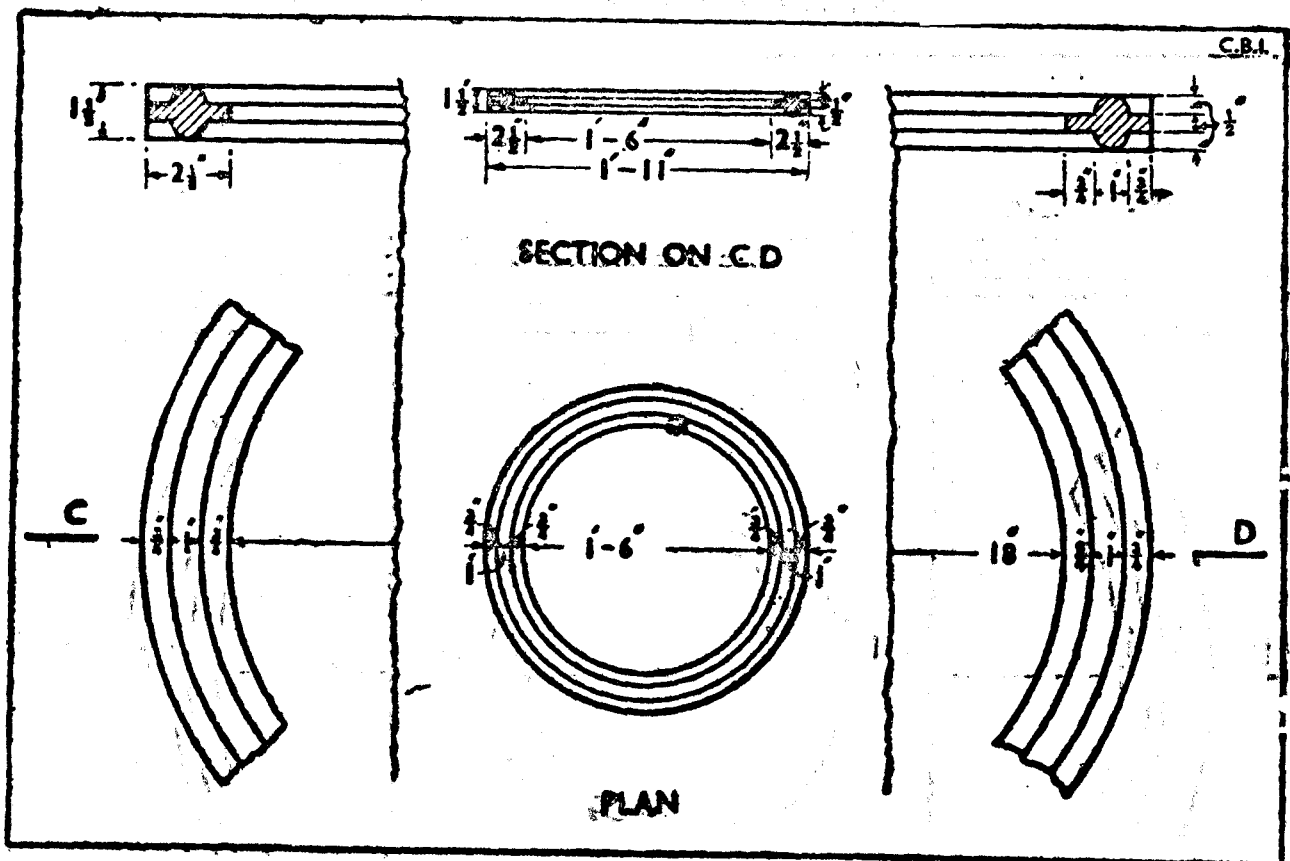


Figure 15C. 6 : Detail of cast iron separator

METHOD OF BORING

The boring is started from ground level. A pit 6 feet in diameter is dug to a depth of 8 feet from ground level. The cast iron cutting shoe is placed in the centre of the pit. Three $\frac{3}{8}$ inch diameter iron bars $12\frac{1}{4}$ feet in length, with bolt heads at one end and threads for 3 inches at the other end, are passed through the holes in the cutting shoe with their bolt heads inside the cutting shoe, and the threads towards the top. Both ends of a cement concrete pipe are cleaned thoroughly with a steel brush and given a coat of hot asphalt. A coat of hot asphalt is also given on the groove of the cast iron cutting shoe, and the pipe is then lowered into the pit with the tongue end to rest centrally on the cutting shoe. The three iron bars passing through the cutting shoe come into vertical grooves of the pipe. The ends of the second pipe are then treated similarly, and the pipe is then lowered with the tongue end resting on the groove end of the first pipe. The third pipe, as shown in Figure 15C.4 (bottom) is then lowered and rests on the second pipe. Care is taken that these three pipes are perfectly in plumb, and their long vertical grooves are all in one straight line, so that the iron bars running from the cast iron shoe just fit into them. At the top of the third pipe a mild steel ring (Figure 15C.9) $\frac{1}{2}$ inch thick and one inch wide, having six holes 60 degrees apart, is placed in the notch at the edge of the pipe so that the three bolts coming from below pass into the three holes of this ring. In this position the mild steel ring is projecting $\frac{1}{4}$ inch outside the pipe, but it does not cause any obstruction during the process of boring. The nuts of the bolts are then screwed tightly. This joins all the three pipes firmly and they work as one length. (Figure 15C.9).

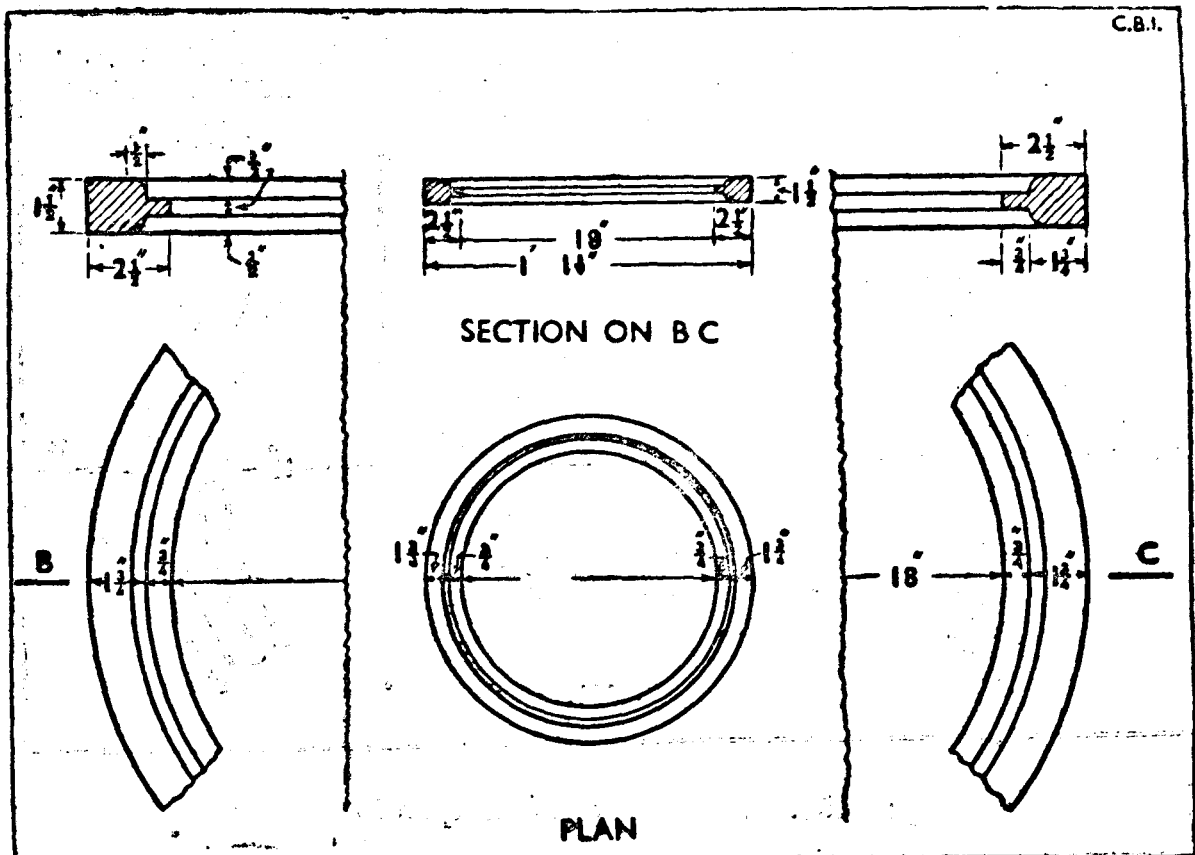


Figure 15C. 7 : Detail of cast iron separator.

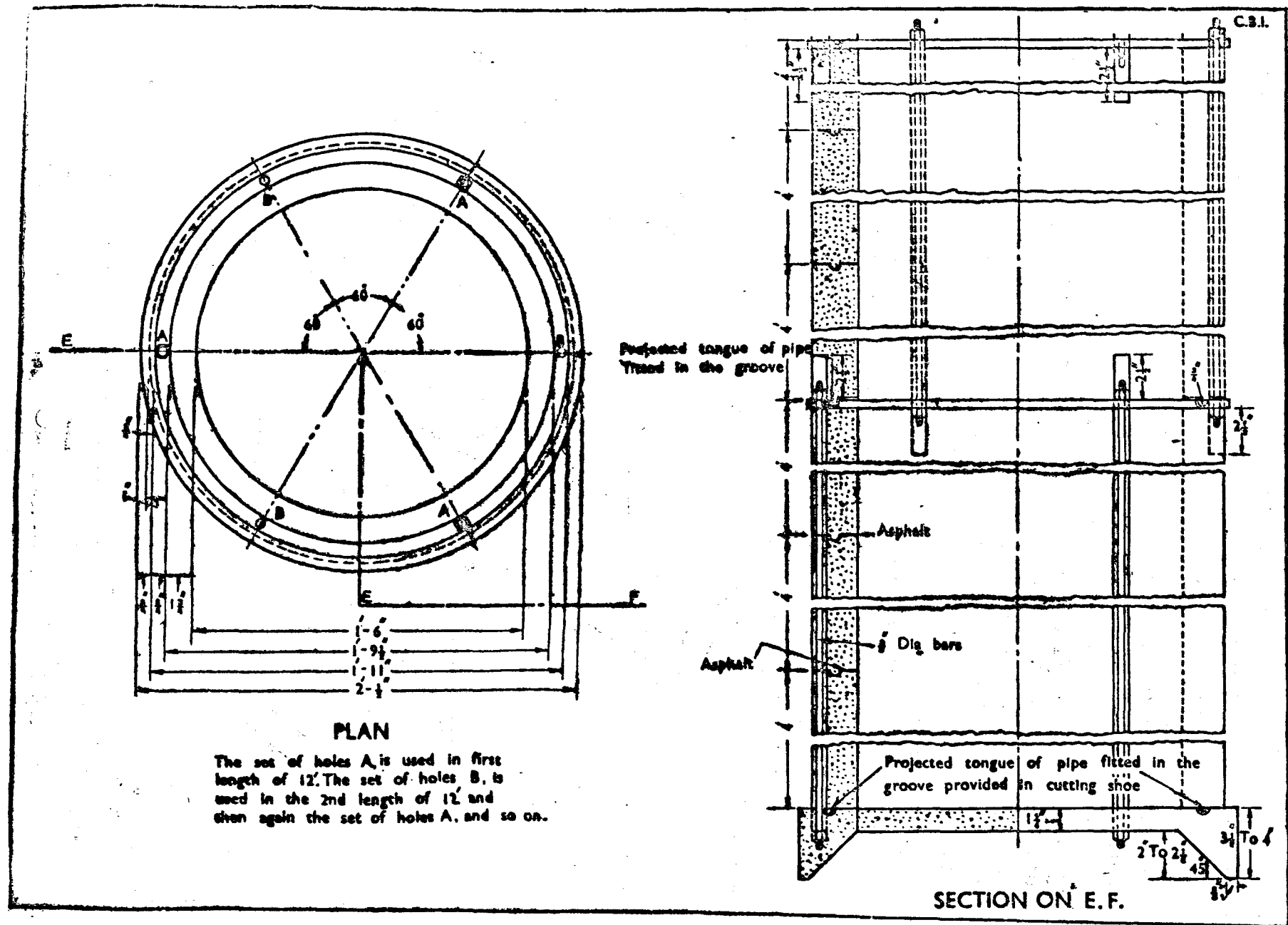


Figure 15C. 8 : Details of 18 inches diameter cement concrete pipe and cutting shoe for bore hole.

An iron clamp is then fitted on to the top pipe and the whole thing is loaded with sand bags. The boring is then started with an eight inches sand pump pouring sufficient water from outside into the pipe. In order to enable the sand pump to work, when the third pipe is sunk upto four feet from ground level the fourth pipe is lowered exactly in the same way as described above with the tongue end resting in the groove of the third pipe, care being taken that the small vertical grooves come just over the ends of the bars coming from below. Three $\frac{3}{8}$ inch diameter iron bars $12\frac{1}{4}$ feet long are then passed into the three remaining holes of the mild steel ring, and they pass through the three long vertical grooves of the fourth pipe, with their bolt heads resting inside the small grooves of the third pipe. The boring is then resumed and another pipe lowered as in the first case. Then the sixth pipe with the notch for mild steel ring is lowered and the process repeated as at the third pipe. By using one set of three holes in the mild steel ring to tie down one lot of three pipes together; and the other three holes, *i.e.*, the alternate holes, to tie the next three pipes; each set of three pipes is tied to the next, so that the whole series from top to bottom are tied together. During the process of boring the $12\frac{1}{4}$ feet iron bars are kept in position by means of U clamps until they are tightened to the mild steel flat ring at every third pipe. They fit in the grooves provided for the purpose in the pipes and check them from going doglegged because they work as one length.

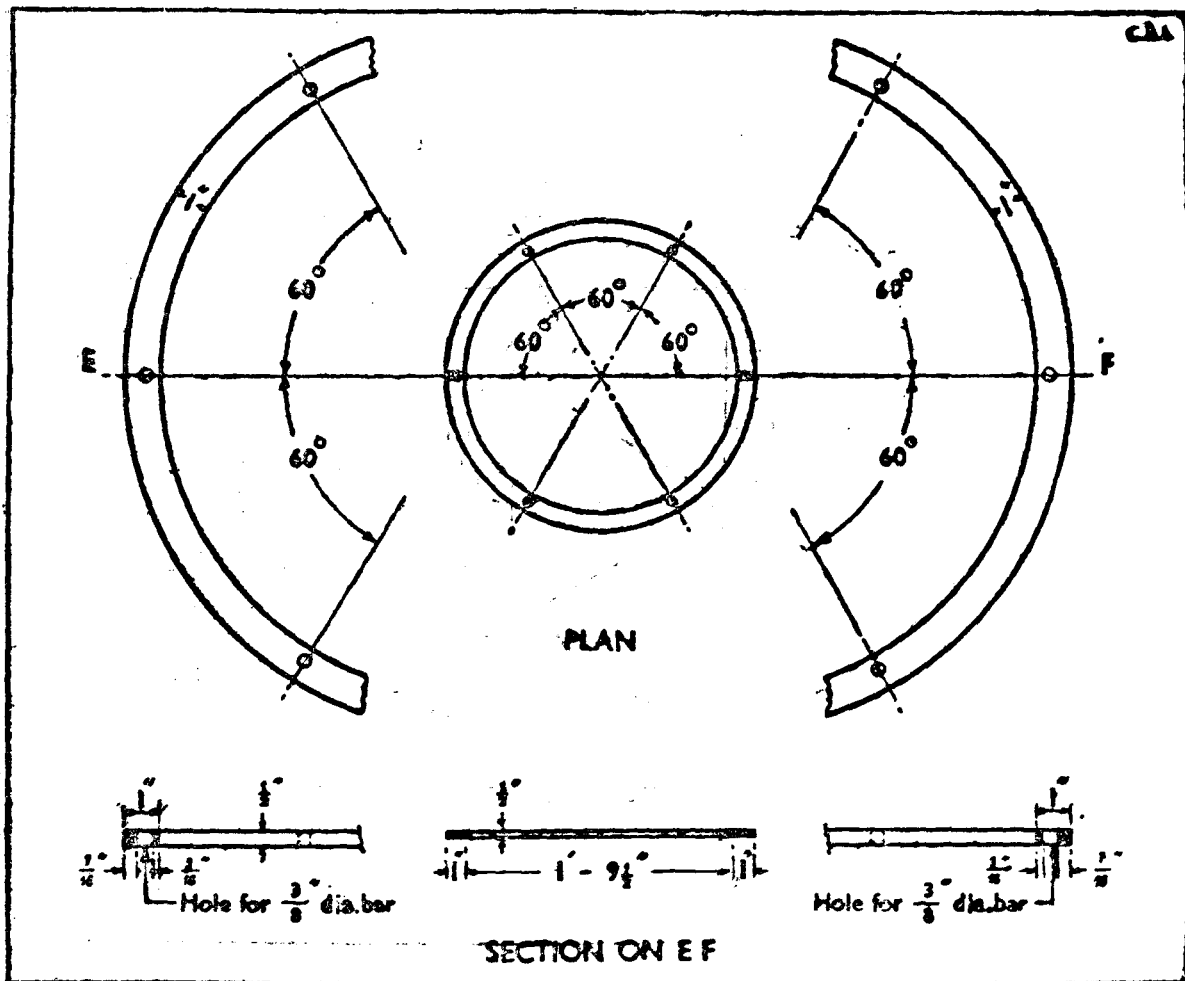


Figure 150. 9 : Details of mild steel ring.

Another method of loading these pipes for boring is to lower one length of 15 inches casing pipe into them with a wooden clamp tightened at the top which will rest on the top of cement concrete pipe. Sufficient weight can be loaded on this clamp. The boring is thus continued upto the desired depth, water being poured from outside until the spring level is reached, and then the boring proceeds on easily.

Details of cost of accessories required for boring with these pipes is given in Table 15C.4.

COST OF BORING

The three trial borings were done from the ground level and the strata encountered was unfavourable. The cost of boring per foot was slightly higher. It is expected that it will be cheaper in ordinary cases. It worked to Rs. 5/8/- per foot but it has been assumed to be Rs. 6/- per foot in working out the comparative cost of bore hole wells.

CONCLUSION

The comparative cost of constructing bore hole wells with cement concrete pipes as compared with 12 inches wrought iron pipes for inserting bore hole pumps has been worked out in Table 15C.5. It shows a saving of Rs. 1,318/- per well if cement concrete pipes are used.

There is no danger of pitting of the pipes due to corrosion within the depressed level. There is ample space for inserting big size bore hole pump if bigger discharges are required.

TABLE 15C.4

Details of cost of accessories required for boring with 18 inches cement concrete pipes for 80 feet

Item No.	Quantity	Description of item	Rate	Per	Cost
1	100 lb.	$\frac{3}{8}$ inch dia iron base 21 Nos. $\times 12\frac{1}{2} \times$	17 0 0	Maund.	21 0 0
2	6 Nos.	.368 lb M. S. Flat Rings complete	7 8 0	Each	45 0 0
3	21 Nos.	with holes Labour charges for making bolts (item 1) including nuts, threading etc.	1 8 0	"	31 8 0
4	1 No.	Cutting shoe	85 0 0	"	85 0 0
5	Job	Cost of asphalt steel brush etc. ..	L. S.		6 0 0
Total ..					188 0 0

Cost of accessories for boring with 18 inches cement concrete pipes for 80 feet—Rs. 188
 \therefore cost per foot—say Rs. 2/6.

TABLE 15C.5

Showing comparative cost of bore hole wells with 12 inches wrought iron pipes and 18 inches diameter unreinforced cement concrete pipes

Serial No.	Particulars	Quantity	Unit	Boring with 18 inches dia. cement concrete pipes		Boring with 12 W. I. wrought iron pipes		Remarks.
				Rate	Amount	Rate	Amount	
1	Digging pit 6 feet dia about 8 feet deep	Job	..	L. S.	20 0 0	L. S.	20 0 0	*
2	Carriage of boring set and pipe set etc.	Job	..	L. S.	125 0 0	L. S.	125 0 0	
3	Cost of pipe (See Table 1)	80 feet.	Rft.	2 5 0	190 0 0	18 0 0	14,400 0 0	Stock rate of 12" dia inches W.I. Pipe Rs. 18 per foot
4	Iron rings, cutting shoes etc. (See Table 3)	80 feet.	..	2 6 0	190 0 0	L. S.	60 0 0	
5	Misc. expenses e.g. bags, sulls asphalt oil, etc.	Job	L. S.	..	50 0 0	L. S.	40 0 0	
6	Boring charges ..	80 feet.	Rft.	6 0 0	680 0 0	5 0 0	400 0 0	
7	Expenditure in cutting and re-threading four lengths of pipe for making them 10 feet to 12 feet long for use in boring	8 Nos.	Each	11 0 0	88 0 0	
8	Providing extra sockets ..	4 Nos.	Nos.	50 0 0	200 0 0	
					1,055		2,373	

Net difference—Rs. 1,318 per well

DISCUSSION BY THE RESEARCH COMMITTEE

MR. CHATURVEDI introduced item (1) and said that the technique of boring with 18 inches un-reinforced concrete pipes was actually developed by Mr. Hutchinson (Chief Engineer, United Provinces Irrigation Department). This note had already appeared in the July 1947 issue of the Central Board of Irrigation Journal.

RAI BAHADUR BIJAWAT pointed out that a saving of Rs. 1,318/- per well has been worked out in Table 15C.5 by using 18 inches diameter unreinforced concrete pipes as compared with 12 inches diameter wrought iron pipes for housing the deepwell borehole pump. He stated that the rate for wrought iron pipes quoted in Table 15C.5 was probably correct during the war period but the rates have gone down considerably since the close of the war. He was getting 12 inches pipes at the rate of Rs. 7/8/- per foot length. The reduction of cost on this item alone would wipe out the saving worked out in the comparative statement. In this case also unreinforced concrete pipes were

*In case of cement concrete pipes the carriage of extra weight will be covered by the extra distance involved in carriage of 12 inches wrought iron pipes as the former would be available at each pipe centre and the latter at Central Stores.

used because of the difficulty and cost of obtaining wrought iron pipes. With the reduction in rate of wrought iron pipes there is no longer any advantage in using concrete pipes over wrought iron pipes.

The other advantage claimed for the concrete pipes was the danger of pitting due to corrosion within the depression zone in the case of iron pipes. This advantage was very illusory as no such pitting is likely to take place during the life time of a tubewell which is estimated to be only about 15 years.

MR. CHATURVEDI associated himself with Rai Bahadur Bijawat's remarks and said that conditions had now changed (as regards costs) and that with bigger bore hole pumps there might be lesser trouble.

MR. H. P. MATHRANI enquired what sort of couplings were used for these pipes.

RAI BAHADUR BIJAWAT replied that end of one pipe was inserted into the other end then it was soldered at the site.

(iv) **EXTRACTION OF TUBES**

(v) **OPERATING MACHINERY AND THEIR TESTS**

(vi) **ORGANISATION**

There was no contribution under the above sub-heads.

DISCUSSION BY THE BOARD

(i) TYPES

THE SECRETARY reported that only one item was discussed by the Research Committee.

(ii) CONSTRUCTION

THE SECRETARY said that only one item was discussed under this head.

There was no contribution under other sub-heads.

MR. NARASIMHAIYA wanted to know if there was any organisation to send round equipments to bore tubewells.

THE PRESIDENT said that there was the Central Ground Water Section but unfortunately their representative was absent that day.

MR. A. R. KHANNA wanted Mr. Chaturvedi to tell the house about the radial tube-wells.

MR. CHATURVEDI said that the construction of tube-wells in the fields had been taken up only recently and not much of a headway had been made in this connection.

MR. BALBIR SINGH said that the work had just started. There was not much of progress made. Three tube-wells of this type, minimum diameter 16 feet, were being bored.

(i)

INDEX

	Pages		Pages
A			
Abreo, Mr. F. D.	17, 191, 714, 751, 752, 753	Asghar, Dr. A. G.	17, 303, 304, 305, 310, 332, 337, 338, 339
Advani, Shri P. B.	13	Assessment of tubewell supplies	825
Afforestation	144, 148	Atmospheric pressures	312
Afforestation of waste lands	206, 208	Attracting impermeable groyne	451
Agarwal, Shri P. C.	(i), 23	Attracting island	782
Airfield runways	306	Attrition: Rate of Sand—in channels	233, 240, 241, 251
Akhihebbal	159	Automatic recorders	150
Alkaline soils: Reclamation of—	334	Aye Mounq, Shri U.	(i)
Alluvium:		Ayer	146
Design of channels in—	234	Ayyanger, Shri N. Govindaraja	(i), 15, 151, 412, 416, 419, 433, 454, 457, 536, 621, 622, 625, 717
Flow in channels in coherent and incoherent—	231, 233	B	
Ambedkar, Hon'ble Dr. B. R.	24	Bably Compensators	610
A method of determining the shrinkage factors of soils	289	Baffle	518, 530, 621, 644, 646, 694, 706, 712, 716, 734, 735, 752, 767
Amount of water utilized per acre of sugarcane by sugar factories in the Bombay Deccan Canal areas	348, 352	Bahl, Shri M. L.	(i)
Analysis of observations on silt exclusion at the headworks of Damodar Canal at Rhondia	754, 760	Bahri, Shri J. C.	600
An analysis of hydraulic data from selected discharge sites along Barrage canals during 1946	233, 244	Baker, Mr. L. Y.	76, 123, 127, 128, 129
Anand, Shri D. B.	395	Balbir Singh, Kanwar	(i), 23, 371, 839
Ananda Rao, Shri U.	17, 429	Baldev Singh, Hon'ble Sardar	24
Anderson, Col. W. G. Lang	(i)	Banerji, Dr. S. K.	23, 75, 130, 131, 132, 133, 134, 137, 140, 204, 213, 440, 441, 442, 443
Anderson Earth Dam	308	Barakar River	132, 136, 142, 146, 148, 172, 291
Anderson Weir at Rhondia	757, 760	Barakar at Asansol Railway Bridge Site	154, 156, 172
Angle of off-take: Effect on silt distribution of—	718, 719, 722, 723, 751	Barakar at Barhi Road Bridge Site	154, 156
Annual Water Statements	131	Barakar River	132, 136, 142, 146, 148, 172, 291
A note on surkhi morta—by M. C. Sampathiengar	610, 611	Bara Tal Reservoir	592
Anti-dune	395, 397	Barhi	154, 156, 172
Anti-malarial aspects of Water Utilisation Projects	380	Bariarpur Reservoir	592
Discussion by the Board on—	383	Basic experiments for determining co-efficients for radial gates	718, 723, 751
Discussion by the Research Committee on—	383	Basic experiments to investigate the effect on silt distribution of (a) angle of off-take, (b) varying discharges	718, 719
Preliminary Note on—	380	Battery of siphons	671
The Year's Work on—	382	Baxter, Mr. D. C.	16, 429
An unprecedented flood in Malwa and Pravara Valleys and estimated loss of flood damage	462, 481	Bazaz, Shri S. L.	16, 23
Any other business	436	Bearing capacity	289, 294, 296
Discussion by the Board on—	436	Bearing power	308
Discussion by the Research Committee on—	436	Beas River: Railway Bridge at Dhillwan	490, 506, 516
Preliminary Note on—	436	Bed-load	396, 397, 589, 590
A. P. M. Flumes	801	Bed trench method of obtaining—	
Appendices	437	Samples	185
Appendix I	439	Experiments on the measurement of—	177, 185
Appendix II	444	Ratio of—to suspended load	188
Apportionment of costs	374, 375, 376	Sampling of—	176
Aquifer	211	Bed load characteristics of Brahmaputra	177
Artesian formations	212	Bed material	97
Artificial rainfall	132, 133		
Experiments abroad on—	131, 133		
Experiments in India on—	133		
Asansol Bridge	136, 154, 156		

	Pages		Pages
Bed material load	396, 397	Canal Falls	709
Bolj Ram, Shri	17	Consideration by the Board of—	717
Bermo	117	Discussion by the Research Committee on —	714
Bhakra Dam	132, 595, 621	Preliminary Note on —	709
Bhakra Dam Spillways	537, 682, 696	The Year's Work on —	709
Bhakra Project	372, 373, 374	Canalisation	207, 208
Bhandari, Dr. N. N.	17, 685, 687, 688, 689, 698	Capacity surveys of reservoirs	588, 597, 598
Bhatia, Shri K. M.	153, 176, 192, 310, 395	Capillarity of soil	333
Bhawar Reservoir	596	Capillary	306
Bhonsle, Major N. K.	(i) 13	Capillary action	332
Bidyadhari River	563, 565, 566	Capillary fringe	333, 339
Bijawat, Shri M. C.	16, 23, 151, 204, 219, 333, 354, 355, 356, 385, 595, 822, 827, 828, 829, 838, 839	Capillary rise	216, 217, 304, 313, 316, 322, 323, 333
Birnasi	462, 463, 464, 465, 466, 470, 472, 488	Capillary zone	213, 338
Birupa	462, 465	Capital accounts for generation and trans- mission	282
Blench, Lt. Col. T.	(ii), 15, 18, 21, 69, 152, 159, 175, 196, 208, 219, 250, 251, 333, 339, 340, 362, 433, 434, 514, 534, 595, 596, 597, 608, 609, 623, 689, 707, 715, 717, 752, 801, 822, 826.	Catchment :	
Bokhoro	196	Drained — s	198, 199, 200, 201, 202, 203
Bore-hole-wells	829, 837, 838	Experimental — s	199
Bose, Dr. N. K.	(ii), 15, 21, 23, 69, 70, 127, 128, 129, 132, 133, 134, 140, 144, 149, 150, 151, 152, 174, 175, 191, 192, 203, 204, 211, 213, 233, 249, 250, 251, 274, 303, 304, 305, 310, 332, 333, 339, 395, 397, 429, 433, 439, 440, 441, 442, 443, 453, 455, 456, 457, 509, 510, 511, 512, 514, 515, 530, 531, 534, 535, 544, 545, 583, 564, 577, 581, 582, 690, 691, 692, 700, 706, 708, 793, 794, 800.	Undrained — s	198, 199, 200, 201, 202, 203
Bottle sampler	177, 178	Water shed characteristics of—	589
Boulder Dam	595, 621	Cauvery	66, 143, 145, 150, 603, 620
Brahmani: Unit Hydrograph of the — above Nalhati	50	Cauvery at Chunchinkatte	161, 174
Brahmaputra	145, 149, 177, 238, 249	Cavitation	634, 639, 650, 657, 631, 631, 692
Distribution curve of — bed silt at Jogigopa	182	Cavity well	827
Gauge-discharge curves of the river — at Jogigopa	140, 141	Cellular cofferdam at Ramapadasagar	328, 329
Relation between different discharge elements for the river — at Jogigopa	233, 236	Cement :	
Brahmaputra near Goalpara	236	Effect of mixing—with soil	292
Brahmaputra Sand	241, 251	—soil blocks	303, 306
Bridges: Scour downstream of—	518, 519	Central Board of Irrigation :	
Brij Narayan, Shri	16, 151	Annual Dinner of the—	24
Brown, Mr. K. I.	23, 140, 273, 279, 280, 284	Annual Meeting of the—	22
Buctons' machine	292	—Guests	24
Buff meter	165, 166, 167, 168, 169, 170, 171, 172, 174	List of members of—as on December 1, 1947	(i)
Bukkar Gorge	531, 764	Presidential Address	4
Bukkar Island	764	Central Soil Board	207
Burna, Dr. R. V.	(ii)	Chambal	145
Burt, Mr. T.	(i)	Chandana River	536, 537, 538, 539, 540, 541, 544, 545
	C	Chandra, Shri R.	(i)
Caissons	400	Channels :	
Canal drops with counter acting jets for the Nervasa Tenpathi Channel	709, 712	Design of—in alluvium	234
		Design of—in Mor Project	233, 234
		Design of irrigation	254
		Flow in—in coherent and incoherent alluvium	231, 233
		Flow in—with rigid and quasi-rigid boundaries	231
		Rate of sand attrition in—	233, 240
		Theory of flow and design of—	229
		Chas Road Bridge Site	136, 249
		Formula for velocity at—on the Damodar	233, 235
		Unit hydrograph of the Damodar Catch- ment above—	65

	Pages		Page
Chaturvedi, Shri R. S. (ii), 15, 21, 23, 123, 129, 195, 203, 204, 212, 213, 251, 273, 274, 303, 304, 305, 306, 307, 353, 354, 357, 417, 433, 455, 456, 489, 490, 531, 535, 594, 596, 682, 689, 690, 691, 693, 695, 697, 698, 699, 715, 717, 752, 797, 821, 826, 827, 838, 839		Correlation between regeneration and flood discharge	197
Chenab River	490, 507,	Correlation between regeneration and local rainfall	197
Chunehinkatte	159, 174	Correlation between the gauge heights at the Samthia Railway Bridge Site and the Tilpara Barrage Site on the River Mor	140
Church, Dr.	130, 131	Correlation of surface velocity observations with current meter observations at 0.6 depth	153, 159
Clay content	289, 296, 303	Costs of generation of hydel power	282
Climatology	211	Critical velocity: Kennedy's—theory	234
Cloud control	133	Critical velocity to start abrasion	241
Cochrane, Mr. G. A. D. 16, 69, 128, 152, 195, 306, 596, 623, 625, 717, 827		Critical velocity to start attrition	241
Co-efficient of discharge 601, 603, 604, 605, 606, 634, 635, 636, 640, 642, 643, 657, 662, 663, 664, 667, 668, 669, 670, 671, 672, 673, 674, 675, 676, 677, 678, 681, 684, 685, 688, 689, 690, 692, 694, 695, 696, 697, 698, 718, 726, 728, 729, 730, 732, 733, 751, 753,		Crop: Maximum yield of—	348, 349
Co-efficient of discharge for the Ramapadasagar Spillway Dam	636, 642	Crops: Quality of waters in relation to—	214
Co-efficients of evaporation	213	Consideration by the Board of	228
Co-efficient: Transmission	210	Discussion by the Research Committee on—	218
Cofferdam 328, 329, 330, 331, 400, 521, 523, 524, 525, 526, 532, 533, 535, 536		Crops: Rotation of—	332, 338, 356
Coffor Dams for the Ramapadasagar Project	518, 521	Water requirements of	347, 349
Compaction	308, 595	Cross regulator	719, 723
Compaction method	303	Cross Regulator at Rustom	718, 733, 734, 752
Compaction test	292, 302	Current Meter:	
Comparative statment showing the discharges obtained at the turbulent sites with various meters as directly gauged and as deduced from gaugings done at the regular site.	170	Comparison of velocity observations by the float method, and the — method..	153
Comparison of different formulæ for Evaporation	318	Correlation of surface velocity observations with — observations at 0.6 depth	153
Comparison of velocity observations by the float method and the current meter method	153, 154, 155, 156, 157	Experiments conducted to find the effect of temperature on the results of ratings of— by K. K. Mahadevan	154
Compressive strength	292	Velocity by —	157
Conductivity	216, 217	D	
Conduits and penstocks	626, 633	Dam Appurtenances:—	599, 625, 626
Construction of dams	699, 600, 625	Consideration by the Board of—	633
Construction of tubewells	824, 829, 839	Discussion by the Research Committee on—	632
Contact load	395, 397	Preliminary note on—	626
Contour ploughing of agricultural land	148	The Year's work on—	626, 628
Co-ordination of research work for the ensuing year	433	Dam models	297, 301
Discussion by the Board on—	434	Damodar 136, 145, 146, 148, 172, 439, 440, 441, 793.	
Discussion by the Research Committee on—	433	Correlation between gauge levels of the river—at Raneegunge and Rhondia and the gauge discharge curve at Raneegunge	140, 142
Corelation between annual evaporation and annual rainfall	195	Formula for velocity at the Chas Road Bridge site on the —	233, 235
Correlation between cane acreage, hydro-isobaths and damaged area	342	History of—floods	146
Correlation between gauge levels of the river Damodar at Raneegunge and Rhondia and the gauge discharge curve at Raneegunge	140, 142	Hydrological possibilities in—	145, 146
		Unified development of the— valley	146, 148
		Unit Hydrograph of the — catchment above Chas Road Bridge Site	65
		Water balances investigation of the — catchment	23, 129, 133
		Damodar at Anderson weir	755
		Damodar at Ramgarh Road Bridge Site	154, 156
		Damodar at Raneegunge and Rhondia	149
		Damodar at Rangamati.	757
		Damodar at Sindu	210

	Pages		Pages
Damodar at Sudamdih Railway Bridge Site	154, 156	Development and Research Division	
Damodar Canal	754, 755, 760	Sind: Summarised Annual Report	33
Damodar Valley	132	Development of power	389
Dam profile	297	Development of power for canals for domestic, commercial and agricultural purposes and for the extension of agriculture	269
Dams	297, 625, 634	Resolution passed by the Board in 1946 on—	259
Dams and Weirs	599, 625	Dharla river	480, 491, 509
Consideration by the Board of—	625	Dhandraul reservoir	588
Discussion by the Research Committee on—	608, 620	Dhukwan reservoir	584, 585, 592, 594, 596
Preliminary Note on—	599	Diameter:	
The Year's Work on—	601, 610	Nominal—	395
Das, Shri S.	17	Sedimentation—	395
Datar Singh, Sardar	24	Sieve—	395
Definition of terms: in common use	395, 825	Dickens	111, 123, 124, 533
Consideration by the Board of—	397	Dickens Equation	121
Preliminary Note on—	395	Dildar Hussain, Syed	13, 15, 18, 151, 308, 307, 332, 512, 638, 624, 633, 690, 753
Deflector	516, 517, 530, 646, 647, 649, 683, 736, 767	Discharge and other hydraulic observations	440, 441, 442
Deolbari	146	Discharge elements: Relation: between different—for the river Brahmaputra at Jogigopa	233, 236, 238
Depression head	210, 265, 827, 828	Discharge sites: Analysis of hydraulic data from selected—along Barrage Canals during 1946	233, 244
Depth: Relation between plant root and shoot development and — of water table	320	Distributary Heads	718
Depth to which plant roots can penetrate in search of water	320	Consideration by the Board of—	753
Design of Canal Falls	709, 713	Discussion by the Research Committee on—	608
Design of Channels in Alluvium	234	Preliminary Note on—	599
Design of Channels in the Mor Project	233, 234	The Year's Work on—	601
Design of dams	599	Design of distributary heads: Central Board of Irrigation Publication—	718
Consideration by the Board of—	625	Design of drains in irrigated and tidal areas	255
Discussion by the Research Committee on—	608	Consideration by the Board of—	256
Preliminary Note on—	599	Discussion by the Research Committee on—	256
The Year's Work on—	601	Preliminary Note on—	255
Design of distributary heads: Central Board of Irrigation Publication—	718	The Year's Work on—	256
Design of drains in irrigated and tidal areas	255	Design of irrigation channels	254
Consideration by the Board of—	256	Consideration by the Board of—	254
Discussion by the Research Committee on—	256	Preliminary Note on—	254
Preliminary Note on—	255	Design of Lower Sind Barrage at: Kotri for effective sand exclusion from canals	754, 770
The Year's Work on—	256	Design of pavements of regulators on clay and sand foundations	701
Design of irrigation channels	254	Design of tubewells	824, 829
Consideration by the Board of—	254	Design of weirs on permeable foundations	701
Preliminary Note on—	254	Deterioration of the river Kultigong	545, 546
Design of Lower Sind Barrage at: Kotri for effective sand exclusion from canals	754, 770	Determination of the optimum number of rain recording stations required in catchments for estimating runoff— by V. Stekapan and V. Vittal Sharma	70
Design of pavements of regulators on clay and sand foundations	701	Detritus	395
Design of tubewells	824, 829	Sub-Committee for drafting instructions for the determination of—load in rivers and canal	23
Design of weirs on permeable foundations	701	Development and Research Division	
Deterioration of the river Kultigong	545, 546	Sind: Summarised Annual Report	33
Determination of the optimum number of rain recording stations required in catchments for estimating runoff— by V. Stekapan and V. Vittal Sharma	70	Development of power	389
Detritus	395	Development of power for canals for domestic, commercial and agricultural purposes and for the extension of agriculture	269
Sub-Committee for drafting instructions for the determination of—load in rivers and canal	23	Resolution passed by the Board in 1946 on—	259
		Dharla river	480, 491, 509
		Dhandraul reservoir	588
		Dhukwan reservoir	584, 585, 592, 594, 596
		Diameter:	
		Nominal—	395
		Sedimentation—	395
		Sieve—	395
		Dickens	111, 123, 124, 533
		Dickens Equation	121
		Dildar Hussain, Syed	13, 15, 18, 151, 308, 307, 332, 512, 638, 624, 633, 690, 753
		Discharge and other hydraulic observations	440, 441, 442
		Discharge elements: Relation: between different—for the river Brahmaputra at Jogigopa	233, 236, 238
		Discharge sites: Analysis of hydraulic data from selected—along Barrage Canals during 1946	233, 244
		Distributary Heads	718
		Consideration by the Board of—	753
		Discussion by the Research Committee on—	608
		Preliminary Note on—	599
		The Year's Work on—	601
		Design of distributary heads: Central Board of Irrigation Publication—	718
		Design of drains in irrigated and tidal areas	255
		Consideration by the Board of—	256
		Discussion by the Research Committee on—	256
		Preliminary Note on—	255
		The Year's Work on—	256
		Design of irrigation channels	254
		Consideration by the Board of—	254
		Preliminary Note on—	254
		Design of Lower Sind Barrage at: Kotri for effective sand exclusion from canals	754, 770
		Design of pavements of regulators on clay and sand foundations	701
		Design of tubewells	824, 829
		Design of weirs on permeable foundations	701
		Deterioration of the river Kultigong	545, 546
		Determination of the optimum number of rain recording stations required in catchments for estimating runoff— by V. Stekapan and V. Vittal Sharma	70
		Detritus	395
		Sub-Committee for drafting instructions for the determination of—load in rivers and canal	23
		Development and Research Division	
		Sind: Summarised Annual Report	33
		Development of power	389
		Development of power for canals for domestic, commercial and agricultural purposes and for the extension of agriculture	269
		Resolution passed by the Board in 1946 on—	259
		Dharla river	480, 491, 509
		Dhandraul reservoir	588
		Dhukwan reservoir	584, 585, 592, 594, 596
		Diameter:	
		Nominal—	395
		Sedimentation—	395
		Sieve—	395
		Dickens	111, 123, 124, 533
		Dickens Equation	121
		Dildar Hussain, Syed	13, 15, 18, 151, 308, 307, 332, 512, 638, 624, 633, 690, 753
		Discharge and other hydraulic observations	440, 441, 442
		Discharge elements: Relation: between different—for the river Brahmaputra at Jogigopa	233, 236, 238
		Discharge sites: Analysis of hydraulic data from selected—along Barrage Canals during 1946	233, 244
		Distributary Heads	718
		Consideration by the Board of—	753
		Discussion by the Research Committee on—	608
		Preliminary Note on—	599
		The Year's Work on—	601
		Design of distributary heads: Central Board of Irrigation Publication—	718
		Design of drains in irrigated and tidal areas	255
		Consideration by the Board of—	256
		Discussion by the Research Committee on—	256
		Preliminary Note on—	255
		The Year's Work on—	256
		Design of irrigation channels	254
		Consideration by the Board of—	254
		Preliminary Note on—	254
		Design of Lower Sind Barrage at: Kotri for effective sand exclusion from canals	754, 770
		Design of pavements of regulators on clay and sand foundations	701
		Design of tubewells	824, 829
		Design of weirs on permeable foundations	701
		Deterioration of the river Kultigong	545, 546
		Determination of the optimum number of rain recording stations required in catchments for estimating runoff— by V. Stekapan and V. Vittal Sharma	70
		Detritus	395
		Sub-Committee for drafting instructions for the determination of—load in rivers and canal	23
		Development and Research Division	
		Sind: Summarised Annual Report	33
		Development of power	389
		Development of power for canals for domestic, commercial and agricultural purposes and for the extension of agriculture	269
		Resolution passed by the Board in 1946 on—	259
		Dharla river	480, 491, 509
		Dhandraul reservoir	588
		Dhukwan reservoir	584, 585, 592, 594, 596
		Diameter:	
		Nominal—	395
		Sedimentation—	395
		Sieve—	395
		Dickens	111, 123, 124, 533
		Dickens Equation	121
		Dildar Hussain, Syed	13, 15, 18, 151, 308, 307, 332, 512, 638, 624, 633, 690, 753
		Discharge and other hydraulic observations	440, 441, 442
		Discharge elements: Relation: between different—for the river Brahmaputra at Jogigopa	233, 236, 238
		Discharge sites: Analysis of hydraulic data from selected—along Barrage Canals during 1946	233, 244
		Distributary Heads	718
		Consideration by the Board of—	753
		Discussion by the Research Committee on—	608
		Preliminary Note on—	599
		The Year's Work on—	601
		Design of distributary heads: Central Board of Irrigation Publication—	718
		Design of drains in irrigated and tidal areas	255
		Consideration by the Board of—	256
		Discussion by the Research Committee on—	256
		Preliminary Note on—	255
		The Year's Work on—	256
		Design of irrigation channels	254
		Consideration by the Board of—	254
		Preliminary Note on—	254
		Design of Lower Sind Barrage at: Kotri for effective sand exclusion from canals	754, 770
		Design of pavements of regulators on clay and sand foundations	701
		Design of tubewells	824, 829
		Design of weirs on permeable foundations	701
		Deterioration of the river Kultigong	545, 546
		Determination of the optimum number of rain recording stations required in catchments for estimating runoff— by V. Stekapan and V. Vittal Sharma	70
		Detritus	395
		Sub-Committee for drafting instructions for the determination of—load in rivers and canal	23
		Development and Research Division	
		Sind: Summarised Annual Report	33
		Development of power	389
		Development of power for canals for domestic, commercial and agricultural purposes and for the extension of agriculture	269
		Resolution passed by the Board in 1946 on—	259
		Dharla river	480, 491, 509
		Dhandraul reservoir	588
		Dhukwan reservoir	584, 585, 592, 594, 596
		Diameter:	
		Nominal—	395
		Sedimentation—	395
		Sieve—	395
		Dickens	111, 123, 124, 533
		Dickens Equation	121
		Dildar Hussain, Syed	13, 15, 18, 151, 308, 307, 332, 512, 638, 624, 633, 690, 753
		Discharge and other hydraulic observations	440, 441, 442
		Discharge elements: Relation: between different—for the river Brahmaputra at Jogigopa	233, 236, 238
		Discharge sites: Analysis of hydraulic data from selected—along Barrage Canals during 1946	233, 244
		Distributary Heads	718
		Consideration by the Board of—	753
		Discussion by the Research Committee on—	608
		Preliminary Note on—	599
		The Year's Work on—	601
		Design of distributary heads: Central Board of Irrigation Publication—	718
		Design of drains in irrigated and tidal areas	255
		Consideration by the Board of—	256
		Discussion by the Research Committee on—	256
		Preliminary Note on—	255
		The Year's Work on—	256
		Design of irrigation channels	254
		Consideration by the Board of—	254
		Preliminary Note on—	254
		Design of Lower Sind Barrage at: Kotri for effective sand exclusion from canals	754, 770
		Design of pavements of regulators on clay and sand foundations	701
		Design of tubewells	824, 829
		Design of weirs on permeable foundations	701
		Deterioration of the river Kultigong	545, 546
		Determination of the optimum number of rain recording stations required in catchments for estimating runoff— by V. Stekapan and V. Vittal Sharma	70
		Detritus	395
		Sub-Committee for drafting instructions for the determination of—load in rivers and canal	23
		Development and Research Division	
		Sind: Summarised Annual Report	33
		Development of power	389
		Development of power for canals for domestic, commercial and agricultural purposes and for the extension of agriculture	269
		Resolution passed by the Board in 1946 on—	259
		Dharla river	480, 491, 509
		Dhandraul reservoir	588
		Dhukwan reservoir	584, 585, 592, 594, 596
		Diameter:	
		Nominal—	395
		Sedimentation—	395
		Sieve—	395
		Dickens	111, 123, 124, 533
		Dickens Equation	121
		Dildar Hussain, Syed	13, 15, 18, 151, 308, 307, 332, 512, 638, 624, 633, 690, 753
		Discharge and other hydraulic observations	440, 441, 442
		Discharge elements: Relation: between different—for the river Brahmaputra at Jogigopa	233, 236, 238
		Discharge sites: Analysis of hydraulic data from selected—along Barrage Canals during 1946	233, 244
		Distributary Heads	718
		Consideration by the Board of—	753
		Discussion by the Research Committee on—	608
		Preliminary Note on—	599
		The Year's Work on—	601
		Design of distributary heads: Central Board of Irrigation Publication—	718
		Design of drains in irrigated and tidal areas	255
		Consideration by the Board of—	256
		Discussion by the Research Committee on—	256

	Pages		Pages
		E	
East, Mr. L. R. (Australia) :		Emergent measures for preventing serious damage to the central sluices for the Mahanadi Anicut at Cuttack, Orissa	518
Message from—	22	Energy dissipating devices (dissipators)	518, 519, 638, 677, 687
Eastern Ghats	231	Energy dissipation	636, 638, 644, 647, 648, 649, 650, 666, 669, 670, 671, 672, 674, 682, 685, 690, 693, 694, 696, 699, 700, 706,
Earth dam :		Erosion	207
Hydraulic gradient in homogeneous—	301	Erosion Control	148
Protection to water face of—	307	Erosion of the river Torsa	490, 491
Seepage of water through an—	297	Estimation of evaporation losses in reservoirs	195
Earth dams and embankments	297, 307, 309	Estimation of soil moisture under field conditions	324
Discussion by the Board of—	307	Estimation of water power	261
Echo sounding apparatus	236	Evaporating power	312, 331
Economical use of water	348, 349, 353	Evaporating regime	324
Economics of lining	193	Evaporation	312, 331, 333
Economics of Staunching	818, 822	Correlation between annual — and annual rainfall	195
Edgecombe, Mr. A.R.B.	395	Effect of surface cover on —	318, 319
Effect of angle of approach on the shape of the apron of a Throated span	490, 508	Effect of upward movement of salts on Experimental farms recording—	319
Effect of certain salts on the black cotton soil	322	Formula for calculating—from soil and free water surfaces	317
Effect of concentration of salts on capillary ascent	322, 323	Mean daily—	314
Effect of heavy cane irrigation on subsoil water-table in deep soil area near Jinti Distributary 18, Nira Right Bank Canal	340	Pan—	196
Effect of mixing sand with the Poona soil on capillary ascent	323	Penman's formula for—	318
Effect of perennial cane irrigation on the sub-soil water table and consequent damage to lands	340, 342	Piche—	196
Effect of scale on dam models	297	Poona formula for—	318
Effect of sewage irrigation, on black cotton soil	219, 220	Portion's formula for —	318
Effect of sodium salts on soils	215	Quantity lost by—	198
Effect of surface cover on evaporation	318, 319	Rate of —	333
Effect of tubewell pumping on sub-soil waterlevel	209, 824	Relation between—and distance between water table and the evaporating surface	317
Consideration by the Board of —	213	Evaporation and transpiration losses	213
Discussion by the Research Committee on	211	Consideration by the Board of	213
Preliminary Note on— }	209	Evaporation from free water surface	317, 331
The Year's Work on—	211	Evaporation from soil surface	312, 315, 316, 317, 331
Effect of upward movement of salts on evaporation —	319, 331	Evaporation inside different crops	314
Effluent :		Evaporation losses	195, 204, 313, 319, 332, 333
Sewage—	218	Consideration by the Board of—	204
Water—	217	Discussion by the Research Committee on—	195
Well waters in—zone	214	Estimation of—in reservoirs	195, 313
Electrical Analogy : The development of—		The Year's work on —	195
Methods in India	449	Evaporation variation with environments	314
Electrical analogy method	262, 263, 265, 266, 268, 825, 826	Evaporimeter :	
Electrical Analogy, Model of Ramapada-sagar Dam	328	A comparison of evaporation recorded with different types of—	314
Electrical Research	274, 275	Open pan —	312, 314
Electricity tariffs, accounts and procedure	282, 284	Piche —	312, 314
Elephant Butte Reservoir	595	Pickering s—	312
Embankments	144, 297, 308	Soil—	314, 315, 317, 319, 320, 321, 322, 331, 332
Construction of—	144		
Protection to the water face of	397		

Evaporimeter—contd.	Pages	Pages
U.S.A. Type of —	312, 314, 317	
Wild's —	312, 314	
Excavations	400	Factors affecting area irrigated and growth of crops 347
Excluders and ejectors	754	Fair incidence of cost of irrigation water in different classes of crops 347
Central Board of Irrigation Publication on —	754	Fall at R.D. 129, 500 Pakpattan Canal 709
Discussion by the Board on —	797	Fate of rainfall or irrigation 313
Discussion by the Research Committee on —	793	Ferozepur Headworks 132
Preliminary Note on —	754	Feteley and Steam's Meter 166
The Year's Work on —	754	Field capacity 296, 297, 303
Executive Committee of the Central Board of Irrigation—		Field capacity in terms of clay content 289, 296
Meeting of the —	23	Financial aspects of waterpower works 282
Election of members of the —	24	Consideration by the Board of — 284
Exhibition: Meeting of Research Officers to discuss the details of — to be organised at the Thomason College Centenary	23	Preliminary Note on — 282
Experimental farms recording evaporation	314	The Year's Work on — 284
Experimental stress analysis: Photoelastic methods	601, 602	Financial aspects of water utilisation projects 372
Experiments: Some — done at Krishnarajasagar to study the effect of turbulence on the velocity records obtained with cups and propeller types of meters, by K.K. Mahadevan	154	Discussion by the Board on — 376
Experiments: Water account —	198	Preliminary Note on — 372
Experiments conducted to find the effect of temperature on the results of ratings of current meters, by K.K. Mahadevan	154, 161	The Year's Work on — 372
Experiments in connection with the determination of (a) optimum moisture content (b) bearing capacity and (c) calcium chloride (Hygroscopic) required for optimum bearing power on different typical soils	289, 294	Fish culture 628, 629, 632
Experiments on dissipating the energy of flow from volute siphons	636, 669	Fish farms 632
Experiments on one span (part model) of Siphon in connection with proposed additions and alterations to the existing Tando Mastikhan Fall at R.D. 119, 117, of Rohri Canal.	709, 710	Fish passes 599, 626, 627, 628, 629, 630, 631, 632, 633
Experiments on Shaft Spillways for tanks	636, 672	Consideration by the Board of — 633
Experiments on staunching of canals with surkhi mortar	819, 820	Discussion by the Research Committee on — 632
Experiments on the effect of altering the shape and number of volutes in a volute siphon	636, 667	Preliminary Note on — 636
Experiments on the measurements of bed load	177, 185	The Year's Work on — 636
Experiments on the measurement of suspended load	177, 183	Fisheries Research Station 633
Experiments on the variation of co-efficient of discharge under varying degrees of submersion of siphon outlets	636, 667	Float:
Experiments on transmission co-efficients of sand in Sind	312, 324	Comparison of velocity observations by the — method and the current meter method 183
Experiments to determine water requirements for sugar cane and wheat crops for the maximum yield with the most economical use of water	348, 349	Velocity by—method 187
Extraction of tubes of tubewells	824, 839	Flood 137
		Analysis of — data 139
		Collection of — data 138
		Definition of major — 138
		High—s in Krishnarajasagar 140, 143
		Major— 137
		Maximum — 138
		Maximum — discharge from small catchments 132
		Publication of — data 139
		Flood absorption in Krishnarajasagar 462, 473
		Flood control 135, 144, 147
		The Collection and study of statistical data pertaining to river — 458
		Flood protection works 458
		Discussion by the Board on— 482
		Discussion by the Research Committee on — 482
		Preliminary Note on — 458
		The Year's Work on — 462
		Floods 135, 137, 458, 482
		Floods and their control: Resolution on 137, 458

Pages	Pages		
Floods in Sutlej and Jumna	130		
Flow : Theory of — and design of channels	229	Gadgil, Hon'ble Shri N.V.	23, 24
Flow in channels in coherent and incoherent alluvium	231, 233	Task before the architect : Inaugural speech at the Annual Meeting of the Board	1
Consideration by the Board of —	252	Gadkary, Shri S.A. (i), 23, 424, 436	
Discussion by the Research Committee on —	249	Galleries	213
Preliminary Note on —	231	Gandikota (Pennar) Project	643
Flow in channels with rigid and quasi-rigid boundaries	231	Ganesh Iyer, Shri V (ii), 634, 684, 685, 697, 699	
Consideration by the Board of —	252	Ganesh Iyer Volute Siphons	See Spill-way siphons.
Discussion by the Research Committee on —	233		
The Year's Work on —	231	Gangao Reservoir	592
Flow in rivers and canals	231	Gangapur Dam near Nasik	367
Consideration by the Board of —	252	Ganges River	145, 151, 292, 581
Discussion by the Research Committee on —	233, 249	Garg, Shri Gita Ram (i), 23, 424, 436	
Preliminary Note on —	231	Garhya	592
The Year's Work on —	231	Gauge discharge curves for the river Brahmaputra at Jogigopa	140, 141
Fluming ratio	717	General—Section G	389
Flaming the Tungabhadra Low Level canal between miles 27/4 and 28/4	802	Germination of crops	349, 350
Forests : as a factor affecting infiltration	211	Ghaggar Canal Head	718, 725
Formal confirmation of the minutes of the sixteenth Research Committee Meeting	22	Ghats : Eastern —	231
Formative discharge	573	Ghotankar, Shri S.T.	17, 487, 530
Formulae for calculating evaporation from soil and free water surfaces	317, 318	Goalpara	141, 149, 236
Comparison of different —	318	Godavari River	145, 151, 214, 233, 328, 343, 524, 532, 536, 631
Formulae for velocity at the Chas Road Bridge Site on the Damodar	233, 235	Kondapondi Gorge of the —	231
Foundation of Khatima Power House, Sarda Canal	262	— Water	241, 218
Foundation pits	400	Gokhale, Dr. V. K.	363, 364
Foundation rock of the Ramapadasagar Dam	610	Gokhale, Shri and Shrimati B. K.	24
Foundations	307, 400	Gole, Shri C. V.	17, 797
Discussion by the Board on —	307	Gorari River	537, 539
Foundation trench of Ramapadasagar Dam	312, 328, 330	Gorrie, Dr. R.M.	16, 206, 207, 208
Foy, Mr. T.A.W. (i)			
Framji, Shri K.K. (ii) 15, 21, 23, 69, 70, 127, 128, 136, 149, 150, 151, 152, 153, 175, 176, 192, 203, 204, 250, 251, 274, 306, 362, 363, 397, 429, 433, 434, 435, 452, 453, 454, 455, 456, 457, 461, 489, 490, 510, 512, 513, 514, 515, 516, 517, 529, 530, 531, 532, 533, 534, 535, 544, 545, 564, 580, 582, 583, 689, 690, 691, 692, 693, 697, 698, 699, 700, 715, 716, 717, 752, 753, 794, 795, 796, 797, 798, 800, 827		Govinda Rao, Shri N.S.	16, 21, 69, 70, 150, 153, 174, 196, 305, 332, 333, 355, 362, 451, 488, 534, 535, 610, 622, 623, 624, 625, 684, 685, 690, 691, 692, 693, 697, 796, 821, 822
Friction coefficient	535	Grading of soil particles	292
Fundamental research in soil mechanics	289, 307	Grand Coolce	652
Discussion by the Board on —	307	Grazing Control	206, 208
Fundamentals : Sub Committee on — of Soil Research	23, 310	Green, Mr. Kenneth	17, 18, 339, 433
		Ground Slope : as a factor affecting infiltration	211
		Ground water	211, 212, 400
		Velocity of —	212
		Ground water resources	131, 212
		Grouting	823
		Consideration by the Board of —	
		Preliminary Note on —	823
		Grouting for staunching	
		Grouting of foundations	823
		Growth of crops	394, 350

	Pages		Pages
Rate of —	210	Jogigopa	149, 177, 178, 233, 233, 237, 249
Infiltration (absorption by ground) factors which determine the portion of rainfall which infiltrates while also considering the effects of plants, the configuration of the soil etc., and the importance of the influences, by D.V. Rao	211	Distribution curve of Brahmaputra bed silt at —	182
Infiltration galleries	210, 827	Gauge—discharge curves for the river Brahmaputra at —	140, 141
Influence of the distance between the Soil surface and the free watertable on the root and shoot development of plants.	319	Jogigopa Rock	238
Informal Winter Meeting of the Research Committee	15	Joglekar, Shri D. V.	16, 21, 23, 173, 210, 211, 233, 251, 305, 456, 512, 515, 564, 578, 580, 582, 624, 685, 619, 692, 695, 696, 697, 716, 717, 796, 816, 817
Inglis, Sir Claude C	203, 395, 586, 588, 594, 753, 766, 797	Jog Power Scheme	384, 666
Inglis, Modified Formula	112, 121, 123, 124, 553, 676	Jog Volute Siphons	667
Inglis' Original Formula	114	Jumna River	490, 494, 512
Inlet and outlet works	626, 633	Floods in —	132
Insulating material; Research on —	273	Regenerations of — above Okhla	197
Integrating rain—gauges	439	Jumna river at Tajewala	754, 792
Intensities of Irrigation	335, 337	K	
Intensity : Watering —	336	Kabraji, Dr. K. J.	16, 69, 128, 129
International conference on Soil Mechanics and Foundation Engineering : Meeting of the National Committee for India	23	Kahai, Sardar, H. S.	363, 364
Investigations (Dams and Weirs)	599	Kanva	684
Consideration by the Board of —	625	Kanwar Sain, Shree	23, 132, 133, 203, 376
Preliminary Note on —	601	Kapur, Shree B. K.	176, 192, 395
The Year's Work on —	601	Karamnasa river	676, 725
Invisible condensation of water vapour	325	Karim, Mr. A.	15, 152, 210, 305, 338, 362, 363, 622, 624, 795, 822, 827, 828
Irrigation	145, 147, 338, 345	Katikund	68
Artificial —	333	Katjuri	462, 463, 464, 465, 467, 470, 471, 472, 487, 488
Effect on yield of varying frequencies of —	349, 350	Kennedy's Critical Velocity Theory	234
Effect on yield of varying intensities of —	349, 350	Kennedy's diagram	234
Efficiency of —	198	Keti Island	754, 784, 785
Fate of —	313	Khandala drainage scheme	203
Heavy cane —	340	Khanna, Shree A. R.	23, 839
Intensity of —	333, 335, 337	Khanna, Shree D. K.	18
Perennial cane —	340, 342	Khatima Power House	262
Spray —	392	Khosla, Shree A. N.	(i), 15, 23, 24, 130, 132, 192, 204, 253, 273, 274, 280, 284, 346, 376, 377, 379, 383, 384, 386, 412, 415, 416, 417, 418, 423, 424, 439, 440, 441, 453, 454, 455, 456, 457, 531, 532, 536, 563, 564, 590, 594, 595, 598, 609, 610, 628, 621, 622, 623, 624, 625, 670, 689, 690, 691, 692, 698, 699, 700, 701, 702, 703, 706, 707, 708, 826, 839
Sub-soil —	319, 322	Presidential Address delivered at the Annual Meeting of the Board	4
Irrigation Enquiry Committee (Bombay)	352	Presidential Address delivered at the Annual Meeting of the Research Committee	18
Irrigation of fruit gardens	392	Kistna	145, 159
Irrigation Research Station Madras : Summarised Annual Report	34	Konar	140
Islam Headworks	708	Koncha Bhanvar Reserwei.	592
Islands	572, 573, 574, 575, 579, 580, 581	Koncha Reservoir	586
Artificial — for training of rivers	494, 511, 512, 517, 573	Kosi	145, 632
Attracting —	782, 799		
Iyer, Shri V. Ganesh	See Ganesh Iyer.		
Jaswant Singh, Lt.-Col.	16, 23, 383, 385, 386, 387		
jetty	400		
Jharam Kaver	490, 507		
Jobra	462, 463, 465, 518		

	Pages		Pages
Krishna, Dr.	363, 364	Irrigation	444
Krishnarajasagar	617, 618, 699	Irrigation Cess	445
Combined stream gauging and raingauging station at —	66	Land acquisition, improvement etc.	446
High floods in —	140, 143	Minor irrigation	445
Hydraulic Research Station : Summarised Annual Report	44	Miscellaneous	446
Rating curves of —	66	Navigation	445
Some experiments done at — to study the effect of turbulence on the velocity records obtained with cups and propeller types of meters, by K. K. Mahadevan	154	Private irrigation	445
Krishnarajasagar catchment area	66	River causeway and improvement	446
Krishnarajasagar Dam 376, 473, 601, 603, 610, 611, 616, 620, 622, 623, 624	384	Tank improvement	445
Krishnarajasagar Project	384	Tubewells and underground waters	445
Krishnarajasagar Rating tank 161, 162, 164, 172, 174	66, 143, 462, 473, 481, 488	Legislation to control and assist in the proper development and conservation of the water resources of the country	378, 379, 444
Krishnarajasagar Reservoir	66, 143, 462, 473, 481, 488	Lightning resistivity of soils	140
Runoff from Catchment around — water spread	65	Limitations of Model Experiments and comparison of Models and Prototypes	449
Kuakhai	462, 463, 470, 471	Consideration by the Board of —	457
Kultali	565, 566, 567, 568, 578	Discussion by the Research Committee on —	452
Kultigong River	545, 546, 547, 563	Preliminary Note on —	449
Kumar River	537	The Year's Work on —	450
Kumar, Shree S. S.	23	Limiting salt content when water can be said to be injurious	215
Kutar, Mr. Phiroz	418, 423	Lining : Economics of —	193
Kutter's N.	249	Lining material	819
Kuttiammu, Jenab T. P.—Sahib Bahadur (ii), 16, 21, 174, 233, 332, 417, 454, 532, 535, 536, 608, 620, 621, 682, 683, 684, 685, 689, 697, 707, 715, 816, 817	249	List of existing Indian Legislations regarding the utilisation and conservation of natural waters	444
Lacey, Gerald	771	Lloyd Barrage	702, 754, 764
Lacey's basic equation	237	Loam : Bed —	396, 397
Lacey's channel	714	Bed Material	396, 397
Lacey's diagram	235	Contact —	395, 397
Lacey's 'f'	250, 251	Saltation —	395, 397
Lacey's formula	236, 237, 556, 557, 558, 716	Sediment —	396
Lacey's regime condition	250	Wash —	395, 397
Lacey's silt factor theory	235	Loamy soil	294
Lachura Reservoir	592	Losses from Reservoirs	584
Lakhavalli Project	384	Lower Bhavani Project	643
Lakhmanathirtha	66, 143	Lower Sind Barrage at Kotri	754, 770, 771, 784
Lalitpur Dam	302, 303, 371	Luni River	490, 493, 494, 512, 573, 579
Land Reclamation	338, 357, 358	Lysimeter experiments	312, 331, 337
Questionnaire on —	334		
Land Utilization Board	205, 206, 207, 208, 425	M	
Leaching	337	Madras Irrigation Research Station : Summarised Annual Report	34
Legal aspects of water utilisation projects Consideration by the Board of —	378 379	Mahadevan, Shri K.K.	17, 154, 161, 164
Preliminary Note on —	378	Mahanadi River	145, 462, 463, 465, 466, 470, 471, 472, 487, 488, 518, 530, 583
Legislation for acquisition of land for transmission of power	280, 281	Mahanandi Hill	521, 526
Legislations : List of —	444	Mahbub, Mr. S. I.	801
Drainage	446	Maithon Dam	136, 146, 291
		Majumdar, Shri S. C.	(i), 23, 379
		Malampuzha Project	643
		Malaprabha River	510
		Malaria Control	143, 233
		Malegaon Cut	383
		Malhotra, Shri S.L.	(ii) 28, 397

	Pages		Pages
Marconhalli	699	Measurement of precipitation in hilly catchments with special reference to problems of runoff and other hydrologic studies, by L. Y. Baker	76
Marikanave Dam	611	Measures adopted to reduce turbulence in the Hooghly river downstream of the knuckle at the Titaghur Jute Mill (Bengal) on ———	565, 569
Materials for dam construction	599	Mechanical analysis	292, 294, 296, 327
Consideration by the Board of ———	625	Meetings of the Board and the Research Committee	15
Discussion by the Research Committee on ———	620	Mehra, Shri S. R.	16, 23, 304, 306, 308
Preliminary Note on ———	599	Mehta, Shri D.	(i), 23, 376, 583
The Year's Work on ———	610	Mehta, Shri M. L.	363, 364
Materials for grouting	823	Members : List of ——— of the Central Board of Irrigation as on December 1, 1947	(i)
Materials for staunching	818, 819, 822	Meshwa Canal Project, Bombay : Design of the weir and head regulator on pervious foundations, by Shri N. P. Gurjar	367, 369
Mathews, Mr. H. M.	(i)	Metering of supplies	718
Mathrani, Shri H. P.	16, 21, 196, 303, 412, 414, 416, 417, 418, 419, 423, 429, 454, 455, 456, 530, 531, 532, 564, 622, 716, 796, 839	Meters :	
Mathrani, Shri M. P.	(i), 13, 15, 18, 21, 70, 137, 204, 306, 309, 363, 530, 531, 564, 620, 623, 707	Rating of ———	153, 159
Matla	565, 566, 567, 568, 578	Some experiments done at Krishnarajasagar to study the effect of turbulence on the velocity records obtained with cups and propeller types of—by K.K. Mahadevan	154
Matla—Peali System	565, 566, 578	Method of assessment of tubewell waters	825
Maximum bearing power	296	Method of coordination for the full utilisation of water resources for irrigation in the United Provinces, by J. W. Russell	388
Maximum consolidation	295	Mettur Dam	624, 631
Maximum dry density	292, 293, 302	Meyer's formula	112, 124
Maximum flood discharges from small catchments, by A. R. Thomas	111	Minor tanks : Restoration of ——— in Mysore	367
Maximum moisture content	304	Miscellaneous (Projects)	388
Maximum runoff discharge	70, 132	Consideration by the Board of ———	388
Consideration by the Board of ———	129	The Year's Work on ———	388
Discussion by the Research Committee on ———	123	Mitra, Shri, J. N.	23
Maydeo, Shri A. G.	15, 21, 123, 128, 196, 203, 218, 219, 228, 306, 310, 340, 346, 353, 354, 357, 488, 596, 623, 624	Mittal, Shri A. C.	17
Mead, Dr. Daniel W. "Notes on Hydrology"	145	Model experiments for exclusion of silt from Damodar Canal	754, 755
Meander	552, 561, 562, 564, 579	Model experiments for the prevention of erosion at Kurigram by the River Dharla	490
Meander belt	552, 563	Model experiments for the revival of the Matla-Peali System	565
Meander conditions	532	Model experiments for the revival of the River Chandana	536, 537
Meander length	556, 558, 770, 785	Model experiments in connection with scour below cross regulator over Sind Canal at Rustom	718, 733, 752
Meandering	458, 459	Model experiments in connection with scour hole below head regulator over Rohri Canal	450, 452
Discussion by the Board on ———	583	Model experiments in connection with silting of Dalore and Patoi distributaries ex Jamrao Canal	718, 741, 752
Discussion by the Research Committee on ———	563	Model experiments in connection with silting of Jamrao Canal	718, 746, 752
Preliminary Note on ———	461	Model experiments in connection with silting of Saidkhan distributary ex Hyderabad Branch	718, 736, 752
The Year's Work on ———	545	Model experiments on the exclusion of sand from a feeder channel taking off from Pennar River	754, 790
Meandering of rivers	458		
Board's Resolution on ———	461		
Meandering rivers and flood control, by K. B. Ray	462, 485		
Mean moisture per cent. expressed on dry basis	324		
Mean velocity : Relation between surface velocity and—	161		
Measurement of flowing water	153		
Consideration by the Board of ———	175		
Discussion by the Research Committee on ———	172		
Preliminary Note on ———	153		
The Year's Work on ———	153		

Pages	Page
Model investigations for the proposed Mor Barrage at Tilpara	490, 491
Model investigations regarding the design of Nagwa Dam	637, 676
Model investigations regarding the design of Pathri Power House	262, 269
Model investigations regarding the Project of Mohammadpur Power House	262, 268
Model limitations as regards bank erosion and accretion upstream and downstream of a tree groyne	450, 451
Mohammadpur Power House	262, 266, 268, 269
Mohinder Singh, Sardar	600
Mohsin Ali, Mr.	(i)
Moisture :	
Hygroscopic	324
Movement of — in saline soils	303, 322, 323
Movement of — in saturated and unsaturated soils	310
Consideration by the Board of —	334
Discussion by the Research Committee on —	332
Preliminary Note on —	310
The Year's Work on —	312
Zone of constant —	338
Moisture capacity	333
Moisture conservation	319
Moisture content	289, 290, 291, 295, 296, 306, 308, 309, 324, 332, 339, 595
Moisture movements	312, 331, 335
Mondal, Shri G. B.	23, 442
Mor Project	250, 490, 491, 492, 708
Design of channels in the —	233, 234
Mor River	140, 149
Correlation between the gauge heights at the Sainthia Railway Bridge site and the Tilpara Ghat Barrage Site on the —	140
Mountbatten : Their Excellencies Lord and Lady — At Home	24
Movements of water and salt through soils	323
Mula	462, 481, 482, 483, 484
Murum soils	306
Mustafa, Syed Ghulam	(i)
N	
Naagamvala, Shri J. P.	(ii), 23
Nagpur Sewage Farm	220
Nagwa Dam	302, 303, 637, 676, 698, 725
Nagwa Dam Site	677
Nalhatti—	
Bridge at —	68
Unit Hydrograph of the Brahmani catchment above —	50
Nandur Khandala Drainage Scheme	199
Nangal Hydrel Project, by Shri S. A. Gadkary	372, 376
Nangal Project	373, 374
Nangal Transmission System	373, 374
Naraj	462, 463, 464, 465, 466, 467, 471, 472, 487, 488
Narasimhaiya, Rajakaryaprasakta, M.	(i), 23, 24, 137, 275, 358, 367, 369, 370, 371, 376, 377, 384, 385, 386, 387, 434, 699, 839
Narayanan, Shri R. L.	(i), 13, 23, 273, 279, 284
Navigation	144, 145, 148, 233
Nayar River	595
New simple method of estimating soil moisture	324
Nira	199, 200, 201, 202, 203
Nirgajni Power Station	262, 271
Nizamsagar Project	356
Nomograms	399
Preliminary Note on —	399
Notation : Standard —	398
Note on trade waste experiments in the Bombay Province	227
O	
Observations of suspended load at Dhukwan	584
Off-taking channel	718, 723, 741
Okhla : Jumna above —	197, 203, 204
Open pan evaporimeter	312
Operating costs of water power works	282
Operating machinery of tubewells and their tests	824, 839
Optimum bearing capacity	296
Optimum bearing power	289, 294, 296
Optimum consolidation	294
Optimum moisture content	289, 292, 293, 294, 296, 302, 303
Optimum number of rain-recording stations : Determination of—required in catchments for estimating runoff	70
Optimum shape of the nose of Keti Island at Kotri for the Lower Sind Barrage	754, 784
Organisation of tubewell system	825, 839
Other (hydraulic) works	626, 633, 802
Consideration by the Board of —	817
Discussion by the Research Committee on —	816
Preliminary Note on —	802
The Year's Work on —	802
Outlets or sluices	801
Consideration by the Board of —	801
Discussion by the Research Committee on —	801
Preliminary Note on —	801
The Year's Work on —	801

	Pages		Pages
Paddy: Experiments to determine requirements of water by — crops	351, 353	P—contd	
Padma River	537, 538, 539, 540, 541, 544, 545	Power House and equipment	262
Pahari Reservoir	588, 592, 594, 595	Board's Resolution on —	276
Pahuj Reservoir	592	Consideration by the Board of —	273
Pahuj Weir	636, 657, 662, 684, 693,	Discussion by the Research Committee on—	273
Palampur	592	Preliminary Note on —	262
Panchang, Shree G. M.	16, 124, 125, 126, 127, 129, 203	The Year's Work on —	262
Paricha Reservoir	595	Power Transmission lines	278
Pathri Power House	262, 269	Land under —	280
Peadi	565, 566, 567, 568, 578	Pramnik, Shri H.R.	17, 609, 793
Pellicular zone	338	Pravara	199, 200, 203, 343, 462, 481, 482, 483, 484
Penetration depth of different intensities of irrigation	335	Precipitation	132
Penetrations	296	The measurement of — in hilly catchments with special reference to problems of runoff and other hydrologic studies	76
Pennar Project	643	President: Election of —	23
Pennar River	754, 790	Presidential address, by Shri A.N. Khosla Annual Meeting of the Board	4
Panstocks	626, 633	Annual Meeting of the Research Committee	18
Percolation	211, 212	Pressure distribution on Ogee Spillways: Tungabhadra Spillway Dam	636, 639
Percolation Losses	342	Pressure distribution on the Ramapadasagai Spillway Dam	636, 640
Percussion rigs	828	Pressures on Lloyd Barrage Floor	702
Permeability Coefficient	328	Preventing scour below Ganesh Iyer Volute Siphon Spillways by mutual impact of jets from adjacent siphon outlets	636, 665
Permeability of rock formation	212	Price Meter	165, 167, 169, 170, 171, 172, 174
Permeable groyne	451, 452, 453, 495, 497, 498, 500	Priming	634, 668, 689, 690, 710, 711, 816
Photo-elastic methods	601, 602, 608, 609	Priming Depth	657, 662, 663, 664, 666, 667, 668, 669, 681, 684, 685, 694, 695, 696, 697, 699
Phreatic line	330	Priming Head	634
Phreatic line of seepage flow	328	Priming Siphon	695
Piche Evaporimeter	196, 312	Problem of Fish Culture in Irrigation and Hydro-electric Reservoirs, by S. L. Hora	628
Piekering's Evaporimeter	312	Proctor's Moulds	292
Piers: Scour around—	518, 519, 527, 534, 559, 560, 561	Project for constructing a dam at Gangapur near Nasik, Bombay: Feasibility of constructing the dam in earth, by Shri N.P. Gurjar	367, 369
Pitched embankment for training of river	494, 512	Proposed railway bridge near Shahabad on the Ramganga River	545, 551
Pitched islands for training of rivers	494, 507, 512, 517, 529, 569, 572, 776, 782, 790	Protecting the foreshore of the Anglo-India Jute Mill on the Hooghly River (Bengal)	565, 575
Plant:		Protection of Usehat Bund for Wave Wash with stabilized soil	289, 292
Depth to which — roots can penetrate in search of water	320	Prothero, Mr. E.L.	(i), 13, 18
Root and shoot development of —s	319, 320, 321, 322, 331	Proximity of power transmission lines and the tele-communication lines	280
Plant accessories	276	Public Electricity Supply All India Statistics	282, 283
Polarisation	609	Pumping: Recharging the underground basin for future—	209
Polariscope	610	Punjab Irrigation Research Institute, Lahore: Summarised Annual Report	36
Pollution: River —	391	Puri Siltometer	241
Poona Irrigation and Research Division, Poona: Summarised Annual Report	33		
Poondi Regulator	702, 706		
Porosity	212		
Power	147		
Power and telecommunication engineers meeting	279		
Power House:			
Khatima —	262		
Mohammadpur —	262, 266, 268, 269		
Pathri —	262, 269		

Q	Pages	R—contd.	Pages
Quality of Irrigation Waters	214, 357, 358	Ramapadasagar Overflow Dam	536, 650
Quality of waters in relation to (i) crops (ii) soils, (iii) structures and (iv) indus- tries	214	Ramapadasagar Project.	233, 518, 521, 532
Consideration by the Board of —	228	Ramapadasagar Spillway Dam	536, 540, 641, 642, 643, 652, 655, 683, 693
Discussion by the Research Committee on —	218, 228	Ramdas, Dr. L.A.	310, 331
Preliminary Note on —	214	Rameshwar Saran, Shri	829
The Year's Work on —	214	Ramganga River	545, 551, 561, 562
Quality of waters in relation to indus- tries	228	Ramgarh	154, 156, 157
Quality of waters in relation to soils	219	Randomised block system	349, 350, 351
Quality of waters in relation to structures Consideration by the Board of —	227 228	Raneegunge	140, 142, 149
Discussion by the Research Committee on —	228	Rao, Shri D.V.	211, 212
Quality of yield of tubewells	824	Rapti River	489
Questionnaire on Land Reclamation	334	Rasul Hydel Scheme Falls	709, 713, 716
R		Rasul Hydel Scheme Spillway Siphon	637, 681
Radial gates	718, 726, 728, 729, 730, 732, 751, 753	Rasul Power Station	308
Radials	825, 826, 827	Rate of sand attrition in channels	233, 240
Radial tubewells	825, 826, 827, 828, 829	Rating of meters	173
Raghubir Singh, Sardar (East Punjab)	600	Effect of the non-uniformity of velocity on —	159, 174
Raghubir Singh, Sardar (Patiala)	23	Effect of temperature on —	162
Raghunathchak	142	Effect of velocity range on —	159, 173
Rainfall	212, 349	Method of —	161
Amount and intensity of — as factor affecting infiltration	211	Rating of meters under normal conditions and with the introduction of turbulence	171
Artificial —	See Arti- ficial Rain- fall.	Ratio of the bed load to suspended load	188
Correlation between annual evaporation and annual —	195	Ravi River : Spur at Baherian	490, 507
Fate of —	313	Ravi River at Dhanna Weir	754, 792
Relation between rate of silting and —	584, 585, 586	Ray, Shri K.B.	462, 485, 488, 489
Rainfall and evaporation	440, 441, 442	Recharging the underground basin for future pumping	209, 825
Rainfall Runoff	49	Consideration by the Board of —	213
Preliminary Note on —	49	Discussion by the Research Committee on —	212
The Year's Work on —	50	Preliminary Note on —	209
Rainfall Runoff Experiments in Trays	54	The Year's Work on —	211
Rain-recording stations : Determination of the optimum number of — required in catchments for estimating runoff	70	Reclamation	335, 337, 340
Raja Raja Varma, Shri K (i),	13	Land —	334
Raja Rao, Shri	17, 251	Reclamation by drainage	340
Raju, Dr. S. P. (ii), 15, 21, 23, 69, 70, 129, 150, 152, 196, 207, 280, 305, 356, 364, 385, 429, 434, 455, 456, 534, 608, 610, 621, 822		Reclamation of gullied lands	207
Ramanathan, Dr. K.R.	23, 108, 127, 130, 133, 134, 439, 442	Reclamation of <i>thur</i> soils	303
Ramapadasagar	533	Reclamation rice	331, 332
Ramapadasagar Dam	312, 328, 417, 521, 524, 610, 622	Reclamation watering	335
		Recovery of water	199
		Re-distribution of sand load between the Mahanadi and Katjuri systems at Cuttack	462
		Reduction of afflux at sluices, culverts and inverted siphons	802, 813
		Regenerated flow	199
		Regeneration	144, 197, 204
		Consideration by the Board of —	204
		Discussion by the Research Committee on —	203
		Preliminary Note on —	194
		The Year's Work on —	197
		Regeneration and losses	193, 594, 818,

	Pages	Pages	
R—contd.			
Regeneration of the river Junna above Okhla	197	River Research Institute Bengal : Summarised Annual Report	30
Regime	234, 395	Rivers :	
Regime flow	238	Characteristics of principal — of India	145
Regional planning	444	Meandering of —	458
Regrassing	148	Rivers and canals : Sub-committee for drafting instructions for the determination of detritus load in —	23
Regulation	277	Rivers Chenab and Jhelum at Trimmu	490, 507
Rehbock's Dentated Sill	646	River Sutlej at Samasata Railway Station	490, 508
Relation between different discharge elements for the river Brahmaputra at Jogigepa	233, 236	River training	144
Relation between rate of silting and catchment area	584, 586	Central Board of Irrigation Publication on —	459
Relation between rate of silting and rainfall	584, 585	River training by artificial inslands	494
Report on soil moisture and evaporation investigations being conducted by the Agricultural Meteorological Section at Poona, by Dr. L.A. Ramdas	312	River training works	458
Research Committee ;		Discussion by the Board on —	582
Informal Winter Meeting	15	Discussion by the Research Committee on —	509
Presidential Address	18	Preliminary Note on —	459
Seventeenth Annual Meeting	15	The Year's Work on —	490
Research Officers : Meeting of — to discuss the details of the exhibition to be organised at the Thomason College Conventary	(ii)	Roads and runways	367
Reservoirs :]	584	Discussion by the Board on —	367
Discussion by the Board on —	597	Rocks : The geological and structural features of the underlying — affecting infiltration	211
Discussion by the Research Committee on —	594	Rohri Canal : Model experiments in connection with scour hole below head regulator over —	450, 452
Estimation of evaporation losses in —	195	Roller bucket for the Ramapadasagar overflow dam	636, 650
Losses from —	584	Rotary rigs	209, 824
Preliminary Note on —	584	Rotation of crops	332, 338, 356, 357, 358
Silting of —	584	Rowtherp, Carl —'s formula	313, 331
The Year's Work on —	584	Roy, Mr. James L.	(i)
Resources surveys	152	Roy, Shree C.B.	(i)
Retaining walls	400	Roy, Shree P.B.	17, 578
Retaining walls, revetments and trenches	307	Rugosity	535
Discussion by the Board on —	307	Runoff :	
Return flow	199	Determination of the optimum number of rain recording stations required in catchments for estimating —	70
Figures of —	198	Maximum —	132
Return flow by seepage into natural drainages (nallas) in the canal areas	197	Maximum — discharge	70
Revetments	307	Rate of —	589
Rhondia	140, 142, 149, 439, 793	Relationship between period of concentration and — discharge in a given catchment	117
Rigid structures : Scour downstream of —	518	The measurement of precipitation in hilly catchments with special reference to problems of — and the hydrologic studies	76
Ripdaman Singh, Sardar	17	Total volume of — including the standardization of hydrographs	50
River behaviour	135, 231, 458, 582	Runoff and erosion experiments	440, 441, 442
Consideration by the Board of —	137	Runoff from catchment around Krishnarajasagar Reservoir Waterspread	65
Preliminary Note on —	135	Runways	306, 307
River behaviour, training and control	458, 461	Rupar Headworks	132
River Control	458	Rupnarain River : The — upstream of the goods-yard, Kolaghat Railway Station upstream of the Bangal Nagpur Railway Bridge	490, 499
Discussion by the Research Committee on —	486, 508		
Preliminary Note on —	458		
The Year's Work on —	462		
River Pollution	391		
Consideration by the Board of —	391		
Preliminary Note on —	391		

	Pages		Pages
		R—contd.	S—contd.
Russell, Mr. J. F.	(i), 137	Schoklitch's formula	210, 690
Russell, Mr. J.W.	388	Scour	459, 636, 637, 638, 644, 648, 650, 651, 652, 653, 654, 657, 665, 667, 683, 685, 687, 689, 693, 712, 713, 714, 718, 725, 733, 736, 752, 788, 789, 790
Ryves	111, 553	Scour and erosion	458
Ryves' Equation	121, 123	Discussion by the Board on ———	583
		Discussion by the Research Committee on ———	529
		Preliminary Note on ———	460
		The Year's work on ———	517
Sabri	231	Scour downstream of a five vented spillway dam for various combinations of shutter openings	636, 637
Saddle type siphon	See siphon.	Scour downstream of rigid structures in wide channels—prototype data regarding depth of scour downstream of bridges	518, 519
Sadhbelo Island	764	Scraping of grass	208
Saha, Shri B.	17	Sedimentation	152, 306, 534
Sainthia Railway Bridge	140, 149	Sedimentation diameter	395
Correlation between the gauge at ——— site and the Tilpara Ghat Barrage site on the river Mor	140	Sedimentation discharge	151, 152, 395
Saline lands	334, 335	Sedimentation load	396
Saltation load	395, 397	Sedimentation of reservoirs	514, 517
Salt content of waters	216	Discussion by the Board on ———	593
Salts :		Discussion by the Research Committee on ———	594
Removal of ——— by reclamation	337	Factors affecting ———	594
Vertical movement of ———	335, 336, 337	The Year's work on ———	584
Sampathiengar, Shri M.C.	610, 611	Sedimentation tanks	564
Sampler : Bottle ———	177, 178	Sediment investigation : River Jamna at Tajewala Head works	754, 792
Sampler for use in deep rivers	192	Sediment investigation : River Ravi at Dhanna Weir	754, 792
Samples :		Sediment investigation : River Sitlej at Hussiniwala Headworks	754, 792
Analysis of ———	178	Seepage	323, 329, 330, 820
Bed trench method of taking bed load ———	185	Relation between reduction in ——— and time	819, 820
Collection of ———	177, 178	Return flow by ——— into natural drainages (nulls) in the canal areas	197
Collection of suspended load ———	183	Seepage into the foundation trench of Ramapadasagar Dam	312, 328, 303
Sampling of bed and suspended load	176	Seepage lines	191, 201, 818, 821
Consideration by the Board of ———	192	Seepage losses	191, 201, 818, 821
Discussion by the Research Committee on ———	191	Consideration by the Board of ———	204
Preliminary Note on ———	176	Preliminary Note on ———	193
The Year's work on ———	177	The Year's Work on ———	195
Sand exclusion	754, 770, 777, 779, 780, 783, 789, 790, 792, 797	Seepage losses from canals and their prevention	193, 818
Sand stowing	148	Seepage of water through earth dam	297
Sand wave	395, 397	Selection of subjects for discussion at the following meeting	426
Sauwal, Shri K. D.	209, 824	Consideration by the Board of ———	432
Sardari Lal Sakni, Shri	17	Discussion by the Research Committee on ———	429
Sarup Singh, Sardar 4 (i), 23, 24, 132, 254, 275, 363, 370, 371, 376, 379, 435, 708, 797		Preliminary Note on ———	426
Satakopan, Shri V.	16, 70, 123, 124, 127, 128, 129, 331, 332, 333	Sen, Shri S	17, 23, 412, 415, 416, 417, 423
Saturation deficit	313	Senapati, Shri G	23, 583
Saturation gradient	305, 306	Separation	681, 685, 692, 693
Saturation line	297, 298, 299, 300, 301, 302, 303, 304, 305.	Sethi, Dr. R. L.	16, 218, 219, 346, 354, 355, 357, 362, 369, 364, 514
Effect of dam profile on ———	297		
Tracing of ———	302		
Saxena, Shri B.P.	17, 23, 584, 588, 594, 595, 623, 690, 827,		
Sea's effect	297, 301, 303, 697, 728, 723		

	Pages		Pages
		S—contd.	
Sethna, Shri T. R.	(ii), 15, 18, 21, 23, 173, 174, 175, 192, 207, 303, 331, 416, 453, 454, 455, 530, 531, 706, 707, 708, 754, 764, 797	Silting of reservoirs	584, 588, 590, 591, 592.
Seventeenth Annual Meeting of the Research Committee	15	Silting of Reservoirs, by Shri A.N. Khosla	598
Sewage effluent	218	Silting of Reservoirs, by Shri B. P. Saxena	594, 588
Sewage Irrigation	217, 218	Silt load	206, 588
Effect of — on black cotton soil	219, 220	Siltometer	182
Sugar cane experiments under —	219	Puri—	241
Sugarcane experiments under varying dozes of N in the form of — with special reference to the quality of gur	219	Silt Surveys	177, 189
Shaft Spillways	See Spillways	Sind: The Development and Research Division — Summarised Annual Report	38
Shape of dam profile	297	Sind Canal	718
Shape of spur apron for conventional types of spurs	490, 508	Sindri	210
Sharma, Shri V. Vittal	70, 124	Sinha, Shri G.	17, 68, 149, 172, 249
Shaw, Mr. J.	(i)	Singer, Lt. Col. C. M.	16, 23
Sheep's Foot Roller	308	Siphon	634, 635, 682, 684, 709, 710, 715, 716, 717
Sheet erosion	589	Auxiliary volute —	693
Sheet piles—	329, 400, 401, 402, 403, 404	Ganesh Iyer's volute —	See spillway siphons.
Types of —	401, 402, 404, 405, 406	Inverted —	802, 813
Sheet piling: Standardisation of steel —	400	Mutual interference of —	666
Sheffer, Dr.	131	Saddle — s.	657, 658, 659, 666, 684, 692, 693, 694.
Shelter belts	207	Self priming — s.	635
Shenoy, Shri R.C.	17	Volute —	662, 665, 667, 669, 671 684, 689, 698
Shifting sands	207, 208	Volute priming —	657, 684
Shrinkage curve	291	Siphon conduit	660
Shrinkage Dish	290	Siphoning of the Cumbum Tank Sluice	802, 815
Shrinkage factor of soils	239	Siphon outlets	636, 663, 665, 669, 670, 671, 672, 673, 674, 675, 684, 685, 689, 695.
Shrinkage limit	289, 290, 291	Siphon spillways	See spillway siphons.
Shrinkage ratio	289, 290, 291	Siphon Spillways for P'ahuj Weir	636, 657
Shrouding	827, 829	Sixteenth Research Committee: Formal Confirmation of the Minutes of —	22
Sieve diameter	395	Sluices	801, 813, 814
Silbit weir	676	Small catchments: Maximum flood dis- charge from — by A. R. Thomas	111
Sill	643, 644, 645, 646, 647, 648, 649, 650, 683, 685, 687, 689, 697, 767	Snow surveys	130, 131
Silt:		Soil:	
Distribution curve of Brahmaputra bed — at Jogigopa	182	Effect of mixing cement with —	292
Harmful — load	189	Effect of Sewage irrigation on black cotton	219, 229
Origin and nature of —	589	Stabilised —	289, 292
Tot I — load	189	Soil cement	306
Transportation of —	589	Soil conservation	206, 207, 208, 425.
Silt characteristics	234	Consideration by the Board of —	425
Silt charge	183, 189	Preliminary Note on —	425
Silt control	148, 718	Soil Conservation Bureaux	207
Silt distribution	718, 719, 722, 723	Soil cover as a factor affecting in filtration	211
Silt exclusion	754, 755, 760, 775, 793, 794, 797	Soil Denudation as an adverse factor on River supplies and agriculture	205, 425
Submerged vane as a device for —	777, 778, 779, 797, 798	Board's Resolution on —	208
Silt exclusion works at Lloyd Barrage at Sukkur, by T. R. Sethna	754, 764		
Silt Factor: Lacey's — Theory	234		
Silting	718, 735, 741, 746		
Relation between rate of — and catchment area.	584, 586		
Relation between rate of — and rainfall	584, 585, 586		

	Pages		Pages
Consideration by the Board of —	208	Soil Thermometer	324
Discussion by the Research Committee on —	206	Soluble Salt Content	337, 340
Preliminary Note on —	205	Some experiments done at Krishnarajasa- gar to study the effect of turbulence on the velocity records obtained with cups and propeller types of meters, by K. K. Mahadevan	154, 164
The Year's Work on —	206	Sonalapur Dam	136, 146
Soil Mechanics 144, 287, 308		Son River	145
Discussion by the Board on —	307	Son River Catchment	71
Discussion by the Research Committee on —	303	Sood, Shri M. L.	16, 204
Fundamental Research	289	Spacing to be maintained between power transmission lines of the Electricity Branch and the telephone and telegraph lines of the Indian Posts and Telegraphs Department, by S. A. Gadkary	278
Preliminary Note on —	287	Special instrument for the investigation of the movements of water and salt through soils	235
The Year's Work on —	289	Spill	458
Soil moisture 312, 331		Discussion by the Board on —	588
A new simple method of estimating —	324	Discussion by the Research Committee on —	544
Estimation of — underfield conditions	324	Preliminary Note on —	461
Soil moving machinery 207, 208		The Year's Work on —	586
Soil profiles : Depth to which moisture moves in —	335, 337	Spillway 637, 640, 654, 673, 677, 682, 683, 684, 696	
Soil Research : Sub-Committee on Fundamentals of —	23	Spiral — chutes	636
Soils : 219		Spillway dam	636, 637
Alluvial (normal) — from the Punjab 315, 317		Overflow — 634, 636, 643, 682, 696	
Bari (Alkali) — from the Punjab 315, 317, 319		Spillways	634
Black Cotton — from Poona 315, 316, 317, 319, 321, 322, 323.		Automatic siphon —	635
Clay — 294, 295, 296		Central Board of Irrigation Publication on Siphon —	634
Clay loam 294, 295, 296		Closed tunnel —	682
Compaction tests of — 292		Discussion by the Board on —	699
Compressive strength of — 292		Discussion by the Research Committee on —	682
Lightning resistivity of — 140		Ganesh Iyer Volute Siphon — 634, 636, 665, 666, 693, 699	
Mechanical analysis of — 292		Gated —	637
Red — from Bangalore 315, 316, 317		Glory Hole —	634
Sand — 294, 295, 296		High Head Siphon —	635
Sandy — from Travancore 315, 316		Inclined tunnel —	634
Sandy loams — 294, 295, 296		Ogee — 636, 639, 682	
Shrinkage factors of — 289		Preliminary Note on —	634
Wave action on — 292		Shaft — 634, 636, 672	
Soil Science : Section E 285		Siphon — 634, 635, 637, 681	
Soil Surface 319		The Year's Work on —	636
Soil Survey of the area commanded by the Mor Project 359		Spray blocks	647, 649
Surveys 357, 358, 359, 360, 361, 362, 363		Spray Irrigation	392
Discussion by the Board on — 363		Consideration by the Board of —	394
Discussion by the Research Committee on — 361			
The Year's Work on — 359			

	Pages		Pages
Discussion by the Research Committee on —	394	Strainers	326, 826, 827, 828
Essential parts of — system	392	Stream lines	305
Preliminary Note on —	392	Stress analysis by method of least work	601
Spreading floor of falls	715	Study of individual projects	367
Spurs	531	Discussion by the Board on —	369
Armoured —	498, 514	The Year's Work on —	367
Conventional types of —	490, 508	Sub-Committee: Permanent — to draft a directive on standardisation of methods and equipment for soil testing	287, 304
Hockey —	508, 515	Sub-Committee for drafting Instructions for the determination of Detritus Load in Rivers and Canals	23, 176, 192
Inverted hockey —	508	Sub-Committee of Research Officers for details of exhibition at the Thomason College Centenary	23
Repelling —	498, 514, 515, 516	Sub-committee on Fundamentals of Soil Research	23, 305, 310, 333
Straight —	508	Sub-Committee on Water Balance Investigations of the Damodar Catchment	23
T-headed —	490, 508, 509, 515, 516	Sub-Committee to draft standards for (a) calibration methods for current meters and (b) discharge observations and equipment	175
Stabilisation —	306, 307	Sub-grade soil	291, 306
Stabilisation with calcium chloride	296	Sub-grade soil constants	286
Stabilised soil	289, 292, 303	Sub-soil flow	144
Bituminous —	303	Sub-soil flow and regeneration	440, 441, 442
Stage—discharge relations	136, 140, 231, 458, 582	Sub-soil reservoir	213
Consideration by the Board of —	152	Sub-soil rock	213
Discussion by the Research Committee on —	149	Sub-soil valley	343
Staggered blocks	519, 530, 677, 683, 706, 713, 714, 715, 717, 725.	Sub-soil water	343
Standardisation of steel sheet piling	400	Sub-soil water level	340, 342, 346
Discussion by the Board on —	423	Effect of tubewell pumping on —	209
Discussion by the Research Committee on —	412	Sub-terranean waters	209, 824
Preliminary Note on —	400	Consideration by the Board of —	213
The Year's Work on —	403	Preliminary Note on —	209
Standard Notation	398	The Year's Work on —	210
Preliminary Note on —	398	Sudamdih	154, 156
Standing wave flume	173	Sugar-cane : Effect of — irrigation on sub-soil water level	340, 341, 342, 343, 344, 345, 346
Water surface profiles observed in a five feet wide — with parallel sides for various stages of flow	153	Experiments to determine water requirements for — crops	348, 349, 352, 353
Stanley Meter	165, 166, 167, 168, 169, 170, 171, 174	Sugar-cane experiments under sewage irrigation	219
Staunohing of canals	193, 818, 819	Sugar-cane experiments under varying doses of N in the form of sewage irrigation with special reference to the quality of gur	219
Staunohing of canals and embankments	818	Suitability of drainage waters for irrigation: Baramati quadrats	214, 215
Consideration by the Board of —	822	Suitability of drainage waters for irrigation: Shingwa area	214
Discussion by the Research Committee on —	82	Sukkar Barrage	416, 517, 518, 552
Preliminary Note on —	818		
The Year's Work on —	819		
Stevenson's Screen	196, 314		
Stilling basins for overflow dams	636, 643		
Stilling of whirlpools in the Kondepudi Gorge of the Godavari River	231		
Storage	212		

(XX)

	Pages	Terms—	Pages
Saleimanki	517	Definition of — in common use	395
Sumer Chand, Shri (i),	23	Glossary of technical and vernacular — in connection with irrigation in India together with standard notation	395
Summarised Annual Reports of Research Stations	25	Terraces	206
Sundaram, Shri G. (i)		Terracing	148, 208
Sundar Rao, Shri H.	16, 23	Thomas, Mr. A. R. 123, 124, 253,	395, 509
Surface application	335	Maximum flood discharge from small catchments by —	111
Surface drainage	313	Thomason College Centenary	23
Surface reservoir	212	Thana Reservoir	586, 588
Surface velocity: Correlation of— observation with current meter observa- tions at 0.6 depth	153, 174	Theory of flow and design of channels	229
Relation between — and mean velocity	161, 175	Thur	337, 338
Special arrangements to observe—	154	Tidal action including that in estuaries Discussion by the Board on —	458 582
Burkhi mortar 610, 611, 619, 620, 622, 819, 820, 821		Discussion by the Research Committee on —	577
Surua	462, 470, 471	Preliminary Note on —	462
Suspended and bed load characteristics of the river Brahmaputra at the Railway Bridge Site near Jogigopa	177	The Year's Work on —	565
Suspended load 396, 589, 590, 591		Tidal model of the river Timor (Australia) Tilaiya	433 146
Average weighted mean diameter of—	189	Tilling of crops	349, 350
Characteristics of Brahmaputra—	177	Tilpara Ghat Barrage	132, 149
Experiments on the measurement of—	177, 183	Correlation between gauge heights at the Sainthia Railway Bridge Site and the — site on the river Mor	140
Observations of — at Dhukwan	584	Timber pile groynes for training of rivers	496
Ratio of bed load to —	188	Tintometer	217
Sampling of —	176	To find a device for controlling side erosion at foot bridge at mile 8-4-232, Marihan Branch—Mirzapur Canal	709, 712
Sutlej: Floods in —	132	To find a suitable device for eliminating bed scour below Ghaggar Canal Head	718, 725
Sutlej at Hussainiwala	754, 792	To remedy silting above the left forebay of Nirgajni Power Station	262, 271
Sutlej at Samasata	451, 452, 490, 498, 508	Torrent Training	208
Sweating from the Krishnarajasagar Dam	610, 620	Torsa river	490, 491, 509
T			
Tajewala	197, 203	Total volume of runoff including the standardisation of hydrographs and their analysis	50
Talbots' formula	114	Consideration by the Board of —	70
Tank Rehabilitation Scheme	349	Discussion by the Research Committee on —	68
Tapti River	716	Trace elements	219
Task before the architect: Inaugural speech—by Shri N. V. Gadgil at the Annual Meeting of the Board	1	Trade-waste: Note on — experiments in the Bombay Province	227
Taylor, Mr. S. G. (i)		Trade waste disposal	327
Technical proceedings	47	Trade waste treatment	229
Telephone and telegraphlines	278	Training	459
Temperature—		Training of rivers with permeable tree groynes	490, 500
Effect of — on rating of meters	162, 174	Training of the Jumna River at Delhi Gate Pump House by means of a pitched embankment	490, 494
Experiments conducted to find the effect of — on the results of ratings of current meter—by K. K. Mahadevan	154, 161		

	Pages		Pages
Training of the river Indus above the Sukkar Barrage to prevent slips of the right bank below the nose of the approach channel	517, 518	Tunnels	626, 633
Training of the Sutlej River at Samasata about six miles downstream of the Empress Bridge at Adamwahan	490, 498	Turbidity	216, 217
Training of the Watrak River at Kaira	490, 494	Turbulence	572, 579, 589, 590, 657
Training the River Luni at Railway Bridge No. 3. of Samdari Raniwara Branch of the Jodhpur Railway	490, 493	Effect of — on rating of meters	169, 171 172, 174
Transmission	278	Introduction of —	171
Consideration by the Board of —	279	Reduction of —	565
Preliminary Note on —	278	Some experiments done at Krishnarajasagar to study the effect of — on the velocity records obtained with cups and propeller types of meters—by K. K. Mahadevan	154
The Year's Work on —	278	Two crore scheme for the restoration of minor tanks in the Mysore State—by M. Narasimhaiya	367
Transmission Coefficient 210, 312, 326, 327, 328		Type of material and its effect on hydraulic gradient	297, 300
Temperature correction of —	210	Types of tubewells	824, 825, 839
Transmission coefficient of sand samples taken between ground level and 12 feet depth from three test borings and its verification with transmission coefficient obtained from yield test results in the Damodar River at Sindi	210		
Transmission losses	820	U	
Transpiration	306	Underground basin : Recharging the — for future pumping	209
Evaporation and — losses	213, 440	Undrained catchment	198, 199
Trays—See Rainfall Runoff Experiments on—		Unified plan of development of the Damodar Valley	147
Tree Groyne 452, 453, 455, 456, 457, 490, 495, 496, 497, 499, 500, 501, 502, 503, 504, 505, 506, 515		United Provinces Irrigation Research Station : Summarised Annual Report	41
Model limitations as regards bank erosion and accretion upstream and downstream of a —	450, 451	Unit Hydrograph of the Brahmani Catchment above Nalhati	59
Trench : Bed—method of taking bed load samples	185	Unit Hydrograph of the Damodar Catchment above Chas Road Bridge Site	65
Trenches	307, 400	Uplift pressures	702
Trimmu Barrage	707	Uppal, Dr. H. L.	17, 490, 514, 515, 516, 517, 535 580, 715, 716, 717, 796, 797
Tubewell borings with 18 inches unreinforced cement concrete pipes—by Rameshwar Saran	829	Upward movement of salts	331
Tubewells pumping : Effect of—on subsoil water-level	209	Usar soil	819, 821
Tubewells	210, 326, 392, 824,	U.S. A. Type of evaporimeter	312
Discussion by the Board on —	839	Usehat Band	289, 292, 303
Discussion by the Research Committee on —	826, 838		
Handbook on —	209	V	
Preliminary Note on —	824	Vashvant Sagar Dam	635
The Year's Work on —	825	Vagai Project	643
Tube-wells required to lower water-table at Khatima Power House	263	Vaidhianathan, Dr. V.I.	17, 23, 303, 304, 305, 306, 308, 310, 332, 456, 457, 608, 609, 610, 827, 828, 829
Tubewells, their causes of failure and remedies	825	Vanivilas Sagar Dam	611, 613, 614, 615, 316, 618, 622, 623, 624
Tungabhadra Canal	518, 533	Vapour pressure : Mean saturation —	317, 318
Tungabhadra Dam	601, 622, 637	Vapour tension	325
Tungabhadra Spillway Dam	636, 639, 640, 643	Variation of coefficient of discharge with reservoir levels for full opening of sluices in Krishnarajasagar Dam	601, 603

	Pages		Pages
Varying discharges : Effect on Silt distribution of —	718, 719, 722, 723	Effect of — on the vertical movement of salts	337
Vasishth, Shree R.P.	16	Reclamation —	335
Vaughanlee, Col. G. H.	(6)	Water level : Effect of tubewell pumping on sub-soil —	209
Vegetation as a factor affecting infiltration	211	Waterlogged lands	334, 335, 340
Velocity :		Waterlogging	334, 337, 338, 342, 343, 349, 350
Effect of the non-uniformity of — on rating of meters	150	Waterlogging and land reclamation	334
Relation between surface — and mean —	161	Water Power :	
Some experiments done at Krishnarajasagar to study the effect of turbulence on the — records obtained with cups and propeller types of meters—by K. K. Mahadevan	144	Board's 1946 Resolution on — — development	259
Surface method of observation of —	159	Estimation of —	261
Two points method of observation of —	159	Water Power Works	257, 259
Velocity observations : Comparison of — by the float method and the current meter method	153	Consideration by the Board of —	260
Velocity observations at 0.6 depth	159	Financial aspects of —	262
Venkatacharya, Shri A.R. (6), 15, 21, 23, 24, 132, 134, 150, 333, 356, 357, 379, 385, 434, 490, 582, 608, 610, 624, 625, 696, 699, 717, 828.		Preliminary Note on —	259
Venkatramayya, Shri V	16	The Year's Work on —	260
Verification of the observed lowering of water table at Mohammadpur Power House by electrical analogy method	262, 266	Water requirements for sugar-cane crop	348, 349
Verman, Dr. Lal C	23	Water requirements for wheat crops	348, 349
Vertical movement of salts under reclamation waterings	335, 336	Water requirements of crops	347, 355, 356, 357
Volute priming siphon	657, 663, 664, 684	Discussion by the Board on —	357
Volutes	636, 666, 668, 669, 673	Discussion by the Research Committee on —	352
Number of —	636, 667, 668, 684	Preliminary Note on —	347
Shape of —	636, 662, 667, 669, 684	The Year's Work on —	348
Volute Siphon Spillways	See Spillway.	Water requirements of crops and vegetation	440, 441, 443
		Water resources	131, 144, 145, 152, 206, 444
W		Ground —	131
Warabandi	336	Watershed characteristics of catchments	589
Wash load	395, 397	Water supply	148
Water :		Water surface profile observed in a 5 feet wide standing wave flume with parallel sides for various stages of flow	153, 157
Amount of — recovered in <i>nallas</i>	198	Water table	306, 315, 317, 319, 320, 321, 322, 331, 332, 333, 346.
Annual — Statements	131	Definition of permanent —	213
Economical use of —	348, 349	Reclamation of alkaline soils by lowering of —	334
Recovery of —	199	Sub-soil —	340, 342
Underground —	131	Water table at Mohammadpur Power House	262, 263
Water account experiments	198, 200, 201, 202	Water utilisation projects	365
Water balance investigation of the Damodar catchment : Meeting of the Sub-Committee	23, 439	Water utilised in catchments	198
Watering intensity	336	Water vapour	325
Waterings	340, 346	Waterways, Irrigation and Navigation Act	378, 444
		Waterways of India	444
		Watrak River	490, 494, 512
		Wave action	292, 295

(xxiii)

	Pages		Pages
Wave wash	289, 292, 303	Y	
Protection against—	307	Yield (Tubewells)	209
Weather forecasting	144	Consideration by the Board of —	213
Weathering : Intensity of — affecting infiltration	211	Discussion by the Research Committee on —	210
Weighted mean diameter	233	Maximum —	211
Weirs : Dams —	599	Preliminary Note on —	209
Well records	152	The Year's Work on—	210
Wells : Yield in —	211	Yield of tubewells in relation to sub- soil	324
Well Waters	215	Yield of crops	348, 349, 350, 351, 354, 356
Well waters in effluent zone	214, 217	Younus, S.M.	(6), 15, 151
Wheat : Experiments to determine water requirements of — crops	349, 353		
Wild's evaporimeter	312	Z	
Wind velocity	313, 317	Zone of constant moisture	323

**LIST OF PUBLICATIONS ISSUED BY THE CENTRAL BOARD OF
IRRIGATION**

1. Digest of Technical Notes 1931 (Reprint)

A summary of views of Chief Engineers on the following subjects :—

- (i) Waterlogging and Reclamation.
- (ii) Effect of deforestation and afforestation on river floods.
- (iii) Design of works on sand foundations.
- (iv) Fluming of canal works.
- (v) Regulation at canal head works.
- (vi) Design of canal falls.

2. Digest of Technical Notes, 1932.

A summary of views of Chief Engineers on the following subjects :—

- (i) Waterlogging.
- (ii) Design of works on sand foundations.
- (iii) Design of canal falls.
- (iv) Staunching of canals.

3. Catalogue of books (upto December, 1933) in the Library of the Central Board of Irrigation, Simla, and its 1st and 2nd Annual Supplements (1934-36)

4. Hydraulic Diagrams : Energy of flow, Pressure—plus—Momentum Diagrams for the Simple graphic Solution of problems involving a change of section in a stream of water (1934) Revised edition (1948).

5. Glossary of Technical and Vernacular Terms in connection with Irrigation in India, together with Standard Notations. (1934). (1st Revised Edition 1941).

6. Fluming : A series of examples illustrating the use of Hydraulic Diagrams, C. B. I. Publication No. 4 (1934).

7. The Standing Wave or Hydraulic Jump. (1934).

8. Observation and Record of Pressures below Works on Permeable Foundations, by Shri A. N. Khosla. (1935).

9. Stabilization of Soils by the use of Bitumen Emulsion—by Brigadier C. H. Haswell. (1935).

10. Irrigation Canal Falls. (1935).

11. Annual Report (Technical) of the work of the Central Board of Irrigation, India, (1934-35).

12. Design of Weirs on Permeable Foundations—by Shri A. N. Khosla, Dr. Nalini Kanta Bose and Dr. E. McKenzie Taylor. (1936).

13. Annual Report (Administrative) of the work of the Central Board of Irrigation, India, (1935-36) (For the use of Board Members only).

14. Annual Report (Technical) of the work of the Central Board of Irrigation, India, (1935-36).

15. Annual Report (Administrative) of the Central Board of Irrigation India, (1936-37) (For the use of Board Members only).

16. Annual Report (Technical) of the work of the Central Board of Irrigation, India, (1936-37).

17. Notes on Waterlogging and land Reclamation in the form of a Questionnaire. (1938).

LIST OF PUBLICATIONS

18. Annual Report (Administrative) of the work of the Central Board of Irrigation, India, (1937-38). (*For the use of Board Members only*).
19. Annual Report (Technical) of the work of the Central Board of Irrigation, India, (1937-38).
20. Regime Flow in Incoherent Alluvium—by Gerald Lacey. (1939).
21. Annual Report (Administrative) of the work of the Central Board of Irrigation, India, (1938-39). (*For the use of Board Members only*).
22. Annual Report (Technical) of the work of the Central Board of Irrigation, India, (1938-39).
23. Annual Report (Administrative) of the work of the Central Board of Irrigation, India, (1939-40). (*For the use of Board Members only*).
24. Annual Report (Technical) of the work of the Central Board of Irrigation, India, (1939-40).
25. Annual Report (Administrative) of the work of the Central Board of Irrigation, India, (1940-41). (*For the use of Board Members only*).
26. Water Weeds and their eradication from canals,—by C. C. Inglis, and V. K. Gokhale, with a note on Eradication of Bulrush by V. K. Gokhale and S. N. Punekar. (1941).
27. Annual Report (Technical) of the Central Board of Irrigation, India, (1941).
28. Annual Report (Administrative) of the work of the Central Board of Irrigation, India, (1941-42). (*For the use of Board Members only*).
29. Annual Report (Technical) of the Central Board of Irrigation, India, (1942).
30. Annual Report (Administrative) of the work of the Central Board of Irrigation, India, (1942-43). (*For the use of Board Members only*).
31. Annual Report (Technical) of the Central Board of Irrigation, India, (1943).
32. Annual Report (Administrative) of the work of the Central Board of Irrigation, India, (1943-44). (*For the use of Board Members only*).
33. Annual Report (Technical) of the Central Board of Irrigation, India, (1944).
34. Annual Report (Administrative) of the work of the Central Board of Irrigation, India, (1944-45). (*For the use of Board Members only*).
35. Annual Report (Technical) of the Central Board of Irrigation, India, (1945).
36. Role of Glaciers and Snow on the Hydrology of Punjab Rivers—By Shri Kanwar Sain.

(XXVI)

LIST OF PUBLICATIONS

37. Annual Report (Administrative) of the work of the Central Board of Irrigation, India, (1945-46). (*For the use of Board Members only*).

38. Annual Report (Technical) of the work of the Central Board of Irrigation, India, (1946).

39. Annual Report (Administrative) of the Central Board of Irrigation India, (1946-47). (*For the use of Board Members only*).