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Director : K. K. FRAMJI, M.B.E., I.S.E.

RESEARCH PUBLICATION No. 11

ANNUAL REPORT (TECHNICAL) OF WORK DONE

DURING THE YEAR 1946

ΒY

K. K. FRAMJI, M.B.E., B.A. (Bom.), B.Sc. (Lond.), A.K.C. (Lond.), I.S.E.,

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AND

STAFF AT KHADAKVASLA



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Symbol No.	Research Publi- cation No.	Subject.	P	rice	Year of Publica- tion.	
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IHR. 2	8	Note on "The theory, design and construction of Gibb Modules which, without moving parts, give a constant discharge within working limits irrespective of variations in upstream and downstream water levels" by 0. 0. Inglis and Rao Sahib D. V. Joglekar.	3	8	0	1941
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IHR. 1·41	. 5	Annual Roport (Technical) of work done during the year 1940-41 and index for 1937-41 by C. O. Inglia, A. R. Thomas, Rao Sabib D. V. Joglekar and staff at Khadak- wasla.	7	8	0	1942
IHR. 1·42	. 6	Annual Report (Technical) of work done during the year 1941-42 by C. C. Inglis, J. S. Roid, Rao Bahadur D. V. Joglekar and staff at Khadakwasla.	7	8	0	1944
IHR. 1·49	. 7	Annual Report (Technical) of work done during the year 1942-43 by C. C. Inglis, J. S. Reid, Rao Babudur D. V. Joglekar and staff at Khadukwasia.	7	8	0	1945
IWE. 1·43 🔍	. 8	Annual Report (Technical) of work done during the year 1943 by C. C. Inglis, Rao Bahadur D. V. Joglekar and staff at Khadakwasla.	7	8	0	1945
IWE. 1·44	. 9	Annual Report (Technical) of work done during the year 1944 by C. O. Inglis, Rao Bahadur D. V. Joglokur and staff at Khadakwasia.	7	8	0	1945
IWE. 1.45	. 10	Annual Report (Technical) of work done during the year 1945 by Rao Bahadur D. V. Joglekar and staff at Khadakwasia.		•••		Under print.

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ii	9 from top	economic	economics.
1	21 from top	Tahmankar	Tamhankar.
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97	Table 3 XVI-ii	Bore of allotted screen	Bore of slotted screen.
97	Table 3 XVI-ii	Length of allotted screen.	Length of slotted screen.
98	2 from top	open	upon.
	Fig. 4/XVI-ii opposite page 98. Heading of bot- tom figure.	Infiltration gallery in right bank.	infiltration gallery near the right bank.
99	Under para. 2 "Model Scale Ra- tios".		Add "The model extended equivalent to 84 miles up- stream and 64 miles down- stream of the proposed barrage".
107	(4) Slope scale 3rd line from bottom.	=6.33 with V.E.=	=6.25 with V.E.=.
132	5 from top	Analyses	Analysis.
134	37 from top	6½ cusecs	6 ¹ / ₂ lakh cusecs.
136	12 from top	contant	constant.
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1. STAFF AND THEIR ACTIVITIES

Director: Mr. K. K. Framji, M.B.E., B.A. (Bom.), B.Se. (Lond.), A.K.C. (Lond.), I.S.E.

Deputy Director: Rao Bahadur D. V. Joglekar, B.E. (on leave from 1-2-1946 to 31-5-1946 and 16-10-1946 to 15-12-1946; on deputation from 1-6-1946 to 15-10-1946).

Assistant Research Officers

1. Mr. G. M. Panchang, M.Sc. (Lond.), (for Statistics), (till 21-5-1946).

2. Mr. S. T. Ghotankar, M.Sc.

3. Mr. S. C. Desai, B.E., (till 31-5-1946).

4. Mr. C. V. Gole, B.E.

Research Assistants

1. Mr. S. N. Mulekar, B.Sc. (Elec. and Mech.).

2. Mr. S. V. Chitale, B.E.

3. Mr. D. P. Kulkarni, B.E.

4. Mr. P. K. Kulkarni, B.So. (Elec. and Mech.).

5. Mr. G. T. Wadekar, B.E.

6. Mr. C. G. Tahmankar, B.Sc. (Elec. and Mech.).

7. Mr. P. M. Damle, B Sc.

Work Assistant

Mr. N. K. R. Iyengar.

Computors

Mr. S. B. Barve.

Mr. R. P. C. Varma, B.Sc.

The Khadakwasla Welfare Centre, which is run partly by voluntary work, and financed by contribution from the Red Cross, the Rotary Club, Collector's Fund and Members of the Staff of the Station, has done excellent work throughout the year.

Figure 1 shows :---

(i) the 13 places visited by the Director during the year,

(ii) the 12 places where advice was given without model experiments, and

(iii) the 8 places where advice was given after model experiments.

The 12 cases in which advice was given without model experiments were as under:-

1. Savitri River.—(1) Prevention of the formation of a sand bar at the mouth of the River where it falls into the Sea near Ratnagiri in Bombay Province and siltation and erosion of the Konkan creeks; and

(2) effect of the jetties constructed and the determination of the type of jetties most suitable for the existing conditions.

2. Suitability or otherwise of the site for the proposed bridge over the Brahmaputra at Jogigopa, Assam, and the determination of the probable maximum scour at the proposed bridge site.

3. Prevention of scour at the bridges over the Beas River at Beas Railway Station and over the Chenab River (Alexandra Bridge) near Wazirabad on the N. W. Railway.

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Fig. 1.

4. Best top section for a broad-crested overfall weir in the Bhavnagar State.

5. Comments on Notes on Experiments with low earth dams for flushing streams in Assam by Maj. W. E. C. Peltman, R.E.

6. Protection of the Railway Embankments of the proposed Railway Bridge across the Ramganga River on the C. K. Railway branch line of E. I. Railway from the effect of erosion in the event of meanders acquiring the maximum swing towards the embankments.

7. Protection works for the Bridges over the Sukri, Jawai, Sagi and Khari Rivers on Jodhpur Railway.

8. Suitable profile of a H. C. weir in Hyderabad State (Dn.).

9. Prevention of damage from floods in the Subernarekha River near Jalleswar in Orissa.

10. Suitable design for weirs on sand and shingle foundations in Baluchistan.

11. Prevention of scour at the retaining wall of Shri Harihareshwar Mahadeo Temple on the bank of the Saraswati River at Patan in Baroda State.

12. Flood control in the Khari River, Ajmer.



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On 7th and 8th March 1946, the Director visited Ajmer for the purpose of investigations in regard to the prevention of damage by floods in Ajmer Merwara, where owing to concentrated and excessive rainfall in 1943, thirtyeight small storage tanks breached simultaneously with an abnormal rise in the river Khari, which resulted in an enormous loss to life and property.

On 29th April, he visited Cuttack at the request of H. E. the Governor of Orissa and inspected the severe damage to the Jobra weir across the Mahanadi and suggested measures for the dissipation of energy below the weir and the avoidance of the recurrence of similar damage in future.

As desired by the Secretary, P. W. D., Bombay, the Director visited Surat along with the Honourable Dr. Gilder, the Minister for Public Works, Bombay, and other officers of the Bombay P. W. D. on 10-8-1946 in connection with the flood problem of the Tapti River. For the flood-control problem, the Director recommended hydrographic, industrial, power loads, and other surveys to examine the feasibility of multipurpose reservoirs on T. V. A. Lines.

At the request of the Chief Engineer, N. W. Railway, Administration, the Director and the Chief Engineer inspected, from 22nd to 24th September the site of erosion by the Sutlej River near the Samasata Station Yard for the purpose of suggesting measures to prevent further inroad by the river; and the Wazirabad Bridge over the Chenab. [See para. II (vi) (4) regarding the former case.]

On 26th September 1946, he paid a second visit to Ajmer regarding the flood problem of the Khari River, when he inspected the tanks at Todghar, Devghar and Bhim.

At the instance of the Consulting Engineer to the Government of India for Waterways, Irrigation and Navigation, New Delhi, the Kshipra River at Ujjain in the Gwalior State was inspected by the Director on 25-10-1946, on his way back from Karachi where he had flown to attend the session of the Indian Roads Congress. This river is receding away from the infiltration gallery of the Ujjain Water Works and advice is sought of the Director for ways and means of restoring the river to its original course.

The Director was again requested by the Secretary, P. W. D., Bombay, to give his advice, after inspecting the Watrak River near Kaira in Gujerat, where the river has been eroding some land in the village of Hariala; advice was required as to the exact protective measures necessary to prevent this erosion, see para. II (vi) (3).

Rao Bahadur D. V. Joglekar, Deputy Director, was appointed Officer on Special Duty, from 1st June to 15th October, to work at Delhi in the Consulting Engineer's Branch for preparing detailed proposals for buildings and equipment for the proposed re-organised Station at Delhi.

One of the Officers of the Station (Mr. S. T. Ghotankar, Asstt. Res. Officer) was deputed to Delhi to construct two models of the Mahanadi Valley, "as it is" and "as it will be" respectively, on behalf of the Central Waterways, Irrigation and Navigation Commission. These Models were demonstrated at the exibition—opened by the Hon'ble Pandit Jawaharlal Nehru—held under the auspices of the Central Board of Irrigation from 28th November to the 2nd December 1946, and were immensely popular, proving to be the major attraction of the Exhibition. Skilled artisans of this Station worked under Mr. Ghotankar for a month on making the two 40' × 12' attractive relief models of the Mahanadi Valley.

In the latter model were depicted the Hirakud, Tikarpara and Naraj dams controlling the floods, their canal systems irrighting the fields, navigation, power houses, distribution mains, terracing of hill slopes, well-planned cities, etc., showing the tremendous improvement that the multipurpose project would make to the development of the valley : while the former model showed a lack of flood control, the country inundated in parts and barren in others, properties damaged, etc., according to the existing conditions.

Besides these, there was an unprecedented collection of technical and instructive models exhibited by research organisations and Government Departments all over India, covering a wide range of subjects such as navigation, river training, Irrigation works, meteorological apparatus, Survey, Soil Science, Physics, Engineering, Chemistry, etc.

Hon'ble Pandit Jawaharlal Nehru was surprised to learn that this was the first exhibition of its kind in India and stressed the need of having a series of such interesting exhibitions all over India.

His Excellency the Viceroy of India (Lord Wavell) and His Excellency the Governor of Bombay (Sir John Colville) visited the Station on 13th January 1946; Sir Andrew Clow, the Acting Governor of Bombay saw the Station on 6th October 1946; all the three distinguished visitors appeared to be greatly interested in the work being done at the Station. Other prominent visitors to the Station during the year were :--

Maj. Genl. Philip B. Fleming, Head of the U.S. Federal Public Works Administration, U.S. A.

Mr. Thomas Harris Macdonald, Commissioner of Public Road, U.S. Federal Public Roads Administration, U.S.A.

Mr. W. H. R. Nimmo, Official of the Department of Co-ordination of General Public Works, Australia.

Mr. D. G. Garland, Bridge Engineer of the Queensland, Main Roads Commission, Australia.

Mr. Mohsin Ali, Member for Irrigation, C. W. I. N. C., New Delhi.

Rao Bahadur A. R. Venkata Acharya, Chief Engineer for Irrigation, Madras.

Mr. B. S. Narasinga Rao, Superintending Engineer, Irrigation Circle (K. R. S. Works), Mysore.

Mr. G. B. Mondal, Chief Engineer, Bengal.

Dr. N. K. Bose, Director, Bengal River Research Institute.

Mr. B. P. Saxena, Superintending Engineer, U. P.

Mr. N. D. Gulhati, Secretary, Central Board of Irrigation, Simla.

Dr. S. P. Raju, Director, Engineering Research Department, Nizam's Dominions, Hyderabad (Dn.).

Mr. J. K. Hunter, Chief Engineer, Associated Consulting Engineers (India).

Mr. J. H. Talati, Chief Engineer, Baroda State Railway.

Students and Professors from the following colleges and institutions also visited the Station during the year :—

(1) Kala-Bhavan Technical Institute, Baroda.

(2) Engineering College, Osmania University, Hyderabad (Dn.).

(3) College of Engineering, Poona.

(4) College of Engineering, Bangalore.

(5) College of Engineering, Guindy, Madras.

(6) Victoria Jubilee Technical Institute, Matunga.

During the year, 20 investigations were in progress, of which 13 were specific in character and 7 basic.

Five notes on "Basic Problems" were written during the year; and notes were also written on a number of specific problems.

PART II-SPECIFIC RESEARCH

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I. FIELD RESULTS OF MEASURES ADOPTED AFTER MODEL EXPERIMENTS.

I-1. TRAINING OF THE MALAPRABHA RIVER TO PROTECT THE LEFT Abutment of the M. and S. M. Railway Bridge No. 8 at Hole Alur on the Hotgi-Gadag Line, Bombay

Having developed a cut-off one mile below the Railway bridge, the Malaprabha river was trying to dissipate the excess energy by eroding its banks. The left bank erosion near the bridge was severe; the bridge was threatened to be outflanked; and experiments were carried out at this Station to devise means to protect the left abutment and relieve it from attack.

The recommendation of cutting the right bank about 1/2 mile above the bridge was adopted in the river and spoils were dumped in front of the left abutment.

In continuation of the results reported in the A. R. (T.) 1945, Fig. 1/I ^{Fig. 1/I}shows the deep channel surveyed after the floods of 1945. The deep channel has shifted completely to the second span from the left end of the bridge. The apex of the meander appears to be about 150 ft. downstream of the Railway bridge, as against about 200 ft. above the bridge before 1945 floods. The results thus appear to be satisfactory.

I-2. PREVENTION OF SCOUR BELOW THE CENTRAL SLUICES OF THE MAHANADI ANICUT AT CUTTACK (ORISSA)

In May 1945, the Director was asked to inspect and advise regarding the methods of protecting the Central Sluices of the Mahanadi Anicut. Those sluices had been giving trouble for a long time and the measures annually adopted by the local officers did not remove the root cause. In 1944, staggered rows of control blocks had been constructed at the 3rd dwarf wall below the sluices but they were washed away unevenly, worsening the already bad condition of flow. Photos 3088 and 3098 show the condition when the Director Photos 3088 and 8098. inspected the site. The local proposals-of providing stone and concrete in wire mattresses sloping down to the bed - were considered unsuitable as they attempted merely to treat the symptoms. The Director, therefore, suggested a glacis with a series of baffles on the pavement below-the only practicable measures based on a diagnosis of the root trouble which could be completed in the very short time available before the commencement of the ensuing flood season. Photos 4003 and 4004 show the satisfactory conditions obtained after working of the Photos 4408 and 4004. suggested measures for one flood season.

Photos 4053A and 4054 show the resulting effective dissipation of energy Photos 4053A below the sluices in the river. The scour hole has silted substantially and 4064. (by over 13 ft.), the under cutting has ceased, and the structure is now safe.

I-3. MODEL LIMITATIONS AS REGARDS BANK EROSION AND ACCRETION UPSTREAM AND DOWNSTREAM OF A TREE GROYNE

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 [vide para. II, (vi) 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 — thereby reducing the velocities and bank erosion became imperceptible.

Actually, in the river it is expected that banks would continue to crode 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.

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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.

2. 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.

3. 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.

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.

II. RIVER BEHAVIOUR AND CONTROL : (i) MEANDERING

11-i. (1) PROPOSED RAILWAY BRIDGE ACROSS THE RAMGANGA RIVER ON THE PROPOSED C. K. RAILWAY (A BRANCH OF E. I. RAILWAY).

1. Introduction

The Consulting Engineer forwarded a Report of Mr. M. S. Ahmed, Executive Engineer in-charge of C. K. Railway Survey, to the Director for his considered opinion, particularly on the following points of the Report:—

(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.





3088. Photo taken from the left flank divide wall from a point opposite the third dwarf wall showing the damaged control blocks and dry stone pitching.



3098. Photo showing the stepped nose of the left divide wall. Looking upstream.

Central Sluices of the Jobra on the Mahanadi (Cuttack).



4003. Photo showing the silted scour-hole below the Central Sluices of the Mahanadi Anicut (Cuttack) after 1946 floods. Note the shallow depth of water, where there used to be a deep scour.



4004. Same as Photo 4003, showing a distant view.



4053A. Photo taken on 18-11-46 showing the clear standing waves with glacis and baffles when crest shutters of the anicut were up. Upstream gauge = 68:3 and downstream gauge = 59:5. Dwarf shutters of the central sluices were up.



4054. Photo taken on 26-9-46 showing the clear standing waves with glacis and baffles giving satisfactory dissipation of energy. U/s gauge=68.0; d/s gauge=61.0. Shutters of the central sluices were down. All the crest shutters of the anicut except 70 Nos. adjacent to the central sluices on its south were up.

2. 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, in the Director's opinion, 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 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 4 to 6 miles, but just upstream of the Sukkur Barrage the swing is much greater, being nearly 12 miles⁽¹⁾.

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 1/II-i). Only seven out of 150 years river courses had been $\tau_{able 1/11-1}$ supplied; so it had to be assumed that the seven river courses shown on the $2^{\prime\prime}$ to 1 mile plan were faithfully representative of all the years for which data were available. Even a study of these limited courses, however, did not lead to the same conclusion as Mr. Ahmed's in respect of the maximum arc/chord ratio; according to him the worst ratio was $2\cdot 1$ (year 1895) while according to to the Station analysis it was $2\cdot 67$ (year 1806). The correct estimation of this ratio is of some considerable importance, both for the positioning of the approach embankments and for fixing the length of the guide banks.

Fig. 1/II-i shows meanders on this river in four different stages of develop- Fig. 1/11-1 It will be observed that as the arc/chord ratio increases from 1.26 to ment. 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. $\operatorname{arc/chord} = 1.26$, $M_{\nu} = 1.25$), it can develop to as much as $M_b = 1.80$ miles, and the minimum distance between the edge of 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 unforesceable disturbing factors. Accordingly, the minimum distance for a loop such as of

the year 1895 would be $1.80 - 1.25 + \frac{25}{100} \times 1.8 = 1.0$ mile.

While the minimum distance from the existing 1945-46 loop $\left(\frac{B}{C} = \frac{\text{length along bend}}{\text{chord}} = 2.11.$ $M_b = 1.75$, see fig. 1/II-i) would be $(1.80 - 1.75 + \frac{25}{100}1.8) = 0.5$ mile.

Thus, from this consideration, the line of the proposed embankment should be AB (in fig. 2/II-i), so far as it is possible to fix it with the incomplete meander Fig. 2/II-i. courses available.

(2) Though ordinarily, where a river is controlled by a fixed exit and 2. 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 not

Fig. 1/11-i.

⁽¹⁾ Annual Report (Tech.) for 1943 of the Indian Waterways Experiment Station, Poona, page 13.

TABLE 1/II-i

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		M	Mı	м _b .	м _b	Batio C	Radius of curvat	ure "r" in miles.	r Vomar	
Course of year.	No. ot dofined meanders.	Length of the meander in miles.	Average length for the year in miles.	Meander balt in miles.	Average meander belt for the year in miles.		As measured from plan.	As calculated by formula I.	Where values of "r" are as given in 8(b) × 5280.	Where values of "r" ars as given in 8(b, x 5280.
· 1	2	8	4	5	6	7	8 (n)	8 (b)	9 (a)	9 (b)
1790	4	3.70		1.25		$\frac{9\cdot 25}{7\cdot 3}=1\cdot 25$	1.05	1.13	10-11	12-88
		2.26	2.77	0.77	1.53	$-\frac{5\cdot 50}{4\cdot 5}=1\cdot 22$	0.45	0.77	6-89	8-44
		1.80		1.75		$\frac{8\cdot 0}{3\cdot 6} = 2\cdot 20$	0.10	0.51	4 • 57*	5.58 * Cut-off appears to have occurr-
		8.40		2.85		$\frac{13\cdot75}{6\cdot8}=2\cdot02$	0.69	0.86	7.70	ed. 9.40
1806	8	6-0		1.3		$\frac{13\cdot 25}{12} = 1\cdot 11$	1,40	2.48	22.20	27.15
		1.50	8-33	1.8	1.61	$\frac{8}{3} = 2 \cdot 67$	0-20	0.47	4.20	5 - 15
		2.50		1.75		$\frac{8\cdot 5}{5} = 1\cdot 7$	0+60	0.69	5 64	6+90
1883	2	2.50	2.87	0.85	1.20	$\frac{7\cdot 5}{5} = 1\cdot 5$		0.84	7 - 52	9.20 Development imperfect.
		8-25		1.55		$\frac{11}{6\cdot 5} = 1\cdot 7$	0+60	0.84	7.52	9-20 Do.
1895	8	4.50		1.00		$\frac{11\cdot 4}{9} = 1\cdot 27$	1.80	1.94	17.37	2.13
		1.25	2.87	1.85	1.37	$\frac{5\cdot 5}{2\cdot 5} = 3\cdot 2$	0.20	0-86	· 3-22	8-94
		2.85		1.75		$\underbrace{10}_{5\cdot7}=1\cdot76$	0+95	0·71	6 - 35	7.77
1906	8	5.0		0.9		$\frac{11}{10} = 1 \cdot 1$	1.80	3.70	\$3.15	40.50

Aver	ago	2.99 miles=	15,800 ft.	<u> </u>	1.84 miles=1,075 ft.	1.67	0.77 mile= 4,075 ft.	1.05 miles = 5,520 ft.	<u> </u>	<u>. </u>
		2.25	2.78	1.75	1.10	$\frac{9\cdot 5}{4\cdot \delta} = 9\cdot 11$	0.60	0-53	5 ·18	6-35
1945-46	2	8.25		1.50 .		$\frac{10}{6\cdot 5} = 1\cdot 54$	0.50	0.85	7-61	9.30
1938—39	1	8-00 ⁻	8.00	1.2	1-20	$\frac{7\cdot75}{6}=1\cdot29$	0-92	0.84	7.52	9.20
4 1 		8.25		1.75		$\frac{11}{6 \cdot 5} = 1 \cdot 7$	0.95	0-82	7.84	8-96
د		1.75	8-34	1.43	1.36	$\frac{6}{3\cdot 5} = 1\cdot 72$	0.20	0-45	4.03	4.93 Out-off occurred.

Note (1)-

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(i) Worst B/C=3.67 (Year 18(6).
 (ii) Four soute meanders B/C=2.0 to 3.2.
 (iii) Out-off appears to have occurred betwonn 1895 and 1900 at B/C=2.2.

Note (2)-

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For 'r' formula below is used:

$$\frac{M}{4} = \frac{Mb - W}{9} \left(2R - \frac{Mb - W}{9} \right) = \frac{Mb - 1500}{2} \left(2R - \frac{Mb - 1500}{9} \right)$$

Note (3)---

Maximum discharge adopted for calculations in Col. 9(a) = 35×1,000 cs., and dominant dis-charge adopted for calculations in Col. 9(b) = 2/3 Qmax = 233.500 os.; as actual measured discharges for these years are not known.

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be lost sight of. On the right (or west) bank of the river exists, 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 approximately four miles near Shahabad (as given by Mr. Ahmed) – owing to incompleteness of river courses it is not possible to verify this figure—the approach should nowhere be less than 4 miles away from the river axis, in a reach of, say, 8 to 10 miles upstream of Shahabad. (The hard inerodible bank on the right side appears to terminate at Shahabad). This gives, approximately, the line C D on fig. 2/II-i. 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 compared to the proposed position shown in the plans received from Mr. Ahmed.

3. The calculations of the flood discharge

3.1 In the absence of flood discharge data Mr. Ahmed had calculated the flood discharge by almost all the well-known empirical formulae,—Dickens, Ryves, Inglis, etc., which gave widely varying results.

He had estimated the discharge by the Mississippi formula, recommended by Sir Francis Spring, assuming depth = 27.69 ft. with 10 ft. scour at H. F. L. which, in the Director's opinion was on the low side. With depth = 35 ft., velocity (for slope of 6.76 ins. per mile) = 6.3 ft./sec., area=20,2(.0 + $17.7/10 \times 10,000 = 37,500$ Sq. ft., discharge= $37,500 \times 6.3 = 2,36,000$ cusecs. Making the same per cent allowance for error in reported H. F. L., as made by Mr. Ahmed, the discharge would be $2.5/1.72 \times 2,36,000 = 3,43,000$ cusecs.

The flood discharge was also estimated on the analogy of the Bareilly Bridge, assuming a velocity of 6 ft./sec., while for rivers of this size the mean velocity in flood would more likely be at least 8 ft./sec.; with 8 ft./sec. 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, ⁽¹⁾ $Q = K^n \frac{7000A}{\sqrt{A+4}}$ where for Deccan catchments n = 1/3; but U.P. catchments are known to have less run-off for the same area of catchment so n = 1/2 may be assumed. Shape factor, $K = \frac{2}{\sqrt{II}} \frac{\sqrt{A}}{L} = \frac{2}{1\cdot77} \times \frac{7\cdot07}{168} = \cdot54$, $\therefore Q = K^{1/2} \frac{7000A}{\sqrt{A+4}} = 0.54^{1/2} \times 4.95,600 = 0.735 \times 4.95,800 = 3.64,000$ cs.

(1) Indian Waterways Expt. Stn., Annual Report (T) for 1942-43, page 34.

Fig. 2/II-i.

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3. (2) The flood discharges obtained by the various methods used by Mr. Ahmed but with necessary modifications suggested by the Station, are set out in table 2/II-i below:—

Formula used.	Computed maximum discharge (os.) (as by Mr. Ahmed).	Romarks.		
1	2	3		
(i) Dicken's	4,90,100			
(ii) Dredges	2,14,000			
(iii) Inglis'	4,95,800	Corrected for shape factor, assuming		
(iv) Cousins	6,19,655	n = 1/2, discharge will be 3,64,000 cusees.		
(v) Ryve's	1,96,500			
(vi) Mississippi	1,72,354 (2,50,000 cs. assumed for error in reporting H. F. L.)	Total depth has been elsewhere calculated by Mr. Ahmed as 30.9 ft. as against 27.7 ft. assumed here. With depth of 35 ft. (later verified), maximum dis- charge would be 3,43,000 cs.		
(vii) From Bareilly bridge section and assumed flood velo- city of 6 ft./sec.	2,15,172	With 8.85 ft./sec. mean velocity, as cal- culated below from Lacey's formulae for R=35 ft. against 6 ft./sec. assumed by Mr. Ahmed, discharge would be 3,17,000 cs.		
(viii) From Morada- bad Bridge section plus tributaries and assumed flood velocity of 6 ft./sec	2,59,176	With 8.85 ft. sec. velocity (vide Lacey's formula, see below) discharge would be 3,82 000 cs.		

TABLE 2/II-i

Table 2/11-1

3.3 Based on consideration of the figures given in column 2, Mr. Ahmed had proposed taking Qmax = 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.

3.4 There are two other ways of calculating the maximum discharge of the river, one by Lacey's formulae, and the second from the Station correlation between Meander length and maximum discharge.⁽¹⁾

3.5 Lacey's Method.—requires a correct estimation of the hydraulic mean depth below H. F. L. and of the energy slope. For these cross-sections taken during high flood would be of the greatest value. An inquiry was, therefore, made from Mr. Ahmed 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 ft.

⁽¹⁾ Annual Report (T) of I. W. E. Stn., Poona, for 1939-40 page 10 and A. R. (T) for 1912-43, page 54.

And, by Lacey's "general" flow formula, $V = \frac{1.3458 \text{ R}^{3/4} \text{ S}^{1/2}}{N_{\bullet}} \text{ where } N_{\bullet} = .0225 \text{ f}^{-1/4}.$ Now f = 0.77 (not 1.0 see para. 4 (A) (c) below—) $\therefore N_{\bullet} = .0225 (.77)^{1/4} = .0211.$ $\therefore V = \frac{1.3458 \times 40^{3/4}}{.0211} \times \frac{(.1065)^{1/4}}{(.1000)^{1/2}}$ $= \frac{1.3458 \times 15.8 \times .0103}{.0211} = 10.4 \text{ ft./sec.} \qquad \cdots \qquad (ii).$

The average of (i) and (ii). gives-

$$V = \frac{8.85 + 10.4}{2} = 9.6$$
 ft. per sec.

Mr. Ahmed gives area up to H. F. L. = 20,220 sq. ft. when R is about 17.7 ft. and 10,000 sq. ft. extra for 10ft. additional scour; hence for R=40 ft. the area would be—

$$20,220 + \frac{40 - 17.7}{10} \times 10,000 = 20,220 + 22,300 = 42,500$$
 sq. ft.

under maximum scour at maximum H. F. L.

This would give discharge = A. V. = $42,500 \times 9.6$.

=4,08,000 cusecs.-

For this discharge,
$$R = 0.473 \left(\frac{Q}{f}\right)^{1/3} = 473 \left(\frac{4,00,000}{0.77}\right)^{1/3}$$

Hence R = 40 ft assumed is apparently too much.

Assuming R = 35 ft. from results in (i) and (ii) above, from "normal" flow equation, $V = \frac{8 \cdot 85 \times 35^{2/3}}{40^{3/4}} = 8.1$ ft./sec.

and from "general" flow equation $V = \frac{10.4 \times 35^{3/4}}{40^{3/4}} = 9.6 \text{ ft./sec.}$

Average velocity = $\frac{8 \cdot 1 + 9 \cdot 6}{2} = 8 \cdot 85$ ft./sec.

Area, corresponding to 35 ft. depth = $20,200 + \frac{35 - 17 \cdot 7}{10} \times 10,000$

=37,500 sq. ft.

Hence, discharge = $37,500 \times 8.85 = 3,32,600$ cusecs.

For this discharge,
$$R = 0.473 \left(\frac{3,32,000}{0.77}\right)^{1/3} = 35.7$$
 ft.

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 cusecs.

3.6 Flood discharge determined from meander length

From table I/II-(i) the average meander length of the seven years for which river courses were available=2.99 miles, or 15,000 ft.; and the maximum meander length of any year is 3.34 miles, or 17,600 ft. 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 Poona Station formulae⁽¹⁾ correlating maximum meander length with Q (maximum) is:- $M_L = 29.6 \sqrt{Q_{max}}$... (1)

$$M_L = 29.6 \sqrt{Q_{max}}$$

... $Q(max) = \frac{17,600^3}{29.6} = 3,53,000$ cusecs.

Thus, from meander length considerations, the flood discharge works out to be of the same order as from Lacey's formulae.

Inglis' formula (with correction for shape factor).	$\dots = 3,64,000 \text{ cs.}$
Mississippi formula (for $R = 35$ ft.)	= 3,43,000 ,,
Vide Bareilly bridge (V=8.85 ft./sec.)	= 3,17,000 "
Vide Moradabad bridge and tribu	itaries
(8·85 ft./sec.)	= 3,82,000 ,,
Lacey's formulae	= 3,50,000 ,,
Meander length formula	= 3 ,53,000 ,,

Average = 3,51,500 cs.

Say, 3,50,000 cusecs.

Hence the Station estimate of the flood discharge would be 3,50,000 cs.

4.0 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 scour 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 $1.5 \times \text{depth}$ of scour. This means that the design should be based on 1.7 times "the greatest discoverable scour below low water level", which, according to Mr. Ahmed's report, is 35 ft. 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 ft. below low water level, or 91.0 ft. below H. F. L., the difference between H. F. L. and low water level being 12 ft.

(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. $35+11\cdot5=46.5$ ft. below low water level, or 58.5 ft. below H. F. L.

(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 ft. 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.⁽²⁾

⁽¹⁾ See Annual Report (Tech.) of Indian Waterways Experiment Station for 1939-40, page 10 and Augual Report (Tech.) for 1942-43, page 54.

⁽²⁾ Vide page 44, and paper no. 167, Inst. of Civil Engineers, London. S 41-4

(i) At the head. Thus, at the head, maximum $scour = (35 + \frac{33}{100} \times 35) + \frac{50}{100}(35 + \frac{33}{100} \times 35) = 69$ ft. below L. W. L., or S1 ft. below H. F. L.

(ii) At the shanks, maximum scour = $(35 + \frac{33}{100} \times 35) + \frac{25}{100}(35 + \frac{33}{100} \times 35) = 58$ ft, below L. W. L. or 70 ft. below H. F. L.

(c) According to Lacey. $R_{Lacey} = 473 \ [Q/f]^{1/3}$

The mean diameter of the bed-sand used in Mr. Ahmed's 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 Mr. Ahmed's reports are based on sieves of British Standard Specifications, the mean diameter is only '191 mm., which will give a bigger depth of scour.

... f=0.77, not 1.0 as assumed.

$$\therefore \ \mathbf{R}_{\text{Lacey}} = 473 \left[\frac{3,50,000}{.77} \right] = 36.4 \text{ ft. below } H. F. L.$$
(i) At head:

Deepest scour⁽¹⁾=2.0 to 2.5 R; the smaller depth being for the larger radius curves. Assuming depth below H. F. L.=2 R,

maximum scour depth below H. F. L.=72.8 ft.

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.⁽²⁾ Accordingly, maximum scour to be adopted in design of aprons would be $2.75 \times 36.4 = 100$ ft. below H. F. L.

(ii) At shank.⁽³⁾—Deepest scour = $1.5 \times R. = 54.6$ ft. below H. F. L.

Recapitulating, according to various authorities, the maximum scour in ft. below H. F. L. would thus be:---

Position.	Spring. (ft.)	Gale. (ft.)	Lacey. (ft.)	Remarks.
Nose or head of large-radius guide bank.	91	81	72.8	(100 ft. according to Indian Water- ways Experiment
Along shank.	58.5	70	54.6	Station results).

It was, therefore, recommended that the atrons should be designed for 100 ft. depth of scour at the nose and 60 ft. along the shanks.

(b) Maximum scour depth and depth of founds for Bridge piers.

(a) According to Spring:-

Maximum scour adjacent to hard or rigid surface $= 35 + \frac{33}{100} \times 35 = 46.5$ ft. below L. W. L., or 58.5 ft. below H. F. L. According to Spring's curve (reproduced in Mr. Ahmed's Report) the foundations should be taken down to 103 ft. below H. F. L.

(b) According to Gale:—The deepest known scour at a soft bank being 35 ft., deepest anticipated scour in vicinity of bridge = $35 + \frac{33}{100} \times 35 = 46.5$ ft.

Add for Class A Rivers, 25 per cent of 46.5=11.6 ft.

Add for grip in Class A Rivers 50 ft.

- ... Depth of foundation below low water level = 108 ft.
- ... Depth of foundation below H. F. L. = 120 ft.

⁽¹⁾ Vide C. B. I. Publication No. 12, page 155.

⁽²⁾ Fide page 78 of Annual Report (Tech.) for 1944 of the Indian Waterways Experiment Station, Poons.

⁽³⁾ Page 155, C. B. I. Publication No. 12.

PART II

(c) According to Lacey. For piers without high-level protections, depth of scour = 2R = 72.8 ft. below H. F. L.; and allowing, according to current practice⁽¹⁾ a grip of R, the level of the foundation of the piers should be 3R below H. F. L., i.e. 109.2 ft. below H. F. L.

(d) At the existing bridges on the Ramganga River, the depth of foundations below H. F. L. appears to be 100 (below L. W. L.) + 12 = 112 ft. at Moradabad, and 85 + 12 = 97 ft. at Bareilly.

In consideration of (a), (b) and (c) above, it was recommended that the foundations should be taken to a depth of 110 ft. below II. F. L., basing particularly on the results of the examination of depths of scour at piers of existing bridges⁽³⁾; if the width of the bridge is made 2,000 ft. 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 (allow-

ing for 20 per cent concentration as usual) $q = \frac{120}{100} \cdot \frac{(3,50,000)}{2,000} = 210 \text{ cs/ft. and } R$

=
$$\cdot 91 \quad q^{2/3}/f^{1/3} = \frac{\cdot 91 \quad (210)^{2/3}}{(\cdot 77)^{1/3}} = 35 \cdot 5 \text{ ft.}; \text{ hence, depth of foundations required} = 3R = 3R$$

106.5 ft. below H. F. L. The fact that the Bareilly bridge foundations carried only 100 ft. below H. F. L. are safe should not be made to influence the required depths at another site: on the other hand, the figure of 120 ft. (required according to Gales) may be reduced to 110 ft., since it only means that the grip of 50 ft. required according to Gale is reduced to 40 ft., which according to modern concepts, is quite sufficient for a maximum depth of scour of 120 ft. below H. F. L. Hence, if the bridge is made 2,000' long, the foundations were recommended to be taken 110 ft. below H. F. L.

5. Length of Guide Bank.

The detailed study of the meanders made for the first point of reference (see para. 2 above) revealed that the proposed length of guide bank—namely L = 1,800 ft., 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 ft.³; for the Kosi, with arc/chord ratio of 2.25 Gales recommended a guide bank length of 2.33 L;⁴⁰ while in the present case of the Ramganga, the maximum arc/chord ratio is 2.67 (see Table 1/II-i).

Mr. Ahmed has stated that experiments have proved that a length L is satisfactory—without quoting the source of his information. If it is the Hardinge Bridge he has in view, that was a specific case where the optimum length happened to be L, when determined experimentally; but the result is not for general application; without specific model experiments made to ascertain the most suitable river training works for the conditions of the Ramganga, 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.

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 1/II-i., taking the worst meander of 1806, arc/chord = 2.67, $M_1 = 1.5$

⁽¹⁾ Major T. McIntyre's Paper on well foundation for bridges, Railway Board Quarterly Technical Bulletin No. 78.

⁽²⁾ Vide page 78 of A. R. (T.); 1944 of the I. W. E. S., Poona.

⁽³⁾ Page 315, December 1938 Journal of Institute of Civil Engineers, London.

⁽⁴⁾ Page 15, Paper No. 5167 of Institute of Civil Engineers, London.

miles == 7,920 ft., $M_b = 1.8$ miles = 9,504 ft., and estimated Qmax = 3,50,000 cusecs, the radius of curvature can then be computed thus⁽¹⁾:--

A B² = AC (2R - AC)
∴
$$\left[\frac{M_{e}}{2}\right]^{2} = \frac{Mb - W}{2} (2R - \frac{Mb - W}{2})$$

∴ $\frac{7920^{2}}{4} = \frac{9504 - 1500}{2} (2R - \frac{9504 - 1500}{2})$
∴ $1980^{2} = \frac{8004}{2} (2R - 4002)$
∴ R = 2,490 ft.

Fig. 3/11-i.

1.1. Figure 3/II-i. shows that, allowing a margin of safety of L/2=900 ft., in the conditions at Ramganga, B/C=2.67, R=2,490 ft., a guide bank of length 4,700 ft.=2.6 L is necessary; and even without any margin of safety—though some is essential—the guide bank would be 3800 ft.=2.1 L.



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 by the Fig. 2/11-i. Station, are shown, in relation to the river courses, in Fig. 2/11-i. The falling aprons of the guide banks should, as far as possible, overlie a subsoil stratum of sand only; and small adjustments should be made in the alignment of the guide bank, if necessary, to secure this advantage.

⁽¹⁾ Page 10, Annual Report (Tech.) of Indian Waterways Experiment Station, 1939-40.

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 to the Station.

(*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 to the Station; the larger scale plan was assumed as the correct one for showing the proposed alignments of the Railway Line.

6.0. It was advised that the superficial evidence of stability of a river course is not a sufficient criterion to justify the final selection of a site of an important bridge on an alluvial river; and that, therefore, it was most essential to take trial borings along the bridge line, extending far into both the banks, to confirm the apparent stability.

II RIVER BEHAVIOUR AND CONTROL

(ii) SCOUR

II. (ii)./ TRAINING THE RIVER INDUS ABOVE THE SUKKAR BARRAGE TO PREVENT SLIPS OF THE RIGHT BANK BELOW THE NOSE OF THE APPROACH CHANNEL.

Introduction

An approach channel was constructed, in the year 1940-41, to exclude excess bed material which was entering the right bank canals of the Sukkur Barrage.

It was expected from the results⁽¹⁾ of the experiments made in this connection that favourable curvature of flow would establish along the right bank, upstream of the approach channel, and a part of the sand exclusion would occur at the nose, the rest—the major part—being effected 2,500 ft. downstream at the Fig. 1 (a)/11 raised sill. Forward flow was also expected throughout the approach channel.

The actual conditions near the entrance of the approach channel were however, not so good. A deep scour-hole, reaching nearly to R. L. 135, developed at the centre of the mouth of the approach channel, giving rise to Fig. 1/11-11. high velocity flow in the middle with a strong return flow along the nose and slack return flow along the right bank.

Due to the very slack velocities, considerable silting, up to R. L. 190, took place along the right bank; while for a length of 400' downstream from the entrance of the approach channel, the right guide bank assumed a steep slope of 2 (horizontal) to 1 (vertical) to $2^1/2$ to 1 due to the deep scour in the middle. When the water level upstream of the barrage was lowered to R. L. 185, the silt and send deposited, being unable to withstand the steep slope—steeper than the natural angle of repose—slipped into the scour-hole, pulling along with it the pitched bank. Three such slips have occurred in the last two years. This has endangered the road to Sukkur, which runs quite close to the right bank.

Mr. Hickin, late Executive Engineer, Barrage Division, discussed, during his visit to the Station in 1941, whether it would be advisable to remove the berm on the right bank; the arguments in favour of this suggestion were:⁽²⁾

(1) Curvature of flow, which tends to prevent bed sand entering the Right Pocket and Canals, is at present, established only downstream of the Pumping Station Section -1,000 ft. downstream of the approach channel entrance. If the berm upstream of Pumping Engine section was removed and water followed that bank, curvature would be established further upstream and sand exclusion would be improved.

⁽¹⁾ CIHR station Annual Report (Tech.) for 1941-42, pp. 16-18.

⁽²⁾ CIHRS Annual Report (Tech.) for 1941-42 (pp. 18 and 19).

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(2) If the face of the inner right bank were pitched, flow would be attracted to it. Hence, once favourable conditions were induced (by dredging), the flow might be held permanently along the right bank from a point further upstream than at present.

(3) It was agreed that filling the scour-hole at the nose of the outer bank in the approach channel from R. L. 134 to R. L. 166 would improve the conditions as regards curvature of flow; and there would then be less tendency for the berm to reform; but considering the amount of stones which would be required to fill the scour-hole,—see Appendix II—the proposal had to be ruled out.

Experiments⁽¹⁾ showed that conditions were likely to improve if dredging were done as suggested above.

It was, however, pointed out, at the same time, that there was a danger of some berming occurring again at the end of Abkalani, under conditions which could not be reproduced in the model; and that dredging might not be quite so successful in the river as indicated by the experiments, particularly if the adverse curvature of flow causing the berm was not eliminated.

Dredging was subsequently done in the approach channel; but berming took place again as predicted.

2. Object

The primary object of the present 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.

3. Experiments in 1/300: J/40 full-width model

Preliminary experiments were done in 1/300: 1/40 full-width Sukkur Barrage model to obtain velocity distribution at KL and outfall sections and to observe general lines of flow for 3, 4 and 5 lacs cusecs discharges, the bed being laid to corresponding conditions.

As the above scales were found to give correct vertical exaggeration and curvature of flow in the previous experiments,⁽³⁾ these were adopted. No proving of the model was, therefore, considered necessary.

4. Experiments in 1/80: 1/40 part-width model

Discharge scale 1/19500.

Time scale 1/12.

1/80:1/40 part-width model was constructed to test possible alternatives to improve conditions in the approach channel. These scales were adopted as they were found to be satisfactory in the previous experiments.⁽³⁾

The part-width model was first constructed up to the outfall section; but as the entry length was found to be insufficient, it was extended upstream to the Bunder section. This was necessary as otherwise the existing nose would have been too close to the entrance to the model.

It was decided to test all modifications for one flood discharge of 4,00,000 cusecs only so as get comparative results quickly. As the conditions at the nose are quite severe for 4,00,000 cusecs, any modification which would give relatively satisfactory results for this discharge is expected to be all right for other discharges; the best measure was later, however, tested for other discharges.

Fig. 3/11-ii

From the preliminary experiments in $1/300: 1/40 \mod 1$, it was observed that 38 per cent of the total river discharge passes through the width of the river reproduced in the part-width model; this required a discharge of 78 cusecs in the $1/80: 1/40 \mod 1$ and represented condition= $4,00,000 \mod 1$ cusecs in the river.

(3) Annual Report 1940-41 of CIHRS, Poona, (page 14, 15)

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Appendix II.

⁽¹⁾ OIHRS Annual Report (Tech.) for 1941-42 (pp 18 and 19)

⁽²⁾ Annual Report 1939-40 of CIHRS, Poous, (page 12)





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5. Proving the model

Experiment 1-

The extended model was laid in the same sand, viz. dam sand (mean dia. 0.32 mm.), which was used in the earlier experiments, descr bed in the Annual Report for 1940-41. The slope exaggeration was the same as before, viz. 3.5.

A discharge equivalent to 4,00,000 cusecs was run; the bed movement was found to be satisfactory and the gauges were correctly reproduced.

The velocity distribution at the Outfall section in the part-width model was compared with that obtained in the full-width model. Higher velocities were obtained along the right bank and lower on the left as compared to the full-width model. In order to set this right, the section on the left bank at the entrance to the model was made deeper, in gravel, to increase the discharge intensity along the left bank. After a few trials, a section was arrived at, which faithfully simulated the velocity distribution observed in the full-width model.

After 4 hours run when the model had stabilized, bed and surface lines of flow and depths were observed. General flow conditions were found to be similar to those in the full-width model.

6. Experiments to improve conditions in the approach channel

The present trouble of deep scour near the nose and occasional slips in the right bank are mainly due to the restricted width at the entry and steep slopes of the nose, which always induce deep scour, and, hence, concentrate the flow. So it was considered that an extension of the present outer bank with a wider, flared entrance, and flatter slopes to the new nose on the approach channel side, would improve conditions.

Pig. 8 (a)/11-(ti)

Fig. 9 (b)/11-(11)

Experiment 2-

The outer bank of the approach channel was, therefore, extended by about 400 ft. The nose was given a side slope of 5 to 1 on the approach channel Fig. 6/11-(6, side, of 4 to 1 at the nose, and 3 to 1 on the river side. A smooth transition was given in between the present 3 to 1 slope and 5 to 1 of the extended nose.

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The model was laid to 4 lacs cusecs condition and the equivalent discharge was run. In the beginning for about 2 hours, there was a part-width, central flow with two regions of return flow on either side of the approach chonnel for about 460 ft. length from the entrance. After 4 hours run, the flow in the approach channel became forward throughout, consequent upon the accretion in the scour-hole at sections 2 and 3. Scour developed around the new nose to R. L. 168 only, the original bed as laid being at R. L. 177.

It was considered possible that the accretion in the approach channel might have been mainly or in part due to the deposition of sand washed out from the scour-hole around the new nose-a model limitation-and in that case, the improvement could not be wholly attributed to the extension of the approach channel.

Experiment 3-

To isolate this effect, only sections 2, 3 and 4 where accretion had occurred were relaid to existing 4 lacs cusecs conditions, leaving the rest of the bed and scour around the noseas obtained in Expt. 2.

Discharge equivalent to 4,00,000 cusses was run; 2.5 cft. of bed material was injected per hour, this being arrived at after trial and error-so that there was uniform bed movement. Depths were observed at 1 hour intervals and lines of flow after 12 hours run.

Fig.4(a)Il,ii)

Depths at sections 2, 3 and 4 are shown in Fig. 4 (a)/II (ii) after 6 and 12 hours run. There was progressive accretion at these sections. A forward flow was established throughout the approach chennel after about 9 hours (=4.5 days).

The extension of the approach channel by 400 ft. gave satisfactory results; but as its construction would be very difficult and costly due to the deep scourhole around the existing nose, it was considered better to lay a truncated nose, 130 ft. long, out of the proposed extension of the outer bank, leaving a gap of about 270 ft. between it and the existing nose. The truncated nose was given the same side slopes as in Expt. 2, its rear slope being 3 to 1.

With the truncated nose in position P, the following experiments were done Fig. 6/11 (ii) to obtain the best positions and numbers of right bank groynes, necessary to hold the right bank, in the initial stages, till accretion of the scour-hole occurs and the return flow stops.

Experiment 4---

Fig. 6/11 (ii)

Truncated nose position P and five, 100 ft. long, 30° pointing upstream, semi-permeable, tree groynes to protect the right guide bank.

The experiment was done with 4,00,000 cusecs as before. The following points were noted:-

Photos 5047 5056 and 5057.

There was cross flow though the gap from the approach channel side to the river side for about $2\frac{1}{4}$ hrs. $(\equiv 2\frac{1}{2}$ days) in the beginning. From $4\frac{1}{2}$ to 7 hours, there was no perceptible cross flow in either direction.

Quite appreciable accretion occurred in the gap; the bed at section 2 which was laid to R. L. 168 had built up to R. L. 185. In the beginning, bed maternal moving along the approach channel deposited in the gap, while later the material from the river side accumulated there. So, after 12 hours (=6 days), the gap was completely filled up to R. L. 185 and there was slight cross flow from the river to the approach channel; but, at this stage, the outer bank can be extended to the new nose at little cost.

Appreciable accretion occurred at sections 2, 3 and 4. There was some slack return flow just upstream of groyne 1, but as the scour-hole around the $(b^{Fig.4})$ in nose filled up, the return flow progressively subsided. After 12 hrs. (=6 days), the flow was forward throughout the approach channel.

Fig. 4

RIGHT BANK APPROACH CHANNEL AT SUKKUR



Photo 5047. Experiment No. 4. Flow conditions on River Side, looking upstream towards the truncated nose.



Photo 5056. Experiment No. 4. Conditions . after 12 hours run showing accretion at 2 & 3 sections.

Photo 5057. Experiment No. 4. Return flow along the right bank and cross-flow in the gap in the beginning.




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Experiment 5.-

In order to reduce the cost, only three, 50 ft. long, semi-permeable tree groynes were tested, in positions shown in fig. 6/II (ii), with new nose in Fig. 6/II-ii. position P.

The flow conditions in the gap were similar to those in experiment 4. Fig. 5 (6)/11-There was accretion in the gap to R. L. 185, as in the previous experiment. In The slight return flow in front of the threegroynes and upstream of groyne No. 1 subsided and forward flow was established throughout the approach channel as Fig. 4 (c)/11accretion progressed at sections 2, 3 and 4. This showed that three, 50 it. long, ii. tree groynes were equally effective.

Experiment 6.-

The truncated nose in position P and two 50' long impermeable stone Fig. 0/11.41. groynes on the right bank.

As stone is readily available at Sukkur, it would be easier—and perhaps even less costly—to construct stone groynes. So two, 50 ft. long, atone groynes were tested. Accretion in the gap and at sections 2, 3 and 4 was quite satistication in the gap and at sections 2, 3 and 4 was quite satistication.

factory and comparable to the previous experiment; but even after 12 hrs. run, Fig. 4 (d)/llthe return flow persisted in the vicinity of the groynes. Wider spacing of the groynes was tried which only worsened the flow conditions. Conditions were, thus, not so favourable as in the previous experiment.

Experiment 7.—

Though the truncated nose in position P was found to be quite satisfactory, the rear slope of the nose came right into the deep scour-hole at the existing nose and so it would be very difficult and expensive to construct and maintain it. The position of the nose was, therefore, moved further upstream so that the rear slope was clear of the edge of the scour-hole. The alignment of ^{Pix. 6/11-ii.} the new nose in position Q, with respect to the centre line of the nose, was kept the same as before. The proposed side slopes were also the same.

As the deep scour, which is the main cause of slips in the right guide bank, would be accreted after the construction of the truncated nose allowing a wider entrance to the approach channel, it is expected that groynes will not be necessary to hold the bank, which is pitched and should stand fairly high forward velocities. No groynes were, therefore, provided in this experiment.

The model was laid to 4 lacs cusecs condition with the scour round the new nose as obtained previously. Discharge equivalent to 4 lacs cusecs was run. The following points were observed:—

(i) The cross flow from the approach channel to the river mainly hugged the existing nose.

(ii) There was a region of slack return flow at the rear side of the new nose.

(*iii*) The process of accretion in the gap was slower with nose Q than in the previous experiments with the new nose in position P. Even after 12 hours run, there was some cross flow from the approach channel to the river along the existing nose. The bed in the gap had built up to nearly R. L. 180 after 12 hours run, as against R. L. 185 in the previous experiments.

Some accretion in the return flow region is expected to occur in the river, which could not be reproduced in the model—this is a model limitation. This would partly accelerate the process of building up of the bed in the gap.

(iv) The accretion in the approach channel was also a bit slower, the bed levels after 12 hours run being slightly lower than those obtained at $p_{ig. 4}(r)/11$.

The flow in the approach channel became forward throughout after 12 hours run except for a small region of return flow near the present nose.

The outer bank was then extended to the truncated nose Q and the experiment was repeated with river discharge $\equiv 4,00,000$ cs. and the approach channel discharge $\equiv 45,000$ cs., in order to see if a similar scour would develop at the same relative position inside the extended approach channel as it does under the existing conditions. As was anticipated, the flow was smooth throughout the approach channel with forward velocity along the right bank and there was no sign of any scour developing inside the approach channel.

1/80 geometrically-similar, part-width, Sukkur Barrage model to test stability of the truncated nose for a 7. super-flood of 9,00,000 cusecs.

Due to vertical exaggeration in the 1/80: 1/40 model, it was considered that the local scour at the truncated nose would be shown more accurately in a 1/80 scale, geometrically-similar model, with enforced entry conditions for higher discharges. These experiments are described below :---

The experiments were designed to test the following two points :---

(a) Previous experiments " for the design of the nose of the outer bank had shown that "gulleting" would occur if the side slopes were flatter than 1 in 3. As flatter slopes were employed in the proposed nose, it was tested for stability in 1/80 geometrically-similar model.

(b) To obtain maximum depths of scour around the truncated nose in position Q under 6 and 9 lacs cusecs flood conditions, so as to provide proper apron protection.

Preliminary experiments were done in 1/300, 1/40 full-width model to obtain velocity distribution at Outfall section, for 6 and 9 lacs cusecs, with truncated nose in position Q and 45,000 cs. in the approach channel. Velocity distribution at Outfall section was not perceptibly affected by variation of approach channel discharge from 30,000 cs. to 60,000 cs. Depths at sections downstream of Bunder were observed after the bed had stabilized for 6 lacs cusecs discharge after a run of 7 hours.



(1) Annual Report 1939-40, CIHRS. Poons (pr. 18, 19).





7 (a) Experiments to test gulleting

Experiment 8.—

The model was relaid for 6 lacs cusecs according to depths obtained from 1/300, 1/40 full-width model. Correct velocity distribution, as observed in full-width model, was obtained by slightly changing the Bunder section as in Fig. 9.1046. experiment 1.

The truncated nose in position Q was made with a sand core, pitched with 3/16'' dia. gravel equivalent to "one man stone"⁽¹⁾. Discharge was gradually increased from 6 lacs to 9 lacs cusecs allowing sufficient time for the bed to stabilize.

For discharges above eight lacs cusecs, considerable gulleting occurred at the truncated nose on the river side which was given **a** slope of 4:1. No gulleting was observed on the approach channel side.

(1) Annual Report 1939-40, C. I. and H. R., Poons (pp. 50, 51).

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Experiment 9.-

In order to prevent gulleting at the nose, the slope was altered to 3:1. In order to prevent guilding. Inappreciable gulleting occurred and that Similar flood cycle as before was run. Inappreciable gulleting occurred and that Similar flood cycle as before was tud. As such super-floods are very for only the highest-ever 9 lacs cusecs discharge. As such super-floods are very for only the highest-ever 9 lacs cusecs are very small—the altered at tor only the highest-ever s late variation will be very small—the altered slope 3:1 rare,—and if they occur their duration will be very small—the altered slope 3:1 rare, — and it they occur man address in reserve to meet the conditions conse-may be adopted; stones may be kept in reserve to meet the conditions consequent upon any such unprecedented super-flood occurring.

7(b). Maximum depth of scour for opron protection

In the 1/80 geometrically-similar, part-width model, the truncated nose was constructed in position Q. The sides were plastered and were stuck with was constructed in position Q. The slats which preserves that was given on the 3/16'' gravel to reproduce correct roughness. A slope of 5:1 was given on the approach channel side up to R. L. 180, the top of the nose being at R. L. 205. Below R. L. 180, a steeper slope of 3:1 was continued down to R. L. 100 to simulate a launched apron. The rest of the nose was given a slope of 3:1 throughout.

The following experiments were done :---

Experiment 10.-

The model was laid in dam sand of mean dia. 32 mm. according to depths obtained in 1/300: 1/40 model for a 6 lacs flood, except at the scour hole around Fig. 10/11-ii the new nose Q and the present nose, which was laid according to 4 lacs cusees conditions obtained in 1/80: 1/40 part-width model to start with.

The discharge was slowly increased to 6 lacs cusecs and was run for Fig. 11/II-ii 14 hours $\equiv 5$ days 6 hrs. till stable depths of scour were obtained. The approach

channel discharge was 60,000 cusecs throughout.

Experiment 11.-

The discharge was further increased to 9 lacs cusecs, keeping the approach channel discharge 60,000 cs. This was run for 8 hours = 3 days, i.e. the Fig. 12/11-ii maximum possible duration of the super-flood. Depths around the truncated nose Q were observed.

Experiment 12.-

The outer bank of the approach channel was extended to position Q.

The model was laid as in experiment 1. The discharge was slowly increased to 9 lacs, and continued for 8 hours ($\equiv 3$ days). The approach channel Fig 13/11-ii discharge was 60,000 cs. Depths observed are as shown in fig. 13/11-ii.

Table 1/II-ii shows R. L.s of deepest scour obtained in various experiments.

	Level o	Water level at	
	River side.	Approach channel.	Outfall Section
Truncated nose:—			
(a) River discharge = 6 lakh cs. and ap- proach channel Q=60,000 cs.	R. L. 141	R. L. 150	R. L. 200 8
(b) River $Q=9$ lakh cs. and approach channel $Q=60,000$ cs.	R. L. 131	R. L. 135	R. L. 204 [.] 1
Extended nose (i. e. continuous outer bank up to Q)			
(c) River $Q=9$ lakh cs. app. ch. $Q=30,000$ cs.	R. L. 119	R. L. 163	R. L. 204 [.] 1
Existing nose:			
River $Q = 4$ to 5 lakh cs. (app. ch. Q = 45,000 cs.)	R. L. 150	R. L. 120	R. L. 196 ^{.8} to 198 ^{.4}

TABLE 1/II-ii

EXPT. 7. - STABLE DEPTHS OF SCOUR AROUND TRUNCATED NOSE IN POSITION Q.RIVER DISCHARGE 4 LAKHS CS., APPROACH







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PART II

As the chances of a 9 lakh flood occurring in the same year in which the truncated nose will be constructed are very rare, the apron may be designed for scour depths obtained with 6 lakh flood for the approach channel side *only*. On the river side, the apron should be designed for full depths obtained with a 9 lakh flood and with the outer bank extended to the new nose (which gave the worst results for scour), when the maximum depth of scour was down to 12. I. 119 on river side. The model must give relatively deeper scour due to side restriction of the left bank; so the design will be reasonably safe.

In experiments 10 and 11, accretion actually occurred at the rear slope of the truncated nose; but in order to prevent any possible slipping of rear slope into the deep scour-hole at the upstream end of the present nose, apron designed for maximum scour to R. L. 160 is provided. For details of the apron design, see Appendix I, Fig. 14/II-ii.

Recommendations.—

App. I and Fig. 14/11-11.

(1) As the truncated nose in both the positions P and Q gave quite satisfactory results, an intermediate position may also be expected to give equally Fig. 6/11.01, good results.

(2) The new nose should be constructed as near as practicable to the existing nose, keeping the alignment with respect to the centre line of the nose AA same as in P or Q. The rear face should make an angle of 85° with the centre line AA, as this gave the optimum results.

Other details such as apron length, thickness are shown in Appendix I and Fig. 14, II-ii.

Appendix II compares the quantity of stone required for (i) filling the scour-hole and (ii) constructing the truncated nose.

8. Effect of extended outer bank or its truncated nose on exclusion of sand in the approach channel

Due to the differential slopes of the extended bank or its truncated nose, the scour at the nose was greater on the river side than in the approach channel. After the scour-hole at the existing nose in the approach channel was filled up, the flow throughout the approach channel was forward. This would tend to establish a more favourable curvature of flow upstream of the approach channel than at present; it is expected that this would improve the sand exclusion. Tests made confirmed this point both by the number and extent of bed seeds entering the approach channel and by the amount of silt trapped in the pits below the right Bank Canals.

9. Afflux due to the proposed extension of the outer bank of approach channel

There was no perceptible additional afflux in the model due to the extension of the approach channel even to position Q.

It was also seen from the 1946 contour plan that a berm had formed on the right bank upstream of the nose. This seems to have been partly croded in 1947. Such berms are formed, at present due to the deep scour-hole at the nose of the approach channel attracting the flow and causing return flow on right bank.

The proposed extension of the approach channel will establish a fairly high forward velocity along the right bank. This will maintain a deeper channel on the right bank than at present. As the velocities will be sufficiently high to carry the fine material in suspension, no such trouble as berming is expected to occur. This will, however, be tested in the model.

APPENDIX I

Design of falliny apron for the truncated nose in position Q

Fig. 14/II-ii. Fig. 14/II-ii shows the depths of scour as obtained in experiments 10 and 12 on which the length of apron is based. The initial thickness of apron provided is 10' and the length provided is based on the assumption that the apron will launch to a 2:1 slope with a minimum covering of 4' of stone.

Part of nose Fig. 14/II-ii.	R. L. of max. scour for which apron is provided (ft.) 2	Quantity (cft.) of stone required per fost length of apron for a minimum covering of 4' of stone. 8	Length of horizontal apron required, in ft. (thickness of apron = 10 ft.) 4	Remarks.
A	160	179	18	Top of apron R. L. 180. Bottom of apron R. L. 170.
в	150	268	27	Do.
С	140	360	36	Do. •
D	125	490	5C	Do.
Е	115	580	58	Do.
F	125	404	41	Do.
G	145	316	32	Do.
н	160	179	18	Do.
J	150	268	27	Do.
к	155	224	23	Do.

TABLE 2/II-ii

APPENDIX II

Approximate quantity of stone required for filling the scour-hole in the approach channel and constructing a pitched, truncated nose in position Q

(1) Quantity required for filling the scour-hole.

Difference in cross sectional area before and after	c. s. 2	c. s. 3	c. s. 4
12 brs. run in expt. 7 :	4,650	6,000	8.000 so ft
Assuming mean area of cross se	ction to i	be filled as 6.000	sa ft
Length of scour-hole=550 ft.			54.10
Total quantity of stone required scour-hole $= 550 \times 6,000$	for fillin	g the	
=3,300,000 cft.			

(2) Quantity of stone required for truncated nose.

(a) Approximate length around which 9' thick stone is to be provided.......180 ft.
Length along the 3:1 slope=100 ft.

 \therefore Quantity of stone = $180 \times 100 \times 9$ cft.



- (b) Length around which 6' thick stone is to be provided = 640 ft. Quantity of stone per ft. run = 140 × 6 cft.
 ∴ Total quantity = 640 × 140 × 6 cft.
- (c) Quantity of stone required for apron 10' thick. Area of apron provided = 42,700 sq. ft.
 ∴ Quantity of stone required = 42,700 × 10
 - = 427,000, cft.
 - ... Total quantity of stone for island
 - (a) $180 \times 100 \times 9 = -162,000$
 - (b) $640 \times 140 \times 6 = 540,000$
 - $(c) \ 42,700 \times 10 = \ 427,000$

Total ...1,129,000 cft.

Quantity of dredged sand for nose.

Mean area of $nose = 31 \times 1,600$ = 58,600 sq. ft.

Quantity of sand = $58,600 \times 32$.

=18,80,000 cft.

Quantity of stone for pitching after the gap is filled.

Length of gap = 550 ft.

Mean slope 3.5:1

(transit from 5:1 to 3:1)

: with bed level at R. L. 180 and top at R. L. 205;

quantity of stone per ft. run for 4' covering = 4×73 cft.

: total quantity for pitching both sides = $4 \times 73 \times 500 \times 2$

=2,92,000 cft.

 \therefore total quantity of stone required = 11,29,000 + 2,92,000 = 1,421,000 cft.

as against 3,300,000 cft. required for filling the scour-hole.

Construction of truncated nose will, therefore, be comparatively cheaper than filling the scour-hole with stones.

II. RIVER BEHAVIOUR AND CONTROL

(ii) SCOUR

II-ii-1. Emergent measures for preventing serious damage to the central sluices of the Mahanadi Anicut at Cuttack-Orissa

(1) Introduction.

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 cs. 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 unsuitable 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.

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. 28

Fig. 15/II-ii, Photos 3088 and 3098 in Para. I. 2.

During the floods of 1945, however, serious damage to the talus occurred which threatened to encroach upon the apron, fig. 15/II-ii and photos 3088 and The scour depth increased to 28' on the southern end and 15' on the northern end. The fourth dwarf wall was completely washed away and the damage to the laterite block floor at the two corners extended up to the third dwarf wall, which had rather shallow foundations.

The local officers proposed to repair the damaged floor, after the floods of 1945, with laterite blocks, laid in lean cement concrete, up to the fourth dwarf wall. Instead of the usual bullah cribs, it was proposed to place blocks of wire mattresses of lengths 10 to 15 ft. and 5' deep in continuation of the last dwarf wall

Fig. 16/11-ii gradually sloping down to the bed of the river in steps.

On invitation, the Director inspected the damage early in May 1946. Though it was essentially a problem for specific model experimentation, yet due to insufficient time (barely 6 weeks) available for construction, advice was given on the spot based on previous experience of this Station. The measures suggested had to be limited to be such as, in the opinion of the local officers, could be executed in time before the commencement of the floods.

(2) Cause of trouble.—As stated in para. 1 above, it was evident that the trouble occurred, not during high floods when the downstream water level was high so that a standing wave could form on the pacea pavement or was drowned, but during low floods, when all the weir crest shutters were raised, and the full river discharge of about 40 to 50 thousand cusecs was passed through the Central Sluices with upstream water level relatively high at retention level (R. L. 685) and downstream level at its lowest (R. L. 610). Under these conditions, hypercritical flow persisted over the dry concrete blocks, which caused scour downstream. This diagnosis was also borne out by the Director's calculations, see Appendix I, and confirmed by the opinions of the local officers

The defects in the existing design responsible for the trouble were evidently:-

(1) The very flat glacis and the downstream cistern level too high and length of pavement too short relative to the level at which the standing wave would occur and the length up to which its disturbances would extend;

(2) the disturbed $5' \times 2' \times 2'$ friction blocks which caused increased turbulence and increased scour ;

(had the friction blocks been suitably positioned and firmly held, they would undoubtedly have been a measure in the right direction, being one of the standard devices for dissipating energy; but their effect, at the best, would have been small on account of the existing, flat and high downstream pavement); and.

(3) scour at the steep noses of the extended divide wall generating excessive scour.

(3) Emergent measures suggested by the Director without model experiments : -

The proposal of the local officers was simply a palliative and would have perpetuated the present unsatisfactory conditions. The following advice was given by the Director, aimed at removing the causes of the trouble :-

Fig. 17/II-ii.

(i) Commencing 15 ft. downstream of the fulcrum of the existing Central Sluice Gates, there should be an upstream glacis 1 in 3, 2' wide crest at R. L. 63.5 (i.e. 5' above present cill level), followed by a downstream slope 1 in 4. This downstream glacis would serve two purposes: (a) it would reduce the intensity of discharge from 95 to 36 cusecs/ft. with the same upstream water level, and (b) it would help to hold the standing wave on the pucca pavement and confine the travel of the standing wave to a narrow range. (Compare Appendix III with Appendix I).

(ii) A baffle-wall 4 ft. high (top R. L. 62.5) on the lst apron wall, i.e allowing a baffle pavement of 20 ft. width, from the downstream too of glacis to downstream face of baffle wall. This baffle would serve two distinct





purposes: (a) it would head up the water above it to a level of R. L. 62.5 against the present minimum downstream level of R. L. 61.0 (see Appendix II)—thus ensuring that a standing wave would form: and (b) it would withstand the actual impact of the high-speed, hypercritical stream of water and so dissipate energy.

The baffle wall with 1 in 4 downstream glacis would probably make the existing downstream pavement level just about low enough to hold the wave; but by itself neither the glacis nor the baffle wall would be sufficient for dissipation of the energy. It is true that the baffle pavement level (ride Appendix III) is required to be at R. L. 570 against the existing level R. L. 585: but the local officers did not consider it possible within the time left, to lower the existing downstream pavement. Calculations in Appendix III ignore friction and for reasons explained by Rai Bahadur Khosla in his Paper (C. B. I. Paper No. 12, page 164), the actual position of the standing wave would be somewhat upstream of the calculated position.

The baffle wall should be very firmly anchored as upon its being securely in position depended the formation of the standing wave on the pucca pavement. Its suggested thickness was 3 ft, at the base and 2 ft, at the top, because the unbalanced pressure against the baffle was small, and if properly anchored, there should be no difficulty in its remaining intact.

The baffle was recommended to have a curved face upstream with radius equal to, and ending at, 2/3rd the height of the baffle.

(iii) For draining out leakage water collected between the front gates and the glacis, in order to facilitate the manipulation of the rear gates in dry, four sluices, each $1' \times 1'$ opening. (or 15'' dia. Hume Pipe) evenly distributed, in the full width of the glacis, were recommended. These sluices should have sliding shutters on the upstream glacis face; and should generally remain shut and be opened only during manipulation (i.e. raising) of the rear gates of the Central Sluices.

(iv) There should be a deflector 1.5' high, on the top of the last safe Apron Wall, which was stated to be Apron Wall No. 3. Though similar in design to a baffle, its purpose is markedly different: while a baffle is primarily required to dissipate energy, a deflector is required to deflect the high velocity stream near the bed and cause a horizontal bed roller to form, which piles up sand downstream of it, generally at a slope of about 1 in 3.

As there would be greater scour near the sides, i.e. adjacent to the divide walls, for a distance equal to 1/4th width (=475/4=120 ft.) from each bank, the deflector height should vary from 2 it. along the divide wall to 1.5 ft. at quarter point, the central 235 ft. being 1.5 ft. high.

The best design of the deflector was a continuous one with gaps 2 ft. wide, left at intervals of 50 ft. or so, to drain off water freely downstream; short walls, 5 ft. wide, being constructed in front of the gaps to prevent water jetting from the gaps.

The deflector walls, like the baffle wall, should be *firmly* anchored to the existing Apron Wall.

(v) The existing divide walls were recommended to be raised to be sufficiently high to prevent water overflowing on to the anicut and the length in which raising was necessary should be limited to the length of the primary wave over the baffle. According to calculations given in Appendix III-A, the height and length of the primary wave would be, respectively, 6.65 ft. and 12 ft. and so the divide walls were advised to be raised for a distance of 10 ft. on either side of the baffle to R. L. 66.0.

On the upstream of the glacis the retention level being R. L. 68.5, the divide walls should be at a level of R. L. 69.0 so that the divide walls were recommended to be raised in a slope, being R. L. 69.0 at the gate and R. L. 66.0 at a point 6 ft. upstream of the baffle: then continued at R. L. 66.0 for a length of 12 ft. (i.e. to a point 10 ft. downstream of the baffle) and joined to the existing wall in any easy slope.

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(vi) Time permitting, the existing nose of the divide wall should be extended by a length of 17.5 ft. beyond Apron Wall No. 4 so that it had a sloping nose of 1 in 5, reducing from 3.5 ft. (existing height) to 0 ft. (R. L. 56.5). The nose should be pretected all round by an apron of one-man-stone, 20' wide $\times 3'$ thick, with bottom laid at R. L. 50.0.

(vii) Downstream of the deflector, a loose stone apron 4' thick of oneman-stone should be laid for a distance of 25 ft. at a slope of 1 in 3. This apron, as well as the apron protection around the divide wall, were intended to launch so as to line the anticipated bed configuration.

(viii) All friction blocks constructed in 1945 should be carefully removed from the talus of dry blocks, broken up and thrown in the scour hole below the deflector. Their presence would largely undo the good done by the proposed measure by causing eddies and disturbances, thereby increasing the downstream scour.

This was the minimum work considered necessary. It was not the optimum solution but was expected to fit in with the final solution as determined from model experiments. Were it possible to lower the baffle pavement level, it was recommended that this should be done to the level suggested in Appendix III.

4. Secondary effects of the proposals.—By these proposals it would be possible to pond to the retention level of 68.5 for a relatively longer period, i.e. up to the time when the river discharge fell to 17,000 cusecs as against about 45,000 cusecs at present. Shore sluices would remain open up to river discharge of 17,000 cusecs. This itself would assist the silting up of the scoured channel below the Central Sluices. Till experience was gained, the change in regulation and the sanding of the central channel would require careful attention and watch.

In spite of the raising of the Central Sluice cill by 5 ft., there would be little or no difference in the upstream H. F. L. to pass the maximum discharge or the afflux: because the small reduction in ventage area of about 3 per cent 5×475

 $(\frac{17.5 \times 475 + 17.5 \times 345 + 5510 \times 11}{17.5 \times 345 + 5510 \times 11} \times 100)$ would be more than compensated by the improvement in the coefficient of discharge caused by the sharpening of the crest of the Central Sluices.

5. Model Tests:-

While collection of materials and preparation for construction of the suggested measures were in hand, hurried model tests were made at Poona to see if any modifications could be effected with a view to minimise the construction work and to test the suggestions already made.

These experiments fully confirmed the Director's diagnosis of the cause of the trouble and the conditions under which damage occurred; it was proved by the model that by far the worst conditions occurred not at high flood but during low floods as explained before; and, save in a minor detail or two, the experiments also confirmed the suitability of the emergent remedial measures suggested.

The Model.—Two central spans with waterway $\equiv 189.58$ ft. were reproduced in smooth cement plaster, in a 1/40 G. S. Model, first under the existing conditions and then with the suggested 'modifications and other modifications detailed below.

Tests were made to observe the coefficient of disharge, the position of the standing waves, the water profile, qualitative nature and conditions of flow, under the existing and the modified conditions proposed for dissipation of energy, both at maximum floods and at low floods (in the latter case all the discharge passing through the Central Sluices).

PART II

With the glacis added, as suggested by the Director, the coefficient of discharge was observed in the model to increase to 3.16 against 3.2 predicted, rida Appendix III. Hence, as in Appendix III, the total discharge the Central Sluices would pass after modification, with upstream water level 68.5 and downstream water level low (clear overfall), would be 17,000 cusees as already predicted. That the raised crest of the Central Sluices, would make no difference to the upstream water level, or the afflux, for passing the high flood discharge was thus corroborated.

(7) Test of Measures to dissipate energy

The following model tests were made:-

(i) Photo No. 5026 shows the flow conditions with the existing profile Photo 5026. of the Central Sluices, upstream water level 68.5 and downstream water level 62[.]0. The photo clearly shows that there is hypercritical flow under these conditions all over the pavement and a standing wave tends to form only at the downstream end of it, at or beyond curtain (dwarf) wall No. 4. This hypercritical jet and the disturbance created by the standing wave occurring in the unprotected area are confirmed as being the primary cause of the trouble.

(ii) Photo No. 5028 shows the conditions, after modification accord- Photo 5029. ing to the Director's proposals, namely a 5 feet high glacis, a 4' high balllo fixed 20 ft. downstream of the glacis, followed by a 1.6' deflector on the last safe (i.e. third) curtain wall. It was observed that the 4 ft. high baffle wall made the trough too full with the result that, though a standing wave formed well up on the downstream slope of the glacis (only 6 ft. downstream of the crest), the dissipation of energy was not fully satisfactory.

It was also observed that the 1.6' high deflector was, in effect, working as a baffle, by controlling the position of the second standing wave, which without it formed just on the third dwarf wall. Further, though there was subcritical flow downstream of the 1.6' deflector, yet, due to the nearly horizontal stone apron below it, the beneficial roller with horizontal axis enabling silt to pile up against the downstream of the deflector (for which the deflector is intended) was absent.

Thus, because of the high level of the pavement (which could not be lowered in the time available) and the resulting persistence of hypercritical flow downstream of the primary baffle wall, the model indicated that this was a case for two or more continuous baille walls, a deflector being largely ineffective unless the pavement was sufficiently lowered to give subcritical flow.

(*iii*) Photo No. 5029 shows conditions as in (*ii*) above but with $\frac{Phone}{No. 5029}$, primary baffle wall (on D. W. No. 1) reduced in height to 3 ft. This lowered the primary standing wave, making it more satisfactory in dissipation of energy-in spite of the relatively lower water level just upstream of it compared to the water level with 4' high baffle-because the trough was too full with the higher baffle.

(iv) Since hypercritical flow persisted downstream of the baffle wall, at least one more baffle wall appeared necessary. Instead of a deflector (which did not fulfil its function), a second baffle wall higher up the pavement was, therefore, tried. Photo No. 5030 shows a 3 ft. high baffle on the first curtain wall [as in (iii) above], followed by 1.6 ft. continuous No. 6030 baffle on the second curtain wall (i.e. 30 ft. downstream of the 3 ft. high baffle). This apparently improved the dissipation of energy, as there were three standing waves, the first two with adequate turbulence, both on the pucca portion of the pavements, and the third, a feeble standing wave, 8 feet below the second baffle, with downstream velocities greatly reduced.

(v) In this test, a 1 in 3 sloping apron was provided downstream of the third dwarf wall, with only a 3' baffle on first dwarf wall and 1.6'baffle on the second dwarf wall.

Photo

With downstream water level \equiv R. L. 62, the conditions were very satisfactory in that the dissipation of energy was good and a roller with horizontal axis formed on the 1 in 3 slope which persisted even when the downstream water level was lowered to R. L. 61; but a standing wave tended to form just on the third dwarf wall which could very effectively be moved upetream to a point nearly 10' below the second baffle by adding a 1 foot baffle-cum-deflector on the third dwarf wall; and then the conditions below the third dwarf wall markedly improved, the sub-critical pulsating flow below the end wall becoming slack and smooth. Even with downstream water levels falling as low as R. L. 580, this gave the best results of all the measures tried.

(vi) Photo No. 5027 shows conditions with a 3 ft. high baffle on 1st curtain wall, 2 ft. high baffle on 2nd curtain wall, and 1.6 ft. high continuous baffle on 3rd curtain wall with a horizontal pavement downstream of the 3rd curtain wall as it exists in the prototype. This gave the second best results and the best of all the measures with a horizontal parement downstream of dwarf wall No. 3 in that there was good dissipation of energy in three standing waves above the 3rd dwarf wall-one-half way up the downstream face of the glacis, the second 20' upstream of the second baffle on the second curtain wall, and the third 20' upstream of the third wall. Though a feeble, fourth standing wave formed on the kacha stone pron downstream of the third dwarf wall. yet considering the dissipation of energy in the first three standing waves it was anticipated that no damage would occur to the apron which had withstood intense hypercritical flow conditions previously. The only way of totally avoiding the formation of the fourth standing wave was to lower the katcha stone apron downstream of the third dwarf wall [vide (v) above] -which, it was stated, was not feasible in the time available, however desirable it might be. This proposal was tested for further retrogression of downstream level down to R. L. 58 and was found reasonably satisfactory, but not as good as proposal (v).

Several other alternatives were tried, with different heights, numbers, and positions of the baffle walls, and with downstream water levels ranging from R. L. 58 to 60, but within the limitations of time and minimum alterations, none was so effective as (v) if the apron could be lowered, or (vi) if it could not.

(8) Modifications of the Measures suggested after model experiments

Thus, as a result of the model tests, the only modifications necessary to the measures already suggested were the following:—

(i) The first baffle wall to be 3 ft. high (top R. L. 61. 5) instead of 4 ft. high;

(ii) an additional, baffle wall, 2 ft. high, 15" wide, rectangular in section equally firmly anchored, was provided between the first and the third walls, on the second dwarf wall; and

(*iii*) slight reduction in the raising of the divide walls to suit the actual water profile obtained in the model.

(9) Conclusions

The ideal solution would necessitate the pucca pavement and the loose stone apron to be depressed as well as providing the glacis, a baffle and a deflector. Lowering the pucca pavement to the correct cistern level is required to ensure the formation of the standing wave at the desired position on it; and lowering the loose stone apron below the deflector to a slope of 1 in 3 for obtaining the beneficial roller with the horizontal axis which enables silt to pile up immediately downstream, precluding undermining.

Without lowering of the pavement and the apron, all that could be done was to suggest measures to reduce the hypercritical jet by dissipation three or four times, through standing waves created by a series of baffles, to a velocity below the normal velocity required to move a one-man-stone.

Fig. 17/II-ii.

CENTRAL SLUICES OF THE MAHANADI ANICUT AT JOBRA (Orissa)



. Standing wave Dwarf (curtain) wall No. 4

Hypercritical flow

5026. Photo showing flow conditions with existing conditions and upstream water level = 68.5; downstream water level = 62.0.



 1.6' wall on 8rd Dwarf (curtain) wall.
 4' baffle on 1st Dwarf wall.

trough too full standing wave.

5028. Photo showing conditions with a 4' baffle on 1st curtain wall and a 1.6' baffle on the 3rd curtain wall upstream water level=68.5. downstream water level=62.0.

CENTRAL SLUICES OF THE MAHANADI ANICUT AT JOBRA (Orissa)



hypercritical flow. 1.6' bafile on 3rd dwarf wall Secondary standing wave. 3' bafile.

Primary standing wave.

5029. Photo showing conditions with a 3' bafile on first curtain wall and 1.6' one on third curtain wall.

> upstream water |evel = 68.5downstream water |evel = 62.0



1.6' Baffle on 2nd Dwarf wall • 3' 1st Dwarf wall

3rd standing wave (feeble) 2nd standing wave. 1st standing wave

> 5030. Photo showing conditions with a 3' baffle on 1st curtain wall and 1.6' baffle on 2nd curtain wall. upstream water level = 68.5 downstream water level = 62.0

CENTRAL SLUICES OF THE MAHANADI ANICUT AT JOBRA (Orissa)



1.6' Baffle on 3rd D. W.
2' Baffle on 2nd D. W.

3' Baffle on 1st D. W.

5027. Photo showing conditions with 3' baffle on 1st curtain wall, 2' baffle on 2nd curtain wall and 1.6' baffle on 3rd curtain wall and upstream water level = 68.5 downstream water level = 62.0.

Feeble 4th S.W. 3rd S.W.

2nd S.W. Primary S.W.

PART II

With the modifications proposed, it should be possible to maintain the required retention level, with the gates of the Central Shices open, throughout the active irrigation season, regulation being done down to river discharges of 17,000 cusees at the Shore Shices. This should obviate the difficulty previously experienced that, once the Central Shices were opened (i. e. down) before a flood, they could not be raised during the season, so that in case of mid-season fluctuations of river levels the pond could not be maintained.

It may be contended that by these proposals the "scouring" capacity of these Central Sluices to keep open a deep channel would be lost. The real scouring sluices are the Shore Sluices, adjacent to the Canal where—rather than in the centre of the river—it should be desirable to concentrate the deep channel. Also, the canal is not perennial and has to be given supplies only till November, by when the river discharge does not fall below the proposed carrying capacity of the Central Sluices. Moreover, on account of the relatively high coefficient, whatever sand comes towards the Central Sluices will be flushed out without difficulty. The local officers do not, therefore, anticipate any difficulties consequent on the reduction of the Central River Channel from 40,000-50,000to 17,000 cusees capacity, particularly in view of the counter-advantage of regulation at the Shore Sluices, which are more convenient to operate.

APPENDIX I

Showing comparative levels and extent of standing wave under low and high floods

(Existing conditions)

		Low f	ioods.	High floo	ds.
Estimated discharge (cusecs)		50,0	000	6,02,00	0
Upstream water level		68	3∙5	76.0	
Velocity of approach (V _s) (from C. S. No. 16).		Negl	igible	7.0	
$h_a = V_a^2/50$				1.0	
Upstream T. E. L.	•••	68	3.2	77.0	
Coefficient of discharge (C_D)	•••	e (clear o	3·0 overfall)	2.0	(drowned)
	1	Low floods.	. E	igh flood	s.
		Central.	Shore.	Central	Weir . Crest.
Crest levels		58.5	58.5	58·5	65.0
H (=U/s T. ECrest level)		10.0	18.5	18.5	12.0
$q = C_{P'} H^{3/2}$		95	· 160	160	85
Width of crest, ft.	•••	474	345	474	5510
Q (flood discharge), cusecs	••••	45,000	55,200	75,840	4,70,000
Total	l			6,01,000	cusecs.
Downstream water level		61.0	74 [.] 0		• · · ·
	(from	App. II)	(Max.]	H.F.L.d	l/s of 1872)
Velocity of recess (V _r) (from C. S. 17).		2.75	6.0		
$h_r = V_r^2 / 50$	•••	0.15	0.72		
Downstream T. E. L.	•••	61.15	74.7		
$\therefore H_{L} (= U/s T. E. L D/s$ T. E. L.)	•••	7.35	2.3		Weir
2. 2. 2. 2.		Central.	Shore.	Central	. Crest.
q (as above)	•••	95.0	160	160	85
∴ Ef₂ (from C. B. I. Paper No. 12).	•••	13.7	16.4	16.4	11.2
$\mathrm{Ef}_{1} (= \mathrm{Ef}_{2} + \mathrm{H}_{\mathrm{L}})$	•••	21.1	18.7	18.7	13.5
.:. From C. B. I. Paper N	o. 12				-
Plate XI.1, D ₂		12.8	14.3	14.3	10.0
Do. D_1	•••	2.8	5.7	5.7	3.3
$\therefore \text{ Level at which S. W.} = (=D/s W. LD_2).$	forms	48.2	59.7	59•7	64·0
Length of disturbance of star wave=5 (D_2-D_1) . (ft.)	nding	50	43	43	34
Hypercritical velocity $V_1 = q/D_1$ (ft/sec).		` 34	28	28	26

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APPENDIX II

(Extracted from Gauge Register of the Jobra Weir)

Showing the maximum upstream and the corresponding minimum downstream river levels when the Central Sluices were fully open and all the Weir Crest Shutters and Shore Sluices were shut

.

	Dates of o Central	peration of Sluices.	Maximum upstroam an downstroam riv	Damaster		
Year.	Date of opening.	Date of closing.	Period or dates.	Maximum u/s levol,	Minimum d/s lovel.	Romarks.
1945 "	9-9-45	12-10-45	${29-9-45 \\ 8-10\cdot45}$	68 ·7 67·95	66 [.] 0 61 [.] 0	
1945	24-7-44	20-10-44	{ 12-10 to 19-10 5-9 to 24-9	68·2 68·5	$61^{1}4$ $62^{1}0$	
1943	13-7-43	11-10-43	5-10 to 9-10 9-9 and 10-9 19-8	68·0 68·6 58·6	$\begin{array}{c} 62.0 \\ 61.5 \\ 62.2 \end{array}$	Some Shore Sluices open.
1942	6-7-42	11-10-42	{5-10 to 8-10 {25-7 and 26-7	$68.3 \\ 68.4$	60 [.] 8 61 [.] 8	
1941	12-7-41	31-8-41	{ 18-7 to 31-7 1-8 to 8-8 Do.	68-0 68-6 67-2	$\left. \begin{array}{c} 61^{\cdot}4 \\ 61^{\cdot}9 \\ 60^{\cdot}8 \end{array} \right\}$	
1940	7-7-40	18-9-40	{11-9 to 16-9 {11-8 to 15-8	67°6 68°5	61·2 63·0	Some Sh ore Sluices open,
1939 ,	16-7-39	6-10-39	(1-10 to 3-10) and 5-10) 25-9 to 30-9 Do. 1-8 to 18-8 Do.	$ \begin{array}{c} 67.9\\ \begin{cases} 67.4\\ 67.8\\ 69.1\\ 67.1 \end{array} $	60·9 61·3 62·0 64·7 60·5	
1938	15-8-38	1-10-38	27-9 to 29-9	67.6	61-9	
1937	3-8-37	3-10-37	$\begin{cases} 1-10-37\\ 1-9 \text{ to } 5-9\\ 21-9 \text{ to } 30-9\\ 21-8 \text{ to } 24-8 \end{cases}$	66·7 67·7 67·9 68·9	62·2 63·6 63·2 64·4	
1936	27-6-36	29 -9- 36	$ \begin{cases} 21-9 \text{ to } 24-9 \\ 16-7 \text{ to } 19-7 \end{cases} $	67 [.] 6 68 [.] 6	$\begin{array}{c} 61.3 \\ 62.2 \end{array}$	Do.
1935	11- 7- 35	16-8-35	$\begin{cases} 14.8 \\ 17.7 \text{ to } 19.7 \end{cases}$	67 2 67 7	60 [.] 9 61 2	7 Shore Shices open. All closed.
1934	3-7-34	12-10-34	$\begin{cases} 10-10-34 \\ 1-8 \text{ to } 6-8 \\ 18-7 \text{ to } 31-7 \\ \text{Do.} \end{cases}$	$\begin{cases} 67.7 \\ 68.5 \\ 68.55 \\ 67.0 \end{cases}$	$\begin{array}{c} 62.8 \\ 62.6 \\ 62.5 \\ 61.5 \end{array}$	
1933	18-7-33	9-9-33	{ 5-9 and 6-9 20-8 to 25-8 29-7	66·4 67·0 68·6	Not observed. 62·8 61·7	7 Shore Sluices open.
1932	2 4-7-32	17-9-32	{13-9 to 16-9 1-9 to 6-9 25-8 to 31-8	67·8 68·5 68·3	Not observed. Do. Do.	
1931	21-8-31	18-9-31	14-9 to 16-9	67.0	Do.	·

	Dates of c Central	peration of Sluices.	Maximum upstream a downstream r	nd corresponditiver levels in p	ing minimum eriod.	
Year.	Date of opening.	Date of closing.	Period or dates,	Maximum u/s level.	Minimum d/s level.	Remarks.
1930	4-7-30	12-10-30	(1-10 to 2-10 3-10 to 10-10 8-9 to 10-9 14-9 to 22-9 18-5 to 21-8 13-7 to 17-7	$ \begin{array}{c} 68.0\\ 67.5\\ 68.3\\ 68.8\\ 67.5\\ 68.4 \end{array} $	61.0 61.4 62.0 62.1 61.9 62.1	
1929	19-7-29	1-10-29	20-9 to 29-9	67.4	61.5	
1928	- 22-6-28	27-8-28	{12-7 to 18-7 {8-8 to 13-8	68 ⁻⁶ 68 ⁻ 4	62·8 62·1	Gates mani- pulated twice
,,	15-8-28	6-11-28	{17-9 to 28-9 20-10 to 31-10 1-9 to 3-9	68·3 67·9 66·9	$\begin{array}{c} 61 \cdot 2 \\ 62 \cdot 8 \\ 61 \cdot 3 \end{array}$	in this year.
1927	17-7-27	25-9-27	$\begin{cases} 21-9 \text{ to } 23-9 \\ 8-8 \text{ to } 9-8 \\ 22-7 \end{cases}$	66·8 68·0 68·3	Sand 60 9 61 7	
		Note.—	Worst gauges	68 [.] 5	61.0	

APPENDIX II-(contd.)

APPENDIX III

Showing calculations for the Standing Wave, Baffle Pavement, etc., in the proposed design (Crest level 63.5; upstream glacis 1 in 3; downstream glacis 1 in 4)

Central Sluices open, Crest and Shore Sluices shut

U/s T. E. L. = U/s W. L. ($h_a = negligible$)		68.5 R.L.
$C_{D} = Coefficient of discharge$		3 ·2
Crest level	•••	63.5 R.L.
\therefore D=upstream depth below upstream T. E. L.	•••	5.0 ft.
$q = C_{\rm D} \cdot D^{3/2} $	•••	36 cs/ft.
Q = 474.5 q		17,000 cusecs.
D/s water level (effected by baffle of height 4 ft.)		62.5
$h_r = V_r^3 / 50$ (from C.S. No. 17)	•••	0.1
D/s T.E.L.		62.6
\therefore H _L = u/s T. E. L D/s T. E. L.		5·9 ft.
From Bombay T.P. No. 44, d _x =depth d/s of S.W.	over	
baffle pavement level		6·8 ft.
∴ Baffle pavement level (required)		55.8
	against	58.5 available
(but to lower the never and the this start in 1)		

(but to lower the pavement at this stage is said to be *not* practicable). Height of baffle, here, guided by required d/s level = 4 ft. Trength of haffle pavement below too = $5.05 \times d$

Dengen of dame pavement	below toe $= 5$	or25 -	×	d ₂			
if at R. L, 56.0	= 5	5.25	x	1.4	(from	Bombay	ΤP
					No.	44).	1.1.

```
= 7.5 ft.
```

:. length of baffle pavement at R. L. $58 \cdot 5 = 7 \cdot 5 + (58 \cdot 5 - 56) 4 = 17 \cdot 5$ ft. (N.B.—Thus, if desired, the baffle pavement can be lowered at any time with advantage to R.L 56 \cdot 5, keeping the 1 in 4 glacis continuous).

PART 11

APPENDIX III-(a)

Height and length of primary wave (according to Bombay T. P. No. 44, p. 58 et seq.)

$$h/d_2 = 2.5 (h_2/D)^{1.18}$$

$$\therefore h = 1.4 \times 2.5 \left(\frac{68.5 - 58.5 - 1.4}{5}\right)^{1.18} = 3.5 (1.72)^{1.18}$$
$$= 3.5 \times 1.9 = 6.65 \text{ ft.}$$

... Divide walls should be raised to 58.5 + 6.65 = 65.2, say 66.0 R. L., for a length L given by

$$L/D = 6.0 \ h_2/D$$
^{1.18} = 6.0 (1.72)^{1.18} = 60 × 1.9 = 11.4, say 12 ft.

or 10 ft. on either side of bafile.

II-(ii) 3. MINIMUM WATERWAY REQUIRED FOR THE SAGI, KHARA AND SUKRI BRIDGES—JODHPUR—CASES OF ADVICE WITHOUT MODEL EXPERIMENTS

The Jodhpur Railway Authorities sought advice as regards protection works for bridges over the Sagi, Khara and Sukri rivers. The data supplied did not give the information required for a considered opinion.

From such data as was available, it was evident that the present as well as the proposed waterway of the skew bridges across the Sagi and the Khara were insufficient to pass the probable maximum flood without damage at some future date, particularly in the absence of training works.

In the case of the Sagi, it was found advisable to recommend 5 more spans on the left bank in addition to 4 already proposed by the local authorities in order to pass a flood of the order of 1,00,000 cusecs.

In the case of the Khara river, it was even more difficult to assess the maximum discharge and hence the minimum required waterway; and the best measure in the interest of safety was considered to be to span the river from bank to bank, i.e. a distance of 400 ft. as against 281' proposed by the local officers. As the bed was, as indications showed, hard material which will not allow deepening of the bed during floods, adequate waterway was all the more necessary.

In the case of the Sukri River Bridge, to pass the estimated floed of the order of 1.0 to 1.5 lakhs cusecs it was considered necessary to extend by 12 spans of 40 ft. or preferably by 5 spans of 100 ft.—in any case the end span should be 1.0 ft.; thus making the total waterway of about 1,157 ft. as against 1,032 ft. required according to the Lacey formula. This increase of waterway was necessary, firstly because the Jodhpur rivers are flashy and they widen rather than deepen—having a high width : depth ratio—and secondly because the existing foundations are very shallow.

In addition to widening, it was also considered necessary to recommend in this case, a correctly designed guide bank on the right bank.

II-(*ii*)-4. Design of Vent Approaches and Exits of Road Culverts (Advice without Experiments)

The Consulting Engineer to the Government of India (Roads) sought advice of this Station in regard to the best design of approaches and exits of masonry culverts.

In respect of the approaches, it was recommended that, from the point of view of minimum loss of head, a bellmouth upstream and a 1 in 10 divergence downstream would be sound practice. A bellmouth upstream, however, increases constructional difficulties arising from the effects of surcharge from the earthen bank, and increases the length and hence cost of the wing walls; and so a splay

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of 1 in 5 may be adopted instead. The splay may be steepened to 1 in 3 for the smaller culverts where this convergence is not too sharp for the water

For exits, it was suggested that wing walls with hyperbolic expansions, to follow. together with a baffle wall and a deflector, are generally the best, but where economy is a major consideration, straight divergences of 1 in 10 with a baffle and a deflector need only be provided. Without model experimentation to evolve the required measures, steeper divergences would be unwise.

Where severe action, bed scour and bellying are anticipated below the culvert, a sloping glacis at 1 in 5 was recommended up to the optimum floor level necessary to ensure the formation of a standing wave for dissipation of excess energy. The flow can further be stabilised by a baffle and a deflector, if so required in specific cases.

If the necessary data are supplied, complete design would be furnished by this Station for any case.

II. RIVER BEHAVIOUR AND CONTROL

(iv) TIDAL ACTION INCLUDING ESTUARIES.

II-(iv) 1. Measures adopted to reduce turbulence in the Hooghly

RIVER DOWNSTREAM OF THE KNUCKLE AT THE TITAGHAR

JUTE MILL (BENGAL)

1. Synopsis-

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The note describes experiments carried out with 1/300: 1/60 scale model and 1/150: 1/80 part model of the Hooghly .--

(a) to investigate the present conditions 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⁽¹⁾ 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.

2. 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 cs. (\equiv 1,87,000 cs. observed in 1941) was run with W. L. at Konnogar \equiv 13.5 ft. above K. O. D. S. (Kidderpore old Dock Sill as datum).

A horizontal scale of 1/150 was chosen for the large, part model, so as to represent 1.8 miles u/s of Titaghar Jute Mill and 2.1 miles downstream. When this model was run under identical conditions of contours, discharge and W. L., as in 1/300 scale model, with a depth scale of 1/100, (i.e. V. E. = 1.5), and

$$Q = 1.25 \text{ cs.} \left(\equiv \frac{1.87,000}{LD \sqrt{D}} = \frac{1.87,000}{150 \times 100 \sqrt{100}} \right)$$
, heavy scour occurred. So dam sand

Fig. 1(a) was replaced by Londba sand (m = 0.30 mm.); the bed movement was very low. and 1 ^(b) Hence the depth scale was altered to 1/80 (V. E. = 1.8), and Q = 1.7 cs. Figs. 2 ^(a) (\equiv 1,87,000 cs.) and the movement of bed material improved. The lines of flow and and 2 ^(b) isotachs are shown in figs. 1(a) and 1(b)/II-iv respectively. A wooden grid was 11-iv. the unstream end of the model to improve the distribution of the distribution of the model to improve the distribution of the model to improve the distribution of the distribution of the model to improve the distribution of the distributic 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.







PART I-EXPERIMENTS WITH PILES IN 1/150: 1/80 MODEL

3. 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 2 cft/hr. (i.e. 0.4 parts per 1,000).

Duration of experiment was 24 hours = 16 days in all experiments.

4. Experiments.

Experiment no. 1.-In this experiment the L (longitudinal) row of piles was constructed as shown by position 1 in Fig. 3/II. iv. The alignment of cross-row Fig. 8/11-iv. of piles was arbitrarily fixed keeping the top 4 ft. below K.O.D.S. allowing thereby 10 ft. draft at low water. The longitudinal piles were constructed with the top level 32 ft. below K.O.D.S.

Observations:-

(a) Lines of flow are shown in Fig. 4/II.iv. As compared with the Fig. 4/II.iv. 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 Fig. 5 (b)/II.iv. When compared with Fig. 5 (a) & $\binom{b}{11-4v}$. the previous experiment without piles and under similar conditions (fig. 5 (a)/[1]. iv], it was seen that the 50 ft. contour extended towards the right bank showing thereby the tendency to scour on the right side of the knuckle. Scour also occurred d/s of the knuckle at C. S. 5, 4A, etc.

(c) Soundings were taken at C.S. 5B to see the amount of silting Fig. 6/11-1v. 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 9" 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 = 30 ft. 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. 5B were only taken. These showed Fig. 7/11-1v. 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 Fig.3/II.iv.

Observations :---

(a) The surface lines of flow downstream of the knuckle shifted away Fig. 8/11-iv. from the knuckle.

(b) Bed contours show that there was scour between C.S. 5A and 5 Fig. g/11-iv. towards the left Bank: 60 ft. contour which was not obtained in the previous experiment was obtained in this experiment.

(c) Soundings at cross section 5B show that there was no tendency to Fig. 10/11-iv. Slight scour was, however, observed. silt.

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, Fig. 3/II.iv.

Observations:-

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Fig. 11/II-iv.

Fig. 12/II-iv.

(a) Lines of flow showed a concentration of flow at C.S. 5, which might result in a formation of knuckle at that place.

(b) Bed contours showed that there was scour within the bight and outside the L. pile—to the right of L. piles. 60 ft. contour was obtained in the bight.

Fig. 13/11-iv. Sounding at C.S. 5B confirmed the scour in the bight.

Experiment No. 2A.

Fig. 14/II-iv.

As the scour was observed outside the L. piles, an additional row of four piles, with the tops 8 ft. below the tops of L. piles, was constructed. Fig. 14/II.iv 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.

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.

Fig 16/11 iv. Fig. 15/II.iv shows sounding at C.S. 5B; silting was not, however, satisfactory.

5. 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 1 to 15 ft. 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

6. 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 18 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 $\equiv 300$ ft. with top 25 ft. above K.O.D.S., i.e. 4 ft. above H.W.L.; width $\equiv 112.5$ ft. (semicircular 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 by the Station as being unsuitable for training flashy rivers such as the Luni near Samdari (Jodhpur Railway)" where the high formative discharges occur for too short a duration to allow the island to leave its impress on river configuration, this Station considers that

(1) I.W.E. Stn., Annual Report (Tech.) for 1946-para. II. vi. 1.

NR 6174 ATLENNYS EXPT. STY POONA . FIG. J.iv 1 : 1 HOOGHLY RIVER MODEL . PLAN SHOWING THE DIFFERENT POSITIONS OF L- PILES TRIED. 5CALE 1 = 66' 24 CONTOUR ALIGNMENT OF SHIFTED CROSS ROW OF PILES. CROSS ROW OF PILES [je П Π Π Π WHUCKLE- PAFfred D D LONGITUDINAL PILES ALIGNMENT NO. 2. Π " NO. I 1, <u>ц</u> 11 1) 1, 110, 3. EBB. FLOOD -By ... R.T. THAKOR . 27-8-46 . G.HT.P.HOONA 47










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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. -

7. 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 23rd September 1941 was re-produced and repeated 10 times in each experiment.

Lines of flow were invariably observed under 3/4 ebb flow conditions, after 10 tides were completed in each experiment.

Water level recorders were fixed at Damurdaha (u/s 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 fig. 16/11-iv. Out of Fig. 16/11-iv 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 Fig. 17/11-iv 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.

Experiment No. 3.—

As the island in experiment 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 water level), it was considered desirable to shift the island towards the left Fig. 18/11 iv bank, where the depths were considerably less. The results were not satisfactory, as the island did not act.

Experiment No. 4.-

In this experiment, the island was shifted upstream so as to be at a distance of $M_i/4$ from the knuckle. The island was not, however, effective in attracting $_{Fig. 10/11 \text{ iv}}$ the flow towards the right bank opposite the knuckle. Lines of flow were observed in this experiment, also under flood condition. The island was then $_{Fig. 20/11 \text{ iv}}$ removed and the lines of flow again observed under flood condition. This $_{Fig. 21/11 \text{ iv}}$ 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 $_{Fig. 22/11-iv}$ right bank at C. S. 8 flowed along the right side of the island B; but further downstream, it swung back towards the knuckle.

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 $_{Fig. 23/11 \text{ iv}}$ the shoal near right bank between C. S. 7 and 5A.

Experiment No. 7.-

In this experiment, a third island called 'C' was constructed as shown in figure 24/II-iv, the object being to get advantage of scouring action during $_{Fig}$ 21/II-iv flood tide. So the semi-circular end of the island was made to face the flood flow.

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Figure 24/II-iv shows the lines of flow under flood condition. All the Figure 24/11-1V shows the three was no indication of scour on the flow was hugging the left bank and there was no indication of scour on the now was nugging one lett tain 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.

Fig. 25/II-iv

The islands were so constructed so as to form a continuous line. This, it une istands were so considered and in a formation of continuous deep channel, attractwas considered, would result in a concentration of the flow on right bank opposite the knuckle and reducing concentration of the flow on right bank opposite the knuckle and reducing concentration. tration 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 a sceeper side side given at the islands C and B) had a tendency to move 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.

Experiment No. 9.-

The position of the island A in experiments nos. 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). Dradging was also done from C.S. 8A to 8-a distance of about 1/3 mile.

Fig. 26/II-iv

Figure 26/II-iv shows the lines of flow after 10 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 10 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.

Fig 27/II-iv

Figure 27/II-iv 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 10 feet below L. W. L. 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 (1/150:1/80); 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 behavior of islands are also under investigation.

II-iv-2. PROTECTING THE FORESHORE OF THE ANGLO INDIA JUTE MILL ON THE HOOGHLY RIVER (BENGAL)

Introduction

Fig. 28/II-iv.

Figure 28/II-iv shows the plan of the Hooghly from 6 miles upstream of Bansbaria to Konnagar. This figure clealy 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

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REFERENCE:~







<u>FIG. 30/II.iv</u> ANGLO INDIA MILL NO. 3. FORESHORE SECTIONS (SURVEYED BY WILLCOX ON 14# MAR. 1945) & HYDRAULIC GRADIENTS AT VARIOUS STAGES. FT. 20 FT.









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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.

The river is practically held on both banks - especially on the outside of curves — where stone revetments have been constructed for bank protection. During the inspection in May 1945 by the then Director, Rao Bahadur Joglekar, with Mr. Kilford, River Surveyor, and Messrs. Ghotankar and Mulekar (members of the Poona Staff), 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 28/II-iv).

Fig. 28/11-iv.

(1) Along the left bank upstream and downstream of Punhatti.

(2) Left bank near the Titaghar Jute Mill. The downstream end of the drain upstream 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 Nawabgunj.

(5) Erosion near Dunbar Cotton Mill.

(6) Erosion along the right bank between Gondalpara and Chandranagore. New embayment had formed along the right bank opposite Anglo India 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 to the Station 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 und described in the 1944 Annual Report (Tech.). Those in connection with the Titaghar Jute Mill are in progress. An 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 21-2-1942. The collapse of the bank on 3rd and 4th March 1944 was very severe, being more than that which occurred on 21st February 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. Mr. Kilford 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. (Kidderpore Old Dock Sill). The least water level determined from simultaneous tidal curves observed on 25th March 1940 was 1'-9" above local zero. This gave the maximum head due to subsoil water equal to 24'-3" and a saturation gradient of about 1:4. During 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 29/II-iv shows the foreshore of Anglo India Jute Mill and the Fig. 29/II-iv. line of collapse. Figure 30/II-iv shows cross sections.

Recommendations

The existing material, namely, ashes, clinkers, etc., should be removed as far as possible to a depth of 2 to 3 feet and new banks in sand sloping in 1 in 3 should be made as shown in Figure 30/II-iv in red. This should be covered with 2' thick " one man stone " (weighing 70 to 90 lbs). This should be followed by sloping stone apron 4' 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 be laid at the lowert W. L. It is also recommended that below the bottom of 2' 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.

II. RIVER BEHAVIOUR AND CONTROL

(V) FLOOD CONTROL

II-v-1. REDISTRIBUTION OF SAND LOAD BETWEEN THE MAHANADI AND KATJURI SYSTEM AT CUTTACK (ORISSA)

1. Introduction

About 12 miles above Cuttack, the Mahanadi with its peak flood of 15 lakhs cusecs passes through the gorge at Goil Bank to flow through a wide stretch, upstream of the second gorge at Naraj about 5 miles above Cuttack. 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 2 miles long. At Naraj, 56 per cent (8:4 lakhs cusecs at peak flood) of the high river discharges 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 finds 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 and Katjuri loop.

2. The Model

The model reproduced a reach of the river from Goil Bank down to 2 miles below the Jobra Weir on the North of Cuttack and 2 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 2 miles to 5 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.	= 6.06
B. S. E. = S. E.	= 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

Fig, 1/II-v.



small or two 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.

3. Experiments

The experiments reported in the Annual Report (Tech.) 1945 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.

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. With Birnasi gap full open and Banki Road spill discharge Experiment 2: reduced to 50,000 cusecs. With Birnasi gap full open and Banki Road spill discharge reduced to 33,000 cusecs. Experiment 3: With Birnasi gap closed by an embankment ending in a Experiment 4: spur⁽¹⁾ and Banki spill full open. Same as Experiment 4, but Banki Spill discharge reduced Experiment 5: 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 lakks 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.

3. 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

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^{1.} Indian Waterways Experiment Station, Annual Report (Tech.) 1945.

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back to 6 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.

4. Discussion of Results

Figs. 2, 3 and (i) Figures 2, 3 and 4/II-v show the water levels observed in the model at the Kazipatana, Collectorate and Bellevue gauges of the Katjuri in the experiments of Series I.

It will be seen that with Birnasi spill shut off completely and the Banki 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 2 feet which increased to 3 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 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.

Fig. 5/II-v

Fig. 6/II-v

Figure 5/II. v shows the velocities observed in the model, with peak flood along a section parallel to and $\equiv 200$ ft. upstream of the Naraj weir. Computing from this the discharge that flowed through the reduced portion of the Naraj weir, i.e. 35 %⁽¹⁾ with both spills completely closed, 24.8 % in experiment 3 (Series II), and 15.5 % 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% reduction of Naraj weir waterway.

0.982	do.	24.8	do.	do.
0.523	do.	15.5	do.	do.

TABLE 1	/II-v
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Showing the discharges of the Mahanadi below Naraj and below Birnasi gap in lakhs of cusecs.

	Total above Naraj.	Due to re- duction of Naraj weir.	Mahanadi below Naraj.	Birnasi gap.	Mahanadi below Birnasi gap.
According to Rhind (existing conditions).	15.00	Nil.	6*58	C·64	7.22
Series II, Expt, 1, Birnasi gap open, Banki Road spill Q= 50,000 cusecs and Naraj Weir length reduced by 15.5 per cent.	15.00	0.23	7.103	0 [.] 64	7.743
Series II, Expt. 3, Birnasi gap closed, Banki Road spill Q=50,000 cs. Naraj weir length reduced by 24.8 per cent.	15·00	0.985	7-565	Nil.	7-565

(1) Indian Waterways Expt. Station Annual Report (Tech.) 1945.











Assuming that the Mahanadi and the Berupa share the discharge below the Birnasi gap in the same ratio as under existing conditions, viz. 61 lakhs cusees in Mahanadi and 1.12 lakhs cusecs in Berupa, 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.

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

> Length of weir = $6373 \cdot 5$ ft. Maximum W. L. =75.0. Sill R. L. =64.27. h_v or head due to velocity of approach = $(7.4/50)^3 = 1.10$ ft. $D = D_1 + h_v = 10.73 + 1.10 = 11.83$ ft. Q over weir = 6.10 lakhs cusecs. $\therefore C = \frac{6.1 \times 10^5}{6373.5 \times (11.83)^{1.5}} = 2.35.$

Hence, to pass 6.62 and 6.56 lakhs cusecs, the head required will be 12.49 ft. and 12:30 ft. respectively. Thus, the afflux in experiment 1, would be 12:49 -11.83 = 0.66 ft. and in experiment 3 it would be 12.30 - 11.83 = 0.47 ft.

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 ft. in experiment 1 of Series II and 0.806 ft. 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 ft. and 0.50 ft. 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 Table 2/11-v with the existing conditions as shown in table 2/II-v.

TABLE 2/II-V

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

Experiment. No. Experiment. Series III.		Reduction in Narai	Rise in water level.			
	weir waterway.	Model.	Prototype. (equivalent rise.)			
1	Birnasi gap open and Banki Q=50,000 cs.	15.5%	·021 ft.	1·38 ft.		
2	Birnasi gap closed and Bank. Q = 50,000 cs.	24.8%	[.] 029 ft.	1-91 ft. `		

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 lakhs.

B, i.e. length of weir=3,900 ft. Sill R. L. = 72.6

W. L. with peak flood=91.0

 h_v i.e. velocity of approach = $\frac{(10.5)^s}{50} = 2.21$ ft.

 $\therefore D = D_1 + h_v = (91 - 72.6) + 2.21 = 20.61$ ft.

 $\therefore C = Q/BD^{10} = 2.30$

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When the Naraj waterway would be reduced by 24.8% or 15.5%, the discharge in the Katjuri would be reduced by '985 lakhs cusecs or '523 lakhs cusecs, respectively; with C=2.30 the head required to pass the remainder from Table ?/I'-v the 8.4 lakhs cusecs will be as in table 3/II-v.

Reduction in water- way at Naraj.	Q passing into the Katjuri.	$D = \left(\frac{Q}{2\cdot 3 \times B}\right)^{2/3} \text{ (ft.).}$	Increase in afflux (ft.).
Nil.	8.4 lakhs cs.	20 [.] 61	-
15.5%	7.877 " "	22.10	1.49
24.8%	7.415 " "	22·90	2 29

TABLE	3,	/II.	V
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The model rise was slightly less (see Table 2/II. (V). 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.



PART II

					the second s						_		
Details of experiments.	Total quant- ity of sand	Mahanadi.		Katjuri loop.		Surua.			Kuakhai.				
Details of experimental	all branches.	ΣQT/T.	Q ₆ .	x	Σ QT/T.	Q _{s.}	x	ΣQT/T.	Qa.	x	ΣQT/T.	Q	x
1	2	8	4	5	6	7	8	9	10	11	12	18	14
Series I—With unrestricted waterway of Naraj Weir.											_		
$ \begin{array}{llllllllllllllllllllllllllllllllllll$	(a) 14.000 (b) ,,	1·390 "	6.000 "	0·820 "	0.769 "	3.000 "	0·740 "	1-826 H	4.000 "	0+498 "	0-516 "	1.000 "	0-370 "
Experiment 2 with Biranasi 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	0.715 	9.250 9.166	0.796 0.775	1-588 	4 - 125 4 - 010	0+510 0+496	0-510 	1+000 0+974	0+878 0+863
Experiment 3 with Biranası 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 C.808	0.772 	8 • 250 8 • 230	0-797 0-791	1-5:30 	3-500 8-470	0-485 0-493	0.510 	1+850 1+340	0-803 0-495
Experiment 4 with Biranasi gap Q nil. Banki spill Q=1,00,000 cs. (at peak flood).	(a) 14.500 (b) 14.000	1 · 360 	6-00 0 5-795	0.840 0.811	0.770 	8+500 8+350	0-865 0-880	1.550	4-000 8-860	0-490 0-473	0-505 	1+000 +965	0-878 0-860
Experiment 5 with Biranssi gap Q nil. Banki spill Q=50,000 cusecs (at peak flood).	(a) 15.125 (b) 14.000	1·370 	6 000 5 550	0-838 0-718	0.800 	3.7≛0 3.470	0-890 0-528	1.620 	4 - 250 8 - 930	0+498 0+460	0-A98 	1 125 1-040	0-403 0-972
Experiment 6 with Biranasi gap Q nil. Banki spill Q=33,000 cs. (at peak flood).	(a) 16.000 (b) 14.000	1·280 	6.000 5.250	Q∙826 Q•722	0.820 	4.000 8.500	0-930 0-814	1.630 	4.500 8.940	0-525 0-459	0.544	1+500 1+310	0+694 0+459
Experiment 7 with Biranas gap Q nil. Banki spill Q = nil (at peak flood).	(a) 16.75 (b) 14.000	1 · 390 	6-250 5-220	0.855 0.714	0-830 	4.500 3.760	1.030 0.801	1.650 	4-375 8-660	0-508	0.550 	1+625 1+858	0-860 0-468
Series II—With Naraj Weir Waterway reduced.													
Experiment 1 with Biranas ⁱ «ap 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	0·750 	8.000 2.800	0.760 0.710	1 · 420 	4.000 8.720	0-585 0-499	0.450	1-000 -934	0+423 0+394
Experiment 2 with Biranasi 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	0.770 	3 250 2 720	0-801 0-670	1 · 320 	4.500 3.720	0+848 0-541	0.490 	1+500 1+255	0-580 0-485
Experiment 3 with Biranasi gap closed, Banki spill Q=50,000 cs. Naraj weit waterway reduced by 24.8 per cent.	(a) 14.50 (b) 14.000	1·540 	7•500 7•140	0.925 0.893	0·785 	8.000 2.900	0-775 0-749	1.450 	9.000 2.900	0-898 0-879	0 - 470	1 - 000 0 - 065	0+405 0+891
Experiment 4 with Biranasi gap closed, Banki spill Q=38,000 cs., Naraj weir waterway reduced by 28.5 per cent.	(a) 17.25 (b) 14.000	1.6CO 	8.(00 6.500	0.950 0.771	0·720 	3-500 2-840	0+923 0+750	1.410	4-500 8-65	0-607 0-493	0-440 	1 · 250 1 · 010	0-540 0-435

Table 4/II-(v) showing the ratio of the sand drawn by the Mahanadi and its branches to the time-average discharge

Notes:-(1) '(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 collected equal to sand injected.
- (2) $Q = discharge in cusecs; Q_s = quantity of sand collected in cft.; T = time in minutes;$ $X = charge in parts per 1000 (by weight) = Ratio of Qs to <math>\sum QT/T$, where $\sum QT/T$ is the weighted mean discharge with respect to time, computed by taking the sumr of the products of each stage discharge and its time of duration and dividing by the total time of the flood cycle.

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

Table 4/II-v.

Table 4/II-v. 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 4/II-v are the quantities actually collected in the pits and the corresponding sand 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 cft.) as in Expt. 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 Fig. 7/11-v. 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 4/II-v, 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 69 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 $4\frac{1}{2}$ ft. 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 cft. as observed (and 8.25 cft. as corrected) which gradually increased to 10.5 cft. as observed (and 8.78 cft. as corrected) in experiment 7.

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" offtake.

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 Fig. s/II-v. conditions.

Comparing corrected relative sand charges of experiment 1 (Series II) with experiment 1 (Series I), it will be seen that whereas in expt. 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.

⁽¹⁾ Indian Waterways Expt. Stn. Annual Report (Tech.), 1945.





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PART II

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 248 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, 57 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 9 and 10/II-v show the lines of flow with 6 lakhs cusecs flood in Figure 9 and 10/11.v 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 Kazipatana, the surface flow was towards Cuttack side, i. e. left bank, whereas the bed flow was towards the Kazipatana 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.

(vi) 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 experi-mentation 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 distributions are not significantly effected.

(ii) An analysis of table 4/II-v. would appear to show experiment 1 of Table 4/II-v. 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.

year, whenous causing any account of increasing the carrying capacity of the (*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 4/II-v).

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⁽¹⁾ in a limited length. Generally, however, the conditions in the Katjuri system may be expected to improve.

II. RIVER BEHAVIOUR AND CONTROL, (VI) LOCAL TRAINING

II/vI-1. TRAINING THE LUNI RIVER AT RAILWAY BRIDGE NO. 3 OF THE SAMDARI-RANIWARA BRANCH OF THE JODHPUR RAILWAY. EXPERIMENTS IN CONNECTION WITH THE---

(1) Introduction:-

Preliminary experiments were described in the last year's Annual Report Fig. 1/II.vi. (Technical) of the Station. The Luni river debouches from hills near Ajmer, and collecting water on the way from various tributaries, flows westwards till it reaches Gole, where it turns southwards, and flows roughly straight till it meets the Arabian Sea.

> The average width and depth of the river near the Railway bridge, in flood, are 3,800 ft. and 8 ft. respectively, and its slope is 1 in 1,500. The bridge is 1,295 ft. in length and has a clear waterway of 1,160 ft. with 29 piers; while before the construction of the bridge, in 1927, the river width at the bridge site was 3,100 ft. 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, as explained in last year's Annual Report (Technical)].

When the bridge was constructed, the high bank line was continuous up Fig. 2/11-vi. to A, shown in Fig. 2/11-vi; 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 ft.

(1) Indian Waterways Experiment Station Annual Report (Technical, for 1942-43, page 51.





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.

(2) Experiments:

Preliminary experiments described in last year's Annual Report (Technical) were carried out to reproduce the existing conditions. These experiments were continued during the year to obtain optimum flow conditions at the Railway bridge.

The Series of experiments have been sub-divided into Parts I, II and III. Part III experiments were necessitated by a discrepancy in the original survey plan of the river, which came to notice only at about the end of the experiments in Part I, on receipt of the air photographs. The labour over the experiments was, however, not totally wasted since they provided information regarding the inadequacy of artificial, pitched "attractive" islands in training flashy rivers like the Luni, though such islands are recommended by the station" under more favourable circumstances. Experiments of Part I have, therefore, been summarised below :--

Part I: Experiments.

(1) Preliminary experiment with existing conditions.

Fig. 8 (a)/11

(2) Waterway of the bridge extended by $2.5' \equiv 500'$.

(3) Waterway extended and with two islands, side by side, positioned on Fig. 8 (6)/11the existing natural shoals.

(4) Same as experiment 3, but with T headed groyne cut back by 2' = 400'.

(5) Same as experiment 4, but with one, left, island only. (Groyne cut back by 400').

(6) Same as experiment 4, but with two islands in a line. (Groyne cut Fig. Medit.vi back by 400').

(7) Waterway extended, head of groyne cut back and with island below FIR. 3(1)/11.v the bridge.

(8) Waterway extended, head of groyne cut back and with a straight guide Fig. 3(0)/11-vi bank on left side.

(3) Analyses of the experiments-

The object of all the experiments was to reduce the afflux and to centralise and distribute the flow evenly in all the spans, thus avoiding concentration and consequential local scour at the flanks.

The conditions obtained in the various experiments are briefly set ou below (Scales, 1/200: 1/40; V. E. = 5) :--

Results of the Experiment No. (1) have been given in the Annual Report (Tech.) 1945.

Experiment No. 2-

With increased waterway, the maximum velocity through the bridge wig 4(a)/11-v1 dropped from 8 ft./sec. to 6.5 ft./sec. Concentration at the flanks still, however, persisted; but the bed did not scour anywhere to a level below the present foundations; the deepest scour-R. L. 421.40-occurred at the left abutment due to sharply curved flow there.

έ.,

The afflux for the maximum discharge of 250,000 cs. was : 064 ft = 2:56 ft.

Eddies and return flow formed at the T head of the groyne and in the area between the left abutment and the guide bund. No appreciable scour was, however, noticed along the guide bank except at the T head, as in experiment no. 1.

Experiments Nos. 3 to 7-

It was expected that an island or islands, suitably positioned, would attract the flow and control its distribution through the bridge. positions of islands were, therefore, tried in experiments nos. 3 to 7. It was,

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however, observed that islands cannot be fully effective in a flashy river, such as the Luni, because of the very short duration of the high floods which cannot bring the islands into full action.

Experiment No. 3-

Fig. 5/11-vi

Two islands were built on the existing natural shoals, about 3,000 ft. upstream of the bridge. These improved the flow conditions at the bridge by slightly reducing the side concentration and by shifting the high velocity flow a little towards the centre spans. There was also a tendency to form a shoal in the centre; this tendency will be greater in the prototype as accretion occurs relatively more slowly in the model.

At the section across the islands, the right channel deteriorated, while the left one developed and carried about half the total discharge.

The islands had practically no effect on the return flow and scour at the T-head of the groyne.

Experiment No. 4-

The T-headed groyne was cut back by 2'=400'. This checked the development of the left channel and the main flow was "captured" in between the artificial islands at the section across the islands.

A big shoal, however, formed below the left island which extended up to Fig. 4(b)/11-vi and beyond the bridge.

Experiment No. 5 -

The effect of one island-the more central left island-was examined in this experiment with a view to divide the river stream into two channels on either side of it, and prevent a central slack channel leading to shoaling. With the T-headed groyne cut back, the attracting effect of the rocky, right bank pulling the river towards the right side would also be counteracted by an island, set slightly towards the left; this was another reason for positioning the island slightly on the left side.

The island over the left shoal successfully attracted the flow and kept it central till section 6-1,550 ft. upstream-below which onis effect was, however, Fig. 1(e)/II vi not perceptible. A shoal similar to that in the previous experiment, but smaller in extent, was formed at the bridge. This appeared to show, as in the previous experiments, that the island or islands were rather too far upstream to have the desired effect at the bridge. -Had an additional island been positioned near the 6th section the conditions might have improved; but the cost of three islands would be prohibitive; on the other hand, a single island near the 6th section alone is not likely to come into action, since the rocky right bank would then be the dominant attracting feature.

Experiment No. 6 .-

Two islands-one on the left shoal 3,500 ft. upstream and the other at cross-Fig. 1(d)/II-i section no. 6, 1,550 ft. upstream - were, therefore, built. Out of the experiments with islands, this gave slightly better results than the others.

> The high velocity flow continued along the centre line down to the lower island and then distributed itself at the bridge fairly uniformly, producing practically equal velocities in all the spans-average surface velocity being about $1.0 \equiv 5 \text{ft.}/$ sec. The discharge distribution through the bridge was, in general, better than in the other experiments with islands, although it was not entirely even, due to a small shoal formation in the centre. The scour at the left abutment was also less—up to R. L. 423.80—but concentration of discharge and scour in the left end span persisted.

> A shoal also formed below the bridge which, by elongating backwards may join the one above, thus defeating the purpose of centralising the flow. As the model is relatively slower in shoaling, the effects or efficacy of islands would be less in the river than shown by the models; and due allowance must be made for this fact in weighing the relative advantages of various measures.

Experiment no. 7.-

A common phenomenon observed in all experiments with artificial islands was that an island invariably forms a deep channel at its nose and along its











flanks due to its attracting effect; while, downstream of the island, despite the stream-lined shaping of the island, there is an unmistakeable tendency towards shoal-formation. Even if due allowance is made for the model limitation of the scoured material depositing downstream and not getting washed away to tho same extent as in the river, there is still a definite tendency for shoals to form downstream of "attracing" islands. Such shods would vitiate part of the good that the islands might do. Experiment No. 7 was designed to see whether an island, suitably sited below the bridge, would produce advantageous effects without the accompanying undesirable effects of a shoal upstream of the bridge, is should not be too far downstream, lest the pulling effect does not reach the bridge; it should also not be too near the bridge, in case the pier foundations are endangered by excessive bed scour working upstream by retrogression. In the model, as a compromise between these opposing requirements, the island was constructed 5 ft. = 1,000 ft. downstream.

Fig.4 (*)/11vi.

The discharge distribution obtained was not, however, satisfactory nor as uniform as in experiment no. 6. There was acute concentration of discharge and heavy scour in the left end span. Moreover, a shoal formed between the bridge and the island, indicating that the pulling effect of the island was not being felt up to the bridge; but placing the island nearer the bridge was not considered desirable for the reasons stated above.

Part II, Experiment 1—

The length of the left guide bank was kept the same as in experiment 8 of Part I; but the alignment was flared, being made symmetrical to the right Fig. s(n)11natural bank, which flares at an angle of 17°.

With 2.5 cusecs ($\equiv 100,000$ cs.), conditions were very nearly the same at the natural island; at the bridge, however, the change in the alignment induced slightly better results. The discharge distribution became more uniform and the cross-section more even. The tendency to shoal formation in the centre Fig. 4 (0)/11was relatively reduced. No scour was noticed at the left abutment.

(4) Conclusions of experiments in Parts I and II

(a) The best results were obtained with a flared left guide bank, increased waterway, and the T headed groyne cut back by $2' \equiv 400'$.

(b) In a flashy river like the Luni, full efficacy of islands is doubtful. For coming into full action the islands require high floods of long duration: while in flashy type rivers such floods are rare, the floods being generally short sustained. Also, the low floods flowing for a longer period and quick drops in discharges would silt up the scour along the islands, reducing their pull and hence their usefulness. The optimum location of an island in this case was apparently at about 1,500 ft. upstream of the bridge, as in that position it would be most effective in influencing the conditions at the bridge; but even so its effect was small and in nature, would be smaller still.

Part III

(5) Why it was necessary to repeat the experiments !

On completion of the experiments described above, air photographs of the bridge were received which showed a different alignment of the right bank to F_{19} . $\sigma_{1}(1-v)$, that shown in the original survey. This discrepancy was subsequently confirmed by a fresh survey. As this change in the right bank was evidently sufficient to affect the flow conditions at the bridge, it was decided to repeat those experiments which had given promising results in the previous series (Parts I and II).

(6) Experiments, Part III

As a result of the change in survey, the following experiments were carried out :--

Experiment No. 1-

With existing conditions.

F.g. 7/11-vi.

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Experiment No. 2-

With existing waterway but with existing left guide bank removed.

Experiment No. 3-

With conditions as existing in the prototype but with left guide bank Fig. 7/11-vi.removed and an "attracting island" positioned as shown in Fig. 7/11-vi.

Experiment No. 4-

With existing conditions but with a left guide bank symmetrical to the existing right natural bank.

Experiment No. 5-

Waterway of the bridge increased by 500 ft. and with a left guide bank as in experiment No. 4 above.

Experiment No. 6-

Waterway increased by 300 ft. and with the guide bank as in experiment No. 4.

(7) Observations on results of the experiments :--

Experiment No. 1 with conditions as existing in prototype

The repetition of the Experiment No. 1 of Part I was expected to yield more enlightening results, as the breach in the guide bank omitted in that experiment was reproduced now.

Unlike in the old experiment No. 1 of Part 1, the high velocity flow Fhotos 3065 hugging the concave guide bank was materially assisted by water rushing through the breach in causing an enormous scour near the left abutment down Figr. s and sto R. L. 411-80, against the corresponding bed level of R. L. 416-63 in the (a)/II-i. river. This confirmed that the concave curvature of the guide bank is an important cause of the scour at the left abutment—the other principal cause being the inadequate waterway. In spite of high velocities persisting on the right bank of the river upstream, the concave curvature of the left guide bank pulled a bigger discharge to pass through the left half of the bridge.

The dimensions of the scour-hole in the model depend upon the vertical exaggeration, the depth of the scour and the angle of repose of the material and are not susceptible of direct correlation to the dimensions in the prototype: it was, therefore, not expected that the profile of the bed would be exactly simulated in the model.

With $6.25 \equiv 250,000$ cusecs, the scour at the left abutment and the unequal distribution of discharge persisted and the conditions were in no way improved.

Experiment No. 2 with existing conditions but with left guide bank removed

It was evident from the first experiment that the scour in the left end spans could be reduced, if not eliminated, by either suitably realigning the existing left guide bank or even possibly by doing away with it altogether. It was, therefore, decided to test, first of all, the conditions obtained without a guide bank, since it would be more economical and also desirable in certain respects to do without a guide bank, if possible.

The model results, however, showed that by merely removing the guide bank it was not possible to reduce the scour. At section No. 10-across the natural islands—the left channel slightly developed to carry 67 per cent more discharge than in the previous experiment. At the bridge, however, the conditions deteriorated as indicated by the discharge distribution which now became more ununiform.

Experiment No. 3 with conditions as existing in the prototype but with left guide bank removed and an artificial island positioned as shown in Fig. 7/II-vi

Fig. 7/II-vi.

The possibility of getting satisfactory conditions by mere removal of the left guide bank being thus ruled out, it was decided to test an "attracting"

Railway Bridge No. 3 of the Samdari Ranlwara Branch of the Jodhpur Railway, on the Luni River



Upstream

Left abutment Downstream

Photo 3065. Scour at the abutment with a breach in the existing left guide bank (Expt. 1, Part III).



Downstream

Photo 3066. Deep scour at the left abutment obtained by the removal of the upstream present guide bank (Expt. 2, Part III).





island, sited 1,550 ft. upstream of the bridge as in Experiment No. 6, Part I, for centralising the flow. This had been found the best position for an island in the earlier tests.

The island, though successful in attracting the flow towards it by causing scour at its nose, was, however, ineffective in reducing the scour at the abutment; and there were indications of shoaling, as in the previous series of experiments with islands, which was undesirable.

This test, however, conclusively showed again that, in the existing conditions at the Luni Bridge, a guide bank is preferable to an island.

Experiment No. 4—with conditions as existing in the prototype but with a left guide bank symmetrical to the existing right natural bank

A left guide bank being thus proved necessary, experiment No. 4 was designed to decide whether a realigned guide bank without increasing the existing waterway—with a view to minimize cost—would be sufficient.

At section No. 10—across the islands—the distribution of discharge in the three channels was slightly affected, the right channel developing to carry increased discharge. Below, at section No. 6, the flow redistributed equally and passed the bridge with conditions which might have been ideal, were it not for an undesirably deep section in the central spans. Since, with even distribution, $\operatorname{Flg. 9}(h)$ this deep acour occurred, it proved that the existing waterway was insufficient.

Experiment No. 5.—Existing waterway of the Bridge increased by 500 ft. and with left guide bank alignment as in experiment No. 4 above

Since it was clear that the existing waterway of the bridge was insufficient—and since the piers were not sufficiently deep or founded on rock—an increase in the water-way was evidently desirable. As previous experiments had shown that an increase in waterway of 500 ft. together with a left guide bank gave the best results of all the measures tried, this arrangement was again tested with the modified survey reproduced in the present series.

The modified bank line, however, did not appreciably alter the conditions at section no. 10, across the natural islands. The main flow persisted in the central channel which, after gently swinging towards the nose of the sharp bend on the right bank at section No. 6, about 1,500 ft. upstream, again redistributed at the bridge. Unlike in Experiment No. 8, Part I, greater concentration of flow was, however, obtained in the right half of the bridge. No dangerous $F_{ig.10/11.vl.}$ local scour was perceptible near the abutments or in midspans, although comparatively heavier action was noticed at the third right pier. Maximum velocity reduced from 1.17 (in the corresponding experiment of the previous series) to 1.04 ft./sec. and the velocities throughout the section were more uniform. These results were, however, offset by the formation of a shoal in the left half of the channel, extending from the left natural island down to within 1,000 ft. above the bridge. As in other experiments with extended waterway, this shoal formation indicated that the increase in the waterway provided was probably more than required, in spite of the duration of the higher, scouring discharges being exaggerated in the model.

> Experiment No. 6.—Existing waterway increased by 300 ft. only with left guide bank alignment as in experiment No. 4 above

True to expectations the reduction of the proposed increase in waterway say of only 2 model spans=300 ft., i.e. total waterway equal to 1.1 times Lacey's minimum stable width—resulted in markedly improved conditions. Velocities and discharges in the different spans were pratically equal; the bed profile was F_{16} 11/11-vi. more or less uniform; and uneven or heavy local scour was conspicuously absent. It is true that the maximum velocity for dominant discharge 2.5=100,000 It is true that the maximum velocity for dominant discharge 2.5=100,000 1.25 ft./sec. (=6.25 ft./sec. in the prototype) but this was not excessive. The conditions with the maximum discharge of 6.25 cs. = 250,000 cs. were also satisfactory.

Slactory.

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Flow conditions were not appreciably altered at section No. 6. Closing th left end span of Experiment No. 4 apparently affected the conditions at section No. 10, as indicated by the reduction in discharge of the left channel, with a corresponding increase in the adjacent central channel discharge.

Action along the guide bank, probably due to the fluming imposed, indicated the necessity of an apron, especially at the curved head of the bank.

Immediately above the bridge, action was substantially uniform along the bed and no shoal formation was obtained.

(7) Conclusions from experiments in Part III

(A) The model showed that the alignment of the left guide bank had an important bearing on-

(i) the prevention of the dangerous scour at the left abutment; and

(ii) the flow conditions at the bridge.

(B) The model also convincingly proved the justification and advisability of increasing the waterway—by about 300 ft. only—for the following reasons:—

(a) it gives uniform distribution of discharge through the bridge;

(b) helps prevention of deep scour at the abutment and the piers;

(c) causes reduction of the present inordinate afflux; and

(d) results in absence of undesirable shoals upstream of the bridge, which occur with a larger increase in waterway.

(C) The guide bank would require an adequate apron, especially heavy. Appendix I. protection along the curved head (see Appendix I).

(8) Recommendations

(i) New spans providing about 300 ft. 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 atleast to R. L. 39700.

(*iii*) New guide banks, upstream and downstream of the bridge, on the left bank should be constructed. Suitable alignments and sections for these are shown $\frac{Figs. 13 \text{ to } 17}{11 \text{ vi}}$ in figures 13 to 17/II-vi. Two alternative alignments of the guide bank are $\frac{11 \text{ vi}}{11 \text{ vi}}$ 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 Fig. 13/II-vi shown in the fig. 13/II-vi) 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.

Fig. 9 (c) II-vi 58









DESIGN OF THE LEFT GUIDE BANK OF THE RAILWAY BRIDGE OVER LENT

NEAR SAMDARI (JODHPUR RAILWAY)

(1) Alignment of the guide bank to be as shown in figure 13 H-vi. (2) Length of the guide bank from bridge :-

Upstream length ... 1,400 ft.

Downstream length ... 300 ft.

(3) Top R. L. of the guide banks :- At the curved head of the guide bank,

H. F. L. in 1944 floods was 448 00. In view of the proposed 300 ft. increase in waterway, guide bank top may be kept at R. L. 451,00, i.e. free board 3'. (4) Side slopes of the bank :- These should be kept at 2:1 on the river

side and 3:1 with rearberms S' below top, on the land side. (5) Top width-20 ft. (6) Thickness of pitching-2 ft.

(7) Thickness and width of apron:----

(i) at the curved head (i.e. between YY and ZZ).

(ii) at the shanks (i.e. downstream of YY) and

(iii) at the reach adjacent to the retired bank.

For determining these, maximum probable scour must be calculated.

By Lacey's formula $R = 47 (Q/f)^{1/3}$;

with Q = 250,000 cusecs.

 $f = 1.76\sqrt{m} = 1.76\sqrt{.26} = .895.$

... R=30.6 ft.

Actual depth of water at the bridge after the 1944 floods was 16 ft.

This difference in the actual and calculated values of R is due partly to the nature of this river, which has a tendency to widen rather than deepen, attributable mostly to the sudden rise of floods which do not sustain long enough to allow the bed to scour."

Generally, the maximum scour at the head of guide bank is assumed to be A_{PP} I. 2R; in the present case, 1.6 R may be assumed, for reasons explained above; along the shank, the depth of maximum scour may be assumed as reducing from 1.6 R to 1.0 R below YY. These factors give, -

(i) Maximum probable scour at the upstream curved head of the guide bank-49.5 ft., say 50 ft. between YY and ZZ.

(ii) Maximum scour at the shank-

Reducing from 50 ft. to 30 ft. at YY and below.

(iii) Maximum scour between ZZ and retired new bund 30 ft. (e.g. Fig. 13/ c.s. XX in fig. 13/II-vi.

Cubic contents of the apron per foot run are to be such that after launching the slope will be covered by stone of 1.25 times the thickness of the pitching".

Section of the guide bank at the curved head.



(1) 1945 Annual Report (Tesh.).

(2) Page 46 "River Training and Control" by F. Spring.

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(i) At the head, cubic contents per ft. run $=85 \times 2 \times 1.25 = 212$ cft. Width of apron provided = 38 ft. \therefore Average thickness of apron = 212/38 = 5.6 ft. \therefore Apron 38 ft. \times 5.6 ft; extra berm 10' \times 5.6 ft.



(ii) Section along shanks below Y Y. Below YY, cubic contents per ft of the apron of the guide bank.

=29.2 × 2×1.25=73 cft. Width of apron provided = 20 ft. \therefore Average thickness of apron = 3.65 ft. \therefore Apron = 20' × 3.65' Extra berm = 10' × 3.65'.

(iii) Section of the apron in the region adjacent to the junction of the guide bank and the retired bund (e.g. XX)



Cubic contents per ft. = $49.3 \times 2 \times 1.25 = 123$ cft. Width of apron = 35 ft. .: Average thickness = 3.52 ft. .: Apron = 35 ft. $\times 3.52$ ft.

II-(vi) 2-TRAINING OF THE JUMNA RIVER AT DELHI GATE PUMP HOUSE BY MEANS OF A PITCHED EMBANKMENT-DELHI.

(1) Introduction—

As a result of the adverse curvature, the low water course of the Jumna river at the Delhi Gate Pump House recedes, after the floods, to the left bank away from the Pump House, necessitating heavy expenditure in bunding of the low water channel and dredging a channel to feed the Pump and Power Houses.

Experiments to train the river so as to attract the deep channel to the Pump House bank by means of a pitched embankment were reported in Annual Report (Tech.) 1945.

A pitched embankment, which cut a portion of the existing downstream bank was found effective in inducing a low water channel, both under existing and the post-barrage conditions—Fig. 18/II.-vi—. This alignment of the embankment was not suitable to the Delhi Power House authorities in view of the projected extension of the Power House. Further experiments were, therefore, carried out to evolve an alternative suitable alignment of the embankment which did not deprive the Power House of any valuable land.

Fig.17/ II-vi

Fig 18/ II -vi



6 PZ.PPGCHA'47.



After a few trial and error tests, the alignment shown in Fig. 18 II.-vi Fig. 18 II. vi bankment, but without barrage) was found to be even better than that already reported last year.

Subsequently, however, at the meeting held on 5th and 6th June 1946 to consider the problems connected with the water-supply at Delhi, *inter alia*, it was decided as under:--

"The pitched embankment proposed by the Indian Waterways Experiment Station, Poona, has been evolved as a result of experiments based on the river survey of 1941. The river conditions were then about the worst for supplies past Delhi Gate. It could, therefore, be assumed that it would work effectively under the river conditions that would prevail subsequently. It was, however, decided that the Director, Indian Waterways Experiment Stration, Poona, should carry out further experiments on the basis of the survey for 1945-46 with the proposed alignment of the pitched embankment and with another alignment 60' further towards the river, and report results. The Superintending Engineer, Delhi Province, was requested to get the survey of the river in the vicinity of Delhi Gate completed as early as possible after the floods of 1946 and supply a copy of that to the Director, Indian Waterways Experiment Station, Poona, not later than the end of September 1946, so that the latter may run a few test experiments on the basis of that survey as well, to confirm that the pitched embankment as proposed already needs no modification in view of the changed river conditions.....

"It was decided that the Director, Indian Waterways Experiment Station, Poona, would, while carrying out tests of the pitched embankment, also include two intake piers to ascertain how these could best be located in relation to the pitched embankment so as not to vitiate in any way the purpose of this embankment."

As a result of important discrepancies pointed out by the Director, Indian Waterways Experiment Station, Poona, in the survey of the river after the floods of 1944, which could not be reconciled, it was decided by the Superintending Engineer, Delhi Province, to have an accurate triangulation survey made. The plan showing the results of this survey were received complete with the nocessary cross-sections on 27th December 1946. No dependable survey of river conditions after the floods of 1944 being thus available, this report relates only to experiments made with the accurate survey of the river course *after the floods of 1946*.

Fig. 19/II-vi shows the incorrect surveys of 1944 on which the previous Fig. 19/IIexperiments reported last year were based—superimposed on the recent accurate vi. survey of the river course after the floods of 1946.

Though, at a first glance, the new survey appeared to be more favourable for the embankment, because of the more easterly alignment of the left bank above groyne 4, yet a closer study showed that the apex of the meander, as also the shoals and confluences of channels, in the new survey have moved further downstream—i.e., there is a bodily downstream movement of the river course facts which counterinfluence the apparent improvement.

It will also be observed that the old, incorrect survey showed the nose of groyne 5 jutting 160 feet more into the river. It may be recalled that in experiment No. 6—Annual Report (Tech.) 1945—the nose of this groyne had to be receded to reproduce the flow conditions; the model nose was actually receded by 60 ft. more than as existing in the river, according to the correct survey; this was because of the compromise which had to be made between geometric similarity of the action of the nose and the vertical exaggeration of the rest of the model—a model limitation and technique. Secondly, the right bank of the river below the Pump House near Culvert No. 12 was 440 ft. river-wards, i.e. eastwards, in the incorrect survey. The model, however, had been operated to yield its own configuration, which, it is interesting to observe, tallied with the correct survey.

These points clearly bring out the often-stressed, but much criticised statement of this Station that model experimention is an art and not an exact

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science-at least under the present limited knowledge of the various phenomena involved; and that model indications are often more dependable than data of doubtful accuracy.

(2) Experiments-

The following further experiments were carried out :----

Experiment 15 :-- With existing conditions, the model bed being initially laid to the correct survey of the river course as after the floods of 1946.

Experiment 16:-Same as Experiment 15, but adding pitched embankment adjoining the Pump House bank.

Experiment 17:-Same as Experiment 16, but with a Barrage added at Purana Killa (Sill of the barrage at R. L. 58.5; and sill of the undersluices at R. L. 56.5, as stated in earlier experiments).

Experiment 18:-Same as Experiment 17, but with a modified alignment of the pitched embankment to conform more with the downstream river course as well as the expected river course after construction of a Barrage at Purana Killa.

Experiments were also made with two Intake Piers by themselves and the Piers in conjunction with the pitched embankment. These are described in para. 5 below.

(3) The model and procedure of experiments.

The model which represented a reach of the river from just below Wazirabad Pump House to the Okhla Weir had the following scale ratios :----

Length scale ratio	1/150
Depth ,, ,,	1/46
Discharge " "	= 1/22,500
Vertical Exaggeration	= 3.26
Surface S. E. = Bed S. E.	= 6.18

Before commencing this series of experiments with the correct survey, it was verified that the optimum bed movement and proper flow conditions were simulated with the tilted model and scales adopted. The existing inlets and Pump House bank were reproduced in a cement-plastered, vertical bank. Before starting each experiment, the model bed was moulded to the conditions after the 1946 floods, and flood cycles rising from 40,000 cs. to 90,000 cs. and falling back to 40,000 cs. were run with appropriate bed sand charge till the model stabilized. The rate of sand charge was 05 parts per 1,000 with 40,000 cs. rising to .075 parts per 1,000 for 90,000 cs., based on initial "trial and error" tests.

Note.-The model could not be run to discharges lower than 40,000 cs. while scour in the river occurs down to discharges as low as 1,000 cs., so that success of a measure for inducing scour should be greater in the river than indicated by this model.

(4) Discussion of results.

Fig. 20 to Figures 20 to 23/II-vi and photos 5068, 5070, 5071 and 5074 show the 23/II-vi. Photos 5067, low water channel in experiments 15 to 18, from which it will be observed that 5070, 5071 and 5074. while a sand bank was thrown up against the upstream Pump House inlet in Fig. 20/II-vi experiment 15 (without embankment), the deep channel hugged the Pumping Fig. 21/11-vi Station bank in experiment 16 (with embankment). In experiment 15, though flow conditions were apparently favourable, the low water channel appeared to touch the Pumping Station bank near the downstream inlet, this being due to the reproduction of the Pumping Station bank in masonry-which attracted the flow to a certain extent; but this will not occur in the prototype. In experi-Fig. 22/II-vi ment 17 (with embankment and Barrage), though the deep channel was slightly away from the embankment, compared to experiment 16, yet a shallow depth was noted against the embankment; and a shoal was also thrown up in front of the downstream inlet of the Power House.

> In experiment 15, as visual observations showed, adverse curvature effecti.e. bed flow being thrown to the right bank and surface flow away from it,-was

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THE JUMNA RIVER AT DELHI GATE PUMPING STATION



Pump House.

Embankment.

5067. Photo showing low water channel observed in experiment 16 with pitched embankment only.



Pump House. Embankment.

5070. Photo showing low water channel in Expt. 17 with embankment of Expt. 16 and barrage at Purana Killa. Note the shoal in front of the downstream inlet (i.e. downstream of X).

THE JUMNA RIVER AT DELHI GATE PUMPING STATION



Pump House.

Embankment.

5071. Photo showing the low water channel in experiment 18 with modified embankment and barrage at Purana Killa. Though there was water against the embankment yet along it there was a small sand bank; the deepest portion of the channel being away from it. The shoal in photo 5070 did not appear in this experiment.



Pump House.

Embankment.

5074. Photo showing conditions obtained when in Expt. 18 with modified embankment, the barrage was removed and experiment continued. The deep channel was against the embankment.





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PART II

obtained from the upstream inlet to a point about 450 ft. eastwards (i.e. riverwards). The main, high-velocity, forward flow and the deep channel were well away from the Pump House—a result which was in accordance with the experience in the prototype.

While in the prototype, the low river channel crosses over to the right bank from a point midway between groynes 3 and 4, it did not do so in the model on account of a sand bar, which formed during high floods but did not scour away with the low floods, in spite of the channel below the bar being deeper than on the left bank; this is due to the model limitation that it could not be run to equivalent discharges lower than 40,000 cs., while bed movement — and so scouring of the bar-occurs in the river right down to river discharges of 1,000 ousees. With the embankment, as in experiment 16, however, this adverse curvature effect was markedly absent in that at every point along the embankment, the surface and bed lines flowed parallel to each other as in a straight channel. With the embankment and barrage, as in experiment 17, the conditions were not considered equally satisfactory because of the shoal formation in front of the d/s On modifying the curvature of the embankment, as in experiinlet. ment 18, the shoal did not form in spite of the Barrage, apparently because the alignment of the embankment was more in conformity with the probable bank line under the post-barrage conditions. Fig. 23/II-vi shows the low water Fig. 23/II-vi channel in experiment 18; in this experiment though there was water against the embankment, yet the deeper channel was a little away from the embankment. When, however, the same experiment was continued with the barrage removed, the sand bank scoured away and the deep channel began to hug the embankment as in experiment 16-vide Fig. 24/II-vi-which shows the cross Fig. 24/II vi sections Nos. 12, 14 and 16 obtained in the model before and after removal of the barrage. It should be possible to eliminate the adverse effect caused by the barrage-of having the deepest channel section a little away from the bankby modifying the design of the barrage so as not to distort the natural distribution of discharge intensity to any appreciable extent. In the design tested the undersluice crest was only 2 ft. lower than the Barrage crest; this may be insufficient to give the same discharge distribution as in the river without a Barrage and may thus cause distortion which prevents the formation of a deep channel. Experiments to test this point are still to be made.

Figures 25 to 27/II-vi show the lines of flow in the neighbourhood of the Fig. 2. to 27/ Pumping Station in experiments 15, 16 and 17 with a flood of 40,000 cusees. In experiment 15 there was return flow at the upstream inlet, which later disappeared as the bed built up. The return flow was probably due to the initial dredged conditions to which the model was laid, which were not in conformity with the general curvature of flow. The trend of the surface lines in experiment 15 was to flow more to the east, i.e. left of the corresponding bed lines—an adverse curvature effect. In experiment 16, this effect was absent and bed lines were thrown more to the east. In experiment 17, the flow conditions, though better than in experiment 15, were inferior to experiment 16.

In experiment 18 lines of flow were similar to those in experiment 17.

(5) Intake piers.—

Figs. 28 and 29/II-vi show the contours in the neighbourhood of Intake F_{128}^{Figs} 24 A piers tried in experiment 15 and 16, i.e. without and with the pitched embankment respectively. It will be observed that though, in experiment 15, the piers were fixed with their longitudinal axis parallel to the flow line and were well apart (1 ft. \equiv 150 ft. apart) so as not to neutralise the effect of each other yet, in this experiment, as judged by the 658 5 contour, the action of the piers was purely local and by themselves they would be insufficient: whereas in experiment 16 (with the embankment), they added to the pull of the embankment and helped in scouring the channel along the P. S. bank.

(6) Conclusions .--

(1) It appeared that under existing conditions the embankment of experiment 18 would be effective in inducing a deep water channel along the Pump House bank. (2) With a barrage of the design tested, the effect of the embankment slightly deteriorated in that the deep channel was slightly away from it. The narrow, low level, intermediate sand bank along the embankment should not, however, be of significant consequence because, even if the deep channel is a little further away, the pond level would in all likelihood cover the bed in front of the Intakes. The Barrage can also probably be suitably redesigned so as to give the same conditions in respect of the low water channel as obtained with a pitched embankment without the Barrage in experiment 16.

(3) Intake piers would have a beneficial effect on the action of the pitched embankment, but without it, they will be of little use; because the predominating factor is the adverse curvature of flow which is not corrected by the local scour round the piers.

II-vi-3. TRAINING THE WATRAK RIVER AT KAIRA

(BOMBAY PRESIDENCY)

The Watrak flows in an alluvial bed of fine sand (m = 0.26 mm.). There is Fig. 30/II-vi a sharp right-angled bend from about $1\frac{1}{2}$ miles to 3 furlongs upstream of Kaira town. For 20 years or more erosion has been taking place; but until 1923 it was so slow that it did not cause alarm. The earlier erosion appears to have been slower on account of curvature being less sharp than now; and also because the bank was partially protected by a "man"-like material made up of a mixture of slightly consolidated clay and silt. After the floods of 1927, four groynes were constructed along the concave bank above the town. These consisted of teak piles connected together with bullies, filled in between with sand bags and brushwood. Two of these were washed away in 1930; and the remaining two groynes, together with 530' of reinforced brick pitching protecting the Provincial road, and also part of the road itself, were washed away, bit by bit, subsequently.

> In August 1937, the local officers wrote to the Director that the river had further eroded its left bank upstream of Kaira town and that there was a grave danger of its damaging the north-west corner of the old Fort wall. Experiments were accordingly carried out⁽¹⁾ and 3 alternatives were suggested:—

(1) A bridge—submersible or high level—projecting from the northwest corner of the town at 30° upstream, the bridge portion starting 500 ft. from the town wall, with a clear waterway of 300 ft. and a depth of 7 ft. overflow at 1 lakh cusecs, which was the maximum estimated flood discharge.

Fig. 31/II-vi.

(2) A groyne or spur 180 ft. long, sloping down at 1 in 5 from the north-west corner of the town pointing upstream at an angle of 35°, and

(3) to deepen and strengthen the corner of the town wall by pitching.

It was, at the same time, pointed out that this last alternative, though costing least, was objectionable; because it would hold the river permanently at the corner of the old Fort wall, thus perpetuating the dangerous swing u/s and downstream and because the afflux would be shifted towards the north-east corner of the town and the protective strip of land north of the town would be reduced considerably if scour extended.

Actually, the second proposal of constructing the sloping spur was at first approved by Bombay Government; but, subsequently, it was rejected in favour of the third alternative on grounds of cost.

Not being satisfied with the measures adopted, the Director inspected the site in April 1940 and proposed that further experiments should be done to check the erosion of the left bank upstream of the town. It was, however, decided by the local officers that the experiments might be postponed till the effect of the pitching had been sufficiently watched. As predicted by the Station, there was further trouble. Had either of the alternatives (1) and (2) mentioned above

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⁽¹⁾ OIH K Annual Report for 1987-88 (page 7?) 1938-39 (page 18).





been adopted, they would have been very effective in repelling the attack of the river away from the left bank upstream of Kaira and the Fort wall and would have also straightened the course of the river downstream. Instead, the Fort wall is now pegging the river down permanently at the point, while the upstream left bank is always subject to erosive attack. After leaving the Fort wall corner, the river has gone over to the right bank downstream. When the reinforced concrete bridge was subsequently constructed downstream of the town in 1942-43 the right bank "cold weather" channel had to be blocked by an embankment, with the idea of forcing the river through the bridge. Actually, this resulted in a breach of the approach bank virtually outflanking the bridge. The approach bank of the bridge had then (in 1944) to be very strongly armoured with pitching on the slopes and a heavy apron at the toe, at a cost of Rs. 2,35,000. From the above it will be clear that river curvature is being severely distorted with the result that the right bank area of the Hariala village upstream of the bridge has been to be eroded and an avulsion of the river into the low lying area of the village has been feared.

The local officers again approached the Director for advice during the year under report. The site was inspected in December 1946 and the Bombay Government was informed that further model experiments would be necessary for the suggestion of a permanent remedy. But, in view of the urgency of the problem, a temporary solution for training the river with permeable (timber) pile groynes or with tree groynes—which would not conflict with the final solution as indicated by the model experiments—was recommended. F_{18} . $B_{11}(1, v)$.

As trees were available within a reasonable lead, the proposal of tree groynes as recommended is, it is understood, being adopted. Detailed instructions for $_{A_1}$ both types of construction—*vide* Appendices I and II—were furnished.

Appondance

Firs. 32 & 39/11-vi.

1 and 11.

APPENDIX I

(Tree Groynes)

Each tree groyne should be 200 ft. long and originally laid to face Each tree groyne should be to the bank; so that when the groyne some 60° upstream with reference to the bank; so that when the groyne some 60 upstream with reference it assumes a position still facing slightly launches and becomes sand-bound it assumes a position still facing slightly launches and becomes salu-bound to under impermeable groyne which is upstream, say 70° to the bank. Unlike an impermeable groyne should not face upstream, say 10 to the only of the second s generally made to lace ou upstream, a potential against its face; converting it to upstream since it would collect floating debris against disadventages in the upstream since it would context notice that attendant disadvantages in this case, an almost impermeable type, with attendant disadvantages in this case. an annost inpermeasure of the state of the s 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 Fig. 34/ILiv. in Fig. 34/ILiv; They are approximately 300 ft. distant—a distance which should be easily protected by 200 ft. long groynes, as a groyne generally protects 2 to 3 times its length.

Each tree groyne should consist of a thick wire rope (3/4'' dia.) 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' long) per Rft. 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.

APPENDIX II

(Timber pile groynes)

1. Main Verticals.—Country timber piles, 8" minimum diameter, should be embedded a minimum 8 ft. (and preferably 10 ft. or more) with top of half the length of groyne at R. L. 800. This top level is suggested with a view to restrict the required pile lengths to 25 ft. without reducing the penetrations; with longer piles being available there should be 1 ft. extra penetration to every 2 ft. 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.

2. Intermediates, transverses and Struts.—There will be two rows of verticals, spaced not more than 4 ft. apart in each row, the rows being also 4 ft. apart. In between two main verticals there will be two intermediates (2^n-4^n) dia.) embedded not less than 4' below bed. Each row should be either closely intertwined by "lai" brushwood branches going in and out around each vertical, or horizontal walings, 4"-6" 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 4"-6" diameter rafters. Every other main vertical of the rear row should be vertued, the strut being embedded a minimum of 8 ft. below bed (measured vertically) and squarely clamped and bolted to the main vertical.

Fig. 36/II-vi.

3. Filting.—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 of alternate layers of 6 ft. thick brushwood weighted down by 2 ft. 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.

4. 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 3 ft. thick for a width of 10 ft. along the shanks and 20' along the nose.

5. 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.



could not be completed within the time available for it. The present position is that the isolated groyne head has caused heavy accretion below it while there has been further erosion of the left bank upstream of it, and the railway siding is now barely 160 ft. from the river bank.

Before the flood season of 1946, the local opinion appeared to rely on the chances of a cut-off occurring, as the ratio of arc/chord of the Samasata loop was 2.8. (A cut-off occurred in 1943 when the ratio was 2.9). The minimum Figs. 82 and distance across the neck of the loop was 6,200 ft. in 1945; and it has reduced $\frac{19}{20/(1-v)}$. further to 5,800 in 1946; nevertheless, comparing the 1946 loop with the 1942 loop, it will be seen that conditions are still not favourable for a cut-off, which the model has later confirmed. Besides, it is dangerous to depend upon the occurrence of a cut-off merely because the arc/chord ratio has a value at which a cut-off occurred in the past: it may be recalled that a cut-off did *not* occur in 1923 and 1924 even though the ratio was as great as 4.0. The Director and the Chief Engineer, N. W. Railway inspected the site on 22-9-1946 and on the former's advice it was decided to carry out model experiments at Poona to assess the likelihood of a cut-off occurring through the Sharabi Nala—see Fig. 40/II-vi.—, to test alternative measures to protect the bank Fig. 40/II-vi. from further erosion, and to evolve measures which would assist or accelerate the currence of a cut-off.

Table 1/II-vi showing tortuosity = $\frac{\text{length of cold weather channel}}{\text{direct length}}$ of the river Sutlej at Samasata

Year of Survey.	Length along cold weather channel.	Direct length.	Tortuosity (Col. 2/Col. 3).	Remarks.	
1	8	8	4	5	
	<u> </u>		1	1	
1916	40.000'	24,000'	1.66		
1917	40.000'	24,000'	1.66		
1918	48,000'	28,000'	1.71	1	
1919	50,000'	28,000'	1.89		
1920	88,000'	42,000'	2.09		
1921	88,000'	38,000'	2.31	}	
1922	96,000'	38,000'	2.52		
1923	104.000'	48,000'	2.17		
1924	88,000'	54,000'	· 1·63		
1925	88,000'	54,000'	1.63		
1926	88,000'	54,000'	1.63		
1927	72.000'	54,000'	1.33	Bell bunds cons-	
1041	-,,			tructed this year.	
1028	72.000'	54,000'	1.33		
1020	99,000'	58,000'	1.70		
1930	h				
1031	CPlans not avail	able.			
1032	1	-			
1033	92.000	64,000'	1'44		
1091	92,000'	64,000' 1'44		1	
1025	88.000'	60,000' 1.46			
1096	96,000'	58,000'	1.65		
1007	96.000'	58,000'	1.62		
1000	96,000'	58,000'	1.62		
1930	104.000'	64,000'	1.62		
1939	104 000'	64,000'	1.62		
1940	104 000'	6±,000′	1.62		
1941	104 000'	64,000	1.62		
1942	90,000'	64,000'	1.40	}	
1943	95 000'	64,000'	1.48		
1944	95,000'	64,000′	1.48	ļ	
1945	95,000'	64,000′	1.48	1.	
1010			·		

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PART I-EXPERIMENTS IN A VERTICALLY-EXAGGERATED MODEL

2. Model Experiments

The scale ratios initially adopted for the model were:

C BOUND LUGIOU		
Horizontal	=	1/250
Vertical	==	1/80.
V. E.	==	3.12

(See Appendix I for details).

Appendix-I.

Considerations of the discharge, w/d ratio, charge and grade of the material in the model (relative to the prototype) led to the adoption of a slope-exaggeration of 80—in other words, a tilted model was necessary—see Appendix I.

Experiment I. Existing conditions.

Fi3 40/II-v After laying the model to these scales according to contours in figure 40/II-vi it was run to stability. As the survey did not extend far enough upstream the model was suitably extended so as to ensure correct entry conditions. The scales to which the model adjusted itself were:

depth scale = 1/65 or V. E. = 3.85 and S. E. 10.0

These scale ratios were adopted when the model was relaid in various experiments.

Sand charge required to maintain correct conditions (i.e. appropriate flow conditions, bed movement, and slope) was found to be 0.083 parts per 1,000 (by weight).

Figs 41/(a) (b)/II-vi.

Figures 41 (a) and (b)/II-vishow lines of flow and isovels with Q = 2.4 cs. (=150,000 cs.) with existing conditions. These were taken as the basis of comparison with other experiments. The mole head of the breached spur was retained in this experiment. It was noted that the high velocity flow hugged the left bank and some of it was attracted by the mole head. Sharabi Nala across the loop was reproduced in the model as a cut-off was expected to occur through it.

Fig. 41/(a) II-vi.

Experiment 2: To test measures of improving the Sharabi Nala so that early avulsion would take place through it

As in the prototype, the mouth of the Nala was moulded in stiff inerodible clay and the rest of the reach in semierodible material, which, as expected, could not stand the steep bank slopes dictated by the vertical scale, and the Nala soon widened considerably below the mouth. This model limitation should not be mistaken as an indication of the likelihood of the rapid development of the Nala.

This could not, however, affect the discharge of the Nala which was governed by the water surface slope and the cross-sectional area at the entry, which were correctly reproduced in stiff clay in the model.

Percentage river discharge carried by the Sharabi Nala*, at different flood stages, in the model were:---

Per cent discharge Equivalent river Equivalent of carried by the Nala in the Model discharge discharge in cusees. column No. 3, in the river. Remarks. in cuseca. model. 1 2 3 4 5 1.08 cusecs. 67,500 cusecs. 5.55 per cent. Not known. 1.2880,000 ... 6.0 Not known. ... 2.40 " 150,000 ,, 6.04 4.5 per cent.* ,, About correct.

TABLE 2/II-vi

Appendix IL

* Calculations for the Sharabi Nala discharge are given in Appendix II.







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It was observed that a considerable quantity of bed sand was drawn into Fig. 43/II.vi. the Nala. This was apparently due to the following factors:---

(a) the deep scour up to 6' below normal bed level at the protruding nose of the right bank at the mouth of the Nala attracted the bed material and threw it in suspension; and

(b) the curvature effect was subdued at the mouth owing to its position relative to the meander, i.e. the mouth of the creek being at the transition, was in an unfavourable position in respect of curvature. This perhaps explains why the Nala has not sufficiently developed, though existing at least since 1943.

Experiment No. 3

The banks at the mouth of the Nala were cut back, as in figure 37/II-vi, to R. L. 360.00, the total excavation being approximately = 25,20,000 cft.

At the mouth, the depth of scorr on the left bank (near A in fig. 44/II.-vi) Fig. 44/II.-vi) two much reduced—24 ft against 6' in the previous experiment. The deep channel in the Nala was, however, maintained along this bank and all the sand moving along the bed was deflected to the opposite side; thus, it was apparent that widening of the mouth should all be done on its left bank so as to induce the maximum possible flow into the Nala. Heavy action was noticed along the left bank immediately below the entry, indicating the natural tendency of the Nala to widen on that side.

Figure 44/II-vi clearly shows the effect of widening the mouth on the $_{\rm Fig. 44/II-vi,i}$ flow conditions immediately above the entry.

Increase in the discharge of the nala at various flood stages, due to increase in the cross-sectional area at the mouth, was as in the Table 3/II-vi:-

River discharge in cusecs.		Nala dis in cu	charges 15ecs.	Per cent river discharge in the	Per cent river discharge in the Nala in	Increase in the per cent discharge	
Proto.	Model.	Proto,	Model,	Nala in Expt. No.5.	Expt. No. 2.		
67,500 80,000 150,000	1·08 1·28 2·40	·09 ·126 ·335	5,620 7,850 20,950	8·34 per cent. 9·85 " 13·95 "	5·55 per cent. 6·00 ,, 6·04 ,,	2·79 per cent. 3·85 ,, 7·91 ,,	

TABLE 3/II-vi

It is important to recognize, however, that the increase in discharge will be reduced to some extent due to relatively more bed sand entering the nala and retarding its developing tendency. The experiment also showed that the mouth should not be widened by bellmouthing the entrance along the right (or upstream) bank of the nala.

In fact, in order to give the cut-off channel the best chance of quick development, it is desirable to minimise the entry of coarse sand into the nala; development, it is desirable to minimise the entry of coarse sand into the nala, but because this coarse sand would not only raise the bed and choke the nala, but because this coarse sand would not only raise the bed and choke the nala, but because this coarse sand would not only raise the bed and choke the nala, but because this coarse sand would not only raise the bed and choke the nala, but lt was, therefore, suggested that a bund should be constructed at the head of the Sharabi Nala, with its top at the level of the adjacent banks of the nala, made Sharabi Nala, with its top at the level of the adjacent banks of the nala, made up from excavated material by widening along the left (downstream) bank of the up from excavated material by widening along the left (downstream) bank of the from the nala and moved instead into the loop to raise its bed; so that, at so that to high flood, the bund at the head will breach forcibly and will open out and deepen the nala rapidly.

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Conclusions from experiments 1 to 3

The model has shown the unlikelihood of the Sharabi Nala quickly developing as a cut-off channel, with or without further inducement by other training measures. Unfortunately, the Sharabi Nala happens to lie in an unfavourable position for the early development of a cut-off; in its present alignment, it tends to draw heavy bed sand which would not favour its development.

Were a similar cut-off channel available in some such position as XY shown Fig. 39/II-vi in fig. 39//II-vi, it would require little or no inducement for an avulsion to occur: because not only would it then short circuit a more considerable length of the river channel but, what is more important, it would draw only the surface flow, free from the adverse effects of bed sand on cut-off development. A pilot cut along XY to induce avulsion, however desirable, would be prohibitive in expense and would involve earthwork which cannot conveniently be done within one working season. In the event of this cut-off occurring in future, the river is expected to increase its length both upstream and downstream. As the river will be held at the Railway bridge upstream, it may form a loop towards the left bank between the bridge and the upstream end of the cut-off channel; these developments will have to be watched carefully and necessary corrective measures taken in time.

The strongly armoured mole-head of the breached spur, as it stands, is acting as a pitched island, attracting the flow; and it is, therefore, counteracting the tendency for the cut-off to occur. In the existing conditions, the river is likely to acquire, during the next flood season, a permanent set on the left bank, in the vicinity of the Station yard; and in spite of the length of the loop being about 2.8 times the length of the chord, there is little likelihood of an avulsion occurring before there is further serious inroad by the river towards the Station yard.

The first essential, therefore, is to take immediate measures to strengthen the ring bund round the Samasata yard similar to the measures adoped for the protection of the main line near Rohri; but this is to be considered only as the last line of defence and it is essential to remove the attack on the river bank by a repelling spur, before a cut-off occurs upstream and before the river attacks the ring bund of the Station yard; because in the latter event, the apron will launch and there will be a strong and constant "attraction"even a permanent set-towards it, making future counteraction very costly and difficult.

So, further experiments were done to determine the effect of removing the mole-head and to evolve the optimum design of a repelling groyne.

Experiment 4: Effect of removing the mole-head on existing conditions

Experiment 1 was repeated without the mole-head. Slightly lower Figs. 42 (a) Experiment I was repeated without the mole-nead. Slightly lower and (b)/II- velocities were obtained along the left bank and the conditions of flow were slightly better without a mole-head than with a mole-head; so the conditions may be expected to improve; but the difference in velocities being small, the difference in bank erosion, and the difference in the rate at which cut-off will occur, will also be small.

Experiment 5: With a repelling groyne and without the mole-head.

As it was evident that some measures were required to protect the ring bund and to accelerate the cut-off, a repelling spur was next tested.

It is evident that the best position for the repelling spur, in the present conditions, is in the vicinity of the breached spur. The repelling spur could not, however, under present conditions, be located just upstream of the scour-hole caused by the breached spur without causing serious difficulties in holding the new spur because of the steep hydraulic gradient through it. It was, therefore, decided to align the spur downstream of the present mole-head, in the position CDE shown Fig. 45/11- in Fig. 45/11-vi, of course after removing the mole-head as in experiment 4. A very

flat nose sloping at 1 in 10 was provided to reduce the scour; the groyne made an angle of 60° to the upstream bank.



PART II

Conditions markedly improved both as regards flow and velocity distri-Figs. 41(b)/ bution. For example, the maximum velocity was reduced from-11-vi. and 46(b)/11-vi.

1.87	ft./sec.	to 1.48 ft./sec.	at Section AA	
1.59		1.44	BB	
1.73		1.09	CC	

The fast moving current was also deflected towards the right bank.

Experiment 6: (with a shorter impermeable groyne and without mole-head)

In experiment 5, it was noticed that the throw-off of the current to the opposite right bank downstream of the nose was too severe. So, in this experiment, the length of the nose was reduced from 500 ft. to 300 ft. and the alignment of the nose was slightly altered, so that it made an angle of 70° upstream instead of 60° as in the previous experiment, vide CDD'E' in figure 45/III-vi. Fig. 45/II-As it was found that gulleting⁽¹⁾ would occur, if the slope were flatter than vi As it was found that guileting would occur, it the depth of water 1 in 3, the nose was given a slope of 1 in 3 down to 1/3rd the depth of water (i.e. in a length of 30 ft.) and the remaining 170 ft. length was given a slope $F_{ig.47(c)/}_{II-vi.}$

The throw-off was reduced, while, at the same time, the satisfactory Fig. 47(a) conditions upstream, both as regards the flow and velocities, were maintained. and (b)/11. The backwater effect due to the spur would extend beyond the Sharabi nala vi. and would thus help an early cut-off. The afflux upstream of the spur was found to be equivalent to 3 ft.; while it was $\equiv 2$ ft. at the mouth of the Sharabi Nala.

It is extremely important to protect the mole-head (nose) of the new spur as well as the portion of its shank facing the existing scour-hole with a pitching and a falling apron. Protection must be provided on the downstream as well as the upstream side of the new spur; the mistake made in the previous years in not providing sufficient protection on the downstream side of the shank should be avoided, as there will be some action on the downstream side, though in a repelling spur it should not be so severe as on its upstream side. As the spur faces upstream and is designed to have little scour at its nose, in this position it will act as a repelling spur and will not attract the flow to the left bank.

If constructed wholly of stone (for convenience of construction), the section of the spur could be made lighter. For testing the width and depth of the apron required, experiments were done in a geometrically-similar model, as described in Part II of this note.

Experiment No. 7: (with one long permeable tree groyne)

As it may not be possible to collect the material required for adequate Figs. 46(a) protection of the spur or complete the work before the next flood season, an and id (b)/IIprotection of the spin of complete allo work block the here (800' + 650' + vi)alternative suggestion of a permeable tree groyne, 1,650 ft. long $(800' + 650' + 200' a \log DE)$ was also found necessary. Tree groynes have been found very rig. 45/11. successful in Sind in repelling the attack, vide para. II-vi (6). The leaves, bransuccessful in Sing in repeating the astack, *vice* para. 11-v1 (0). The feaves, bran- vi. ches, thorns, and twigs trap the coarse sand moving along the bed as well as hold back the fine material in suspension which cements the trapped bed sand, s of this causing accretion both upstream and downstream; and, eventually, the sand- year's A. Bound spur becomes impermeable. Due to model limitations, however, in the model water continued to flow through the tree groyne, though with reduced Para. 1.3. velocities; it is expected, however, that the tree groyne would be much more effective in the view that we have be the model effective in the river than shown by the model.

Experiment No. 8: (with short tree groynes)

In case neither of the above two suggestions, viz. a long impermeable groyne or a long permeable tree groyne, are practicable in the time available, groyne or a long permeable tree groyne, are protocorto in the standard, a third alternative of short permeable tree groynes was suggested. Short tree Figs. 49 (a groynes were tested in experiment No. 8. The best alignments, positions and b)(II. in and lengths of these groynes were determined in the model, as shown in Fig. 50/II. vi. 50/II-vi.

⁽¹⁾ C. I. and H R. Annual Report (Tech.) for 1939-40, pp. 18, 19. 8 41-19

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Further, in this experiment, a short attracting impermeable groyne, 500' long, facing 45' downstream, was fixed at Z, to intensify bank erosion upstream of the vi. Sharabi Nala and accelerate the occurrence of a cut-off. Though the model vi. Though the model indicated more intensive action in a general manner, it could not quantitatively this year's reproduce the expected bank erosion for obvious model limitations see para. 1.3.

The short tree groynes were found successful in repelling the high velocity flow away from the left bank, while at the same time reducing the velocities Fig 49 (a) & both on the upstream and downstream sides of the groynes. (b)/II.vi.

It must, however, be made very clear that the tree groynes are suggested merely as a temporary protection of the left bank, for one or two seasons only, for the immediate purpose of avoiding the extension of the bank erosion up to the Ring Bund of the Station Yard. These groynes by themselves are little likely to accelerate a cut-off, as they have only a limited throw-off; but they will certainly be better than nothing, till a long impermeable (stone) or permeable (tree) groyne can be properly constructed, with the necessary protection.

Fig. 50/II-vi

A short "attracting" impermeable spur at ZZ was also recommended as it promises to be helpful. It will cost little and can be quickly constructed; and by virtue of the scour-hole it will create at its nose, on account of the steep slopes to the nose, (say 1 in 2), and by virtue of its downstream alignment and position along the meander, it should attract the river u/s and d/s of it, thus accelerating the cut-off.

Fig. 39/II-vi.

Experiment No. 9 (with a cut-off channel along XY shown in Fig. 39/II-vi)

A qualitative experiment was done with a pilot cut along XY to observe the varying percentages of discharge that can be carried by such a channel as it tends to develop. In practice, a cut-off channel develops more by retrogression from the downstream end; as these conditions could not be reproduced in the model, artificial deepening of the bed at downstream end was done; and it was found that the cut which originally carried only 5 per cent of the discharge, soon increased to 50 per cent, thus justifying the assumption that XY is about the most favourable alignment for a cut. This tendency of rapid development was not evident when the same procedure was adopted with the Sharabi Nala. Actually, in the river the development would be more rapid with a pilot cut along XY, because a large portion of sand-free surface water would break through the neck of the loop while the loop itself would get rapidly accreted with the remaining flow moving with a lower velocity and carrying almost all the bed sand. For obvious reasons, model limitations preclude the simulation of this cutting-back effect.

PART II-EXPERIMENTS WITH A GEOMETRICALLY-SIMILAR MODEL FOR THE DESIGN OF THE NOSE OF THE IMPERMEABLE GROYNE.

Design of the nose suggested in experiment No. 6 was tested for stability in a 1/80 geometrically-similar, part-width model of the Sutlej. Part of the groyne and of the nose were reproduced in the model. Curvature of flow at the nose—as obtained in the vertically-exaggerated model—was imposed by altering the entry conditions suitably and a discharge $\approx 250,000$ cusecs (the maximum discharge) was run. The following designs were tested:—

(i) Slope of nose (longitudinal) 10:1, side slope 3:1. Though such flat slopes reduce the depth of scour at the toe of the nose, the stones from such flat slopes are less stable and have a tendency to be easily washed away, thus forming a "gullet".

With the above slopes, heavy gulleting occurred in the model along the 3:1 upstream side slope, as well as at the toe, and stones were washed down-stream.

(ii) Slope (longitudinal) 5:1, side slope 3:1.

The action at the toe was much reduced. Gulleting still occurred along the upstream face of the nose.

(*iii*) Slope (longitudinal) 3:1 from R. L. 380 to R. L. 370 and 5:1 from R. L. 370 to R. L. 355. Side slope 2:1.



0. P2. P. POONA , 1947.







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This design was found to be quite satisfactory as no gulleting occurred either along the side slope or at the toe, for 250,000 cusecs flood discharge.

Recommendations for the protection of the nose of impermeable groynes :---

Fig. 51/II-vi shows the plan and cross-sections of the proposed nose. The ^{Fig. 51/II-vi.} entire nose and 30 ft. of the shank should be made of "one-man-stone", i.e. stones weighing not less than 70 to 90 lbs.

The proposed nose be given a longitudinal slope of 3:1 down to R. L. 370 and 5:1 below.

As 2:1 side slope did not show any tendency of gulleting in the 25 ft. portion of the stone shank, a steeper side slope be given for economy, viz. $1\frac{1}{2}:1$.

The portion of the nose below R. L. 370 be given a side slope of 2:1.

The depth of scour at the nose reached R. L. 310 and the scour-hole had an elongated shape. The Station generally recommends the stone to be laid conforming to the shape of the scour-hole—thus liming the scour-hole; but as this would be very expensive in this case, as it would involve excavating the scour-hole according to its shape and depth, a horizontal apron as shown in figure 51/II-vi was recommended. Some of the stones are likely to be washed Fig. 51/II-vi. away before the apron launches; so an extra provision of some 30 per cent stone was advised.

Due to the changed slopes, the length of the nose was reduced to 80 ft. instead of 200 ft. for the 10:1 sloping nose. In order to get the same effective Fig. 45/11-vi. throw-off, the toe of the nose should be at E' and, therefore, the length of the horizontal portion increased from 100 ft. (previously proposed) to 220 ft.

Final Recommendations

(I) An impermeable stone groyne along CDD'E' as shown in Fig. 45/II-vi. Fig. 45/II-vi.

(2) Alternatively, if it is not found possible to collect the material for completing the construction in the time available before the next flood season, a permeable tree groyne may be constructed along the same alignment.

(3) Failing (1) or (2), short tree groynes shown in Fig. 50/II-vi may Fig. 50/II-vi. be constructed merely as a temporary measure, pending the collection of material etc. required for the designs (1) or (2).

(4) An apron should be provided to the Ring bund of the Station Yard, only as an *additional* line of defence.

(5) In order to accelerate the cut-off channel across the loop, the entry of the coarse sand into Sharabi Nala should be prevented by constructing a bund at the head of the Nala with its top level at the levels of the adjacent banks of the Nala. This bund should be allowed to breach at three-fourths to high flood so as to deepen the Nala rapidly due to rush of sand-free top water entering it.

(6) A short impermeable groyne at "ZZ" shown in Fig. 50/II-vi may Fig. 50/II-vi also be constructed to increase the river curvature by intensive bank erosion upstream of it, thus accelerating the development of the cut-off.

upstream of 10, thus accelerating use determined is along the line XY: The most promising position of the cut-off channel is along the line XY: but with the measures (5) and (6), the maximum chance is given to the development of the Sharabi Nala.

APPENDIX I

(Scale ratios of the Sutlej model at Samasata.)

(a) River data:---

(1) Average width of river Khadir in the model

		9 07 E LL
reach.	•••	э,ото њ.
(2) Average depth		11 ft.
(3) Average maximum discharge		1,61,554 cusecs.
(4) Average surface slope in floods	•••	1 in 8,000
(5) Average bed slope		1 in 5,500
(6) Mean diameter of the bed sand	•••	0 [.] 17 m. m.
As regards item (3), the data was as follows :		

Hydrographs of the Sutlej at the Empress Bridge near Adamwahan are available for the years 1942 to 1946, from which the frequency curves were prepared. Analysis of these is given below:

TABLE	3,	/II-vi	
-------	----	--------	--

Year.		Days for which discharges were $>$					
	60,000 св.	80,000 cs.	100,000 cs.	Remarks.			
1941—42	49	33	21	Oct. to Oct.			
1942-43	40	32	20	Oct. to Oct.			
1943—44 1944—45	27 33	13	3	Oct. to Oct.			
1945-46	27	20	14	Oct. to Sept			

Maximum discharge in, 1942-238,017 cusecs.

1943-169,036 cusecs.

1944-104,974 cusecs.

1945-172,745 cusecs. 1946-123,000 cusecs.

Average maximum discharge = 161,554 cusecs.

(b) Model scales :---

Longitudinal scale: To get a model of workable size-neither too small nor undesirably unwieldy—scale of 1/250 was adopted.

Discharge scale: Where meanders are the essence of the problem, since M_L and M_B , meander length and belt $\infty \sqrt{Q}$,⁽¹⁾ for simulation of river meanders in model, Q scale should be = (Horizontal scale)⁸

$$\therefore$$
 Q scale = L³ = $\frac{1}{62,500}$

Vertical scale:-

Vsand (m=0.21 mm.) was used for laying the model and for injection. ... $f_m = 1.76\sqrt{m} = 1.76\sqrt{0.21} = 1.76 \times .457 = .805$ and $f_p = 1.76\sqrt{0.17} = .176\sqrt{0.17} = .176\sqrt{0.17}$ $1.76 \times 412 = .725$

(1) By Lacey formula, $d = 0.473 (Q/f)^{1/3}$; $d_{s} = \cdot 473 \left[\frac{1}{62,500} \times \frac{\cdot 725}{\cdot 805} \right]^{1/3} = \frac{\cdot 473}{41} = \frac{1}{86\cdot 5}, \text{ say } 1/87;$...V. E. $=\frac{250}{87}=2.88$

(2) By Manning's formula, assuming same "n" for model and river, since grades of sand are not much different;

$$d_{s} = L_{s}^{3/4} = \frac{1}{250^{5/4}} = \frac{1}{62}$$

$$\therefore \quad V. \quad E. = \frac{250}{62} = 4.02$$

(1) Page 10, Annual Report (Tech.) 1939-40 of I. W. E. Station.

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Slope scale:-

B

y Lacey formula, S. E. =
$$\left(\frac{t_m}{t_p}\right)^{43} \times \left(\frac{Q_p}{Q_m}\right)^{16}$$

= $\left(\frac{\cdot 805}{\cdot 725}\right)^{6/3} \times 62{,}500^{1/6} = 1{\cdot}146 \times 4{\cdot}3 \times 2{\cdot}16$

=7.9 or say 8 and was adopted for initial laying.

Discharge Cycle :---

To test the efficacy of various measures designed to expedite the development of a cut-off under the lowest possible flood conditions, 1944 hydrograph was selected for adoption in the model as being the worst.

A discharge cycle from this hydrograph, with time scale calculated according to tractive force method, was worked out. A hypothetical discharge cycle finally designed (see Table 4/II-vi below) was based on this and modified in light of the Table 4/II-vi experience gained at the Station, that low discharges are required to be run in models for longer periods and high discharges for smaller durations to counteract model limitations.

Model times.		Proto. discharges in cusecs.	Model discharges in cusecs.	Remarks.	_
(10 minutes		25,000	0.40	These should be run longer; b	ut
	15 minutes	40,000	0.64	they have httpe enect.	
Rising -	20 "	67,500	1.08	e -	
flood	12 ,	80,000	1.28		
	3 "	150,000	2.40		
	6 "	80,000	1.28		
Falling-	12 "	67,500	1.08		
flood)				

40,000

25,000

TABLE 4/II-vi

Sand charge:-

10 "

6,,

Sand charge required to maintain the bed slope and stabilise the model was determined from preliminary model runs.

0.64

0.40

APPENDIX II

Computations for discharge of Sharabi Nala with a flood of 150,000 cuseos flowing down the river.

Table 5/II-vi

Cross	H F. T. with	Sectional area	Average depth	Velocity	Discharge in cs.	Remarks.
Section No.	150,000 св.	in sq. ft.	111 16.	th address	[[
		1.093	9.95	5.2	5,675	Mean discharge
AA	573-90	1,000	9.95	5-2	8,275	=6080 cs.
BB	372.71	1,594	0.65	4.75	8,625	
1	872.15	1,812	8.00		5.875	
ç	371 - 1 3	1,356	7.53	4.04	0,010	
9	\$70.43	1,324	6.03	3.74	4,950)

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Nala discharges at each of the cross-sections were worked out by the Lacey formula, $V=165~R^{3/3}~S^{1/3}$ with S=2.83 ft. in 8,000, i.e. 0.35 per 1000.

Thus, Nala discharge with 150,000 cusecs in the river =6,680 cusecs =4.45 per cent or, say, 4.5 per cent of the river discharge.

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THE RUPNARAIN RIVER UPSTREAM OF THE GOODS YARD, KOLAGHAT RAILWAY Station (Upstream of the B. N. Railway Bridge), Measures to reduce risk of attack on the right Bank of-

Since the construction of the spur on the left bank, about 4,200 ft. 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 bend 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 local officers requested the Station for advice.

The present conditions were studied in the pilot model (horizontal scale=1/3000; vertical scale=1/3000). 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⁽¹⁾ 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 Fig. 52/II-vi, would reduce the attack, if any, and protect the river bank between XY--chainage 1,500' to 4,500' 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.

⁽¹⁾ For details of construction and launching of groynes please see paras. II-vi. 3 and II.-vi. 6.



· PART II

II.-vi-6 TRAINING OF RIVERS WITH PERMEABLE, TREE GROYNES

Introduction

Tree groynes have been very successfully used in Sind for river training, under suitable conditions, as will be seen from the note below :-

The Director, while he was in charge of Begari Canal and Development and Research Divisions, Sind, had personal experience of the method of construction and the behaviour of the tree groynes during and after floods. It is hoped that the following note will be useful in analogous river problems.

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 cun, 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 groupes—not only upstream but also downstream of the spurs; otherwise there is a danger of their being outflanked. 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 length 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 (acacica 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 have been recently recommended by the Station in the following cases :-

(1) In the case of Rupnarain, upstream of the Railway Bridge (B. N. Rly.) at Kolaghat, to prevent erosion of the Right Bank upstream of the Railway Bridge-see paragraph II.-vi. 5.

(2) In the Watrak at Kaira (Bombay Presidency) to prevent erosion and washing away of the area in Hariala village on Right Bank upstream of the newly constructed Road Bridge, vide paragraph II.-vi 3.

(3) In the Sutlej at Samasata (N. W. Rly.), para. II.-vi. 4.

(2) Was advised without experiments; while in the case of (1) and (3), model tests were actually made.

Note on tree groynes.

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.

Each of these types is described below :--

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.



Jucks (cross groynes) were constructed about 30 ft. apart. Each of these groynes consisted of strong vertical stakes ("munas"), driven about 5 ft. in the grounds, at 10 ft. 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), 1 ft. apart, were placed between the vertical posts. The entire juck-work was made watertight by lining the groynes, on their upstream side, by "lai" bundles, 2 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 7 ft.

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.

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Type (b)—Tree groyne for closing up one river channel and opening out another.

This type was constructed across the river at MN, 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.



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 "dhoro" (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 ft. width and widening of "dhoro" RP to about 400,- 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 "dhoro" owing to the existence of a deep "kumbh" (pit) at R.

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Anchorage consisted of two concrete piles, 100' apart, driven in series by jetting with water under pressure in the sandy katcha. It was hoped to sink these piles to 40 ft. depth but with the pumps available, they could only be taken to a depth of 24 ft., the tops thus being only 2 ft. 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 the sketch (fig. 55/II-vi) below. The trenches housing this anchor were filled up. The anchors were connected by 1-1/8'' cable with trees tied on to it, at 10 ft. intervals, as in case of a tree groyne. It was, thus, intended to prevent deflection of the current to the island NJ.



The groyne was fabricated at the edge of the stream because of the difficulty of keeping the cable stable across a stream 2,000 ft. wide, with semi-rigid anchorage, and tying the trees in situ. The tree groyne was made up of whole green babul trees, about 20 ft. to 30 ft. high, spaced 5 ft. apart, tied to a G. I. Cable 1-1/8" 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 10 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 groyne 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 (Figs. 56 and 57/1I-vi.). (See p. 84).

Figs. 56 & 57/, II-vi.

The site A B (see Fig. 56/II-vi) 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 CB. 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 ft. long was first fixed in cement concrete (1: 3: 6) block, $5' \times 5' \times 8'$, at point B in the island, 260 ft. 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 10 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 1" dia. rope made up of separate small lengths into one length, 1,850 feet, was first tried but snapped; and a new rope 1-1/8''diameter, 1,850 ft. 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 10 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.

Two floating buoys, connected to heavy anchors with strong iron chains 30 ft. long and shackles, were placed on the upstream side of the wire rope line, one 5 cwt. anchor 200 ft. away and another 1 ton anchor 400 ft. 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, $2\frac{1}{2}$ to 3 ft. girth, 25 to 35 ft. 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 *vhole* trees were carted on bullock carts with branches overhanging at the back, the carts being hauled by a team of bullocks and nen. 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 1" diameter, 2 ft. 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 10 ft. long, 3/8" dia., 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 ches, 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 one tree provision of one tree per 10 Rft. 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 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, strain on the rail block B in the island and on the winch, causing more and more 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 as to indicate that the river has silted at its source. Fig. 57/II-vi shows how successful the tree groyne has been in Fig. 57/II-vi 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 free 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 $10' \times 5' \times 3'$, 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 Rft. Babul trees about 3 ft. in girth and 25 to 30 ft. long are the best.

(iv) This kind of groynes can be successfully made in depths of water about 15 to 20 ft. It must have at least one rigid, unyielding anchor fixed in pucca soil.

(v) The cost of the groyne was Rs. 14 per Rft. in 1937-38.

III. FLOW IN RIVER AND CANALS.

ii Flow in channels in incoherent and coherent alluvium.

III. ii-1 FLATTENING THE SLOPE OF THE N. W. CANAL-SIND.

(Advice without experiments)

In order to retard the deterioration in water levels of the Indus river above the Sukkur Barrage it was suggested:^a

"In the case of the North West Canal, the full supply level can be lowered by excavating the bed and flattening the slopes in the main canal and Warah Branch. At worst, a small area of high level near the head of Warah Branch will be turned into lift. This, however, would be a small loss compared to the great advantage of allowing free flow through the Barrage for discharges in excess of 2 lakh cusecs, so as to retard or prevent for the state in the second further deterioration"

With reference to the above statement the Executive Engineer, Barrage Division, made an enquiry as to the flattest slope which could be recommended for the N. W. Canal and the Warah Branch.

From the available data of grade of silt entering the canals at the head, it was observed that the curved, approach channel generally excluded from the N. W. Canal, bed sand coarser than 011 mm. diameter giving Lacey f=0.58. If this were correct, according to Lacey's formula, with a designed head discharge of 5,047 cusecs, the canal should have a slope of 0.055 per 1,000 or 1 in

⁽¹⁾ Annual Report (Technical) for 1945.

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18,000, provided all coarse sand is first removed from the bed—a treatment similar to that recommended for the Rice Canal⁽¹⁾. Further taking $m \propto Q^{1/10}$, m in the Warah branch, with a designed discharge of 1,951 cusecs, would be 0.10 mm., i.e. f=0.56; and the slope, therefore, may be 0.065 per 1,000, or 1 in 15,400.

The canal at present already runs at a flat gradient, viz. 1 in 17,500, for a discharge of 5,000 cusecs; so the main trouble appears to be the Warah Branch which takes off at high level.

V. ACCURACY OF DIFFERENT METHODS OF TAKING DISCHARGES

V-1 DESIGN OF A BROAD CRESTED WEIR FOR THE RANGHOLI IRRIGATION SCHEME, BHAVNAGAR STATE.

The State Engineer approached this Station for the design of a broadcrested section for the Rangholi Irrigation Scheme weir which has 75 automatic gates, each $20' \times 5'$ in size. With 62,000 cusecs (maximum flood), the depth over the weir crest being 6 ft., the coefficient of discharge for these conditions worked out to 2.82, with negligible head due to velocity of approach.

A standard weir section having 1 in 5 upstream and 1 in 3 downstream slope with a flat crest 5 to 6 ft. long, enough for the gates to rest on, which with clear overfall gives a discharge coefficient of $3^{\circ}2$, was recommended with a cistern downstream to hold the standing wave which otherwise proves dangerous, causing scour and undermining.

V-2. DESIGN OF HIGH COEFFICIENT WEIR PROFILE (HYDERABAD, DECCAN)

The Director, Engineering Research Department, Hyderabad State, referred to this Station for a high coefficient weir profile which, while having the crest at R. L. 100 with upstream ground level at R. L. 99 and the downstream ground at R. L. 97, would give a discharge coefficient of the order of 4.5; with a head of 2' to 3'.

In this case, the ratio of the head over the weir to its height worked out to 2 or 3 according as the head was 2' or 3'; and hence weir profile No. $X^{(3)}$ which gave C=469 when D/H varied from 4.79 to 1.38 was suggested, provided the weir worked with a free overfall. It was made clear that fresh experiments would have to be done in case the flow conditions are drowned, or partially drowned.

V-3. DESIGN OF A 410 CUSECS STANDING WAVE FLUME FOR THE JAWAI RIVER PROJECT (JODHPUR)

The Chief Engineer, Jodhpur State, asked for a design of a 410 cusecs Standing Wave Flume for a lined canal having 24 ft. bedwidth, 5 ft. depth and 1:1 side slopes. The design recommended—based on Design of Standing Wave Flume⁽⁹⁾—is shown in Fig. 1/V.

Fig. 1/V.

Fig. 2/V.

V-4. WATER SURFACE PROFILES OBSERVED IN A 5' WIDE STANDING WAVE FLUME WITH PARALLEL SIDES FOR VARIOUS STAGES OF FLOW

1 Introduction

Information was sought by Mr. A. R. Thomas, Executive Engineer, regarding water surface profiles, observed in a long-crested, parallel-sided weir, when the downstream water level was raised through various stages, hypo-critical, critical and hyper-critical.

Since this information was not forthcoming from past records, it was decided to observe these profiles in the L. B. Flume over the 5' wide, parallel-sided hump. A longitudinal section of the hump is shown in fig. 2/V.

⁽¹⁾ Annual Report (Technical) 1945.

⁽²⁾ Annual Report (Technical) 1939-40, page 47.

⁽³⁾ Annual Report (Technical) 1944, para. 90.



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Mr. Thomas wanted the various profiles to be observed for $D_1 = \frac{L}{4}$, $\frac{L}{2\cdot5}$ and $\frac{L}{1\cdot5}$, L being the length of the crest in line of flow; actually, profiles have been observed for $D_1 = \frac{L}{10}$, $\frac{L}{6}$, $\frac{L}{4}$ and $\frac{L}{2\cdot5}$. The profile for $D_1 = \frac{L}{1\cdot5}$ could not be observed owing to the inadequacy of the flume to take the high discharge (q=8\cdot8 cs/ft. approx.). Even the profile for $D_1 = \frac{L}{2\cdot5}$ could not be observed under fully drowned conditions.

2. Discharges

The hump forming the flume had previously been calibrated against a 2' S. W. F. The formula derived statistically was Q=3.03 BD¹⁻³³. Consequently, for

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 $\left[\, D_{1} \! = \! D \right]$, as head due to velocity of approach was negligible]

3. Procedure

For each of the discharges, the downstream water level was gradually raised from clear free flow stage to critical stage when the upstream water level was just affected by the tail water. The downstream water level was further raised until the flow conditions were partly and fully submerged, in stages. Water surface profiles were observed at each stage in the centre of the flume.

4. Graphs and Figures

The profiles have been plotted on the same sheet for a particular discharge to show the various stages for easy comparison—Figs. 3 to 6/V. A very high Figs. 3 to exaggeration of scales has been adopted in order to be able to distinguish 6/v. different profiles.

Figs. 7 to 10/V show the graph showing the relationship between the Figs. 7 to percentage loss of head $\left(\frac{D_1-D_3}{D_1}\text{ per cent}\right)$ and the upstream head over the cill.

Figs. 11 to 14/V show relationship between the per cent drowning ratio and Figs. 11 to the per cent error in discharge. The error increases with the increase in the ^{14/v}. drowning ratio (see Table 1/v); the error up to 90 per cent submergence is, Table 1/v. however, small, viz. only 3 per cent. Above this limit it increases rapidly.

5. Results

The study of the mechanics of flow over a weir reveals that till the critical stage is reached, the upstream water level remains unaffected in spite of the rise in downstream water level; when, however, the critical stage is passed, the rise in upstream water level is fairly rapid with the rise in downstream water level and finally a stage is reached when the rise in upstream water level is the same as the rise in downstream water level.

An analysis of the results shows that the higher the discharge, the lower is the modular limit, when upstream water level is just affected by downstream water level.

With $D_1 = \frac{L}{10}$, D_3/D_1 (oritical) = 87.10 per cent. $= \frac{L}{6}$ = 86.50 per cent. $= \frac{L}{4}$ = 84.74 per cent. $= \frac{L}{2.5}$ = 85.13 per cent.

6. Position of Standing Wave

Between 'c' and 'e', the surface rises rapidly and is much disturbed by spraying and spattering. The whole mass is so milky as to suggest the presence



of much internal impact.¹¹ The milky condition of water reduces its specific gravity and accordingly the surface at the top of the rise is above the normal level; but as soon as all the air bubbles reach the surface so that the water is again transparent, the surface becomes smooth and level.

It must be remembered that the theory of pressure – plus – momentum is used in problems where the friction on the boundary layer may be neglected between two sections close together under conditions of horizontal motion. It cannot be applied to sections in which curvature is prominent, nor to sharply inclined flow unless a correction is made for the vertical component of velocity. Allowance has also to be made for other losses – e.g. loss due to shock in passing from the toe of the glacis to the flat bed of the channel, etc. The standing wave constantly oscillated backwards and forwards and caused fluctuations.

The actual profile will differ somewhat from the ideal owing to the following considerations $^{(2)}:-$

(a) Changes in stream lines cannot be abrupt on approaching a change of section;

(b) A change of velocity or a change of depth postulates a change in frictional losses. This change in frictional losses is neglected in calculations of ideal profile.

This shows that the actual position of the standing wave may-and probably will-be slightly higher up than the theoretical position.

7. Curvature of flow

The experiments ⁽³⁾ to investigate the curvature of flow in the throat of a standing wave flume showed that the pressures at the end of the throat were less than the corresponding depths of water. The negative pressure on the bed at the end of the throat, which is a measure of the convex curvature, apparently increased for higher values of D. This has been confirmed by the present series of experiments also. Last year's experiment also proved that at the entrance there was little difference between depth of water and corresponding pressures, even for $D=1^{\circ}2.FS.D$. The curvature at the entrance of the throat is thus less pronounced.

Table 1/v.

The effect of this convex curvature at the end of the throat is to reduce the value of the critical depth (see Table 1/V) at the downstream end of the throat and to make the critical depth occur at a certain distance further upstream.

As explained by Bakhmeteff ":

"Experiments show that in actual movement over a broad-crested weir, the critical depth $d_{er} = q^{2/3}/c^{1/3}$ is attained by the surface at a certain

(4) "Hydraulics of open channels", by Bakhmeteff, p. 43.

^{(1) &}quot;Design of Weirs on permeable foundations" by A. N. Khosla, I.S.E. (page 159).

⁽²⁾ Energy of flowing water, Punjab Engineering Congress, 1929.

⁽³⁾ I. W. E. Stn., Poons, Annual Report (Tech.), for 1945.

si 41−23	Table showing per cent error in discharge and per cent drowning ratio for various values of D_1																			
	$D_1 = \frac{L}{10} = 0.5'.$ $D_1 = \frac{L}{6} = 0.5'.$								$D_1 = \frac{L}{4} = 0.76'$.					$D_1 \simeq \frac{L}{s \cdot s} = 1 \cdot 2'.$						
	D _{1 ft} .	Q = 3.03 BD 1.53 cusecs.	Actual Q.	per cent error.	D3/D ₁ per cent.	D ₁ ,	$\begin{array}{c} Q = \\ 8.03 \text{ BD}^{1.53} \\ \text{cusecs.} \end{array}$	Actual Q.	per cent error.	D _∂ /D _i .	D ₁ .	$\begin{array}{c} Q = \\ 8.03 \text{ BD}^{1.59} \\ \text{ousecs.} \end{array}$	Actual Q.	per cent error.	^D 3/ ^D 1.	D ₁ .	$\begin{array}{c} Q = \\ 3.03 \text{ BD}^{1.58} \\ \text{cusecs.} \end{array}$	Actual Q.	per cent error.	^D ₃ / ^D ₁ .
	0.300	2.40	2.40		87-0	0.500	5-246	5.246		59 .0	0.75	9.767	9.757		78.12	1.2	20.05	20.05		61.66
	0.800	2.40	2.40		46.67	0.500	.,	,, ·		86 · 5	0.754	9-885		0.8	84.74	1.203	20.09	"	0.2	85.13
	0.302	2.428	2.40	1.14	87-10	0.506	5-248		1.85	88 - 72	0.759	9.933	,,	1.81	85 - 39	1.207	20.2 0		0.75	87.92
	0.902	2.428	, ,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,	1.14	88-74	0.507	5.359		2.16	89-74	0.763	10.001	"	2.54	87.82	1.217	20.46		2.05	89.74
	0.806	2.475	ļ "	8.08	91-89	0.518	5-455		4.00	90 .55	0.772	10.17		4-23	89.52					
	0.819	2.613	н	8.85	91.85	0.516	5-504		4.99	- 90-44	0.782	10.40	"	6-58	89.65					
	0.828	2.657		11 -10	93.19						··· _									

TABLE No. 1/V

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 $[D_1 = D, as head due to velocity of approach was negligible.]$

PART II

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distance before the edge of the weir and the depth d_{er} over the edge is somewhat smaller. The explanation lies in the fact that the critical somewhat smaller. The explanation due the assumption of parallel flow. depth $d_{er} = q^{2/3}/g^{1/3}$ was determined under the assumption of parallel flow. In other words, $d_{er} = q^{2/3}/g^{1/3}$ corresponds to flow with a possible minimum content of energy in parallel flow.

For curvilinear flow, the minimum possible content of energy corresponding to a discharge q differs from content (minimum) corresponding to parallel flow. It is greater for concave flow and smaller for convex movement. Also the critical depth determined as the depth at which flow with minimum content of energy takes place is smaller than $q^{9/3}/g^{1/3}$ for convex streaming and greater than $q^{9/3}/g^{1/3}$ for concave flow ". The water profiles at modular limit bring out the following points:--

With an increase in upstream depths,

(i) the profile became more curved;

(ii) the theoretical 2/3rd depth point came closer to the entrance of the throat (see also col. 4 of Table 2/V);

and (iii) the ratio of minimum depth at end of throat \mathbf{D}_1 decreased and the divergence from the theoretical value (=2/3) increased (col. 5 of Table 2/V).

Conclusions

The experiments clearly showed the existence of curvature of flow.

The curvature at the end of the throat increases for higher values of D. This is in agreement with experiments carried out during the last year, which also showed that the length over which the flow remains parallel in the throat, depends on D and decreases with increase of D.

TABLE No. 2/V

WATER SURFACE PROFILES IN 5' HUMP

Table showing actual depth of water observed on centre lines as compared with D₁ upstream over the cill.

D=depth u/s over the cill in ft.	Depth of water at entrance to the throat in ft. 2	2/3 D ₁ ft.	Position where 2/3 D, of flow was observed measured from the, u/s end of throat in ft. 4	Depth of water at the downstream end of the throat in ft. 5	Drowning Ratio (Modular limit).
	/			·	
$\frac{\mathrm{L}}{10}=0.3$	0.229	0-200	2.82	$0.195 = 0.65 D_1$	87.10%
$\frac{L}{6} = 0.2$	0.370	0.333	2.41	$0.310 = .62 D_1$	86 50%
$\frac{L}{4} = 0.75$	0.540	0 500	1.37	$0.452 = .603D^{1}$	84.74%
$\frac{L}{2.5} = 1.2$	0.880	0.800	1.12	0.714 = 0.595 D	85-13%

 $(D_1 = D_1)$ as head due to velocity of approach was negligible.)

Table 2/V

PART II

XI SPILLWAYS

XI-1. PREVENTING SCOUR BELOW GANESH IVER VOLUTE SIPHON SPILLWAYS BY MUTUAL IMPACT OF JETS ISSUING FROM ADJACENT SIPHON OUTLETS.

(1) Synopsis.

In the high head dams, proposed to be constructed in various post-war schemes, Ganesh Iyer Volute Siphons will suggest themselves for adoption for excess flood disposal. They have a better discharging capacity and a lower priming depth as compared to ordinary saddle type siphon spillways. 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.



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Where rock or inerodible 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.

(2) Object of the experiments

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.

(3) 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 Hirebhasgar siphons. The only change made was in the alignment of the horizontal outlets; thus, in experiment 1, they made an angle of 18° and in Experiment No. 2 of 30°.

Experiments could not be started earlier as the essential details of the volutes were received from the Mysore P. W. D. only by the end of December.

(4) Procedure.

The bed downstream of the spillway was laid in shingle at level equivalent to R. L. 1690 to a depth of 255 ft. 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 2 it. above the crest of bellmouth (M. F. L.), while water level in the river downstream was maintained at R. L. equivalent to 1707, or 17 ft. 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 ft.

(5) Experiments.

Experiment No. 1: (Jet angle=18° and bed material=17.31 mm.)

Fig.2/X [.

Fig.1/XI.

In this experiment, the jets were made to hit at an angle of 18° . Coarse shingle (mean dia. = 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 ft. 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 ft. only).

Experiment No. 2: (Jet angle = 30° and bed material = $17\cdot31$ 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 Fig. 3/XI. experiment no. 1, viz. m = 17.31 mm. Figure 3/XI 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. Photo 5087 shows the model with jets hitting at an angle of 30°. Ganesh Iyer Volute siphons-Dissipation of energy by jets.



5087. Photo (1/24 scale Volute siphon model) Showing flow conditions near the siphon dome and jets issuing from the outlets hitting at an angle of 30°. Looking from top.





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Experiment No. 3: (Jet angle = 30° and bed material = 6.02 mm.)

In this experiment, the bed material was changed from 17:31 mm. dia. to 6:02 mm., to see the effect of the grade of bed material on scour. It has been found elsewhere⁽¹⁾ that scour-holes in such cases is 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). The depth of scour was 2:4 feet, as compared with 2:3 feet in experiment 2 with m = 17.31 mm. This point requires further investigation. Figure 4/XI shows the longitudinal profile of Fig. 4/XI. 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 jets hitting at angle of 30° in Experiment No. 3, the bed material being the same. Figure 5/XI shows the profile of scour-hole. The maximum depth of scour Fig. 5/XI 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.

(6) 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

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⁽¹⁾ Inst. O. E. Paper No. 5127 (1938-39) on "Protection of dams, weirs, and sluices against scour" by R. V. Burns and C. M. White.

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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.

Further Experiments

Experiments with siphon having outlets tilted upwards, so that the jets travel upwards before hitting each other and experiments where the out-going jet is fanned out before incidence of the jets, are next on the programme; after which experiments will be done with revolving jets, singly and impacting against each other.

XVI. TUBE WELLS.

(ii) YIELD IN RELATION TO SUB-SOIL.

XVI-(ii)-1. 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 SINDRI.

In September 1945, the Superintending Engineer, Public Health Engineering Department, Bihar, approached this Station in connection with the Project for water supply to the Government of India Fertiliser Factory, Sindri. The depth of sand in the river bed was reported to be approximately 20 to 25 feet and general information on the water yielding capacities and transmission coefficients was asked for.

A study of the data gave results as explained below :---

Fig. 1/XVI-ii

Figure 1/XVI-ii shows three typical distribution curves of sand m = 0.89; 0.90 and 1.07 mm., used for finding out the transmission coefficient.





The apparatue used for finding out the transmission coefficient of Sindri sand consisted of a 3" dia. steel pipe with holes drilled for insertion of glass tubes. The sand was compacted in the cylinder to a dry bulk density of 975 lbs./Cft. The ends of the pipe were closed with wooden covers and it was made air-tight by sealing the covers with plaster of Paris. Glass tubes were fitted as shown in Figure 2/XVI-ii. Water was made to flow through the sand Fig. 2/XVI-ii column under three different heads and was collected at the other and for a column under three different heads and was collected at the other end for a Transmission coefficient was calculated as below:---known interval.

The value of K, the transmission coefficient, is expressed as the velocity of flow through the soil under the action of a unit hydraulic gradient (i = h/l = 1).

It is obtained from the formula-

$$Q = \mathbf{K} \times \mathbf{h}/\mathbf{l} \times \mathbf{A}$$

where Q = quantity of water in c.c. percolating in one minute,

h/l = hydraulic gradient,

A = Cross-sectional Area in sq. cms. of soil column,

and K = Velocity of water in cms/min.

= transmission cofficient (in cms./min.)

Table No. 1/XVI-ii shows the results and the mean value of K obtained. The mean value of K is 2.2 cms./min., or 0.072 feet/min.

No.	D. B. D. of sand compacted in cylinder.	Quantity of water collec- ted in C. C.	Time required in minutes.	Quantity of water perco- lating in c. c./min.	Tempe- rature of water degree C.	h/1.	A. Sq. cms.	K cms./ min.	Mean value of K in ft./min.
1		222-0	7	31.5	24.5	0.37	52.7	2.21	
		220.0	7						
2.	1.52 gms./c. c.	210.0	8.	70.01	94.5	0.583	52.7	9.27	0-072
	97.5 lbs. per oft.	211.0	8	,					
	l	222-0	4	55-6	94.5	0.501	52.7		
9		213-0	4	0010		2 501	mean = 2.20	2.11	

TABLE No. 1/XVI-ii

(b) Schoklitsch's⁽¹⁾ formula for determining the transmission coefficient from yield test of a tube well is

$$z_1^2 - z_g^2 = \frac{Q}{\Pi K} \log_e \frac{X_1}{X_g}$$

where x_1 and x_2 = horizontal distance in feet from the centre line of tube well,

 z_1 and z_2 = heights in feet of hydraulic gradient line above the bottom of strainer at x_1 and x_2 ,

Q = Discharge of tube well in cu.ft./min. for a particular depression,

and K = Transmission coefficient in feet/min.

Transmission coefficients were calculated by the above method from the yield tests of tube well No. 3, for three different yields; the corresponding cones Fig. 8/X VI.ii of depression are shown Figure 3/XVI-ii.

"Hydraulic Structures. Volume I", by A. Schoklitsch The American Society of Mechanical Engineers" Publication.



No.	Yield of tube well (gallons/ hour.)	Yield in Cu. ft./minute.	Z ₁ (ft.)	Z ₂ (it.)	x 1 (ft.)	*2 (ft.)	$ \begin{array}{c} \mathbb{K} = \underbrace{\mathbf{Q}}_{\Pi} \operatorname{Log}_{\theta} \frac{\mathbf{x}_{1}}{\mathbf{x}_{2}} \times \frac{1}{Z_{1}^{2} - Z_{2}^{2}} \\ \text{it./min.} \end{array} $	K from laboratory experiments.
1	2,000	5·35	15·8	16-2	5	15	0·146	0.072 ft./min.
2	9,500	25·4	15·4	15-9	10	20	0·358	
3	13,500	86·1	15·0	15-58	10	20	0·507	

There is an increase in the value of K for greater depressions. The upper stratum up to 12 ft. depth consists of sand of more or less uniform porosity; below that depth the stratum is more porous due to the presence of pebbles, boulders, etc. For lower values of depression the water passes through the more porous, lower as well the upper sandy strata; but for the higher depressions more water passes through the lower strata. This is clearly shown in Fig.s/XVI-Figure 3/XVI-ii. Due to this, there is an increase in value of K for higher ii.

On the whole the values of transmission coefficients obtained from yield test in the river are higher than from the experiments done at the Station, probably due to the presence of larger particles at site.

Temperature of water has a large effect upon the transmission coefficient. Schoklitsch gives the following formula for evaluating "K" at standard tempera-

$$K = K_{10} (1 + .0845 [T - 10] \sqrt[6]{d_e})$$

= transmission constant in cms/min. at temp. T, where K TZ

$$u_{10} = ", ", ", ", 10^{\circ}C,$$

= Temperature of water, degrees C, T

and d_e = effective size of grain such that 10 per cent of the particles by

weight are smaller and 90 per cent larger than this size in cms. =0.025 in our experiments.

Assuming 20°C as the standard temperature of water under Indian conditions, the formula may be written as : -

$$K = K_{20} \left(1 + 0.0745 \left[T - 20 \right] \times \sqrt[6]{d_e} \right)$$

Value of transmission constant from Laboratory experiments reduced to 20°C works out to 1.89 cms/min., thus :

$$2\cdot 2 = K_{20} (1 + 0.0745 \times 4 \cdot 5 \times 0.478)$$

= K_{20} (1.161)

 \therefore Transmission constant at 20°C=2.2/1.161

=1.89 cms/min.

The values of K, obtained from yield tests, should also be reduced to standard 20°C for proper comparison. The difference in values of K in laboratory and field may partly be due to higher temperatures of water in the yield tests. The temperature of water should, in future, always be observed in yield or recuperation tests.

The yields of the tube wells appear fairly good, being about 500 gallons/hr. per square foot of strainer area for 8.7' depression. The yield can be further increased by using Ashford strainers, thus safely increasing the depression.

Yields of some of the tube wells on Zhob valley Railway⁽¹⁾ are given in the following table for comparison :--

Station.		Pore of allotted screen.	Longth of allotted screen.	Yield gallons per hr.	Drop.	Yield gal.,hr. p :r sq. ft. of screen area.	Yield gallous hour per sq. it of screen/ area for 10'-0" drop.	
Quetta Cantt.	'_ l	10"	13'-6"	18 500	35'-0"	400	114	
Rohtak		6″	6'-0"	5,000	20'-0"	530	275	
Delhi		. 10"	5'-5″	19,100	10'-0"	1,400	1,400	
Campbellpur		10"	20'-0"	12,300	22'-0"	246	110	

TABLE 3/XVI-ii.

The discharge of the Damodar river at Sindri has varied from 15 to 50 cs. while the above tests were being carried out. The original subsoil water level in the tube wells was only 1'' lower than the free water surface of the river. Every year, in the dry season, the discharge of the river is very low, often below 5 cs. for long periods. It is possible that the yields of the tube wells may be affected in such long periods of low river discharge.

^{(1) &}quot;Tube wells on Zhob valley Kailway, Baluchistan" by J. Varion, Executive Engineer, N. W R. Labore.



No.	Yield of tube well (gallons/ hour.)	Yield in Cu. ft./minute.	Z ₁ (ft.)	Z ₂ (ft.)	*1 (ft.)	×2 (ft.)	$K = \frac{Q}{\Pi} \log_{\theta} \frac{x_1}{x_2} \times \frac{1}{Z_1^2 - Z_2^2}$ ft./min.	K from laboratory experiments.
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PART II

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= transmission constant in cms/min. at temp. T, where K

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= Temperature of water, degrees C, т

and $d_e =$ effective size of grain such that 10 per cent of the particles by weight are smaller and 90 per cent larger than this size in cms.

=0.025 in our experiments.

Assuming 20°C as the standard temperature of water under Indian conditions, the formula may be written as : -

$$K = K_{20} \left(1 + 0.0745 \left[T - 20 \right] \times \sqrt[6]{d_{a}} \right)$$

Value of transmission constant from Laboratory experiments reduced to 20°C works out to 1.89 cms/min., thus :

$$2^{\cdot 2} = K_{20} (1 + 0.0745 \times 4.5 \times 0.478)$$

= $K_{20} (1.161)$

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=1.89 cms/min.

The values of K, obtained from yield tests, should also be reduced to standard 20°C for proper comparison. The difference in values of K in laboratory and field may partly be due to higher temperatures of water in the yield tests. The temperature of water should, in future, always be observed in yield or recuperation tests.

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Yields of some of the tube wells on Zhob valley Railway" are given in the following table for comparison :---

Station.		Fore of allotted screen.	Length of allotted screen.	Yield gallong per hr.	Drop.	Yield gal.,hr p :r sq. ft. of screen area	Yield gallous hour per sq. it of sercon/ itea for 10'-0" drop.
Quetta Cantt.		10"	13'-6"	13 500	85'-0"	400	114
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(1) "Tube wells on Zhob valley Railway, Batuchistan" by J. Varion, Executive Engineer, N. W M. Labore. S 41-25 .

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A yield test, carried out when river discharge is very low over a long period will give the minimum yield that can be relied opon. Such tests should be don if it is possible. Even a recuperation test will give useful data from which probable yields can be worked out.

(c) The size-distribution curves of the fine silt and sand deposited after the floods at various points near the proposed site of the infiltration gallery are $F_{18,4}/XvI$ - shown in Figure 4/XVI-ii. Even a layer, a few inches thick, of such fine sand-of mean diameter 0.05 mm.-deposited on the coarse bed sand of the river would cause a decrease in the rate of percolation to the infiltration gallery. as happens on the Sabarmati at Ahmedabad.

The Ahmedabad Municipality has constructed an infiltration gallery in the bed of the Sabarmati river for city supply. After the floods, there is a deposit of fine silt on the river bed and much of the percolating water through the bed is cut off. The Municipal Engineer in his letter, dated 16th November 1940. wrote :--

"The present river bed acts as a filter bed and the top surface occasionally gets choked up, mostly at the beginning or at the end of the monsoon when flood water gets suddenly reduced with a break in rains. The silt deposit is being drawn to the pores of the sand and the supply is suddenly reduced to about 5 lac gallons per hour (usual 12 lac gallons/hr.)

"Every time this happens, the top layer is required to be removed from the channel feeding the wells. From March to May when the discharge in the river is at its lowest, moss is formed in the channel and it exactly behaves as silt, as mentioned above."

Similar precautions will be necessary in the present case.

XXI. EXCLUDERS AND EJECTORS

DESIGN OF LOWER SIND BARRAGE AT KOTRI FOR EFFECTIVE XXI. 1. SAND EXCLUSION FROM CANALS

(1) 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, the present Director-while in charge of the Lower Sind Project Division-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.

Fig. 1 (a)/XXI shows the site of the barrage, tentatively proposed by the Project Division, 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.

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(2) Model scale ratios

Model scale ratios are given in Appendices I and II.

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(3) 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 C. S. $5\frac{1}{2}$ to 7 which indicated either an incorrect survey in this reach or the existence of hard, incrodible 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 following data were observed during the experiments :-

(i) Lines of flow were observed for $Q \equiv 2$, 3, 4 and 5 lakhs cusecs, Figs. 1 (a)/ of which Figs. 1 (a)/XXI and (b) show the lines of flow for $Q \equiv 2,00,000$ and $\frac{XXI}{(b)/XXI}$

(ii) Figs. 2 (a)/XXI, (b), (c) and (d) show isovels for 2, 3, 4 and 5 lakh $\frac{\text{Figs. 2}(a)}{\binom{b}{(d)}(NXI)}$ es at the proposed barrage site. cusecs at the proposed barrage site.

(iii) Fig. 3/XXI is a graph showing the percentage of the total river F_{1g} $_{3/XXI}$ discharge carried by the right channel, as observed in the model and the river.

These data indicate the following :----

(a) The lines of flow showed that the proposed barrage line was somewhat 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 C. S. 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 discbarge-, and (ii) 19,500 cusecs, the maximum discharge under the post-barrage conditions: but some seeds dropped at C.S. $5\frac{1}{2}$ entered the canal for low river discharges (2 lacs 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 Barragewas good.

(4) 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 = 2,00,000 cusecs (the minimum discharge at which bed movement occurs in the river) was run in the model-with upstream pond level at R. L. 680 (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 C. S. 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;

(c) the relatively excessive draw-off of the L. B. 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.

(5) Measures tested for improving sund exclusion out of Fuleli.

(5.1) 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 Fig. 4/XXI. bank so shaped upstream of the Canal—see Fig. 4/XXI 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 bedsand 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.

Fig. 5/XXI.

(5.2) It was then considered that a wholly concave alignment of the guide bank, in line with the curvature d/s of the barrage, so that the whole made one curve would eliminate at least the avoidable of the two drawbacks. Though partially successful, and better than (5.1), exclusion was still unsatisfactory, indicating that an artificial increase of local curvature would not in itself be sufficient.

(5.3) A raised crest at the entrance of the approach channel to the Fuleli, Fig. 6/XXI. 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.

(5.4) 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 make it symmetrical and pull the main flow towards the left bank, thus boosting up velocities along that bank. But this did not remove the trouble from the Fuleli.

Fig. 6/XXI.

(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 Fig. 6/XXI 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' [see Fig. 6/XXI island (c)]. But this had results contrary to expectations. A larger island would not materially improve exclusion 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.







(55) 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 $V_{\text{Biver}}/V_{\text{Pocket}}$ 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)⁸ and the time required to scour the pocket to L⁴--while actual experiments⁽¹⁾ have shown the theoretical approximation of proportionality to L⁴ 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_B/V_P 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.

(5.6) 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.

(5.7) It had 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 2,00,000 cusees—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—2,00,000 and 2,50,000 cusees—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 progressively decreased from the divide wall outwards) did not yield sufficiently satisfactory results;

(ii) nine spans just outside the divide wall with=4' 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. 9 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.

(5.8) The effects of a concave, submerged vane pointing d/s, located at different points, upstream of the Fuleli canal, with the top of the vane at 2/3rd \mathbf{Fig} e/XXI. the depth with ultimate pond level (R. L. 68), were also tested.

⁽¹⁾ Page 2, pars. 9, Bombay Tech. Paper No 45 on Silt Exclusion from Canals, "Still Fond" versus "Open Flow".

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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 Fig. 6/XNI A, 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. J Its optimum alignment is shown in Fig. 6/XXI, its length being 1250', and radius 2,687 ft.

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.

(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 back. 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 canal, 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 silting 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 $Q \equiv 500,000$ 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 avron protection is required to prevent scour occurring as a result of rotary flow and to allow the bed flow to follow the d/s face of the vane towards midstream. To save cost, a sharp slope is desirable; but a sloping apron not steeper than in 5, with a flat apron extending to a distance of 125° 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—

(a) 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;

(b) when the width of apron was extended from 125' to 175';

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 ft. apron would be sufficient there.

Thus, in this particular case, a vane could not be regarded as an all-time satisfactory solution.

(6) 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° upstream, parallel to the previous alignment, and the abutment be moved 600 ft. inwards (riverwards)—instead of 300' 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 where B, the left abutment, would be Fig 7/NNL taken 500 ft. riverwards from the pucca edge to obviate the necessity of crossing the Mukam area and D about 1,125 ft. from the original portion of the R. B. 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 $_{Fig. \theta/XNI.}$ in Fig. $\theta/XXI.$

(7) Results of the experiments with the new alignment (vide fig. 8/XXI):

When the different measures (described above in para. 5) were retested with this changed alignment of the barrage, there was no perceptible improvement in the sand exclusion from the Fuleli cfftake.

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 L. B. regulator.

When the worst discharge for exclusion, of 2,00,000 cusecs was run in the model, it was observed that the exclusion at Fuleli offtake 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 4' accretion above the mean level, are tabulated below :----

		Perc	entage in the righ					
Serial No.	Total discharge in the river.	without barrage.	with barrage (mean level d/s).	with barrage (4' accretion downstream).	Remarks.			
1	200,000	7.35	23.0	19.50	With no difference of levels in the two arms.			
2	່ <u>ສ</u> 00,000	10.80	21.0	16·0	3			
3	400,000	11-20	18 [.] 50	14.0				
4	500,C00	13.00	17	13:50	, , ,			

TABLE 1/XXI

[Note.-Sand exclusion was satisfactory from Pinyari and R. B. 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 2,00,000 cusecs forcing 23 per cent, etc., resulted in 6" difference in water levels between the two arms, at all the discharge stages. It, moreover, improved exclusion in the right arm.

(8) 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 Fig. sixxr. into the Pocket alongside the Pinyari Head Regulator; at the same time the arrangement of spans as well as their crest levels were altered to simplify the

Statement I/XXI.

design. Statement I/XXI sets out the salient features of the proposed and the revised designs, the latter as suggested by the Station.

A few quick tests were made with this revised design before recom-

(9) Merits of the simplified design

:e the main advantages arising out of the proposed changes

gn is uniform and simple with consequent advantages in intenance and operation;

Il be two more spans in the left pocket without adding to · of river and pocket spans; so that the 'still pond' limit will uring a longer period of ungated flow with consequent d postponement of, accretion;

T	ΔA	
т	$\mathbf{U} \pm$	

(c) the total lineal 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;

PART II

(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 3' on the left and 6' 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.)

10. Central Island for control and efficient exclusion on both 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, vide para. XXI-2—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 R. B. 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 $Q_{R} \equiv 2,00,000$ cusces — Bank Canals — i.e. a high Q_{P}/Q_{R} ratio at the worst stage of $Q_{R} \equiv 2,00,000$ cusces when the Fulell Canal was shifted into the Pocket, and brought within the influence of the Island the design gave full 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.

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 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 pitching more than offsets any tendency of the Barrage to cause deterioration of the right arm channel—see table 1/XXI. 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.

(11) Factors affecting the exclusion of sand from the canals

(11) Factors agreeding in the Kotri and Hajipur Barrage sites, have Model experiments, made for the Kotri and Hajipur Barrage sites, have focussed attention on the following factors as affecting the exclusion of

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sand from the offtaking 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 offtakes of the canals in relation to the curve; e.g., the Fuleli canal offtaking 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 riverward 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, Excludors 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 carals.

(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 — necessificating 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 bonk 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 arms by a Double Barrage reduce the Q_B/Q_P ratio in each arm, and admittedly, at low river discharges this is a drawback 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 divide wall of two-thirds this length is generally sufficient.

(1) 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.

APPENDIX I

SCALE RATIOS OF LOWER SIND BARRAGE MODEL

(1) Length scale

It was decided to adopt a length scale of 1/250 for the model with a view :----

(a) to accommodate the model in the area avilable;

(b) to get the maximum discharge of the prototype within the range of 7 to 10 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.

(2) Discharge scale

As this is model of a barrage, with curvature characteristics as a predominant factor-

Q scale = L^2 ; as $M_L \ll \sqrt{Q}$,

where $M_L = meander length$.

 \therefore 1 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 cs.-i.e. within the optimum range.

The bankful discharge is estimated to be 2,50,000 cusecs≡4.0 cusecs in the model.

It was observed that 1/62,500 discharge scale simulated conditions satisfactorily; hence this scale was adopted.

(3) Depth scale

(a) According to Manning's formula for regime conditions-

 $D = L^{3/4} = 250^{3/4} = 63$, i.e. V. E. = 4 (where L=Horizontal scale and D = Depth scale).

(b) Lacey's formula (for no difference in sand) -

 $D = L^{2/3} = 250^{3/3} = 40$; \therefore V. E. = 6.25.

The model was initially laid in the Sangam sand with m='18 mm., i.e. same as the prototype grade. It has been now observed that finer particles have been washed out and coarser sand with m=30 mm. has remained in the model.

Hence allowing for difference in sand we have according to Lacev :---

 $L_p/L_m = (m_p/m_m)^{1/4} (d_p/d_m)^{3/4}$.

 \therefore $L_p/L_m = (-18/30)^{1/4}$. $(d_p/d_m)^{3/2}$.

 $d_p/d_m = 45$ against 40 with same grade of material.

. V. E. = 250/45 = 5.5 against 6.25 with same grade of material.

(4) Slope scale

According to Lacey :--(a) S \prec f^{5/3}/Q^{1/6}. : S. E. = $(\underline{m_m/m_p})^{19/24}$. $(\underline{L_p/L_m})^{5/12}$. (V. E.)^{:74}. =6.33, with V. E. =6.25 and $m_m = m_p = 0.18$ mm. (b) S. E. = $\frac{(250)^{5/12}}{(5\cdot5)^{1/4}} = 8\cdot30,$ with V. E. = 5.5, $m_m = 0.30$ mm. and $m_p = 0.18$ mm.

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Thus, due to coarsening of bed material, while the V. E. (calculated) decreases from 6.25 to 5.5, the S. E. 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=2,00,000 cusecs to high discharge of 8.48 cusecs=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.

APPENDIX II

(a) Time scale for the Lower Sind Barrage model

The time scale was worked out according to tractive force formula :----

$$\begin{split} \frac{T_{\rm p}}{T_{\rm m}} &= \frac{L_{\rm p}}{L_{\rm m}} \cdot \left(\frac{d_{\rm p}}{d_{\rm m}}\right)^{1/2} \cdot \frac{S_{\rm p}}{S_{\rm m}} \cdot \\ \text{For the Lower Sind Barrage--} \\ & L_{\rm p}/L_{\rm m} = 250; \quad d_{\rm p}/d_{\rm m} = 50 \text{ and } S_{\rm p}/S_{\rm m} = 1/5 \\ & \frac{T_{\rm p}}{T_{\rm m}} = \frac{250 \times (50)^{1/2}}{5} = 350. \end{split}$$

i.e. 1 second in model=350 seconds in the prototype.

i.e. 4 minutes in the model=1 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—

1 day=8 minutes has been assumed for Q=150,000 cs.

1 day=1 minute for Q=500,000 cs.

and a smooth asymptotic curve is drawn through these three points for the time scale of other discharges.

(b) Silt charge scale

Fig. 9 (a)/XXI.

Fig. 9 (b)/XXI.

Data of silt charge of the river Indus at Kotri, for the year 1943, according to which a rising and a falling cycle is run in the model, have been analysed and show the following charge in parts per 1,000 for various discharges in the river:

Discharge in cusecs.	Charge in parts per 1000 in the river.	Charge in parts per 1300 injected in the model.			
200,000 250,000 350,000 350,000 400,000 450,000 500,000	$\begin{array}{c} 2:30\\ 2:90\\ 4:80\\ 3:00\\ 4:30\\ 4:30\\ 4:75\\ 4:75\\ 4:75\end{array}$	$\begin{array}{r} \cdot 115 \\ \cdot 145 \\ \cdot 190 \\ \cdot 200 \\ \cdot 215 \\ \cdot 238 \\ \cdot 238 \end{array}$			

It is assumed that 90 per cent of the charge in the river is in suspension and only 10 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.



No.	Item.	Project Division design of Double Barrage.							Proposed revised design of Double Barrage.						Remarks.		
		B. W (ft)	D	epth ft.)	f Slope.	F. S. L.	0. B. L.	Vel. (ft./sec.)	B. W. (ft)	Depth (ft.)	f	Slope.	F.S L.	C. B. L.	Vel. (ft./sec.)		
1	Design of Feeders.																
	Fulsli-cum-Guni	254	1	9.7 0	67 .0537	65.7	46-0	8.7	280	18.0	0.67	.0587	66.0	49.0	3 ∙7	Not absolutely essential to the revise design.	
2	Design of Head Regulators.	Off	ke.	Discharge (cusecs)	Spans. Nog.	Orest lev	zel L. H.	F. S. L <u>U/s</u> d/s	Offtak	. 8	pans.	Orest lev	al L.	н.	F. S. L. <u>u/s</u> d/s		
	Fulcli.cum-Guni	Jams	horo.	19,215	Existing	46.0	0.9	66 · 6 65 · 7	Barrag	e 1	0 × 24'	49.0	0	.5	66 · 5 66 · 0		
	Pinyari	Bar	rage.	16,800	10 × 24'	52.0	0-6	66.5 64.5	Barrag	e	8 × 24′	49.0	0	.5	66 · 5 64 · 5	Two spaus saved.	
	Kalrì-Baghar		,	12,415	8 × 24'	52.0	0+5	66+5 66+0	Barrag	Θ	8×24'	52 •0	0	5	66 · 5 66 · 0		
		' т.	ft non	ket. т	off River.	l Right R	iver Bic	aht nooket	Left n	ochet.	Teft R	ivar.	l Right Rive	r Bive	r pocket.	, 	
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.	u/s po	vement o	rest level.	Ciste	rn level.	Nos.	u/	s pavemei level.	nt Cre	st level.	Cisteri	n level.		
4	Left Pocket spans	•••	6	55	•0	52 -0	9	7.0	8		46.0	t	2.0	37.	0	Raised crest instead of flat regulator crest 6 ft. higher than u/s pavement.	
5	Left River spans		22	40	.0	52.0	97-0		22	22 46.0		1	52-0 37-0		0		
6	Right River spans		7	4	0	55·0	4	0.0	δ		46-0	5	2.0	37.	0	All spans have same u/s crest and d cistern levels in proposed design.	
7	Right Pocket spans		5	55	•0	55.0	4	0.0	5		46.0	5	2-0	3 7 ·	0	Raised crest of normal profile; and regu- later crest 6 ft. higher than u/s pavo-	

STATEMENT I/XXI

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XXI. EXCLUDERS AND EJECTORS

XXI-2. Optimum shape of the nose of Central Island at Kotri for the lower Sind Barrage

1. Object

It is proposed to construct the Kotri Barrage in two parts, roughly onethird 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 a curve provided by nature without the necessity of 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.

2. Model

Fig. 10/XXI.

A 1/500: 1/83'3 scale model of the pitched island was constructed in the 16 ft. parallel-sided L. B. Channel, the island boing centrally located. The intensity of discharge in the right and left arms in the model was \equiv 346 cs/ft, whereas actually in the river the intensities in the two arms are 250 and 333 cs/ft. 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.

3. 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 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 $1/83\cdot3$.

(b) $Time \ scale$.—The time scale was worked out from the tractive force formula as follows:—

 $\begin{aligned} \text{Time scale} &= \frac{\text{Tractive force in prototype.}}{\text{Tractive force in model.}} \\ &= \mathbf{T}_{\nu}/\mathbf{T}_{m} \end{aligned}$

Now, $W \propto d.s$ where w = work done d = depths = slope











PART II

i.e. 1 day in prototype=19 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 $L = \frac{1}{500} \qquad D = \frac{1}{83.3}$ $\therefore \quad \text{Discharge scale} = LD\sqrt{D}$ $= 500 \times \frac{500}{6}\sqrt{83.3} = 380,000$ $= \frac{500 \times 16 \times 346}{380,000} \text{ where } 16' = \text{ width of flume in the model.}$ 346 = Intensity in os/ft.in proto (= 750,000 cs.)

= 7.22 cusecs.

4. 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 cs./ft.=750,000 cs. approx., 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. Same experiments were finally repeated under rising and falling flood conditions.

5. 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 fig. 11/XXI) and figs. 12 to 14/XXI show the Figs. 11 to configuration of the stable scour-hole obtained in each case.

From Table 2/XXI, it is seen that Profile III gave more uniform and less Table 2/XXI 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') 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.

1/500 : 1/83 3 SCALE MODEL OF "CENTRAL" ISLAND (KOTRI BARRAGE)

I Elliptical and long. $1\cdot 22' \equiv R. L 17\cdot5$ $1\cdot 17' \equiv R. L 13\cdot5$ $1\cdot 20' \equiv R. L 15\cdot96$ $1\cdot 8' \equiv 900'$ $3\cdot 8' \equiv 1650'$ $3\cdot 6' \equiv 1800'$ $3\cdot 46' = 1800'$ II Pointed nose and long. $1\cdot 20' \equiv R. L 10\cdot96$ $1\cdot 25' \equiv R. L 20\cdot0$ $1\cdot 20 \equiv R. L 15\cdot96$ $1\cdot 6' \equiv 900'$ $2\cdot 8' \equiv 1400'$ $3\cdot 8' \equiv 1900'$ $3\cdot 6' \equiv 1800'$ $3\cdot 6' \equiv 1800'$ $3\cdot 6' \equiv 1900'$ $5\cdot 6' \equiv 1900'$ $5\cdot 6' \equiv 1900'$ $5\cdot 6' \equiv 1800'$ $5\cdot 6' = 1800'$ $5\cdot 6' $	Profilo,	R. L. of scour at Nose. Lowest R. L. of scour on the Left Flank.		Corresponding lowest R. L. of scour on the Right Flank.	Extent of scour at the nose.	Extent of scour on the Left Flank.	Extent of scour on the Right Flank.	Чр
	I Elliptical and long. II Pointed nose and long. III Elliptical and short. V Do. VI Do. VI Test (A) Do. VI Test (B) Do. VII Do. VII Do. VIII Do.	$1.22' \equiv R. L 17.5$ $1.20' \equiv R. L 18.96$ $1.25' \equiv R. L 20.0$ $1.03' \equiv R. L 20.0$ $1.03' \equiv R. L 1.0$ $1.12' \equiv R. L 9.4$ $1.03' \equiv R. L 9.4$ $1.03' \equiv R. L 6.0$ $6.53' \equiv R. L. + 40.7$ No scour $1.00' \equiv R. L. + .07$ $1.16' \equiv R. L 12.63$	$\begin{array}{l} 1 \cdot 17' \rightleftharpoons \mathbf{R} \cdot \mathbf{L} & -13 \cdot 5 \\ 1 \cdot 25' \rightleftharpoons \mathbf{R} \cdot \mathbf{L} & -20 \cdot 0 \\ 1 \cdot 25 \restriction \mathbf{k} \eqsim \mathbf{R} \cdot \mathbf{L} & -20 \cdot 0 \\ 1 \cdot 17' \rightleftharpoons \mathbf{R} \cdot \mathbf{L} & -13 \cdot 5 \\ 1 \cdot 30' \rightleftharpoons \mathbf{R} \cdot \mathbf{L} & -13 \cdot 5 \\ 1 \cdot 30' \rightleftharpoons \mathbf{R} \cdot \mathbf{L} & -24 \cdot 0 \\ 1 \cdot 50' \rightleftharpoons \mathbf{R} \cdot \mathbf{L} & -41 \cdot 0 \\ 0 \cdot 76' \rightleftharpoons \mathbf{R} \cdot \mathbf{L} & -41 \cdot 0 \\ 0 \cdot 76' \rightleftharpoons \mathbf{R} \cdot \mathbf{L} & +20 \cdot 7 \\ \mathbf{No} \text{ soour} \\ 1 \cdot 07' \rightleftharpoons \mathbf{R} \cdot \mathbf{L} & -5 \cdot 13 \\ 1 \cdot 37' \rightleftharpoons \mathbf{R} \cdot \mathbf{L} & -30 \cdot 12 \end{array}$	$\begin{array}{l} 1\cdot 20' \equiv \mathrm{R. \ L. \ -15\cdot96} \\ 1\cdot 20 \equiv \mathrm{R. \ L. \ -15\cdot96} \\ 1\cdot 30' \equiv \mathrm{R. \ L. \ -15\cdot96} \\ 1\cdot 30' \equiv \mathrm{R. \ L. \ -24\cdot0} \\ 1\cdot 18' \equiv \mathrm{R. \ L. \ -14\cdot4} \\ 0\cdot 95' \equiv \mathrm{R. \ L. \ -14\cdot4} \\ 0\cdot 95' \equiv \mathrm{R. \ L. \ +5\cdot0} \\ 0\cdot 90' \equiv \mathrm{R. \ L. \ +5\cdot0} \\ 0\cdot 90' \equiv \mathrm{R. \ L. \ +5\cdot0} \\ 1\cdot 62' \equiv \mathrm{R. \ L. \ +32\cdot4} \\ \ldots \\ 1\cdot 04' \equiv \mathrm{R. \ L. \ -2\cdot63} \\ 1\cdot 12' \equiv \mathrm{R. \ L. \ -9\cdot3} \end{array}$	$\begin{array}{c} 1 \cdot 8' \equiv 900'\\ 1 \cdot 8' \equiv 900'\\ \cdot\\ 1 \cdot 8' \equiv 900'\\ 1 \cdot 6' \equiv 800'\\ 2 \cdot 4' \equiv 1200'\\ 2 \cdot 4' \equiv 1200'\\ 2 \cdot 4' \equiv 1200'\\ 0 \cdot 3' \equiv 150'\\ \dots\\ 1 \cdot 9' \equiv 950'\\ \cdot\\ 2 \cdot 5' \equiv 1250'\end{array}$	3.3' ≈ 1650' 2.8' ≈ 1400' 3.5' ≈ 1750' 3.1' ≈ 1550' 3.4' ≈ 1700' 3.1' ≈ 1550' 2.8' ≈ 1400' 3.0' ≈ 1500' 	$3 \cdot 6' \equiv 1800'$ $3 \cdot 8' \equiv 1900'$ $3 \cdot 3' \equiv 1650'$ $2 \cdot 9' \equiv 1450'$ $2 \cdot 6' \equiv 1800'$ $1 \cdot 8' \equiv 900'$ $3 \cdot 2' \equiv 1600'$ $3 \cdot 0' \equiv 1500'$	846 cs./ft. Do. Do. Do. Do. Do. Flood cycle. Dominant discharg c. 846 cs./ft. 346 cs./ft.

Table 2/XXI, showing comparative merits of various profiles

-

Photos 5052 and 5058 show the shape of the scour hole obtained with Photos 5182 Profiles I and III. and 5058.

(b) Profile IV .- The natural slope of the island at the nose in nature ap- Fig. 16/XXI. proximates to 1:6 (i.e. 1 vertical: 6 norizontal) and hence it was thought feasible to retain this slope in the dividing island as far as possible. 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, Fig. 16 XXI.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. 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.

Fig. 17/XXI illustrates the bed lines of flow and indicates the probable ex- $\frac{Fig. 17}{XXI}$ clusion. 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 he.

(d) Profile VI.—The effective slope on the left side was made still steeper by F(a, 18)reducing the length of the transition reach. This resulted in the largest depth of scour obtained in the series, viz. to R. L.-41. 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 Fig. 19/ R. L. =135, obtained with profiles I and IV. Fig. 20/XXI shows the surface Fig. 20/ 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 Fig. 21/ from 1:3 to 1:2. The maximum depth of scour was limited to R. L. -513 and owing to the differential scour, the exclusion was satisfactory. Figs. 22 and owing to the differential scour, the exclusion was satisfactory. Figs. 22 and 23 show bed and surface lines of flow. The excluding action at the nose Figs. 22 and 23 show bed and surface lines of flow. The was the profile finally adopted. 23/XXI. 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 unsuccess- Fig. 24/ ful as it again developed a deep scour-hole, as deep as R. L. -30.12. The XXI. ful as it again developed a deep scott into, this case also. The profile of the $_{Fig. 25/}$ bed exclusion, however, was satisfactory in this case also. The profile of the $_{Fig. 25/}$ with shorter lengths of transition reaches. XXI. 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; Fig. 26' but both the depth and extent of scour were smaller as compared with the depth xxI. obtained by running a constant superflood discharge.

Tables 2 & 3

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.

6. 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 tested under the curvature interior in the side. Owing come many disturbing factors, no absolute numerical accuracy is claimed for the values of depth and factors, no absolute numerical accuracy is claimed for the values of depth and extent of scour given in Table 2/XXI. The relative values are, however, useful for assessing the best shape.

S 41-29

PART III. BASIC RESEARCH

II. RIVER BEHAVIOUR AND CONTROL (ii) SCOUR

II-(ii) 1. Scour downstream of rigid structures in wide channels—Prototype data regarding depth of scour downstream of bridges

Experiments were done in the year 1944-1945⁽¹⁾ 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.

Table No. 1/ II-ii.

Such data is being observed for the last 20 years on 21 important bridges on alluvial rivers on N. W. Railway. The N. W. R. authorities have kindly n:ade this data available for analysis.

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 waterway, as was observed in the case of Hardinge Bridge over the Ganges.⁽⁹⁾

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

(a) Depths at 150', 300' and 450' 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' 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' apart, till beyond the point of maximum scour, have been requested for, in case of only 8 out of 21 bridges.

(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 italies in Table 1/II.-ii) where daily discharge measurements are made by the Discharge Division.

(2) Annual Report (T), 1941-1942, C. I. and H. R., page 12.

⁽¹⁾ Annual Report (Tech.) of Indian Waterways Experiment Station, Poona, 1945.

1/500 : 1/83 3 scale Model of Central Island (Lower Sind Barrage at Kotri)

 $g_p = 346 \text{ cs/ft.} \equiv 750,000 \text{ cs. approx.}$

U/s. water level R. L. 74. Top of Island R. L. 84.



Profile I 5052. Photo looking sideways upstream.



Profile III 5058. Photo looking sideways downstream











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G.PZ.P.POONA '47.





PART III

TABLE 1/II-ii

Important Bridges on alluvial rivers on the North Western Railway

S. No.	Name and site of Bridge.	Place at which discharge is observed.	Distance between the site of bridge and place where discharges are	Whether up- stream or down- stream of bridge.	Remarks.
_1	2	3	observed (in miles).		
1	Jhelum at Jhelum	Rasul Head-		_ 5	6
		works. Mangla	21	Downstream	•
2	Jhelum at Haranpur	Rasul H/w	32	Upstream	
8	,, ,. Khusab	Chela	- 28	Do.	1
4	Chenab at Chund		60	Downstream	1
				•••	No observations in the vicinity,
5	" " Wazirabad …	At the bridge		• •••	-
6	", " Chiniot E.	D:	i	· .	1
7	ρ. _{ρ. α} .⊴ W.∫ […]	reivaz bridge	56	Downstream	
8	", ", Shershah …	At the bridge	'		
9	Ravi at Lahore	Sidhnai	5	Do,	
10	., " Abdul Hakim	At the bridge			
11	" " Dera Baba Nanak.	Shahadara	72	Downstream	•
		Madhopur H/w.	59	Upstream	
12	Beas at Beas	Mandi plain	30	Downstread	
13	Sutlej at Phillour	Rupar H/w			
14	", "Gidarpindi …	At the bridge			
15	., "Adamwahan	Do.			
16	Jumna at Delhi				No record in Laho
17	" — " Kalanour				Discharge Divisio
18	Indus at Kotri	At the bridge		1	
19	"""Sukkur	Do.			
20	🟹 " " Kalabagh	Do.			
21	Kurum at Thanedarvala.				No record.

III. FLOW IN RIVERS AND CANALS

(ii) FLOW IN CHANNELS IN INCOHERENT AND COHERENT ALLUVIUM

III/ii-1 RATE OF SAND ATTRITION IN CHANNELS

The experiment, which was first started in September 1938, to determine how far attrition was the true explanation of the progressive reduction of size of bed materials from the hills to the sea, was carried out in a circular tank, in which water was revolved with a uniform velocity. In this tank, experiments could be done only with one velocity at a time.

1943 Apparatus⁽¹⁾

In order to expedite results, a circular tank 7 ft. in diameter and 2 ft. deep, was divided into 4 compartments. Vertical blades were spaced 120° apart, Fig. 1/11.ii. fixed to a central revolving shaft, with the bottom edge of the blades 10″ above the bottom of the tank. The mean velocity of the water in the three compartments was 2.38, 4.16 and 5.93 ft./sec. respectively.

⁽¹⁾ Indian Waterways Experiment Station, Annual Report (Tech.) for 1948, pp. 62-64.

116 ANNUAL REPORT (TECHNICAL) OF THE WORK DONE DURING THE YEAR 1946



This experiment was started on 3rd February 1943 using Brahmaputra sand, passing through a 60 mesh sieve and retained on a 80 mesh sieve; so that the range of variation of grades of sand in the mixture was less than normal.

m of original sand = 185 mm. \pm 10 (or \pm 60 per cent.)

m of sand used in experiment = 254 mm. + 063 (or +25 per cent.)

The sand thus obtained was placed in each compartment in the tank to a depth of 4'' (about 1150 lbs.) and water was maintained at a constant level.

The abrader was run for 1460 hours. Sand samples were collected from each compartment and after thoroughly mixing the sand, they were analysed by the Puri Siltometer. The observations were repeated after 3770 hours (1944), 5650 hours (1945) and 6550 hours (1946) and sand samples were analysed for Table 1/III.ii each period. Table 1/III-ii shows the percentage fraction of various grades to total quantity.

Fig. 2/III-ii. Figure 2/III-ii shows the relation of "reduction in volume of sand": Velocity after 1460 hours, from which a tentative straight line relation was obtained. Similar relations were also obtained after 3770 hours, 5650 hours and 6550 hours of working.

Curve A	Reduction	in	volume	after	$1460 \text{ hrs.} = 00137 (V-2)^{.39}$
Curve B	"	"	**	"	$3770 \text{ hrs.} = 00202 (V - 1.2)^{-39}$
Curve C	"	"	"	,,	5650 hrs. = \cdot 0030 (V - \cdot 92). ⁵⁹
Curve D	"	"	23	,,	$6550 \text{ hrs.} = 0038 (V - 89)^{.36}$

The relation A indicates that the attrition started at velocity 2 ft/sec. The relations B, C and D show that the critical velocities to start abrasion after 3770 hours, 5650 hours and 6550 hours were 1.2, 92 and 89 ft/sec. respectively.

As size of sand decreased, the critical velocity to start attrition also decreased, thus showing that the critical velocity depends upon the grade of material, a lower critical velocity being required to start abrasion in a finer grade material.



PART III

An approximate relation between \widetilde{m} and Veritical, viz. $\widetilde{m}=28$ $V_{ert}^{(091}$ —is

shown in fig. below, where $\overline{m} = mean$ size in the three compartments at the Table 2/III-II end of the four time-intervals, shown in table 2/III-II.



Figures 4/III-ii and 5/III-ii show size distribution curves of sand at Fig. 4 & 5/III different intervals.

These experiments are still in progress.

TABLE 1/III-ii.

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COMPARATIVE STATEMENT SHOWING PERCENTAGE FRACTIONS OF BRAHMAPUTRA SAND TO TOTAL SAND

Collected in Different Compartments of Abrader with Velocities 2.38, 4.16 and 5.93 ft./sec. after 1460 Hours, 3770 Hours, 5650 Hours and 6550 Hours of Working.

	· 	1									<u></u>				,
	Dia annu at	Brahmaputra sand used in Sand Abrader between 60 and 80 mesh.	Inner compartment (Velocity 2-38 it./sec.)			bliddle compartment (Velocity 4.16 ft./sec.)				Outer compartment (Velocity 5.93 ft./sec.)				Brahma-	
Fractions of Sand.	Dis. range of particles in mm.		After 1460 bours per cent.	0 After 3770 hours per cent.) After 5650 hours per cent.) After 6550 hours per cent.	After 1460 hours.	1460 After 3770 ars. hours.	0 After 5650 hours.	0 After 6550 bours.) After 1460 hours,	After 3770 hours.	0 After 5650 hours.) After 6550 hours.	before siev- ing.
Olay	00089														-2
Very fine silt	-00390078									•1			•1	-1	•8
Pine silt	-00780156	-1	-2	•9	•2	.2	-9.	-2	•3	•3	•2	· 2	•4	.4	4.7
fedium silt	·0156- ·0812	-1	-8	•\$	-4	•3	-2	-5	-5	-5	•8.	-5	-6	.9	5.3
oarse silt	·0312-·0625	-4	·4	1-1	1.6	1.5	•5	1.5	1.8	1.8	·6 .	1.4	2.0	2.1	8-8
ery fine sand	-0625125	8-20	4-6	7.1	7.5	7.6	4.8	7.6	8.0	9-0	5.9	8-1	9.2	10.2	17.2
ine sand	·125-·167	8.00	8.1	9.0	8.9	9.9	9.1	10.8	11.2 .	12.6	9.9	10-4	12-1	12.5	10.9
edium fine sand	-16725	81.80	93-2	85-0	36.9	37.8	34.7	85.0	37.2	35-7	35-0	85.7	37.2	86-2	22.6
iedium sand	.2542	56.40	59·2	47.8	44.5	42.7	50-9	45.7	41.0	40.0	48.1	43-8	38+4	88·3	30-0
ean dia in mm		- 2537	-4487	•2419	-2345	·2317	•2438	·2858	- 2267	-2240	· 2408	-2828	-2229	·2194	- 185
" 9		01682	·01588	•01416	01291	•01244	.01449	-01311	·01165	.01124	•01396	-01262	-01106	.01056	
ofference of m ³ from original (0.01682).			-00094	·00216	-00941	· 20389	-00183	-00821	- 00467	-00508	.00286	-00870	-00526	-00576	.





PART III

TABLE 2/III-ii.

Showing Critical Velocities and Mean Diameter in mm. at Different Periods

Period of working in hours.	Velocity in ft./sec.	m in mm.	V _{crt.}	
1460	2.38	.2487		
	4.16	-2438	-2414	8·0
y, *	5.93	•2408	± •0032	
3770	-38	.2419	·2368	1.2
,,	4.16	.2358	± -00308	
"	5.98	• 2328		
5650	2.38	.2345	•2280	.92
. 21	4.16	-2267	+ 0045	
89	5.93	-2298		
6550	2.38	.2317		
,,	4.16	-2240	0.2250	-89
19	5-98	. 2194	± :0050	
		J		

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PART IV-MISCELLANEOUS

I. HISTORICAL ACCOUNT OF THE INDIAN WATERWAYS EXPERIMENT STATION, POONA (1916-1946)

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PART IV-MISCELLANEOUS

HISTORICAL ACCOUNT OF THE INDIAN WATERWAYS EXPERIMENT STATION, POONA (1916-1946)

1. Introduction

The Station, recently put on a permanent basis by the Government of India in January 1944, was originated 26 years ago in the special Irrigation Division, Poona, under the Bombay Government by Mr. C. C. Inglis (now Sir Claude Inglis). Sir Claude Inglis continued to be the Director of this Station from its commencement till March 1945. Mr. K. K. Framji, the present Director took over charge in December 1945. In the intervening period Rao Bahadur D. V. Joglekar, Deputy Director, officiated as Director.

Research was started in the Bombay Deccan on soil and irri-cultural problems in 1916. This work will be summarised in Part I and the research in hydrodynamics will be summarised in Part II. The advance in research in various branches has been the result of well-coordinated, team work under the able guidance of Sir Claude Inglis and it is rather difficult to apportion credit for a particular research to a particular officer or officers because many actually contributed towards it; but this has been done wherever possible. Names of officers or assistants who carried out experiments described in Part II are given at the end of each para.

PART I-RESEARCH ON SOIL AND IRRI-CULTURAL PROBLEMS IN THE BOMBAY DECCAN

$\mathbf{2}$ Soil classification and demarcation of 'block' areas

In 1916 it was found that about 30 per cent of land otherwise suitable for sugarcane, in the Nira L. B. canal tract, was damaged by water-logging, salt efflorescence, or sodiumisation; and that other canals were rapidly becoming similarly affected; so, as all previous scattered attempts to solve these problems had failed, a Division-called the Special Irrigation Division-was opened to focus its efforts on their solution. It was found that successful drainage depended mainly on the porosity of the substratum and so a very simple method of measuring porosity in situ was worked out.") It was also found that damage was largely due to water from irrigation of high-lying, shallow soils finding its way under the clay layer of the valley lands; thus water, under several feet of pres-sure, was found in the clay zones. The first necessity was a soil survey, for which observers were trained to judge the soil profiles to a depth of 10ft. from surface indications, especially weed growth, colour and texture of the surface soil ⁿ. Working with these soil classification plans, land which was unsuitable, either because it would be damaged by irrigation or because its irrigation would cause damage to lower lying lands, was excluded from sugarcane irrigation; (*) the canal areas were divided into zones suitable for sugarcane, fruit trees and seasonal crops; and each type of crop was sanctioned on 6 year permits in "blocks",⁽⁸⁾ in units of $1\frac{1}{2}$ acres, the whole area having been demarcated into 1/2 acre plots to fit in with the standard 3 year rotation usual in the Deccan.

3. Land Drainage

At the same time as this soil classification, land drainage experiments were carried out (4). (6) and (8); it was found that where an outlet to a river or a large nala was available, drainage was economically practicable, and a large part of the area.

(1) Bombay P. W. D., T. P. No. 13 " Soil and Sub-soil surveys in the Deccan Canal areas " by C. O. Inglis and V. K. Gokhaie (1926).

(2) Bombay P. W. D. T. P. No. 49 " Development of Irrigation in the Deccan Canal areas" by C. C. Inglis and V. K. Gokhale (1934).

(3) Bombay T. P. 16. Note on Irrigation on block system in the Deccan Canal tracts" by C. C. Inglis (1927).

(4) Bombay T. P. 14 "A short note on the improvement of Damaged Lands in the Canal Irrigated tracts of the Bombay Decean "by C. C. Ingiis (1927). (5) Bombay T. P. 24 "Report on Land Drainage Experiments at Baramati" by C. C Inglis and V. K. Gokhale (1998).

(6) Bombay T. P. 56 " Land Drainage in the Deccan Canal areas" by W. A. Evershed, with a note by C. C. Inglis (1937).

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when drained, was at once reclaimed. Sodiumised lands resisted reclamation; so this question required greater effort and collaboration of agricultural graduates with irrigation staff. It was found that practically all lands could be reclaimed by the judicious use of gypsum in conjunction with the growing of suitable crops under irrigation and suitable drainage.

4. Controlled Irrigation

Up to 1916 an unknown discharge of water was distributed by low 4·1 paid staff to an unknown area of crops (with different water requirements) for an unrecorded period of time. The result was chaos, waste and dishonesty. Steps were, therefore, taken to restrict water by constructing measuring devices⁽⁷⁾ and outlets of a type which restricted the water supply ^{(8) and (9)}.

4.2. The standard measuring device, adopted at the recommendation of the Special Irrigation Division, was the Standing Wave Flume "; but where the head available was insufficient, i.e. less than 10 per cent of the depth of water in the channel, Venturi Flumes (loss of head less than one per cent at full supply) were recommended.

The outlet adopted was the Gibb module, (10) which has no moving 4.3.parts, yet the discharge is maintained constant within 1 per cent, irrespective of upstream and downstream water levels within working limits, by means of baffles which come into action (when there is excess head) as a result of free vortex flow. These devices were standardised after a long series of experiments and were adopted as standard Canal Outlets (11).

5. Increasing the discharging capacity of Regulators, Outlets and Aqueducts by fluming and remodelling

Nira Left Bank Canal Head Regulator .- In 1916, it was decided 5.1.to remodel the Nira Left Bank Canal to pass 50 per cent more water without interfering with irrigation operations and conditions precluded adding extra spans to the Head Regulator. Model experiments were carried out and it was found that the coefficient of the existing sluices could be increased from 0.83 to 193 by adding a bell-mouth upstream and expanding the barrels downstream⁽¹²⁾ As a result 720 cusecs can now be passed with a loss of head of 0.17 ft. as against the previous loss of head of 0.38 ft. when only 480 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 spans—had this been possible. The head regulators of the Godavari Right Bank and Left Bank Canals ⁽¹³⁾ were similarly treated with satisfactory results.

5.2.Rock cuts were either widened or, where more than 15 ft. deep-so that widening was impracticable-were lined, thus reducing the rugosity factor from 0.038 to 0.013 (14).

Fluming of aqueducts .-- The width of the aqueducts, it was found, could 53. be reduced by providing suitable convergences upstream and divergences downstream as was done in the case of Aqueduct 23 of the Nira Right Bank Canal⁽¹⁵⁾. The nala was also recommended to be flumed. The reduction in cost in such cases would be, ordinarily, of the order of 40 per cent. In this particular instance, however, the saving in cost was only about 20 per cent as it was found more feasible to increase the number of spans due to very high nala banks, which

(7) Bombay T. P. 15" Note on Standing Wave Flumes and Flume Meter Falls" by C. C. Inglis (1928.

(8) Bombay T. P. 19 "Note on the Design of distributaries and their outlets in the Bombay Deccan" by C. C. Inglis (1928).

(9) Bombay T. P. 23 "A note on Standing Wave Pipe Outlets ' by C. C. Inglis (1928).

(10) T. P. No. 13 (Class A), Punjab P. W. D. and C. I. and H. R. Station Publication No. 3 "Note on the Theory, Design and Construction of Gibb Modules, etc. " by C. C. Inglis and D. V. Joglekar (1941).

(11) Bombay T. P. 25 "Moduling operations on Distributary No. 8, Nura Left Bank Canal, and application of the results to other distributaries" by C. C. Inglis and D. V. Joglekar (1923).

(12) Bomby T. P. 54 "Note on experiments with a model of the head Sluices of the Nira L. B. Canal" by C. C. Inglis and D. V. Jogleksr (1935).

(13) C. I. and H. R. Annual Report for 1937-38 (p.32).

(14) Bombay T. P. No. 20 "Note dated November 1927 on model experiments to ascertain the effect of partial lining of the Malegaon Cut in miles 38 and 39 of the Nira L. B. Canal" by C. O. Inglis and D. V. Joglekar (1928).

(15) Bombay T. P. 42 "Notes on flumed Aqueducts, (2) Remodelling an existing canal or Distributary to pass, an increased discharge, (3) Afflux caused by beading up water at regulators of a canal and the extent of back water & (4) Distribution of velocities in channels" by C. C. Inglis (1982).

would have increased cost in earthwork. The idea of fluming masonry works was taken up by Mr. Hawes in Sind,¹⁶ who carried out some experiments⁽¹⁷⁾ in this connection at the Poona Station with the help of Rao Bahadur D. V. Joglekar and H. V. Katakkar.

5.4. Canal remodelling.—Considerable lengths of Canals and Distributaries were remodelled to the correct natural shape⁽¹⁵⁾, $P=2.67 \ Q^{1/2}$ and $R=47(Q/f)^{1/2}$, where P=wetted perimeter and R= Hydraulio mean depth, but elsewhere the banks were widened and raised, and the channels left to reshape themselves to carry the increased discharge naturally-i.e. with widened sections and flattened slopes.

All these modifications were tested out by means of models, and results were far in advance of what was anticipated, both as regards efficiency and cheapness.

6. Disposal of Poona sewage by irrigation

6.1. Pumping Poona sewage for disposal by irrigation was started in 1918, water and effluent mixture being distributed as required through twin channels of Distributary 5 of the Mutha Right Bank Canal. A Venturi meter was installed at the head of this channel to measure the canal water and standing wave flumes to measure the effluent mixture or water-in each of the twin channels. As sewage irrigation was a new practice, the Effluent Experimental Farm (comprising 63 acres of lands) was opened in 1918 and placed under the Special Irrigation Division, 1919. Experience showed that 225 lbs. of nitrogen per year, in the form of effluent, was sufficient to give an average crop of cane; but to allow for less etc. the standard was fixed at 300 lbs. per acre per year. Experiments showed that effluent was a complete manure and gave an appreciably bigger outturn of cane per acre than that produced by the standard manuring practice, and the land and outturns have been steadily improving under effluent irrigation⁽¹⁵⁾. The late Mr. K. K. Bhatarkar, B.Ag., the Farm Superintendent, did a creditable amount of spade work in this connection.

Up to 15 cusecs of sewage effluent is now received for disposal and 6.2.although distributaries 3 to 6 and intermediate watercourses commanding a total area of 1,500 acres are now supplied with effluent, the area is insufficient; so experiments are in hand in the Poona Irrigation Research Division to find cane varieties and methods of irrigation to dispose of as large a dose of nitrogen as possible.

6.3. The Manjri Drainage Scheme, which drains a large part of the Effluent Area, was completed in 1936, and it was found that irriggtion with effluent expedites reclamation of land damaged by salt and sodiumisation.

6.4. A scheme (a) for utilising the gas generated by the sewage to pump the sewage, and (b) to dispose of an anticipated increase of 50 per cent sewage supply, partly by irrigation on a new area uncommanded by a canal and partly by sludge digestion and activation, was prepared⁽¹⁰⁾.

The great success of this scheme of sewage disposal was due to pumping, irrigation and research having been under a single control.

7. Soil surveys in Sind.

7.1. In 1931, staff was lent to the Development and Research Division in Sind to assist in carrying out soil surveys on lines analogous to those followed The whole of the area in the Deccan-with suitable modifications. commanded by the Lloyd Barrage has since been surveyed⁽⁶⁰⁾.

(18) Bombay, P. W.D., T.P. 17 "Note on the Effluent Farm near Poona (1910-19 to 1925-26)" by C. C. Inglis (1927). (19) Bombay T. P. 57 "Disposal of Poona Sewage Effluent" by C. C. Inglis, D. V. Joglekar and R. A.

(20) Bombay Engineering Compress Paper No. 136, Vol. 21 "Sub-soil investigations made in the area commanded by the Lloyd Barrage and Canals construction" by U. G. Hawes (1932).

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⁽¹⁶⁾ Bombay T. P. 31 "Report on experiments carried out at the Karachi Model Testing Station on a model of flumed regulator" by C. G. Hawes & H. S. Kahai with a note by W. A. Evershed (1929).

⁽¹⁷⁾ Bombay T. P. 26 "Report on tests made on a model on the group of Regulators at Oderolal at mile 169 of the Rohri Canal, Sind" by C. G. Hawes and C. C. Inglis (1928).

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7.2. Some two million acres were classified in detail and, from the results obtained, it was found possible to work out a system of classifying lands from surface appearance of the soil, the flora growing on the soil, and the texture of the soil down to 3 ft. as ascertained from borings. This method of classification could be done, at the time, at one anna per acre.

7.3. Sind was advised and guided regarding the system of sub-soil water observation pipes to be placed on a regular grid. Sub-soil observations are now made in the whole command of the Barrage and in a large area of the non-Barrage canals so that Sind has better sub-soil data than most other Irrigation provinces. Regular observations are made throughout the year in these pipes and the results of observations are plotted for April and October as difference contours, showing the difference in sub-soil water table over a 12 months period, and as, hydro-isobaths, i.e. contours showing lines of equal depth below the surface.

8. Water weeds and their eradication from canals

Deccan Canal administration had been seriously affected for many years as a result of heavy weed growth⁽²¹⁾.

The following water weeds are found in the Deccan :---

(1) Vallisneria Spiralis,

(2) Potamogeton pectinatus,

(3) Potamogeton perfoliatus.

(4) Potamogeton indicus,

(5) Ceratophyllum demersum,

(6) Hydrilla verticillata;

but for simplicity, water weeds may be divided into only 2 groups :---

(1) Vallisneria spiralis —a grass like weed, up to 14" in length, which grows in tufts; and

(2) Potamogeton varieties and associates—the worst of which is Potamogeton perfoliatus which grows to a length of 12 ft. or even more.(1) is the first weed to appear, and is, comparatively, unimportant except in

small channels. (2) indicates a much more serious condition of weed growth. It was found that in canals in which there was considerable silt in suspen-

sion weed growth was small, and vice versa; and so it was recommended that water should be fluctuated in the Pick-up Weir Basins at the heads of Canals, to stir up silt as far as practicable. A protracted investigation, carried out in the field and laboratory, showed the relative effect of various factors on weed growth—light, exclusion by silt, velocity etc.,—but the main point established was that water weeds can be killed off by suitably timed canal closures⁴²⁹. Consequently, water weeds have been eliminated from all Deccan Canals except the Mutha Right Bank Canal, which cannot be closed on account of being the source of Poona Water Supply.

9. Rainfall and Run-off

A considerable amount of work was done on this subject by Sir Claude Inglis when he was under the Government of Bombay, and also since then as the Director, Indian Waterways Experiment Station. These investigations are described in Part II.

PART II-RESEARCH IN HYDRODYNAMIC PROBLEMS

10. Introduction

10.1. In the early days of the Poona Station, from 1922 to 1934, experiments were conducted at distributary 5 of the Mutha Right Bank Canal near Hadapsar, about 6 miles from Poona, with models designed to solve various

⁽²¹⁾ Bombay Engineering Congress Paper No. XLVIII "Some factors affecting the design and working of Deccan Canals", by C. C. Inglis and C. I. & H. B. Annual Report (Tech.) for 1937-38, (p. 7).

⁽²²⁾ C. B. I. Publication No. 26 "Water weeds and their eradication from Canals" by C. C. Inglis and V. K. Gokhale (1942).

hydrodynamic and irrigation problems occurring in Bombay and Sind-especially in connection with Sukkur Barrage Scheme and the Deccan Canals, then under construction.

10-2. In 1925 it was found that the discharge available at the Hadapsar Station was insufficient for carrying out several large scale model experiments simultaneously; and, as exceptional facilities existed at Khadakvasla situated about 11 miles from Poona, where clear water can be drawn straight from Lake Fife, new models were constructed there. In 1934 the whole station was transferred to Khadakvasla. In 1937 the station was taken over by the Government of India to serve the whole of India.

10-3. During the last 9 years, over 100 important investigations involving over 2,000 experiments have been carried out at the Station; some of these are listed with references in Appendix I. these investigations covered various $_{Appendix I.}$ fields:

Control of sand entering canals;

the protection of bridges across rivers;

training of tidal and non-tidal waters;

determination of the principles governing the action of spurs and pitched "attracting" islands for controlling rivers;

the factors determining the size and shape of river meanders;

the relation between rainfall and run-off;

the effect of silt and sand washed into the rivers when hill denudation occurs and the rate of silting of reservoirs: the laws governing the movement of sand in rivers and the factors affecting accretion of rivers;

littoral drift; etc.

10-4. As a result of experience gained from the model work and the inspection of actual works, advice is also given in many cases without having recourse to model experiments. Models are now used only in intricate or novel cases. Inquiries from various quarters of India are increasing, as a result of increased confidence in the work done by this Station following the correct and successful diagnosis and recommendations made by the Station and the material savings accruing therefrom. Only some of the notable achievements of the Station are given in the following paragraphs.

11. Curvature of flow in rivers and canals for controlling sand entering canals and training of tidal and non-tidal waters.

Sir Claude Inglis is probably the first Engineer in India to establish a claim to the desirability of locating off-taking channels from the parent channels on the outside of curves, so that only the high-velocity, sand-free, surface water is drawn by the offtaking channel; because favourable curvature of flow exists at such places.

[Before dealing with particular investigations, it is necessary to explain what is meant by "curvature of flow". In general, "curvature of flow" includes all flow in which the filaments follow a curved course; but the special sense in which it is used at the Indian Waterways Experiment Station—which is actually the only relevant case in River practice because free vortex flow is not found— is where the filaments follow curved courses, which do not remain equidistant from the bank but approach or recede from it. It follows from this that a surface filament converges towards the concave bank but at a small depth below dives down (favourable for an offtaking channel) and cross flow is generated; while the bed filament recedes from the concave bank and flows across to the convex (inner) bank rising as it does so. Sometimes such type of flow is called rotary flow, because this gives the idea of complete circulation which is rare in rivers—at other times curvature is erroneously visualised as a confiscere motion which it is not; but in the sense explained above, curved flow is nearly everywhere common in rivers.]

11 (A) Control of sand entering the canals taking off the Lloyd Barrage at Sukkur and other problems relating to the design of the Barrage

In 1928, hydrodynamic questions relating to the Lloyd Barrage and Canals construction were referred to the Poona Station. Basic and specific experiments by means of models ^(23, 24 and 25) conducted in this connection were:—

(a) "Still pond running" versus "open flow".

(b) Effect of raised sill of a canal regulator on the quantity of sand drawn by the canal.

(c) Design of pockets upstream of Barrages to produce favourable curvature.

(d) Siting or locating the off-takes on the outside of curves.

(e) "Diverging guide banks" versus "Divide walls at right angles to the Barrage" and the optimum length of divide walls.

(f) Necessity of maintaining correct velocity ratio inside and outside the pockets for sand control.

(g) Effect of divide walls having a vertical nose and having a sloping nose on sand exclusion.

(h) Distribution of sand entering a series of canals upstream of a Barrage.

(i) Scour at the nose of Divide walls and its effect on sand entering the canals.

In "still pond" regulation, the undersluices are kept closed when canals are open: whereas in "open flow", some undersluices are kept open. Though "still pond" running was more successful, either method was suitable for the left bank canals off-taking from the concave side; but "still pond" gave markedly better sand exclusion from the right bank canals. Predictions were fully confirmed when the canals were opened.

On account of the slight bend upstream of the Sukkur Barrage, it was predicted that a sand bank would form along the right (convex) bank: to prevent this, it was proposed that a long vane should be constructed upstream of the canals on the right bank. A right bank vane, by itself, would have tended to pull the main flow of the river across to the right bank; so a similar vane was proposed for the left bank and the two together would have maintained flow free from heavy sand along both banks (Reports not printed).

This recommendation was not acted on, it being decided to watch results. The amount of sand entering the canals in subsequent years up to 1938 increased in quantity, and though considerable scour occurred each October, which washed down the silted bed level to the order of 1 ft., the canals lower down gradually deteriorated. Conditions got so bad by 1939 that a large expenditure had to be incurred on sand clearance of the canals and the right pocket. The bed of the river on the right bank had, by then, got badly silted up and the original solution was no longer possible. The question was again referred to the Poona Station; the solution "⁶⁰ recommended was to construct an approach channel, starting 5,000 ft. upstream of the Barrage, from which the supply for the canals would be drawn off over a raised sill at a point 2,250 ft. above the Barrage, the rest of the water and most of the bed sand flowing down the tail channel. This investigation required the use of 6 models with different vertical exaggerations—to see the scale effect on curvature of flow and to obtain information regarding the design of—

(a) the shape of the nose of the outer bank of the approach channel;

⁽²³⁾ Bombay T. P. 45 "Note of May 1931 on silt Exclusion from Canals, "still pond" versus "open flow" Part I" by C. O. Inglis (1933).

⁽²⁴⁾ Bombay T. P 46 "Note on silt exclusion from Canals, Part II, first report on experiments with the full-width model of the Sukkur Barrage at Khadakwasla" by C. C. Inglis and D. V. Joglekar (1933).
(25) Bombay T. P. 52 "Factors affecting bed silt from canals taking off from alluvial rivers" by C. O. Inglis, and D. V. Joglekar (1935).

⁽²⁶⁾ Annual Reports (Technical) of the Indian Waterways Experiment Station for 1938-39, 1939-40, 1940-41 and 1941-42.

(b) the junction of the outer bank with the Barrage;

(c) the raised sill at the off-take into the Right Pocket with stone protection upstream and downstream of the raised still;

and (d) the inner bank of the approach channel.

The approach channel was constructed in 1940-41 and has proved very effective in excluding bed sand from the canals⁽²⁷⁾.

(a) to give minimum afflux during floods;

(b) to make the standing wave form just downstream of the Barrage, instead of at the end of the apron, where it would have set up destructive forces, which would have endangered the structure, had the original design been adopted;

(c) to evolve the most suitable design of pier cut- and ease-waters, which were found to be equilateral arcs of circles, though semi-circles were almost as good; they had also the advantage in that they were better when there was cross flow and were less liable to catch floating matter.

[These experiments were carried out by Rao Bahadur D. V. Joglekar, B. E., H. V. Katakkar, B.E., Mr P. D. Jaeel, B.E., Mr. J. M. Parikh, B. E., Mr. P. K. Kulkarni, B. Sc., (Elec. and Mech.) and Mr. N. K. R. Iyengar, Overseer.]

11 (B) Design for the Lower Sind Barrage and sand-excluding heads for the two canals of the Dehri-on-Son Anicut (Bihar)

The Indian Waterways Experiment Station has emphasised⁽²⁹⁾ time and again that the successful working of a Barrage-particularly in respect of sandexclusion from off-takes---depends primarily on its positioning. When the natural meander-length of a river is ignored and an unnatural distorted curvature is imposed at the very point where it is essential to have sweet, natural curvature, it is not surprising that Sand excluders, ejectors and islands are required to reduce the damage done by bad positioning of the Barrage. The designs for the Lower Sind Barrage at Hajipur⁽²⁰⁾ and at Kotri^{20(A)} are being evolved on the principles set out above; Mr. Gole, B.E., has contributed valuable ideas in this direction and the experiments are continued under the direction of Mr. K. K. Framji. Mr. D. P. Kulkarni, B.E., carried out basic experiments regarding co-efficient of discharge etc. "Sand excluding" heads for the two canals of the Dehri-on-Son Anicut (Bihar)(30) were similarly designed with the aid of model experiments, by artificially creating favourable curvature. These latter experiments were creditably carried out by Mr. J. M. Parikh, B.E.

11(C) Control of sand entering the Mithrao and Khipro Canals at Makhi Weir Ex. Nara River (Sind)

The most notable and successful example of controlling sand by favourable curvature of flow is that of the Mithrao canal⁽³⁰⁾. The Mithrao canal derives its supply from the Eastern Nara above the Makhi Regulator; before the Sukkur Barrage the Mithrao Canal off-take was located on the concave side of a bend in the Nara, then a river drawing its supplies from the Indus; with the Sukkur Barrage Project, the Eastern Nara was canalized and given a controlled head above the Barrage and the Mithrao Canal made to take off from a straightened Canal. Owing to scour in the Nara, and an unsuitable off-take from a straight reach, coarser silt than previously drawn, began to enter the Mithrao in 1932; and the previous drop into the canal of 2'6'' (at full supply discharge) gradua y disappeared, so that command was lost.

(27) C. I. & H. R. Annual Report (Technical) for 1943.

(28) Bombay T. P. 23 " Note on experiments carried out with various designs of piers and sills in connection with the Lloyd Barrage at Sukkur" by C. C. Inglis, C. G. Hawes, J. S. Reid, D. V. Joglekar and H. V. Katakker (1929).

(29) I. W. E. Station Annual Report (Tech.) for 1944.

(29A) Indian Waterways Experiment Annual Report (Tech.) for 1945.

(80) C. I. & H. R. Annual Report (Tech.) for 1942-43, 1943 and 1944.

(31) Bombay T. P. 19 "Silt control at Heads of Cauals and Distributaries" by C. C. Inglis and D. V. Joglekor (1938).

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Experiments were, therefore, carried out to determine how to exclude sand from the Mithrao Canal. Several alternatives were suggested :--

(a) A spur or groyne downstream of the Mithrao combined with 1 in 5 upstream warped bank.

(b) 1 in 5 warped convergence combined with a raised sill.

(c) King's vanes for Mithrao and Khipro.

(d) A divide wall opposite Mithrao, combined with a single large-radius vane for Khipro.

(e) The best solution, both from the point of efficiency and constructional facility, was found to be a separate approach channel for the Mithrao from the outside of a bend upstream of it.

It was recommended that the old river bed should be partly re-opened to carry water to the Makhi Regulator and Khipro Canal, the existing straight channel, after reduction in width, being used as an approach channel to the Mithrao Canal. This lay-out was such that the approach channel took off where the beneficial effect of curvature was a maximum, so that only top water containing only fine silt entered the Mithrao approach channel or the Khipro Canal, the heavy bed sand being deflected into the river downstream.

This recommendation was acted on and "command" was at once restored and the drop into the Mithrao at the Head Regulator gradually increased to 1'6".

The problem was again referred to the Poona Station in November 1940, this time to see how silting of the Nara river downstream of the Makhi Regulator could be reduced. That this would occur had been predicted by the Station. Two alternatives were suggested :--

(a) Creating turbulence upstream of the approach channel by constructing short groynes, pointing 45° downstream, the groynes being fixed in the right bank of the bend. This would throw sand into suspension, which would then be drawn by the Mithrao, thus reducing the quantity of sand going down the river.

or (b) closing the approach channel and drawing water for the Mithrao through a cut on the inside of a bend in the Nara River.

The first alternative was adopted because the work could be carried out quickly and cheaply; and has proved a success. These experiments were carried out by Rao Bahadur D. V. Joglekar and N. K. R. Iyengar.

12. Protection of Bridges over rivers and Canals

12(A) Safeguarding the Hardinge Bridge over the Ganges (Bengal-Assam Railway) against the danger of being outflanked

The Hardinge Bridge, 200 miles north of Calcutta, about a mile long, costing 5 crores of rupees and carrying the main line of the Bengal and Assam Railway over the Ganges from Calcutta to the north, was constructed in 1915 and protected upstream and downstream by pitched Guide banks. In 1915 and some years after, the main current was along the left bank of the river at the bridge; but gradually it changed its course towards the right bank. Early on the morning of 26th September 1933, the flow along the right bank became intense and caused a slip in the right bank, thus creating a gap in the guide bank itself. This gap rapidly widened with a very serious danger of the bridge being outflanked. It was temporarily repaired after many train loads of stone had been thrown into the gap. As nearly a crore of rupees had been spent in two years in an endeavour to hold the bridge, a Committee was appointed in August 1934 and the Poona Station was asked to carry out experiments and to report on measures required to safeguard the bridge. A long series of experiments was conducted⁽³⁹⁾and the analyses of results showed that the conditions which led to the breach were due to severe action caused by the Sara Protection

⁽³²⁾ Bombay T. P. 55 "Report on experiments carried out with models in connection with the protection of the Hardinge Bridge" by C. C. Inglis and D. V. Joglekar (1936).

Bank, 19,000 ft. ustream of the bridge on the left bank and accentuated by the Damukdia Guide bank 11,000 ft. upstream on right bank. These guide banks turned the high velocity flow into fiercely eddying, surging flow which undercut the right guide bank and so caused a serious slip, which later developed into a breach.

(a) to remove the Damukdia Guide bank;

(b) to construct by pass channel from the outside of the bend behind the Sara Protection bank to relieve the concentration of flow at the bridge;

(c) to remove the stone from the Sara Protection bank.

The committee adopted (c), being the cheapest of the three. This is reported to be successful and the bridge is safe. Sir Claude Inglis predicted that if this action (c) were taken, the attack would cease in 8 years; while this has actually occurred in 9 years.

[The experiments were carried out by Rao Bahadur D. V. Joglekar, Mr. P. D. Jaeel and Mr. S. T. Ghotankar, M. Sc.]

Supplementary problems relating to the protection of bridges were also dealt with in this connection, namely :---

(a) Falling aprons;

(b) length and alignment of guide banks;

(c) effect of high level pitching round piers on scour d/s.

(a) Falling aprons .(33) A considerable amount of basic work was done by Mr. A. R. Thomas to determine the characteristics of falling aprons; effect of dispositions of stone, and shape and size of stone on falling aprons; he was assisted by Mr. N. V. Halgi, B. Sc.

The behaviour of an apron laid on alternate layers of sand and clay was also investigated with particular reference to the Sara Protection Bank-19,000 ft. upstream of the Hardinge Bridge. Stone protection had to be heavily reinforced every year. Borings taken in this bank showed that the bank consisted of alternate layers of sand and clay. Experiments showed that stones slid down as scour occurred in the case of sand layers; but the cley layers scoured unevenly, the clay scouring into steep cliffs, so that the aprons could not rest on it uniformly but fell to the bottom. It may be said that an apron does not launch satisfactorily unless-

(i) The angle of repose of the underlying material is flatter than that of the stones; and

(ii) the material is incoherent and so scours easily and evenly.

(b) Though basic experiments were not done as regards the optimum design of guide banks, experiments in connection with the Hardinge Bridge revealed that, in that case-

(i) The length of guide banks equal to the length of the bridge-with the length of the bridge correctly designed-gave the best results;

(ii) the guide banks at right anless to the bridge gave good results;

and (iii) it is preferable to have a 120° curved head of the guide banks, the next best results being obtained with 90° heads.

(c) Scour round bridge piers⁽³⁾:--Model experiments carried out in this connection by Rao Bahadur D. V. Joglekar (with the assistance of Mr. N. V. Halgi, B. Sc.) have shown the desirability of laying stone protection round piers at as low a level as possible to conform with the natural river section. Attempts at maintaining a high level pitching have led to serious slips and very deep scour-holes downstream, as pointed out by Sir Claude Inglis to the Engineering Section of the Railway Conference Association in 1942, as under :-

(i) The maximum attack on the loose stone stone protection occurs at the nose of the pier. No stone is necessary at the tail, where sand and stone scoured from upstream are deposited.

⁽³³⁾ C. I. and H. R. Stn., A. R. (T.) for 1937-38 (pp. 68-76) and 1938-39 (pp. 37-39).

⁽³⁴⁾ C. I. and H. R. Stn. Annual Report (Tech.) for 1940-41 (p. 30) and 1941-42 (pp. 12 and 31.).

(*ii*) Where stone pitching is laid round a pier at too high a level, failure may occur due to stone launching downstream from the tail; but normally, failure occurs at the nose.

(*iii*) The stone at the nose remains undisturbed until the attack reaches a critical stage, when the stone suddenly begins to be carried away and a deep scour pit rapidly forms, reaching a depth nearly as great as had there been no protective stone.

(iv) The greater the depth at which stone is laid, the more stable it is, i.e. the more severe the attack it can withstand without being carried away.

(v) The level of the river bed appeared to have little effect on the discharge which causes failure; but such effect as there is, indicates that with a high bed-level, failure occurs with a somewhat lower discharge than if the river bed-level were low.

(vi) Because of (iii) above, it is necessary to place stone at such a level, and to use stone of such size and weight, that it will not be disturbed with the worst attack for which allowance has to be made. Stone laid at higher levels is of little value.

For similar reasons the high level pitching round the piers of the Hope Bridge on the Tapti at Surat³⁵⁾ was condemned because it distorted the natural curvature of flow, thus leading to very deep scour downstream resulting in the collapse of the city wall.

12. (B) Prevention of further erosion of the left bank of the Rupnarain River above Kolaghat Railway Bridge of the B. N. Railway⁽³⁶⁾.

The Rupnarain River—which is tidal—crosses the Bengal Nagpur Railway, 35 miles from Calcutta. Since 1938, the river was eroding its left bank and in 1943 dangerously approached the Railway line to within 1,500 ft. of it. The Railway officers suggested a series of small groynes to protect the left bank; but Sir Claude Inglis was very strongly opposed to this proposal, as the short groynes would have developed a permanent deep channel at their heads along the left bank. Two alternatives were suggested:—

(a) A long curved head upstream of the bridge;

or (b) A repelling $^{(37)}$ groyne 1,100 ft. long on left bank, pointing 60° upstream.

These were tested in tidal models and were found to give satisfactory results. The Railway accepted the second alternative being easier in construction. This was completed in 1944, and the safety of the bridge and the Railway embankment is insured: substantial shoaling has occurred near the left bank (over 12 to 18 ft. deep extending in a width of 5000' opposite the groyne) and the main flow is well away from the Railway line.

[The experiments were carried out by Mr. V. N. Padsalgikar, B.E., and Mr. P. M. Damle, B. Sc.]

13. Preventing erosion along the foreshore of the Hooghly river above Calcutta⁽³⁸⁾; tide producing apparatus.

Severe scour has been occurring along the foreshore of the Hoogly above Calcutta. Erosion takes place in every case towards the lower end of a sharp bend—as at the Dunbar Cotton Mills and the Titaghar Jute Mills—locally called a "knuckle". A "knuckle" apparently forms thus—as demonstrated in a tidal model at Poona: first of all a slip occurs, due probably to scour of the sand underlying an upper clay layer, or due to caving in of the bank by a high sub-soil water-table, or a combination of the two, resulting in the formation of a scour-hole just downstream of the slip, which eventually causes an embayment; and the knuckle is the unscoured portion of the original bank upstream of the scour-hole. During the past 20 years, hundreds of tons of boulders have been dumped along the bank in an endeavour to hold it, and (to quote the words of a local officer) "at times it looked as though this object had been achieved, as the bank built up to a considerable extent; only, however, to collapse again after the freshets."

(33) C. I. and H. R. Annual Report (Tech.) for 1940-41(p. 18), 1941-42 (p. 23); 1942-43 (p. 18); 1943 (p. 25); 1944 (p. 3) and 1945.

⁽⁸⁵⁾ I. W. E. Stn. Annual Report (Tech.) for 1944 (p. 87).

⁽³⁶⁾ L. W. E. Stn. Annual Report (Tech.) for 1944 and 1945.

⁽³⁷⁾ C. I. and H. R. Annual Report (Tech.) for 1942-43 (p. 46).

Experiments at Poona have shown that the scour-hole near Dunbar Cotton Mills can be silted up by a series of submerged piles. The case of the knuckle at Titaghar Mills is under investigation.

The experiments were done by Messrs. P. D. Jaeel, V. N. Padasalgikar under the supervision of Mr. A. R. Thomas and J. S. Reid. These experiments are being continued by Mr. S. N. Mulekar under the supervision of Rao Babadur D. V. Lackhar and M. S. Malekar under the Supervision of Rao Bahadur D. V. Joglekar and under the direction of Mr. K. K. Framji.

The tide producing machine⁽⁸⁹⁾ was designed by Mr. A. R. Thomas under the direction of Sir Claude Inglis and is working satisfactorily. It is wholly operated by water power. Automatic recorders to record the tidal curves in the model were manufactured by the Hindustan Clockworks, Poona.

14. Factors determining the size and shape of river meanders

Sir Claude Inglis analysed the data of meanders of incised rivers and of rivers in flood plains in America and India⁽⁴⁰⁾ and found that meander length and meander belt vary as the square root of the discharge. It has been a common, though erroneous belief, that meanders are a result of water with excess energy flowing with excess slope. On the contrary, the Poona Station Experiments⁽⁴⁾ have shown that a channel begins to meander when it is unable to carry the bed load with the discharge and slope available.

In general, it may be stated that when transporting a charge of material in

(a) In the upper reaches of a river, the charge washed in during heavy rain exceeds what the river can carry with the slope and discharge available and hence primary meandering occurs.

(b) Rarely in rivers, but commonly in canals, the charge in movement just equals what the discharge can carry with slope available. When this conditions persists for a sufficient length of time, the channel develops the natural shape and slope for those conditions and the channel is said to be in equilibrium.

(c) Where the induced charge is less than can be carried by the discharge with the slope available, the excess energy available causes scour and this continues until the charge in motion exceeds the equilibrium charge for the discharge and slope, causing secondary meandering.

The natural meander length depends mainly on discharge (and = 36 $\sqrt{Qdominant}$. In the case of meander belts, however, though they are mainly determined by discharge, charge and grade of material in movement relative to the discharge and slope are also very important factors, which explains why meanders may be much more highly developed in the tidal portion of a river than further upstream.

[The analysis of the river data was done by Mr. Y. K. Patil, B. E. and experiments were carried out and are being continued by Mr. P. M. Damle.

15. Rainfall and Run-off

Rain and River Gauging was started in the Bombay Deccan in 1902, and was handed over to the Special Irrigation Division in 1924. In 1930 the work was summarised," formulae being worked out for maximum floods from 'fan' and 'Deccan' catchments. This report also shows the annual run-off from catchments.

The fan catchment formula as now modified is :---maximum discharge in

 $cusecs = \frac{7000}{100}$ A , where A is the area of the catchment in square miles. Sub- $\sqrt{A+4}$, sequent work (48) at the Poona Station has shown that this formula is of general

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(90)*	O I and H. R. Stn.	Annual	Report	(Tech.)	tor 1940-41 (p. 19).
(39)	0. 1. 110 11. 10. 200,				for 1939-40 (p. 9)-
(40)	Do.		n	**	
	Do			,,	for 1942-43 (p. 51)-

 (41) Do. ", 101.192-43 (p. 0.).
 (42) Bombay T. P. 80 "A critical study of Run-off and floods of calchments of the Bombay Presidency with 4 about note on losses from lakes by evaporation " by C. C. Inglis and A. J. Desouza (1930).
 (43) Bombay T. P. 56 "Summary of the Annual Note on experiments carried out by the Irrigation Research [1986-87" and C. I and H. R. Stn, A. R. (Tech.) for 1940-41 (p. 6); 1942-48 (p. 34); 1943 (pp. 44 and 67). and 67).

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application and holds for all sizes of catchments. It is also shown by the Poona Station that an allowance must be made for the shape of the catchment and that it is entirely wrong to apply the fan catchment formula to elongated catchments.⁽⁴⁾

[The analyses of the data was done by Mr. Y. K. Patil and V. N. Padsalgikar.]

16. Flood Control

In this connection, four outstanding notes written by Sir Claude Inglis may be mentioned :--

(A) Note written for the Orissa Flood Advisory Committee—of which he was a member—on "Our present knowledge regarding the factors affecting silting and scouring of channels."⁽⁴⁶⁾

(B) Rapid westerly movement of the Kosi River⁴⁰—note written with the assistance of Mr. H. M. Vaidya, B.E.

(C) Summary of Technical enquiry into the causes of breaches in river bunds in Sind and steps required to minimise the danger of a recurrence.⁽⁴⁷⁾

and (D) Floods in the Damodar River and their control-note written with the assistance of Mr. S. V. Chitale, B.E.

A. Our present knowledge regarding the factors affecting silting and scouring of channels

In a channel in which the discharge varies, the channel obviously cannot alter its slope, width, and depth, whenever the discharge changes but it will tend to do so; and each channel is the integrated effect of all the various discharges which have flowed. The banks of channels being more resistant to scour than the bed, the biggest changes take place in the bed and it is a well-established fact that after low floods the beds of rivers rise, to be scoured out again in the next high flood. This scour is not due solely to side resistance (flume flow); but even more to the fact that the silt charge which can be carried, increases directly as the effective velocity; and hence, as the discharge increases, the transportable silt per Cft. of water also increases.

The practical application of this to a delta is that the more river channels are allowed to split up, the less the quantity of sand that can be carried.

At a bifurcation, the channel taking off from the outside of a bend takes more water and much less silt than it would do had the approach been straight; the reverse is the case when a channel takes off from the inside of a bend. As a result, the outer channel tends to increase its discharge and deepen, while the inner channel, unable to carry its excess silt burden, silts its bed with coarser silt and sand.

A single channel is much more efficient as a permanent silt carrier than two branches below a bifurcation. The policy, therefore, should be to prevent new channels opening and to help old, dying channels to close. Where, however, it is necessary to maintain both the rivers at a bifurcation, steps must be taken to maintain constant conditiors.

B. Rapid Westerly movement of the Kosi River

In January 1941, Sir Claude Inglis went on deputation to Nepal and Bihar to report on "Factors affecting the westerly movement of Kosi River."

(46) Do. do. for 1941-42 (p. 13).

⁽⁴⁴⁾ I. W. E. Station Annual Report (Tech.) 1944, page 68.

⁽⁴⁵⁾ C. I. and H. R. Station, Annual Report (Tech.) for 1939-40 (p. 6).

⁽⁴⁷⁾ I. W. E. Station, Annual Report (Tech.) for 1943 (p. 10).

In this report, he stated :---

"The Kosi has moved transversely westwards at an average rate of more than a mile a year for 50 years—not with a steady and continuous rate, but the changes had taken place in a series of rapid movements with relatively quiescent periods in between.

The cause of these violent movements of the Kosi is the excessive quantity of sand carried down from the mountains of Nepal. These mountains, which consist mainly of disintegrating shaly rock, weather rapidly.

The contours run almost North to South just below the exit from the gorge and that the lowest land lies along the right bank in this length. Downstream of the Belka Hill, the country slopes up towards the North-West. Lower down, the contours gradually change to East-West; so that conditions near the gorge are actually favourable to an easterly swing; but South of the Bhaptiahi-Durbhanga Railway line there is a wide depression, now occupied by the Tiljuga and Bahm Rivers, into which the Kosi will 'translate; so that until this is filled up to a depth of several feet, the westward progress of the Kosi will not be checked and the cost of preventing the further westerly translation of the Kosi would be prohibitive."

C. Causes of breaches in River Bunds in Sind and steps required to minimise the danger of a recurrence

During the night of 20th July 1942, water in the Indus rose steadily and soon overtopped the Sukkur-Begari Bund, some 20 miles upstream of Sukkur. The Bund breached and the discharge passing through the gaps was of the order of 90,000 cs. The breach water crossed the Shikarpur-Sukkur Road at about mile 14 and breached the Railway Line between Jamra and Ruk in several places. Breaches in the North-Western canal and the Rice Canal also occurred.

The basic reason for floods in Sind is that the Indus is an alluvial river which is gradually building up its bed on a flood plain of its own making, so that the country slopes away from the river on both banks. It was in order to prevent such floods that embankments were built.

There is a widespread belief in India that bunds are the cause of the rise of water levels in the river. Embanking, where the slope of the delta is insufficient to carry the charge of bed sand to the sea and especially embanking on one bank—as in the case of the Damodar in Bengal—is undoubtedly harmful; but when the slope is sufficient, river control by continuous embankments causes the river to be contained to a greater extent in its channels and so double embanking tends to reduce the rate of deterioration. The rate of rise of river beds is largely due to erosion; but little relief is likely in the Indus for some years to come except from dams checking bed movement, in Sir Claude's opinion.

In this report, Sir Claude Inglis also made recommendations regarding the proper design and maintenance of bunds.

(D) Floods in the Damodar River and their control

Sir Claude Inglis served during 1944 on the Damodar Committee, appointed by the Govt. of India, and was also a member of the Bengal Government Damodar Flood Enquiry Committee, which was set up to advise on permanent measures to control floods in the Damodar River and, in particular, to advise on the utility of the scheme of flood regulating reservoirs prepared in the year 1920 by Mr. E. L. Glass, then Executive Engineer, Central Provinces. Sir Claude submitted a note on "How to control-floods in the Damodar River" to the Committee in December 1944, a summary of which is given below :--

The area over which waters from the Damodar used to spill was both healthy and fertile; but the river used to breach periodically and form new channels; so that when the country began to get more settled, flank embankments were built by the people, with the object of preventing heavy spill, breaches, and avulsion into new channels. Breaches still continued to occur. Experience showed that the left embankment was not strong enough to prevent breaches occurring; so it was held that the embankments increased damage; and it was decided, about 80 years ago, to remove a large part of the Right Embankment in order to allow flood water to spill over the right bank to a greater extent, to relieve congestion.

After a period of respite, breaches again began to occur in the left embankment, though less frequently, but people on the right bank claimed that water was thus concentrated in their lands, causing much damage, and when a maximum flood of about $6\frac{1}{3}$ lacs cusecs occurred in 1918, great damage resulted in Burdwan and the surrounding country. Mr. Glass of the Central Provinces, P. W. D., was brought on special duty to prepare a scheme planned by Mr. Addams-William, Superintending Engineer on Special Duty, Bengal, for constructing detention reservoirs on the Damodar and Barakar Rivers. These were to be designed to reduce flood discharges to about half.

These proposals were not, however, acted on. After a flood of only $3\frac{1}{2}$ lacs cusecs had caused the breach of 1943, which put the E. I. Railway and Grand Trunk Road out of commission for several months, it was realized that the whole question must be re-examined, both as an immediate war measure and as a long-term problem. Two alternative methods were suggested by the Committee to meet the danger:

(i) to make the left embankment so strong that the danger of a breach occurring in the near future would be reduced to a minimum;

or (ii) to divert a large part of the water and sand of the Damodar River into the Darkeswar River.

The second alternative was, however, found to be impossible as it involved very heavy cutting through exceedingly tough material. Therefore, the Committee had to recommend alternative No. (i), namely greatly strengthening the left embankment, and giving a 5 ft. free-board above the hignest estimated flood level. It was also proposed that flood detention reservoirs should be built to reduce the maximum flood discharge at Burdwan from $6^{1/2}$ cusecs to one lac cusecs. The major objection raised against building reservoirs was that they would silt up rapidly owing to the very heavy charge of sand and silt brought down by the Damodar and Barakar Rivers; and if the reservoirs were used for developing hydro-electric energy and for flood routing, the rate of silting would be much greater; because flood water would then have to be stored at the very time when the quanti-ty of silt is at a maximum. Under such conditions a considerable part of their capacity would be lost in 50 to 100 years. The Committee, therefore, proposed that reafforestation, land management and sand trapping should be done on a large scale in the catchment. If the Committee's proposals are carried out in full, they will bring the Damodar under complete control, thus removing the threat to the Port of Calcutta, and will also make it possible by building a barrage between Silna and Burdwan to flush and irrigate the country on the two banks of the River, thus improving the fertility of the country and the health of the people.

17. Factors affecting flow in alluvial channels

A considerable amount of work is being done at the Poona Station, both by analysing the hydraulic data of channels in the Punjab, U. P. and Sind and by model experiments to see why channels diverge from regime according to the Lacey theory and how to design channels. In this connection, the following

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PART IV

notes written by Sir Chaude Inglis with the help of Rao Bahadur D. V. Joglekar are of great interest: -

(A) Divergence from regime in stable channels in alluvium⁴⁸.

(B) Cause of accretion of river bed at Sukkur Barrage and how to reduce rate of rise⁽¹⁹⁾.

(C) The design of canals in Sind for new perennial canals and Inundation Canals⁶⁶.

These notes are summarised below :---

(A) Divergence from regime in stable channels in alluvium

While travelling widely in recent years in connection with river training and control, Sir Claude Inglis met many Engineers who complained that a large number of stable channels did not agree with the stable channels according to Lacey formulas. As a result, he wrote a note on the above subject explaining the factors causing divergence from regime.

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

The theory also presupposes that for any particular discharge and mean diameter of bed sand, the 'charge' is normal; that is to say, its size distribution is normal and the channel is moving with "minimum transported load consistent with a fully active bed."

In practice, water temperatures vary throughout the year, the variation being much greater in some Provinces than in others and also greater in small than large channels. Similarly, the silt charge and bed-material size-distribution vary at different times of year; hence it must be clearly understood that the Lacey formulas are based on statistical average values for the conditions inherent in his data.

On plotting the data of mean grade of bed-sand against discharges of the various sites on the Lower Chenab Canal and the Lower Jhelum Canal, Punjab, and on the Jamrao Canal, Sind, it was found that m (mean diameter of bed-material) is not constant as assumed by Lacey but varies with the discharge. The reasons for this variation are:—

(i) dilution;

(ii) attrition;

and/or (iii) sand selection at heads of off-takes.

The analysis of the data definitely shows that in a channel carrying its load forward, the silt factor

$$\begin{array}{l} f_{\rm m} = \sqrt{f_1 \cdot f_{\rm s}} \\ = \sqrt{0.75 (V^2/R) \times 193^{\circ} 15 R^{1/3} S^{2/3}} \\ = 12 \ V. \ S^{1/3} / R^{1/3} \end{array}$$

 f_m , the new silt factor is constant irrespective of whether the sand load is equal to the regime load or not; and under such conditions f_s increases and f_1 decreases with Q, the amount of divergence increasing with divergence of sand charge from regime charge. At regime $f_1 = f_s = f_m$; which was Lacey's original conception of regime.

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⁽⁴⁸⁾ Central Board of Irrigation, Annual Report (T.) for 1941 and C. I. & H. R. Stn. Annual Report (Tech.) for 1941-42 (P. 2).

⁽⁴⁹⁾ Indian Waterways Experiment Station Annual Report (Tech.) for 1944 (P. 99) and for 1945.

⁽⁵⁰⁾ Indian Waterways Experiment Stn, Annual Report (Tech.) for 1943, (P. 21).

The following definitions are given in his Note :---

A channel is said to be "in regime" when the load is natural for the grade of sand and the discharge flowing, giving a "normal population" of bed-sand mix.

A channel is said to be "stable" when it does not alter appreciably from year to year. Stability occurs even though the sand relation does not agree with the regime relation, the charge for the grade and discharge being either greater or less than regime charge.

There will then be a regime charge for every discharge and grade, which may, alternatively, be called a "regime grade for discharge and load."

As m varies in a canal systems, it follows that if f_m is a contant, the formula $f=1.76\sqrt{m}$ can only hold for one discharge—at regime. As regards "divergence from regime dimensions"; this note has clearly shown that this is due to divergence from regime balance between discharge, grade and load, and is not explainable in terms of good or bad maintenance, i.e. shock. Where the load exceeds the regime load, little labitude in design is possible; but where the charge is sub-regime, considerable latitude is possible in shape and slope.

[Basic experiments in this connection were done by Mr. S. C. Desai, B.E., and Mr. N V. Halgi and are being continued by Mr.G.T. Wadekar, B.E.]

(B) Design of canals in Sind.

Sir Claude Inglis was requested by the Chief Engineer, Sind, to advise him on the design of new canals which are to be constructed in the near future. In this connection the above note was written, a brief review of which is given below:—

The Poona Station has repeatedly emphasized that the shape and slope of a canal depend to a marked extent on the quantity of sand normally passing down a canal.

If we compare the Rice Canal taking off from the right bank with the Rohri Canal taking off from the left bank at Sukkur, the designed slopes of both of which are 1 in 12,000, we find that in 1942, though the bed-width was practically the same, yet the slope in the head reach of the Rohri Canal was 1 in 19,800 for full supply discharge (10,800 cusecs), whereas the corresponding slope in the Rice Canal on the right bank was 1 in 7,300 for F. S. Q.=10,200 cusecs.

The very flat slopes which have developed in the Rohri and Eastern Nara Canals are not due to their having been constructed to a wider section than is natural in Sind but to another factor, namely that where the bed is very smooth, as it was originally in these two canals, the sand charge is swept along the bed and the bed gradually scours—due to the high velocity and consequent low depth of flow with resultant retrogression at the head of the canal and downstream of masonry works. This has been confirmed by experiments carried out at Poona. The data on Jamrao Canal and on channels on the Rohri and Dadu Canals was considered in deriving "normal design" formulas, which assumed that excess sand had been excluded at the head of the parent channels. The formulas for perennial canals worked out to

 $S = 22/Q^{-135}$

 $B=1.23 \ Q^{.55}$, where B is the bed-width of a channel

and $D_k = 90 Q^{-30}$, D_k being depth over the central portion.

Side slopes of $1^{1}/4:1$ were recommended for discharges exceeding 300 cusecs, 1:1 for discharges less than 300 cusecs.

The question of Inundation Canals was next investigated. The data for the following channels were considered :-

- (i) Sukkur Barrage Canals,
- (ii) Jamrao Canal,
- (iii) Western Nara Canal,
- (ir) Ghar Canal,
- (r) Fuleli Canal,

The outstanding feature of the inundation Canals data was the excessive slopes in nearly all cases as compared with those in perennial canals; and it appears that this must be almost entirely due to the high degree of tortuosity and varying widths existing in these old inundation canals. It was, therefore, obvious that if it was desirable to flatten the slope so as to gain command or to lower the level of offtake from the river, it might well be that it would be cheaper and better to construct new channels for considerable lengths; but in some cases, remodelling can be done by straightening the canals and making them more uniform in section and constructing regulators at suitable intervals."

(C) Cause of Accretion of River bed at Sukkur Barrage and how to reduce rate of rise.

Some officers in Sind have blamed the approach channel-which was constructed in 1939-40 for excluding bed sand from the Right Bank Canals at Sukkur-for accretion in the river at the Sukkur Barrage. It was, however, shown that accretion was taking place long before the approach channel was constructed and, if anything, the rate of accretion has slowed down since, and thus the approach channel cannot be held responsible for it. The sequence which leads to the rise of water levels is as follows :-

Heading up water at the barrage to pass water into the canals when the river discharge is low, causes ponding and, hence, the deposition of silt and sand upstream.

As the river discharge increases, less and less heading up is required and more and more bed material scoured from the bed passes from the barrage until eventually a stage is reached when the gates are almost fully opened. The water slope and velocity from above Sukkur to below the barrage are then abnormally high, and excess movement of bed-sand occurs, the excess charge being swept through the barrage to accumulate downstream where the slope is insufficient to carry the excess charge forward, causing a rise of bed level downstream; and this is the explanation of the said rise of water levels downstream of Sukkur.

The Sukkur Barrage came into operation in 1933. There was retrogression in the first 2 years, after which there has been a fairly steady rise in water levels, in the immediate vicinity of the Barrage, for specific discharges. This accretion can be largely stopped by reducing ponding, i.e., if the full supply level in the Rice Canal is reduced by 1.5 ft. below the designed full supply level by giving it a flatter slope. To do this it will be necessary to lower the bed at the head by 3 ft.-thus increasing the depth by 1.5 ft. from 11.7 to 13.2 ft., the slope flattening to 1 in 16,000 with the present width. It will also be necessary, at the same time, to remove the coarse sand-the residue of several feet of scoured bed material-so that the canal can be run with the new flatter slope, now rendered possible by the exclusion of most of the bed sand.

18. Water Supply

(A) Augmentation of Bombay Water Supply (51, 154)

- -Note written with the assistance of Mr. S. V Chitale.
- (51) I. W. E. Stn., Annual Report (Tech.) for 1913, page 2.

⁽⁵²⁾ I. W. E. Stn., Annual Report (Tech.) for 1911, page 70.

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(B) Present and future water requirements and supplies in the Jumna at Delhi $^{\rm (32)}$.

-Note written with the assistance of Mr. S. T. Ghotankar and Mr. S. V. Chitale.

(A) Augmentation of Bombay Water Supply.

Sir Claude Inglis had been working during 1943-44 on a Committee of two with Sir M. Visvesvaraya. In their report, the following points were brought out:-

"The bulk of Bombay's water supply is obtained from a large reservoir known as Tansa Lake situated 54 miles north of Bombay. The supply available from this storage is about 35,000 million gallons=5,600 million cft. Water is conveyed from Tansa through two 72" steel pipes, which have a maximum carrying capacity of 90 million gallons per day.

The Committee were asked to report on a scheme to build a dam across the Vaitarna valley to divert supplies from the Vaitarna river through a tunnel five miles long into the existing Tansa Lake. This scheme was not recommended on the grounds that:—

Water from a catchment, through which both the main G. I. P. Railway line and the Grand Trunk Road to Northern India pass, with a population exceeding 22,000, would get mixed with the relatively pure supply from Tansa, causing a considerable deterioration of the purity of the entire supply. This would, it is anticipated, necessitate filtering the whole supply at great cost.

Instead of this, an entirely new scheme, to bring a piped water supply to Ghatkoper from the tail waters of the Tata Power Supply Co. at Bhivpuri and thence to Bombay through the existing pipe system was recommended. The hydraulic gradient available will be $3\frac{1}{2}$ ft. per mile, compared with $2\frac{1}{2}$ ft. per mile from Tansa, and potentially the Bhivpuri supply has a head which is 75 ft. higher than from Tansa, when Tansa Lake is full, and 110 ft. higher at the end of the dry season. With this scheme, the pressure in some parts of the city where pressures are low can be appreciably increased. This scheme can be carried through expeditiously and can be extended gradually, as required. This is important; because should a famine year or 2 years of shortage occur, and such conditions are long overdue, the present supply would be totally inadequate for the present population and industries."

(B) Present and future water requirements and supplies in the Jumna at Delhi.

In June 1943, a meeting of officers of the United Province and the Punjab was held at Nainital at which it was tentatively agreed that two dams, called Tons and Giri, designed to store $1\frac{1}{2}$ million acre-feet, should be constructed, the cost and supply being shared in the ratio of 2:1 between the Punjab and United Province.

A technical Committee (of which Sir Claude Inglis was a member) was appointed by the Government of India in June 1943 to investigate the question of adequate supply of water for the Delhi Electric Power House in the hot weather and the intake of unfiltered water for New Delhi. Sir Claude submitted a report to the Committee which also dealt with the effect which the above storages would have on Delhi water supply. This investigation was of considerable technical interest, as it brought out the following points:---

(i) there is a direct relationship between floods in monsoon and regeneration in the following cold weather, i.e. the bigger the flood discharge in a particular year, the larger is the regenerated water supply lower down, in the following cold weather;

(ii) storages in Tons and Giri catchments would not merely deplete supplies during the flood season, but would also deplete regeneration in the fair season;

⁽⁵³⁾ I. W. E. Stn., Annual Report (Tech.) for 1944, page 91.

(iii) the increased supply drawn by the Western Jumna Caual since it was enlarged has considerably reduced the supply passing below Tajowala and this depletion may be expected to increase;

(iv) the supply in the Jumna at Delhi is already inadequate in years of short supply towards the end of the fair season;

and (v) the passing of increased supplies into the Western Jumma Canal and still more so, the construction of the Tons and Giri dams, would seriously deplete the supplies passing down the river to Delhi, and the supply would be inadequate not merely towards the end of the fair season but also for long periods in the fair season in years of short supply.

19. Dissipation of energy below flumes, falls and bridges

When the designs for the Sukkur Canals were being considered, it was found that none of the standard canal fall designs gave satisfactory results. A new type was evolved—" the proportional, standing wave Flume Meter Fall, with a baffle and deflector". This design, when combined with a Regulator, is cheaper than any alternative and was found to be highly efficient in action-even where, as a result of scour, heavy retrogression had occurred further downstream—. It is also a highly accurate meter. Hundreds of these falls have been constructed and have proved highly satisfactory. (7, 54 and 50).

Experiments for standardising the design were conducted by Mr. J. S. Reid, Rao Bahadur D. V. Joglekar, Mr. S. T. Ghotankar, Mr. N. V. Halgi and Mr. P. H. Wagh, B.Sc.

Deep scour-holes developed below bridges on the Rice Canalis, the North West Canal and the Rohri Canal in Sind. This was due to the rigid structure constricting the waterway across the natural channels thus, increasing the velocity and discharge intensity, resulting in jetting causing deep scour-holes, with return flow on both banks downstream of bridges causing heavy erosion of banks (bellying). Experiments were done to dissipate excess energy by depressing the floor under the bridge and providing a deflector wall and permeable groynes on banks. These have proved entirely successful. Basic investigations on are now in hand to correlate the depth of scour-hole with discharge intensity and depth at the bridge and bed material.

(Experiments were carried out by Mr. J. M. Parikh and are continued by Mr. P. K. Kulkarni)

Specific investigations on models have also been made for dissipating energy below badly-designed flat weirs, which were threatened with disaster by deep adjacent scour-holes and undermining. The remedial measures suggested, such as glacis slopes, baffles, and deflectors have in all cases proved most effective. Some of the examples are given below :-

(1) Tando Mastikhan Fall on the Rohri Canal (Sind).⁽⁶⁴⁾

(2) Protection of apron downstream of approach channel sluices of the Sukkur Barrage.(58)

(3) Prevention of scour downstream of the Marala Weir (Punjab).¹⁶⁰

(4) Baffle and deflector Design downstream of the Mula River Causeway near Rahuri (Bombay).⁶⁰⁾

(5) Dissipation of energy below Bhimgoda Weir on the Ganges at Hardwar (U. P.).(60)

(6) Deflector design downstream of the Sarda Barrage at Banbassa (U.P.).(61)

(54) Bombay T. P. 44 "The Dissipation of Energy below Fails" by O. C. Inglis and D. V. Joglekar.

(55) Indian Waterways Experiment Stn., Annual Report (Tech.) for 1944 (p. 29).

(56) O. I. and H. R. Stn., Annual Report (Tech.) for 1942-48 (p. 21), and for 1944 (p. 28).

(57) I. W. E. Stn., Annual Report (Tech.) for 1945.

(59) Bombay P. W. D., (Tech.) Paper 51.

(60) Bombay P. W. D., (Tech.) Paper 58.

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⁽⁵⁸⁾ O. I. and H. R. Annual Report (Tech.) for 1939-40, (p. 19).

⁽⁶¹⁾ C. I. and H. R. Annual Report (Tech.) for 1940-41 (p. 24).

(7) Deflector design for the proposed, new undersluices of the Son Anicut at $Dehri.^{(62)}$

(8) Dissipation of energy in the waste weir channel of the Spin Karez Reservoir, Baluchistan.⁽⁶³⁾

(9) Dissipation of energy below the masonry waste weir of the Timba Tank at Harsol, Gujarat (Bombay).⁽⁶⁴⁾

(10) Durani Outfall channel at Quetta, Baluchistan.⁽⁵⁾

(11) Prevention of scour downstream of Girns Canal Aqueduct No. 4 $(Bombay).^{(66)}$

(12) Baffle channel for dissipating energy in the tail channel of the proposed Jamshedpur Siphon Spillway. $^{(\sigma)}$

(13) Preventing damage downstream of the Centre Sluices of the Jobra Weir on the Mahanadi at Cuttack.⁽⁸⁾

20. Maximum depth of scour at heads of guide banks and groynes, pier noses and downstream of bridges.⁽⁶⁹⁾

Deep scour occurs-

(i) at noses of piers due to diving flow;

(*ii*) at ends of guide banks and groynes due to concentrated flow swinging round their extremities; and

(*iii*) downstream of bridges due to eddies shed from high pavement or stone protection placed round bridge piers.

In all these cases, the scour is due to the bed velocity being far in excess of what is natural for the discharge, so that v^*/d , which is related to turbulence, is very high. 15 years ago, Lacey gave approximate figures for maximum depths of scour at bends, by relating these to his depth formula for regime conditions in a straight channel.

$D = 47 (Q/f)^{1/3}$

where D = effective depth,

Q = discharge and

f=Lacey's silt factor which approximates to $1.76\sqrt{m}$

(where m =the weighted mean diameter of the bed material).

In a moderate bend, he considered that the maximum depth might be 15 times the regime depth and in a severe bend 1.75 times; while in a right-angled bend, he assumed twice the regime depth.

Though Lacey brought grade of material into the formula to the 1/6 power, recently it has been argued in India that depth of scour downstream of rigid structures is unaffected by bed material; and as the range of bed material is generally small (0.25 to '4 mm.), data could not be taken either to support or disprove this belief; but, though flow pattern is an important factor, indications have been obtained at Poona from subsequent data covering a range of

m=0.06 to 0.37 mm. that f is undoubtedly also a factor and that $f^{1/3}$ is probably of the right order; thus confirming, yet once again, the value and soundness of Lacey's *original* concepts; but scour may be much greater than Lacey estimated.

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⁽⁶²⁾ I. W. E. Station for 1914 (p. 20).

⁽⁶³⁾ C. I. and H. R. Annual Report (Tech.) for 1937-38 (p. 46).

⁽⁶⁴⁾ C. I. and H. R. Annual Report (Tech.) for 1937-38 (p. 84).

⁽⁶⁵⁾ C. I. and H. R. Annual Report (Tech.) for 1938-39 (p. 53).

⁽⁶⁶⁾ C. I. and H. R. Annual Report (Tech.) for 1989-40 (p. 23).

⁽⁶⁷⁾ C. I. and H. R. Annual Report (Tech.) for 1942-43 (p. 20).

⁽⁶⁸⁾ C. I. and H. R. Annual Report (Tech.) for 1946.

⁽⁶⁹⁾ Note by Sir Claude Inglis and Rao Bahadur Joglekar printed in the Indian Waterways Experiment Station, Annual Report (Tech.) for 1944, page 74.

The data so far available show :---

1. (a) the maximum scour downstream of bridges with high level protection of the order of $d_s = 4 D_{Lacey} = 4 \times 473 (Q f)^{1/3}$,

- (b) scour at straight spurs facing upstream is of the order of $d_s = 3.8 D_{Lacey}$, with steeply sloping heads $(1\frac{1}{2}:1)$ and = $2.25 \text{ D}_{\text{Lacey}}$ with long, flat, sloping heads (1 in 20),
- (c) scour at large-radius noses of guide banks, $d_s = 2.75 D_{Lacey}$,
- (d) the maximum scour round bridge piers, $d_s = 2 D_{Lacey}$.
- (e) Scour at spurs along river banks,

$$d_s = 1.7 D_{Lacev}$$
 to 3.8 D_{Lacev}

varying according to conditions controlling severity of flow, such as length of projection, sharpness of curvature, angle, and position relative to the embayment.

21. High Coefficient Weirs

It is important in certain cases to be able to discharge large quantity of water with a small depth over the crest, so that the crest may be set at a higher level for increasing storage capacity without raising the maximum flood level above the safe limit.

By model experiments, a weir profile has been evolved at Poona which has a higher coefficient than any other known profile-i.e. it passes more discharge for a given depth than any weir yet constructed.

A design of a high coefficient weir for Lake Arthur Hill, Bhandardara (Bombay Presidency) was evolved to raise the weir level, so as to increase storage capacity without increasing high flood level. As results were not geometrically-similar, the coefficient had to be determined by extrapolating the coefficients obtained with models of different scales.⁷⁰⁾ This investigation was done by Mr. Thomas with the assistance of Messrs. P. D. Jacel, S. T. Ghotankar, V. N. Padsalgikar and P. M. Damle.

Similarly, designs of profiles to give required conditions of design were worked out for the pick-up weir on the Malaprabha in Bijapur District " and Tansa Dam (19) in Boubay Presidency and Lower Sind Barrage. (70)

22. Siphon Spillways

Experiments were carried out at Poona with models of various designs of siphon spillways including a 500 cs. full-scale siphon. Saddle type siphons are now generally used where gates were hitherto constructed to pass floods from storage reservoirs. They require little attention, are entirely automatic in action, do not leak, and where sufficient head is available, pass a much larger discharge per foot run than other spillways.

The general objection to large saddle-siphons has been that a high coeffi-cient siphon has poor priming qualities, and vice versa. Thus, in the case of the Indore-type siphons-which are provided with a baby siphon to cause early priming— the coefficient was found to be as low as 0.43, the siphons being reported to prime at a depth of 6" above their crest level. The design of saddle type siphons (75) has been improved at the Station to give a coefficient of 0.72 with a priming depth of 6".

[The experiments were carried out by Rao Bahadur R. G. Islur Rao Bahadur D. V. Joglekar, Mr. K. N. Velankar, Mr. P. M. Damle and Mr. N. K. R. Iyengar.]

(73) O. I. and H. R. Station, Annual Report (Tech.) for 1940-41 (p. 27); 1941-42 (p. 26); 1942-43 (p. 24) and 1945.

⁽⁷⁰⁾ C. I. and H. R. Annual Report (Tech.) for 1939.40 (p. 27); 1940.41 (p. 42).

⁽⁷¹⁾ C. I. and H. R. Annual Report (Tech.) for 1939-40 (p. 25).

⁽⁷²⁾ Indian Waterways Experiment Station Annual Report (Tech.) for 1945.

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A Unique design of siphon has been evolved by Mr. Ganesh Iyer (Mysore)—called the Volute siphon—, which has a very high coefficient of discharge, viz. 0.8 and a very low priming depth (2" to 3").

The design of the platform level of these siphons⁽⁷⁴⁾ so as to cause no interference in the working of two or more Volute Syphons as well as other details of these siphons have been evolved or modified after experiments made at the Poona Station. The dissipation of energy and reduction of scour below these siphons is now under investigation at Poona.

23. Model Technique

A detailed note "" was written on the methods followed in fixing model scale ratios and overcoming model limitations, based on experience gained at the Poona Station. This, it is hoped, will serve as a useful guide to research workers serving in the same field. The note explains why some experiments give reliable results, while others depend to a marked extent on making correct assumptions, interpretation and on the experience of the investigator. Pitfalls to be avoided, and difficulties to be overcome, are explained with suggestions as to how to overcome inherent deficiencies in certain types of models.

⁽⁷⁴⁾ I. W. E. Station, Annual Report (Tech.) for 1942-43; (p. 47) and for 1945.

⁽⁷⁵⁾ A Note by Rao Bahadur D. V. Joglekar and Sir Claude Inglis included in the Indian Waterways Experiment Station Annual Report for 1945.
	Subject.	Bombay Tech. Paper No. and year.	File No. and/or year.	Annual Re	port (Tech.)
	River Training				
1.	Experiments with Falling Aprons.			IRD	1935-36
2.	Experiments with models in connection with the protection of the Hardinge Bridge (Ben- gal-Assam Rly.) on the Ganges by C. C. Inglis.	55 (1936) 		•••	
3.	The Ganges above the Hardinge Bridge: Correlating factors, especially discharge, silt charge and slope with field results.			IRD CIHR "	1935-36 1937-38 1938-39 1939-40
4.	The Sina River and Causeway at Mile 5 of the Ahmednagar- Manmad Road (Dist. Ahmed- nagar Bombay)	58 (1938)		CIHR	1937-38
5.	The Mula River at Rahuri (Dist. Ahmednagar, Bombay Presidency) to determine how best to divert the river over causeway which interferes with the working of the form	58 (1938)		CHE	1937-38
6.	The Sarda River above the Bar- rage at Banbassa, U. P.	58 (193 ₂)		CIHR "	1937-38 1938-39 1939-40
7.	Training the Jumna River at Delhi Gate Pumping Station.			iwes	1937-38 1838-39 1939-40 1944 1945
8.	Training the Watrak River to prevent the town of Kaira in Gujerat being damaged by scour			CIHR "	1937-38 1938-39
9.	Training the Nala passing through Aqueduct 14 of the Nira Right Bank Canal.			99] 19	1937-38 1938-39
10.	A model of the Brahmaputra and the proposed Bengal Assam Railway Bridge at Amingaon to determine effect of bridge			11 22	1937-38 1938-39
11.	The Jumna River near Palla village (Delhi Province).			ور	1938-39
12.	The Auranga River to protect Bhadeli village, Dist. Surat, (Bombay)			" 1	933-39
13.	Remodelling the intake for the Raunahi Pumping Station of Fyzabad Pumping and Gogra			,, ,	1933-39
14.	Electric Supply Scheme (U.P.) The Jumna River at Wazirabad.			, 1 , 1	939-40 942-43

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Key:—IRD71 = Irrigation Research Division File No. 71.

CIHR = Central Irrigation and Hydrodynamic Research. IWES ≡ Indian Waterways Experiment Station.

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16.	The Malaprabha River: Experi- ments to compare relative merits of various alternative measures for relieving heavy scour at Railway Abut- ment.			CIHR IWES	1942-43 1945
17.	Deterioration of deltaic rivers: Experiments with the model of Nuna and Pothi Rivers in the Mahanadi Delta (Orissa).			CIHR	1942-43
18.	The Indus River upstream of the Bukkur Gorge in con- nection with the protection of the threatened N. W. Railway embankment.		· · · · ·	CIHR IWES	1942-43 1943
19.	Use of "attracting" spurs and is- lands for training a river up- stream of a masonry work.		•••	CIHR IWES	$1942-43\\1943$
20,	Advice regarding training the Beas River by means of spurs.		•••	CIHR	1942- 43
21.	Narava Maghadri River, Vizaga- patam Port.	Case pending.			
22.	The Mahanadi and Katjuri Rivers near Cuttack (Orissa).			IWES	$1943 \\ 1945$
23.	Jamshoro Bund on the left bank of the Indus near Hyderabad (Sind).			IWES	1943
24.	Steps required to prevent damage to the Railway embankment west of the Alexandra Bridge over the Chenab River near Wizirabad.			"	1943
25.	Realignment of the Rohri-Quetta Railway necessitated by the danger of breaches in the mar- ginal bunds of the Indus above Sukkar.		•••	"	1948 -
26.	How to train and control Beki, Balkadoba and Manas Rivers (Bengal-Assam Railway).		•••	35	1943
27.	How to prevent outflanking of Barrage on Khari River at Raipur, near Ahmedabad.			"	1943
28.	Advice regarding whether river ghat on Brahmputra was likely to be eroded.			>>	1943
29.	Control of erosion of Kshipra River at Ujjain.			*	1943

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31.	Prevention of further erosion of the left bank of the Rupnarain River above Kolaghat Ruilway Bridge of the Bengal-Nagpur Railway.		· · · · ·	., 1944 ., 1915
32.	Factors favourable for opening a new cut-off channel.	•••	•••	. 1914
33.	Damage to lands bordering the Tapti and Narbada and Sabar- mati Rivers.		•••	n 1914
34.	Damage in 1944 to the Surat- Rander Road and the City wall of Surat downstream of Hope Bridge and recommendations to ameliorate present condi- tions.			., 1911
35.	River Training and control by pitched "attracting" islands.		•••	" 1914
36.	Cause of accretion of river bed at Sukkur Barrage and how to reduce rate of rise.	•••	••••	1944 1945
37.	A stone wall built to protect a palace acts as attracting groyne and holds a river permanently along its face.			1944
38.	Proposed diversion of 40,000 cusecs to relieve flooding at Gorakhpur on Oudh and Tirhut Railway.			, 1944
39.	Bridge on Beas River near Beas Station on Lahore-Ludhiana Section, N. W. Rly.	•••		., 1944
40.	Training works at Dingra Ghat on the Mahananda River in Bengal.	•••		" 1944 "
41.	Embankment near Nanpara (U. P.) to divert the whole of the Saraju river into the Gograi, thus preventing spill flowing into the Sarju-Tirhi channels.			, 1 044
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44.	Advice regarding Samsata spur on Sutlej River on the N. W. Railway to prevent dangerous erosion.			33	1944
45.	Advice how to secure an adequate supply of water for the river- side Power Station of the Cawnpore Electric Supply Corporation.			11	1944
46.	Advice regarding action to be taken regarding erosion of spur 2 in the Indus River near Umerkhel.		•••	"	1944
47.	Prevention of erosion at Neamati in Assam.			"	1011
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1.	Note on Orissa Flood Advisory Committee.	***	•••	CIHR	1939-40
2.	Discussion by C. C. Inglis on Institute of Engineers (India) Paper (1941) "Ganges Flood and its lessons".		1941		•••
3.	Note on Denudation, . Erosion and Floods.		•••	IWES	1943
4.	Proposals to prevent recurrence of damage resulting from breaches in the Damodar River (Left Embankment).			33	1943
5.	Proposal to construct a Barrage on Tank Zam in the N. W. F. Province to split up water between several areas.			**	1944
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1.	Notes on the visit to the Sutlej valley Project by C. C. Inglis.	•••	IRD 115 (1930)	•••
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З.	"Note on Silt Exclusion from Canals—First report on experi- ments with the full-width model of the Sukkur Barrage at Khadakvasla" by C. C. Inglis and D. V. Joglekar.	46 (1933)		
4.	Control of sand entering the head regulator of Nabisar Branch Ex-Mithrao Canal, Sind.		IRD 77 (1933-34)	
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6.	The Indus and the Lloyd Bar- rage at Sukkur-Control of sand entering the Right Bank Canals.		IRD 76 (1933-34)	CIHB 1937-38 1938-39 1939-40 1940-41 1941-42 IWES 1943
7.	Factors affecting exclusion of bed sand from canals taking off from alluvial rivers: (a) Curvature of flow induced by the regulation of Barrage Gates (b) Discharge per ft. width entering canal:. total discharge heing the same.		IRD 78 (1934-35)	
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11.	Measures to prevent excess sand entering the Vadavar Channel of the Lower Colercon System (Madras Presidency).			**	1937-38
12.	Exclusion of bed sand from Jamrao Canal (Sind).			"	1937-58
13.	Silting of the Grand Anicut Canal, Cauvery River (Madras).	•••		"	1938-39
14.	Discussion by C. C. Inglis on Punjab Engineering Congress Paper No. 249 "Silt and its Transportation" by Ishardas.		1941		
15.	Discussion by C. C. Inglis on Punjab Engineering Congress Paper No. 242 "Control of flow on Curves in open channels" by A. N. Wilson.		1941		
16.	Model experiments to verify the cfliciency of the proposal made by the Station to exclude bed sand from the Sarda Canal by providing a regulator in the short Banbassa Spur.	••••		CIHR	1941-42
17.	Remodelling the Son Anicut at Dehri (Bihar), for Control of Sand entering the canals.			." IWES "	1941-42 1942-43 1943 1944
18.	Advice regarding control of shingle entry into the Western Jumna Canal, Pujab.			CIHR	1942-43
19.	Fluctuations of grade of materials exposed on the beds of the Eastern Nara and Rohri Canals (Sind).		•	IWES	1943
20.	Design of Lower Sind Barrage at Hajipur.	•••		IWES	1944 1945
21.	Advice regarding whether it was possible to prevent sand enter- ing the Surada Reservoir at its Padma Intake in Orissa.	•••		**	1944
	5. Flow in Alluvial Channels				
1.	Discussion by C. C. Inglis of Punjab Engineering Congress Paper No. 192 on "Silt move- ment and designs of channels" by Dr. N. K. Bose.		1936		

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35 niles downstream of Kala bagh, N. W. F. Province).			" 1943
18. Transportation of peoples in pebble-bed flumes.		· • •	1943
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2.	Record of turbulence by Hot wire and Oscillograph.		•••	CIHR	1938-39
3.	Manning and Lacey rugosity co- efficients with smooth boundaries.			**	1939-40
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2.	Erosien at Ratuagiri			CIHR	1942-43
3.	How to reduce waves in a dock caused by swell in a harbour.			IWES	1943
4.	Advice regarding the control of erosion along the shore of Viza- gapattam harbour.		Case pending data awaited.		•••
9.	Falling Aprons & Guide Banks.				
1.	Effect of length of Guide Banks of the Hardinge Bridge on flow.	58 (1938)			•••
2.	Falling aprons-factors affecting the	58 (1938)		CIHR "	1937-38 1938-39
	0. Scour at and below Rigid Structures.				
1.	Design of glucis and deflector downstream of the Mula River Causeway to prevent under- mining.	58 (1938)		CIHR	1937-38
2.	Godavari Right Bank and Left Bank Head Regulators—Pre- vention of scour downstream of the—			23	1937-38

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ort (Tech.)	Annual F	File No. and/or year.	Bombay Tech. Paper No. and year.	Subject.	
		5 647	58 (1938)	The sluices of the Bhimgoda Weir across the Ganges at Hardwar (U.P.)-To ascertain	8.
•	•			how best to prevent jetting and abrasion of the downstream pavement.	
138-39 39-40 40-41 1945	CIHR " IWËS	•••		Protection of bridge piers against scour.	4.
39-40	"			Giraa Canal Aqueduct No. 4 (Bombay Presidency),-Preven- tion of scour downstream of the	5.
89-40	• 33			Estimated scour at proposed Damodar Bridge at Burdwan (Bengal).	6.
1 40-41	**		, 	Son Anicut at Dehri (Bihar), Scour at noses of divide walls upstream and downstream of Anicut and undersluices.	7.
40-41			 	Scour downstream of Sarda River at Banbassa (U.P.)	8.
140-41	39			Scour at spurs in river models	9.
41-42	"		•••	Protection of bridge piers against scour and minimum safe level at which to lay stone around piers.	0.
141-42	CIHR		•••	Effect of high level bridge pitch- ing laid round bridge piers on the downstream scour.	1.
41-42	"			Scour round piers—Influence of upstream bed level and mate- rial of bed on maximum scour at vier.	2.
142-43	5) -	`	,	Failure of piers on the bridge under construction over the Ajoy River in the Santal Par-	3.
)42-43-	CHIR			ganas, Binai, during 2000 Maximum scour profile between the guide banks of the Ganges at the Hardinge Bridge.	ŀ.
142-43	37			Experiments with the model of the Rice Canal (Sind) in connection with protection of the Railway Bridge at R. D. 75 000.	i.
943	IWES			Prevention of collapse of Rohri- Multan Road Bridge over Rohri Canal at B. D. 52,000.	3. . ·
.944	_ 11		·	Son at Dehri, Bihar, Protective measures below the new	7.

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	Subject.	Bombay Tech. Paper No. and year.	File No. and/or year.	Annual Ro	eport (Tech).
 18.	Dieep scour downstream of the N. W. Railway Bridge over the N. W. Canal at mile 18/13-4 near Ruk (Sind).			IWES	1944
19.	Experiments carried out to evolve a suitable design of pavement and side expansion to induce accretion.			,,	1944
20.	Effect of shortening Sukkur Barrage on retrogression and accretion in the Indus.		•••• •	73	1944
21.	Protection of retaining walls from scour by means of repell- ing spurs (Baluchistan).	•-•		13	1945
-22.	Dissipation of energy down- stream of Lower Sind Barrage.	•••		27	1945
	11. Dissipation of Energy lielow Falls.				
1.	Discussion by C. C. Inglis, Punjab Engineering Congress Paper No. 147 on "Damaged falls on the Khadir Branch of the Pakpattan Canal".		IRD 116 (1931)		•••
2.	Experiments with the model of super-passage 2-A and 8-A on the Nira Left Bank Canal and Deck Bridge below, to dissipate energy.		IRD 71 (1931-52) IRD 116		
3.	Flume Moter Baffle Fall: Standardization of—	·		ird Iwes	$1931-32 \\ 1935-36 \\ 1944$
4.	Stability of Baffles constructed downstream of flumed falls.	•••	IRD 78 (1935)	$_{ m IRD}^{ m IRD}$	1934-35 1941-42
5.	10 ft. Fall on Sarda Deoha Feeder (U. P.).		IRD 78 (1935)	IRD	1984-85
б.	Experiments carried out with a model of Hardoi Falls (U.P.)		IRD 78 (1935)	IRD	1934-35
7.	Influence of the upstream face of bailles and deflectors.		Do	"	1934-35
. 8.	Experiments with a model of the Sukkur Barrage to show the most suitable design of battles and arrows to control standing wive and to prevent scour downstream.			51	1935-36
9.	Design of a Proportional Standing Wave Flume Meter Fall at R. D. 40, 280, Montgomery- Pakpattan Link (Punjab).	•••	••••	CIHR "	1927-38 1939-40 1940-41 `
10.	Dissipating energy below the masonry waste weir of the Timba Tank at Harsol, (Dist. Ahmedabad).		· ···	"	1937-38

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Waste weir Channel of the Spin			CIHR 1937-38
Karez Reservoir (Baluchistan)			
	,		1040 43
Durani Quilali at Quella.		•••	,, 1940-41
Non-proportional Standing Wave Flume Falls.			,, 1940-41
Dissipation of energy below " W ", type fall.			" 1940-41
Allowating Josiana of haffin		1 	. 1942-43
Alternative designs of bailing platform downstream of the Jamshedpur Siphon Spillway			,,
for dissipating energy.	1999 - 19 1		
	, , , , , , , , , , , , , , , , , , ,		1
12. Fluming of Outlets, Bridges			
and Aqueducts, etc.			
Model experiments with a group of Regulators at Oderolal at	26 (1928)		•••
mile 169 of the Ronn Canal, Sind, by C. G. Hawes and C. C. Inglis.			
Experiments carried out at the	31 (1929)		
a Flumed Regulator, by C. G. Hawes and H. S. Kahai, with			
a note by C. C. Inglis.		TRD 116	
Fluming of Aqueduct 23, Nira Right Bank Canal.		(1939)	
Warned Amednets.		IRD-116	
lote on Linner Education		(1931)	
Notes on Flumed Aqueducts and	42		
Afflux caused by heading up	(1952)		
extent of back water etc." by			
C. C. Inglis.	ro		
Head sluices of Nira Left Bank	(1938)		
stentrance, in barrel; in expan-		ł .	-
sion and at exit, in order to work out a general formula for			
losses of head.			
lead sluices of Nira Lieft Bank	5 <u>4</u> (1935)	•••	
Canal-providing a Den-industri upstream and expansion down-	,	1.11	
stream to increase the discharg-			
to 720 cusecs.			CIHR 1937-38
Godavari Right Bank and Left	د.:		-
Bank Head Regulators-Inc-			
Dharma Storage Reservoir-Out-		· ···	,, 1939-40
lets (Dharwar, Bombay			• • • • •

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	Subject.	Bombay Tech. Paper No. and year.	File No. and /or year.	Annual Report (Tech.).
1 in	3. Modules and Semi-modules coluding Standing Wave Flumes,			
1.	Modules and Semi-modules for irrigation. (Bombay Engine- ering Congress Paper XLVII by C.C. Inglis).		1920	
2.	"An adaptation of the Standing wave principle to hydraulic testing" by C. C. Inglis, Bombay Engineering Congress Paper LXXIV.		1923	
3.	Kent 'O' type module.		IRD 40 (1924)	•••
4.	Experiments with Kirkpatrick Flumes.		IRD 46 (1924-28)	
5,	"Standing Wave Flumes" by C. C. Inglis, Bombay Engineering Congress Paper No. CIV.		1926	
6.	Experiments with Dethridge meter.		IRD 45 (1926-27)	
7.	Experiments with Crump Adju- stable Proportional Module.		IRD 45 (1927)	
8.	"Experiments with the Lindley Type Standing Wave Flume" by C. C. Inglis and D. V. Joglekar.	22 (1928)		
9,	"Standing Wave Pipe Outlets" by C. C. Inglis.	23 (1928)		
10.	"Notes on Standing Wave Flumes and Flume Meter Falls" by C. C. Inglis.	15 (1928)		•••• ·
11.	Proposal for fixing "Hydrautomat Siphon Module" at heads of Distributaries 8 and 24 of Nira Left Bank Canal.		IRD 122 (1930)	·
12.	Experiments with 1 cusec spiral modules.		IRD 78 (1931-32)	
13.	Theory, Design and Construction of the Gibb Modules.	•••	CIHR Publn. 3	CIHR 1933-34 " 1939-40
14.	Experiments to determine the best shape of a skimming baffle in the throat of a standing wave flume, so designed to cause no obstruction at F. S. Q., but maximum afflux at discharges exceeding F. S. Q.		IRD 78 (1933-34) IRD 78 (1934-35)	,, 1 3 40- 1 1
15.	Coefficient of discharge of the Sukkur Barrage Gates.	•••	IRD 77 (1933-34)	
16.	Coefficient of discharge of the Tando-Mastikhan Flume Fall (Rohri Canal, Sind).	•••	IRD 77 (1933-34)	

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	Subject.	Bombay Tech. Paper No. and. year.	File No. and/or year.	Annual R	eport (Tech.).
17.	Modular limit of a standing wave flume with a stable parabolic glacis slope.		IRD 77 (1933-34)		•••
18.	Experiments with Venturi Flumes.	 /	IRD 76 (1935-36)	-	···· .
19.	Design for Standing Wave Flume for the Jodhpur Water Supply Channel.		•••	CIHR	1937-38 .
20.	Design and calibration of a tra- pezoidal standing wave flume for the Poona Drainage outfall sewer at Bhairoba Pumping Station.	58 (19 38)			•••
21.	Calibration of a Standing Wave Flume without side contractions	•••	·	CIHR	1938-39
22.	Approximate formula`for Ven- turi Flumes.	•••	•••	.))	1938-39
23.	Design of a Standing Wave Flume and the Venturi Flume at the head of the Mutha Right Bank Ganal (Poona).		•••	p	1938-89
24.	Stabilization of flow downstream of Standing Wave Flumes.	•••	 ~	ы	1939-40
25.	Effect of piers on coefficients of flumes.	•••	•••	"	1939-40
26.	Design of a truncated Standing Wave Flume for Spin Karez Project-Baluchistan.		、	32	1940-41
27.	"Note on the Theory, Design and Construction of Gibb Modules which, without moving parts, give a constant discharge within the working limits irrespective of variations in upstream and downstream water levels" by C. C. Inglis and D. V. Joglekar.		CIHR Publn, 3 1941		
28.	Modularity of 20 cusecs Standing Wave Flume, Jodhpur.		•••	CIHR	1942-43
29.	Discharge formula for Jeur Aqueduct on Nira Left Bank Canal, (Bombay).		•••	IWES	1943
30.	Standing Wave Flumes and Flume Meter Baffle Falls.	•••		у. ⁻	1944
31.	40 cusecs Standing Wave Flume for the masonry conduit, taking off from the Haleji Reservoir, Karachi Water Supply Scheme.	••• ••	•••	""	1944
				,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,	1944

- 32. Design of a 500 cusees Standing Wave Flume in Jodhpur State.
- 33. Curvature of flow in the throat of a Standing Wave Flume.

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	Subject.	Bombay Tech. Paper No. and year.	File No. and/or year.	. Annual Report (Tech.).
	14. Siphon Spillways.			
1.	Siphon Spillway experiments with flared outlet.		IRD 77 (1933-34)	•••
2.	Unsuitability of a Siphon Spill- way over the Waste Weir of the Lake Arthur Hill at Bhandardara (Bombay).			IWES 1938-39
3.	Design of a Siphon Spillway for the Jamshedpur Water Supply Scheme, Tatanagar, (Bihar).	•••		IWES 1939-40 ,, 1940-41 ,, 1941-42
4.	Siphon Spillway experiments with Ganesh Iyer Volute Siphons; siphons with flared outlets; limiting negative pressure be- fore cavitation occurs.		•••• •••	" 1942-43
5.	Permissibility or otherwise of reducing spillway capacity, Berar (Akola Municipality).	• •••	· · · · · ·	" 1944
6.	Erinpura Irrigation and Hydro- electric Siphon Spillway, Jodh- pur State.	•••		" 1944
7.	Ganesh Iyer's Volute siphons	••••		" 1945
	15. High-Co-efficient Weirs.			
1.	Mhaswad Tank Waste Weir- Experiments with crests of weir to improve the co-efficient of discharge.	••••	IRD 37 (1925)	•••
2,	Experiments with models of High-Co-efficient Weirs.	•••	IRD 76	IRD 1935-36 CIHR 1939-40
3.	Investigations to determine the weir section which while design- ed to pass a maximum flood, will store the maximum possible supply at Lake Arthur Hill (Wilson Dam) at Bhandardara (Bombay Presidency).	58 (1938)	•••	CIHR 1937-38 " 1939-40
4.	To improve the design of the Alandi Weir across the Indrami for the Alandi Water Supply Project (Dist. Poona) in order to reduce afflux.	58 (1938)		••• • • •
5.	Pick-up weir across the Mala- prabha River at Shivyoga Mandir (Bijapur Dist., Bombay Presidency).			CIHR 1939-40
6.	Full-scale test of :High-Coeffi- cient, free flow, weir.		·	" 1940-41
7.	Effect of surface roughness on the co-efficient of a High-Co- efficient weir.		•••	" 1940-41
8.	Drowned weirs			,, 1940-41
	1		1	

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	Subject.	Bombay Tech. Paper No. and year.	File No. and/or year.	Annual Report (Tech.).
9.	The fluid forces acting on a weir.		in the set of the set	CIHR-1940-41
10.	Co-efficient of discharge of anto- matic Gate Spillway for the Tansa Dam (Bombay).	•••• 1 1	••• • # • • • • • • • • • • •	IWES 1945
	16. Flow at Bridges.	· ,		
1.	Submersible bridge (C. P.)	•••	1.44	CIHR 1938-39 1939-40
2.	Sluices for Abdulpur-Nawab- gunj, B. A. Railway.	•••	• • • • • • • • • • • • • • • • • • •	" 1939-40
3. [.]	Suitable shape for cut-and ease- waters of bridge piers.	29 (1929)		", 1940-41
1	7. Design of Rigid Channels and Rigid Structures.	-	2 	ante (alexa) and a constant de la constant de la constant de la constant de la constant de la constant de la constant de la constant
1.	Rugosity of cement-sand plaster.			CIHR 1940-41
2.	Spin Karez Diversion Channel, Quetta.) - 1	•••	,, 1940-41
3.	Design for a dry masonry sill for control of Sur Lake sea-cut with marginal embankments	•••	د میرون میشورد باش مرد دور مارید مرد دور مارید	1 ,, 1940-41
4.	Impact losses due to form drag of slabs and piers under axial flow conditions.	• ••• 1.0.0		·,, 1941-42
5.	Impact losses due to form drag of bridge piers and exposed foundations under axial flow.	• • • • •	• • • • • • • • • • • • •	" 1942-48
18.	Models and their Scale Ratios.			
1 .	Experiments to determine equi- valent size and specific gravity of stones and blocks in a model.			CIHR 1939-40
2.	Wave and Surge velocity in tidal and non-tidal channels.	•••		.,, 1940-41
3	"Hydraulic Models as an aid to Engineering Skill" by C. C. Inglis-Presidential Address, Sogring of Engineering, 28th		1941	
	Indian Science Congress, Benares.	· · · · · · · · · · · · · · · · · · ·	en de la composition de la composition Composition de la composition de la comp	n an
4 .	Methods followed in fixing model scale ratios and overcoming model limitations.	•••	••••••••••••••••••••••••••••••••••••••	IWES 1945
•	19. Dams, Reservoirs and Barrages, etc.	•		• • • • • •
1.	Balancing Tanks and Siphon Spillway at Ozar-Proposals for investigations for additional storage for Pravara.	•••	1928	1

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	Subject.	Bombay Tech. Paper No. and year.	File No. and/or year.	Annual Report (Tech.).
2.	Co-efficients of discharge of Khadakwasla Waste Weir Sluices.	 '	1928	
3.	"Experiments carried out with various designs of piers and sills in connection with Lyod Bar- rage at Sukkur" by C. C. Inglis, C. G. Hawes, J. S. Reid, D. V. Joglekar and H. V. Katakkar.	29 (1929)		
4.	Experiments in connection with the proposed Solani Khadir and Balta Zeel Reservoirs (U. P.).	•••		IWÉS 1944
5.	Desilting and Dredging of tanks in Sangli State (Bombay).	•••		" 19 <u>44</u> -
•	20. Rainfall and Run-off.			
1.	"A critical study of run-off and floods of catchments of the Bombay Presidency with a short note on losses from lakes by evaporation" by C. C. Inglis and A. J. DeSouza.	30 (1930)		••• •
2.	Flood run-off from catchments.	58 (1938)		
3.	Run-off from catchments in India.	•••		CIHR 1940-41
4.	Discussion by C. C. Inglis on Punjab Engineering Congress Paper 245 of 1941 entitled "Run-off" by S. D. Khangar and N. D. Gulhati.		1941	
5.	Discussion by C. C. Inglis on Bombay Engineering Congress Paper No. 195 of (1942) on "Use of graphs for working out information from a catchment knowing the depth and period of flow over a Waste Weir of a lake based on Capt. Garrett's formulas".	 	1942	
6.	Analysis of catchment charac- teristics.			CIHR 1942-43
7.	Empirical Run-off formulas			IWES 1943
8.	Heavy run-off from a small catch- ment u/s of the bridge of the M. & S. M. Railway, which caused the river to overflow the embankment and railway track, causing an accident.	•••		" 1943
9.	Maximum possible discharge from catchment of Thorala Tank, Rajkot State.			" 1943

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•	Subject.	Bombay Tech. Paper No. and year.	File No. and/or year	Annual Report (Tech.)
10.	Rainfall and Run-off-Note stres- sing the importance of shape of catchment in working out the maximum run-off.	•••		IWES 1944
11.	What constant in Dicken's formula of discharge would be most suitable for a small catch- ment area of a river in Santal Parganas, District Bihar.		•••	IWES 1944
12.	Advice regarding probable run-off from catchment in Ceylon.	•••	•	IWES 1944
13.	Information regarding run-off formulas and rate of silting of reservoirs required in connec- tion with a scheme on Zambesi River in Transval, South Africa.			IWES 1944
14.	Sunspot cycles as a medium of predicting probable dates of occurrence of river floods.	··· •/		IWES 1945
	21. Silting of Reservoirs.			
1.	Accumulation of sand and silt in Reservoirs.	· ··· .	•••	CIHR 1940-41
2.	Silting of Ekruk Tank (Bombay).	•••		CIHR 1942-43
	22. Irrigation in Bombay Deccan Canals.	•		
1.	Duty of irrigation water in rela- tion to the capacity of canals in the Deccan. (Bombay Engi- neering Congress Paper by C. C. Inglis).	•**	1918	
2.	Some important factors affecting the design and working of Deccan Canals. (Bombay Engi- neering Congress Paper No. LXVIII, by C. C. Inglis).	•••	1922	• ••• • • • • •
3.	"Irrigation on the block system in the Deccan Canal Tracts" by C. C. Inglis.	16 (1927)		***
4.	"Effluent Farm near Poona (1918-19 t. 1925-26)" by C. C. Inglis.	17 (1927)	·	•••
5.	Note on "Appreciation of land values in the area commanded by Godavari and Pravara Canals" by B. G. Islur.	••••	1927	* •••
6.	"Outturn and profit from sugar- cane crops grown in the Deccan Canal Tracts" by C. C. Inglis and V. K. Gokhale.	21 (1928)	• • • •	
7.	Suitability of soils for different types of crops.		IRD 147 (1928-30)	· · · · ·
8.	Prevention of irrigation in iso- lated area.		IRD '28 (1930)	•••

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	Subject.	Bombay Tech. Paper No. and year.	File No. and/or year.	Annual Report (Tech.).
9.	Block System—Introduction of— on Major Canals. Recommen- dations regarding—		IRD 118, (1928) IRD 149 (1933)	
10.	Irrigation Bunds.		IRD 118 (1928-33)	•••
11.	Perennial irrigation within 1/4th mile limit of villages.		IRD 118 (1928-33)	
12.	Variety of cane suitable for effluent irrigation.		IRD 71, 76 (1931-32) (1935-36)	
13.	Transpiration experiments with canes.	•••	IRD 71, 76 (1931-32) (1935-36)	
14.	Factors affecting utilization of effluent.		IRD 71, 76 (1931-32) (1935-36) -	
28	3. Design of canals and distri- butaries in Bombay (Deccan).			
1.	Weeds in Deccan Canals (A supplement to Bombay Engineering Congress Paper No. LXVIII.)		1922	
2.	Note on "The Design of Distribu- taries and their outlets in the Bombay Deccan" by C. C. Inglis.	19 (1928)		
3.	Note dated November 1927 on "Model Experiments to ascer- tain the effect of partial lining of the Malegaon Cut in miles 38 and 39 of the Nira Left Bank Canal" by C. C. Inglis and D. V. Joglekar.	20 (1928)		
4.	"Moduling operations of Distri- butary No. 8, Nira Left Bank Canal and application of the results to other Distributaries" by C. C. Inglis and D. V. Joglekar.	25 (1928)		
ð.	"Methods suitable for reducing losses from Deccan Canals" by C. C. Inglis, Bombay Engineer- ing Congress Paper.	•••	1928	
6.	"Notes on Remodelling an exist- ing Canal and Distributary to pass an increased discharge and distribution of velocities in channels" by C. C. Inglis.	42 (1932)		•••
2:	4. Soil and Sub-Soil Surveys.			
1.	Losses in percolation and absorp- tion from Deccan Canals and lining of Canals.		IRD 3 (1916-22)	
2.	"Soil and Sub-Soil surveys in the Deccan Canal areas" by C. C. Inglis and V. K. Gokhale.	13 (1926)		

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	Subject,	Bombay Tech. Paper No. and year.	File No. and/or year.	Annual Report (Teoh.).
3.	Advice regarding soil and sub- soil analysis and water logging in Sind.		IRD 114 (1930-81)	
4.	Advice regarding the investiga- tion of the soil and sub-soil conditions occurring in the Barrage area in Sind.		IRD 114 (1931)	•••
5.	Soil protection and reclamation, SindNote for Meeting of Officers on Field Research held	•••	IRD 116 (1931)	••••
6.	Experiments to ascertain condi- tions under which a baffle built on sand will fail and what occurs in a sand column under various pressure conditions and a critical head at which the	•••	IRD 116 (1931-32) IRD 71	
7.	sand begins to move. Percolation experiments in mile 31 of Nira Left Bank Canal.	*** .	IRD 55 (1932)	· •••
25	. Absorption, Regeneration and Water Supply.		Ĩ	• 1
1.	Cawnpore Water Supply.	•••		CIHR 1941-42
2.	Proposed extension of the Water Supply of Bombay.		•••	IWES 1944
3.	Present and Future Water Requirements and Supplies in the Jumna at Delhi.	•••		" 1944
4 .	Danger of water-logging in the tract to the west of the Eastern Nara, Khairpur State, Sind.	••••	•••	» , 1 944
.26	Land Drainage and Reclamation.			
1.	"Improvement of Damaged Lands in the Canal Irrigation Tracts of the Bombay Deccan" by C. C. Inglis.	14 (1927)	•••	•••
2.	Factors affecting removal of alkali salts and reclamation of damaged lands.		IRD 71	IRD 1931-32 1933-34
່ ູ ອ.	Action of salts and effluent water in several types of drainage pipes.	••••	 ,	IRD 1931-32
4.	Reclamation studies on soil re- action, movement of water and alkaline salts in the soils- Changes in the soil on applica-	•••	IRD 78 (1934-35)	••••••••••••••••••••••••••••••••••••••
•	tion of mechanical fertilizations with special reference to Baramati Experimental Salt Area			
5.	Behaviour of soil types under irrigation.	•	IRD 78 (1935-36)	ана
6.	Relative rise of salt solutions in different types of soil.	••••	IRD 78 (1935-36)	*** 1
7.	Reclamation of alkali lands: Field and Laboratory experi- ments in connection with the-		IRD (1935-36)	•••

MISCELLANEOUS

2. Rain Gauge Station Maps of India, Burma and Ceylon.

Maps on a scale of 1''=32 miles, showing raingauge stations in India, were printed in November 1944. The Meteorological Office, Poona, gave this station all the facilities in this connection. There was such a large demand from various Provinces and States for these maps that almost all the copies have been sold out, though a large number was printed. It was, therefore, considered desirable to reprint these maps. These are attached in a separate volume of the Report. An Index Sheet showing the sections 1, 2, 5, 6, 7, 8, 10 and 12 into which the map is divided is also given.

There are about 3,100 stations in India whose rainfall data are published in monthly tables by the Provincial Governments as shown below:—

Assam		135
Bengal		269
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Rainfall is measured once a day, at 8 hrs. local time, at all these stations. Symon's Pattern of rain-gauge, 5" diameter, is in use at most stations, except in Mysore and Travancore, where a Stanley Gauge of 4.7" diameter is used.

At the time of the first printing of these maps, rainfall stations in Ceylon had not been included. There are more than 400 raingauge stations in Ceylon; and, in order to avoid overcrowding, about 80 stations only have been shown in the map of Ceylon on sheet no. 10. All these stations, however, are indicated in a separate large scale map of Ceylon (Scale 1"=16 miles), which appears as sheet 10 A in the present series. It may be added that the names of the rainguage stations have been taken from the report on Colombo Observatory (1944) issued by the Superintendent, Observatory Ceylon, Survey Department. This map has been supplied by the Meteorolgical Office, Poona.

In the first printing, the stations in sheet 10 were found to be crowded; hence while reprinting this sheet, the names of various stations, particularly in Travancore, have been re-arranged to facilitate easy reference; and where the names were too crowded, only serial numbers are shown in the map and the corresponding names of stations are given on one side.

The Station is very grateful to the Meteorological Office, Poona, for the facilities given in this connection.

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