Nature and Extent of Damage

The extensive damage to the fall can be visualized by photographic exhibits Plate II & III. The concrete glacis had severed off the crest at its junction in all the bays 1 to 6, wide cracks having developed and cavities as deep as 2·5′ were measured below the glacis. These cracks continued in the vanes being almost vertical in the first three vanes and increasingly oblique in the next three. The cracks continuously widened with the passage of time and extended beyond bay No. 6. The cistern floor downstream of baffle wall settled by about 4·5 ft. at the deepest portion. This settlement commenced from the left wing wall and continued beyond the centre line of the structure. Later the cracks also appeared at the junction of upstream glacis and the crest. The left wing wall ultimately collapsed from where it has cracked due to earth slipping from behind.

Condition Before Damage

The practice has been to close Rohri Canal for its annual repairs in the third week of December. This year a departure was made from the accepted practice and decisions at ministerial level postponed the closure to 15th March, 1957. The canal was closed on this date and was reopened on the evening of 26-3-1957. On the morning of 27-3-57 when hardly a discharge of 2000 cs. was flowing over the structure, the fall had virtually disintegrated. During this closure of ten days, all exposed parts of the fall were inspected by the field staff; Two bays were closed at a time by placing gunny bags at the upstream end. No visible signs of any damage were reported. The downstream floor was not dewatered. The record does not reveal any instance when downstream was dewatered except perhaps once in 1943. The fall is located in an area of high water table. Rohri Canal when flowing full contributes extensively to the water table and this has been the basis of a long drawn out dispute between the Ex-Khair Pur State and the former Sind Government. During closure the canal acts as a drain attracting seepage flow along its entire length. So even in closure the water level was at R. L. 173·04 which is one ft. above the crest level and is capable of discharging about 300 to 400 cs. The water table during closure at up-stream of fall was 175·18 and down-stream it was 170·65. The water surface level in the canal downstream of fall was measured to be 161·3. The downstream cistern level is 159·8 and thus a depth of water at least equal to 1·5 ft. is all the time standing on the down-stream floor and this hides the floor from visual inspection.

The Design of Tando Masti Khan Fall

Before we proceed to analyse the factors that led to the failure of fall, it is imperative that an appreciative study be made of its design characteristics. This can be studied broadly under two main heads.

(a) Hydraulic Design

(b) Structural Design

Behaviour of the Fall

From the surface flow conditions the fall behaved perfectly well during the past 26 years. No problem of bed scour or side erosion was created and this indicated that surplus energy was not left over after the formation of standing wave, redistribution of velocity was satisfactory and roller formation if any did not extend beyond pucca pavement. In this context a reference may be made to the regime conditions of Rohri Canal. The canal was designed with a bed slope of 1 in 11,700. In its actual working the canal adopted a flatter slope and the field data supplied by Barrage Division Sukkur and analysed in C.D.O. indicated that certain reaches have adopted as flat a slope as 1 in 20,000. This caused a general retrogression in the canal. The canal bed swung round the control structure up-stream. This process laid open the shallow foundations of certain masonary structures, quite a number of which collapsed. Due to the general retrogression, the prevailing full supply level at the down-stream of fall was 172·4. Acomparison of the designed data with that prevailing under running conditions is given below.

	Designed Data	**Prevailing Data.**
Upstream Full Supply Level	182·3	183·6
Down-stream F.S.L.	174·2	172·4
Drop	8·1	11·2

The rise in up-stream full supply level was due to afflux caused by excessive fluming at the fall. The effect of lowering downstream F.S.L. and also that of increased drop due to afflux was to shift the position of hydraulic jump down the toe of glacis. Calculations attached as appendix II make it clear that under the prevailing conditions formation of hydraulic jump on the glacis in case of a simple glacis fall was not possible. The baffle wall was a blessing in disguise for the structure and violent churning action helped to dissipate energy which otherwise might have been unmanageable and threatened the safety of structure much earlier than its actual failure. However, the forces of disintegration did not act through this media and adopted other agents to achieve their end.

(b) The Structural Design of Fall

The behaviour of two dimensional sub soil flow under a structure on permeable foundation has been a subject of extensive research work under the guidance of several eminent engineers and scientists. The present day practice is based on theories propounded in the middle of thirties. For the design of Tando Masti Khan Fall no attempt was made to study sub-soil flow with its attendant hydraulic gradient and uplift pressures. The structure was designed on the basis of Bligh's creep theory which held the field upto that time.

(1) The downstream cut off is inadequate and hence sufficient factor of safety is not available for exist gradient.

(2) There is no intermediate cut off to act as a second line of defence. Once the floor downstream the baffle wall got damaged, exist gradient could not be controlled and undermining started by sand blowing.

(3) The thickness of floor down-stream of baffle wall is not adequate to withstand the uplift pressures caused by the actual working conditions. It has been indicated earlier that the prevailing conditions subjected the structure to a differential head of 11·2 ft. instead of the designed head of 8·1. Plate IV gives a comparison of the required thickness of floor to those existing at site. Actually the critical conditions occurred during closure. Data observed during the last closure of the ill fated fall gives upstream water level as 173·04 and downstream water level as 161·3, the head across being 11·74. Hence the worst condition occurs during closure when there is only 2′ of water in the cistern downstream of baffle wall against 12 ft. under running conditions.

(4) As discussed earlier the depth of water in the cistern downstream of baffle is just sufficient to form hydraulic jump according to design data and insufficient according to prevailing data.

Causes of Failure

The soundings and probling of the cistern floor taken after the failure showed that a portion of the floor has settled downstream of the baffle wall. This observation interpreted in the light of calculation of Appendex I indicates that failure has taken place in the portion where the cistern floor was weakest from the point of view of uplift pressure. The probable reason for the failure of the work appears to be the development of cracks in the cistern floor downstream of baffle wall. The cracks remained unnoticed for a long time and continued blowing out sand from underneath the structure, due to excessive exit gradient with the result that cavities were formed under the work. The cavities progressively worked their way up-stream towards the junction of crest and glacis slope and when the vanes could no longer be supported, the glacis collapsed resulting in cracks which developed along lines of weakness such as the junction of crest and glacis slopes. The question can be asked why the structure has stood for the past 26 years if it was unsafe against uplift pressure? At the time of construction of the Fall, the subsoil water levels in the area were very low. As the subsoil water level rose due to the running of Rohri Canal, the structure was subjected to a cross head during closures which was even more than the head during flow. Another very important point was that the subsoil water levels on the left were higher than those on the right which also period of one year. The failure was not sudden in the meaning that the factors which caused failure were very rapid. The fact remains that the process of undermining was a continuous one and the failure was just the culmination of this process. This suggests that there is some basic omission in the procedure of inspection due to which the early detection of damages is not ascertained. When the fall was inspected 2′ of water was standing on the D/S floor. No arrangements were made to dewater the floor or to take soundings and probings of the floor, after making sure that the head across the structure was not more than that for which it was designed due to the sub-soil water levels upstream and downstream of the work. Had such observations been made the cavities and the settlement of floor which preceded the failure must have been disclosed and some prompt remedial measures taken.

Such observations had not been taken as a matter of routine and perhaps have been taken only once in 26 years when in 1943 certain springs were noted in the cistern and plugged with cement. The dewatering of floor for visual inspection and a record of soundings and probings is very essential for the upkeep of structures and must be enforced as a codal rule. True, the soundings and probings cannot detect the cracks in floor, but the next stage of the development of damage can be found out and arrested.

In this context it must be mentioned that for the structures on Canals flowing through the heavily waterlogged tracts, as the Rohri Canal, a closure period of ten days is insufficient to put in a bund across the structure and to dewater it. The time factor is important as the Canal in closure acts as a drain and it takes quite some time to subside the seepage inflow, and hence the closure periods should be longer.

(b) The important structures built prior to the development of modern concepts of design of hydraulic structures on sand foundations should be checked up and necessary corrective measures be taken in time in a systematic way. It evinces a lack of insight not to be able to diagnoze an impending danger to a structure. Any complacency will be suicidal for the reasons that since a structure has stood well for so many years in the past it can serve for an indefinite period. The maintenance engineers must acquaint themselves with the design aspects of structures under their charge and formulate proposals to bring the designs in line with the modern concepts. The very first aspect, the provision of which should invariably be checked in the old hydraulic works is the downstream cut off. Though the importance of thickening the downstream floor to withstand the uplift pressures cannot be ignored, yet it must be remembered that the failure starts with the undermining of structures, due to excessive exit gradient and hence a water-tight cut off at the downstream end is the minimum we should try to achieve in old structures the safety of which is otherwise doubtful.

history of these two syphons clearly demonstrates how works originally safe can become unsafe with a rise in the spring level". The development of modern concept of design started with data from Jaurian & Dugri syphons which lie in an area of high sub-soil water level, but later the pressure pipe observations from Panjnad H/W provided the necessary data and hence the problem of cross flow from the flanks was ignored due to the long width of the work to arrive at some workable system of designing the works. The issue of three dimensional flow under the weirs got relegated to background, as naturally the simpler laws of two dimensional flow were to be investigated first. Later the issue was pushed to oblivion & only passing references were made by certain authors to the probable effects of three dimensional flow on the safety of structures. Mr. Khosla wrote in CBI Publication 12 :—

"In weirs the flow will be mainly two dimensional as the width of a river is considerable so that flow at any cross section of a weir is not appreciably influenced by any cross flow from the sides. At the flanks of a weir and at syphons or other narrow structures the influence from the sides will be considerable & the flow will approximate to three dimensional".

Mr. Haigh in his paper presented to Enginering Congress in 1935 dealt with the subject more explicitly & concluded that the converging flow increases the pressure gradient at the downstream end & consequently necessitates a deeper cut off. It may be noted that no complete solution of three dimensional flow & its application to design of works have been given so far. As a general guide to the safety of structures both Mr. Haigh & Mr. Khosla recommended that besides the sheet piles on U/S & D/S sides the foundations of abutments & flanks should go down to the level of the bottom of these pile lines. With such an arrangement of levels for the flank foundation it may be possible to obtain the conditions that the uplift pressures under the weir due to flank flow may not be more than those due to direct flow under the weir. This is just a precaution. The exact theoretical approach to the design of the floor for three dimensional flow is yet to be investigated. With the spread of water-logging in the irrigated areas progressively increasing number of structures are being subjected to these unknown forces for which we have not made adequate provision in the past. This points to the extreme urgency of starting immediate investigation in this direction. Preliminary thoughts on the subject suggest a solution hinted by Mr. Haigh in his criticism of Mr. Khosla's paper in 1930. He proved that convergent flow towards a point follows a hyperbolic law and not the logarithmic law, which was only applicable to flow converging to a plane. The convergent flow towards a point represents three dimensional flow and that converging to a plane represents two dimensional flow. The theory of two-dimensional subsoil flow as applied to the calculation of pressure gradients in different weir profiles suggests that it follows a linear law along the horizental floor in the region bounded by the sheet piles. In all probability in a three dimensional flow the pressure gradient along the horizental flow will also follow some where in the path of stream lines will result in the steepening of gradient locally, may be to bursting point."

The effect and exact magnitude of these factors can only be evaluated by further rasearch work but it must be admitted that variation in permeability in section and also in plan does alter the distribution of pressure from the recognized pattern of flow in uniform soil.

Conclusions :

The failure of this fall has focussed our attention on the need of greater alertness on the part of maintenance engineers to inspect the works thoroughly. These engineers should be competent enough to predict the behaviour of the structures under their charge with special reference to structures built on older concepts of design.

The designers of irrigation works should realize the absolute necessity of downstream cut offs and for very important structures should not even grudge the provision of secondary defence lines in the shape of intermediate cut offs with suitable cut offs at flanks and wings. The thickness of the floor in the cistern should be adequate to withstand the uplift pressure calcuated on the most modern concept of sub soil stream line flow. With research workers the problem of flow under structures in high water table areas should obtain priority. This is not only for academic interest but also for facing the menace to hydraulic structures in water logged areas.

ACKNOWLDGEMENT.

My thanks are due to Mr. M. Khan, Director, Designs & Research, Irrigation Department, West Pakistan, Lahore, but for whose inspiring guidance I could not have ventured with the paper. I am also grateful to Dr. Mushtaq Ahmad, Hydraulic Officer, Irrigation Research Institute for lucid exposition of problems under discussion and from whose notes I stand benefited in writing this paper.

APPENDIX II

Optimum Cistern Depth for the Formation of Standing Wave

The test is made for the actual water levels and not the designed levels.

$Q=10616$
B at throat$=100'$
U.S.L.$=183.6$

D.S.L.$=172.4$ (actual)

$HL=11.2'$

$$q=\frac{10616}{100}=106{\cdot}16$$

For $HL=11.2'$

And $q=106.16$

From HL, q curve.

$Ef^2=15{\cdot}6$

OR $Ef^2=D^2+h^2$

$15.6=D^2+V^2/2g$

$$\text{Now } V=\frac{106{\cdot}16}{13{\cdot}21}(D=183.6-170.39=13.21)$$

$=8{\cdot}03$

$V^2=64{\cdot}48$

$V^2/2g=1{\cdot}007$

Now $D^2=15.6-1{\cdot}007=14.593$

Cistern level downstream of the baffle is at R.L. 159.82 actual depth in the cistern$=172.4$-$1598=12.4'$

The actual depth is less than the desired depth by about $2.2'$ i.e., the retrogression. But for retrogression the depth is correct. The fall has been behaving well from the view point of sarface hadraulic because of the action and secondary drop at the baffle wall.

APPENDIX III

Exit Gradient Assuming Different Lengths of the Work to be Impervious.

Case I. (Length AD=260)

Let H=11 feet.

b=260′

d=5.5′

Now $L = b/d = 260/5.5 = 47{\cdot}2$

and $H/d = 11/5.5 = 2$

Therefore GE for $H/d = 2$ and $L = 47{\cdot}2$ is

GE=0·182 from the curve.

Case II. (Length AC=166′)

Let H=11′

b=166′

d=6.0

L=b/d

$$\text{Now } L = b/d = \frac{166}{6} = 27.6$$

and $H/d = 11/6 = 1.83$

Therefore GE for $H/d = 1.83$ and $L = 27{\cdot}6$

GE=0.155

Case III. (Cength BD=222′)

Let H=11′

b=222′

d=5.5′

$$\text{Now } L = b/d = \frac{222'}{5.5} = 40 \text{ approx.}$$

and $H/d = 11/5.5 = 2'$

Therefore GE for $H/d = 2$ and

$L = 40$ is

GE=0.140 from the curve.

Case IV. (Length BC=121′)

Let H=11′

b=121′

d=6′

$$\text{Now } L = b/d = \frac{121}{6} = 20 \text{ Approx.}$$

and $H/d = 11/6 = 1.83$

Therefore GE for $H/d = 1.83$ and $L = 20$ approx.

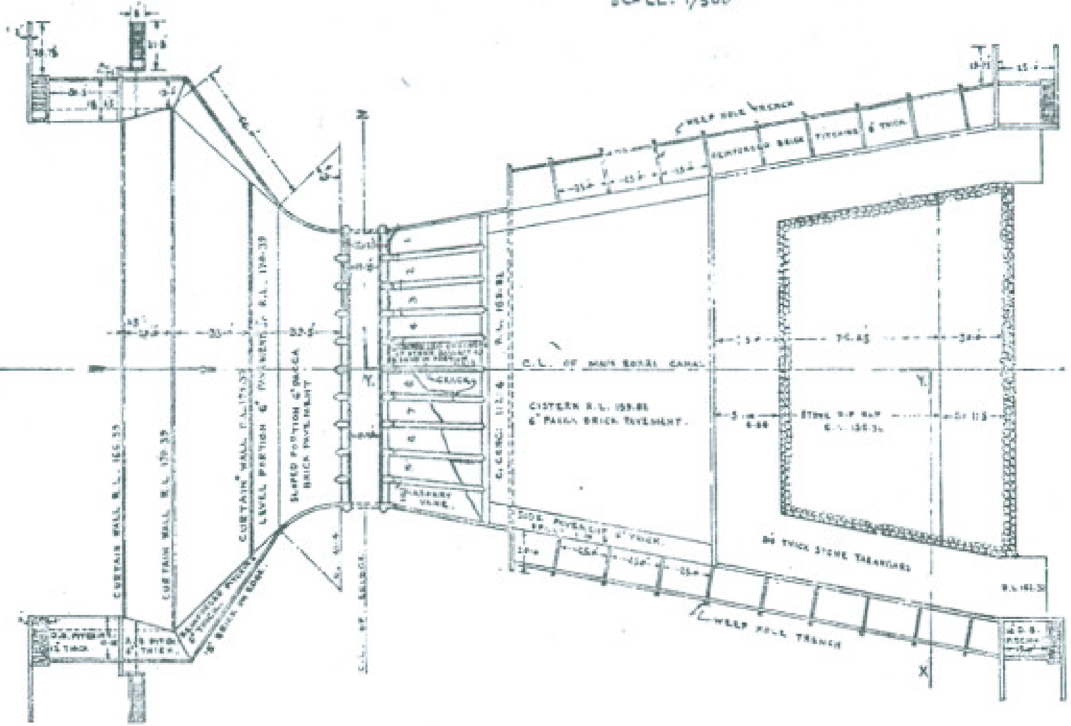
is =0.182 from the curve.

The Lengths AD. AC. BD and BC, of the work assumed impervious are shown in fig. 5, PAPER NO 331

PLATE 1

GENERAL PLAN OF FALL AT TANDOMASTI KHAN AT MILE 24 OF ROHRI CANAL

SCALE: 1/500

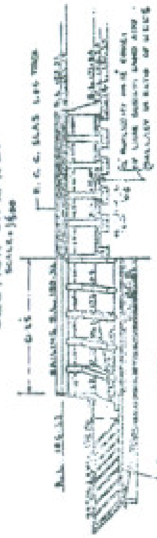


CROSS SECTION OF MAIN CANAL U/S

SCALE: 1/500

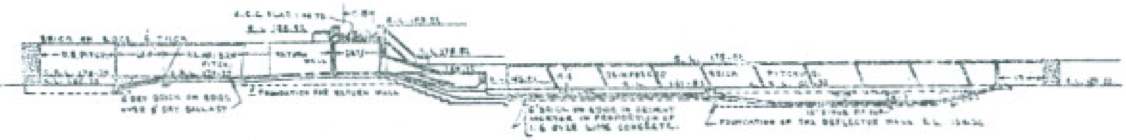
SECTION ON X.Y.Z.

SCALE: 1/500



LONG: SECTION ON BAY No. 6

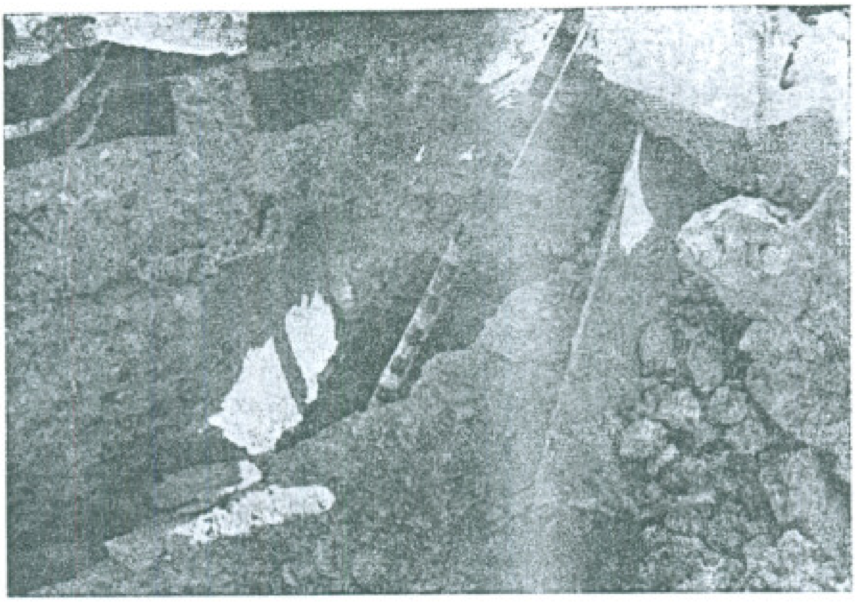
SCALE: 1/500



No.	DESCRIPTION	U/S.	D/S.
1.	DISCHARGE IN CUSECS	10555	10555
2.	FULL SUPPLY DEPTH IN FEET	11.0	12.0
3.	BED WIDTH IN FEET	240.0	240.0
4.	BED R.L.	170.39	161.32
5.	FULL SUPPLY LEVEL	182.39	174.32
6.	CREST LEVEL OF THROAT	172.24	
7.	MEAN VELOCITY THROUGH CANAL IN FEET PER SECD.	3.50	3.50
8.	MEAN VELOCITY THROUGH THROAT IN FEET PER SECD.	10.33	
9.	WATER WAY AS PER CANAL SECTION IN SQ. FEET	30.36	30.24
10.	WATER WAY PROVIDED THROUGH THROAT IN SQ. FEET	10.33	
11.	ACTUAL COST IN RS.	128692/-	
12.	RATE PER SQ FEET OF WATER WAY PROVIDED IN RS.	124.34	
13.	RATE PER CUSEC IN RS.	212.15	

PAPER No. 331

Tando Masti Khan Rohri Canal.

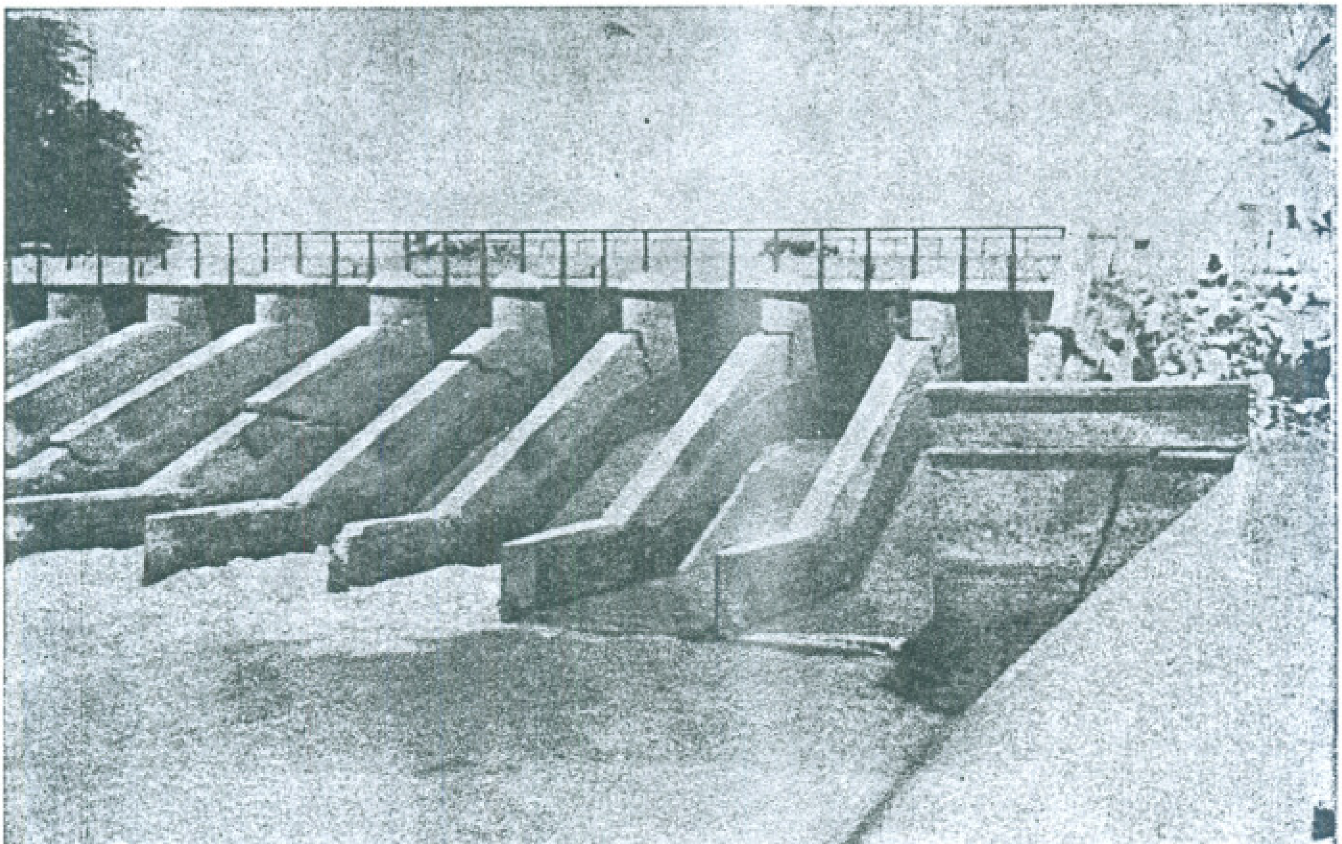


Showing cavities under the glacis slope in Bay 6. The depth of cavity is 2·6′.

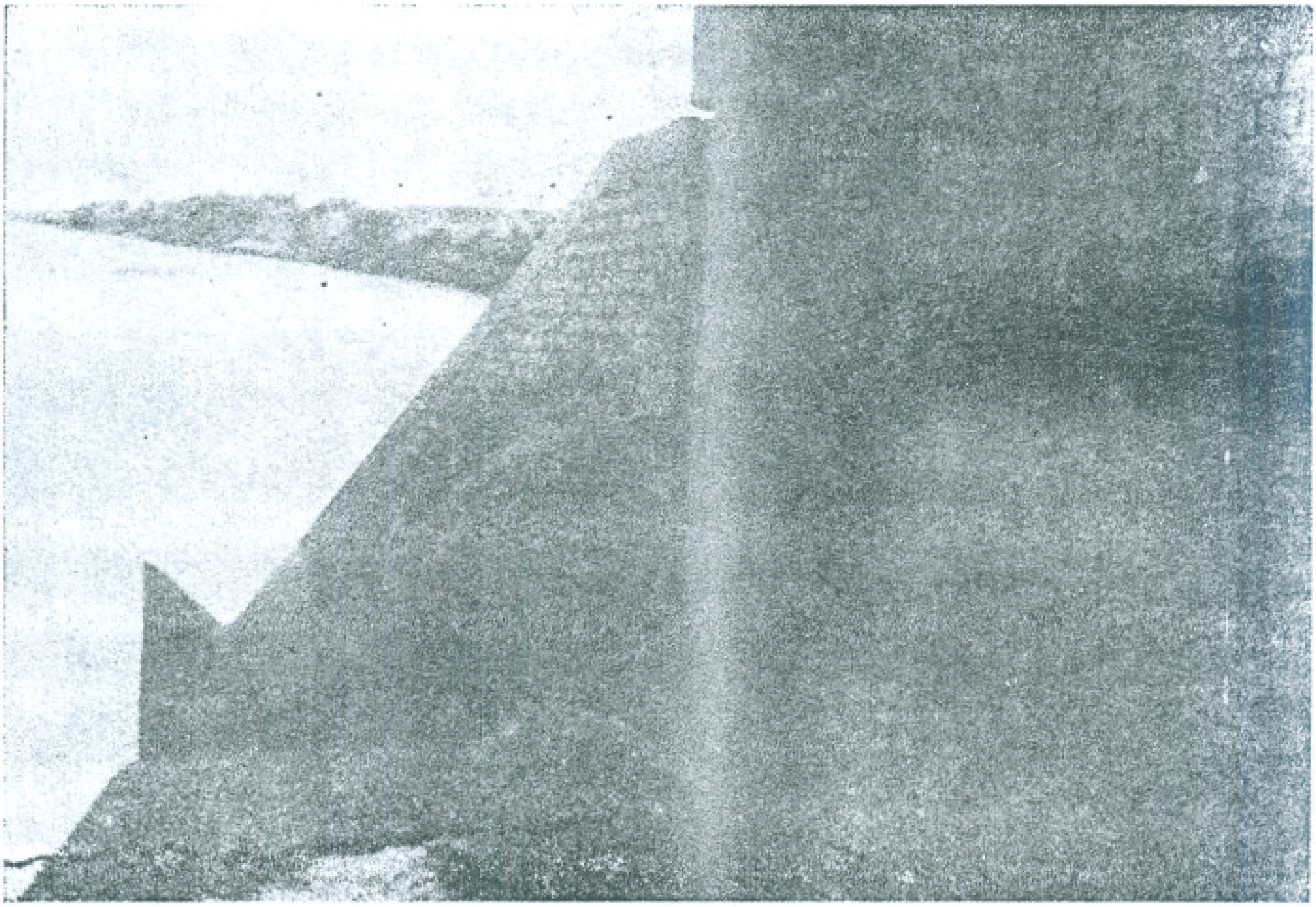
Cavity under the glacis slope.

PAPER No. 331

Tando Masti Khan Fall Rohri Canal.



Showing cracks in piers and flank walls. These cracks are continuation of horizontal cracks on the crest.



Showing cracks in piers and joint of crest and glacis Bay No.

PAPER NO 331

TANDO MASTI FALL

ϕ- VALUES FOR TANDO MASTI FALL

SCALE :- 1/300