

Report of the  
commission of inquiry for enquiring  
into the failure of the Panohet and  
Khadakwasla dams.



Government of Maharashtra

**REPORT**  
OF THE  
**COMMISSION OF INQUIRY FOR ENQUIRING**  
**INTO THE FAILURE OF THE PANSHET**  
**AND KHADAKWASLA DAMS.**

**PART I**

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**PART I**

# PART I

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## APPENDIX I

## STABILITY ANALYSIS OF PANSHET DAM SECTION

1.00. The 'as executed' section of Panshet Dam in temporary waste weir portion has been analysed for safety under full reservoir and rapid drawdown conditions for upstream slope and steady seepage condition for the downstream slope. Since both the slopes of the dam section rest on plastic clayey material placed at 3 per cent. wet of optimum, failure is likely to occur by sliding along this layer constituting a plane of weakness. Stability computations have, therefore, been made by both the methods viz. assuming rotational failure as per conventional analysis and by considering equilibrium of the slope against sliding along the base under horizontal pressure computed by Coulomb's theory.

1.01. The following values of densities and shear parameters have been used for stability analysis of the dam section. These values are based on undrained direct shear tests on samples moulded at various moisture contents ranging from 20 to 30 per cent. and changing the compactive effort to obtain variation in dry density from 107 to 86 lbs./cft. and adopting minimum values therefrom. These values are on the conservative side since they have been worked out in terms of total stresses and additional allowance has been made for pore pressures in the stability computations.

Material	Dry density	Cohesion	tan
1. Casing material at OMC (14 per cent.)	114 lbs./cft.	150 lbs./sft.	0.5
2. Hearting material at OMC (18-20 per cent.)	98 lbs./cft.	500 lbs./sft.	0.35
3. Hearting material at 3 per cent. wet of OMC	94 lbs./cft.	300 lbs./sft.	0.30

1.02. The minimum factors of safety under different conditions work out to as below by the two methods. Since the soaking of embankment materials on filling up of reservoir will lead to reduction in their cohesion values, the factors of safety have also been worked out assuming reduced values of cohesion equal to zero for casing material and 300 lbs./sft. for hearting materials :—

Item	Minimum factor of safety	
	Rotational failure method.	Sliding wedge method.
(a) Upstream slope—		
(i) Reservoir full conditions	1.55	1.55
(ii) Rapid drawdown condition assuming 50 per cent. drainage for casing material.	1.21	1.2
(b) Downstream slope in steady seepage condition	1.49	1.53

1.03. These values of factor of safety are based on the laboratory values of Proctor's maximum dry density for different materials. The average dry density values of materials as actually placed in temporary waste weir portion are slightly lower, being 100 lbs./cft. for casing material and 94 lbs./cft. for hearting material as worked out from compaction data for the reach. The actual factor of safety of the dam under various conditions will be slightly lower. However, even allowing for some reduction on the account, the values of safety factor are considered adequate in view of minimum recommended value of 1.3-1.5 (*vide* Soil Mechanics in Engineering Practice by Terzaghi and Peck, page 388).

## APPENDIX II

## ASSESSMENT OF LOADING OVER CONDUIT ARCH IN TEMPORARY WASTE WEIR SECTION AND COMPUTATION OF STRESSES IN THE ARCH RIBS

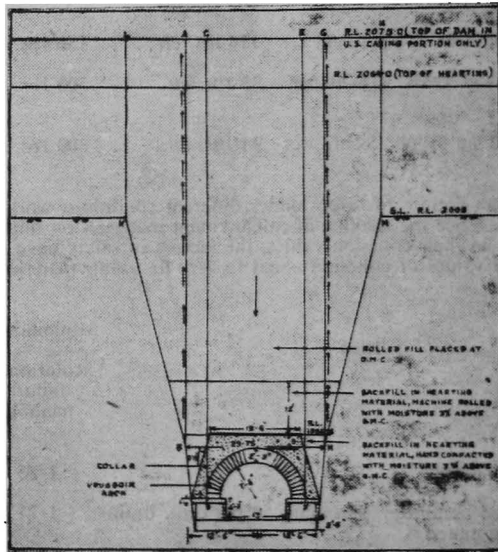
1. The assessment of the load of overlying embankment that may have developed on the conduit arch has been one of the most controversial points and the following very divergent views have been advanced :—

(i) Complete ditch condition \* the Central Design Office computations based on this assumption.

(ii) Complete projection condition.

(iii) Complete project condition with additional drag exerted by the casing material on the hearing zone due to sliding or yielding tendency of the former.

2. The following figure shows a vertical cross section through the conduit at the centre of hearing portion as constructed i.e. at the point from which the downstream portion of the arch has been washed away :—



It may be clarified here that for the purpose of ascertaining whether the failure of Panshet Dam was caused by the collapse or failure of the conduit arch, it is necessary to determine what can reasonably be expected to have been the load on the conduit arch up to the date of the failure of the dam and not the load that should be adopted for the design of conduit under similar conditions in accordance with Conservative design practice.

3. The fill above the conduit arch may be considered to consist of a central prism CDFE vertically above the arch flanked by two truncated side prisms, ABDC and EFHG. The latter adjoin the prisms JKBA and IMHG respectively each vertically above the steep sloping sides of the conduit trench. The load coming on the arch will depend on the yield of the central prism relative to the adjoining side prisms. If the central prism settles or yields more, the side prisms will tend to oppose its yield leading to mobilisation of shear stresses in the upward direction along the contact planes and this would reduce the load on the arch. On the contrary, if the side prisms settle more, they will tend to drag the centre prism, thus tending to increase the load on the conduit.

\*See Spangler M. G., Underground Conduits—An appraisal of Modern Research, Proceedings A. S. C. E., June 1947, page 855.



4. The relative settlement of the prisms may occur due to yielding of conduit and foundation material, and relative consolidation of the fill in the prisms. In the present case, the first two factors are immaterial due to rigid masonry arch resting on rocky foundation. The only factor controlling relative settlement of the fill is its internal consolidation. The lower portion of the fill up to a height of 3 ft. above the crown of the conduit arch consists of hand tamped backfill at 3 per cent. above optimum moisture content and at somewhat lower dry density. Initially, therefore, the two prisms ABDC and EFHG adjoining the central prism will tend to consolidate most and will be supported partly by the central prism and partly by the two end prisms resting on the side slopes of the trench. The load on the conduit arch in the condition may, therefore, be taken as the weight of the central prism plus  $\frac{1}{3}$  the weight of the two thin prisms immediately adjoining it. Also in view of the conditions mentioned above, there will be no horizontal thrust on the arch. After the differential consolidation and internal adjustment in the fill inside the trench have been completed, the load would correspond practically to Spangler's ditch condition with some horizontal thrust which may be taken as equal to  $\frac{1}{3}$  of the vertical load in accordance with U.S.B.R. Practice. This condition of loading known would have taken a long time to materialise and is unlikely to have occurred by the date of the failure of the dam on July 12, 1961. In any case, there is no possibility of existence of projection condition since development of such a condition requires greater settlement of the end prisms resting on the steeply rising side slopes than that of the central prism, which is obviously not possible in this case.

5. In regard to the suggestion that the loading on the central hearing portion of the conduit might be increased very considerably due to relative drag between the casing and the hearing materials, such a drag will come into existence only in the event of some yield or movement along the interface. Such yield can come into play in the event of faster consolidation of casing material or a sliding tendency. Since the casing material is likely to settle more on soaking as a result of sudden filling, the existence of such a drag is possible and may lead to increase in the load on the conduit roof under the hearing portion depending on the rate of settlement of casing material as the reservoir fills up. However, since the seat of this drag is along the interface between hearing and casing material, its effect will mostly be confined and there will be no appreciable increase in the load at the vertical section at the centre line of the hearing zone or sections further downstream which has been washed away.

6. Thus, a realistic picture of the stresses in the conduit arch at any section upto the time of failure can be obtained by considering the total weight of the fill and of reservoir water directly above the conduit arch (central prism) section plus half the weight of the two side prisms immediately adjoining the central prism together with weight of reservoir water, if any, thereon. This is also in accordance with the U. S. B. R. practice of designing conduits in earth dams for load equivalent to total overburden load *vide* U. S. B. R. Design Supplement No. 2, Chapter 13 Outlet works, para. 13·21, C-1 extracted as below :-

"The external load on the top of the conduit is a uniformly distributed vertical load equal to the weight of fill and reservoir water directly above the spring line of the conduit. The external load on the sides of the conduit is a uniformly distributed horizontal load equal to the one-third the vertical load, or no horizontal load. The vertical reaction on the bottom of the conduit is equal to the vertical load, either uniformly distributed, or triangularly distributed."

7. The U. S. B. R. practice assumed a value of horizontal pressure equal to  $\frac{1}{3}$  of vertical pressure or no horizontal pressure depending on the circumstances of the case. In case of Panshet trench, no horizontal pressure could be expected on the conduit arch as, in the circumstances explained already the weight of the two thin prisms immediately adjoining the central prism is expected to be shared by the central prism and the end prisms on the sloping sides of this conduit. For purposes of calculating the most probable load on the conduit arch upto the time of failure, horizontal pressure on the arch may be taken as 'nil'. Thus, the vertical load on the conduit arch will correspond to a 25 ft. wide prism at the horizontal plane passing through the crown and in the computations for determination of rib stresses for this load no horizontal stresses will be taken (*vide* Table 1). Stress computations in the arch rib have also been made for comparison for a total overburden load of wedge equal to conduit width (20·5 ft) and for ditch condition with lateral pressure equal to  $\frac{1}{3}$ rd of vertical pressure (*vide* Tables 2 and 3).

8. It has been suggested that the horizontal thrust equal to pore pressure will always be exerted in the sides. This is correct in respect of hydrostatic pressures in the zone of saturation but not so for pore pressures developing as a result of compression of pore fluid as in the case of construction pore pressures. Once the upper soil mass arches over the side prisms immediately adjoining the central prisms due to much greater depth of relatively compressible soil under the former as explained earlier, and the load on the lower soil mass reduced, the pore fluid will expand and any pore pressures existing prior to arching, will

be relieved. When working out stresses in the conduit arch in the sections under the deepest portion of impervious hearing material for which section the downstream portion of the arch has been washed away, it appears but reasonable to assume that the saturation line could not have developed within the short period of two weeks of reservoir filling after which failure of the dam occurred. On the upstream side, in this portion under the upstream casing zone where soil mass could have become saturated, the horizontal force could have developed and would improve thereby the stability of the arch conduit in their sections which although under vertical load almost equal to the portion under the centre of the hearing zone, have not been washed away and do not show signs of distress due to overloading.

9. The maximum permissible stresses in the stone masonry of the conduit arch can be taken to be of the order of 500 psi compression and 100 psi tension. The maximum stresses that must have occurred in the portion of the conduit arch under the hearing which has been washed away, are much higher and, therefore, it is practically certain that this portion of the arch must have been overstressed. However, the arch could not have collapsed on this account as even a slight deformation of the arch would cause corresponding yield of the central prism of the earth embankment over it, leading to greater mobilisation of upward acting frictional resistance. This would result in large reduction in the vertical load on the conduit arch and would bring the loading down to that corresponding to ditch condition. Horizontal thrust on this conduit arch caused by the loading of the side prisms would also develop, thus bringing down the stresses on the conduit arch to safe limits as shown in column 4 of the Table of stresses. In this connection, an extract from *Trans. A. S. C. E.* 1960, Vol. page 293 Garrison Dam Test Tunnel, Symposium is reproduced :—

“If the concrete lining is so stiff that its high bending resistance initially carries most of the vertical load with resulting higher fibre stress, cracking of the lining will then make the ring more flexible so that it reaches equilibrium in carrying further increments of the vertical load by building up the passing load, V.H. As a limiting case in a considerably cracked condition, the lining might approach a segmented arch as a series of hinged blocks able to take high thrust but low moment and possessing considerable flexibility. This would be somewhat like the English practice, of using precast concrete segments in lieu of cast iron segments, with a thin sheet of compressible material in each circumferential joint to introduce some flexibility. Such a segmented ring is entirely safe and is actually able to carry a considerably higher load without overstress than the stiffer ring could before cracking, provided the deformations are small and the ring maintains its integrity without buckling.”

10. It is seen from the above, that collapse of the conduit arch could not have occurred under the load of the embankment above it, as the arch voussoirs would readjust themselves resulting in reduced vertical loading and increased horizontal passive load. It is, however, practically certain that joints would open out at points of high tension and mortar would be crushed at points of high compressive stress.

11. The stresses in arch rib of a conduit under various conditions are tabulated below :—

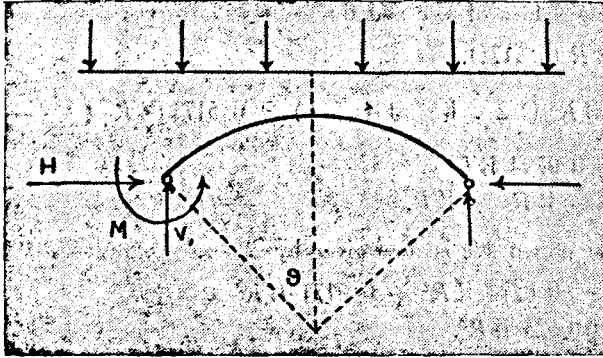
*Table showing Stresses in Conduit Arch Ribs*

Location		Full overburden load in width of wedge = 25 ft. Ph = 0	Full overburden	Ditch condition ph = pv/3
			load in width of wedge = 20.5 ft. ph = pv/3	
Crown	.. .. Top	.. +827 psi	+635 psi	+560 psi
	.. .. Bottom	.. -330 psi	-113 psi	-99 psi
Springing	.. .. Top	.. +1220 psi	+715 psi	+620 psi
	.. .. Bottom	.. -256 psi	+85 psi	+74.5 psi

## STRESSES IN CONDUIT ARCH

The stresses in conduit arch masonry have been worked out on the assumption that it will behave as a fixed arch. Values of moments and horizontal thrust for a fixed arch including rib shortening effect are given by :-

(Vide R. J. Roark's Book, pp. 163, case 30).



Horizontal thrust  $H$  is given by

$$H = WR \left[ \frac{1/4 (S^2 C/O - S) + 1/6 S^3 + \dots}{\frac{(O - S)^2}{2} - 3/2 O + 2S - \dots} \right]$$

$$\alpha \left( \frac{O}{4} - \frac{1}{2} - OC^2 + \frac{1}{4} SC \right)$$

$$\frac{1}{2} SC - \alpha \left( \frac{O}{2} + \frac{SC}{2} \right)$$

The moment  $M_1$  is given by

$$M_1 = WR^2 \left( 1/2 S^2 - 1/4 + 1/4 \frac{SC}{O} \right) - HR \left( \frac{S}{O} - C \right)$$

Vertical thrust  $V_1$  is given by

$$V_1 = WR_s$$

$$\alpha = \frac{I}{AR^2} \text{ where } A = \text{Cross sectional area}$$

$B = \text{Radius of the arch}$

For the specific case of Panset arch

$$R = 7.5 + \frac{2.75}{2} = 8.875 \text{ ft.}$$

$$\alpha = \frac{b d^3}{12} \cdot \frac{1}{b d} \cdot \frac{1}{R^2}$$

$$= \frac{(2.75)^2}{12 \cdot (8.875)^2} = 0.008002.$$

Since the arch is semicircular,  $\theta = \frac{\pi}{2}$

$$S = \sin \theta = 1$$

$$C = \cos \theta = 0$$

Substituting values of  $s$ ,  $c$  and  $\theta$

$$H = WR (0.5171)$$

$$M_1 = WR^2 (1/2 \cdot 1 - 1/4 + 0) - 0.5171 WR^2 \left( \frac{2}{\pi} - 0 \right)$$

$$= -0.0794 WR^2$$

$$V_1 = T_s = WR_s = WR$$

Moment at crown  $M_c$  is now given by

$$M_c = -0.0794 WR^2 + 0.0171 WR^2$$

$$= 0.0623 WR^2$$

#### CASE I.

*Only vertical load (uniformly distributed) is acting*

*At Crown*

$$\text{Then } H = 0.5171 WR$$

$$M_c = 0.623 WR^2$$

Where  $W$  = load per rft.

$R$  = effective radius of the arch

$$p \text{ top, } p \text{ bottom} = pt, b = \frac{H}{\pi} \pm \frac{Mc Y}{I}$$

$$= \frac{0.5171W \times 8.875}{2.75} \pm \frac{0.0623W \times 6 \times 8.875^2}{2.75^2}$$

$$\text{or } pt = 5.563W \text{ lb./sft.} = 0.0386W \text{ psi.}$$

$$pb = -2.223W \text{ lb./sft.} = -0.015W \text{ psi.}$$

*At Springing*

$$M_1 = 0.0794 WR^2$$

$$T_s = WR$$

$$pt, b = \frac{W \times 8.875}{2.75} \pm \frac{6W \times 0.0794 \times 8.875^2}{2.75^2}$$

$$= 3.23W \pm 4.966W$$

$$pt = 8.196W \text{ lbs./sft.} = 0.0589W \text{ psi.}$$

$$pb = -1.736W \text{ lbs./sft.} = -0.012W \text{ psi.}$$

## CASE II

When lateral load due to earth pressure is taken into consideration

Assuming lateral pressure equal to  $\frac{pr}{3}$ , the values of moments and thrust work out as below :—

At Crown

$$H = 0.658 WR$$

$$M_c = 0.0485 WR^2$$

$$\begin{aligned} pt, pv &= \frac{H}{\pi} \pm \frac{Mcy}{I} = \frac{0.658W \times 8.875}{2.75} \\ &\quad \pm \frac{0.0485W \times 6 \times 8.875^2}{2.75^2} \\ &= 2.12W \pm 3.04W \end{aligned}$$

$$pt = 5.16W \text{ lbs./s.ft.} = 0.036W \text{ p.s.i.}$$

$$pb = -0.92W \text{ lbs./s.ft.} = -0.006W \text{ p.s.i.}$$

At Springing

$$T_s = WR$$

$$M_1 = 0.0407WR^2$$

$$\begin{aligned} pt, b &= \frac{W \times 8.875}{2.75} \pm \frac{0.0407W \times 8.875^2 \times 6}{2.75^2} \\ &= 3.23W \pm 2.54W \end{aligned}$$

$$pt = 5.77W \text{ lbs./s.ft.} = 0.04W \text{ p.s.i.}$$

$$pb = 0.69W \text{ lbs./s.ft.} = 0.0048 \text{ p.s.i.}$$

## CASE I—

Full overburden load, width of wedge = 25 ft.

Horizontal pressure =  $pt = 0$ .

Load of soil wedge =  $120 \times 125 \times 25$  lbs.  
= 3,75,000 lbs.

Load per r.ft. of on effective span of conduit

$$= \frac{3,75,000}{17.75} = 21,100 \text{ lbs.}$$

$$\text{Self load of arch} = \frac{\pi \times 8.875 \times 150}{2 \times 8.875} = 236 \text{ lbs./r.ft.}$$

$$\begin{aligned} \text{Total load per r.ft.} &= W = 21,100 + 236 \\ &= 21,336 \text{ lbs.} \\ \text{or} &= 21,400 \text{ lbs. say} \end{aligned}$$

Table 1

	Location		Stress in p.s.i.
Crown	Top	... 0.0386W	827
	Bottom	... - 0.0154W	- 330
Crown	Top	... 0.0569W	1220
	Bottom	... - 0.012W	- 256

## CASE II—

Full overburden load, width of wedge = 20.5 ft.

$$\text{Horizontal pressure, } ph = \frac{pv}{3}$$

$$\begin{aligned} \text{Load of soil wedge} &= 120 \times 125 \times 20.5 \\ &= 3,08,000 \text{ lbs.} \end{aligned}$$

$$\begin{aligned} \text{Load due to earth fill per r.ft. on effective span} &= \frac{3,08,000}{17.75} \\ &= 17,350 \text{ lbs./r.ft.} \end{aligned}$$

Self load of arch

Total load,  $W$

$$= 236 \text{ lbs./r.ft.}$$

$$= 17,586 \text{ lbs./r.ft.}$$

$$\text{or} = 17,600 \text{ lbs./r.ft. say}$$

Table 2

	Location		Stresses in p.s.i. ( $W = 17,600 \text{ lbs.}$ )
Crown	Top	... 0.036W	635
	Bottom	... - 0.0064W	- 113
Springing	Top	... 0.04W	715
	Bottom	... 0.0048W	85

## CASE III—

Ditch condition—

$$H_c = 120 \text{ ft.}$$

$$B_d = 29.25 \text{ ft., } B_c = 20.5$$

$$\frac{H_c}{B_d} = \frac{120}{29.25} = 4.1.$$

$$W = 125 \text{ lbs./c.ft.}$$

$C_d = 2.55$  (from curves on page 401 of Spangler's Soil Engineering  
Total load per r.ft. =  $W_c = C_d \times W \times B_d^2$

$$= \frac{2.55 \times 125 \times 29.25 \times 29.25}{20.25}$$

$$= 13,300 \text{ lbs./r.ft.}$$

$$\text{Load on the effective span of conduit} = \frac{13,300 \times 20.25}{17.75}$$

$$= 15,200 \text{ lbs./r.ft.}$$

$$\text{Self weight of Arch} = 236 \text{ lbs./r.ft.}$$

$$\text{Total load, } W = 15,436 \text{ or Say } 15,500 \text{ lbs./r.ft.}$$

$$\text{Horizontal stress } ph = \frac{pv}{3}$$

Table 3

		Location.	Stress in p.s.i. ( $W = 15,000 \text{ lbs.}$ )	
Crown	Top	...	$0.036W$	560
	Bottom	...	$-0.0064W$	- 99
Springing	Top	...	$0.04W$	620
	Bottom	...	$0.0048W$	74.5

APPENDIX III

Extract 1

LIST OF EARTH DAM FAILURES

[ Extracted from—(i) 'Engineering for Dams' by Creager, Justin and Hinds, Vol. III, pages 660-661.

(ii) Rao, K. L., 'Failure of Earth Dams', International Society of Soil Mechanics and Foundation Engineering, Regional Conference (Asia), 1960. ]

Serial No.	Name	Location	Height (ft.)	Date of failure	Core	Remarks
1	2	3	4	5	6	7
1	Fort Peek *	.. .. Montana	.. 225	1938	Silt hydr. fill ..	.. Foundation failure, low shear strength.
2	La Regadera*	.. .. Colombia, So. Amer	.. 108	1937	Clay ..	.. Foundation failure in plastic clay.
3	Marshall Creek*	.. .. Kansas ..	.. 70	1937	Clay, silt ..	.. Foundation failure in plastic clay.
4	Clendening*	.. .. Ohio ..	.. 62	1934	Imper. rolled material ..	.. Slight shear failure of plastic embankment.
5	Tappan*	.. .. Ohio ..	.. 52	1934	Rolled material 25 per cent. clay.	.. Movement from excess consolidation.
6	Belle Fourche*	.. .. S. Dakota ..	.. 115	1933	Rolled earth ..	.. Upstream slide, sudden draw-down.
7	Alexander	.. .. Hawaiian Islands	.. 125	1930	Hydraulic fill ..	.. Internal liquid pressure.
8	Balsam	.. .. New Hampshire	.. 60	1929	Concrete ..	.. Spillway discharge eroded toe caused sloughing.
9	Pleasant Valley	.. .. Utah ..	.. 63	1928	Puddle cutoff earth fill ..	.. Settlement and piping.
10	Table Rock Cove	.. .. Greenville, S. C.	.. 140	1928	Clay ..	.. Partial failure due to broken outlet pipe.
11	Puddingstone	.. .. California ..	.. 50	1926	Conc. facing ..	.. Overtopping.
12	Apishapa	.. .. Colorado ..	.. 115	1923	Concrete ..	.. Settlement cracks, caused piping.
13	Calaveras *	.. .. California ..	.. 240	1918	Clay ..	.. Liquid pressure on shells.
14	Lower Otay	.. .. California ..	.. 136	1916	Steel ..	.. Insufficient spillway.
15	Weisse, Passe River	.. .. Bohemia ..	.. 42	1916	Steel ..	.. Seepage along conduit.
16	Lyman	.. .. Arizona ..	.. 65	1915	Puddled ..	.. Piping and sloughing.
17	Horse Creek	.. .. Colorado ..	.. 56	1914	None ..	.. Piping and sloughing.
18	Lake George	.. .. Colorado ..	.. ..	1914	Puddled ..	.. Piping.



19	Hatchtown	..	..	Utah	..	65	1914	....	Seepage along culvert.
20	Hebron	..	..	New Mexico	..	56	1914	....	Water through gopher holes.
21	Davis Reservoir	..	..	California	..	39	1914	....	Piping. No cutoffs on gate structure.
22	Supulrida Canyon	..	..	California	..	65	1914	Rein. concr. core	.. Insufficient spillway.
23	Colorado Springs	..	..	Colorado	..	50	1912	....	Partial failure due to piping.
24	West Julesburg	..	..	Colorado	..	50	1910	None	.. Seepage along ledge rock.
25	Zuni	..	..	Black Rock, N. Mex.	..	70	1909	None	.. Hydraulic fill and rock piping.
26	Necaxa*	..	..	Mexico	..	193	1909	Clay	.. Sloughing during construction.
27	Debris Barrier No. 1, Yuba River.	..	..	California	..	..	1907	....	Insufficient spillway.
28	Lake Avalon	..	..	New Mexico	..	48	1904	....	Piping.
29	Greenlick, Scottsdale Dam	..	..	Pennsylvania	..	60	1904	....	Piping.
30	Utica Reservoir	..	..	New York	..	70	1902	....	Steep slopes, poor construction.
31	Lake Francis	..	..	California	..	50	1899	None	.. Settlement and seepage along outlet conduit.
32	Snake Ravine	..	..	California	..	64	1898	Hydraulic	.. Poor construction.
33	Johnstown	..	..	Pennsylvania	..	72	1889	None	.. Insufficient spillway.
34	Ashti*	..	..	India	..	58	1883	Puddled	.. Partial foundation failure; lower shear strength.
35	Swansea	..	..	South Wales, Great Britain.	..	80	1879	Puddle	.. Piping.
36	Saluda Dam	..	..	Columbia, U.S.A.	..	208	1932	Impervious	.. Lack of proper drainage facilities on the downstream side.
37	Kingsley Dam	..	..	Nabraska	..	160	1941	....	Upstream facing failed due to wave action sucking out the underlying fine material.
38	Harrogate Dam	..	..	U.K.	..	29	1953	....	Due to severe drought, there was shrinkage of clay fill which gradually crept downward towards the toe.
39	Chingford Embankment	..	..	U.K.	..	34	1937	Puddle Core selected fill	.. The Section of dam designed was not safe.

APPENDIX III—contd.

Serial No.	Name	Location	Height (ft.)	Date of failure	Core	Remarks
1	2	3	4	5	6	7
40	Palakmati Dam ..	.. M. P., India	.. 48	1953	Selected material	.. Slip of the upstream slope for a length of 400 ft.
41	Ahraura Dam ..	.. U. P., India	.. 75	1953	Homogenous ..	... Breach 100 ft. wide in the embankment near the intake of a sluice.
42	Arwar Dam ..	.. Rajasthan India	.. 41	1956	Impervious ..	.. Due to unsatisfactory construction and inadequacy of proper cutoff below the dam. Bonding of the new work in the breach was not properly done.
43	Guddah Dam ..	.. Rajasthan, India	.. 93	1956	....	Due to faulty design of right sluice retaining wall and inadequacy of the cutoff.
				1956	....	Bad bonding of new and old work.
44	Kaddam ..	.. Andhra Pradesh, India...	74	1958	Composite ..	.. Inadequate spillway capacity. Incorrect operation of gates.

\* Slide or movement during construction, dam later repaired and completed.

APPENDIX III—*contd.***Extract 2**

(Extracted from 'Engineering for Dams' by Creager, Justin and Hinds, Vol. 1, page 202.)

In estimating spillway requirements for relatively low dams, the failure of which would not cause extreme property damage or constitute a serious threat to life, the margin of safety may be made consistent with economic analysis. However, where high embankments are involved, the damages that would result from failure of the dam because of an inadequate spillway would be greater than could be repaired by the owners of the project, even though equitable compensations were practicable in such cases. The social repercussions and hazards to life resulting from such a failure are not susceptible of economic evaluation.

If danger to the structures alone were involved, the sponsors of many projects would prefer to rely on the improbability of an extreme flood occurrence rather than to incur the expense necessary to assure complete protection. However, when a major portion of the risks involve downstream interests, a very conservative attitude is required in developing spillway design-flood criteria. Probable future development in the downstream flood plain, as well as existing conditions, must be taken into consideration in estimating potential damages and hazards to human life that would result from failure of a dam.

**Extract 3**

(Extracted from U. S. B. R. Design Supplement No. 2., Chapter 13, Outlet Works, Para. 13.21 C.)

The distribution of the vertical loads on the foundation depends on the stiffness of the invert and the elastic qualities of the foundation material. Analysis of the stresses in the conduit with the foundation reaction assumed to be either uniformly distributed across the base or triangularly distributed with a maximum at the edge of the base and zero at the center, are believed to represent the two extremes that need be considered in design. With the size and shape of conduit determined by the hydraulic requirements, a thickness of shell may then assumed and the following loads determined for use in design :—

(1) The external load on the top of the conduit is a uniformly distributed vertical load equal to the weight of fill and reservoir water directly above the spring line of the conduit. The external load on the sides of the conduit is a uniformly distributed horizontal load equal to one-third the vertical load, or no horizontal load. The vertical reaction on the bottom of the conduit is equal to the vertical load, either uniformly distributed, or triangularly distributed.

(2) The internal load is a radial load equal to the full hydrostatic pressure at the center line of the conduit.

**Extract 4**

(Extracted from U. S. B. Design Supplement No. 2, Chapter 13, Outlet Works, Para. 13.21 F.)

In all conduit layouts, provision should be made to minimise the seepage of water along the contact plane of the outlet and the dam embankment by the construction of projecting fins or cutoff collars around the exterior circumference of the conduit, particularly through the upstream and central portion of the impervious section of the dam. These collars should be from 2 to 3 feet in height, from 12 to 18 inches in width, and spaced at intervals of from 7 to 10 times their height, thus increasing the length of the path of percolation by 20 to 30 percent. Where the foundation consists of good sound rock, and tight contact between the concrete and the rock may be assured, the collars need not be extended across the foundation under the conduit. However, in these cases the lower ends of the collars along the sides of the conduit should be projected into the rock surface to prevent seepage under them. The weight of the conduit and the weight of the fill tend to increase the watertightness of the foundation joint. Where the foundation material is not absolutely tight, the collar should be extended across the foundation under the conduit to increase the path of seepage and, in addition, porous rock foundations should be grouted. The collars should be separated structurally from the barrel by providing a water-tight bituminous joint filler one half inch or more in width between the collar and the conduit barrel. This joint will permit lateral slipping of the collar on the barrel, eliminate secondary stresses which would otherwise be caused by the stiffening effect to the ribs, and avoid transmitting torsional stresses through the ribs to the barrel if horizontal movement or displacement of the embankment should occur. In tunnel construction, cutoff collars are very difficult and expensive to build, and it is doubtful whether they are of sufficient value in increasing the length of the path of percolation to warrant the additional work and cost. It is believed that grouting of the rock and grouting behind the tunnel lining is much more effective in reducing the seepage, whether the surrounding material is firm and stable, or soft and disintegrated.

APPENDIX III—*contd.***Extract 5**

(Extracted from "Engineering for Dams" by Creager, Justin and Hinds, Vol. III  
Page 710-11.)

Pipes placed in trenches excavated in the foundation should have concrete collars. Where the concrete collars are to be constructed, a cross trench should be dug in the bottom and into the sides of the main trench at least 24 in. Exposed faces of collars should have a vertical batter not steeper than 1 in 10 so that embankment when deposited, will tend to reach in tighter contact with it as its consolidation progresses. The concrete used for collars should be 1 : 2 : 4 mix or equivalent, and just wet enough to tamp readily as more water will cause shrinkage.

There should be not less than three cutoffs throughout the width of the impervious portion of the embankment. After the cutoff forms have been removed, the pipe trench should be carefully refilled; the earth being deposited in layers 3 or 4 in. and rammed thoroughly, with compressed air power tampers, before the next layer is placed. Before the next layer is deposited, the material in place should be lightly sprinkled with water, if it is not sufficiently moist, so as to insure bond between layers. Trenches of this kind should not be puddled, as puddling may cause the fill to shrink away from the walls of the trench.

**Extract 6**

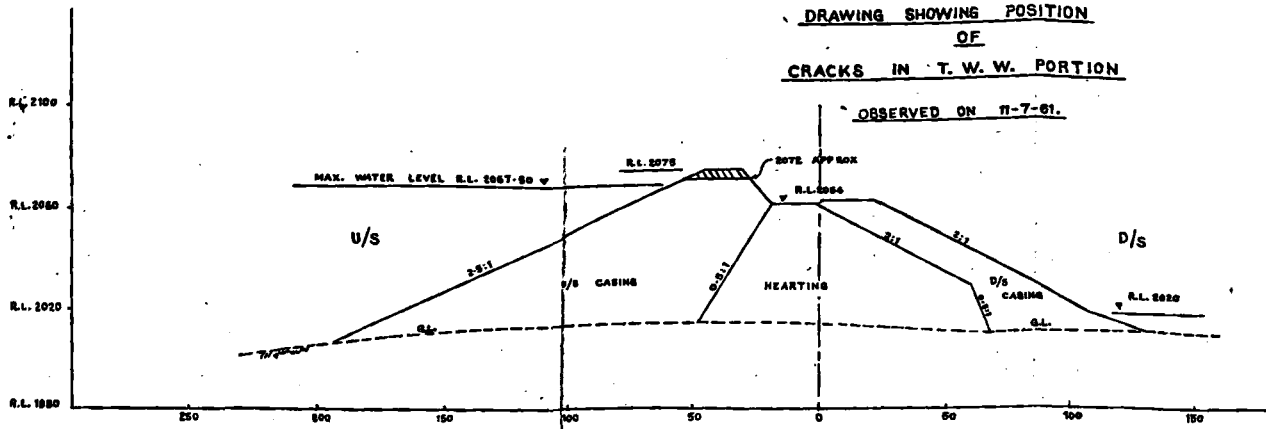
(Extracted from U. S. B. R. Design Supplement No. 2, Chapter 8, Page 17.)

"In the design of any embankment, careful consideration is given to the abutment contracts. Even though suitable cutoffs are provided as a means of intercepting seepage flow, any continuous void spaces caused by failure to effect an intimate contact between the impervious portion of the embankment and the abutments may result in dangerous seepage and even failure. All undesirable and overhanging rocks or formations are removed and such slopes established as necessary to make sure that the embankment materials can be properly compacted to form the desired intimate contact....."

**COMMISSION OF INQUIRY  
PANSHET AND KHADAKWASLA DAMS**

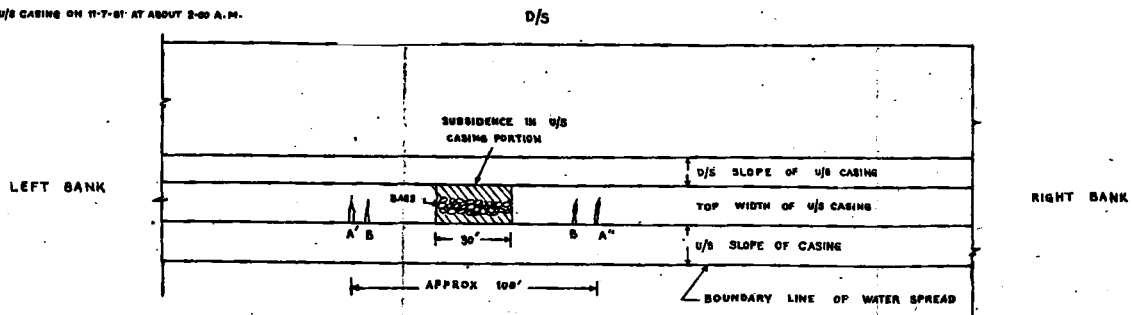
**DRAWING SHOWING POSITION  
OF  
CRACKS IN T. W. W. PORTION**

OBSERVED ON 11-7-61.



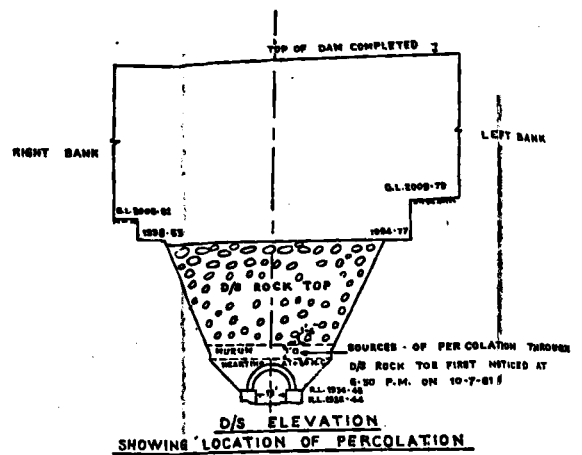
**CROSS SECTION OF PANSHET DAM IN THE T.W.W. PORTION  
(PORTION BELOW G.L. NOT SHOWN)**

NOTE:-  
THE SUBSIDENCE WAS FIRST  
OBSERVED IN THE HATCHED PORTION  
OF THE U/S CASING ON 11-7-61 AT ABOUT 2-00 A.M.

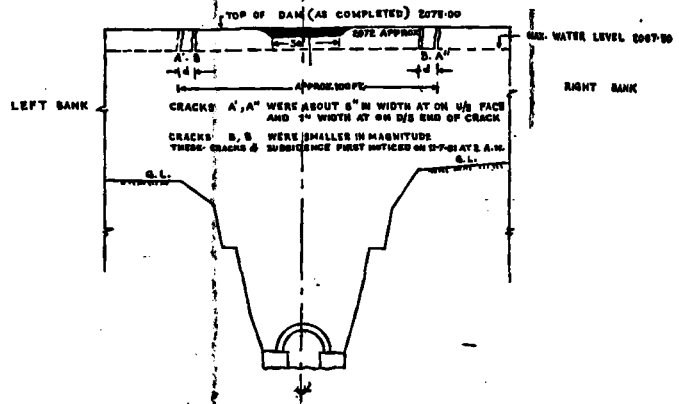


**PLAN SHOWING THE EXTENT AND SHAPE OF CRACKS**

NOTE :-  
(a) THE CRACKS A, A', B, B  
WERE WEDGE-SHAPED BEING  
WIDER ON U/S FACE  
(b) A' CRACK SLIGHTLY SMALLER THAN A



**D/S ELEVATION  
SHOWING LOCATION OF PERCOLATION**



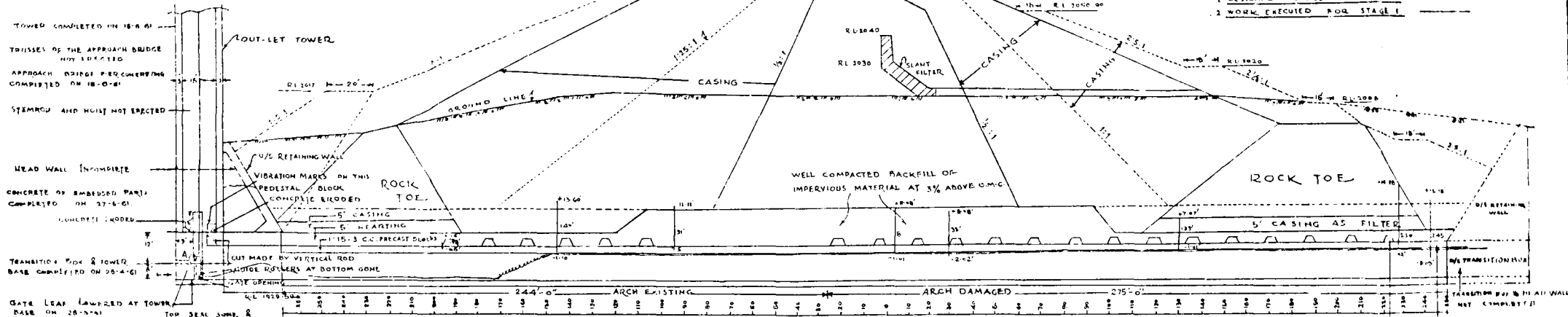
**UP STREAM ELEVATION  
SHOWING SUBSIDENCE & CRACKS IN U/S CASING.**

COMMISSION OF INQUIRY  
PANSHET AND KHADAKWASLA DAMS

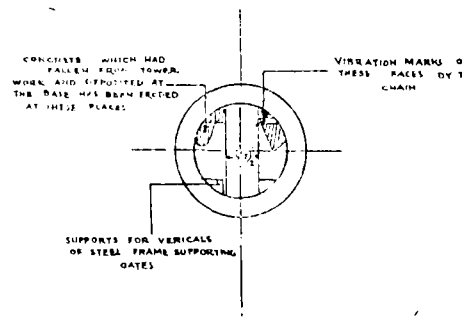
PLATE NO III

— DRAWING —  
SHOWING DETAILS OF THE LINTEL DAM SECTION & DAMAGE  
TO GATE LEAF AND CONDUIT AFTER FATIGUE

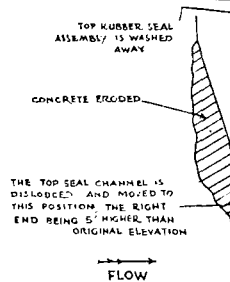
- REFERENCES:-  
1. DESIGNED PROFILE FOR STAGE I  
2. WORK EXECUTED FOR STAGE I



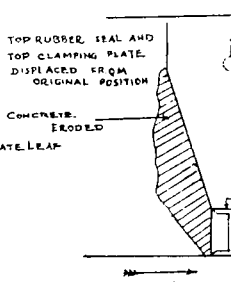
ARCH OF THE CONDUIT COMPLETED ON 25-9-61  
STANCHION RINGS COMPLETED ON 15-1-61



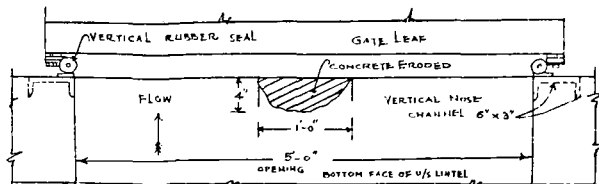
PLAN SHOWING DAMAGES TO TOWER BASE AT SILI LEVEL R.L. 1935-50  
Scale 0" = 15 feet



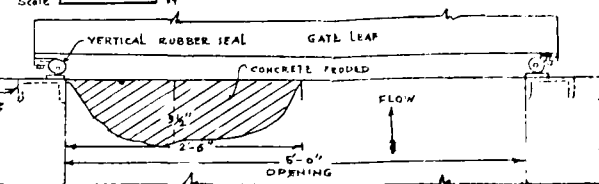
LEFT GATE END VIEW AT (A)  
Scale 0" = 1' Ft.



RIGHT GATE END VIEW AT (A)  
Scale 0" = 1' Ft.



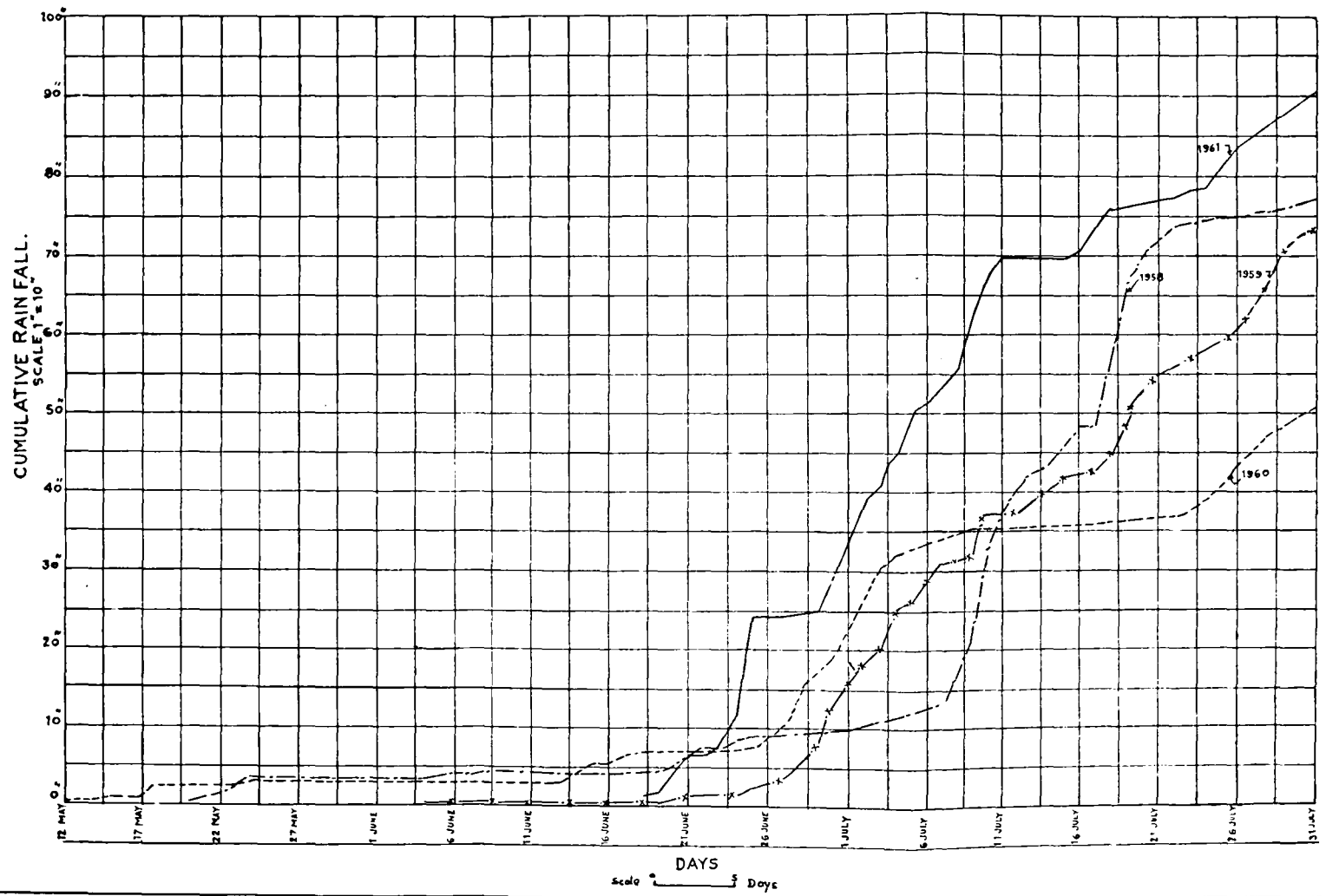
PLAN BOTTOM OF U/S LINTEL OF THE GATE VIEWED FROM BELOW LEFT SIDE OPENING  
Scale 0" = 1' Ft.



PLAN BOTTOM OF U/S LINTEL OF TOWER BASE VIEWED FROM BELOW RIGHT SIDE OPENING  
Scale 0" = 1' Ft.

PLAN SHOWING DAMAGE TO TOWER BASE  
Scale 0" = 15 feet

GRAPH SHOWING CUMULATIVE RAIN FALL  
AT PANSET DAM FOR THE YEARS  
1958, 1959, 1960, & 1961.

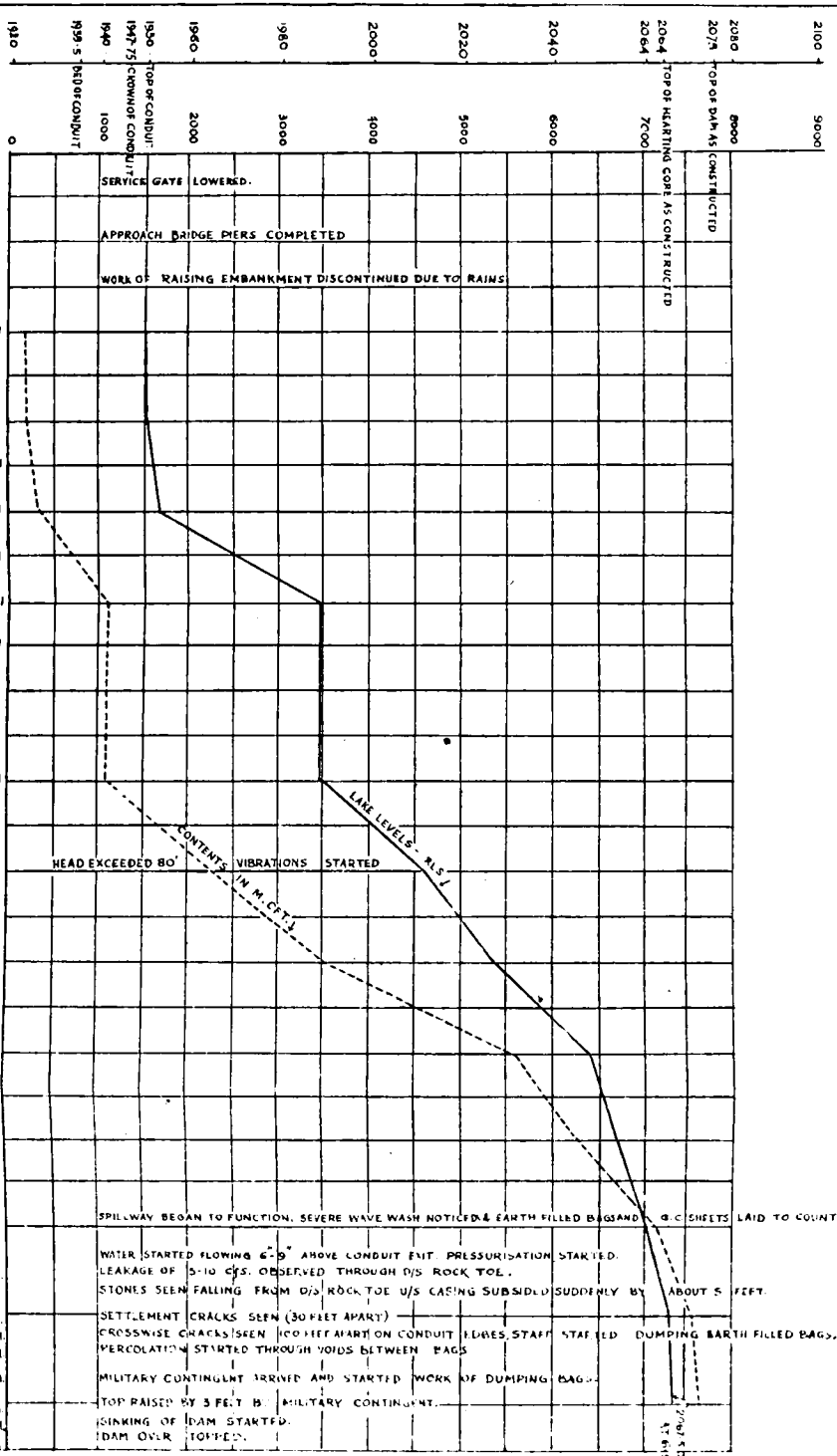


**COMMISSION OF INQUIRY**  
**PANSHET & KHADAKWASLA DAMS**

GRAPH FOR THE PERIOD FROM 10-6-61 TO 12-7-61  
 SHOWING

- 1 WATER LEVELS IN THE LAKE (MS) [SCALE: 1" = 30']
- 2 CONTENTS IN THE LAKE IN M.C.F.T. [SCALE: 1" = 1000 M.C.F.T.]
- 3 SEQUENCE OF EVENTS FROM 10-6-61 TO 12-7-61

WATER LEVEL CONTENTS  
 IN IN  
 THE LAKE M.C.F.T.



SERVICE GATE LOWERED.

APPROACH BRIDGE PIERS COMPLETED

WORK OF RAISING EMBANKMENT DISCONTINUED DUE TO RAINS

HEAD EXCEEDED 80'

VIBRATIONS STARTED

LINE LEVELS - 11.1

CONTENTS IN M.C.F.T.

SPILLWAY BEGAN TO FUNCTION. SEVERE WAVE WASH NOTICED & EARTH FILLED BAGS AND 6 C SHEETS LAID TO COUNTERACT IT.

WATER STARTED FLOWING 4'-5" ABOVE CONDUIT ENT. PRESSURISATION STARTED. LEAKAGE OF 5-10 CFS. OBSERVED THROUGH D/S ROCK TOE. STONES SEEN FALLING FROM D/S ROCK TOE U/S CASING SUBSIDED SUDDENLY BY ABOUT 5 FEET.

SETTLEMENT CRACKS SEEN (30 FEET APART) CROSSWISE CRACKS SEEN (100 FEET APART) ON CONDUIT EDGES STAFF STAFFED DUMPING EARTH FILLED BAGS. PERCOLATION STARTED THROUGH VOIDS BETWEEN BAGS

MILITARY CONTINGENT ARRIVED AND STARTED WORK OF DUMPING BAGS

TOP RAISED BY 3 FEET BY MILITARY CONTINGENT.

SINKING OF DAM STARTED.

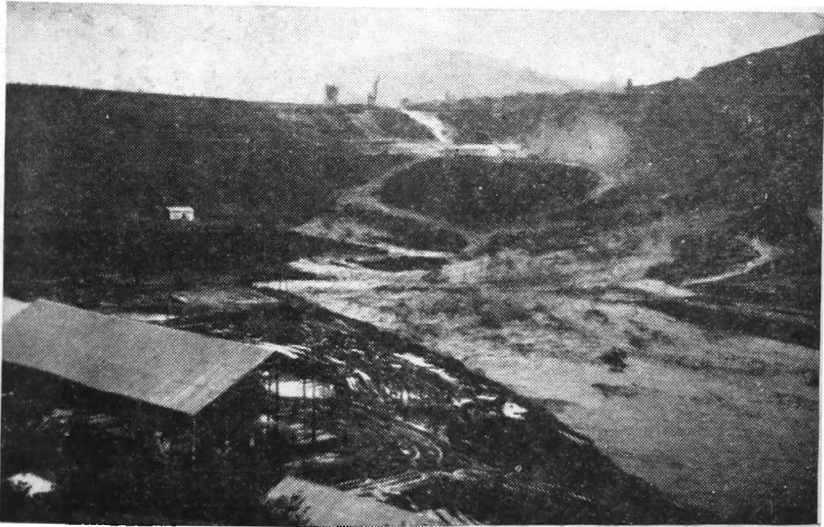
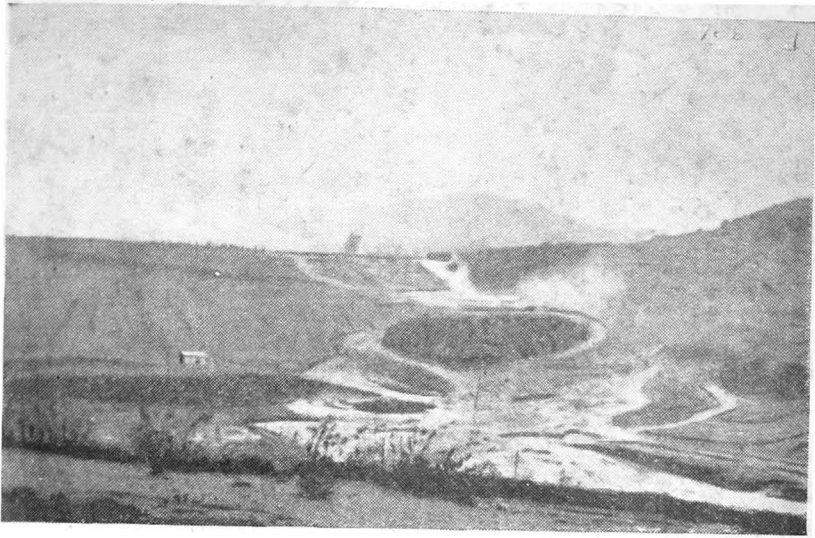
DAM OVER TOPPED.

TOP OF DAM AS CONSTRUCTED  
 TOP OF MARTIN CORE AS CONSTRUCTED

S. N. R. P. U. O. M. A. S. W. 22-11-61



PLATE No. V (a)



**Photographs showing programme undermining of the dam in Temporary Waste Weir portion after overtopping**

PLATE No. V (b)

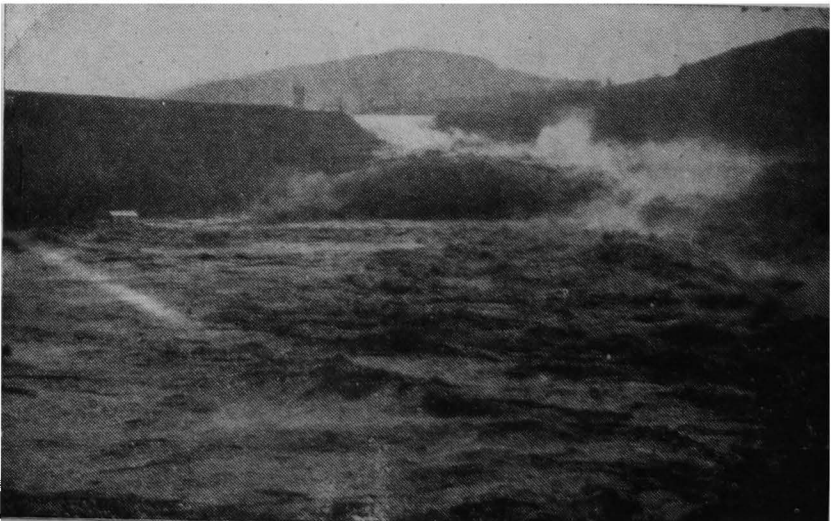
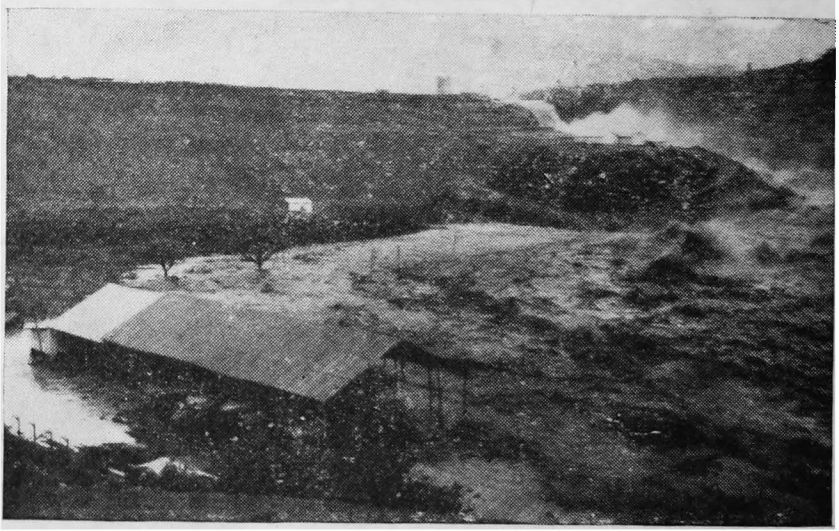


PLATE No. V (c)

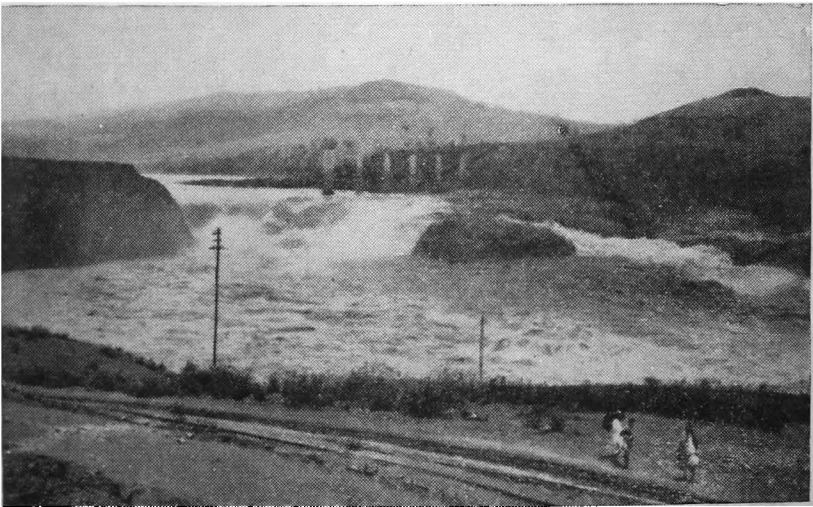
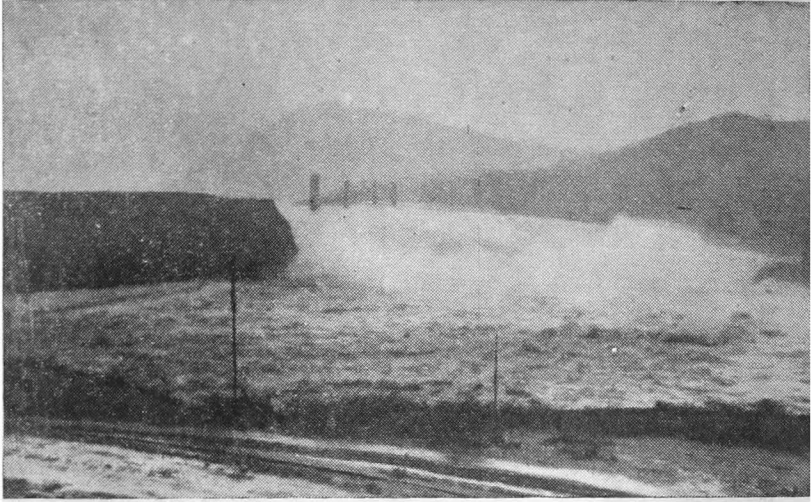


PLATE No. V (b)

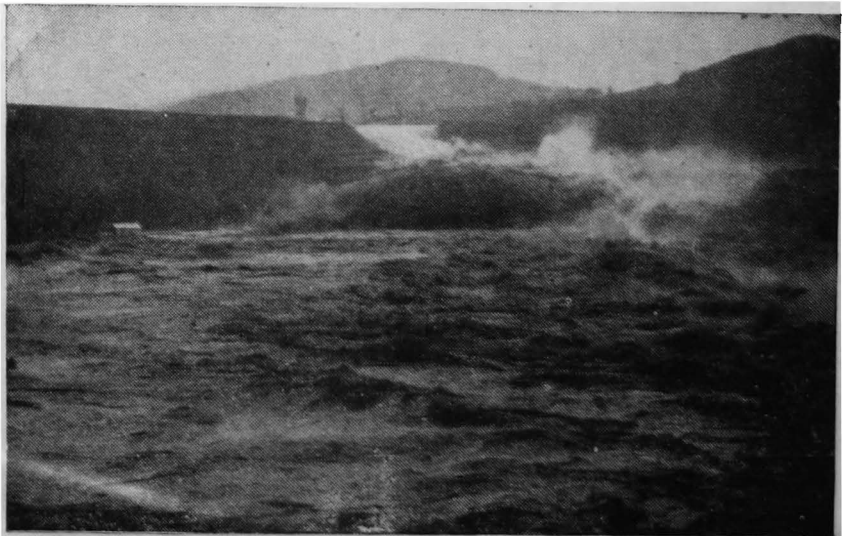
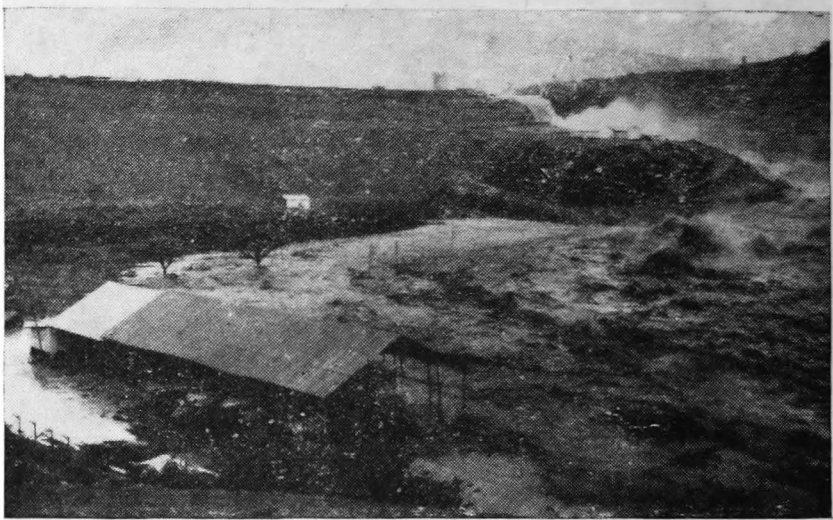


PLATE No. V (c)

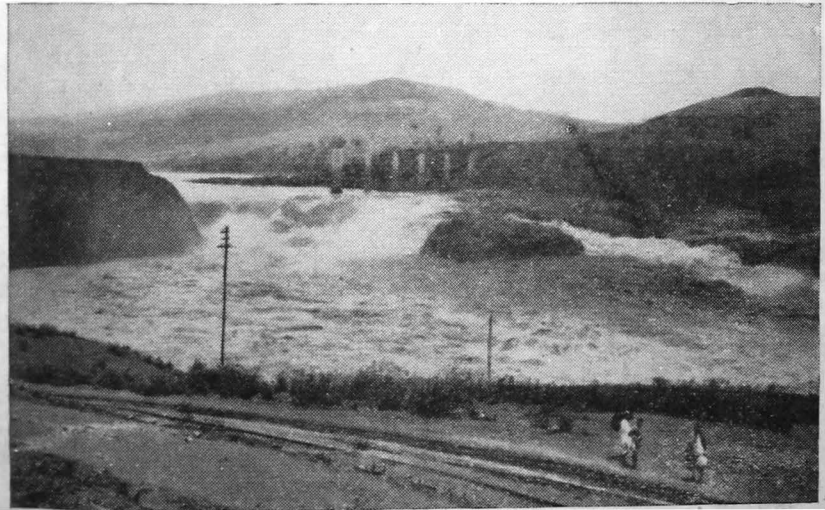
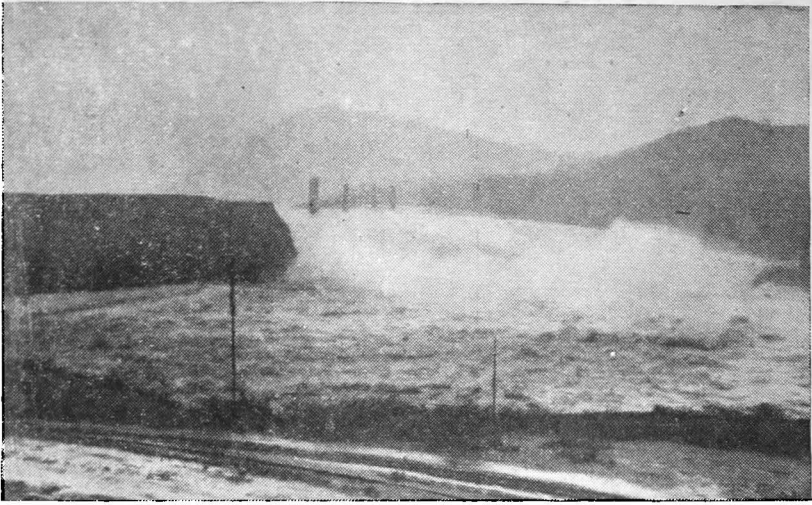


PLATE No. VI



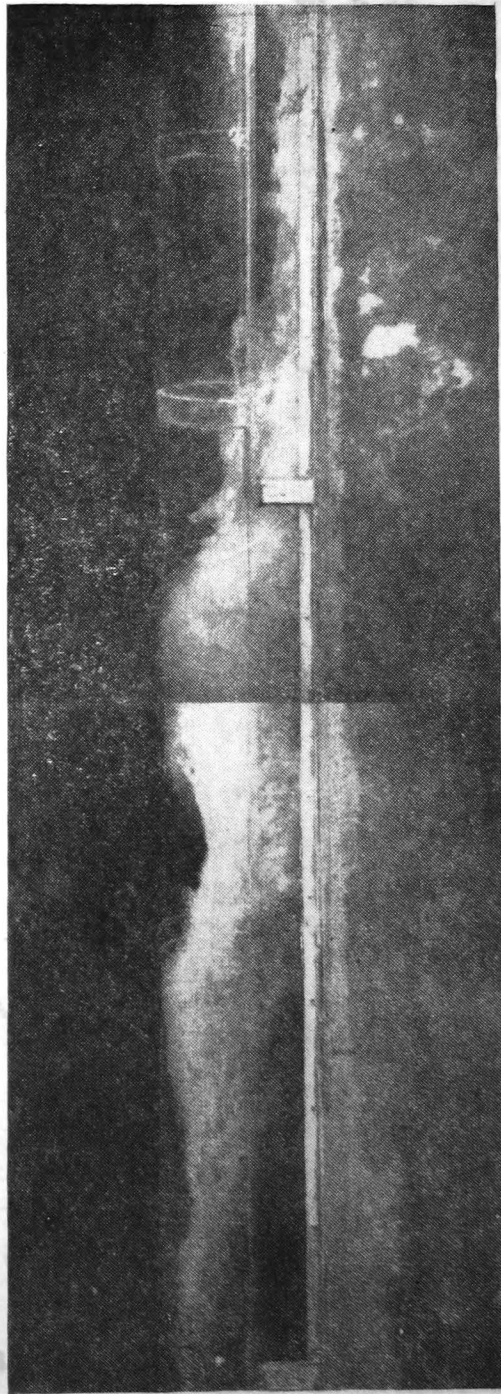
Photographs showing cavitation damage in control tower and conduit

**PLATE No. VII**

*Please see Photograph at Plate No. VII.*

**Photograph showing flow conditions in hydraulic model**

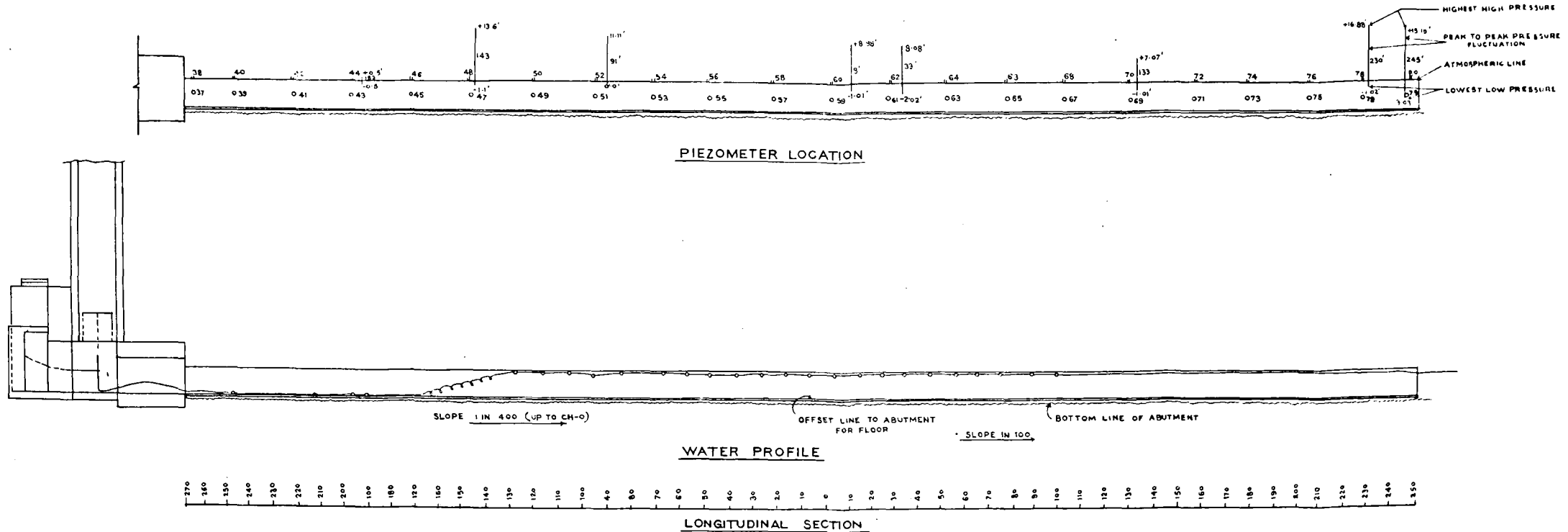
PLATE No. VIII



Photograph showing typical profile for hydraulic jump  
in hydraulic model



COMMISSION OF INQUIRY  
PANSHET AND KHADAKWASLA DAMS  
PANSHET DAM SLUICE OUTLET  
SHOWING  
WATER PROFILE FOR RESERVOIR R.L.2067.50  
AND PIEZOMETER LOCATION ON THE CONDUIT ARCH



# COMMISSION OF ENQUIRY PANSHET AND KHADAKWASLA DAMS

STABILITY ANALYSIS OF PANSHET DAM BY SWEDISH  
SLIP CIRCLE METHOD (ROTATIONAL FAILURE)

SCALE 1"=30'  
FEET

U/S SLOPE IN SUBMERGED CONDITION.

FORCES	CASING MATERIAL AT			HEARTING MATERIAL AT			CLAYEY MATERIAL +3% OF O.M.C. AT			ROCK TOE MATERIAL DRY.	TOTAL KIPS.
	SATUR-ATION	SUBMER-SED.	O.M.C.	SATUR-ATION	SUBMER-SED.	O.M.C.	SATUR-ATION	SUBMER-SED.	+3% OF O.M.C.		
SLIDING FORCE $T$	-	260.8	41.8	-	48.0	29.4	-	0.1	-	-34.8	ET = 345.3
RESISTING FORCE $N \tan \phi$	-	342.6	19.00	-	17.3	8.3	-	5.5	-	84.7	EN $\tan \phi = 47.4$
RESISTING FORCE $c$	-	-	15.6	-	-	48.3	-	-	26.8	-	ELc = 30.7
RESISTING FORCE $c$ WITH REDUCED VALUES OF C.	-	-	-	29.0	-	-	24.8	-	-	-	ELc = 55.8

FACTOR OF SAFETY =  $\frac{ELc + EN \tan \phi}{ET} = \frac{30.7 + 345.3}{345.3} = 1.66$  [C HEARTING = 500 LBS./SFT]  
 C CASING = 150 LBS./SFT  
 FACTOR OF SAFETY WITH REDUCED VALUE OF COHESION DUE TO SATURATION ... =  $\frac{55.8 + 477.4}{345.3} = 1.55$  [C HEARTING = 300 LBS./SFT]  
 C CASING = 0 LBS./SFT

U/S SLOPE IN DRAW DOWN CONDITION

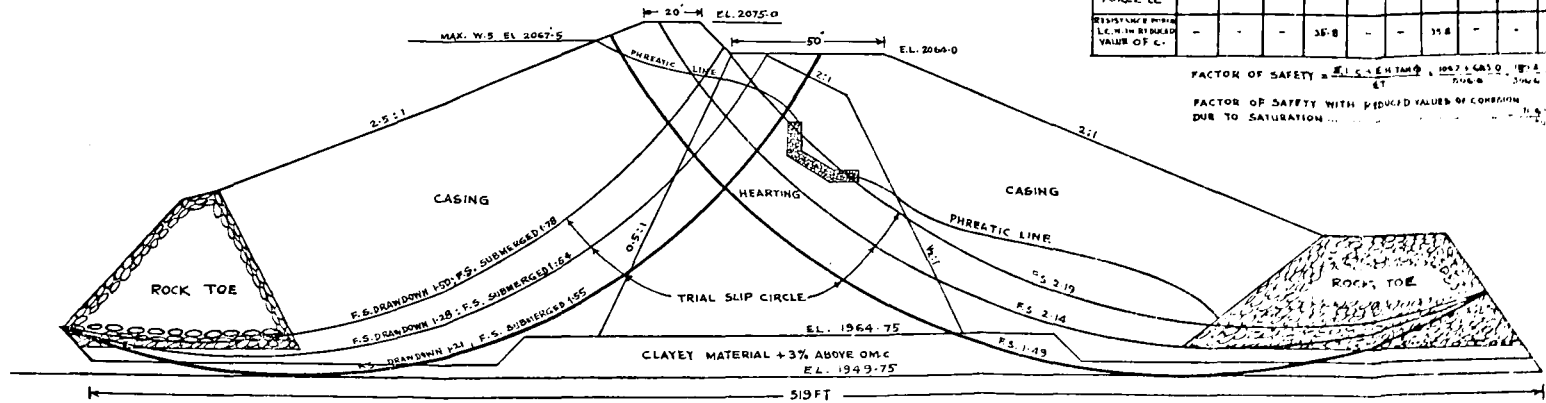
FORCES	CASING MATERIAL AT			HEARTING MATERIAL AT			CLAYEY MATERIAL +3% OF O.M.C. AT			ROCK TOE MATERIAL DRY.	TOTAL KIPS
	SATUR-ATION	SUBMER-SED.	O.M.C.	SATUR-ATION	SUBMER-SED.	O.M.C.	SATUR-ATION	SUBMER-SED.	+3% OF O.M.C.		
SLIDING FORCE $T$	487.8	-	41.8	88.5	-	29.4	0.2	-	-	-85.0	ET = 602.1
RESISTING FORCE $N \tan \phi$	-	491.1	19.00	-	17.3	8.3	-	5.5	-	133.7	EN $\tan \phi = 67.3$
RESISTING FORCE $c$	-	-	15.6	-	-	48.3	-	-	26.8	-	ELc = 30.7
RESISTING FORCE $c$ WITH REDUCED VALUES OF C.	-	-	-	29.0	-	-	24.8	-	-	-	ELc = 55.8

FACTOR OF SAFETY =  $\frac{ELc + EN \tan \phi}{ET} = \frac{30.7 + 602.1}{602.1} = 1.27$  [C HEARTING = 500 LBS./SFT]  
 C CASING = 150 LBS./SFT  
 FACTOR OF SAFETY WITH REDUCED VALUE OF COHESION DUE TO SATURATION ... =  $\frac{55.8 + 674.9}{602.1} = 1.21$  [C HEARTING = 300 LBS./SFT]  
 C CASING = 0 LBS./SFT

D/S SLOPE IN STEADY SEEPAGE CONDITION

FORCES	CASING MATERIAL AT			HEARTING MATERIAL AT			CLAYEY MATERIAL +3% OF O.M.C. AT			ROCK TOE MATERIAL DRY.	TOTAL KIPS
	SATUR-ATION	SUBMER-SED.	O.M.C.	SATUR-ATION	SUBMER-SED.	O.M.C.	SATUR-ATION	SUBMER-SED.	+3% OF O.M.C.		
SLIDING FORCE $T$	118.0	-	200.0	219.7	-	371.0	0.3	-	-	-81.1	908.6
RESISTING FORCE $N \tan \phi$	-	117.1	228.5	-	56.1	28.2	-	8.1	-	244.8	611.0
RESISTING FORCE $c$	-	-	10.7	-	-	39.7	-	-	35.8	-	168.2
RESISTING FORCE $c$ WITH REDUCED VALUES OF C.	-	-	-	35.8	-	-	35.8	-	-	-	71.6

FACTOR OF SAFETY =  $\frac{ELc + EN \tan \phi}{ET} = \frac{168.2 + 908.6}{908.6} = 1.18$  [C HEARTING = 500 LBS./SFT]  
 C CASING = 150 LBS./SFT  
 FACTOR OF SAFETY WITH REDUCED VALUES OF COHESION DUE TO SATURATION ... =  $\frac{71.6 + 908.6}{908.6} = 1.08$  [C HEARTING = 300 LBS./SFT]  
 C CASING = 0 LBS./SFT



SOIL CHARACTERISTICS

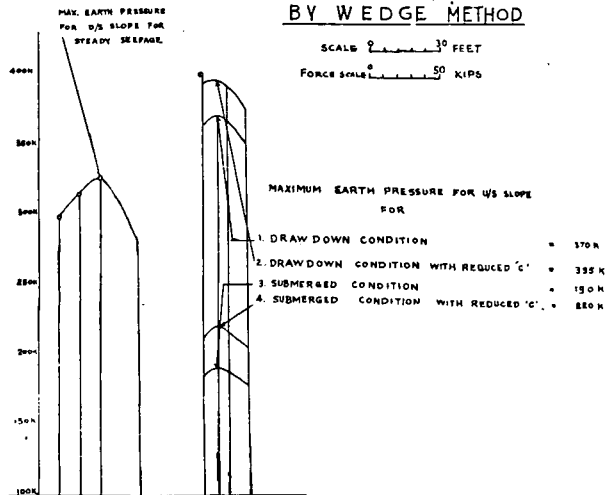
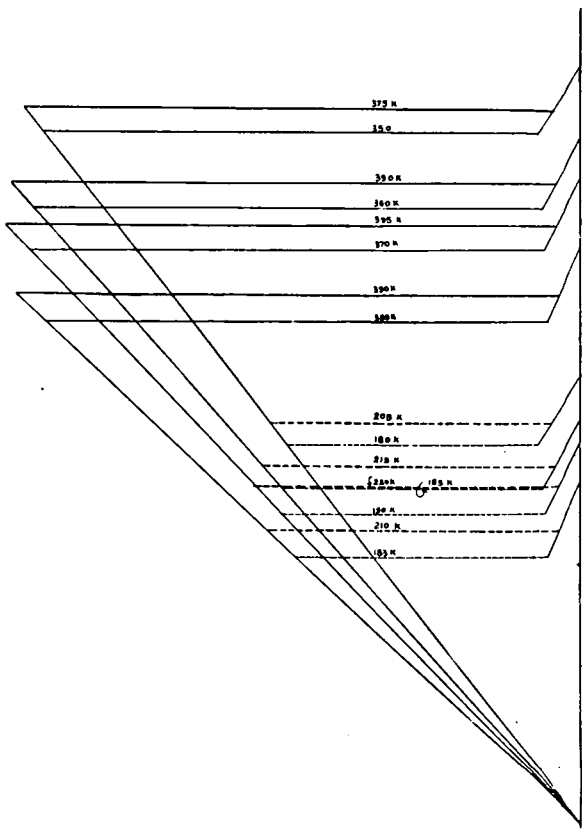
MATERIAL	DENSITY LBS./CFT.			COHESION LBS./SFT		ANGLE OF INTERNAL FRICTION (DEGREES)	
	AT O.M.C.	AT SATURATION	SUBMERSED	AT O.M.C.	AT SATURATION	AT O.M.C.	AT SATURATION
HEARTING	98.0	123.5	61.0	500	300	19.5	19.5
CASING	114.0	134.5	72.0	150	0	24.5	26.5
CLAYEY MATERIAL + 3% O.M.C.	94.0	119.5	57.0	300	300	16.5	16.5
FILTER ROCK	120	138.5	76.0	0	0	45.0	45.0

COMMISSION OF ENQUIRY  
PANSHET AND KHADAKWASLA DAMS

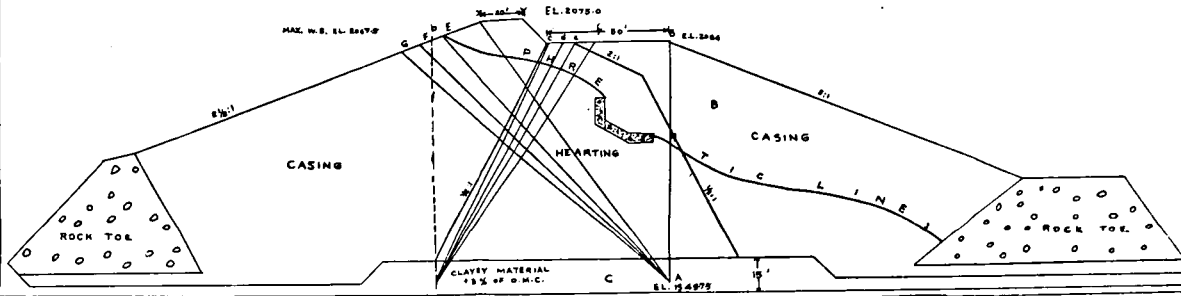
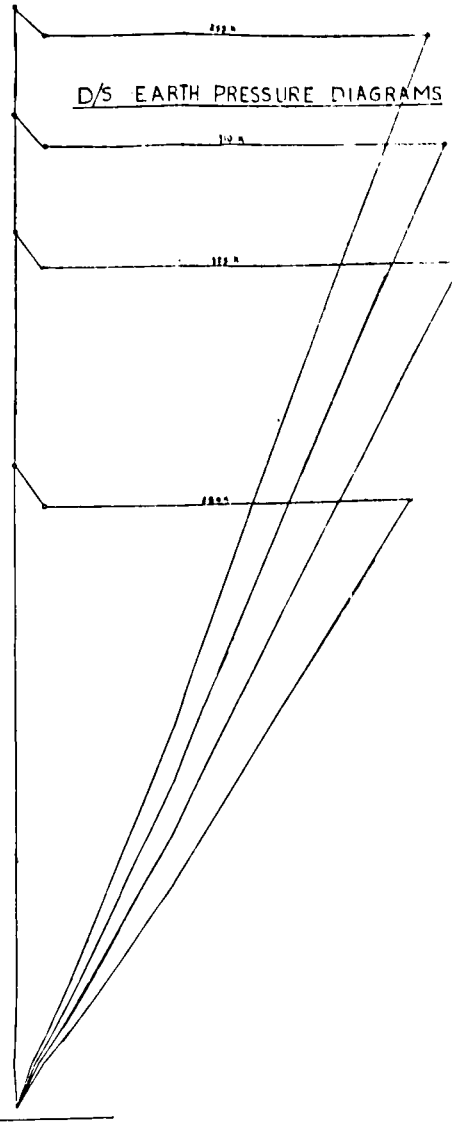
— SHOWING —

STABILITY ANALYSIS OF PANSHET DAM  
BY WEDGE METHOD

U/S EARTH PRESSURE DIAGRAM



D/S EARTH PRESSURE DIAGRAMS



DRAWDOWN CASE (FOR U/S SLOPE)

WEDGE	EARTH PRESSURES				STABILITY CALCULATIONS				
	W (KIP)	C (HIP) AT SAT	P (HIP) AT O.M.C	HT (FT) UP	W (KIP)	E.L.C	TAN $\phi$	WTANG $\phi$	S.F. AT
ab xyc	482	37	37	380.0	36.0	—	—	—	—
ab xycd	471	38	38	385.0	37.0	1.55	0.3	4.18	1.20
ab xycd	453	40	39	390.0	36.0	—	—	—	—
ab xycd	394	48	39	375.0	35.0	—	—	—	—

SUBMERGED CASE (FOR U/S SLOPE)

WEDGE	EARTH PRESSURES				STABILITY ANALYSIS				
	W (KIP)	C (HIP) AT SAT	P (HIP) AT O.M.C	HT (FT) UP	W (KIP)	E.L.C	TAN $\phi$	WTANG $\phi$	S.F. AT
ab xyc	285	37	37	210	185	—	—	—	—
ab xycd	280	38	38	220	190	1.07	0.3	2.07	1.55
ab xycd	297	40	39	215	185	—	—	—	—
ab xycd	328	40	38	208	180	—	—	—	—

SOIL CHARACTERISTICS

MATERIAL	DENSITY LBS/CUYD			COHESION LBS/50 FT		ANGLE OF INT. FRICTION	
	O.M.C	SAT	SUB	O.M.C	SAT	O.M.C	SAT
HEARTING	80	125.8	81	500	300	19.5	19.5
CASING	114	154.5	72	150	0	20.5	24.5
CLAYEY MATERIAL 15% OF O.M.C.	84	125.5	87	300	300	16.7	16.8
ROCK TOE	120	158.5	76.0	0	0	45.0	45.0

STEADY SEEPAGE (FOR D/S SLOPE)

WEDGE	EARTH PRESSURES				STABILITY CALCULATIONS				
	W (KIP)	C (HIP)	P (HIP)	HT (FT) UP	W (KIP)	E.L.C	TAN $\phi$	WTANG $\phi$	S.F. AT
ABY XE	478	37.3	280	—	—	—	—	—	—
ABY XE	638	31	325	158	21	0.3	4.77	1.53	—
ABY XE	712	31	310	—	—	—	—	—	—
ABY XE	736	30	275	—	—	—	—	—	—

LEGEND:-

- 1 DRAW DOWN
- 2 DRAW DOWN WITH REDUCED C
- 3 SUBMERGED
- 4 SUBMERGED WITH REDUCED C
- 5 STEADY SEEPAGE