ANALYSIS OF FAILURE AND RENOVATION OF PANSHET DAM

A Dissertation
submitted in partial fulfilment
of the requirements for the degree
of
MASTER OF ENGINEERING
in
WATER RESOURCES DEVELOPMENT

by

N.P. DUSE

WATER RESOURCES DEVELOPMENT TRAINING CENTRE
UNIVERSITY OF ROORKEE
ROORKEE

September 1971

CERTIFICATE

Certified that the dissertation entitled "ANALYSIS OF FAILURE AND RENOVATION OF PANSHET DAM" which is being submitted by Shri N.P. Duse in partial fulfilment for the award of the Degree of MASTER OF ENGINEERING in WATER RESOURCES DEVELOPMENT of the University of Roorkee is a record of candidate's own work carried out by him under my supervision and guidance. The matter embodied in this dissertation has not been submitted for award of any other Degree or Diploma.

This is further to certify that he has worked for a period of more than nine months from February 1967 for preparing this dissertation.

Countersigned

[illegible]

(V.R. DEUSKAR)
CHIEF ENGINEER (KOYNA)
MAHARASHTRA
EXTERNAL GUIDE

Dated:- 19-9-71

A C K N O W L E D G E M E N T S

The Author takes this opportunity to place on record his deep sense of gratitude to Shri V.R. Deuskar, Chief Engineer, Koyna and Prof. Prahlad Das, Professor Designs (Civil), W.R.D.T.C., his external and internal guides respectively for sparing their valuable time in giving their able guidance and constant encouragement during the period of preparation of this dissertation.

Deep and sincere gratitude is due to Shri H.B. Kulkarni, Superintending Engineer, Poona Irrigation Projects Circle, Poona for so kindly permitting to collect and make use of relevant data and photographs of the Panshet Project for writing this dissertation. The author also thanks Shri S.T. Deokule, Superintending Engineer, Sattara Irrigation Project Circle for his guidance.

The help and cooperation extended by others by way of making available the relevant data to the Author is also gratefully acknowledged.

N.P. DUSE

CONTENTS

SYNOPSIS

Construction of an earth dam at Panshet (25 miles away from Poona City of Maharashtra State) across river Ambi, a tributary of Mutha River, to store 7200 Mcft of water was started in the year 1957. It was practically completed by 20th June 1961 but a few items such as concreting the bed of the conduit, installation of hoisting equipment for the service gates, construction of an approach bridge to the outlet tower and some quantum of earth work over the temporary waste weir portion remained to be completed due to very heavy and untimely break of monsoon on 20th June. The continuous rain from 20th June had resulted in rapid rise in the lake level and within a very short period the lake got filled and the spillway started functioning on 10th July. Leakage in the downstream toe was simultaneously noticed as also subsidence in the upstream casing which could not be controlled inspite of all possible efforts and the dam finally breached on the morning of 12th July.

The Part I of the dissertation deals with the analysis of the circumstances and the causes of failure of the Panshet Dam.

Renovation of the Panshet Dam was undertaken in the year 1964 and the dam is now practically complete. The Part II of the dissertation deals with the analysis of the various possible layouts utilizing the undamaged part of the embankment on the right flank and the details of the final scheme adopted and its sequence of construction.

CHAPTER - 1

INTRODUCTION

1. A masonry dam 107 feet high was constructed in the year 1876-79 across river Mutha, a tributary of the Bhima River which in turn is a tributary of Krishna River. The small storage of 3092 Mcft available at the Khadakwasla Dam was subsequently found inadequate to meet the requirements of Irrigation. The water supply needs of the Poona City and the Cantonment was increasing day by day and the Government of Maharashtra decided to undertake a new Khadakwasla Project envisaging:

(i) Construction of an earth dam at Panshet across river Ambi, a tributary of Mutha to store 7500 Mcft of water.

(ii) Construction of an earth dam at Warasgaon across river Mose, a tributary of Mutha, to store 7500 Mcft of water.

(iii) Strengthening of the existing Khadakwasla Dam.

(iv) Suitable canal system to meet the requirements of water supply for the greater Poona and to provide perennial irrigation to 77000 acres of land in Poona and Dhond Talukas.

2. As there was no approach road either to Panshet or to Warasgaon, the approach road to the existing dam at Khadakwasla on the right flank of the Mutha River was extended to the Panshet Dam site. On its completion, the construction of the Panshet Dam was started in the year 1957. By the middle of June 1961 the work had

been practically completed but for the full dam section in the temporary waste weir cut and some works in the irrigation outlet.

3. Due to very heavy and untimely break of monsoon on 20th June 1961, there was very rapid rise in the lake level and the spillway started operating from 10th July. The lake level continued to rise and the dam breach on 12th July 1961 at a reservoir level of 2067.5.

4. Government of Maharashtra appointed a commission of enquiry for going into the causes of the failure. They also subsequently appointed another committee to recommend the most suitable plan for the development of the water resources of Mutha Valley. The reconstruction of Panshet Dam was taken up in the year 1964 after reports of both the committees had been received and decision for its renovation had been taken.

5. The dissertation deals with analysis of the causes of failure and the scheme of renovation of the Panshet Dam. The dissertation has been divided into two parts:-

Part I deals with the Analysis of Failure and Part II with the reconstruction scheme under the following headings:-

PART I Analysis of Failure:

(i) Inception of the project

(ii) General features of the scheme

(iii) Construction Programme and its achievement

(iv) Course of events leading to failure

(v) Commission of enquiry and model studies

(vi) Various theories about the cause of failure

(vii) Findings of the commission

PART II Reconstruction Scheme

(i) Study of alternative schemes of reconstruction

(ii) Detailed features of reconstruction scheme

(a) Earth Dam

(b) Masonry Dam & Divide Wall

(c) Spillway

(iii) Construction Details

6. The reconstruction works are practically over and the dam stored 7200 Mcft of water during this season i.e. 1971-72.

PART - I

ANALYSIS OF FAILURE

CHAPTER - 2

INCEPTION OF THE PROJECT

The eastern parts of the Poona District receive small amount of rainfall and even that is not uniformly distributed throughout the monsoon. The eastern part comprises of the Talukas of Haveli, Bhimthadi, Dhond and Indapur. The uncertain and scanty condition of rainfall in this area has been the cause of frequent failures of crops and of much suffering and distress among the people. In 1863-64 an unusually severe draught produced great distress just falling short of famine conditions which made the Government to realize the urgency of protecting the District by providing irrigation facilities.

1.2. The problem of planning the most suitable project was entrusted to Colonel Fife, who proposed the following scheme:-

(i) to construct a masonry dam 107 feet (32.61 m) high at Khadakwasla (about 12 miles (19.2 kilometres) above Poona City) holding a storage of 3092 Mcft (87.81×10^6 cu.m)

(ii) to provide a right bank canal 70 miles (112 km) long with a capacity of 412 cusecs (11.70 cumecs) at its head

(iii) to provide a left bank canal 18 miles (28.8 km) long with a capacity of 38.5 cusecs (1.09 cumecs) at its head

The project on the above lines was sanctioned by the Government of India in 1869 and the above works came into operation in 1874-75.

1.3. The small storage provided by the dam at Khadakwasla was soon after found inadequate to meet the requirements of irrigation. The water supply needs of the growing city of Poona and Cantonments of Poona and Kirkee were also rapidly increasing. In view of this shortage and for protecting the Indapur Taluka from famine, creation of additional storages upstream of Khadakwasla Dam were considered necessary and investigation work was taken up. The catchment area of the Mutha River at Khadakwasla is 196 sq.miles (5094 hectares) yielding an average runoff of 47,412 Mcft (1346.5×10^6 cu.m) annually. The valley affords an excellent site for a high dam downstream of the existing Khadakwasla dam. Mr. Beale therefore, proposed the following scheme in the year 1909:-

(i) to provide a storage of 22,000 Mcft (624.8×10^6 cu.m) by a new dam at Khadakwasla, 160 feet high (48.7 m)

(ii) to widen the existing canal upto mile 13 (20.8 km) to carry a discharge of 1000 cusecs (28.3 cumecs), &

(iii) to take off a new canal from mile 13 (20.8 km) for Indapur Area.

The scheme could command an area of 3,40,000 acres (1,36,000 hectares) of which 1,26,780 acres (50,712 hectares) could be irrigated annually. No further progress on the proposal could be made due to financial stringency caused by World War I.

1.4. The investigation of the project was reviewed in the year 1919 and the following three alternative proposals were considered:-

(i) Raising the existing dam at Khadakwasla

(ii) Constructing an altogether separate dam downstream of the present dam at Khadakwasla

(iii) Constructing two masonry dams at Panshet and Warasgaon upstream of Khadakwasla

1.5. The first proposal involving raising of the existing dam was rejected, due to:-

(a) the problems involved in making the old and the new masonry sections structurally monolithic

(b) risk to the stability of the existing dam while deeper foundations (as indicated by drill logs) for the extension were being excavated.

(c) danger to the existing dam by blasting required for foundation excavation for the new work.

(d) danger to the safety of the existing dam by the removal of earth berms (provided on the downstream face for the safety of the existing dam) for foundation excavation for the extension.

(e) as no risk to the safety of the existing dam could be run due to the vital importance of the dam for domestic water supply to the town of Poona

1.5.2. The proposal for constructing two masonry dams at Panshet and Warasgaon was not found to compare favourably with an enlarged storage by a new dam at Khadakwasla. Ultimately, therefore, the proposal for constructing a new masonry dam about 200 feet (60.96 m) downstream of the present structure at Khadakwasla to store 15,000 Mcft (426 x 10^6 cu.m) of water was framed and submitted to the Central Water & Power Commission in October 1954.

1.6. It was suggested by Dr. K.L. Rao, the then Member (D & R) Central Water & Power Commission, that the possibilities of construct-ing two earthen dams, one at Panshet and another at Warasgaon may be investigated. And accordingly necessary investigations were made and rough cost estimates prepared (See Appendix I) which indicated that the alternative suggested was economical and feasible. The proposal also had the advantage of permitting phased construction of 7500 Mcft (213 x 10^6 cu.m) storage at a time and making part benefits available (on completion of one dam) much earlier i.e. by the end of the Second Five Year Plan.

1.7. The project finally approved provided:-

(i) Construction of two earthen dams at Panshet and Warasgaon to impound 7600 Mcft (216 x 10^6 cum) each

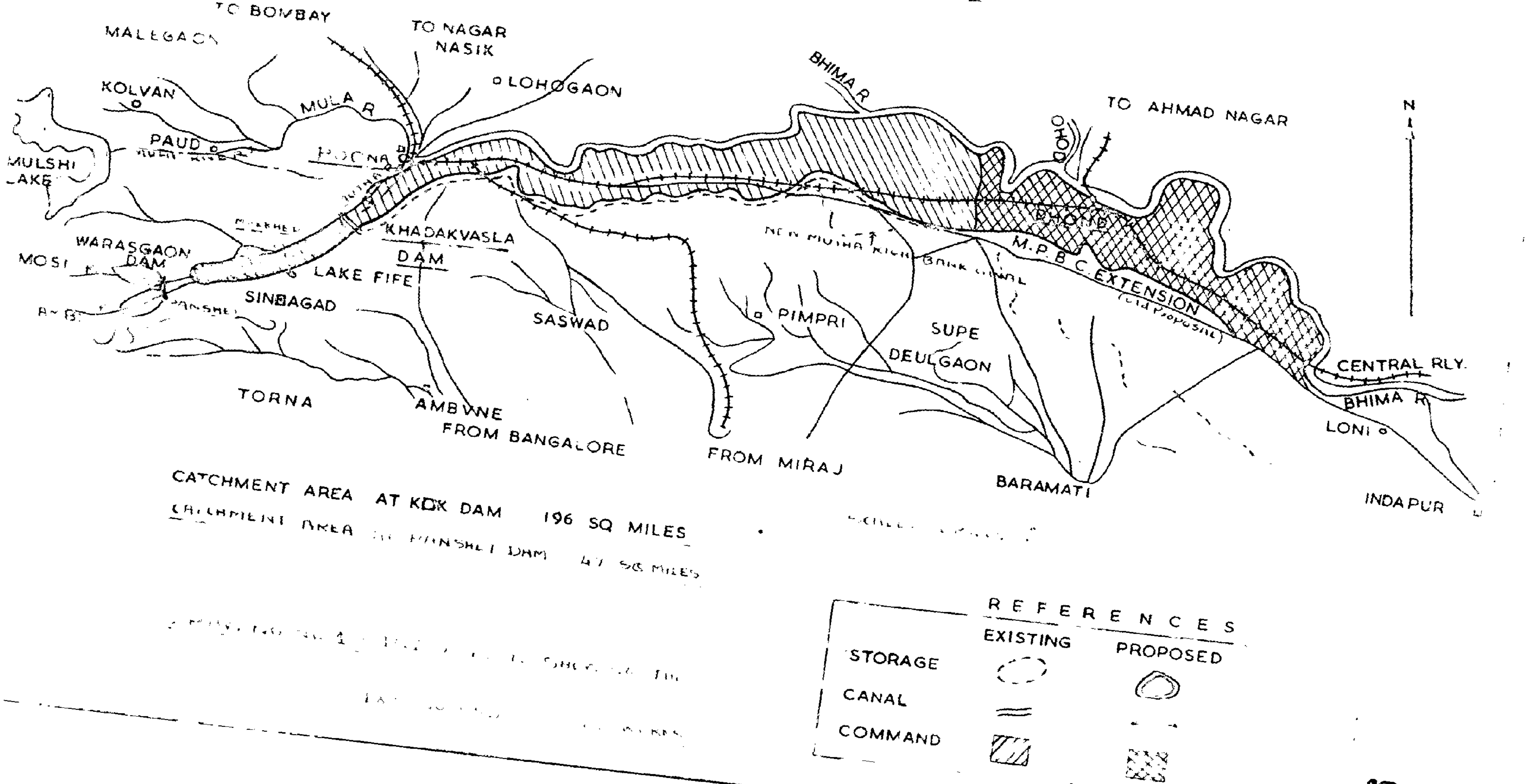
(ii) Release of waters from upper dams into Khadakwasla lake

(iii) Construction of a new canal offtaking from Khadakwasla of capacity 1100 cusecs (31.24 cumecs) using after remodelling as much of the old canal as possible.

(iv) Strengthening of the existing Khadakwasla Dam.

An index plan showing the abov-e proposal is enclosed as Drawing No.1.

KHADAKWASLA PROJECT



CHAPTER - 3

GENERAL FEATURES OF THE SCHEME (PANSHET DAM)

The headworks of the Panshet Dam can be broadly grouped into three categories viz.

(I) Earthen Dam

(II) Surplussing Arrangement

(III) Outlet

The broad details of the layout and their design features are discussed hereunder:-

1.1. (I) EARTHEN DAM

An earthen dam from Ch.500 to 2900 as shown in the longitudinal section (Drawing No.2) was constructed across the tributary named Ambi. The various control levels being

(i)	Top of dam	R.L. 2080.00	(R.L.634.0 m)
(ii)	H.F.L.	R.L. 2072.00	(R.L.631.54 m)
(iii)	F.S.L.	R.L. 2062.00	(R.L.628.50 m)
(iv)	Waste Weir Crest level	R.L. 2062.00	(R.L.628.50 m)

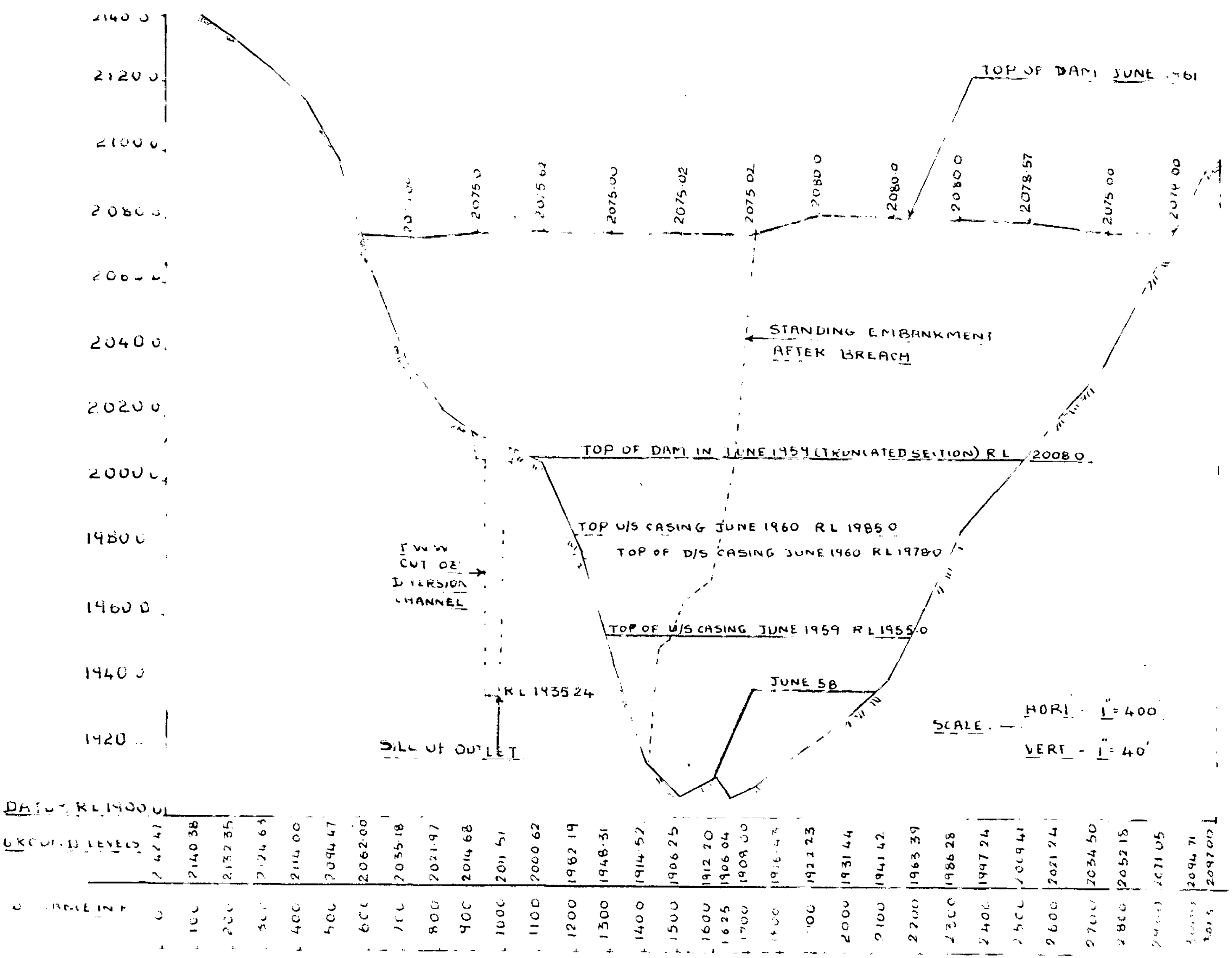
1.1.2. The embankment was zoned rolled fill comprising:-

(i) Central hearting core of clay

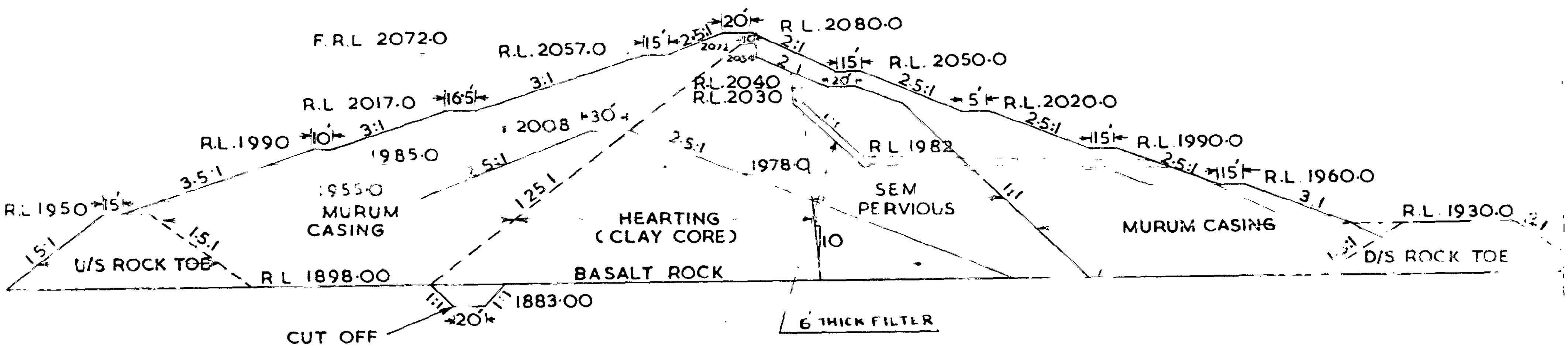
(ii) Downstream portion of mixed soft and hard murum (semipervious to impervious)

(iii) Substantial casing on either side of hard murum (pervious)

(iv) A chimney drain (introduced at a later stage when the earth work had been completed upto about R.L. 1980.00 (R.L.603.5 m))



DRAWING NO 2 : LONGITUDINAL-SECTION OF PANSHET DAM



NOTE: 1. WORK COMPLETED BY JUNE 59 [ ]
2. " " " JUNE 60 [ ]
3. " " " JUNE 61 [ ]

CROSS SECTION OF PANSHET DAM (JUNE 61)
(SEASONWISE WORK DONE)
SCALE 1" = 100'

DRAWING. No. 3

A section of earth dam showing various zones is shown in the Drawing No.3.

1.1.3. The deccan trap available at site is very compact and massive having practically nil water intake. No grout curtain was therefore considered necessary. A cut-off trench 3' to 4' (1 m) deep in the rock and backfilled by impervious material was, however provided. The stability analysis of the earth dam was done by the conventional slip circle method taking the pervious casing material (not free draining) as 50% buoyant.

A free board of 8' (2.44 m) was provided for a wind velocity of 80 miles per hour (128.7 km per hour).

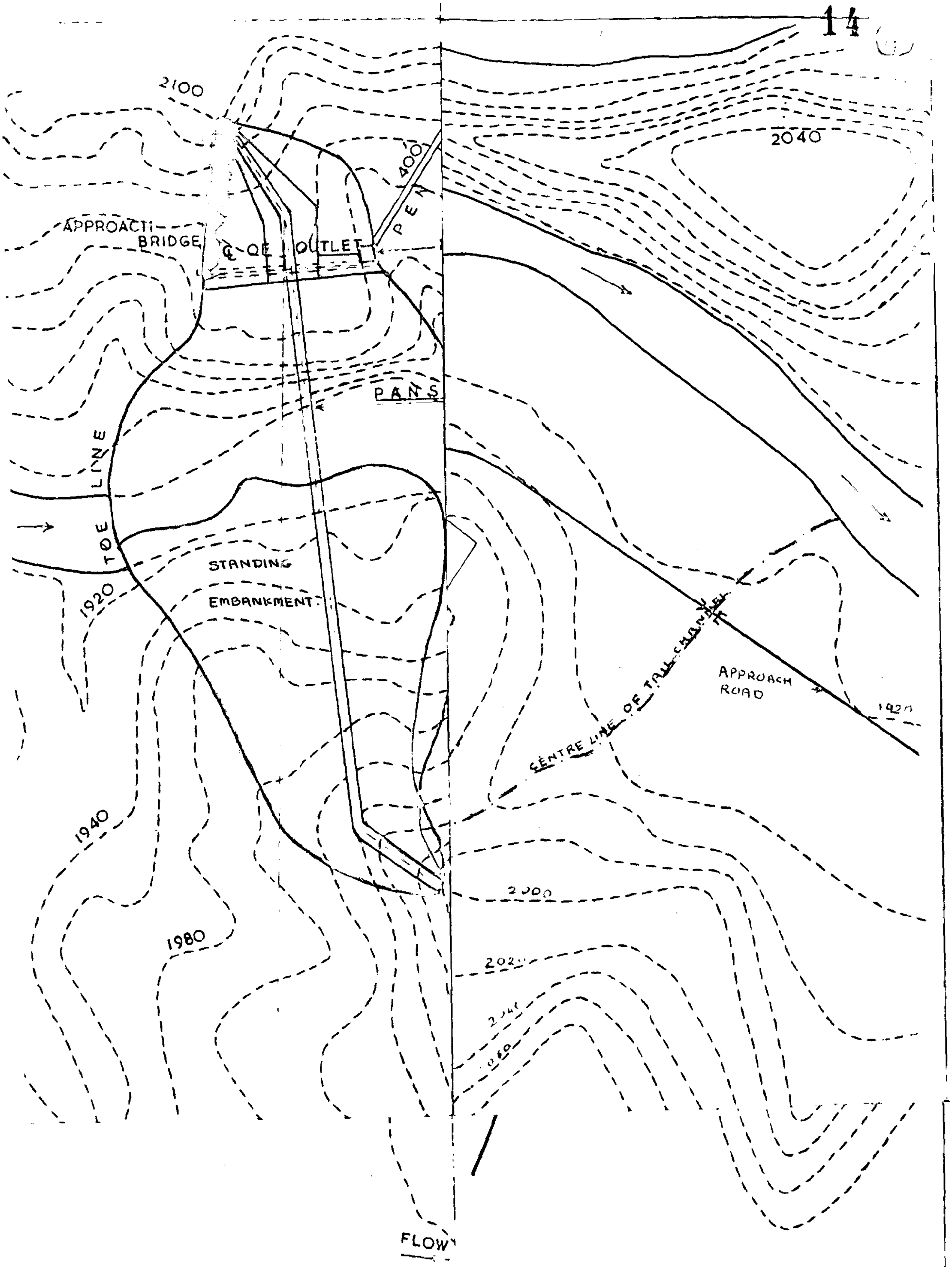
II SURPLUSSING ARRANGEMENT

An ideal site for providing spillway was available in the natural saddle on the right flank. Approach and tail channels were excavated and it was planned to direct the floods through this channel which would work as open channel to start with. The bed level of this channel was kept at R.L.2062.00 (R.L.628.50 m) and the channel of 172 feet (52.42 m) width was constructed. Ultimately it would be provided with an ogee crest with three 41' x 27' (12.5 m x 8.23 m) tainter gates to cater for a spillway design flood of 48000 cusecs (1363 cumecs). The water thus diverted through the tail channel was led into the natural river as shown in Drawing No.4.

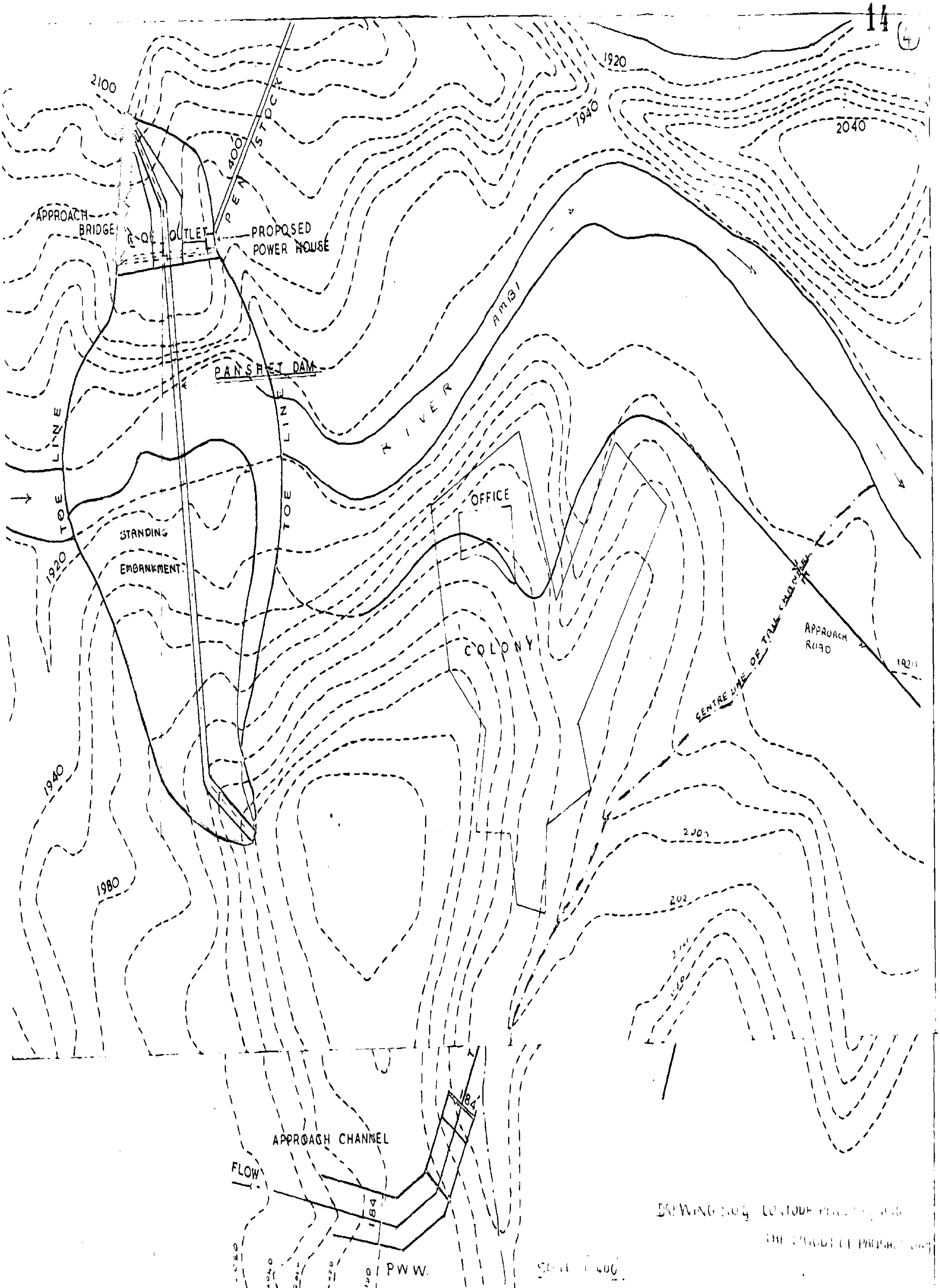
III OUTLET

(a) DISCHARGE REQUIREMENT

The outlet at Panshet was designed for a discharge of



DRAWING NO 4 : CONTOUR [illegible]
THE [illegible]



DRAWING NO 4 CONTOUR

1000 cusecs (28 cumecs) to meet the fair weather canal requirements.

(b) LOCATION

The most convenient and economical location for the outlet was the temporary waste weir channel which was excavated to about R.L.1950 (R.L.594.36 m) where good rock foundations were available. Later, it was found that intertrappean layer was met within the foundation between R.L.1950 (594.36 m) and R.L. 1940 (R.L.591.31 m) The foundations of the outlet were therefore, taken below this geru layer down to hard rock. (R.L. 1932 to R.L.1935.0) i.e.(R.L.588.87 m to 589.78 m).

(c) LAYOUT

An R.C.C. tower having 15 feet (4.57 m) internal diameter was provided outside the earth dam. A control cabin was proposed in this intake tower to operate the service gates. The tower was made approachable from the left flank through an approach bridge. A conduit consisting of voussoir-arch (pre-cast cement concrete blocks set in cement mortar) was provided underneath the earth dam section in the diversion channel. (i.e. temporary waste weir cut). The layout for the tower, approach bridge, and conduit is shown in Drawing No.4.

(d) DESCRIPTION OF THE CONDUIT

The outlet was provided with two service gates 5 feet x 8 feet (1.5 m x 2.43 m) each capable of discharging five hundred cusecs (140 cumecs) at one foot (0.3 m) driving head. The regular mode of operation of the outlet was tolet down the discharge through both the openings. Each outlet opening was provided with a service gate and an emergency gate. The conduit was designed to accommodate a penstock of eight feet diameter for the powerhouse to be installed at a future date. Before the installation of the penstock, the conduit was to flow as an open channel. The conduit was 608 feet (185.31 m) long from the centre of the tower and it was given a bed slope of 1 in 400 from the service gate to the centre line of the dam. Beyond that it was given a slope of 1 in 100. The height of the culvert to the crown of the intrados was 11 feet 9 inches (3.58 m). The abutments of the conduit arch were constructed in concrete (cast-in-situ).

(e) CONTACT SEEPAGE

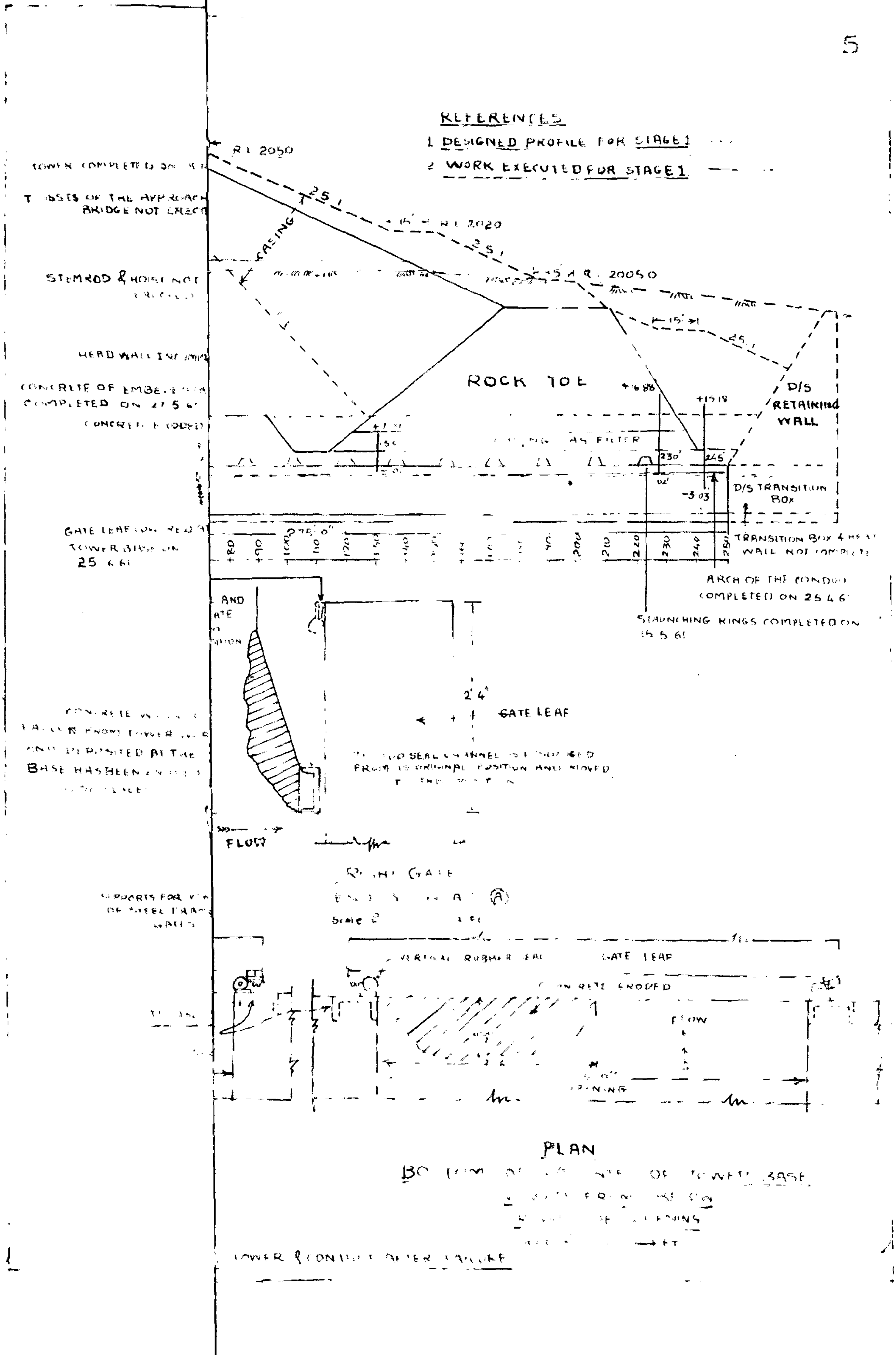
In order to guard against seepage along the surface of the conduit, 21 cut-off rings at 17.5 feet (5.33 m) centres were provided all round the conduit. In addition to the above, a plastic fill at 103 percent of optimum moisture content was laid over the conduit for a depth of five to fifteen feet as shown in Drawing No.5. The plastic fill was raised upon a layer of asphalt over the conduit. This provision was intended to offer additional resistance to seepage of water along the contact path.

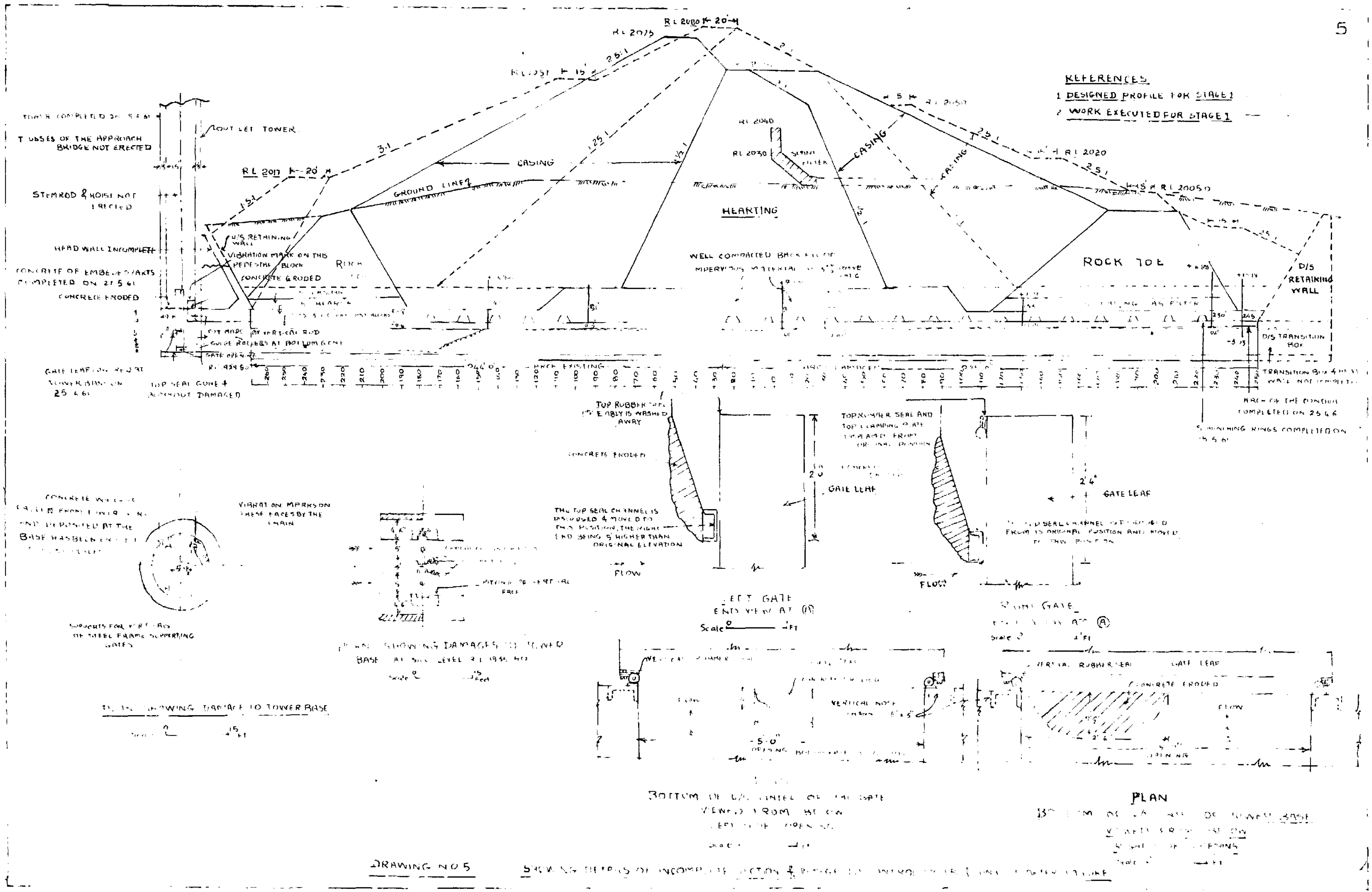
(f) TOWER BASE BOX AND TRANSITION BOXES

The tower base box and transition boxes were in reinforced cement concrete.

(g) UPSTREAM AND DOWNSTREAM RETAINING WALLS

Retaining walls both on the upstream and downstream as shown in Drawing No.5 were proposed for retaining the earth in


REFERENCES
1 DESIGNED PROFILE FOR STAGE 1
2 WORK EXECUTED FOR STAGE 1
R L 2050
CASING
R L 2020
ROCK TOE
D/S RETAINING WALL
D/S TRANSITION BOX
STEMROD & HOIST NOT
HEAD WALL
GATE LEAF
TOWER BASE
TRANSITION BOX
ARCH OF THE CONDUIT
STAUNCHING RINGS COMPLETED ON
2' 4"
GATE LEAF
FLOW
RIGHT GATE
GATE LEAF
FLOW
PLAN


5
REFERENCES
1 DESIGNED PROFILE FOR STAGE I
2 WORK EXECUTED FOR STAGE I
RL 2080
20'
RL 2075
2.5:1
15'
RL 2017
20'
3:1
1.5:1
CASING
GROUND LINE
HEARTING
RL 2040
RL 2030
CASING
5'
RL 2050
2.5:1
RL 2020
RL 2005.0
ROCK TOE
D/S RETAINING WALL
D/S TRANSITION BOX
OUT LET TOWER
STEMROD & HOIST NOT ERECTED
HEAD WALL INCOMPLETE
U/S RETAINING WALL
VIBRATION MARK ON THIS PEDESTAL BLOCK
CONCRETE ERODED
CONCRETE ERODED
ROCK
WELL COMPACTED BACK FILL OF IMPERVIOUS MATERIAL
ARCH EXISTING
TOP SEAL GONE & ALMOST DAMAGED
TOP RUBBER SEAL ASSEMBLY IS WASHED AWAY
CONCRETE ERODED
THE TOP SEAL CHANNEL IS DISLODGED & MOVED TO THIS POSITION, THE RIGHT END BEING 5" HIGHER THAN ORIGINAL ELEVATION
FLOW
GATE LEAF
LEFT GATE
END VIEW AT (A)
TOP RUBBER SEAL AND TOP CLAMPING PLATE
2'4"
GATE LEAF
FLOW
RIGHT GATE
VIBRATION MARKS ON THESE FACES BY THE CHAIN
SUPPORTS FOR VERTICAL OF STEEL FRAME SUPPORTING GATES
SHOWING DAMAGES TO TOWER BASE
DAMAGE TO TOWER BASE
VERTICAL RUBBER SEAL
GATE LEAF
CONCRETE ERODED
FLOW
VERTICAL NOTE
5'-0"
OPENING
BOTTOM OF U/S LINTEL OF THE GATE
VIEWED FROM BELOW
PLAN
DRAWING NO 5

the temporary waste weir channel. The retaining walls were provided directly above the transition box and these were in uncoursed rubble masonry.

(h) SERVICE GATES

The gate was designed with upstream skin plate supported on horizontals which in turn, were supported by end verticals. The sealing was effected by use of a rectangular seal for the bottom and musical note seals for the three sides fixed to the gate leaf. The gate was stem operated and was guided into the slot by guide rollers. A worm and gear type hoist was proposed for the irrigation outlet stage. For the power outlet stage, the gates would be required to be closed, in a very short time and therefore, the hoist would be either hydraulically operated or the gates fitted with retractable seals so that it could be closed under its own weight. The layout of the gate slots and the gate leaf was finalised keeping this in view.

(i) TAIL CHANNEL

The releases from the conduit were ultimately led down into the river proper through the diversion channel which was already constructed.

CHAPTER - 4

CONSTRUCTION PROGRAMME AND ITS ACHIEVEMENT

ORIGINAL PROGRAMME

In the original project report, a tentative dam section with 4:1 side slopes was adopted for estimation of quantities and the dam section was to be finalised after ascertaining the availability of construction materials and their properties. Based on the above assumed cross-section, the total earth work was worked out to be of the order of 124 Mcft (351×10^4 cu.m) for the dam fill and the dam was scheduled for completion in five working seasons, as detailed below:-

S.No	Works	Year	Quantities of excavation/ Embankment as per original project	
			Embankment	Excavation
(1)	(2)	(3)	(4)	(5)
1.	Preliminary work and filling cut-off trench in the gorge required for embankment upto R.L.1990.0 and outlet	One Season	6 Mcft (17×10^4 cum)	Nil
2.	Blocking the natural gorge by embankment having truncated section and excavation of temporary waste weir (i.e. diversion channel)	2nd Season	30 Mcft (85×10^4 cum)	1 Mcft (2.83×10^4 cum for P.W.W. channel & 5.25 Mcft for T.W.W. (14.91×10^4 cum
3.	Completing the truncated section upto R.L.1990.0	3rd Season	30 Mcft (85×10^4 cum)	6 Mcft for P.W (17×10^4 cum)
4.	Raising the embankment to full height except over the temporary waste weir channel	4th Season	30 Mcft (85×10^4 cum)	3.0 Mcft for P.W.W. (8.5×10^4 cum)

(1)	(2)	(3)	(4)	(5)
5.	Blocking the temporary waste weir gorge and completing the embankment & permanent waste weir channel for diverting full floods	5th Season	28 Mcft ($79x10^4$ cum)	2.82 Mcft for P.W.W ($8.00x10^4$ cum)
	Total		124 Mcft ($351x10^4$ cum)	12.82 Mcft for P.W.W & 5.25 Mcft for T.W.W ($36.30x10^4$ cum for P.W.W & $14.91x10^4$ cum for T.W.W

2.0. PROGRESS TILL JUNE 1959

The yearwise progress is shown below:-

(i) YEAR 1957-58

Preliminary works, excavation and back filling of cut-off trench in the river portion and part earth work on the right flank.

(ii) YEAR 1958-59

The work of diversion arrangements i.e. excavation for temporary waste weir channel was completed and the floods were diverted through this diversion arrangement. The work of filling the gorge was completed by construeting a truncated section upto R.L.2008.00 (R.L.612.03 m) (Refer Drawing No.3). The bottom of diversion channel was kept at R.L.1936.24 (R.L.588.98 m)(Please refer to Drawing No.2) and it was designed to take care of full Inglis flood discharge i.e. 48,000 cusecs (1363 cumecs).

3.0. <u>REVISED PROGRAMME</u>

As the construction proceeded much data was available for finalising the earthwork section and accordingly the quantum of earth work involved in building the dam upto R.L.2080 (R.L.634.0 m) reduced to 94.85 Mcft only. Dr. K.L. Rao, Member, Central Water & Power Commission paid a visit to the project in February 1959 and desired that the work should be speeded to complete the dam by 1961. Revised construction programme for completing the dam in four working seasons i.e. by June 1961 was, therefore, framed.

3.1. Here it may be mentioned that by the end of working season of 1959-60, earth work to the extent of 50.85 Mcft (144.41×10^4 cum) has been done, out of the total of 94.85 Mcft (269.37×10^4 cum) thus leaving about 44 Mcft (124.96×10^4 cum) of earth work to be done in the working season of 1960-61. The success of the above target enabling the dam to be brought into operation during the monsoon of 1961 was mainly dependent on the following:-

(i) Completion of the dam. For this the availability of extra earth moving equipment for the increased target of 44 Mcft (124.96×10^4 cum) in one season was essentially required. (the available earth moving equipment was matching for an yearly target of 25 Mcft (71.00×10^4 cum) only)

(ii) Completion of the permanent waste weir channel (i.e. spillway channel)

(iii) Completion of the outlet conduit with control tower and approach bridge including installation of embeded parts, gates, hoisting equipments etc.

3.2. To gear up the project activities, additional staff was sanctioned, foreign exchange was released for additional earth moving equipment and the additional machinery came into operation in December 1960. Monthwise programme was prepared for the above groups but due to the various difficulties the actual progress achieved during each month was slightly behind the schedule.

A statement showing the monthwise programme and the actual progress achieved during each month is given below:-

(I) EARTH WORK

S.No. (1)	Month (2)	As programmed (in Mcft) (3)	As actually executed (in Mcft) (4)
1.	October 1960	2.00 (5.66×10^4 cum)	2.55 (7.21×10^4 cum)
2.	November 1960	4.00 (11.32×10^4 cum)	3.85 (10.89×10^4 cum)
3.	December 1960	4.00 (11.32×10^4 cum)	4.10 (11.6×10^4 cum)
4.	January 1961	7.00 (19.81×10^4 cum)	7.06 (20×10^4 cum)
5.	February 1961	7.00 (19.8×10^4 cum)	6.56 (18.56×10^4 cum)
6.	March 1961	7.00 (19.8×10^4 cum)	5.84 (16.52×10^4 cum)
7.	April 1961	7.00 (19.8×10^4 cum)	5.89 (16.66×10^4 cum)
8.	May 1961	6.00 (16.98×10^4 cum)	5.90 (16.69×10^4 cum)
9.	June 1961	Nil	4.50 (12.73×10^4 cum)
	Total	44.00 Mcft (124.5×10^4 cum)	46.25 Mcft (130.86×10^4 cum)

NOTE:- (i) The monthwise quantities shown in Column 4 were very approximate and the quantities of earth work carried out outside the dam section for roads, ramps etc. were included.

(ii) The actual shortfall of earthwork in the temporary waste weir portion embankment was reported to be 1.5 Mcft (4.24×10^4 cum)

(iii) The actual levels reache-d by June 1961 are shown in Drawing No.2.

(II) PERMANENT WASTE WEIR

As it was decided to keep the permanent waste weir channel at R.L.2062 (R.L. 628.5 m), the quantities contemplated in the original project report were substantially reduced and the estimated quantity for excavation of permanent waste weir channel upto R.L.2062 (R.L. 628.5 m) worked out to 5.15 Mcft (14.57×10^4 cum) only. As against this, some work was already completed and the excavation work to the extent of 3.55 Mcft (13.04×10^4 cum) was only to be planned during the season of 1960-61. The monthwise programme and the actual executed quantities were as under:-

S.No.	Month	As Programmed (in Mcft)	As executed (in Mcft)
1.	June 1960	1.60 (4.52×10^4 cum)	1.60 (4.52×10^4 cum)
2.	July & August 1960	Nil	Nil
3.	September 1960	1.05 (2.97×10^4 cum)	0.35 (0.99×10^4 cum)
4.	October 1960	0.50 (1.41×10^4 cum)	0.30 (0.849×10^4 cum)

S.No.	Month	As programmed (in Mcft)	As executed (in Mcft)
5.	November 1960	0.50 (1.41×10^4 cum)	0.315 (0.89×10^4 cum)
6.	December 1960	0.50 (-do-)	0.345 (0.97×10^4 cum)
7.	January 1961	0.50 (-do-)	0.26 (0.73×10^4 cum)
8.	February 1961	0.50 (-do-)	0.28 (0.79×10^4 cum)
9.	March to July 1961	Nil	Mostly all the excavation work completed by July

(III) OUTLET WORKS		Date of Completion As scheduled	Actual
(a)	Arch Conduit		
	(i) Abutment	By end of March 1961	March 1961
	(ii) Arch (consisting of pre-cast blocks)	-do-	25th April 1961
	(iii) Concreting the bed	-do-	Remained
	(iv) Staunching rings	-do-	15th May 1961
(b)	Tower	-do-	15th June 1961
(c)	Approach Bridge		
	(i) Piers	By end of May 1961	15th June 1961
	(ii) Trusses	-do-	Only two trusses i.e. two spans out of six errected

			Date of Completion As Scheduled	Date of Completion Actual
(d)	Gates			
	(i)	Service Gate	By end of May 1961	Service gate lowered on 6th & 7th June 1961, but emergency gate and hoist for service gate could not be installed
	(ii)	Emergency Gate	-do-	
	(iii)	Hoist	-do-	

3.3. From the above it would be seen that the programmed dates could not be achieved. All the works like construction of arch conduit, R.C.C. tower, errection of embeded parts of the gates etc were concentrated in the short width of 35' (10.66 m) in the temporary waste weir cut and completion of each work was rather dependent on the progress of the other. The following agencies had to work in the restricted space.

(i) The main civil contractor doing the work of transition box, conduit, approach bridge and tower and subsidiary contractor employed to expedite work on the tower.

(ii) Civil Department executing earth work over the arch conduit

(iii) Mechanical department doing errection of the embeded parts for the service gates and emergency gates

3.4. The progress of these critical works also suffered on account of heavy rains in the month of May. The monsoon broke on 20th June 1961 with a long spell of heavy unprecedented rains. The heaviest rainfall occurred on 24th - 25th June 1961 being 12.71 inches in (32.28 cm) a day. It started raining again after three days and continued unabated from 28th June 1961 to 12th July 1961 aggregating to another 57 inches (144.78 cms) in the fifteen days period.

3.4.1. The unprecedented rainfall in the catchment led to rapid filling of the reservoir. The reservoir level shot up by over 30.5 feet (9.29 m) in a day i.e. from R.L.1958.5 (R.L.596.95 m) on 25th June 1961 to R.L.1989.0 (R.L.606.24 m) on 26th June 1961. Thereafter, the level remained steady for four days till 30th June 1961 when it again started rising steadily and went up by another 78.5 feet (23.93 m) to R.L.2067.5 (R.L.630.17 m) in a period of twelve days. The full reservoir level viz. the spillway crest level of R.L.2062 (R.L.628.50 m) was reached on 10th July 1961 when the spillway started running.

3.5. With the break of monsoons on 20th June 1961, and early filling of the reservoir the works came to stand still and the following items remained incomplete.

(I) <u>EARTH WORK</u>

(a) The height of the dam in the temporary waste weir had reached R.L.2075 (R.L.632.46 m) as against the designed height of R.L.2080.0 (R.L.634.0 m)

(b) The level of the hearting in that section had reached R .L. 2064.00 (R.L.629.10 m) as against the designed level of R.L.2072.0 (R.L.631.54 m)

(c) The downstream casing in the same section was at a level of R.L.2064.0 (R.L.629.10 m) only as against the designed level of R.L.2080.0 (R.L.634.0 m)

(d) No pitching could be laid on the upstream face in the temporary waste weir and adjoining portions of the dam.

(II) PERMANENT WASTE WEIR

The excavation of the channel was practically complete but the bed and sides remained very rough because final finishing of the rock surface could not be done.

(III) OUTLET WORKS

(A) - CONDUIT WORKS

(a) The conduit bed could not be lined with concrete beyond the upstream transition box. The bed concrete had been left in two steps of 2' and 1 foot in a distance of 10' (3.048 m) from the transition box. There was a reverse step at the downstream end of the conduit as the tail channel was not proposed to be lined.

(b) The three feet wide (longitudinal) divide wall in the upstream transition box could not be completed. The stream lining of the upstream entry into the conduit was also only partly done.

(c) The upstream retaining wall which was to support the upstream heel of the embankment in the temporary waste weir portion was only partly constructed upto R.L. 1976 (R.L. 602.28 m)

(d) Work could not at all be started on the downstream transition box and the retaining wall meant to support the downstream slopes of the embankment. On this account, the upstream and downstream slopes were made somewhat steeper than provided in the original design, but a substantial rock toe was provided at the foot of the downstream slope.

(e) The outfall channel beyond the downstream end of the conduit could not be excavated properly and was left at a flat slope of about 1 in 400.

(B) GATES AND HOISTS

Gates, hoists and other component parts could not be manufactured and installed in time.

(1) SERVICE GATES

The two service gates were manufactured in the Dapori Workshop (State Government Workshop) and were lowered in the grooves and adjusted to 2 feet (0.6 m) partial opening on the 6th and 7th June 1961. The service gates were slung from the tower by steel wire ropes. For the left gate a pulley block was suspended from a temporary suspension frame of steel sections. The wire ropes of the right gate were tied to wooden sleepers resting on steel channels.

(ii) HOISTS

Stem rods and hoists were received by 12th June 1961 but could not be erected.

(iii) EMERGENCY GATE

This was not ready and could not be erected.

(C) TOWER AND APPROACH

The bridge could not be completed making the tower unapproachable for resetting of the gate during the monsoon.

(D) The trash racks were not fixed; but temporary screening arrangements were made.

CHAPTER - 5

COURSE OF EVENTS LEADING TO FAILURE

Before dealing with the various theories regarding causes of the failure, it would be worth examining the course of events that took place one or two days prior to the breach of the dam and the evidence of data available after the breach of the dam.

2.0. OBSERVED PHENOMENON

(I) EARTH WORK

In the temporary waste weir section the earth work had been carried out upto R.L. 2075.00 (R.L. 632.46 m) and that too in the upstream casing zone as against R.L. 2080.00 (R.L. 634 m). The hearting and downstream casing had been completed to R.L. 2064.0 (R.L. 629.1 m) only. The highest water level reached at the time of failure i.e. overtopping was at R.L. 2067.5 (R.L. 630.17 m) i.e. 3½ feet (1.0 m) above the hearting top and 7½ feet (2.29 m) below the upstream casing.

(II) WAVE WASH

The water level on 10th July had reached R.L. 2062.0 (R.L. 628.50 m). Heavy rains continued in the Panshet catchment and were accompanied by a strong wind. On this day the wind velocity recorded at Central Water & Power Research Station, Khadakwasla was of the order of 53 to 62 km/hour. Upstream pitching was not carried out in the zone of the temporary waste weir section and severe wave action was noticed on the upstream portion of the dam, particularly on the left side between chainage 600 to 1200; which included the temporary waste weir and conduit section. Gunny bags (4800 Nos) and corrugated iron sheets were put on the upstream slope to counteract the wave action and the wave wash was practically brought into control. The height of the waves as observed was about 4 feet to five feet (1.2 to 1.5 m).

(III) PERCOLATION OR LEAKAGE THROUGH THE DOWNSTREAM ROCK TOE ABOVE THE CONDUIT

At about 7.30 p.m. on the 10th July some water emerging through the downstream rock toe about 6 feet (1.8 m) above the top of the conduit arch on its left side was detected by the patrolman. He saw a few stones falling down from the downstream rock toe and heard some noise. The construction engineers saw this flow and estimated the same to be of the order of 5 to 10 cusecs (0.14 to 0.28 cumecs). They also noticed that the colour of the flow was the same as that of the lake water upstream. It was also observed that the flow was constant and neither increased nor decreased in the course of about 36 hours i.e. till overtopping on the 12th of July.

(IV) NATURE OF FLOW IN THE CONDUIT

On 11th July it was noticed that the conduit was flowing intermittently full. The downstream end was sometimes fully submerged and sometimes open when water level in the tail channel was 6" below the crown of the arch. Whenever the water level went down there was a hissing noise as of an escaping air.

(V) SUBSIDENCE

At 2.0 a.m. on 11th July, the overseer noticed that the embankment opposite the tower i.e. above the conduit in a length of about 30 feet (9.14 m) sank to an appreciable degree and reported the matter to the construction engineer. The top of the embankment in this portion was then only 1 foot to 2 feet above the waves, which showed that the initial subsidence was to the extent of about 3 feet (0.9 m). The embankment continued to subside. From about 5.30 a.m. or so, sand bags were laid on the sunken portion with a view to raise the embankment above the level of water. The engineer incharge who observed the sinking operation did not get any indication or had a feeling that the material of the embankment was moving towards the direction of the lake and thought that it was sinking down. (Refer Drawing No.6).

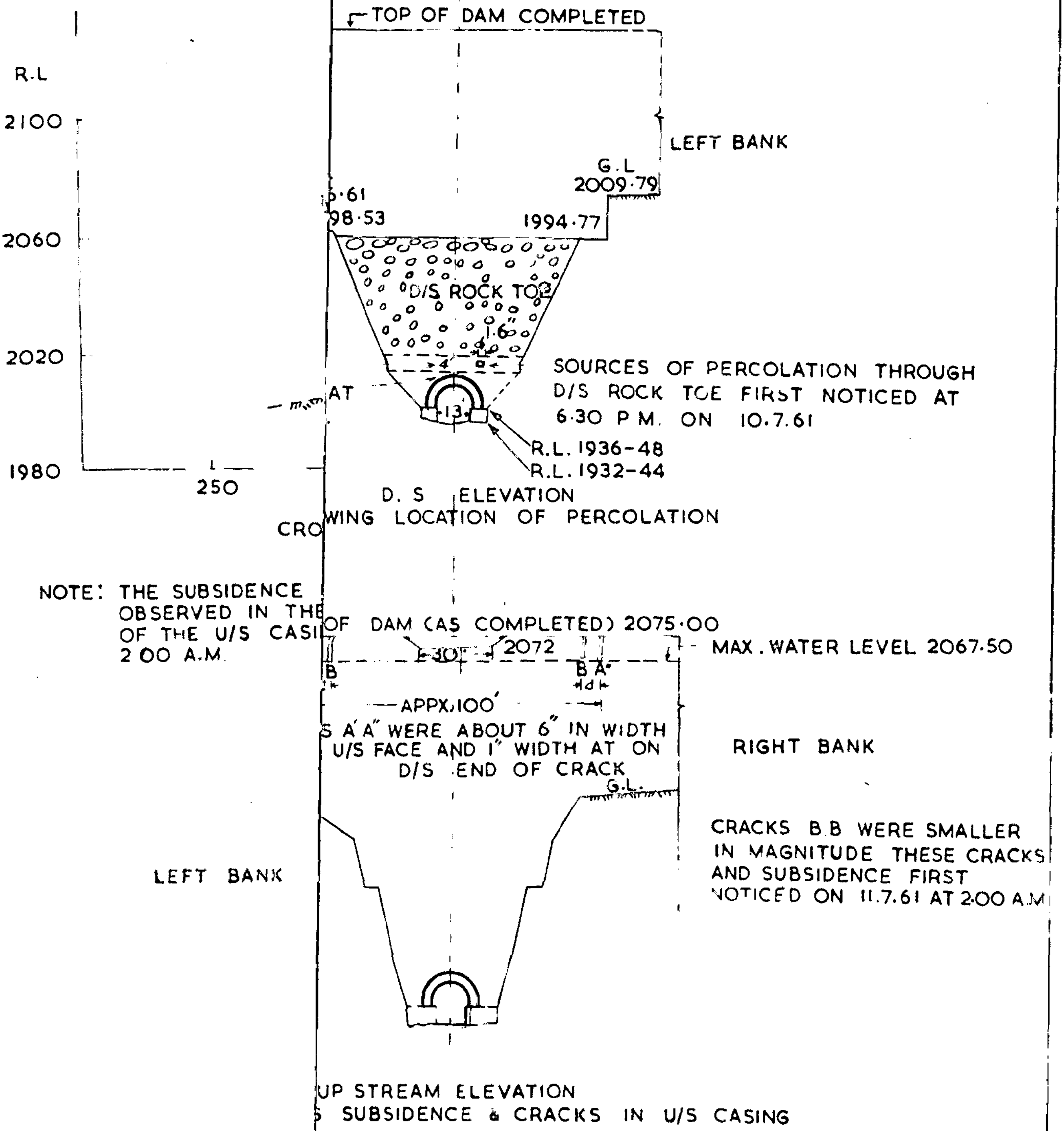
(VI) <u>CRACKS IN THE UPSTREAM AND DOWNSTREAM CASING</u>

Transverse cracks appeared 100 ft. (30.48 m) apart i.e. approximately over the conduit trench edges. These cracks were in half the top width of the casing and were wedge shaped being 1 foot wide (0.3 m) at the upstream edge. Cracks were noticed on 11th morning in the downstream casing also. These cracks were also 100 feet (30.48 m) apart, (i.e. over the edges of the trench) but were thinner being about ½ inch wide. Transverse cracks in the upstream portion were wider on the left side than those on the right side. (Refer Drawing No.6).

(VII) <u>HEARTING AND DOWNSTREAM CASING</u>

Inspection of the dam on 11th July and 12th July showed that the hearting portion was not affected by subsidence and remained intact till the time of overtopping. No slip or scour

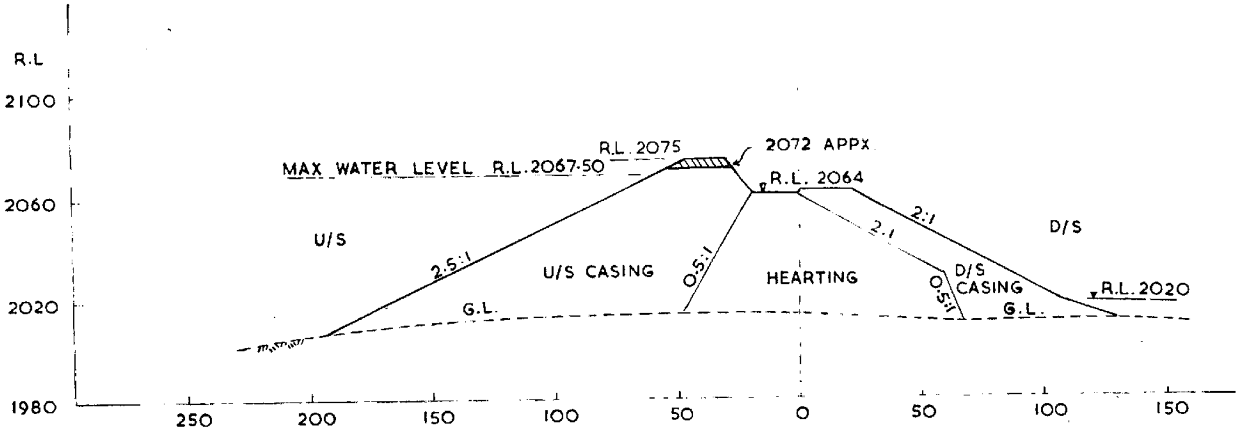
6



TOP OF DAM COMPLETED
R.L
2100
2060
2020
1980
250
LEFT BANK
G.L
2009·79
·61
98·53
1994·77
D/S ROCK TOE
1·6
4
13
AT
SOURCES OF PERCOLATION THROUGH D/S ROCK TOE FIRST NOTICED AT 6·30 P.M. ON 10.7.61
R.L. 1936–48
R.L. 1932–44
D. S ELEVATION
CROWING LOCATION OF PERCOLATION
NOTE: THE SUBSIDENCE OBSERVED IN THE OF THE U/S CASI 2 00 A.M.
OF DAM (AS COMPLETED) 2075·00
30
2072
MAX. WATER LEVEL 2067·50
B
B A
d
APPX. 100'
S A'A" WERE ABOUT 6" IN WIDTH U/S FACE AND 1" WIDTH AT ON D/S END OF CRACK
G.L.
RIGHT BANK
CRACKS B.B WERE SMALLER IN MAGNITUDE THESE CRACKS AND SUBSIDENCE FIRST NOTICED ON 11.7.61 AT 2.00 A.M
LEFT BANK
UP STREAM ELEVATION
S SUBSIDENCE & CRACKS IN U/S CASING

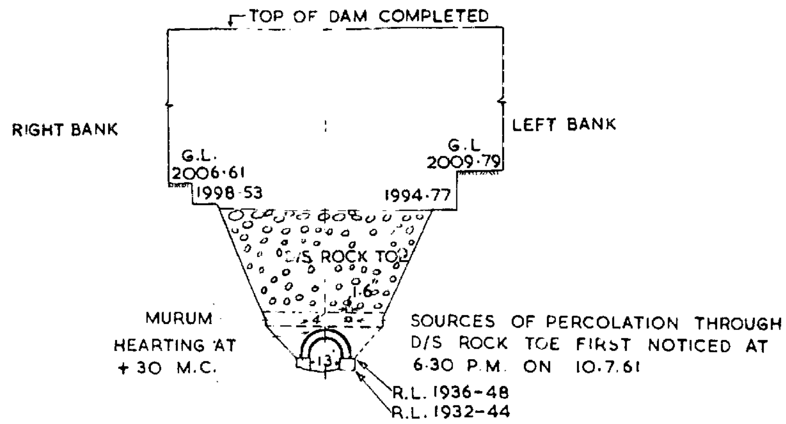
DRAWING SHOWING POSITION OF CRACKS IN T. W. W. PORTION

(OBSERVED ON 11.7.61)

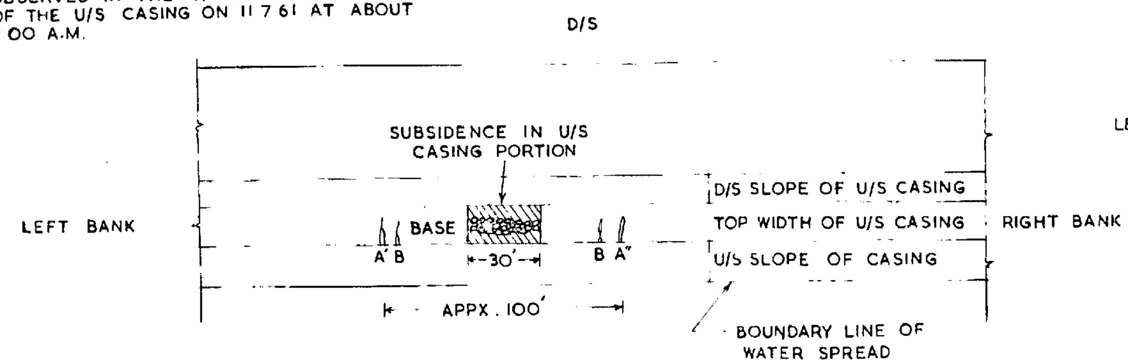


CROSS SECTION OF PANSHET DAM IN THE T.W.W. PORTION
(PORTION BELOW G.L. NOT SHOWN)

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.



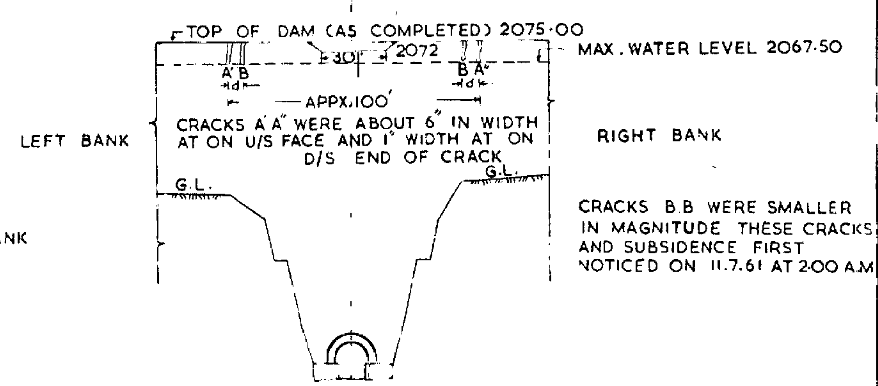
D. S. ELEVATION
SHOWING LOCATION OF PERCOLATION



PLAN SHOWING THE EXTENT AND SHAPE OF CRACKS

NOTE: (a) THE CRACKS AA" BB. WERE WEDGE SHAPED BEING WIDER ON U.S. FACE.
(b) A" CRACK SLIGHTLY SMALLER THAN A'.

0 40 FEET



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

DRAWING. No 3

was seen on the downstream face of the dam besides the small cracks in the casing. No subsidence of the downstream murum slope was noticed either.

(VIII) FEELING OF VIBRATIONS

No vibrations were felt by people working at the top of the dam or on the ground level near the downstream end of the conduit.

(IX) OVERTOPPING

Immediately after overtopping of the dam at 7 a.m. on 12th July, the overflowing water started eroding the dam section over which it cascaded down to the river bed in a mighty fall of about 140' (42.6 m). Chunks of compacted downstream casing and stone pieces from the downstream rock toe fell first and finally the hearting got eroded.

3.0. PHENOMENA NOTICED AFTER THE FAILURE OF THE DAM

(I) CONDUIT ARCH

It was found that out of the total conduit length of 519 feet (158.19 m), 275 feet (83.82 m) length of arch on the downstream side had been washed away completely and the remaining 244 feet (74.37 m) on the upstream side was practically intact. The voussoirs (precast cement concrete blocks) in the last 20 feet (6.09 m) of the standing portion showed signs of distress. Two or three rings of the arch in this portion of 20 feet (6.09 m) have open joints and show slight movement.

Some rock debris and cement concrete blocks were found inside the conduit in the portion which was standing.

(II) ABUTMENTS

The corners of the abutments in the portion where the conduit arch had been washed away were found to have rounded off due to abrasive action of the flow. Damage also occurred by rock fall from the sides of the conduit trench (i.e. T.W.W.)

(III) LEAKAGE THROUGH JOINTS

Some rain water had accumulated in the haunches of the intact portion of the arch conduit and was found to be leaking all along into the conduit through the joints between the skew backs and arch stones.

(IV) PORTION OF THE REMAINING DAM

A longitudinal section of the remaining embankment shown in Drawing No.2 indicates that the old river course has been practically re-established. The front face of the existing portion of the dam was found to have slightly bulged out, obviously as the result of the sudden drawdown of water in the reservoir. Large longitudinal cracks were noticed at the upstream edge. The-se were reported to have appeared after the breach. Pitching was done upto R.L.2060 only (R.L. 627.88 m).

(V) TRANSITION BOX

Cavitation damage has occured in the side walls downstream of the gate opening. There was a big patch of damaged concrete on the roof of the transition box.

(VI) TOWER

(a) Sides of the cement concrete blocks supporting the gantry were found to have worn out by oscillations of the chains.

(b) Some concrete was seen to have fallen out as a result of continued erosion at a number of points.

(VII) CONDITION OF THE GATE SUSPENSIONS & DAMAGE TO STRUCTURE

(A) RIGHT SIDE SERVICE GATE

(i) Upstream skin plate and seals were found damaged.

(ii) Top seal of nose channel was dislodged.

(iii) Bottom guide rollers were sheared off. One roller was found lodged within the gate wheel slot. Guide roller brackets were intact.

(iv) Cavitation damage was seen on the slot base and top seal block-out.

(v) The channel iron block out was torn off from concrete and was found to be abutting against the gate skin plate.

(vi) The stem rods were found lying in the conduit about 120 feet upstream of centre line. One of the stem rod had been dented badly and also bent.

(vii) Heavy erosion in the concrete of the side wall was seen just downstream of the gate slots.

(viii) The sides of the channel downstream of the gate were seen pitted.

(B) LEFT SIDE SERVICE GATE

(i) The divide wall, slot base and top seal block out were seen pitted as a result of cavitation.

(ii) Three guide rollers were found fractured and damaged.

(iii) Nose channel was found dislodged. Top seal was torn off and missing.

(iv) Damage to reinforced concrete was observed as a result of oscillating stem rods.

The downstream bottom flange of the gates was found not to have been cut according to the design. With the gate open of more than 9" (22.86 cms), the top seal went out of action.

(VIII) UPSTREAM RETAINING WALL (DIRECTLY ABOVE THE TRANSITION BOX)

(i) The wall was found to have tilted slightly upstream

(ii) Cracks were noticed in the retaining wall and a crushed stone was noticed.

(iii) Damage was caused at the base of the wall on the downstream side.

(iv) Weep hole pipes were found to have crushed.

(v) Construction joint on the right was found to have been dented.

(IX) CONDUIT ABUTMENT

Pitting was seen on the concrete surface of abutment in the regions where voussoirs had been washed away.

CHAPTER - 6

COMMISSION OF ENQUIRY AND MODEL STUDIES

Immediately after the failure of the Panshet Dam Government of Maharashtra had formulated one man commission for investigating the possible causes of the failure of the dam. Justice V.A. Naik was holding the chair. Reports from various experts regarding the possible causes of the failure were received by the commission. Shri G.N. Pandit, the retired Chief Engineer (Irrigation & Power Department) of Maharashtra State in his report visualised the possibility of the conduit being press urised as a result of higher discharge than was meant to pass under design conditions through the conduit and also as a result of the floor of the conduit not having been concreted. The commission, therefore, directed the Director of the Central Water and Power Research Station, Khadakwasla to reproduce the conditions of the Panshet outlet on the following lines:-

(i) Condition of floor in the arch conduit under high head with a 2' (0.6 m) opening on the prototype gate, the configuration of the jet downstream of the gate and the flow condition in the conduit as it was constructed upto its tail end, should be simulated.

(ii) The model reproduction of the permanent spillway regarding its dimensions and levels relative to the full reservoir level and also intermediate stages of the reservoir should be made.

2.0. Accordingly, a geometrically similar hydraulic model was made to 1/12 scale by the Khadakwasla Research Station.

The model reproduced the entire length from the emergency gate slots upstream to the outfall channel downstream covering a total length of 144' upto its tail. This model was moulded in transperent perspex for flow visualisation. It was laid according to the prebreach surveys and was roughened by pebbles and stone metal. Vibration and pressure pick-ups were fitted for recording gate vibrations and fluctuating pressures in the conduit.

2.1. The model indicated adverse flow conditions in the tower, transition box, and conduit. In each gate slot, a very high velocity vortex was seen to be spinning with considerable entrained air. The vortex created a spiralling motion in the slot and led the flow upwards. For heads above 50 feet (15.34 m) the discharge emerging from the upstream top seal gaps high in the tower and dropping down again through the air vent in the downstream part of the tower, caused intermittent choking of the air vent, giving highly erratic and unpredictable flow conditions.

2.2. The flow through the gate opening was supercritical. Due to the roughness of the b ed and two steps in the conduit bed, hydraulic jump was formed in the conduit itself. The position of the hydraulic jump was found to vary with discharge For low heads the jump formed at the step about 50 feet downstream of the gate and there was free flow in the remaining length of the conduit. As the discharge increased with rise in reservoir level, the jump travelled downstream and became less defined at a head of about 70 feet (21.33 m), when an undulating jump was formed. As the head increased to 100 feet (30.48 m), the undulating jump was transformed into a steeply rising jet intermittently touching the roof of the conduit at about 210 to 220 feet (64 to 67 metres) from the upstream end of the conduit. With still further increase in the discharge, the rising jet suddenly changed into highly turbulant and erratic movement of water to form a hydraulic jump at a distance of about 120 ft (36.57 m) from the upstream end of the conduit for the maximum head of 131 ft (39.92 m). The location of the hydraulic jump was, however, extremely sensitive when sealing of the arch crown occured at the downstream end of the conduit. At the downstream end, where the conduit emerged into the outfall channel, intermittent submergence of the crown of the arch accompanied by hissing noise and spray formation was observed. The model indicated a total discharge of 1370 cusecs (38.36 cumecs) through the conduit at a head of 131 feet (39.32 m).

2.3. PRESSURE FLUCTUATIONS IN THE CONDUIT AND AT THE GATE

When the reservoir water level reached R.L.2067.5 the distribution of pressure was as under:-

(i) Pressures in the gate slots were highly fluctuating and the negative pressure was of the order of - 17.5' (minus 17.5').

(ii) Pressures on the sides of the transition box were slightly negative.

(iii) The pressure in the top seal slot was observe-d to be -6 to -7 feet in the model.

(iv) The pressures in the upper region of the conduit were slightly negative upto the hydraulic jump.

(v) The following pressure variations in the conduit were observed

(a)	At 150 feet upstream of the centre line of the conduit	-(minus) 1' to +13.60' = 14.60'
(b)	At Ch. o	-(minus) 1.01' to +8.00' = 9.00'
(c)	At Ch.245 ft d/s of the centre line	-(minus) 3.03' to +15.15' = 18.18'

The above variations are shown in Drawing No.7.

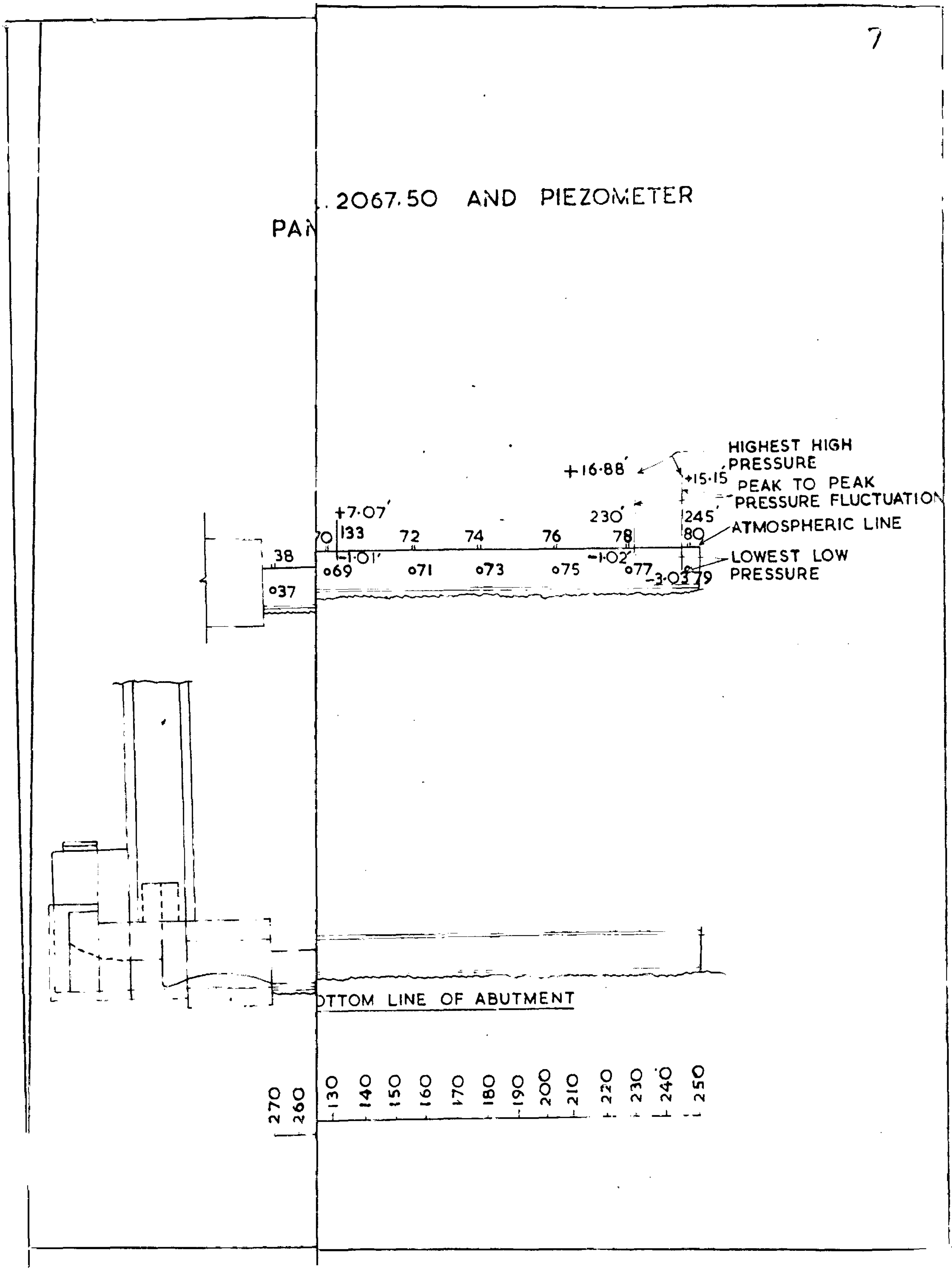
2.4. CONCLUSIONS DRAWN BY THE CENTRAL WATER & POWER RESEARCH STATION, KHADAKWASLA AFTER THE MODEL STUDIES

(i) Through the two foot gate openings and top seal gaps, a combined discharge of 1370 cusecs was observed to pass in the model.

(ii) The test results proved the existance of dangerous cavitation conditions in the seal gaps and the gate slots. The existance of gate vibrations was also established qualitatively, owing to the model limitations. In the absence of test facility of cavitation tunnel, the existance of cavitation could not be proved conclusively in the model tests.

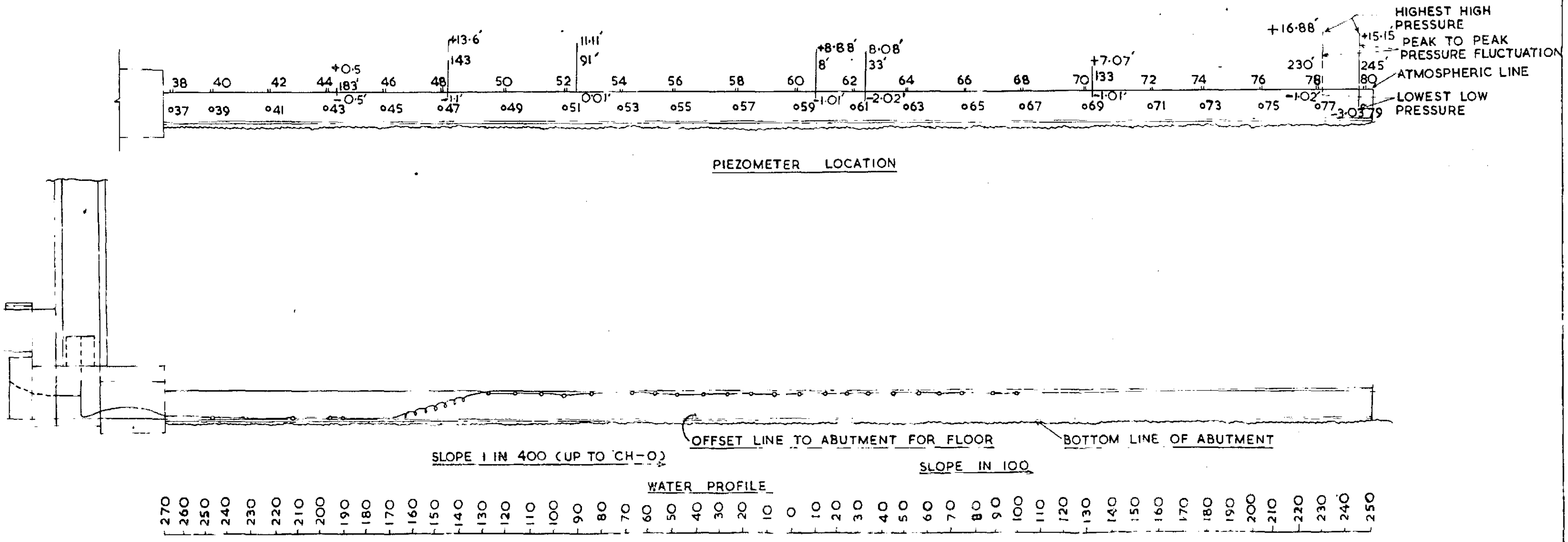
(iii) The adoption of a non-regulatory type design for the service gates should have been avoided. At least, the existing data on the demerits of this type of gate ought to have been weighed carefully before its selection and further improvisations made to remove its defects.

(iv) The existing service gates suffered from faulty top seal


7
. 2067.50 AND PIEZOMETER
PAN
HIGHEST HIGH PRESSURE
+16·88'
+15·15
PEAK TO PEAK PRESSURE FLUCTUATION
+7·07'
230'
245'
ATMOSPHERIC LINE
70
133
72
74
76
78
80
38
-1·01'
-1·02'
LOWEST LOW PRESSURE
o69
o71
o73
o75
o77
-3·03
79
o37
OTTOM LINE OF ABUTMENT
270
260
130
140
150
160
170
180
190
200
210
220
230
240
250

7

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



DRAWING. No. 7

design, severely defective fabrication of lip details and inadequate armour protection to the gate slots and the side of the channel. Cavitation and consequent severe vibrations and pitting were a result of the service gate deficiencies. The gates were also subject to vibrations as they were subjected to underflow and overflow and were hung on improvised elastic suspensions.

(v) The normal design of a separate air vent would have escaped fouling from the seal gap discharge. In the present case, the air vent efficiency was impaired partially and intermittently although a liberal size was provided for in the design. The fouling of the air vent added to unsteady flow conditions at the gate.

(vi) As a result of the higher than design discharge passing through the conduit and non-concreting of the conduit bed to design gradient and surface smoothness, the depth of flow in the conduit built up and a hydraulic jum-p formed inside the conduit. This also resulted in pressurisation of the conduit downstream of the hydraulic jump which was of the order of 10 to 15 feet above hydrostatic limit in the upper parts of the conduit and 15 to 18 feet in the tail portion of the conduit. The conduit arch was thus subjected to highly pulsating pressures.

(vii) The outfall channel downstream was not designed. This should have been done and constructed accordingly. The existing channel combined with the present conduit tended to build up higher water levels.

(viii) Cavitation and consequent vibrations would be highly calamitous forces arising due to the conduit flow. On the other hand pressurisation of the downstream part of the conduit and rapid pressure fluctuations due to turbulent and unsteady flow conditions inside the conduit would also be severely unwarranted forces. To what extent and in what manner these forces contributed to the tragic failure of the dam is beyond the scope of the direct analysis of model studies. It can best be left to rational inferences from hypothesis and conjectures.

2.5. Hydraulic model tests were also done after rectification of the defects and the shortfalls in the construction of the conduit and it was observed.

(i) that the distress on the arch due to turbulant flow conditions and hydraulic jump forming inside the conduit would not have occurred had the bed concreting been done in the conduit.

(ii) The air demand would have been then satisfied from the downstream end of the conduit although this would not have affected the flow conditions at the gate.

(iii) Of all the successive gate openings upto 3.5, the 2 feet gate opening proved very critical so far as high negative pre ssures at and near the gate slots were concerned.

2.6. MODEL STUDIES OF P.W.M.

Discharging capacity of the waste weir channel was found to be lower than designed, viz. against designed discharges of 6800 and 17,200 cusecs at lake levels of 2067.8 and 2072.0, actual capacities were only 3370 and 10,000 cusecs respectively.

It is however, clear that the failure of the dam was not caused as a result of insufficient capacity of the spillway as the depth of water flowing through the spillway was 8.8 feet against the designed flood lift of 10 feet.

LIMITATIONS OF THE MODEL

(i) In the case of open air models, since cavitations conditions cannot be reproduced, the inference about the negative pressures can be only qualitative. The model, however, had shown definite indications that cavitation conditions must have prevailed in the prototype.

(ii) Since rugosity cannot be correctly reproduced it was not possible to assess correctly the volume of discharge. It was, however, certain that the discharge must be higher than the estimated discharge. How much higher is a matter of opinion.

(iii) The model had given definite indications of the build up of high positive pressures in the downstream of the conduit. How high these pressures have been in prototype is a matter of opinion.

CHAPTER - 7

VARIOUS THEORIES ABOUT THE CAUSES OF FAILURE

Before discussing the likely theories about the failure of the Panshet Dam, it would be worth examining the possible causes leading to embankment dam failures. The failure can be due to:-

(I) External Erosion by Flow of Water over the Dam

(a) Due to overtopping caused by inadequate spillway capacity either due to spillway being unable to pass the designed discharge or due to the extraordinary heavy inflow much above that for which the spillway and the dam is designed.

(b) Due to inadequate free board

(c) Due to excessive settlement or subsidence by seismic or other type of vibrations.

(d) Reduction in section as a result of erosion of upstream face by wave action.

(II) Internal Erosion

(a) Piping or seepage through the dam coming out at the downstream slope.

(b) Flow of water through cracks in the embankment caused by shrinkage or differential settlement.

(c) Flow of water along the junction of the dam with foundation or abutments, conduit or other structures embeded in or abutting the dam.

(d) Flow of water along the joint between layers of embankment with different quality of materials or with different degrees of compaction.

(III) Foundation Failure

(a) Due to settlement or heaving of foundation under the load of the dam and reservoir water or due to seismic phenomenon.

(b) Piping through foundation: Boiling or sloughing due to stratification or presence of soft material seams.

(c) Shear failure due to presence of low strength material in the foundation or reduction of effective strength due to transmission of high uplift pressures along previous seams.

(d) Liquefaction of foundation soil due to severe vibratory force of seismic or other origin.

(IV) Slope Failure

(a) Slope failure due to insufficient shear strength of embankment material or under drawdown.

(b) Shear failure along the base or any other plane of weakness.

(V) Conduit

Failure of conduit or other masonry structures embeded into or abutting the dam resulting in collapse of the dam.

(iii) Hearti[illegible]ream casing portion did not show any apparent subsidence and it was intact till the overflow took place. Only small cracks were observed which were 1" to ½" wide.

(iv) Post failure inspection showed that the conduit arch had been washed away completely in a length of 275 feet (83.82 m) on the downstream end only. The conduit in the remaining upstream portion was intact (for a length of 246 feet)(74.37 m).

2.1. If leakage through the embankment is taken into consideration the cause of failure can be contributed to internal erosion (i.e. either due to II(a) or II(c). If the sinking phenomenon is considdred then the cause of failure can be attributed to either

(i) External erosion - may be I(a); I(b); I(c) or I(d)

Or

(ii) Upstream slope failure IV(a).

2.0. Before formulating any conclusion or cause for the failure of the Panshet Dam, one has to take into account the following evidence and then fit the case of the Panshet failure in any of the possible causes listed in para 1 above.

(i) Leakage of 5 to 10 cusecs (0.14 to 0.28 cumecs) was first observed through the downstream rock toe and the leaking spot was at the left side of the conduit 6' (1.8 m) above the extrados of the conduit.

(ii) Sinking of the upstream face was observed on 11th July and sinking was downwards and not towards the lake.

(iii) Hearting portion and the downstream casing portion di(not show any apparent subsidence and it was intact till the overflow took place. Only small cracks were observed which were 1" to ½" wide.

(iv) Post failure inspection showed that the conduit arch had been washed away completely in a length of 275 feet (83.82 m) on the downstream end only. The conduit in the remaining upstream portion was intact (for a length of 244 feet)(74.37 m).

2.1. If leakage through the embankment is taken into consideration the cause of failure can be contributed to internal erosion (i.e. either due to II(a) or II(c). If the sinking phenomenon is considdred then the cause of failure can be attributed to either

(i) External erosion - may be I(a); I(b); I(c) or I(d)

Or

(ii) Upstream slope failure IV(a).

If the cracks in the embankment are considered as the cause of failure, then this could be classified under category II(b) or III(a).

If the failure of the arch on the downstream side is considered as the cause of failure, then this can be classified under category V.

2.2. From the above it would be seen that the problem is rather complex and the cause of the failure could be one or more of the above possibilities.

3.0. From the indisputable evidence available prior to the breach of the Panshet Dam, one fact is clear that the failure was caused by overtopping of the incomplete dam section over the conduit due to subsidence of the upstream casing zone below the maximum water level attained in the reservoir. The cause of the subsidence of the upstream casing is however, obscure and must, therefore, be inferred from the available evidence prior to failure and from the observations of the remaining works after the failure. It would, therefore, be necessary to examine each, possible cause in detail to visualise whether the same is applicable or otherwise.

4.0. DETAILED EXAMINATION OF EACH POSSIBLE CAUSE OF FAILURE

4.1. (I)(a) Inadequacy of Spillway

The spillway of the Panshet Dam was designed on the basis of Inglis Emperical formula and the inflow was worked out to have a peak of 48,000 cusecs (1363 cumecs) with a duration of 7 hours. The spillway discharging capacity was worked out after allowing flood absorption capacity. The peak inflow of 48000 cusecs (1363 cumecs) was assumed to impinge upon a full reservoir spilling with the base flow of 4000 cusecs. The spillway design flood worked out to 17,000 cusecs which the spillway has been designed to handle at the reservoir elevation of 2072 ft. (R.L.631.54 m)

Model tests conducted in the Central Water & Power Research Station, Khadakwasla indicated that the actual spillway capacity of the prototype was likely to be appreciably lower than the design capacity. Moreover, the Inglis Formula is purely emperical and does not provide for any abnormally heavy localised precipitation intensity as is met within the region of the Western Ghats particularly so for small catchments. The inadequate spillway capacity exposed the dam to risk of overtopping. However, as the maximum level reached was only of the order of R.L.2067.5 (R.L. 630.17 m) as compared to the designed level of R.L.2072.0 (R.L.631.54 m) the dam did not fail due to inadequacy of spillway capacity. Similarly the maximum discharge that had passed or calculated to have been passed through the spillway channel was only of the order of 6800 cusecs (193.12 cumecs) as compared to the spillway designed discharge of 17,000 cusecs (482.8 cumecs), it would be clear that the dam did not fail due to extra-ordinary heavy inflow than that for which the spillway and dam was designed.

4.1.1. (I)(b) <u>Due to inadequate Free Board</u>

The designed top of dam was at R.L.2080 (R.L.634.0 m) with the hearting top at R.L.2072.00 (R.L.631.54 m). The maximum flood level anticipated during the peak floods was R.L.2072.00

(R.L. 631.84 m). Except in the temporary waste weir section, the dam had been raised to full height. In the temporary waste weir section the upstream casing could be raised to R.L.2073.0 (R.L. 632.00 m) with hearting at R.L.2064.00 (R.L. 629.1 m) only. The maximum water level in the reservoir recorded just before the breach of the dam was R.L.2067.80 (R.L. 630.17 m). The wave height including ride up reported was of the order of 6 feet (1.8 m). The minimum level of the casing was 2073 (R.L. 632.00 m) and therefore the dam failure cannot be attributed to this cause.

6.1.2. (I)(c) <u>Due to excessive settlement or subsidence by seismic or other type of vibrations.</u>

From the evidence available prior to failure, it is clear that there was a subsidence in the upstream casing. The post inspection of the conduit, tower, gate slots etc. also revealed cavitation damage at such places. The model studies subsequent to the failure also indicated heavy cavitation and vibrations in the gate slot region of the conduit. Dr. B.S. Govindrao, Advisor, Central Board of Irrigation and Power, Dr. D.V. Joglekar, Retired Director, Central Water & Power Research Station and Dr. Wadekar, C.R.O. of Central Water & Power Research Station attributed the failure to excessive vibrations. The detailed reasonings advanced by them would be discussed later.

6.1.3. (I)(d) <u>Saturation in casing as a result of erosion of upstream face by wave action</u>

According to the available evidence, the wave action had been brought under control by putting corrugated galvanised iron sheets etc. The failure, therefore, cannot be attributed to this cause.

4.2. (II) INTERNAL EROSION

4.2.1. II(a) Piping or Seepage Through the Dam coming out at the downstream slope

Dr. K.L. Rao, the then Member (D & R), Central Water & Power Commission thought piping to be the possible cause of failure. The piping visualises that damage first develops at the downstream end of what may be called a pipe in which seepage flow emerges under too steep an exit gradient for the type of soil. Such a condition gets aggrevated due to the enlargement of the pipe by gradual leaching of the material, leading ultimately to subsidence of the dam above the pipe and its failure.

At Panshet, a flow of 5 to 10 cusecs had appeared suddenly through the downstream rock toe about 6' above the crown of the conduit which was constantly kept under observation. This flow of 5 to 10 cusecs (0.14 to 0.28 cumecs) remained constant and did not increase. As such, flow through the downstream rock toe would need some other cogent explanation and the dam could not have failed due to conventional piping.

4.2.2. (II)(b) Flow of Water through cracks in the embankment caused by shrinkage or a differential settlement

On 11th July, some small cracks were noticed both in the upstream and downstream casing, but no cracks were seen in the hearting zone. There is no evidence of water coming out through these cracks. The dam failure cannot, therefore, be attributed to this cause.

4.2.3. (II)(c) <u>Flow of water along the junction of the dam with foundation or abutments, conduit or other structures embeded in or abutting the dam</u>

The constant flow of 5 to 10 cusecs (0.14 to 0.28 cumecs) noticed at the downstream end could be attributed to this cause. Shri G.C. Dhanak, Ex-Chief Engineer, Gujarat State and Shri G.N. Pandit, Retired Chief Engineer, Maharashtra State and many others attributed this leakage to be the cause of the failure.

4.2.4. (II)(d) <u>Flow of water along the joint between layers of embankment with different quality of materials or with different degrees of compaction</u>

From the site eivdence, the cause can be ruled out.

4.3. (III) <u>FOUNDATION FAILURE</u> <u>(a) to (d)</u>

The foundations were sound and the positive cut-off had been taken into solid compact basalt. There is no other evidence indicating possibility of a failure. Hence it can be ruled out.

4.4. (IV) <u>SLOPE FAILURE</u>

Normally for steady seepage condition, the downstream slope is critical, From all available evidence, the stability of the downstream slope was never in danger. It was in the upstream zone that subsidence was noticed which could lead to a doubt about the stability of the upstream slope.

Stability analysis of the dam carried out by the slip circle method indicates that the dam section even in the temporary waste weir portion was stable under all normal conditions.

The minimum factor of safety for the upstream slope under submerged conditions, according to the computations worked out to 1.55 which gave an adequate factor of safety. There was no drawdown of the reservoir before the failure of the dam but even for the full draw down condition, the upstream slope in the weakest section (i.e. temporary waste weir portion) gave a factor of safety of 1.27 The failure of the dam could not therefore, have occured due to this cause.

4.5. (V) CONDUIT

Shri S.B. Joshi on behalf of the Institution of Engineers Poona and Bombay Centre, had put forth the hypothesis of the collapse of the conduit arch due to overstress caused by load of embankment above it. The design criteria adopted for the conduit was examined in detail by Shri A.C. Mitra, Assessor to the Commission of Enquiry with Shri Joshi. They concluded that although the arch could not have collapsed on this account but there could be no doubt that joints (between the two voussoirs) would open out at points of high tension and mortor would be crushed at points of high compressive stress and that the effect would be most marked in the central portion of the conduit where the embankment was highest (i.e. approximately in the portion of the conduit from 200 ft to 300 ft (60 to 90 metres) from the end of the upstream transition box). The dam could not therefore, have failed due to collapse of the conduit arch under superimposed load. The commission also agreed to this view of the Assessor.

4.6. From the above it would be seen that the probable cause of failure could be attributed to one or more of the following:-

(i) Cavitation and vibration

(ii) Internal Erosion

5.0. Views expressed by the following experts in regard to the detailed mechanics of failure will now be discussed:-

(1) Prof. N.S. Govindrao, Advisor, Central Board of Irrigation & Power

(2) Dr. D.V. Joglekar, Retired Director of Central Water & Power Research Station, Khadakwasla

(3) Dr. Wadekar, C.R.O. Incharge of High Head Section, Central Water & Power Research Station, Khadakwasla

(4) Shri G.N. Pandit, Retired Chief Engineer (I & P) Maharastra State

(5) Shri G.G. Dhanak, Ex-Chief Engineer, Gujarat State

(6) Shri A.C. Mitra & Shri C.L. Handa, Assessors of the Enquiry Committee.

5.1.1. Views of Prof. Govindrao

Referring to the model studies conducted in the Central Water & Power Research Station, Khadakwasla, he thought that vibrations had been set up in the conduit due to successive pressurization and de-pressurisation. There was also evidence that

(i) Top seals of the gates were destroyed

(ii) Connection to a bottom guide roller had sheared off

(iii) Concrete block-outs had been damaged.

These damages indicated that cavitation conditions prevailed in the gate region which must have caused increased vibrations in the conduit portion. Due to this cavitation condition and entrainment of air, a number of cavitation bubbles must have travelled along the flow from the vicinity of the gate into the conduit. These bubbles originating in local regions of low pressure moved into regions of high pressure, collapsed in the body flow; causing stress waves in all directions. In short severe vibrations were caused in the conduit due to the following reasons:-

(i) Due to air entrainment and release in the conduit

(ii) Due to oscillatory nature of flow of the hydraulic jump

(iii) Due to cavitation bubbles collapsing in the high pressure regions of flow.

The vibrations so caused in his opinion had components in the horizontal plane along the longitudinal axis of the conduit, along a transverse axis at right angles to the longitudinal axis and along the vertical axis perpendicular to the other axis.

5.1.2. Prof. Govindrao while explaining the effects of vibrations on the conduit divided its length into the following four parts:-

(a) Length of the conduit below the hearting zone.

(b) Length of the conduit under hearting zone

(c) Length of the conduit between the commencement of the hydraulic jump and the hearting.

(d) Length of the conduit between the transition box and the commencement of the hydraulic jump.

(a) Length of the conduit below the hearting zone

The longitudinal component of the stress waves and consequent high pulsating pressures should have resulted in shear stress waves especially at the crown and also at other joints of the arch. Many of the twenty thousand joints between the voussoirs which were parallel to the longitudinal axis of the conduit should have been subjected to sliding or shear stresses in between them. This should have resulted in the abrasion of mortar, as they were ground, and powdered. As the powdered mortar fell off, the joints should have offered less and less resistance to flow through them in the vertical direction. The transverse components of the stress waves should have achieved the same purpose in the direction of the innumerable transverse joints. The vertical components of the stress wave might have further aggravated this abrasive action on the mortar.

With every hissing sound, there was an upsurge. Water should have pierced through the joints in arch (in the form of spray) into the clay shroud and entered the filter zone. When the upsurge pressure lasting for a moment fell down, the water which could have tried to re-enter the conduit found an easier path to escape downstream through the rock too. This leakage through the rock too became constant when the lake reached its maximum level.

(b) Length of the Conduit under Hearting Zone

Similar weakening of the voussoir joints had also taken place resulting in water to spurt into the joints every time when there was an upsurge. During the low pressures, the water should have tried to flow back to the conduit, carrying along with it some of the clay of the hearting material. No doubt this phenomenon must have caused some internal erosion and local sloughing causing subsidence of the hearting. But before any serious consequences in this length could take place, much more serious action had already taken place in the upper reaches.

(c) Length of the Conduit between the commencement of the hydraulic jump and the hearting

This portion was lying in the region where the vibrations due to air entrainment and release, started the amplitude of the vibrations. It was also closer to the centres of cavitation damage on solid boundaries like gates, sill etc. As before the innumerable joints of the voussoirs got loosened. The water jet spray due to momentary upsurge got into the murum soil of the casing, after piercing through the clay shrouding. A certain amount of clay from the clay shrouding might have gone into the conduit through the joints. A part of the jet spray got mixed with the clay shrouding due to shaking given by the vibrations and its resistance to deformation got weakened considerably. The same thing had happened to the murum in the casing zone. The casing portion above the clay shrouding should have got mixed with water more thoroughly than the clay. Due to constant vibrations, with every upsurge, almost synchronous with the hissing noise heard, water spray jet had shot through the murum. The vibration components in the transverse and longitudinal direction further helped this process. This weakened its shearing resistance very considerably. It is also well known that, in gritty soils like murum, there is likelihood of increase of pore pressures when subjected to vibration causing reduction of strength. Thus a phenomenon similar to liquifaction might have taken place.

(d) Length of the conduit between the junction of the transition box and the conduit and the commencement of the hydraulic jump

The cavitation conditions in this reach had effected in creation of stress waves only.

5.1.3. Upstream Casing Subsidence

Due to the phenomenon explained in para 5.1.2. (c) above, subsidence of the material of the upstream section of the bund started as the bottom layers were unable to sustain the weight. The longitudinal component of the vibrations had shaken this semi-solid mass violently. The clay portion forming the conduit shrouding and the clay of the hearting had resisted the impact and movement of this semi-solid murum mass. Thus the only direction in which it could flow was, towards the rock toe. A flow was set up with the result that some of the water-murum material was pushed by the vibrations towards the rock toe on the upstream giving it a kick. The longitudinal, the transverse and the vertical components of the vibrations specially from the cavitation damage had their effect in disloding the loose stones with which it was built. This together with the kick it received from the semi-solid murum mass, every time, there was an upsurge resulted in the rock toe flattening out. This in turn, resulted in the sliding of the rock toe and consequent bulging of the bund on the upstream side resulting in subsidence.

5.1.4. <u>Washing away of part length of the arch</u>

After the dam was overtopped, there should have been a gush of mud slurry consisting of a mixture of water and earth dam material moving down the rear slope dragging with it the stones of the rock toe. These stones might have heaped up at the downstream end of the conduit thus obstructing passage for the water passing through the conduit. The arch might have, therefore, been subjected to full hydrostatic pressure equivalent to reservoir head and the arch stones might have been pushed away one by one, along with the flowing water. As soon as the uplift was stopped (i.e. after all the stones collected at the D/S end of the conduit were dragged away) the removal of arch stones might have been stopped.

5.2. <u>Views of Dr. Joglekar</u>

(a) <u>Downstream Leakage</u>

Considering the limitations of the model, it was to be expected that in the prototype, a discharge of 1500 to 1600 cusecs might have passed through the conduit. The flow must have filled the conduit intermittently with entrained air which led to vibrations and pressure fluctuations. The high pressure head at the downstream end of the conduit, probably might have removed some of the mortar in the joints of the arch stones because at the D/S end of the conduit the overburden was very much less. These open joints admitted water from inside the conduit to flow outside and along the conduit and this water might have appeared at the downstream side.

(b) Upstream Subsidence

Due to severe vibrations, cracks appeared in the upstream murum zone and settlement began over the 30 feet width and in line of the conduit alignment. The vibrations in the three elements viz the gates, the tower and the conduit must have synchronised and led to the dislocation of the rock too. The sliding of the rock toe which was holding the upstream filling, had naturally led to continued subsidence of the upstream murum zone.

(c) Washing away of part length of the conduit

After overtopping, the arch might have been uplifted and caused the removal of the part arch.

5.3. Views of Dr. Wadekar

Dr. Wadekar does not subscribe to Dr. Joglekar's view that the rock toe could be disturbed as a result of vibrations set up in the transition box and the retaining wall. According to him the possibility of the retaining wall being disturbed was not more than 25%. He described the mechanism of failure as under.

The vibrations would create stress waves which would be transmitted to clay. If there is a momentary rise in pore pressure during the transmission of the stress waves, "flow" becomes the only means of relieving the stress thus produced. The flow combined with the already reduced shear strength, would lead to a progressive flattening of the slope starting from bottom upwards. A large scale sloughing of the slope might have started when cavitation vibrations assumed alarming proportions under high head-s which increased gradually as the vibratory loads continued. Observation of subsidence exactly over the 30 feet width of the conduit though apparently baffling, was perhaps due to transmission of the vibrations to the earth slopes. Yielding occured in the form of gradual sloughing of the slope which was enough for the dam to be overtopped.

The appearance of the flow at the downstream end of the conduit has been explained to be due to disintegration of mortar joints under alternating compression and tension waves and spurting out of water through these joints.

5.4. Views of Shri G.N. Pandit

Shri Pandit's evidence before the Commission had taken place prior to the model studies. He predicted the adverse flow conditions in the conduit and opined that pressurisation and severe vibrations might have prevailed in the conduit. The dynamic forces created and accompanied by alternate suction and pressure might have led to disintegration of mortar in the joints of voussoir which may or may not have resulted ultimately to the disloding of the voussoir . Sufficient material would have been sucked in through the open joints of the voussoir blocks to form a cavity within the impervious zone above the arch. This cavity would progressively extend in an oblique direction towards the upstream face of the impervious zone. The cavity would remain within the impervious zone by arch action in the cohesive material. Within the murum zone however no arch action was possible and the material for this zone would continue to be drawn in the cavity in the impervious zone causing a progressive collapse of the murum zone.

The leakage at the downstream too was explained as due to spurting out of the water under pressure through open joints.

5.5. Views of Shri G.G. Dhanak

Referring to the test results of the model experiments and considering the limitations of the model, he opined that the pressure fluctuations in the prototype might have been perhaps of a higher order. The conduit was of course, not designed to run full bore or for any internal fluctuating pressures because full design discharge would have been obtained by smooth hypercritical flow confined well below the springing of the arch. The effect of the internal fluctuating pressure on the arch must have been to shake the arch rings from within and since the frequency of such pressure was high, there would be continuous hammering under the arch ring beginning from almost the 7th July, when the conduit started running full bore to the 12th July. With these very large internal impacts superimposed on the high frequency vibrations to which the entire system was subjected, there must have been considerable amount of fatigue in the mortar resulting in loss of much of its strength. In addition, perhaps due to the disintegration of the mortar, water must have surged past the joints disrupted the asphaltic layer and reached plastic clay fill. The repeated application of water pressure and its reversal must have washed out some clay whenever the joints opened to an adequate magnitude. This could have happened anywhere along the downstream half of the conduit where the formation of standing wave and positive pressures are indicated in the model. The local washing out of the clay from joint to joint could tend to establish at places a continuous path along the extrados of the arch and the leakage of such joints in or ahead of the rock toe must have appeared as 5 to 10 cusecs flow steadily passing along the downstream face of the rock toe. This washing out of the clay must have resulted into unbalanced earth pressure on the arch. This coupled with the loss of mortar strength and the impacts of internal fluctuating water pressure must have thrown the arch to a condition which would lead to formation of local opening probably at more than one place. Into these small openings that were created in the arch, water must have surged up and down a number of times, washing out each time some quantity of clay which progressively increased the size of the cavity. Since the material was plastic and the direction of the maximum load was upwards, the skin of the cavity must have gradually fallen out progressively towards the top. Why exactly it should have travelled to the upstream edge of the top of dam is highly speculative. At this point, the physical evidence lends support to this conception of failure. In the downstream slope of murum, two parallel cracks were observed which must have been due to the loss of support from underneath. This progressive formation of the cavity and washing out of the clay was relatively a slow process, because of the heavy compaction of the clay.

When the water was cascading down not only from the tower side but also from the other two arms of the U formed by the conduit cut, it must have fallen with a terrific impact over the conduit and washed away its entire downstream part which was already shaken. The upstream half of the arch still remained intact probably because this arch was not weakened by internal impacts and the water cascade did not act on this part for a long period as the breach must have swung down to the river bed before water had an opportunity to destroy the remaining part of the arch.

Shri Dhanak while admitting that there would be intense vibrations, pointed out that the vibrations could not be of such a severe character, and of such amplitude as to cause movement of the rock toe.

5.6. Assessor's Opinion about the mechanism of failure

(i) The voussoir joints of the conduit arch were overstressed in the central portions of the conduit leading to crushing of mortar at points of high compressive stress and opening out of joints at points of high tensile stress.

(ii) The vibrations at the gate structure were transmitted to the portion of the conduit arch and its intensity was severe in the portion adjacent to the transition box. Vibration would have developed around 2nd July and increased progressively with the rise in reservoir level.

(iii) These vibrations caused readjustment in the backfill adjacent to the conduit arch by consolidation of the 5 ft thick layer of comparatively lower density clayey material placed at consistency of 3 percent over optimum moisture content in haunches and immediately over the arch.

(iv) The readjustment of material in the backfill accompanied by some loss of material underlying the rock toe into the conduit through partial opening of joints in this reach, caused consequential settlement leading, in its turn, to development of cracks and openings in the superposed casing material in the region adjacent to rock toe. These cracks permitted ingress of reservoir water under pressure upto the wet consistency material underlying the hearting zone.

(v) The roughness of the unconcreted conduit floor gave rise to pressurised and pulsating flow. The fluctuating outward pressure rendered the stability of voussoirs vulnurable against outward radial thrust and caused some of these to be displaced outwards leading to formation of openings due to widening of joints.

(vi) Such openings provided passage for inflow hearting material locally due to tiny jets working in alternating spurts as a result of persistent surging action of pressurised water.

(vii) The openings formed by the loss of hearting material got interconnected above the conduit surface to form a continuous path from the upstream face of the dam into the conduit through the displaced voussoir joints. Leakage of reservoir water through the passage tended to erode out finer material from the upstream casing zone. The casing material tended to flow out more easily since the little cohesion it had under placement condition, would be practically lost on soaking and cracking in lower layers due to vibration. The size of the continuous channel increased gradually as more and more material was eroded and flowed into the conduit without being initially observed at the downstream toe. Later, when the path had enlarged sufficiently due to progressively increasing erosion, and the resulting discharge could not pass through the openings in the conduit, it started making way downstre-am until it got access to the downstream rock toe through which it flowed freely and where it deposited the coarser suspended material. Thus the leakage of 5 to 10 cusecs appeared suddently on 10th July. By this time, considerable internal damage had already occured to the dam by way of loss of material from the upstream casing zone into the conduit. The upstream slope ultimately started sinking from the top when the uppe-r layers could not stand any longer due to loss of material from the lower layers. The subsidence could not be made up by dumping earth filled bags since leaking water was carrying more material than could be resorted by dumping. The subsidence led to overtopping of the dam.

(viii) After overtopping started and overflow of reservoir water got into its stride, chunks of compacted embankment material and stones from rock toe must have been eroded due to the force of gushing water and deposited in front of the conduit exit, leading to its getting temporarily choked. Failure of the conduit arch by bursting in the downstream portion, where the embankment had been washed away, resulted because of high internal pressures due to choking of the exit.

To summarise, the failure of the Panshet Dam has been ascribed by them to a combination of the following causes and circumstances:-

(i) Unfavourable hydraulic flow conditions (pressurisation and surging) in the conduit.

(ii) Malfunctioning of the gate control equipment resulting in severe vibrations and cavitation.

(iii) Overstressing of the conduit arch of voussoir construction and consequent opening out of joints.

(iv) Incompleteness of the section of earth embankment in respect of top level of hearting and casing in the temp. W.W. section where overtopping took place.

(v) Extraordinary adverse meteorological conditions which led to extremely rapid filling of the reservoir. This prevented internal adjustments and self-healing of the dam.

CHAPTER - 8

FINDINGS OF THE COMMISSION

1.1. From the various views regarding the cause of the failure already discussed in Chapter 7, it would be seen that there is a considerable difference of opinion about the cause of the failure. The commission of enquiry expressed the following views:-

> "Judged by any standard, the theory which I am subscribing to, is more sound than the one-sided views put forward by the protagonists of different theories, each of whom is emphasizing the importance of the breach of study to which he has devoted his life. I do not, however, claim any originality for my theory. I have only tried to take a whole and integrated view and combined only the most rational elements in some of these theories, which appear to me to be sound."

1.2. VIEWS OF THE COMMISSION REGARDING THE CAUSE OF THE FAILURE

Cavitation had occurred and as a result of which intense vibrations had been set up in the conduit. The main reason why cavitation conditions arose in the transition box and the region of the gates, was the partial opening of two feet in both the gates which were not fit to be used as regulatory gates. Such gates were suitable for full opening or full closure. Had the gates been completely closed, no flow would have entered inside the conduit and the whole mischief would have been avoided. The other factor regarding the gates was that they were kept suspended on wire ropes and not fixed in a proper way. There was also a defect in the manufacture of the bottom flange with the result that the jet would alternatively cling and spring out clear. When the gate was raised more than nine inches, the sealing effect of the top seal was nullified and water gushed in through the opening, fouling the air vent and creating an additional discharge. Negative pressures were created in the region of the gates. The conduit also had been pressurised and conditions akin to water hammer had occurred in the downstream portion of the conduit. Vibrations had been intensified as a result of the fluctuating flow inside the conduit. Stress waves must have radiated from these vibrations. It is difficult to say whether these stress waves led to unsettlement of the plastic fill above the conduit bringing down the upper layers of casing in the process of flow. That possibility, however, cannot be completely ruled out.

The commission, was however, inclined to the view that some of the joints between the voussoirs must have opened out on account of vibrations due to internal pressures superimposed upon cavitation vibrations. The water spurting out from these joints at different places could create cavities in the clay. The cavities might have grown bigger and joined each other as the process of leaking of water continued. The Commission, however, thought that the process thus set up would take a long time for the formation of a pipe for the passage of water. It was more inclined to the view that there could be thorough shaking of the voussoir stones by the combined effect of cavitation vibrations, internal fluctuating pressures, joints opening out and widening by the continuous spurt of water. The stones of the arch, which was already overstressed, might have moved or shaken or thrown off. A few stones could have been hammered out above the normal outer surface of the arch. On account of the above impact, wedge action of the arch was lost and when this happened, a stone or two could fall down and then some more stones in that area could also fall down in the conduit. A microscopic movement of one stone in the outer direction would make the adjacent stones free from constraint. Once a stone was removed from the constraint of the thrust of the linear arch, it could be pushed up by the outward radial thrust of the pressurised and pulsating flow. When one stone had thus moved out, the small opening would have become bigger on account of further washing out of the clay from behind the opening and on account of the internal fluctuating pressures, the stones would have fallen down into the conduit. This would create local openings in the arch rings. The location of the occurrence of the phenomenon of one or two stones falling down could be, in the first instance, somewhere below the downstream of the centre line of the arch. The process of displacement of the voussoirs would progress upwards and the pressurised spurts gushing through the opened joints would create cavities. These cavities would get inter-connected, thus creating a widening passage for the loss of soil material. The process described above would lead to subsidence of the upstream casing. The dislodgement of the stones would also lead to a flow of water which would appear at the downstream end through the rock toe. These two phenomenon viz., the flow at the downstream end of the conduit and the subsidence of the upstream casing, were the effects of the same cause. At the same time, the two phenomenon might not necessarily be connected with one another nor was it ncessary that they should occur simultaneously.

The weakening of the arch and the actual falling down of some of the voussoirs would make the arch flexible with the result that either as a result of uplift pressure developed at the downstream end, or as a result of overflow after the dam was overtopped, the downstream portion of the arch could be washed away. Although, in a sense, the washing away of the downstream portion of the arch, whether it had been due to uplift pressure or whether it was a result of cross flow was an independent phenomenon and was not necessarily connected with the failure of the dam and had in fact taken place after the overtopping. Still, the commission had grave doubts as to whether the arch could have been washed away merely as a result of uplift or cross flow unless the arch had been thoroughly shaken, weakened and fallen down at places. In other words, the displacement and removal of a few vourssoirs, was the starting point of the process of the failure of the dam. The cause of the displacement and removal of the stones was the combined action of cavitation vibrations and pressurisation of the conduit.

PART - II

RENOVATION SCHEME

CHAPTER - 9

STUDY OF ALTERNATIVE SCHEMES OF RECONSTRUCTION

1.0. INTRODUCTION

After the breach of the Panshet and Khadakwasla Dams on 12th July 1961, Government of Maharashtra, while appointing a one man commission to go into the causes of failure of those dams directed that until the commission's findings were available, no reconstruction works on both the dams should be undertaken.

1.1. There was lot of public criticism regarding the advisability of constructing embankment dams in high rainfall areas like Panshet and Warasgaon. As the Panshet and Khadakwasla storages had ceased to exist, there was a good opportunity to review all previous proposals regarding building of storages in the Mutha Valley. Accordingly the Government of Maharashtra, appointed a high level committee to study the complete development of the Mutha Valley water resources available at the existing Khadakwasla Dam site and to recommend suitable storage schemes. After preliminary examination the committee considered the following to be the possible alternative schemes:-

Scheme I - Construction of a new dam at Khadakwasla

Scheme II - Construction of a new dam near village Malkhed i.e. (in between Panshet and Khadakwasla Dam sites)

Scheme III - Strengthening of the existing Khadakwasla Dam and construction of two storage dams one at Panshet and the other at Warasgaon.

1.2. Due to the breach of the Khadakwasla Dam, the water supply to the Poona City and its Cantonment, non-irrigation supplies to the industrial area around Poona, perennial irrigation on the existing Mutha Right Bank Canal upto mile 12 and seasonal irrigation upto mile 30 was seriously affected. The domestic and industrial water supply was temporarily restored by pumping water from Mulshi dam in a neighbouring valley.

The committee after scrutiny and preliminary investigations of Scheme II for which investigations had not been made earlier, finally recommended Scheme III for adoption. While making their recommendations the committee had in mind the very high cost of providing pumped water supply to Poona City and Cantonment from Mulshi Dam and that the adoption of Schemes I and II would necessitate this arrangement to be continued over a much longer period.

2.0. PROPOSAL FOR BUILDING STORAGE AT PANSHET

Once the scheme for constructing two independent dams at Panshet and Warasgaon was finalised by the committee, the following two alternatives for the Panshet site were examined:-

(i) Whether an entirely new dam near the old site would be preferable.

or

(ii) Whether the existing dam could be reconstructed at the existing site utilizing the remaining dam and other works as much as possible.

2.1. Naturally, from economic considerations one had to go for the proposal (ii) above if found technically sound. The quantity of earth work which was standing still was of the order of 59.65 Mcft costing approximately Rs.1.2 crores. Besides, there were several other existing structures such as outlet tower, approach bridge, piers etc which could be made use of. However, before deciding on the adoption of the renovation scheme it was necessary to:-

(a) examine the condition of the site after the failure

(b) establish the reliability of the standing earth dam portion.

2.2. <u>CONDITION OF THE SITE AFTER THE FAILURE</u>

2.2.1. <u>EARTH DAM</u>

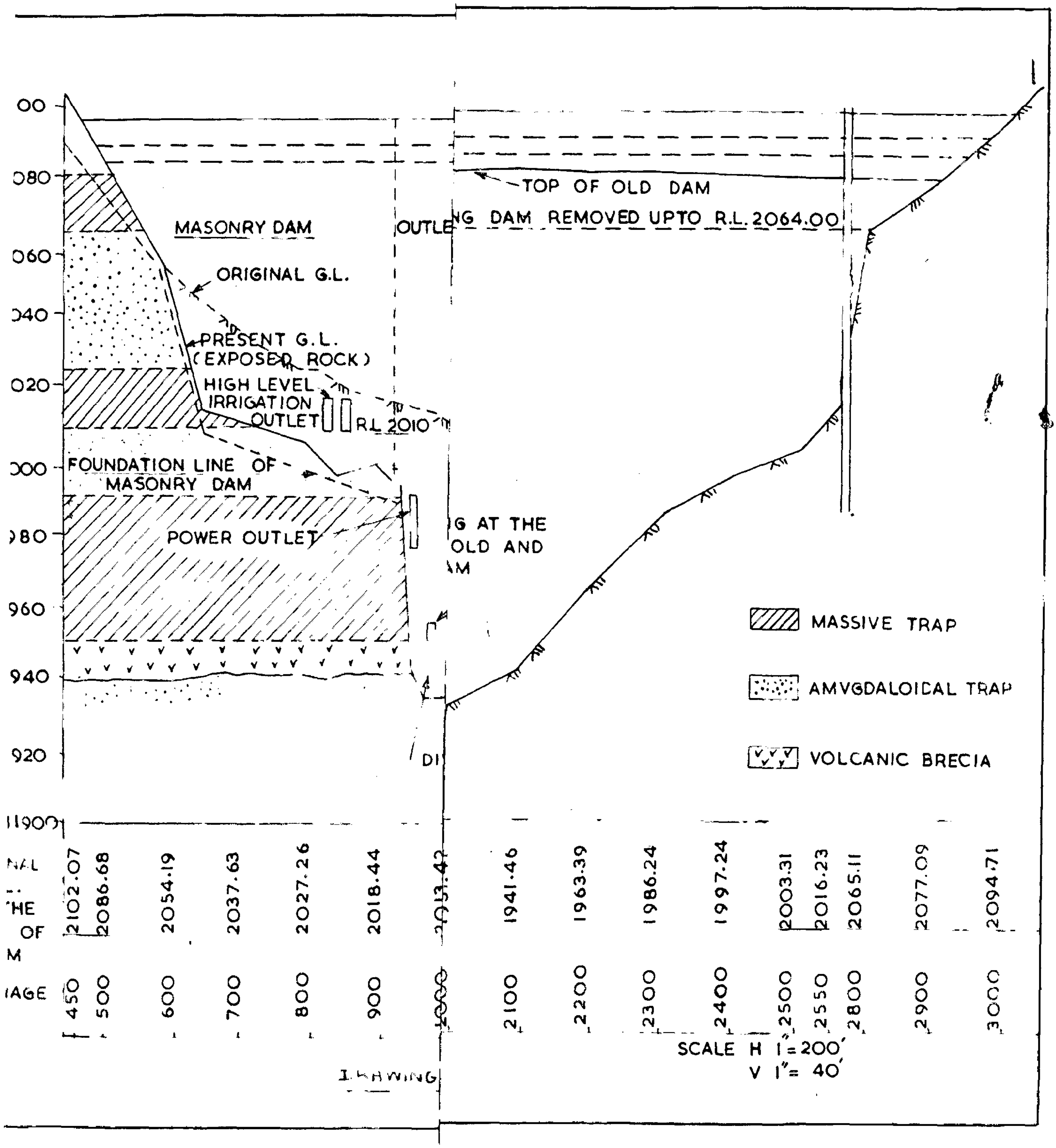
(i) The earth work of the dam between Ch.700 to 1600 was washed away leaving nearly a vertical face at Ch.1600 as shown in Drawing No.8.

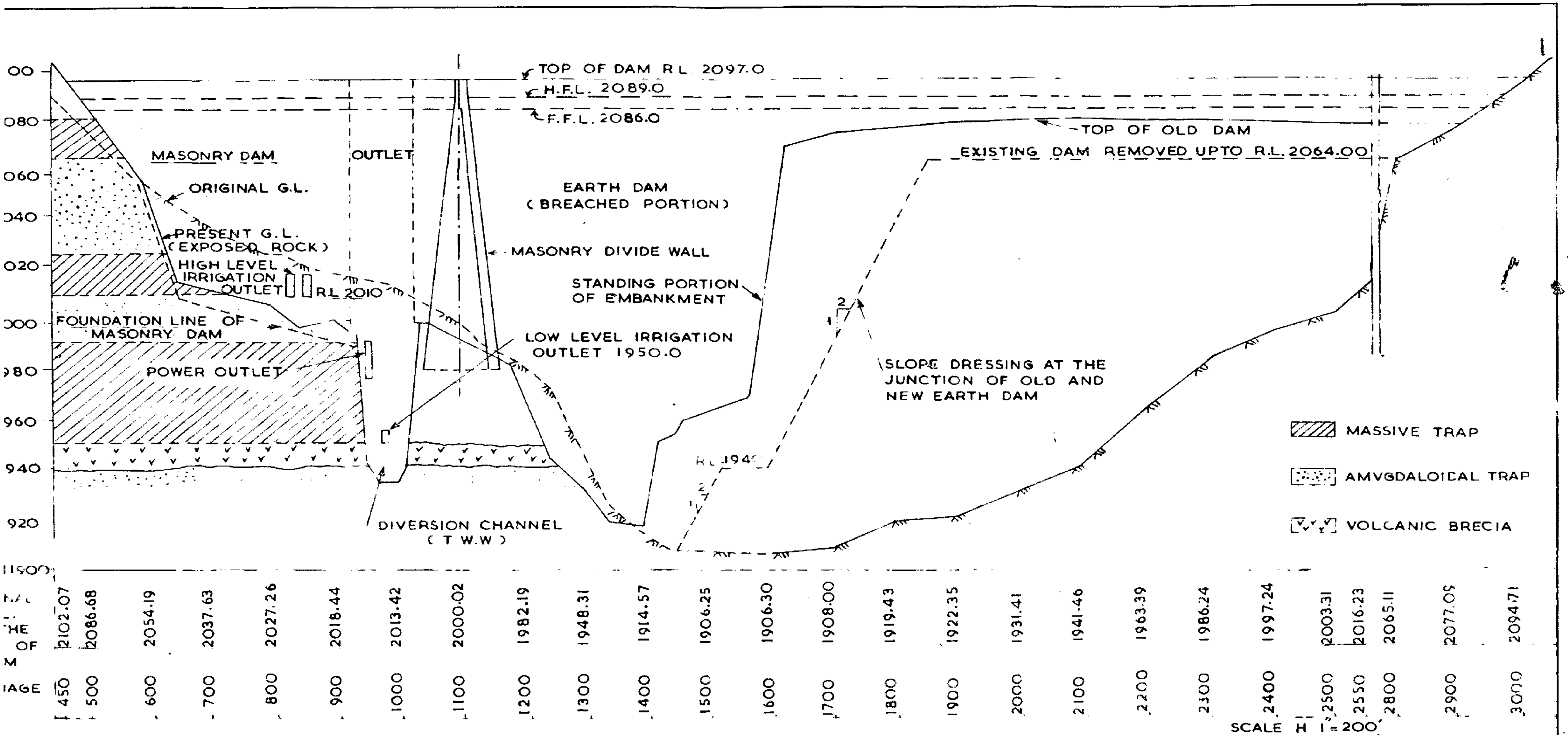
(ii) All the overburden of soil and murum on the left flank had been washed away by the cascading flow after the breach leaving exposed bare rock (including the portion between the waste weir cut and the natural river course)

(iii) The natural river course was re-established.

(iv) The upstream casing zone near Ch.1600 had slightly bulged and slushy material was lying in the natural river course.

(v) Some longitudinal cracks were observed at the top of dam.


TOP OF OLD DAM
MASONRY DAM
OUTLE
NG DAM REMOVED UPTO R.L. 2064.00
ORIGINAL G.L.
PRESENT G.L.
(EXPOSED ROCK)
HIGH LEVEL
IRRIGATION
OUTLET
R.L. 2010
FOUNDATION LINE OF
MASONRY DAM
POWER OUTLET
G AT THE
OLD AND
AM
MASSIVE TRAP
AMYGDALOIDAL TRAP
VOLCANIC BRECIA
2102.07
2086.68
2054.19
2037.63
2027.26
2018.44
2013.42
1941.46
1963.39
1986.24
1997.24
2003.31
2016.23
2065.11
2077.09
2094.71
450
500
600
700
800
900
1000
2100
2200
2300
2400
2500
2550
2800
2900
3000
SCALE H 1"=200'
V 1"= 40'



DRAWING No 6. LONGITUDINAL SECTION OF PANSHET DAM

(vi) The rock on the downstream was disturbed in some portions even beyond Ch.1600

(vii) The tower, and the conduit arch for a length of 240 ft on the upstream side remained in tact.

(viii) Except for the few cracks mentioned in (v) above, the remaining earth work of about 59.03 Mcft was intact and appeared serviceable.

2.2.3. SPILLWAY SITE

A deep gulley was noticed in the tail channel after functioning of the spillway and this gulley was due to the volcanic breccia layer met with at R.L.2060.00.

2.3. INVESTIGATIONS FOR ESTABLISHING RELIABILITY OF THE STANDING EARTHWORK

2.3.1. Though record tests were available, it was considered necessary to take undisturbed samples from the standing embankment to ascertain the in-situ characteristics. Undisturbed samples from bores at Ch.1900; 2100 and 2500 were collected by the Central Water & Power Research Station, Khadakwasla and the following tests were conducted:-

(I) Atterberg's limits

(II) In-situ moisture content and dry density

(III) Undrained triaxial tests for the hearting material.

The results of these test samples are given hereunder:-

(I) Atterberg's Limit - near about 50

Plastic Limit - near about 30

Plasticity index - near about 20

(ii) & (iii) In-situ moisture contents, dry densities and effective stress parameters for a few samples:-

Chainage	Insitu moisture in %	Dry density in lbs/cft	Effective Stress (Undrained test)		
			Cohesion in p.s.f.	Angle of internal friction	
				Degrees	tan Ø
2100	27.7	102.7(1650 kg/cum)	1440(70 kg/sq.cm)	30	0.57
2300	(i) 29.5	102.3(1640 kg/cum)	1584(0.77 kg/sq.cm)	28	0.53
	(ii) 26.4	100.8(1610 kg/cum)	1296(0.63 kg/sq.cm)	32	0.62
1900	28.4	94.7(1520 kg/cum)	1728(0.84 kg/sq.cm)	40	0.83
	28.6	96.2(1540 kg/cum)	576(0.28 kg/sq.cm)	34	0.67

The designed values were as under:-

Dry Density 95 lbs/cft (1520 kg/cum)

C = 600 lbs/sft (0.29 kg/sq.cm)

tan Ø = 0.35 i.e. Ø = 19° - 30'

Thus the in-situ characteristics were found to compare very favourably with those adopted for design.

2.3.2. To assess in-situ permeability of both casing and hearting zones of the existing embankment, bore hole permeability tests were taken for the casing material according to the procedure laid down, under designation E-18 of the U.S.B.R. Earth Manual.

The results of the permeability tests of the casing were found to be as under:-

Chainage	Location	Permeability
2100	On U/S berm at R.L.2057	10,600 ft/year (0.01 cm/sec)
2300	-do-	8,200 ft/year (0.007 cm/sec)
2400	-do-	13,200 ft/year (0.013 cm/sec)

The above test results indicated that the casing zone was sufficiently pervious to justify the design assumptions made regarding the draw-down pore pressures in the upstream casing zone in the stability analysis.

Similarly, after collecting the undisturbed samples from the hearting zone at Ch.2100, permeability test was carried out by the C.W.P.R.S. and it was found that the permeability of the hearting material was of the order of 4 ft/year (3.8×10^{-6} cm/sec). This quite clearly showed that the hearting material available from the standing embankment was impervious and could be relied upon as a "core".

2.2.3. To ascertain the depth of the longitudinal cracks which were noticed at the top of dam after the breach, Several trial pits were taken. It was noticed that the cracks did not extend below 8 feet (2.4 m) depth. The width of the cracks was varying from ½" to 1" which pinched off where it met the hearting zone at a depth of 8' (2.4 m). The orientation of the cracks was nearly vertical. These cracks were entirely local and no movement or slides were noticed both on the upstream and downstream slopes. Even the pitching on the upstream of the standing embankment had remained intact. The embankment was therefore considered usable after stripping the top 16 feet.

2.3. After the above examination and investigations, it was decided to adopt the scheme of renovation of the existing dam.

3.0. <u>ALTERNATIVE SCHEMES FOR RENOVATION</u>

3.1. The following aspects need to be considered for accepting the suitability of layout:-

(i) The junction between the old embankment and the new work

(ii) Location of outlet

(iii) Diversion arrangement

(iv) Peak work load during one season

(v) Creation of partial storages so as to enable the dam to be sub-jected to gradual filling each year.

3.1.1. <u>Junction of old and New Earth Work</u>

Obviously the existing embankment should be stripped back to such a slope as to admit proper compaction at the junction with normal compaction procedure. After examining the practice adopted elsewhere and discussion with U.S.B.R. engineers it was decided to strip the dam face to a slope of 2 H to 1 V and to provide additional keys in the hearting zone so as to obtain a proper bond between old and new earth work.

3.1.2. Location of Outlet

The only suitable and economical location for outlet is in the temporary waste weir cut which was adopted.

3.1.3. Diversion Arrangement

The old arrangement of diverting the floods during monsoon (i.e. through the temporary waste weir cut having its bed at R.L.1935 (R.L.589.88 m)) was considered most suitable and was adopted.

3.2. ALTERNATIVE LAYOUTS

The following likely alternative layout proposals were examined from the point of view of their suitability of keeping the peak load within reasonable limits and the requirement of admitting gradual filling of the reservoir:-

Proposal I

Utilization of all the existing structure except the arch conduit, provision of a R.C.C. box culvert for the irrigation outlet with masonry back filling in the cut upto El.1955 and setting back of slopes of the cut to 3 H to 1 V for proper earth compaction and restoration of the earth dam.

Proposal II

Same as Proposal No.I with masonry back filling upto the top of the cut viz. El.2000. (no setting back of the slopes of cut required).

Proposal III

Same as Proposal No.I with masonry back filling in the cut upto El.1970 and setting back of slopes above El.1970.

Proposal IV

(1) Provision of a gravity non-overflow masonry dam on the left flank and over the T.W.W. cut (with all outlets in the body of the dam). The masonry dam to continue for a length of about 80 feet (24 metres) on the right flank.

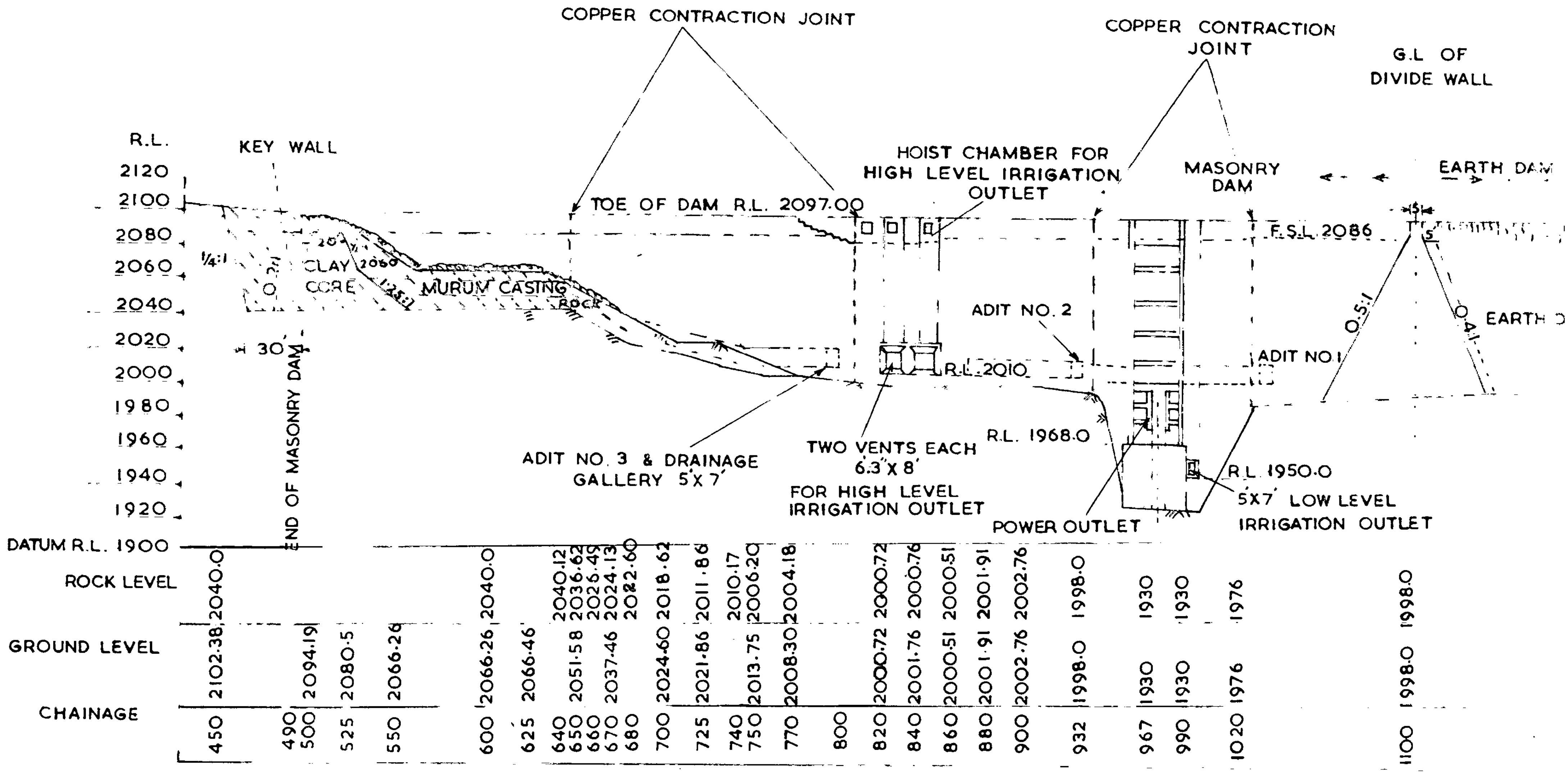
(2) Provision of a divide wall at the junction of the masonry and the earth dams.

(3) Restoration of the earth dam on the right flank beyond the divide wall (See Drawing Nos. 9 & 10).

3.2.1. The relative merits of these proposals will now be discussed.

Proposal I

The programme of works in this layout would have been to divert the floods of the 1st and 2nd season through the temporary waste weir cut i.e. over the masonry back filling at R.L.1958.00 (R.L.595.88 m). In the third working season the backfilling of the T.W.W. cut would be required to be completed to the top (El.2097) to pass the floods through the P.W.W. (the bed of which is at R.L.2062.0). Thus the dam would be subjected to practically the full hydrostatic head of 107' in one season which as already stated, was not considered desirable from the point of view of differential settlements of the old and the new embankments and the joint. This proposal was consequently found unacceptable.



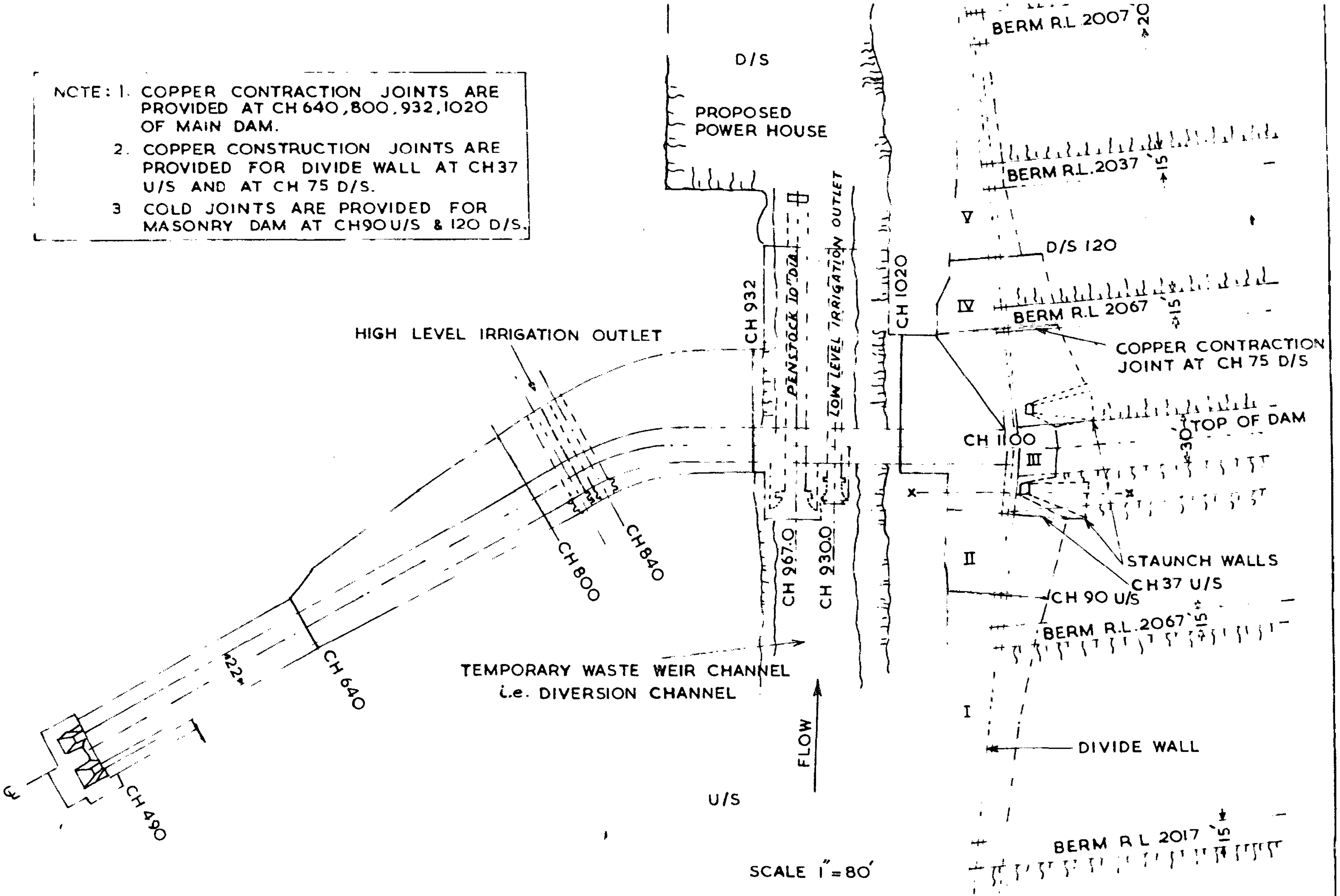
LONGITUDINAL SECTION FOR MASONRY DAM

DRAWING. No. 9

NOTE: 1. COPPER CONTRACTION JOINTS ARE PROVIDED AT CH 640, 800, 932, 1020 OF MAIN DAM.

2. COPPER CONSTRUCTION JOINTS ARE PROVIDED FOR DIVIDE WALL AT CH 37 U/S AND AT CH 75 D/S.

3 COLD JOINTS ARE PROVIDED FOR MASONRY DAM AT CH90 U/S & 120 D/S.



PLAN FOR MASONRY DAM AND DIVIDE WALL

DRAWING No 10

Proposal II

In this proposal, the diversion arrangements could be one of the following:-

(i) Do the masonry filling upto R.L.1970 in the temporary waste weir cut in one season and pass the flood over it and complete the rest of the masonry and the earth to a safe height to pass the flood in the P.W.W. channel.

(ii) Complete the entire masonry in the cut and the dam to a safe height for passing the flood in the P.W.W. channel in one season.

The alternative (ii) in effect is similar to Proposal III which will be discussed later.

In the alternative (ii), the quantity of masonry involved between R.L.1955 (R.L.549.88 m) to R.L.2000 (R.L.609.6 m) would be of the order of 2.5 Mcft (7×10^6 cum). Such a huge quantity of masonry would take 6 to 7 months for completion due to space restriction. The execution of earth work involved above R.L.2000.00 (R.L.609.6 m) in the T.W.W. portion being of the order of 18 Mcft (80×10^6 cum) was not easy to accomplish in the remaining 2 month period and involved an element of risk. The scheme was therefore not considered feasible.

Proposal III

In this proposal the T.W.W. cut until filled with masonry upto El.1970 can be used to pass the flood waters. Thereafter, it would be necessary to complete the dam to a safe height for passing floods through the P.H.H. channel in one season. The only objection in this layout was that the junction between the old earth and new earth work would have been subjected to a hydrostatic head of 92' (28.04 m) (R.L.2062-1970) at one go which was not considered safe enough. This proposal, though superior to I and II, was consequently rejected.

Proposal IV

Though in this layout the tower, approach bridge piers, trash racks etc. could not be utilized, it was considered most acceptable from the following considerations:-

(a) Earth work quarries available nearby were already consumed during the construction of the dam prior to the year 1961, thus the required quantity of material for the entire earth dam especially the hearting material was not available within reasonable leads. Therefore, any proposal involving reduction in the quantities of earth work was preferable.

(b) This layout permitted gradual raising of the storage which was considered very desirable from safety considerations.

(c) Partial storages during the construction period would be created which could be utilized to develop irrigation simultaneously.

(d) The masonry dam in the temporary waste weir cut provided considerable flexibility to the construction programme for the earth work. Depending upon the progress of the work in the earth dam portion, the raising of the masonry monolith in the temporary waste weir portion could be easily adjusted to suit diversion requirements.

This layout was, therefore, finally decided upon for adoption.

CHAPTER - 10

DETAILED FEATURES OF RECONSTRUCTION SCHEME

1.0. STORAGE

In order to utilise the 75% reliable yield at the Khadakwasla Dam site of about 37,000 Mcft, the high level committee appointed to suggest the most suitable scheme of development of Mutha Valley recommended creation of the following storages:-

(i) Existing Khadakwasla Dam = 3000 Mcft

(ii) Panshet Dam = 10500 Mcft

(iii) Warasgaon Dam = 13000 Mcft

The existing Khadakwasla Dam has been strengthened and rebuilt where it was breached, for storing the same quantum of water. The Panshet Dam as constructed was designed for a live storage of 7000 Mcft at R.L.2062.0 (top of dam R.L.2080.00). The reconstruction scheme takes into account the increased storage requirement of 10,500 Mcft. The dam will have a storage of 7000 Mcft upto the spillway crest level. The full storage of 10,500 Mcft would be available when gates are installed as demand for water increases. The control levels for the dam as built in 1961 for its renovation are given hereunder:-

S.No.	Details	Control Levels for the dam as built in 1961	Control Levels for renovated dam after its final develoment
1.	River Bed	R.L.1900	R.L.1900
2.	Irrigation outlet sill	R.L.1935	R.L.1950
3.	Low water level	R.L.1950	R.L.1950
4.	F.S.L.	R.L.2062	R.L.2086
5.	H.F.L.	R.L.2072	R.L.2089

6.	Top of Dam	R.L.2080	R.L.2097
7.	Type of Waste weir	Open Channel 178' wide with bed at R.L.2062.00	Ogee crest at R.L. 2078.0 with four taintor gates 40'x14'
8.	Live Storage	7000 Mcft	10,800 Mcft
9.	Gross storage	7200 Mcft	10,800 Mcft

2.0. <u>WORKS INVOLVED IN RENOVATION</u>

The renovation of the Panshet Dam involved the following works:

(I) <u>Earth Dam</u>

(i) Construction of a new earth dam between Ch.1100 and 1600.

(ii) Strengthening and raising of the existing earth dam between Ch.1600 to 3000.

(II) <u>Masonry Dam & Divide Wall</u>

(i) Construction of a masonry dam between Ch.1100 to the left abutment (Chainage 490)

(ii) Construction of a masonry divide wall at the junction of the earth and masonry dams.

(III) <u>Spillway</u>

(i) Widening the tail channel from 178 ft to 186 ft to suit the revised requirements

(ii) Protection of the tail channel (against retrogression due to the volcanic breccia layer)

These works have been discussed in the subsequent paragraphs.

3.0. EARTH DAM

3.1. ZONING AND DAM DETAILS

The section adopted for the new earth work consists of the following zones. (Refer Drawing No.11).

(i) Hearting

Top width 10' with its top at R.L.2093.00

U.S. slope 1:1 and D/S slope ½:1

(ii) Filter

On the downstream side of the hearting, a filter drain (7' thick sand followed by 5' thick gravel) has been provided from El.2086 to 2' above ground level.

(iii) Semi-Pervious

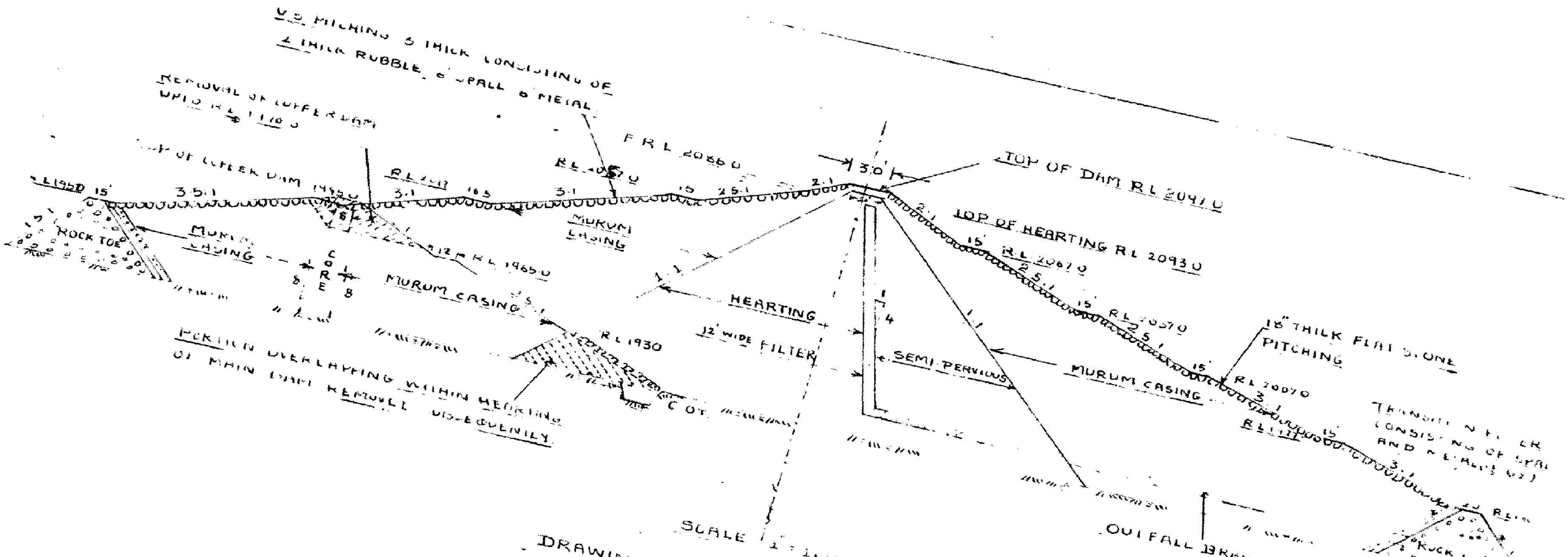
The filter drain is followed by the semipervious zone (randum fill). The soil used in this zone has coarser texture and better shear strength. The D/S slope of this zone is 1:1.

(iv) Upstream and D/S Casing

Both the hearting and the semi-pervious zone is protected by the pervious casing consisting of good hard murum. The upstream slopes are varying from 2.5:1 to 3.5:1 and the D/S slopes are varying from 2:1 to 3:1 as shown in Drawing No.11.

(v) Rock Toe

Rock toes of height equal to h/6 (where h = head of water on the upstream) have been provided. The inside face of the rock toes has been provided with quarry



DRAWING NO 11 : SECTION OF EARTH DAM IN BREACHED PORTION spalls and 1½" to 2" stone metal to act as transition filter between the casing and rock toe.

(vi) U/S Slope Pitching

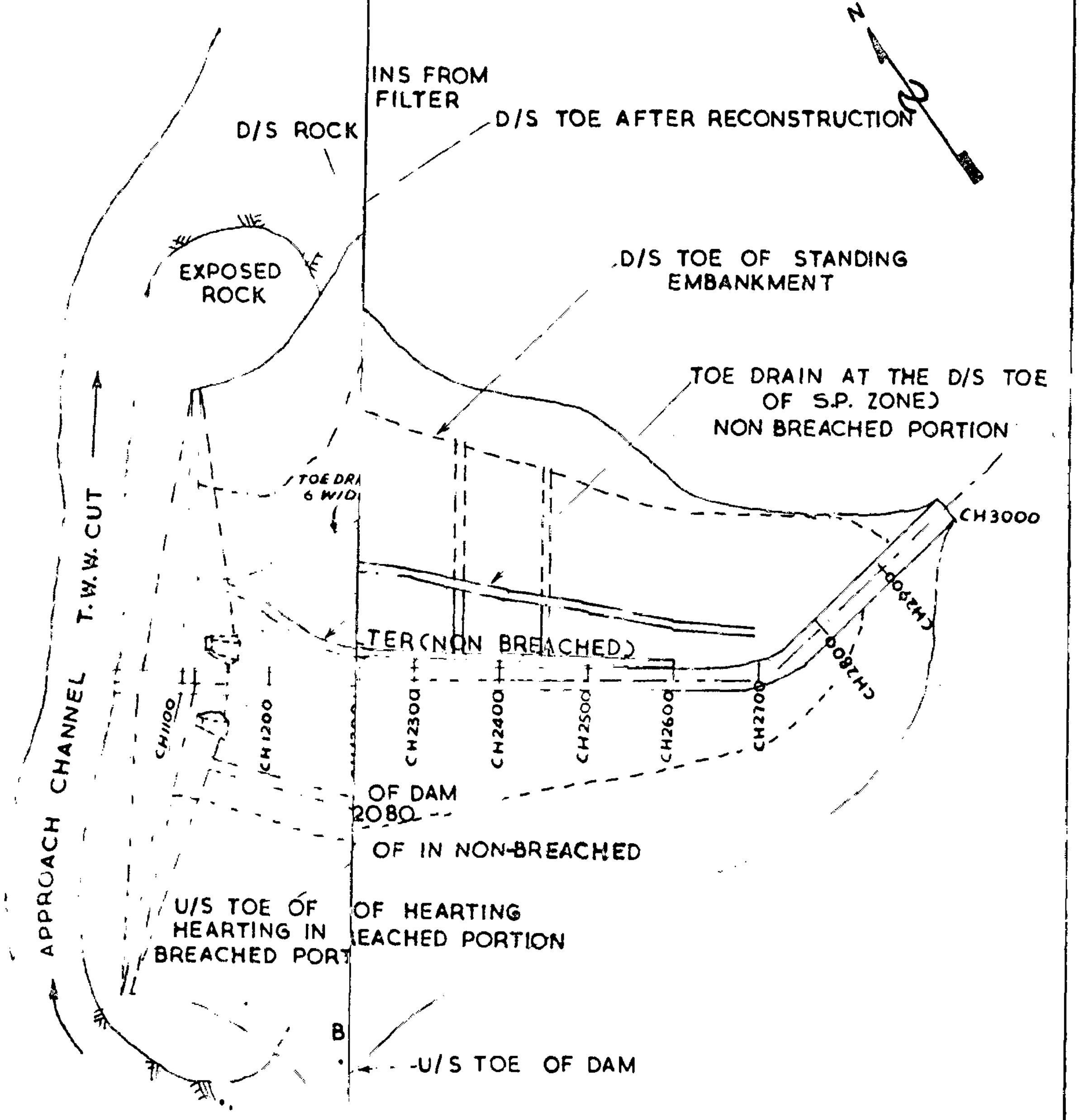
Two feet thick blanket of hand packed rubble placed on two layers, one of 6" thick quarry spalls and the other of 6" thick 1½" to 2" size metal is provided on the entire face, to protect the slope from wave wash.

(vii) Downstream Slope Pitching

12" thick flat pitching underlain by 6" quarry spalls has been provided to avoid rain cuts. The Panshet dam is located in 100" rainfall isohytol and as such protection of the downstream face is essential.

From the above it would be seen that the zones in the breached portion of the earth dam (i.e. new earth work) have been maintained nearly the same as adopted earlier (i.e. section prior to breach) except for the upstream slope of the hearting which has been reduced from 1.25:1 (H to V) to 1:1. This was done to reduce the quantity of hearting which had to be transported from comparatively longer loads.

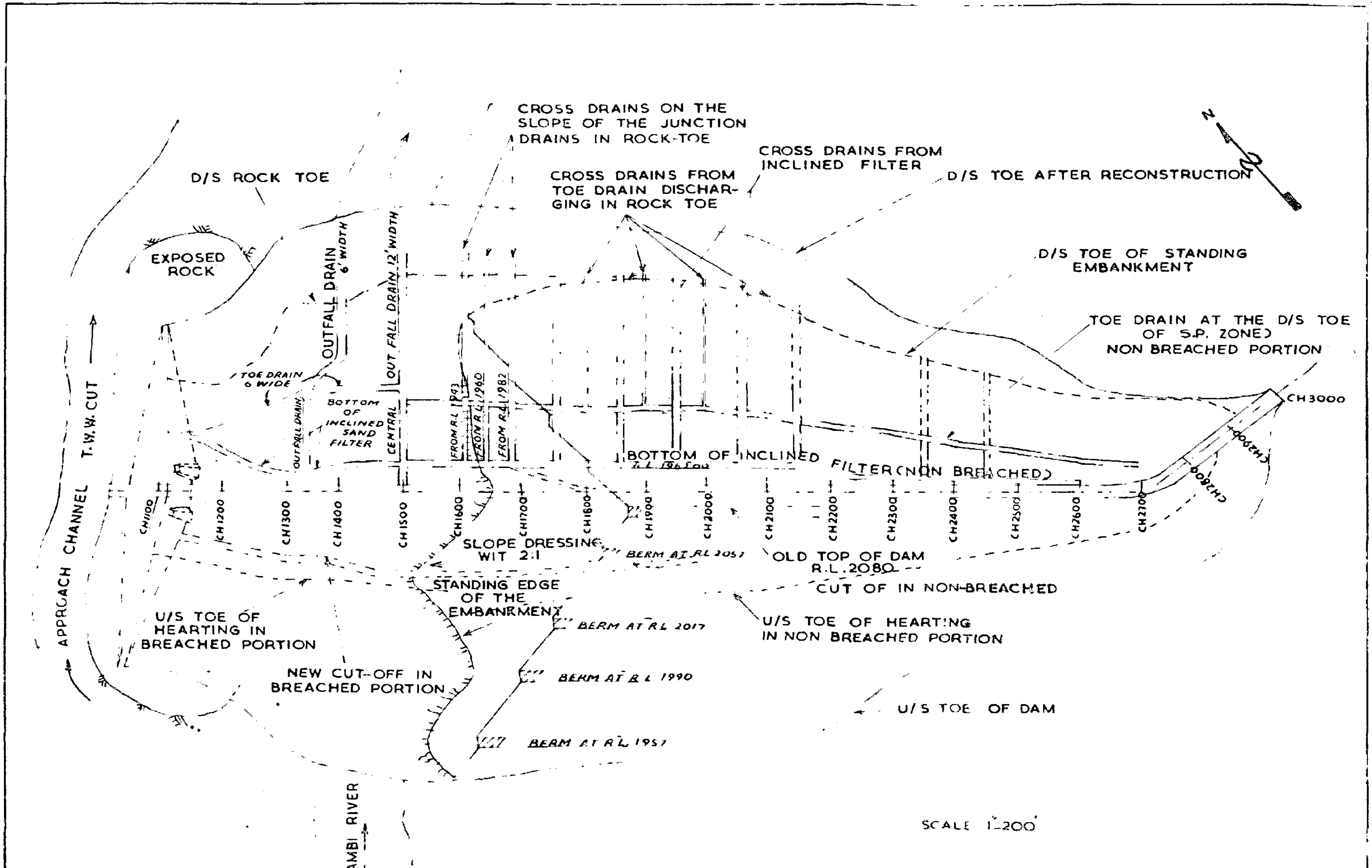
The other principal deviation from the old zoning pattern is the provision of an inclined filter drain in the body of the dam from R.L.2080 to approx. 2' above ground level. This filter drain is connected to cross-drains as indicated in Drawing No.12. An inclined filter drain was also provided in the old dam at an intermediate stage of construction from R.L.1980.00 to R.L.2040.00 which has been extended to R.L.2080 vertically as shown in Drawing No.13.



SCALE 1"=200'

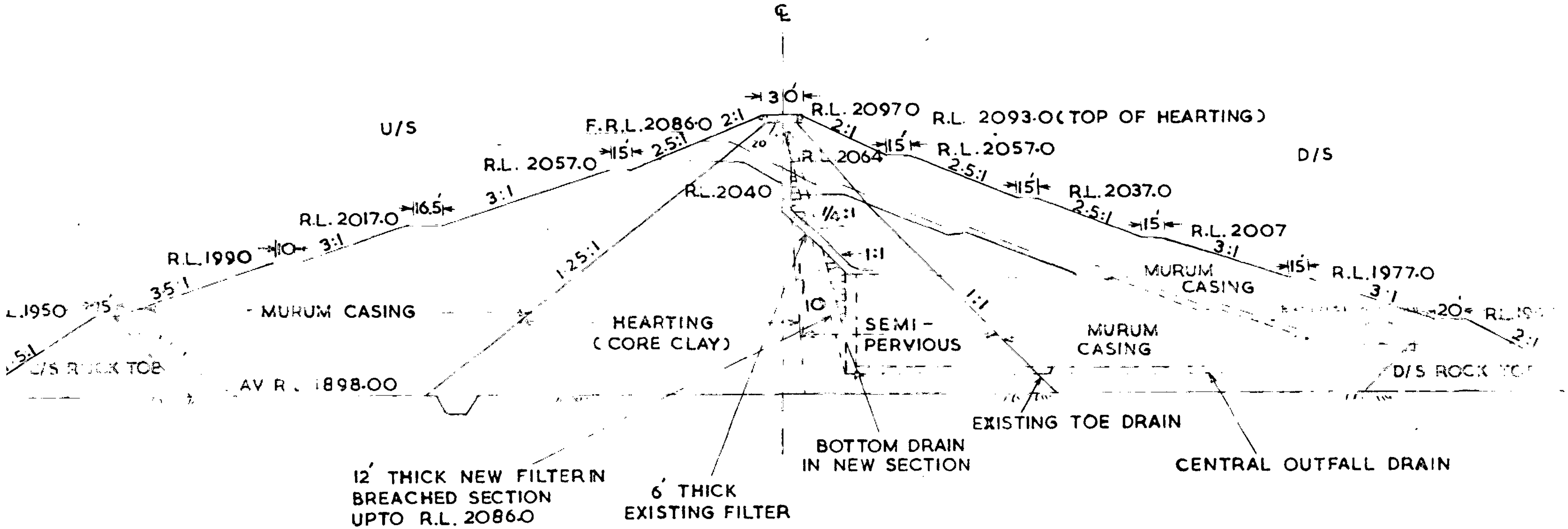
RAINAGE ARRANGEMENTS

No. 12



PLAN OF EARTH DAM SHOWING DRAINAGE ARRANGEMENTS

DRAWING. No. 12



NOTE : HATCHED PORTION SHOWS THE JUNCTION OF OLD AND NEW FILTERS

CROSS SECTION OF OLD DAM SHOWN THUS
RAISING AND WIDENING (FOR STAGE II) SHOWN THUS

CROSS-SECTION OF OLD AND NEW DAMS
SCALE 1" = 100'

DRAWING No. 13

The old and new filters have been joined by increasing the thickness of the filter at the junction. (Refer Drawing No.13)

In the non-breached portion (standing portion) the top portion of the dam upto R.L.2066.00 was removed before it was raised and strengthened. The details are shown in Drawing No.13 & 8..

S.2. EMBANKMENT MATERIAL CHARACTERISTICS

As stated earlier, the in-situ characteristics of the standing embankment were ascertained. In addition, the data collected for the original design of the embankment and the placement data collected during construction of the dam (prior to the breach) was analysed for deciding the design parameters for the various zones.

Fresh borrow area investigations were also carried out to assess the availability of embankment materials. Representative samples were collected from these borrow areas and tested for different properties. The shear parameters were obtained by conducting consolidated quick (undrained) shear tests in box shear apparatus on saturated samples compacted at modified proctor effort at O.M.C.

The design values for the stability analysis of the breached (i.e. new earth work) and non-breached (i.e. standing embankment) portions of the dam are summarised below:-

S.No	Soil Property	Hearting		Semi-pervious		Casing	
		B.P.	N.B.P.	B.P.	N.B.P.	B.P.	N.B.P.
1	Dry density in lbs/cft	95	95	98	98	108	108
	(d-o- kg/cum)	(1430)	(1430)	(1478)	(1478)	(1728)	(1728)
2	Saturated Wt in lbs/cft	118	118	123	123	130	130
	(-do- in kg/cum)	(1888)	(1888)	(1968)	(1968)	(2080)	(2080)
3	Cohesion in lbs/sft	600	600	500	600	200	200
	(-do- kg/sq.m)	(2929)	(2929)	(2441)	(2929)	(976)	(976)
4	Tan Ø	0.35	0.35	0.4	0.35	0.58	0.58

Note:- B.P. = Breached Portion (i.e. new earth work)

N.B.P. = Non-breached portion (i.e. standing embankment after breach)

Rock toe properties are assumed to be the same as those of the casing zone which assumption is conservative.

3.3. STABILITY ANALYSIS

Both theory and experience of dam failures show clearly that the following three conditions which may be critical from the stand point of shear failure which must be analysed:-

(I) End of construction condition

(II) Sudden draw down condition for the upstream slope

(III) Steady seepage condition for the downstream slope

(I) End of Construction

The factor of safety for either the upstream or downstream slope during construction could be critical where high construction pore pressures are likely be developed. In Panshet Dam high construction pore pressures were not expected to develop due to the following reasons:-

(a) Working season being limited to 8 months in a year.

(b) Construction being spread over a long period of 6 years.

(c) The rate of raising of the dam being only about 60 feet in a working season.

(d) Provision of a sand filter in the body of the dam

(e) The impervious zone having been laid at ±2% of O.M.C.

It was consequently not considered necessary to test the stability for the end of construction condition.

3.3.1. METHOD OF ANALYSIS

The stability analysis of both the upstream and downstream slopes was carried out by the Standard Slip Circle Method. The graphical method for plotting N & T forces without accounting for the effect of slide forces was used. Numerous slip circles were tried to locate the critical circle.

3.3.2. The assumptions made while carrying out the analysis are discussed hereunder:-

(I) Sudden Draw Down

The upstream slopes for both the breached and non-breached portions have been tested for drawdown from maximum water level to low water level. For facility of computation for accounting of pore pressures on sudden draw down, the following assumptions were made:-

Zone	Location	Driving Forces	Resisting Forces
Hearting			
i)	Above L.W.L.	Saturated	Submerged
ii)	Below L.W.L.	Submerged	Submerged
Casing			
i)	Above L.W.L.	Saturated	50% submerged
ii)	Below L.W.L.	Submerged	Submerged

Though the casing is sufficiently pervious, some pore pressures have been accounted for in casing zone above L.W.L. by considering the resisting forces in this case as 50% submerged.

(Note:- The design parameters are based on consolidated quick test results on saturated samples compacted at O.M.C. by modified proctor method. These values are on the conservative side since these have been worked out in terms of total stresses and additional allowance has been made for pore pressures in the stability computations).

The minimum factor of safety for this condition worked out to:-

(i) Breached portion (i.e. new earth work) 1.63

(ii) Non-breached portion (i.e. standing embankment after raising and widening) 1.38

The acceptable factor of safety for this condition, is 1.25.

The upstream slope has also been checked for earthquake with 0.1g acceleration for the following conditions:-

(a)	Full reservoir with full intensity of earthquake = 0.1g	F.S. = 1.20 (Accepted F.S = 1.0)
(b)	Sudden drawdown with half intensity of earthquake is the normal practice (adopted in the Beas & Ramganga) but the upstream slope has been tested for α = 0.075g	F.S. = 1.06 (Accepted F.S.=1.0)

(II) Steady Seepage Condition

The unit weights for computations of driving and resisting forces acting on the downstream slope are as under:-

Zone	Location	Driving Forces	Resisting Forces
i) Hearting and semi-pervious	Above T.W.L.	Saturated	Submerged
	Below T.W.L. or phreatic line	Submerged	Submerged
ii) Casing	Below T.W.L.	Submerged	Submerged
	Above T.W.L.	Saturated	a) Moist b) 50% submerged

Two conditions are assumed for the downstream casing viz.

(a) in which the downstream casing would remain moist, &

(b) in which some portion of casing would be saturated due to heavy pour.

The first condition is likely to prevail in view of a sand filter and drainage system. Condition (b) is a very remote probability and a lower factor of safety is considered acceptable. The minimum factors of safety worked out to:-

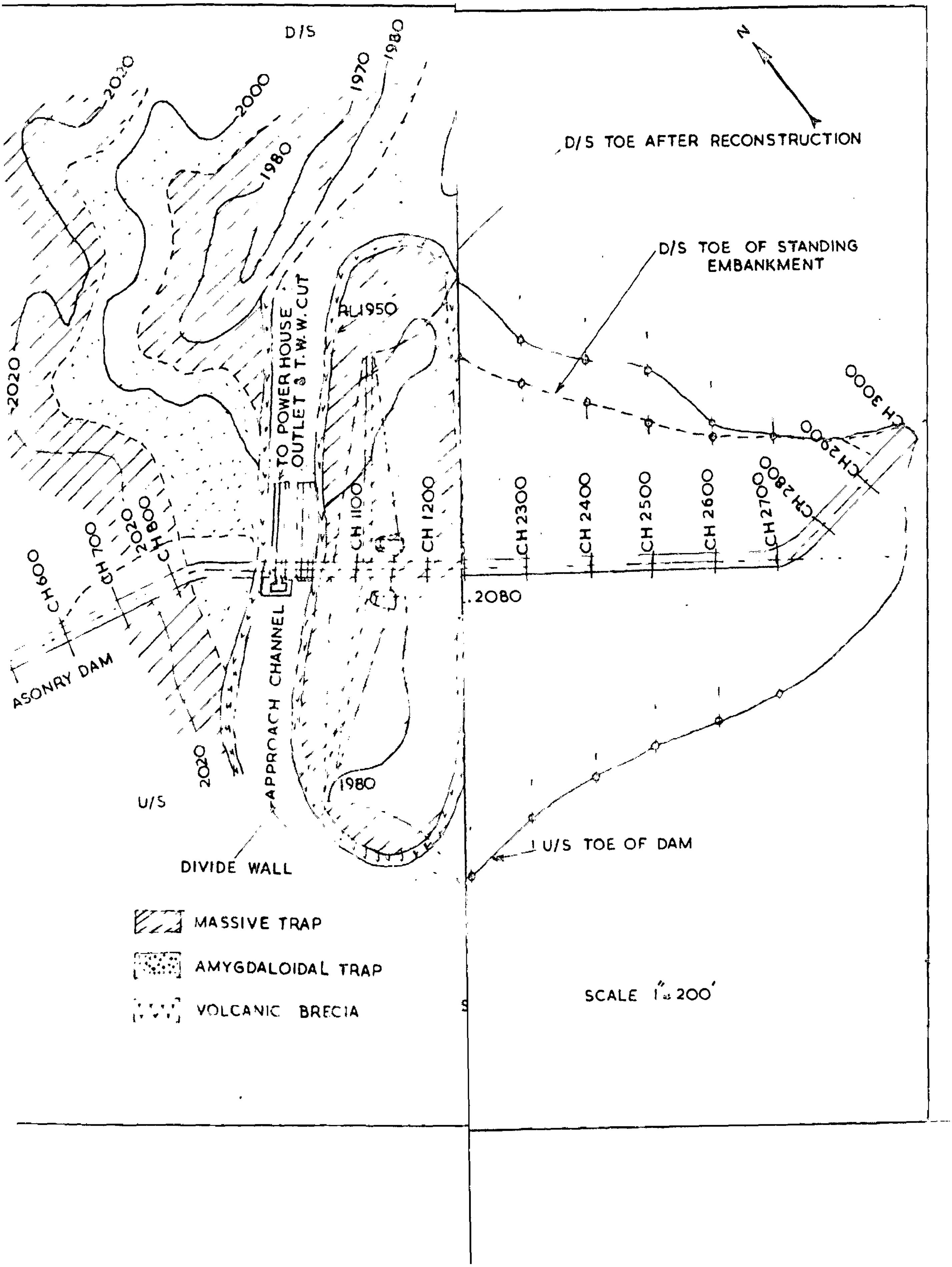
(i) Breached portion (a) 1.64 (b) 1.36

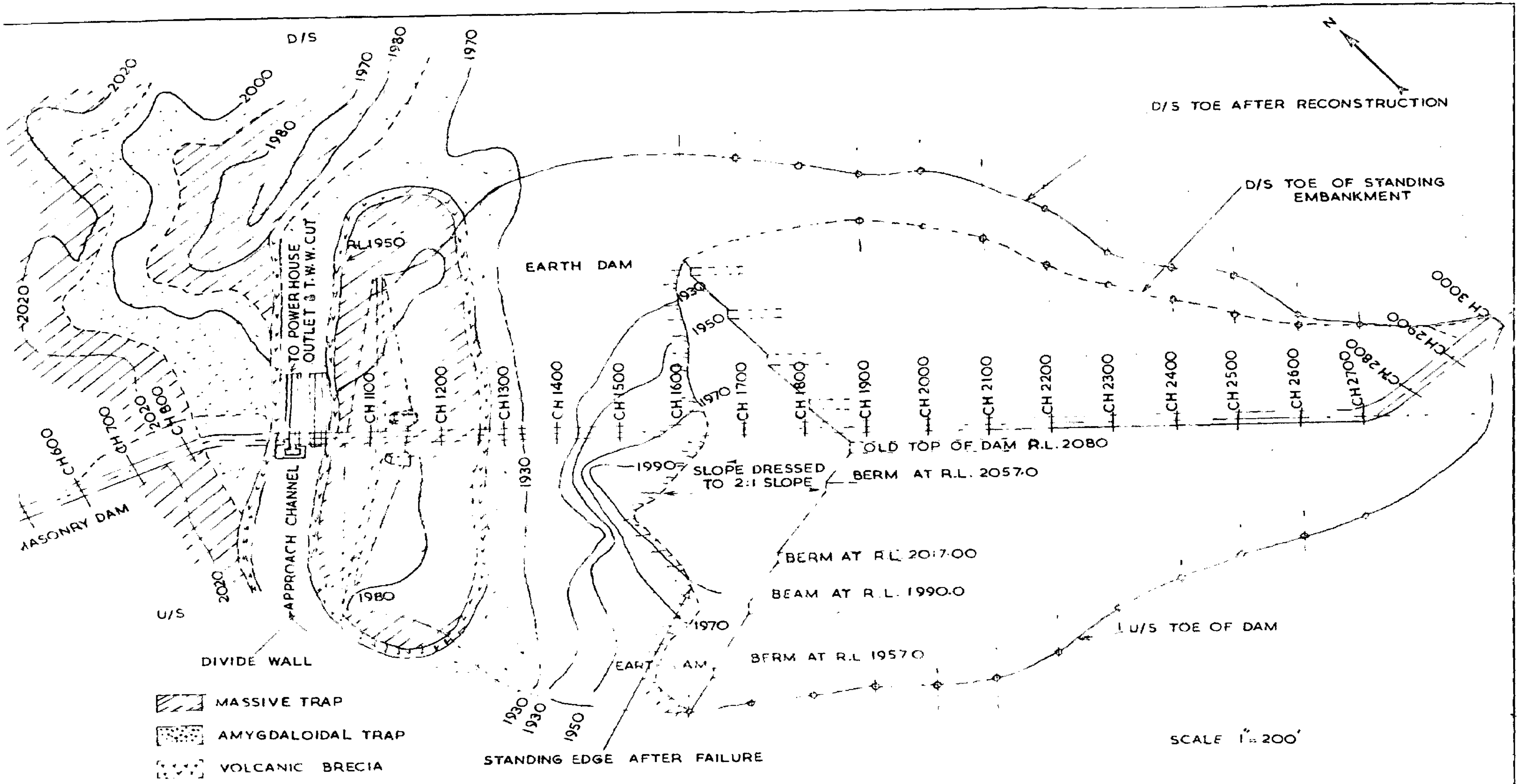
(ii) Non-breached portion (a) 1.86 (b) 1.45

The downstream slope has also been checked for earthquake with 0.1g acceleration. It is a normal practice to test the downstream slope for full reservoir condition with full intensity of earthquake. The factor of safety for condition (b) worked out to 1.07 as against the acceptable factor of safety of 1.0.

3.4. FOUNDATION TREATMENT IN THE BREACHED PORTION

The cut-off trench provided earlier in the left flank and gorge (i.e. breached portion) was practically obliterated. This trench was deepened by 2' to 3' so as to remove all loose rock and expose fresh rock surface. An additional cut-off trench 20' wide and 2' to 3' deep has been provided. The new cutoff trench has been joined to the cutoff in the standing embankment as shown in Drawing No.14. Water intake tests showed practically nil intake and therefore no special foundation treatment like grouting etc. was considered necessary. To have proper bond between the foundation rock and earth work, two key trenches 10' wide going 2' to 3' in rock have also been provided as shown in Drawing No.14.


D/S
D/S TOE AFTER RECONSTRUCTION
D/S TOE OF STANDING EMBANKMENT
N
2020
2000
1970
1980
RL1950
TO POWER HOUSE OUTLET & T.W.W. CUT
CH 600
CH 700
CH 800
CH 1100
CH 1200
CH 2300
CH 2400
CH 2500
CH 2600
CH 2700
CH 2800
CH 2900
CH 3000
2080
ASONRY DAM
APPROACH CHANNEL
U/S
DIVIDE WALL
U/S TOE OF DAM
MASSIVE TRAP
AMYGDALOIDAL TRAP
VOLCANIC BRECIA
SCALE 1" = 200'



PLAN SHOWING RENOVATION SCHEME

DRAWING. No. 14

3.5. DRAINAGE ARRANGEMENT

3.5.1. Internal Drains (Refer Drawing No.12)

In the existing non-breached portion of the dam, a longitudinal drain at the downstream toe of the semi-pervious zone and cross-drains at 100' C/C connecting it to the downstream rock toe had already been provided. An inclined filter drain with cross-drains at 100' centres was also introduced at a later stage of construction when the dam had already been raised upto R.L.1960. These cross-drains have now been extended along the downstream slope of the existing earthwork and taken to the rock toe before widening the section over the rock toe.

In the newly built portion a longitudinal drain is located at the downstream toe of the hearting into which the inclined sand filter drain ends. An out-fall drain 12' wide at Ch.1800 is provided as shown in Drawing No.12. In addition, a toe drain beyond the D/S toe of the S.P. zone has also been provided. On the sloping portion of the standing embankment additional three cross drains have been provided. (Refer Drawing No.12). The inclined filter drains for the breached and non-breached portions have been connected as shown in Drawing No.13. The outfall drain at Ch.1800 starts from R.L.1917 and ends in the rock toe at R.L.1913.00.

3.5.2. SURFACE DRAINS

Cross drains at every 400' along the downstream slope of the embankment connected to a longitudinal drain on each berm have been provided and these can be seen in photograph No.16 The water from all these cross drains and longitudinal drains is finally led into the river.

Collecting Drain

At the downstream toe of the dam, a collecting drain is provided to drain effectively the downstream casing.

3.6. ABUTMENT CONTACTS

Due to the provision of a masonry-cum-earth dam at Panshet, the following three contacts are involved in addition to the contact between the old and new earthwork already discussed:

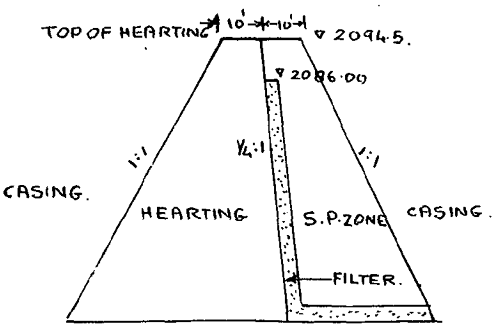
(i) Junction of masonry divide wall with earth.

(ii) Left flank abutment

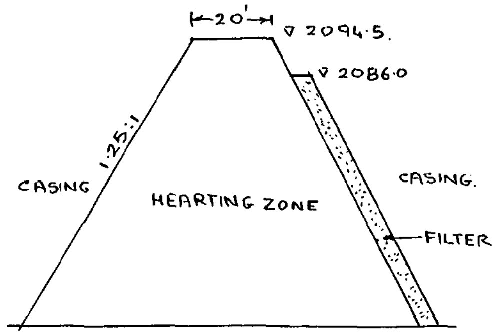
(iii) Right flank abutment

3.6.1. JUNCTION OF MASONRY DIVIDE WALL WITH EARTH

The width of hearting in the earth dam section adjacent to the masonry divide wall has been increased so as to increase creep resistance along the junction. The upstream slope of hearting is increased from 1:1 to 1.25:1 (H to V) and on the downstream side the semipervious zone is replaced by the hearting zone. The details of the earth section in breached portion and at the divide wall are as under:

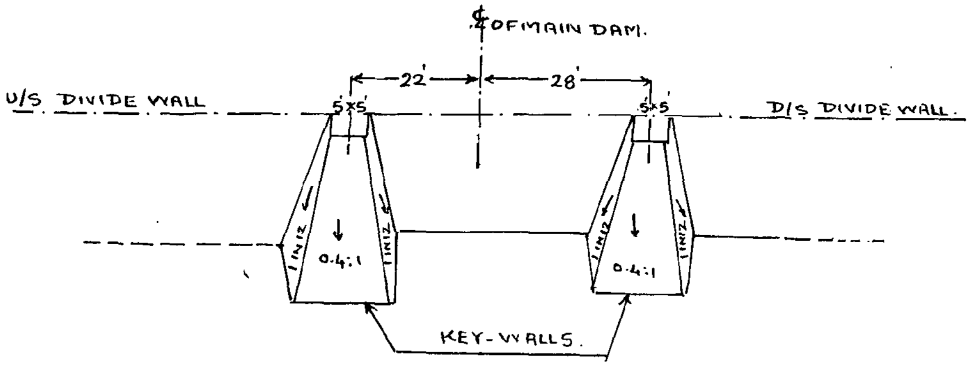


SECTION IN BREACHED PORTION.



SECTION NEAR DIVIDE WALL.

In addition to the above precaution, two key walls have been provided at this junction as indicated in Drawings No.9 & 10. These key walls are 5' x 5' at top and slope out to provide a bigger section at the base as shown below



The top of the upstream key wall is at R.L.2086.00 and that of the downstream key wall at R.L.2075.0. These two keys would further serve to increase the length of path of percolation.

The length of creep at different levels in the hearting near the contact of the divide wall with the earth dam has been calculated and are as under:-

R.L.	Length of creep in feet	Height of water in feet with reference to F.R.L. (2086.00)	Creep/Head Ratio
2080	63	6	10.5
2060	137	26	5.25
2040	203	46	4.4
2020	266	66	4.03
2000	330	86	3.84
1980	396	106	3.72

From the Lane's weighted creep ratio, it would be seen that the following creep ratio is recommended:-

(i) Soft clay 3.0

(ii) Hard clay 1.8

ans as such the above creep ratios worked out at various levels are within the safe limits.

3.6.2. LEFT FLANK ABUTMENT

The masonry dam is taken right upto the left flank. The minimum foundation level at the end is at R.L.2040.00. There exists a volcanic brecia layer between R.L.2045 to R.L.2055 and there could be possibility of leakage along the junction. In order to get over this problem a compacted clay filling has been provided between the end of the masonry dam and the left abutment. To increase the creep length along the junction of masonry and clay, key walls and key trenches have been provided as shown in Drawing No.(9 & 10). The key walls and the end of the masonry dam have been provided a batter of 1/12:1 and the cut-off trench (taken into the left abutment) a batter of ½:1.

These batters were provided so that with the consolidation of the clay fill the joint may get tighter.

3.6.3. RIGHT FLANK ABUTMENT

A cutoff trench 20' wide going about 30' deep in the abutment (about 2 feet into hard rock) has been excavated and backfilled by impervious soil. In addition to this, the hearting width is widened by providing the hearting a downstream slope of 1:1 instead of ½:1 and by deleting the semi-pervious zone. The entire width of the hearting upto R.L.2080 has been tied, to massive rock in the abutment. It is above R.L.2080 only that the hearting has not been placed against massive rock (the rock face is covered with huge boulders) and only the cutoff trench has been tied to massive rock in order to save some stripping.

3.7. WAVE HEIGHT AND FREE BOARD

The straight fetch at maximum water level is about 8 miles. The wave height as calculated by Stevenson's formula as modified by Molitor for a wind velocity of 100 miles per hour came to 5.87 ft. Allowing for the wave ride up and some extra factor of safety, a free board of 11 feet above the full reservoir level of R.L.2086 was provided. This crest level of 2097 is safe for a 50 mile per hour gate coinciding with the design flood.

3.8. SETTLEMENT ALLOWANCE & CAMBER

A settlement allowance equivalent to 2% of the height of the dam has been provided over the design crest level of 2097 to account for post-construction settlement (both normal and due to earthquake). This automatically provided a camber to the crest of the dam.

4.0. MASONRY DAM AND DIVIDE WALL

4.1. MASONRY DAM

The total length of the masonry dam is 610 feet. (From Ch.490 to Ch.1100). For most of this length, rock is exposed at the surface as such there is no difficulty of foundation. Water intake tests for the seat of the dam gave practically nil intake and therefore no special foundation treatment was considered necessary.

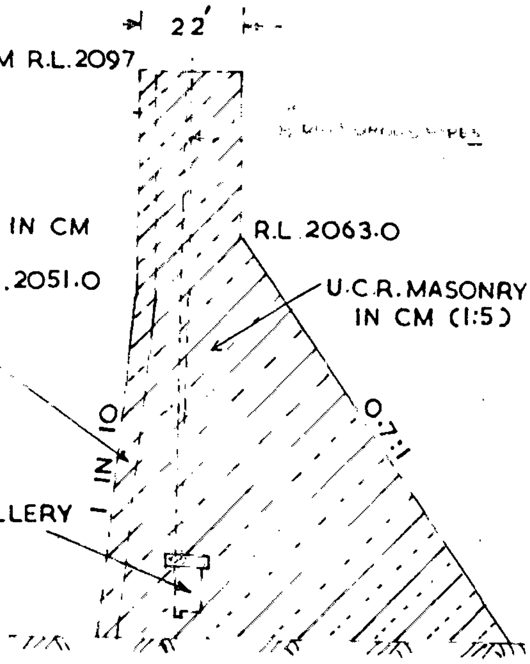
Two different sections of masonry dam, one for the portion outside the temp. W.W. cut and the other for the portion in the T.W.W. cut in which penstock and low level irrigation outlet are situated have been designed. The masonry dam on either side of the T.W.W. cut was designed and constructed with a top width of 22' and suitable upstream and downstream batters as shown in Drawing No.15 prior to the earthquake at Koyna. This section was consequently checked for 0.1g earthquake acceleration and found to be safe. The section in the temp. W.W. cut has 38 feet top width, U/S slope vertical and D/S face slope 0.82:1 (H:V) starting from 36 ft from top (Refer Drawing No.15). As no construction was started in the T.W.W. cut prior to Koyna earthquake, a fresh design was prepared for this monolith with an earthquake acceleration of 0.1g (both horizontal and vertical). The design assumptions and a table of stresses, coefficient of friction and shear friction factor for the two sections are given in Appendix II.

4.1.1. DRAINAGE GALLERY

In the body of the masonry dam a drainage gallery of 5' x 7' has been provided for reducing the uplift pressures and to facilitate grouting and drainage of the foundation if found necessary subsequently. The layout of drainage gallery is shown in Drawing No.(9 & 10).

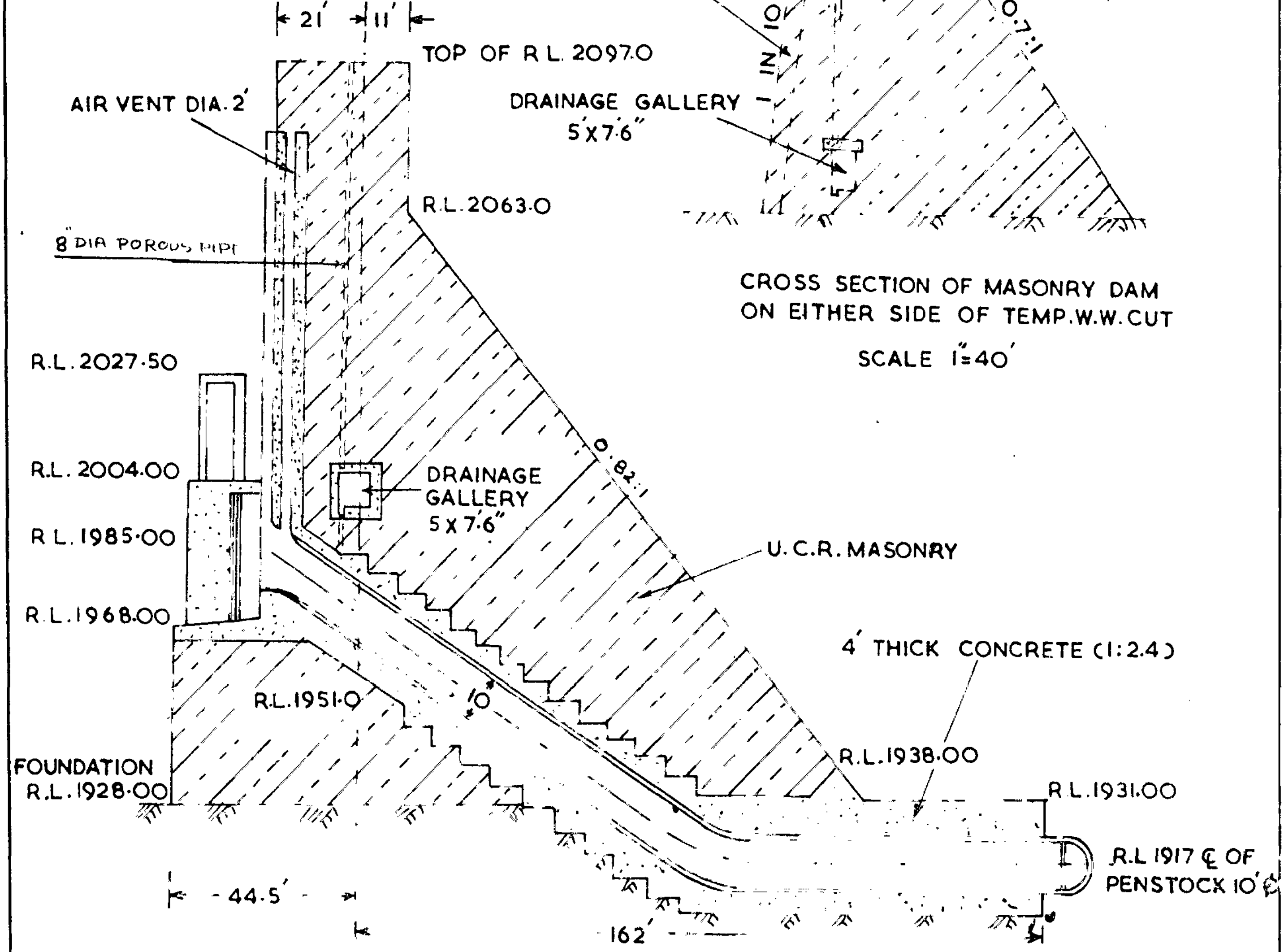
6" dia porous pipes at 20' centres surrounded by coarse sand have been provided to the top of the dam.

An additional drainage gallery at the foundation level in the temporary waste weir cut was also originally proposed. But as there were already too many openings in a width of 38' only, it



CROSS SECTION OF MASONRY DAM ON EITHER SIDE OF TEMP.W.W.CUT

SCALE 1"=40'



CROSS-SECTION OF PANSHET MASONRY DAM AT CH 967

SCALE 1"=40'

DETAILS OF MASONRY DAM SECTIONS

DRAWING. No. 15

was subsequently decided after two dimensional photo-elastic studies carried out in C.W.P.R.S. to do away with this gallery. A 24" Ø pipe has, however, been provided at R.L.1940 in the temporary W.W. cut masonry to drain the water collected by the porous pipes provided from the bottom of drainage gallery to R.L.1940.

4.1.2. MONOLITHS

The foundation level in the T.W.W. cut monolith is about 70 feet lower than that of the adjacent monoliths. The monolith in the temporary waste weir cut has been rigidly tied to the rock by anchors upto R.L. 1980. The rock on either side of the cut was very undulating and there was danger of weak joints between masonry and rock. Therefore the anchor bars provided in the rock abutments have been embedded in 1:2:4 cement concrete. At the rock abutments, grout buttons have also been provided to carry out grouting at these junctions from the gallery at R.L.1997 if required at a later stage. Above R.L.1980, copper contraction joints at Ch.932 and Ch.1020 have been introduced so that the masonry above the temporary waste weir cut becomes a separate monolith. Copper contraction joints have also been provided in the masonry dam at chainage 640 and chainage 800. The above joints were considered necessary to facilitate construction. The contact surfaces of the adjoining monoliths have been provided with plaster finish to permit relative movements between blocks.

4.1.3. TYPE OF MASONRY

Upstream 5' width of the section is constructed in uncoursed rubble masonry in C.M. (1:3). The rest of the section is constructed in U.C.R. masonry in cement mortar 1:5. But the masonry in T.W.W. cut upto R.L.1980 has been done in c.m. 1:4. The design strengths of c.m. 1:3; c.m. 1:4 and c.m. 1:5 were taken as under on the basis of laboratory tests.

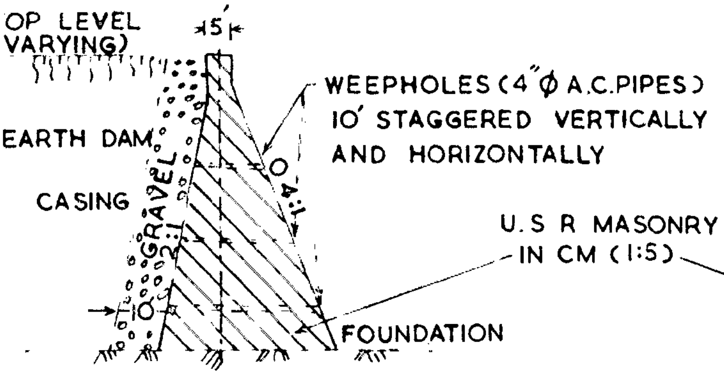
(i)	C.M.	1:3	1600 lb/sq"
(ii)	C.M.	1:4	1300 lbs/sq"
(iii)	C.M.	1:8	800 lbs/sq"

4.2. DIVIDE WALL

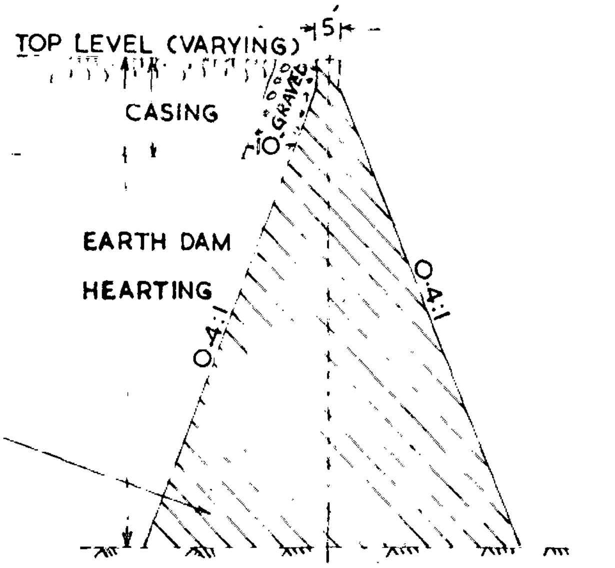
The total length of the divide wall to retain the earth dam at Ch.1100 is 640 feet (198 m). It is located on the bare rock available in between the temporary waste weir cut and the river proper in such a way that the alignment follows highest possible rock contours subject to its toes not falling in the T.W.W. cut or low ground on earth dam side. In keeping with this criteria the downstream portion could be aligned at right angles to the centre line of the earth dam while the portion on the upstream of the centre line had to be turned towards the left bank by 10°.

The divide wall has been constructed in monoliths with joints at suitable points such that each monolith has to retain only one type of earth material as far as possible. The sections adopted for each zone are shown in Drawing No.16 and the location of the joints in drawing No.10.

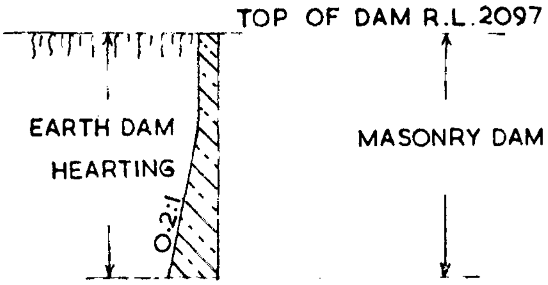
The design conditions and assumptions made for different zones are given in Appendix III.



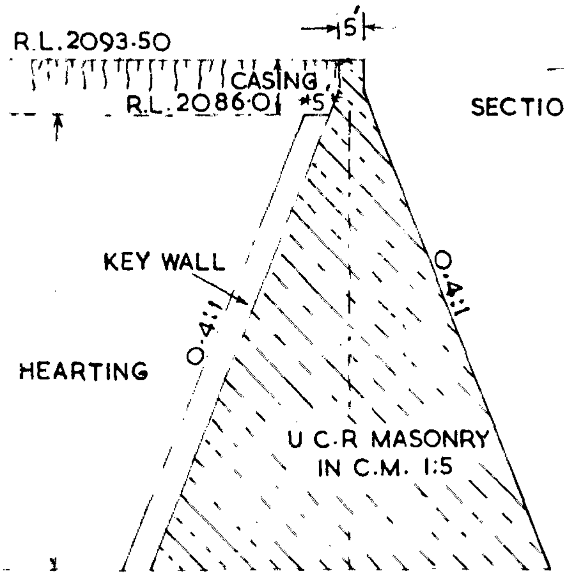
SECTION FOR DIVIDE WALL AGAINST U/S CASING
(ZONE I)
SCALE 1"=40'



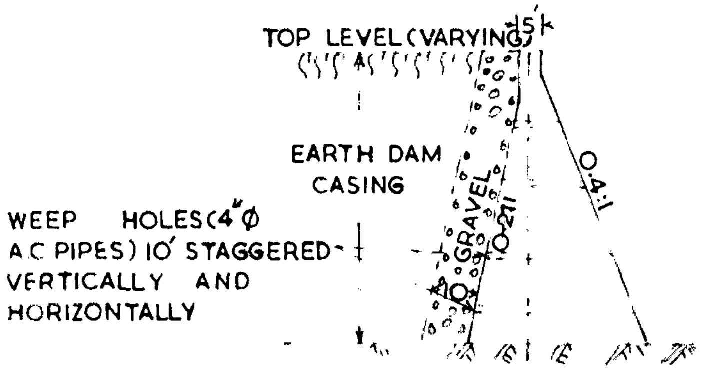
SECTION AGAINST PARTLY CASING AND PARTLY HEARTING
(ZONE II & IV)
SCALE 1"=40'



SECTION ON YY ALONG CENTRE LINE OF DAM
(ZONE III)
SCALE 1=40'



SECTION XX
THROUGH UPSTREAM KEY WALL
SCALE 1"=40'



SECTION AGAINST D/S CASING
(ZONE V)
SCALE 1"=40'

DETAILS OF DIVIDE WALL

DRAWING No.16

6.3. TREATMENT OF TUFF VOLCANIC BRECCIA LAYER

A 10 foot thick tuff volcanic breccia layer is exposed in the temporary waste weir cut and in the reservoir area between R.L.1940 and 1950. This breccia layer contains red bole as large lenses and also in the ground mass. The Superintending Geologist, Engineering Geology Section, Southern Region was of the opinion that there was no harm in founding the masonry dam and the divide wall on the exposed rock underlain by the volcanic breccia. It was decided to found the dam and the wall accordingly and to give the following treatment as suggested by him

(i) Consolidation grouting was done in the entire masonry dam and divide wall seats right upto the volcanic breccia layer. But there was hardly any grout intake.

(ii) On the upstream of the masonry dam in the T.W.W. cut, the volcanic breccia layer was blanketed by masonry and concrete upto the upstream toe of the earth dam and upto 300 ft on the left face. (Refer photo No.8

(iii) On the downstream of masonry dam in the T.W.W. cut the volcanic breccia layer was blanketed by masonry and concrete for a length of 80' & the portion beyond this is proposed to be gunited.

(iv) In the breccia exposures in the T.W.W. cut, the breccia was excavated to expose fresh surface which was covered with 2 foot thick R.C. slab with 5 ft overlap, in sound rock to check transfer of load to breccia.

4.4. OUTLETS

In order that the irrigation outlet may not be required to be operated against a water head exceeding 70 feet, two outlets one at R.L.2010 and another at R.L.1950 have been provided in the body of the masonry dam to let out irrigation releases till the power house is brought into use and later during its shutdowns. The power intake has a 10' dia steel penstock embeded in the T.W.W. cut monolith as shown in Drawing No.15. In short there are following three outlets and the details of which are discussed hereunder:-

(i) High level irrigation outlet at Ch.840

(ii) Low level irrigation outlet at Ch.990

(iii) Power intake at Ch.967

4.4.1. HIGH LEVEL IRRIGATION OUTLET

Two openings 6'-3" x 8' with sill at R.L.2010.00 have been designed to give free flow discharge of 1360 cusecs at a minimum water level of R.L.2020.0. This outlet would be used for depleting the lake from F.S.L. 2086 to R.L.2020.0 The service gates provided are of fixed wheel type with upstream skin plate and musical note type seals and are operated by means of stem rods and hoists. These gates would be operated from a chamber located at R.L.2080 in the body of the dam as shown in Drawing No.9. Provision for an emergency gate has also been made.

4.4.2. LOW LEVEL IRRIGATION OUTLET

This has also been provided with two gates in tandem, a service gate and an emergency gate. The size of the service gate is 6' x 7' and the size of emergency gate is 8'-4" x 11.20'. The service gates are similar to those provided in the high level outlet. This gate has to withstand a head of 140 feet in the closed position but would need to be operated for lake levels between R.L.2020 to R.L.1960. The sill of the outlet is kept at R.L.1950.0 and it is designed to discharge 520 cusecs at a minimum reservoir level of R.L.1960.0. The outlet in the body of the dam is of 5'x7' rectangular opening with semi-circular roof on its top and is having a bad slope of 1 in 15. The flow through this outlet is designed to be free flow under all conditions of operation and the released water is led into the river proper through an open channel.

4.4.3. POWER INTAKE

The power house (Refer Drawing No.10) is proposed near the toe of the masonry dam in the T.W.W. cut and the excavation required for the draft tube, power house etc. has been done. A 10 feet dia penstock with its sill at R.L.1974.78 to pass a discharge of 1000 cusecs is embeded in the body of the masonry dam in T.W.W. cut. The penstock lining is of ⅝" thick steel plates embeded in 4 feet thick R.C.C. ring throughout its length. The reinforcement is designed to take the full bursting pressure including 25% allowance for water hammer without taking the steel lining into consideration. For the time being the downstream end of the penstock has been closed by semi-circular bulkhead.

5.0. SPILLWAY

The permanent waste weir channel already excavated with its bed at R.L.2062 prior to the breach would be used as such to pass the floods for one year(to keep the reservoir level low) after which the ogee crest would be constructed. The gates would be provided later when requirement arises for extra storage.

In the ultimate design, an ogee crest at R.L.2072.00 for a length of 184 feet between abutments with four radial gates of 40' x 14' and 8' thick piers is planned. After construction of the ogee and installation of the gates, the high flood level has been worked out as R.L.2089.0 on the following considerations:-

(i) The full reservoir level of R.L.2086 is already attained and the spillway is discharging a base flow of 32,000 cusecs when the storm impinges.

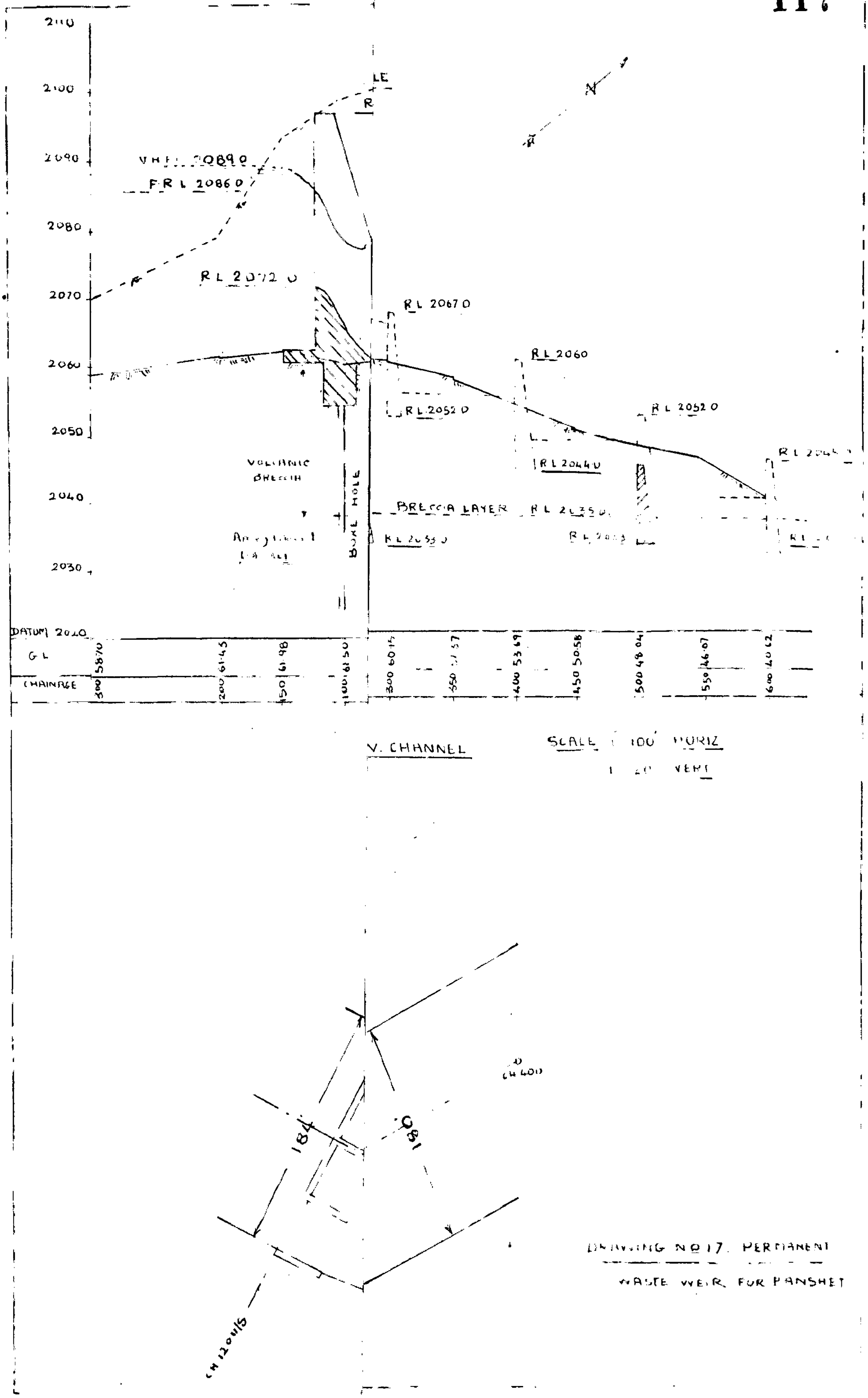
(ii) The following Koyna storm pattern has been adopted which is considered oversafe:

Duration (Hours)	3	6	9	12	24	48	72
Precipitation in inches	8	11	13.5	16	24	37	50
Incremental average hourly intensity in inches	2.67	1.0	0.8	0.8	0.67	0.54	0.54

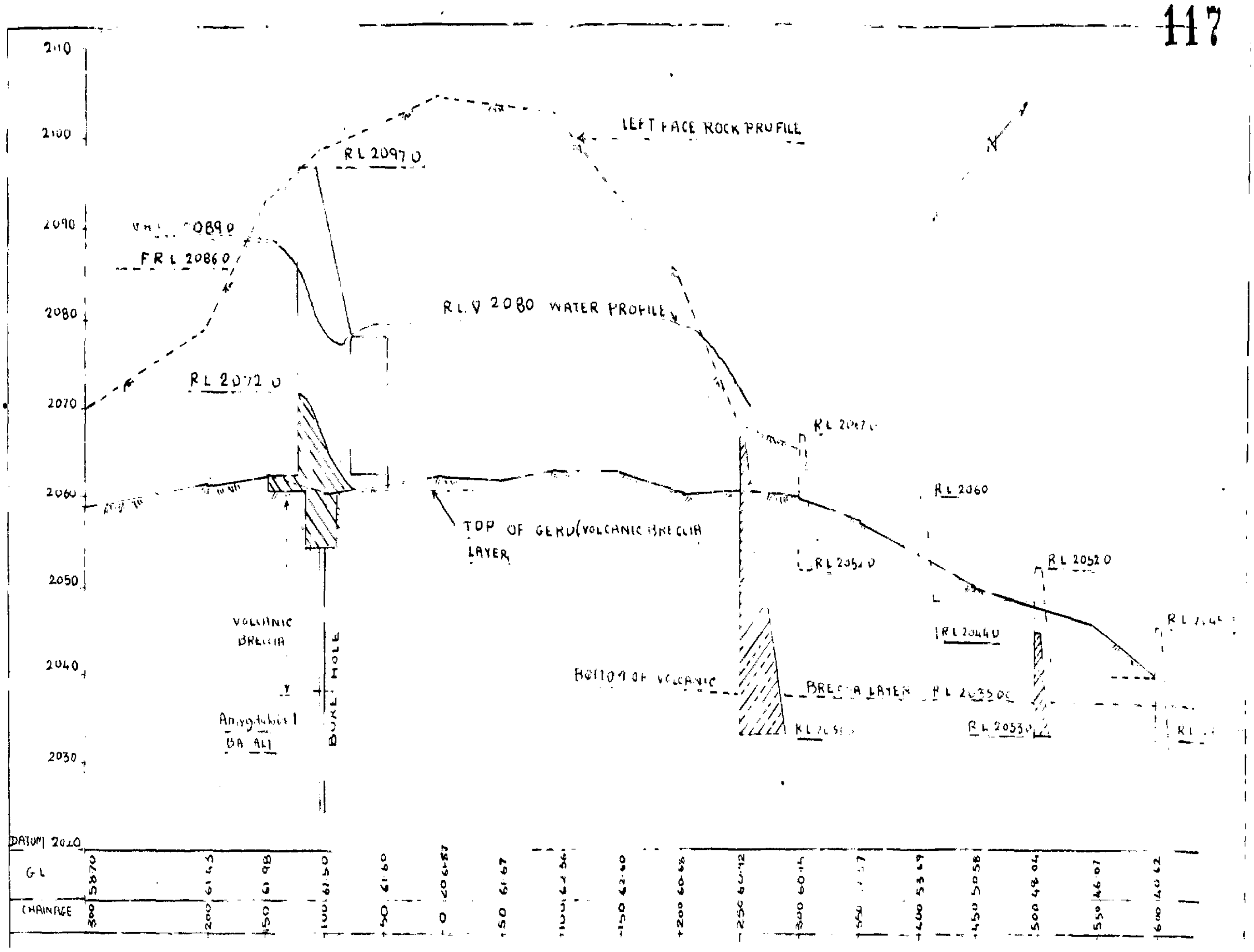
(iii) Flood absorption capacity between F.S.L. & H.F.L. has been accounted for.

5.1. PROTECTION ARRANGEMENTS

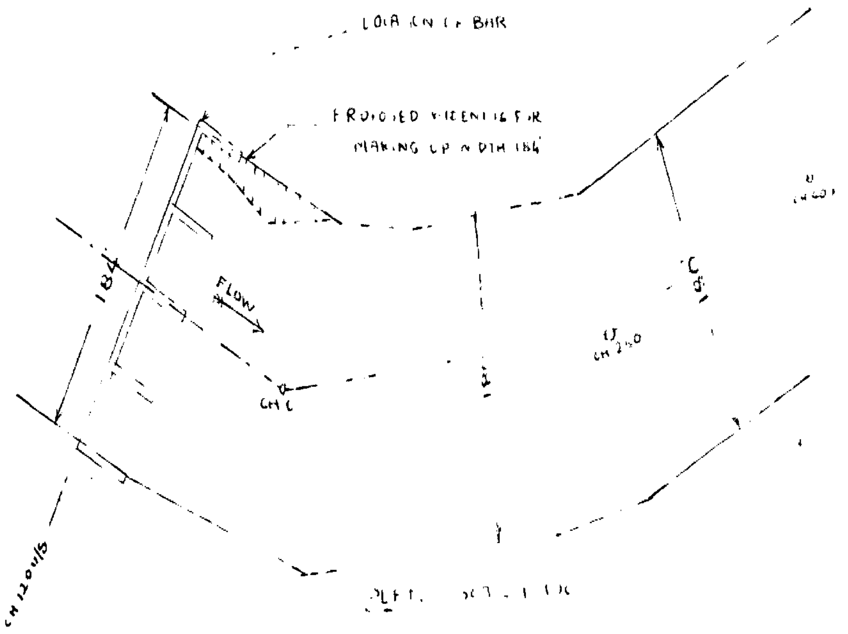
From the drawing No. 17 ,it would be seen that the tail channel beyond D/S chainage 250, is steeply sloping and during the 1961 floods a deep gully was formed in the breccia layer. In order to avoid retrogression it was considered necessary to have some protective arrangements for this volcanic breccia



DRAWING NO 17. PERMANENT
WASTE WEIR FOR PANSHET



LONGITUDINAL SECTION OF PWW CHANNEL   SCALE 1" 100' HORIZ
1" 20' VERT



DRAWING NO 17. PERMANENT
WASTE WEIR FOR PANSHET

layer which is met with between R.L.2035 to 2060.

Alternative schemes of providing falls were studied and it was found most economical to provide two falls one at Ch.250 and the other at Ch.500 (Refer Drawing No. 17) both founded on sound rock below the breccia layer. Beyond Ch.500 to the river junction, it has been left as such. The alignment is shown in Drawing No.4. Any further treatment or protection required would be provided after watching its behaviour.

CHAPTER - 11

CONSTRUCTION DETAILS

1.1. Renovation of the Panshet Dam which involves the use of 56.89 Mcft of standing embankment envisages the following works :

(i) Cutting back the slopes of the standing embankment to remove all loose (not well compacted) material and removing top 16' of the embankment.

(ii) Constructing new embankment from Ch.1100 to 1600

(iii) Raising and widening of the standing embankment to suit the final stage requirements.

(iv) Constructing a masonry divide wall at Ch.1100 at the junction between earth and masonry dams including key walls.

(v) Constructing a masonry dam between Ch.490 to 1100

(vi) Constructing high level and low level irrigation outlets and installation of 10' Ø penstock in the power intake.

(vii) Excavation for the power house.

(viii) Dismantalling the tower, transition box conduit etc

(ix) Constructing spillway and its protective works.

1.2. The quantities involved in the major items of work are:

(I) Earth Dam

(a)	Stripping of the existing embankment	8.51	Mcft
(b)	New earthwork (including coffer dam) 5.09 Mcft)	64.00	"

(i)	Hearting	10.83	Mcft
(ii)	Casing and S.P zones	48.88	"
(iii)	Filter	1.23	"
(iv)	Rock toes	0.83	"
(v)	Pitching U/S & D/S	2.23	"
		64.00	"

(II) Masonry Dam & Divide Wall

(a)	Dismantling tower, trashrack & conduit	Job	
(b)	Masonry for dam & divide wall	3.38	Mcft
(c)	Excavation in rock for power house & tail channel	2.59	"

2.0. The renovation works of the Panshet dam (except crest and gates in the spillway) have already been completed. Works executed in each season are discussed below:-

2.1. WORKING SEASON 1964-65

The following works were executed during the season:-

(i) The control tower, conduit and trash racks were dismantalled.

(ii) Coffer dam upto top R.L.1997.0 was completed to divert the floods from the temporary waste weir cut into the diversion channel.

(iii) Part stripping of the standing embankment

Photographs No.1 & 2 show the condition of works after the breach, as also the coffer dam put up to divert the flow in the diversion channel.

The section of the coffer dam (which finally becomes the U/S. toe of the dam) is shown in Drawing No.11. Later, during construction of the main embankment, the hearting of the coffer dam was removed upto low water level, so as not to prevent drainage of the upstream casing during drawdown, and the D/S. casing and rock toe fouling with the hearting of main dam, was removed.

2.2. <u>WORKING SEASON 1965-66</u>

<u>Earth Dam</u>

Since the diversion arrangement was already complete, it was possible to tackle the river gorge portion early during this season. The following works were taken up:

(i). Dewatering the foundations

(ii) Removal of the slushy material and cleaning the site

(iii) Opening the old cut-off trench and backfilling the same.

(iv) Excavation in rock for new cut-off trench for the embankment in the breached portion.

(v) Taking water intake tests in the cutoff trench (referred to as C.O.T.)

(vi) Joining the new C.O.T. to the old C.O.T. of the existing embankment. (Refer Drawing No.15)

(vii) Construction of longitudinal & cross-drains.

The condition of the works in progress as on 12-4-66 is shown in photograph No.3.

The downstream slope of the standing embankment was removed for a width of about 5' so as to expose a fresh compacted surface for the abutting the new earth work. This stripping was mostly carried out by dozers and some by a dragline. Simultaneously, the work of cutting back the breached face of the dam to the required slope was in progress.

It was possible to complete only 7.17 Mcft of earthwork in full width of the main dam and part work of stripping during the season.

Masonry Dam

The work was given out on contract which was finalised only in April 1965. Therefore, 0.13 Mcft of masonry in the divide wall could be carried out during the season. Photograph No.2 shows the condition of works at the end of the season.

2.3. WORKING SEASON 1966-67

Earthen Dam

During this season the work was taken up in three shifts and the earth work to the extent of 18.65 Mcft was completed departmentally with the help of following machines:

(i) General Shift

Trucks and tippers (Forgo, T.M.B. Leyland, capacities varying from 1.20 cft to 1.8 cft) were brought into use which were loaded manually in the respective quarries.

(ii) Shovel dumper Unit (One shovel 2¼ cu.yd capacity with 5 to 6 caterpiller dumpers of 15/17 cu.yds capacity) was used in two shifts.

In addition the work of stripping was continued.

Masonry Dam

The work of divide wall was in full swing. Masonry work on either side of the T.W.W. cut for the main dam was also taken up, and the total quantity of masonry carried during this season was about 1.09 Mcft. As the work of excavation of the power house at the toe of the masonry dam in the T.W.W. cut was not complete, no works in the temporary waste weir cut could be taken up.

2.4. WORKING SEASON OF 1967-68

Earth Dam

Same three shift working was carried out during this working season also and it was possible to complete the earth work to the extent of 18.99 Mcft. The work of removal of the top 16' of the old dam was carried out by a shovel dumper unit and the usable material was used in the new earth work. The work of stripping and slope dressing was completed. The filter in the old dam at R.L.2040 was opened by dragline and raised along with the embankment.

At the important junction between earth work and masonry dam the following precautions were taken:-

(i) All loose mortar on the face of the divide wall was removed.

(ii) Mud wash was given to the face of the masonry divide wall against which earth work was to be placed which was always kept 1' or 2' above the general earth work level. The compaction was normally done by sheepsfoot rollers. In adverse locations dozers were used where possible, otherwise recourse to compaction by pneumatic hand tampers was adopted. Near the junction, the placement moisture content of the fill was kept +2% of O.M.C. Sufficient field density tests were carried out to ensure proper compaction near the junction. Photograph No.6 shows the work in progress.

On the downstream side of the divide wall in the casing zone 7' thick gravel backed by 10' thick hand packed rubble was placed to ensure complete drainage immediately behind the wall. Photograph No.6 shows the work in progress.

Masonry Dam

The total masonry carried out during this season was about 0.99 Mcft at the following locations:-

(i) Masonry dam on either side of the T.W.W. cut.

(ii) Masonry dam in the T.W.W. cut. It was completed upto R.L.1970 in the upstream portion only and the remaining D/S portion was at foundation level. Photograph No.7 shows the position of work at the end of season. The monsoon floods were diverted over this masonry. The low level irrigation outlet was closed during the monsoons by lowering the service gate with the help of a temporary arrangement.

(iii) The volcanic breccia layer was covered by concrete & masonry as detailed in para 4.3 (CHPT 10) Photograph No.8 shows the completed work.

2.5. WORKING SEASON 1968-69

Earth Dam

Only truck unit was used and the earth work was completed to the extent of about 10 Mcft. The work continued as planned.

Masonry Dam

The work of installation of penstock was taken up and completed partly. Photographs No.9 & 10 show the penstock work in progress. The penstock fabrication and erection work was construted to the Indian Hump Pipe Company.

The masonry in the temporary W.W. cut was raised upto R.L.1996 and the total masonry work completed during this season was of the order of 0.60 Mcft. The breccia treatment as explained in para 4.3 (CHPT 10) was also done. Photo No.12 shows the treatment in progress. Water was again allowed to spill over this masonry during the monsoon season. Photograph No.11 shows the spilling during monsoons. Both the irrigation outlets were kept closed by service gates during the monsoon.

Left Flank Junction

As the masonry dam in its last monolith was nearing completion, the cutoff trench taken deep in the left flank abutment was taken up for backfilling with impervious material. Till a workable area was available, the backfilling was compacted by pneumatic hand tampers and dynapacs (petrol operated earth rammer). Photographs No.13 & 14 show the work in progress. The junction between the masonry dam and the backfill (between the masonry and the rock face) was treated in the same manner as described earlier in para 2.4.

2.6. WORKING SEASON 1969-70

Earth Dam

The earthwork continued as planned in the day shift with the truck unit. About 2.90 m cft were placed during this season and practically all the earth dam upto R.L.2095 was completed.

Photograph No.15 shows the work in progress. It also shows the cutoff taken in the right abutment. The sides were dressed to 1:1 slope and the backfilling by impervious soil was carried out upto R.L.2094.00.

The pitching of upstream and downstream slopes was also completed simultaneously. Photograph No.16 shows the view of downstream pitching and surface drains (i.e. cross drains at every 400' along the slope and the longitudinal drains on the berms).

<u>Masonry Dam</u>

The construction sequence adopted was in keeping with the design criteria of subjecting the dam to gradually increasing heads. The following table gives the maximum retention levels attained and additional head to which the dam was subjected to in each year so far

Year	Retention Level	Additional head to which dam was subjected
(i) 1964-65	No storage	Nil
(ii) 1965-66	Upto R.L. 1935.0 (i.e. upto the bed of diversion channel	34'
(iii) 1966-67	Upto R.L.1935	Nil
(iv) 1967-68	Upto R.L.1970	35'
(v) 1968-69	Upto R.L.1995	25'

Though it was possible to complete the monolith in the temporary W.W. cut to its final level of 2097 and to divert the flood waters through the permanent waste weir channel, the work was restricted to R.L.2040 only so as to restrict the additional head on the dam to 45 feet only.

To implement this decision it b-ecame necessary to provide a temporary ogee crest (to improve hydraulic conditions) and guide walls (to prevent spilling of flood waters beyond the T.W.W. cut). Photo No.17 shows the stage of works at the end of the season.

Spillway

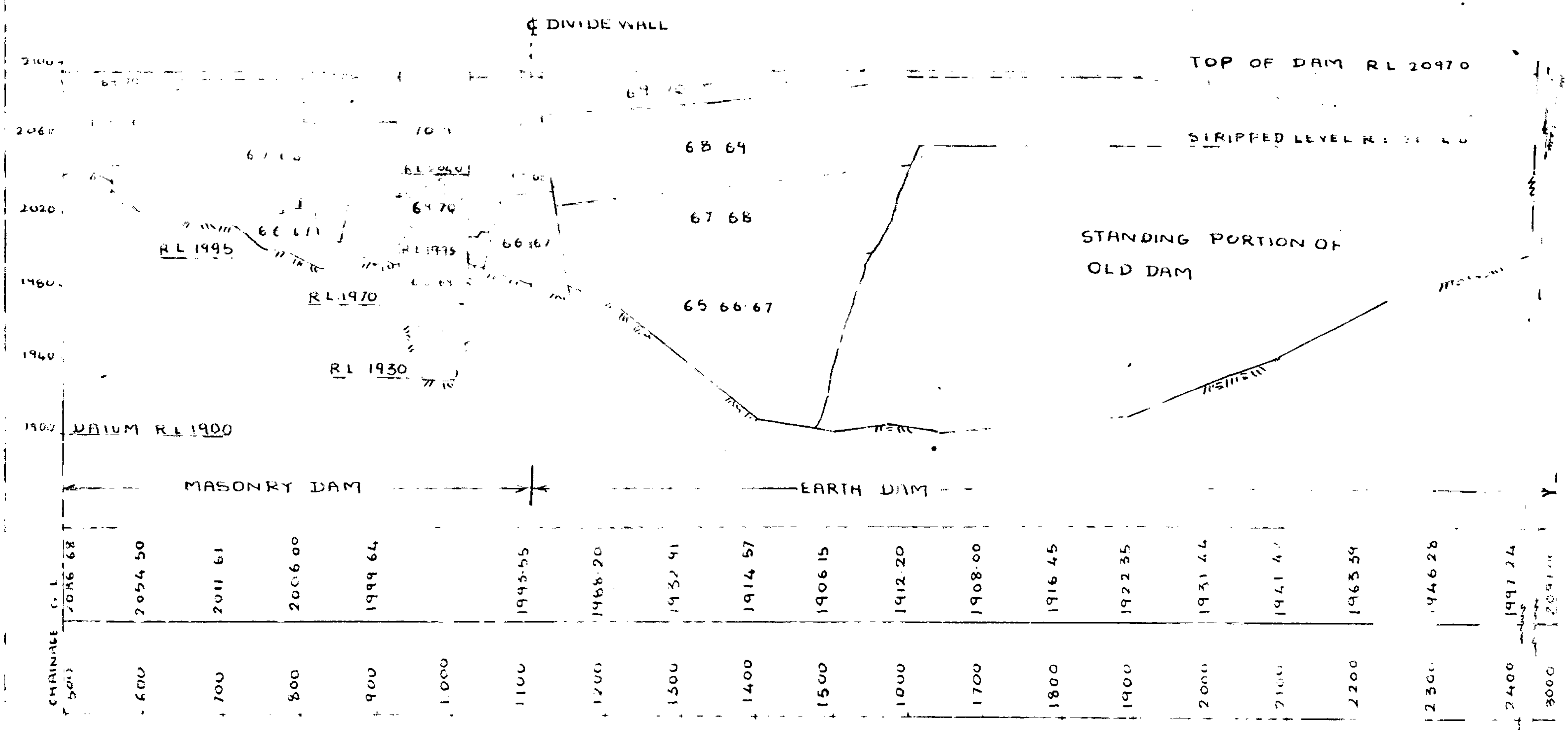
No major work was undertaken except widening of the channel wherever necessary.

2.7. WORKING SEASON OF 1970-71

During this season both the earth and masonry dams were fully completed. The spillway channel protection works were also taken up and completed. Photograph No.18 shows the protection work in progress. The flood waters during the monsoon were passed in the spillway channel at the close of the season. Season-wise work carried out in nutshell is shown in Drawing No.18.

2.8. WORKING SEASON 1971-72

It is proposed to complete the ogee crest and the superstructure etc. of the spillway control arrangement in this season. The gates, as already mentioned earlier, will be installed later when demand for water increases.



DRAWING No 18 : LONGITUDINAL SECTION OF DAM SHOWING WORK DONE EACH YEAR

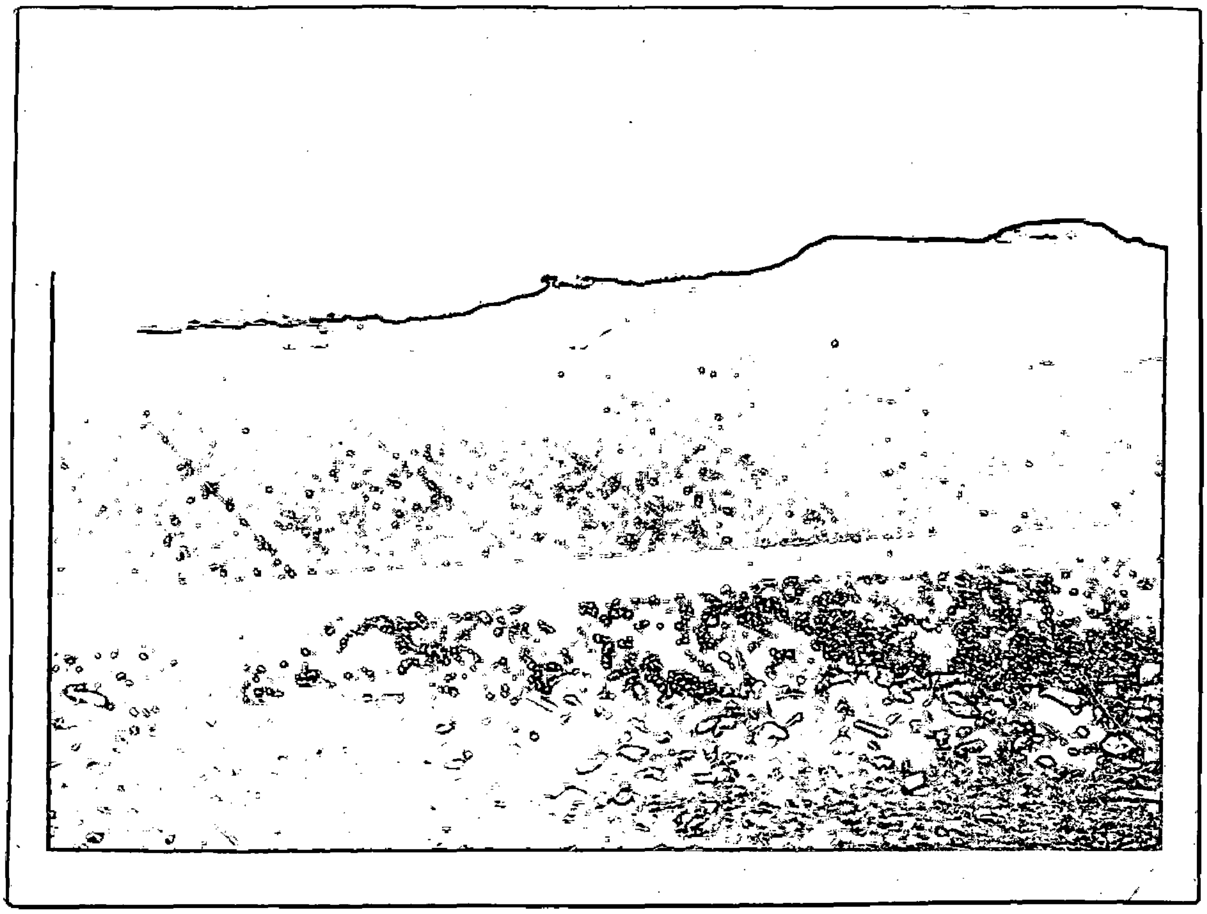
CHAPTER - 12

CONCLUSIONS

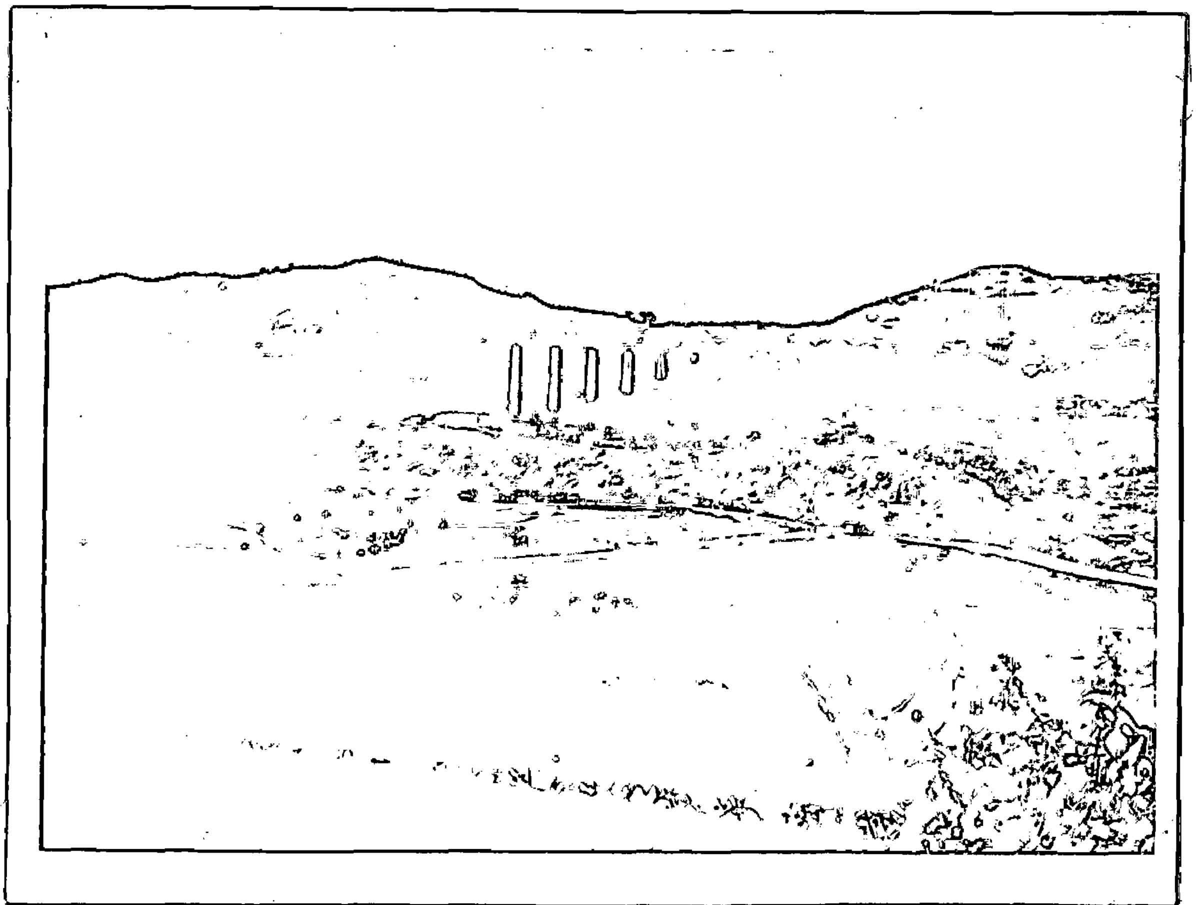
1. The irrigation outlet conduit of Panshet dam was essentially a light structure with inlet control and a roof arch of voussoir construction (precast c.c. blocks cemented together). It had been provided in the body of the dam in a deep open cut. The failure can be mainly ascribed to its use before final completion in respect of finishing of the conduit bed and the control arrangements. Even so, the disaster could possibly have been averted had the cut been filled partly or wholly with masonry or concrete to form a hole in a massive masonry or concrete block.

2. It is well known that provision of an outlet conduit in the body of an high earth dam is not free from risk unless structural integrity of the outlet (under differential loading conditions and the type of foundation), proper compaction of the embankment material round it, adequacy of seepage path at the junction and non-possibility of differential settlement cracks, if provided in a deep cut, can be fully ensured. Ensuring all this is not easy. The history of earth dam failures is replete with instances where a conduit in its body was in one way or another the cause of its failure. Therefore, conduits in the body of high embankment dams are avoided as far as possible. Although the failure of Panshet Dam was due mainly to the incompleteness of work of the conduit, it does bring out the inherent weaknesses of the type of construction that was adopted.

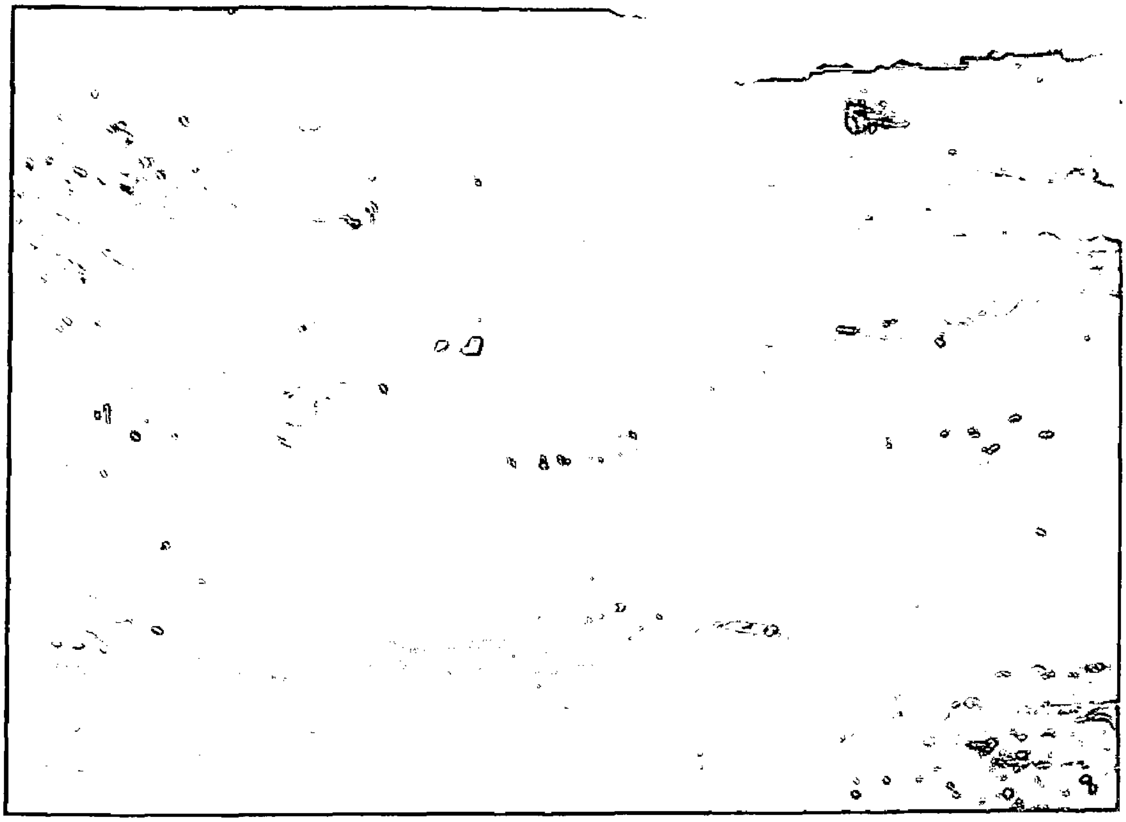
3. In the proposals for the renovation of the Panshet Dam, the alternatives considered involved either restoration of the earth dam with the outlet conduit inside massive masonry back filling or in a part masonry dam on the left flank. The latter alternative was adopted mainly because it admitted of gradual filling of the reservoir below the spillway crest in a period of 3 years so as to subject the vulnerable junctions of the new and old earth dam sections and the earth dam and the masonry divide wall (at its junction with the masonry dam section) to gradual increase of hydraulic head. This is a consideration which deserves attention wherever conditions permit as in the case of earth dams with masonry or concrete spillway sections.



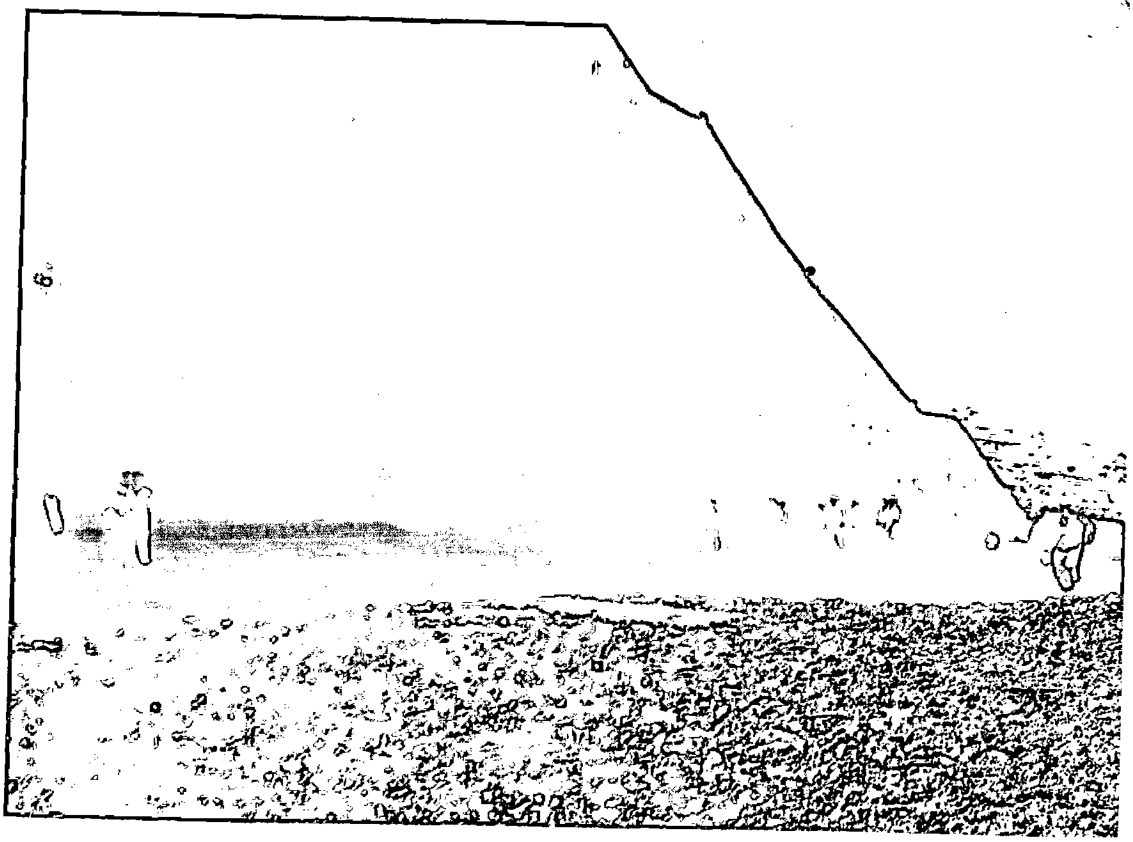
Photograph No.1 - View taken from Upstream side of the dam on 17-6-(



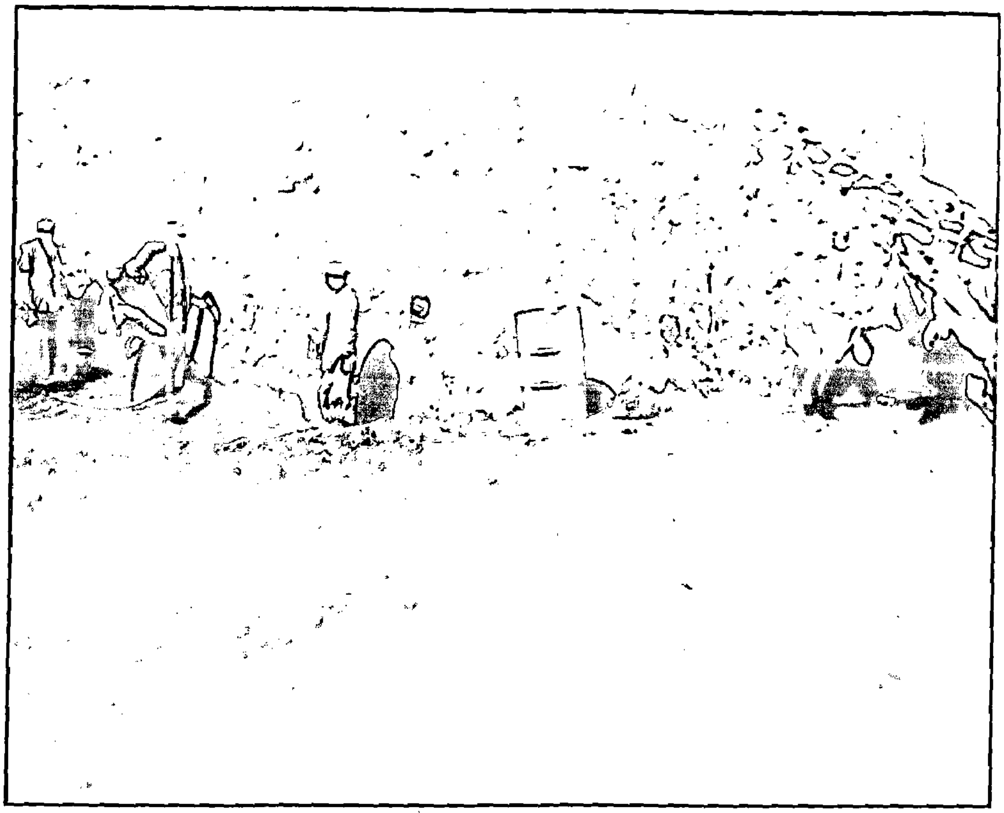
Photograph No.2 - View taken from downstream side of the dam on 20-10-(



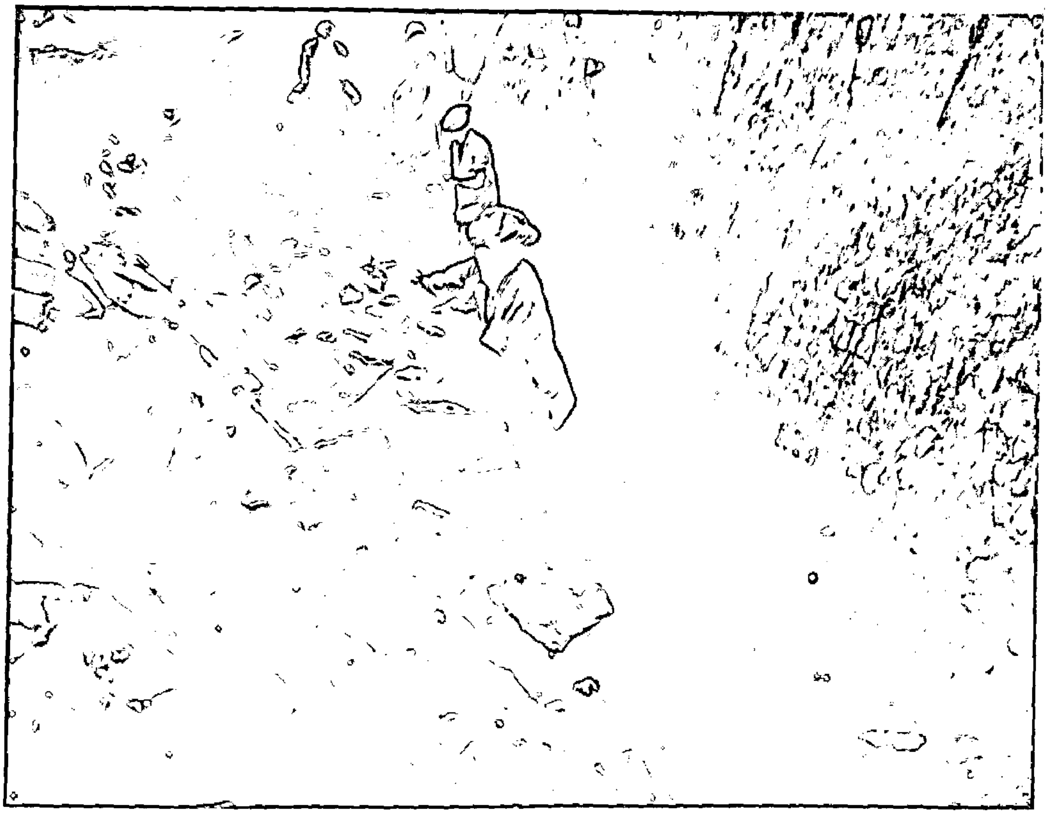
Photograph No.3 - Condition of works as on 12.4.66



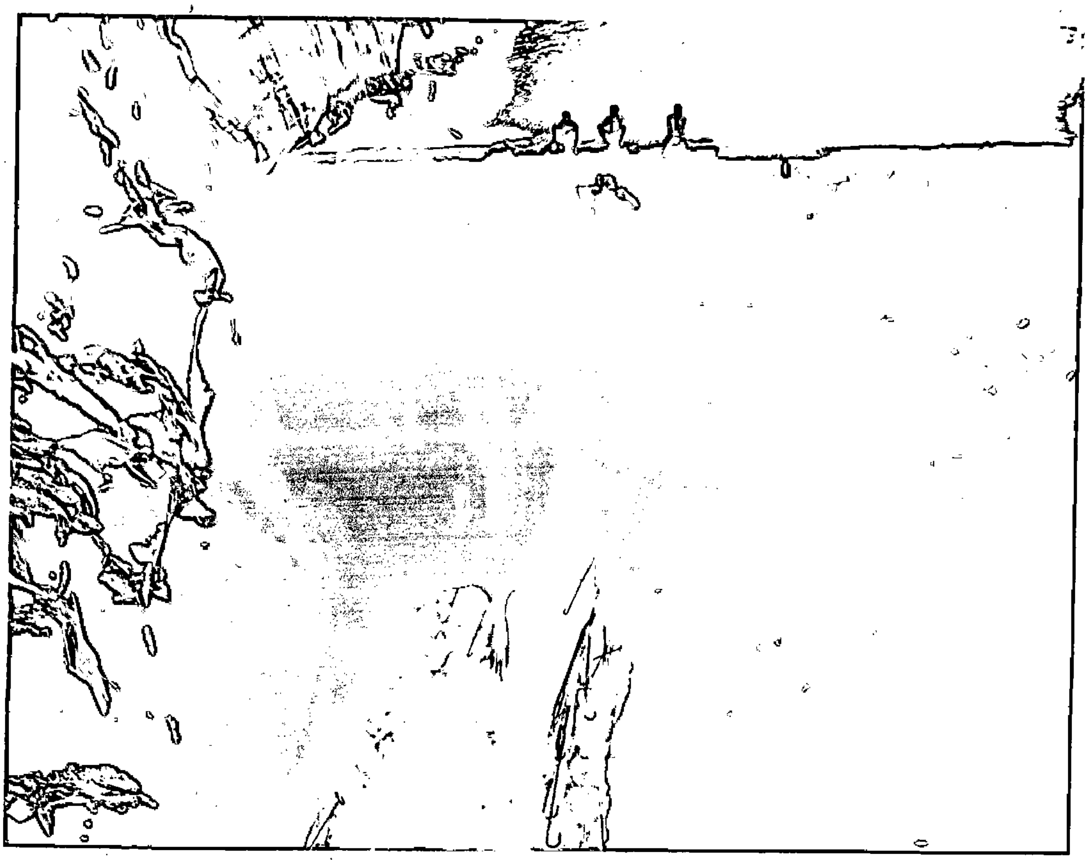
Photograph No.4 - Treatment at the Divide Wall Junction in progress as on 30-5-68



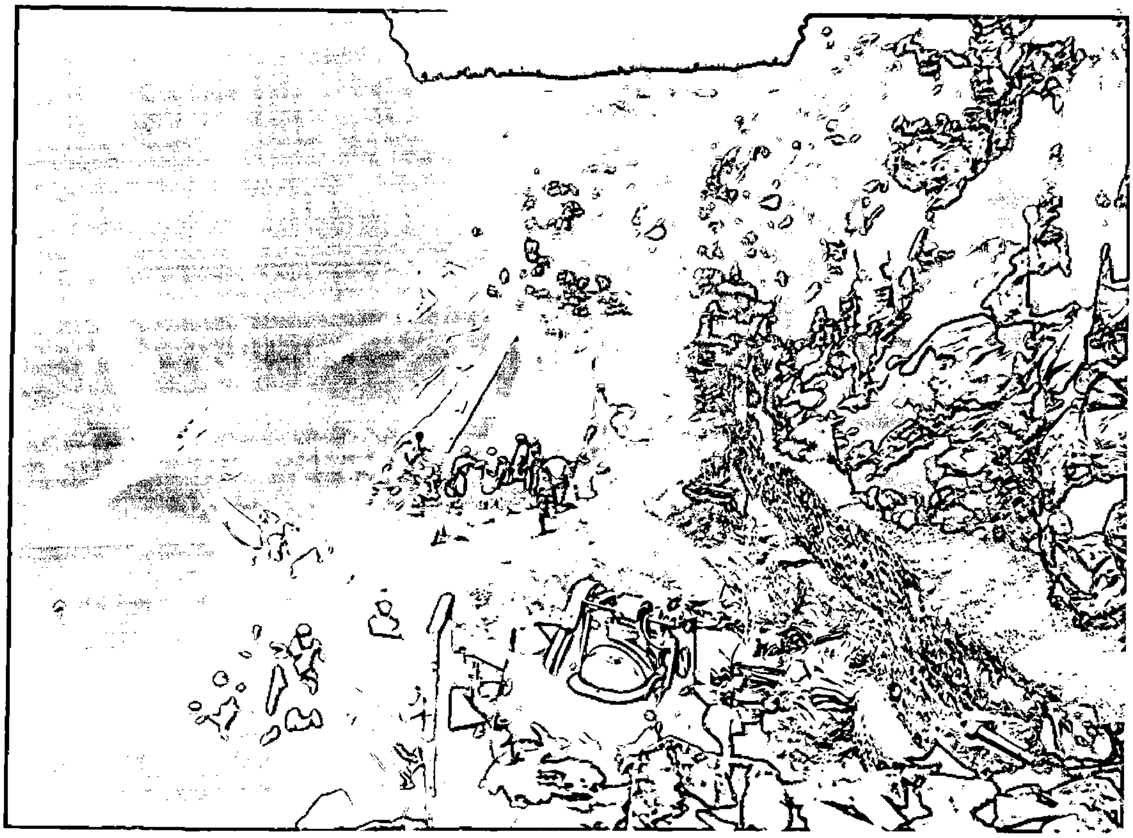
Photograph No.5 - Treatment at the Junction at Higher Levels



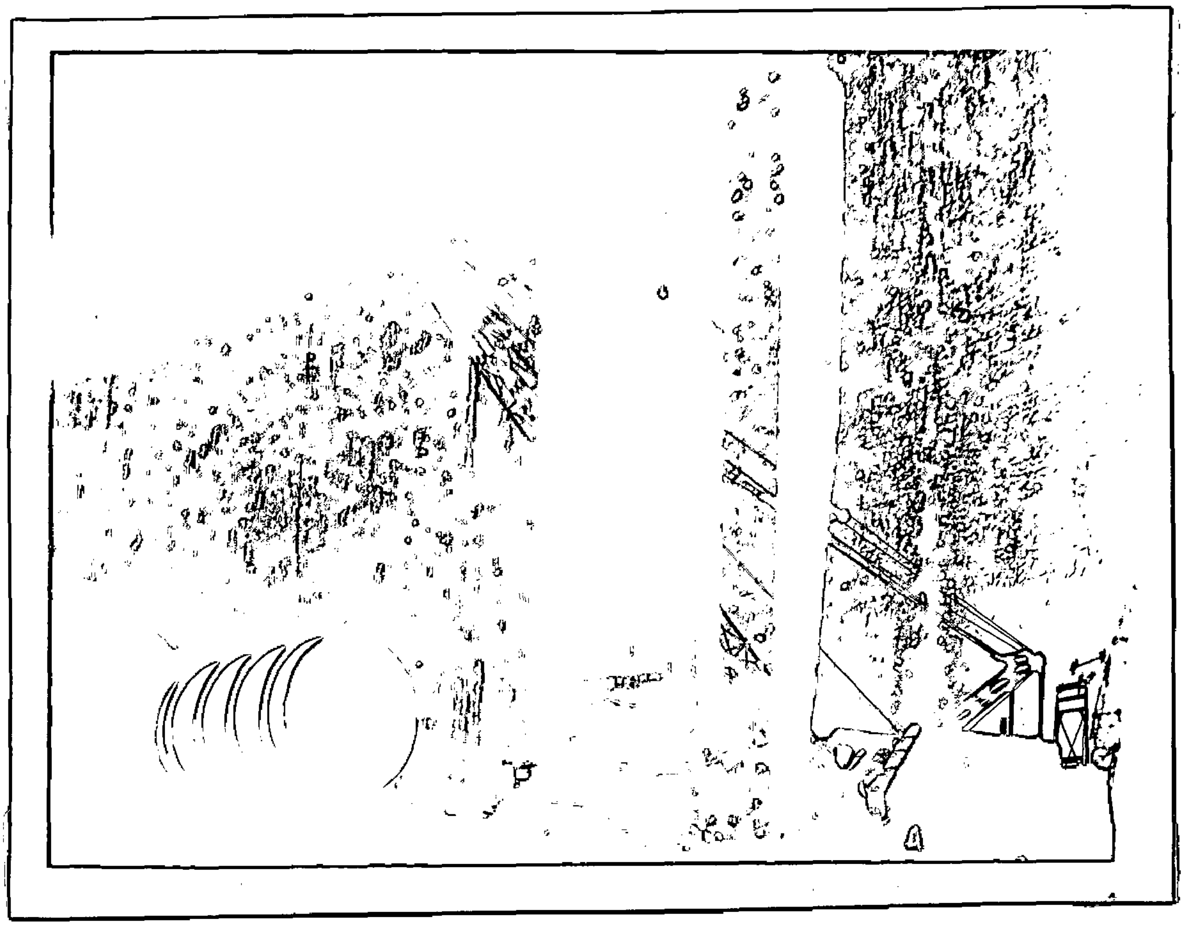
Photograph No.6 - The Drainage Arrangement at the Back of Divide Wall as on 30-5-68



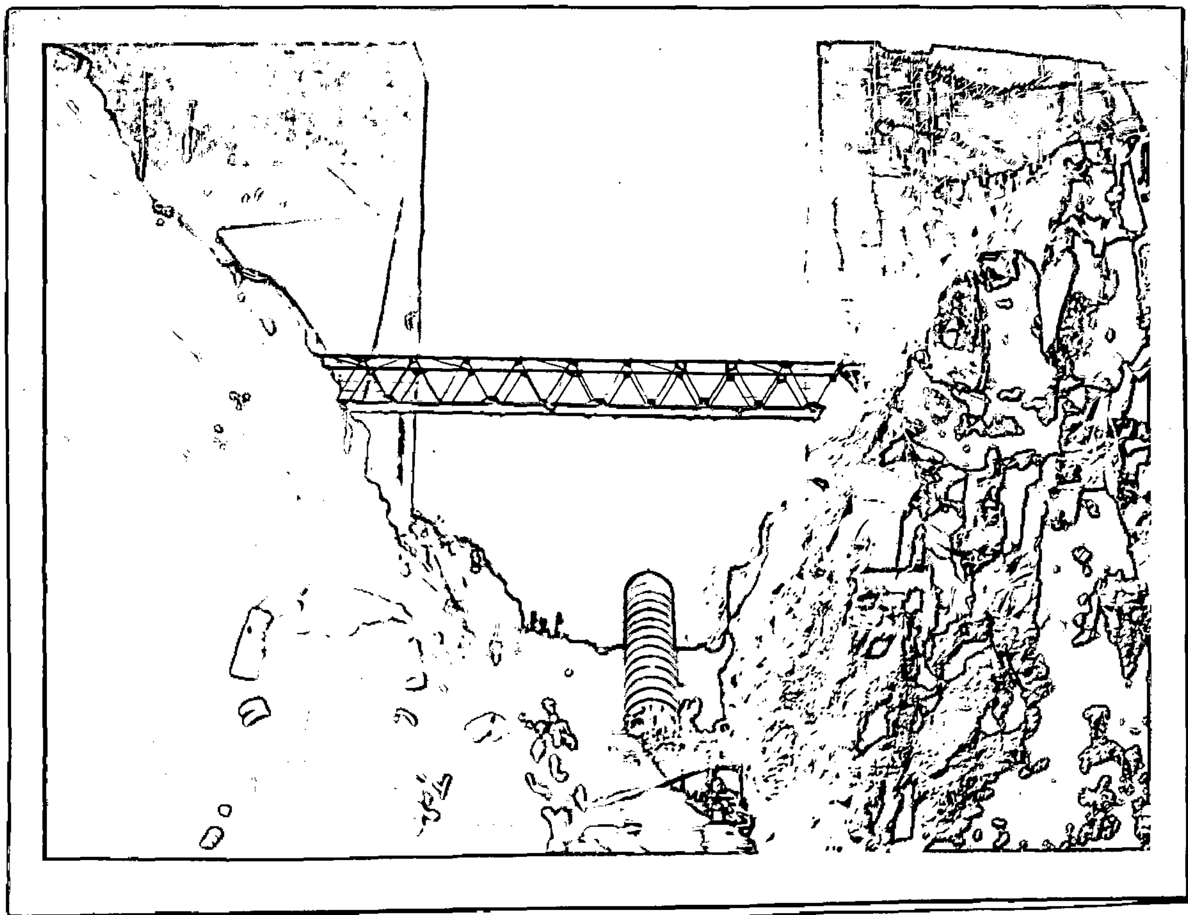
Photograph No.7 - Position of Works in the T.W.W. Cut at the end of working season 67-68



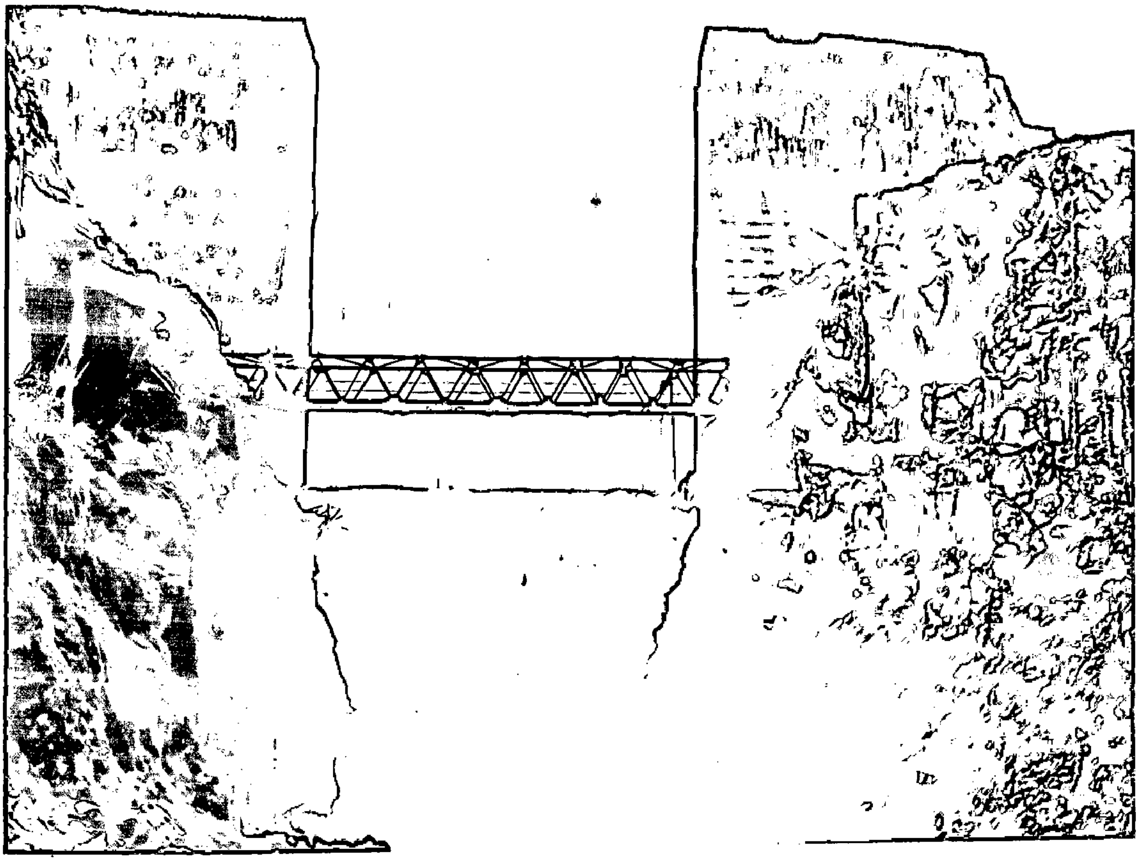
hotograph No.8 - Treatment of Volcanic Breccia Layer in the Upstream portion of the Temp. W.W. Channel - as on 30-5-68



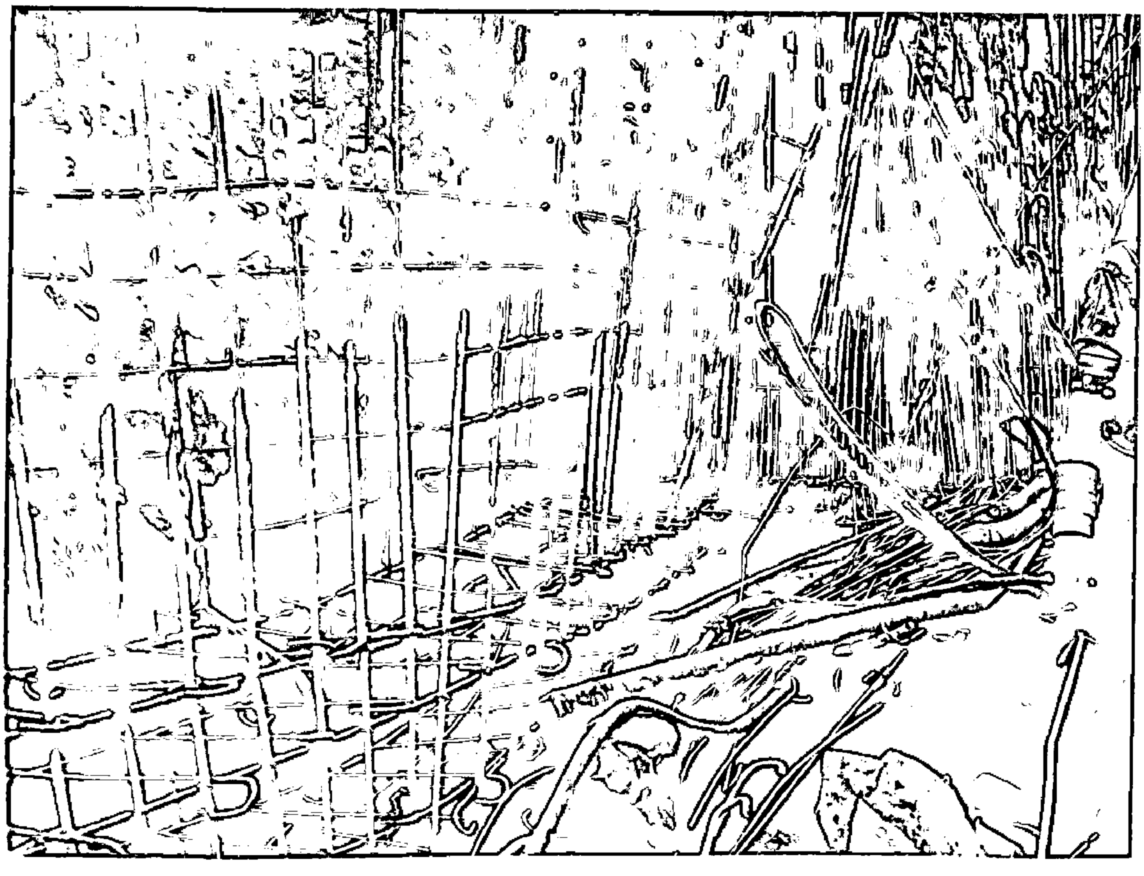
Photograph No.9 - Work of Installation of 10' dia penstock in progress as on 20-1-69



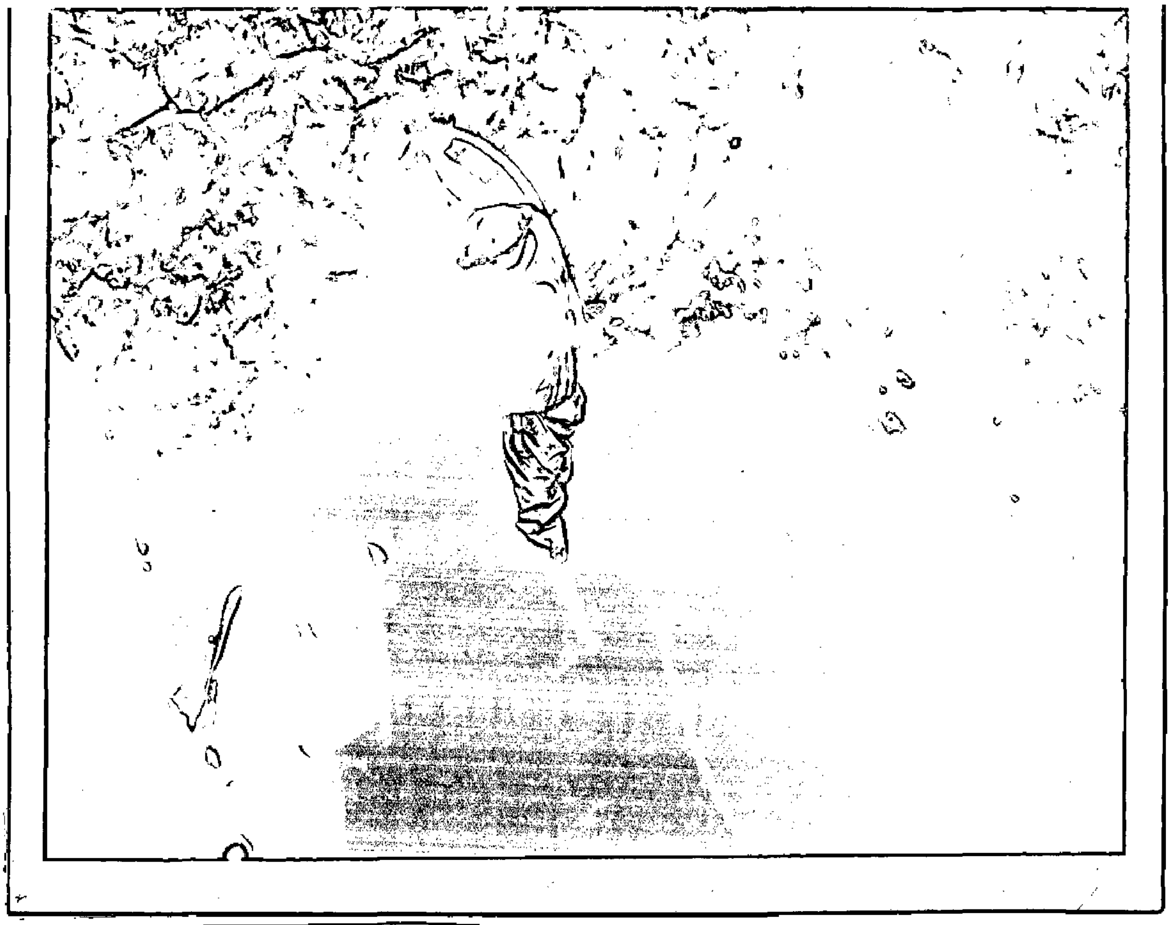
hotograph No.10 - Work of Installation of 10' dia penstock in



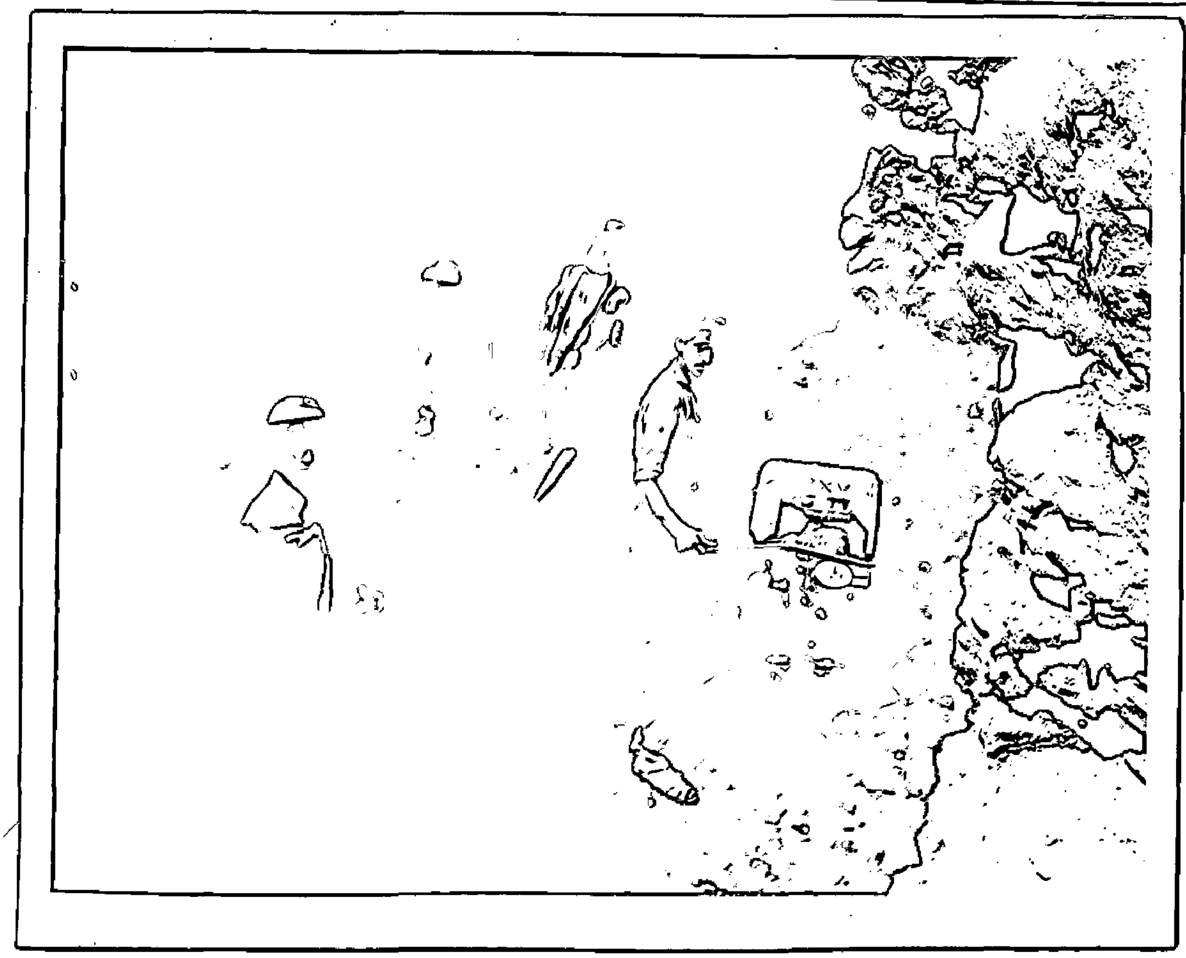
Photograph No.11 - Spilling Arrangements at the end of 1968-69 Season as on 16-7-69



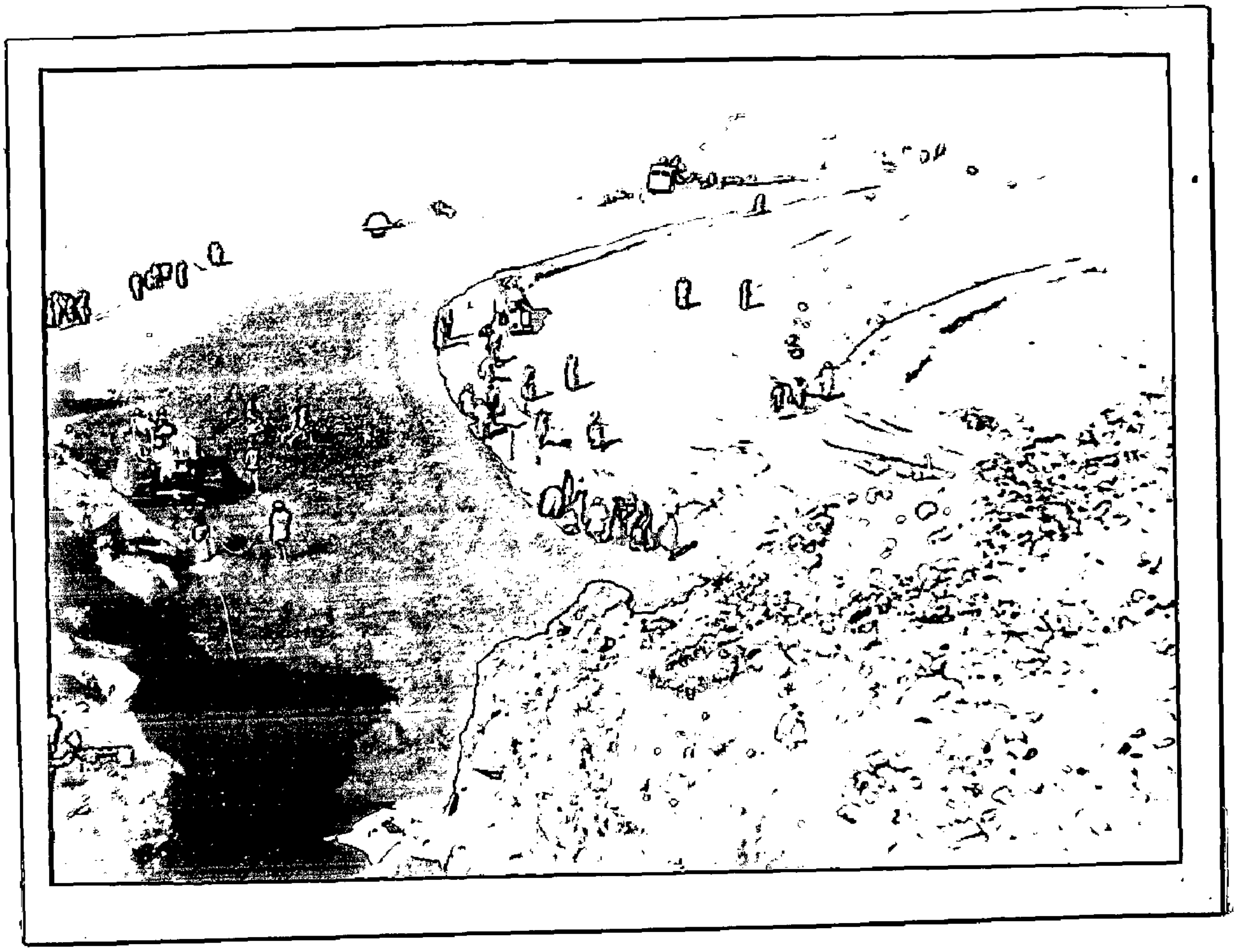
hotograph No.12 - Treatment of Volcanic Breccia Layer in the Dam portion of the T.W.W. Monolith



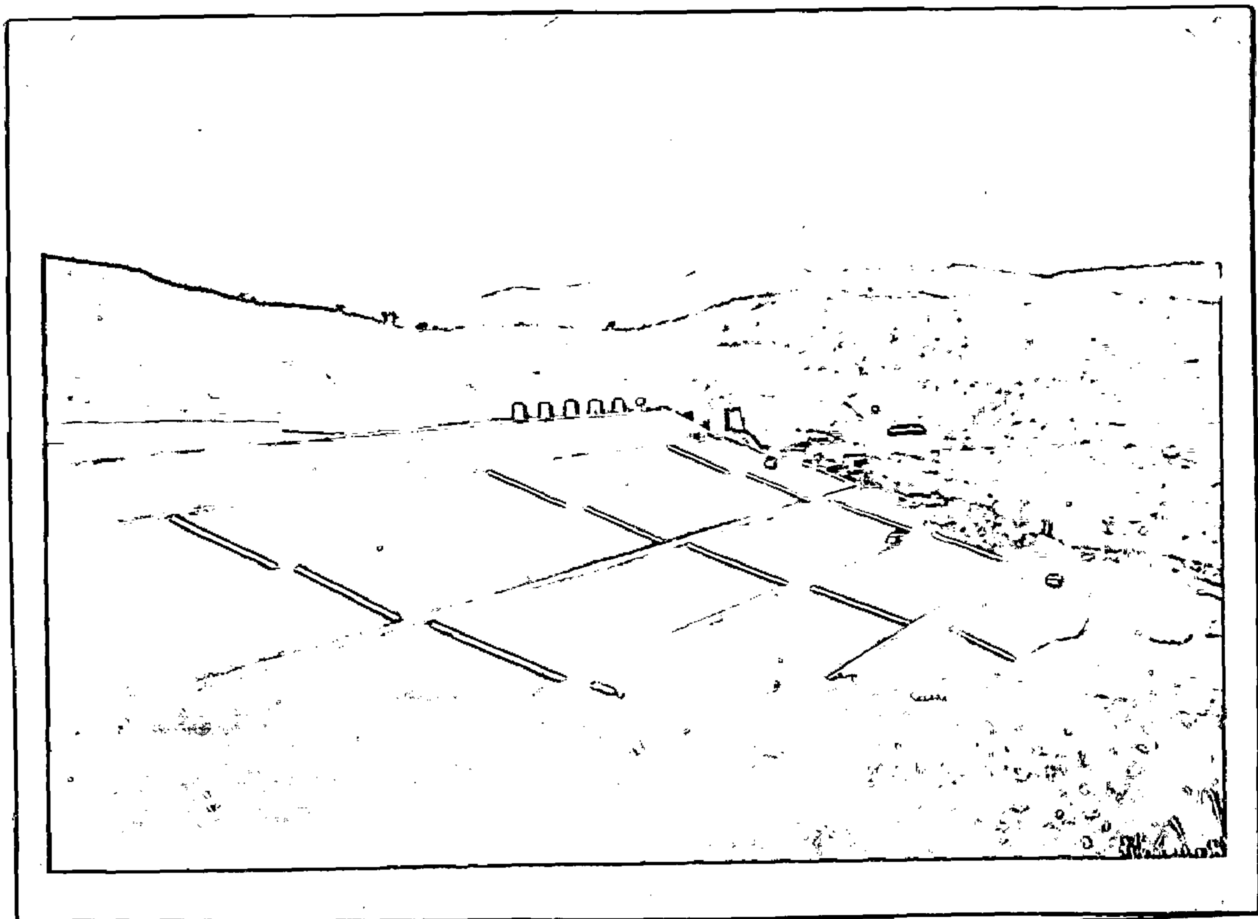
Photograph No.13 - Work of Backfilling the last monolith by impervious soil in progress as on 2-4-69



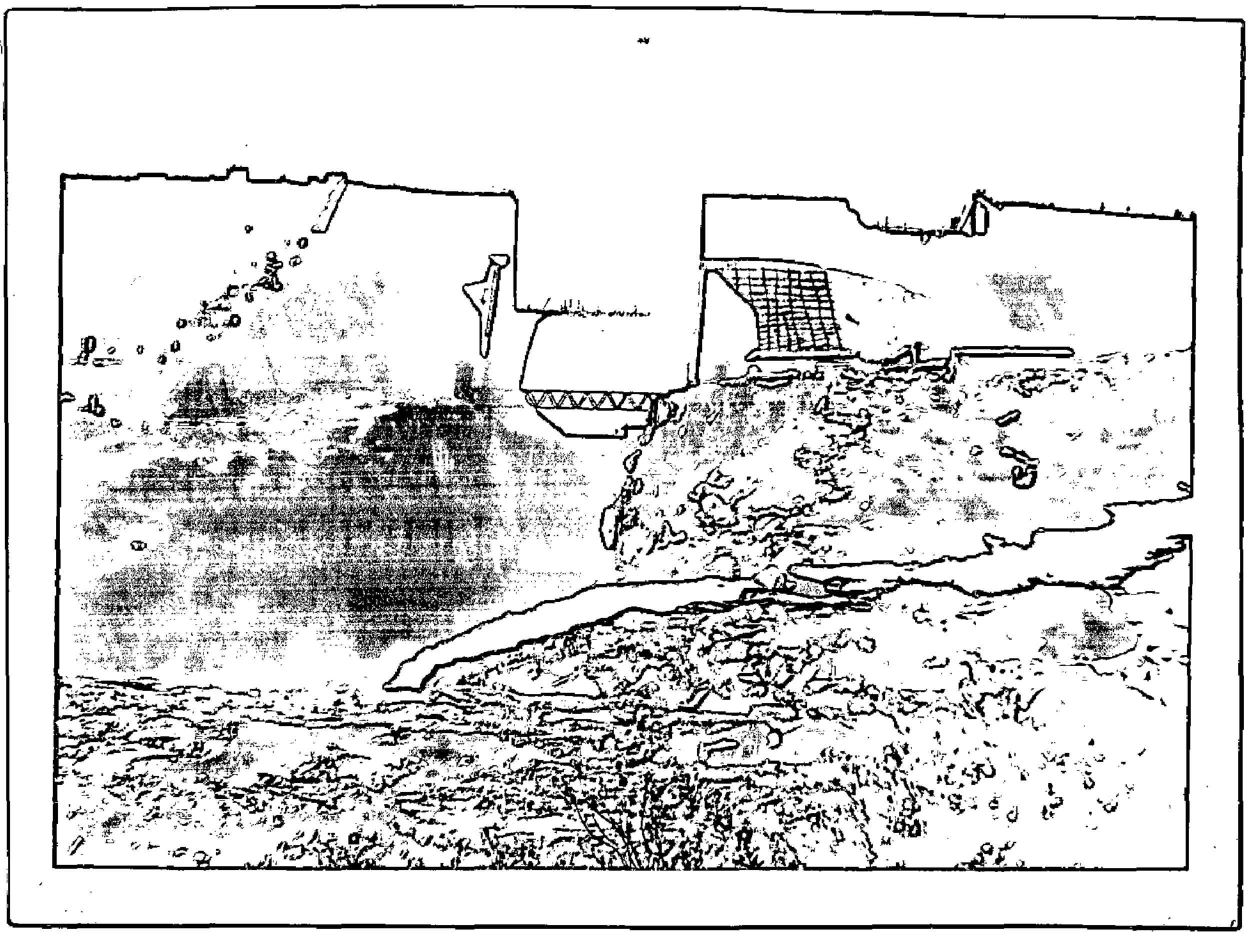
Photograph No.14 - Work of Backfilling the last monolith by impervious soil in progress as on 2-4-69



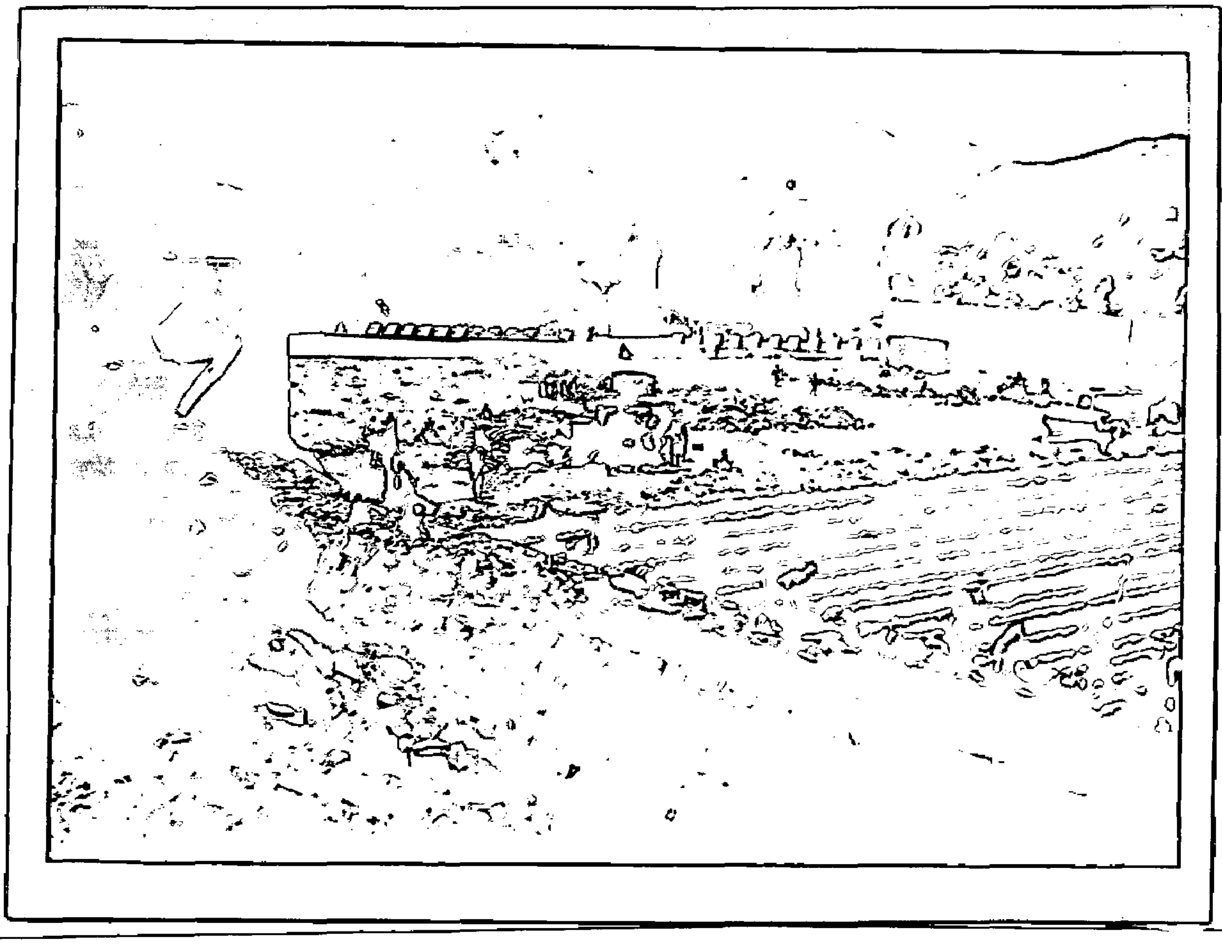
Photograph No.15 - Work of Earth Dam in Progress as on 28-1-70



Photograph No.16 - View of Earth Dam from the Downstream side as on 23-10-70



Photograph No.18 - Work of Spillway & its protection arrangement in progress as on 2-4-71



Photograph No.17 - Stage of Works at the end of working season

APPENDIX NO.1

EXTRACT FROM THE KHADAKWASLA PROJECT 1957 REPORT

STATEMENT SHOWING THE ROUGH COST OF EARTHEN DAMS AT PANSHET AND WARASGAON COMPARED WITH MASONRY DAM AT KHADAKWASLA PREPARED BY THE EXECUTIVE ENGINEER, IRRIGATION PROJECTS DIVISION (CENTRAL) POONA

ABSTRACTS OF COSTS IN RUPEES

.No (1)	Items (2)	Warasgaon Dam (3)	Panshet Dam (4)	Total of Warasgaon & Panshet (5)	New (masonry) Dam at Khadakwasla (6)
1.	'A' Preliminary Expenses	2,25,000	2,42,000	4,67,000	1,44,777
2.	'B' Land	26,72,000	26,27,000	52,99,000	53,28,272
3.	'C' Works	1,65,81,695	1,86,94,992	3,52,76,687	6,16,38,100
4.	'K' Buildings	13,26,536	14,95,600	28,22,136	28,37,687
5.	'L' Earthwork	-	-	-	11,46,278
6.	'M' Plantation	40,000	40,000	80,000	40,000
7.	'O' Miscellaneous	4,50,000	4,50,000	9,00,000	8,93,293
8.	'P' Maintenance	1,79,082	2,01,906	3,80,988	6,52,880
9.	'Q' Spl. Tools & Plants	9,94,902	11,21,700	21,16,602	13,72,475
	Total	2,24,69,215	2,48,73,198	4,73,42,413	7,40,53,762
	Say			Rs.4.75 crores	Rs.7.40 crores
10.	Strengthening & improving existing Khadakwasla Dam & bringing it to standard			Rs.1.00 "	
	Grand Total			Rs.5.75 "	Rs.7.40 crores

APPENDIX - II

DESIGN ASSUMPTIONS FOR THE MASONRY DAM

1.0. The following loads have been considered for the design of the masonry dam:-

(i)	Weight of masonry	150 lbs/cft	
(ii)	Weight of water	62.5 "	
(iii)	Weight of silt	125 "	(assumed upto R.L. 1950.0)

(iv) Horizontal water pressure

(v) Horizontal silt pressure

(vi) Uplift pressure - on 100% area factor equivalent to the head of water at the upstream toe and zero at the downstream toe.

(vii) Wave pressure (for a wind velocity of 80 m.p.h)

(viii) Wind pressure

(ix) Earthquake force on masonry and water (Acceleration 0.1g)

2.0. Two different dam sections, one for the portion outside the T.W.W. cut and the other for the portion in the T.W.W. cut in which penstock and irrigation outlets are situated have been designed to satisfy the following conditions:-

(i) No tension in the section with water at H.F.L. of 2069.0 with no earthquake forces.

(2) No tension in the section with water at F.R.L. of 2086.00 and an earthquake of light intensity = 0.02g both horizontal and vertical)

(3) Maximum tension of 20 p.s.i. at upstream heel and 30 p.s.i. at D/S toe for full reservoir condition and lake empty condition respectively with maximum earthquake forces (= 0.1g both horizontal and vertical).

The stresses, coefficient of friction and shear friction factor are given in the following table. (next page)

Panshet

S.No.	Location	Stresses at R.L.	Loading Condtion	Stresses in kips/sft U/S Heel	Stresses in kips/sft D/S Toe	Coeff. of friction	Shear friction factor	REMARKS
1.	Section in T.W.W. cut	1930.00	1) U/S water at H.F.L. & no earthquake	+2.28	+12.90	0.74	6.00	
			2) U/S water at F.R.L. & earthquake force 0.02g	+2.17	+13.20	0.755	5.75	
			3) a) U/S water at F.R.L plus earthquake force with 0.01g	-1.92	+17.25	0.95	4.55	
			b) U/S water at L.W.L plus earthquake force with 0.01g	+26.60	- 3.34			
2.	Section outside T.W.W. cut	1997.00	1) U/S water at H.F.L. & no earthquake	+2.03	+10.16	0.593	9.00	
			2) U/S water at F.S.L. plus earthquake with 0.02g	+2.28	+12.04	0.59	8.95	
			3) a) U/S water at F.R.L plus earth =0.1g	-0.10	+12.42	0.74	7.15	
			b) U/S water at L.W.L plus earthquake force with =0.01g	+16.73	-0.93			

APPENDIX - III

DESIGN ASSUMPTIONS FOR THE DIVIDE WALL

ASSUMPTIONS

(i)	Weight of masonry	145 lbs/cft
(ii)	Weight of water	62.5 "
(iii)	Coeff. of friction (min)	0.6
(iv)	Shear friction factor	5
(v)	Shear strength	13T/sft

The design background for each zone of the divide wall is detailed below:-

ZONE I

This section is to retain upstream casing.

(i) Length:

Between toe of casing to upstream toe of hearting

(ii) Design Section:

Just upstream of the toe of hearting where the maximum height i.e. about 60 ft.

(iii) Design Condition:

Sudden draw down condition of earth dam assuming that upper half height of the casing material is drained and the bottom half is in bouyant condition.

(iv) Weight of Casing:

127 lbs/cft (buoyant Wt 64.5 lbs/cft)

(v) K_n for casing : 0.6 acting at 0.4h

(vi) Water pressure:

Due to height of water equal to bottom half height of casing.

(vii) Uplift:

Due to height of water equal to bottom half height of casing.

ZONE II

This section is to retain mainly upstream hearting material.

(i) Length:

Upstream toe of hearting zone to U/S key wall

(ii) Design Section:

Just upstream of key wall where there is maximum height i.e. about 100 ft with 85' hearting and top 15' casing.

(iii) Design Condition:

Sudden draw down in the earth dam.

(iv) Weight of casing:

127 lbs/cft (assumed drained)

(v) Weight of hearting:

120 lbs/cft (57.6 lbs/cft in buoyant condition)

(vi) K_n for casing:

0.6 acting at 0.4h

(vii) Kn for hearting:

0.7 acting at 0.4h

(viii) Water pressure:

Due to weight of water equal to hearting

(ix) Uplift:

Due to weight of water equal to hearting.

ZONE III Section Between Key-Walls

As there is masonry backing from other side this section is not designed. However, a slope of 0.2:1 has been provided on the earth dam side and on the masonry dam side (in the key portion) the same slope as in zone II or IV has been continued.

ZONE IV Section Against Downstream Hearting :

(i) Length:
Between the downstream key wall to D/S toe of hearting

(ii) Design Section:
The design section is taken at 36 feet D/S of centre line of dam where there is no masonry dam backing from the other side and maximum height of hearting is retained by the section

(iii) Design Condition:
The casing is drained and hearting is in buoyant condition.

(iv) Weight of Casing :
127 lbs/cft

(v) K_n for casing:
0.6 acting at 0.4h

(vi) Weight of Hearting:
120 lbs/cft (57.5 lbs/cft buoyant)

(vii) K_n for Hearting:
0.7 acting at 0.4 h

(viii) Water pressure:
Due to the height equal to the height of hearting.

(ix) Uplift:
Due to the height equal to the height of hearting.

ZONE V Section against Downstream Casing:

(i) Length:

Between D/S toe of hearting to D/S toe of casing.

(ii) Design Section:

Just downstream of hearting toe where height is maximum

(iii) Design Condition:

Heavy rainfall condition

(iv) Weight of Casing:

127 lbs/cft

(v) K_n for Casing:

0.4 acting at 0.33h (Immediately behind downstream casing there is gravel filter behind which there is 10' rock filling backed by selected murum filling (Photograph No. 6)

(vi) Water Pressure; Nil

(vii) Uplift Pressure: Nil

Weep Holes

In order to facilitate drainage in the casing zone, 4" dia A.C. pipes 10' centres bothways staggered, have been provided in the masonry divide wall .

BIBLIOGRAPHY

(1) Report of the Commission of Inquiry for Enquiring into the failure of the Panshet and Khadakwasla Dams - Part I
GOVT. OF MAHARASHTRA - 1963.

(2) Journal of the Institution of Engineers (India) Volume 47 - Number 11, Part CI 6 - July 1967

(3) Journal of the Institution of Engineers (India) Volume 48 - Number 5, Part CI 3 - January 1968

(4) Earth Manual - U.S.B.R.

(5) Design of Small Dams - U.S.B.R.

(6) Earth and Rockfill Dam Engineering - George F. Sowers

(7) Engineering for Dams - Vol.III, Creager, Justin & Hinds

(8) Earth-Rock Dams: Engineering Problems of Design & Construction Sherard & Others

(9) Lessons for Dam Disasters - Engineering 15 May 1964

(10) Fundamentals of Irrigation Engineering - Bharat Singh

(11) Top Most Dams of the World - Cementation Co. Ltd., London

(12) Ninth International Congress on Large Dams - Q.34, Sept. 1967

(13) Proceedings Symposium on Earth & Rockfill Dams Department of Civil Engineering, University of Roorkee December 1961