PROVISION OF CUTOFF IN TENUGHAT DAM AND ITS PERFORMANCE

A DISSERTATION
submitted in partial fulfilment of the
requirements for the award of the Degree
of
MASTER OF ENGINEERING
in
WATER RESOURCES DEVELOPMENT

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Grateful thanks are also due to the various authors and agencies whose literature has been of help in bringing out this dissertation.

21.7.75

Ayodhya Prasad

C E R T I F I C A T E

Certified that the dissertation entitled "Provision of Cutoff in Tenughat Dam and Its Performance" which is being submitted by Sr. Ayodhya Prasad in partial fulfilment of the requirements for the award of the Degree of Master of Engineering in Water Resources Development of the University of Roorkee is a record of the student's own work carried out by him under my supervision and guidance. The matter embodied in this dissertation has not been submitted for any other degree or diploma.

Certified further that Sr. Prasad has worked for a period exceeding nine months in connection with the preparation of this dissertation.

Bharat Singh

Dr. Bharat Singh
Professor Design
W.R.D.T.C.
(INTERNAL GUIDE)

Roorkee -)
21st July 1975)

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C O N T E N T S

SYNOPSIS

Tenughat Dam built across river Damodar is a water supply project for Bokaro Steel Plant and other industries around the steel plant. The dam is 6460 m long and over 50 m high above the river bed. No dam in this area was possible without crossing the faulted junction of the sedimentaries and the metamorphics, and the Tenughat Dam had therefore to be built across this fault.

The cutoff for the dam in the river portion had to be provided through a sand stratum mixed with pebbles and at places boulder stratum of 5 m-to-15 m thickness. On test, river-bed sand was not found prone to liquefaction or excessive settlement.

Various alternatives of providing an effective foundation cutoff in the river bed were examined and a double-row concrete diaphragm wall with sand inbetween grouted, was adopted from time and economy consideration. The upstream diaphragm wall was extended 33.6 m into the right bank and 134 m into the left bank to provide effective bondage between open trench cutoff and diaphragm wall.

Long-term performance data for such diaphragms are not available. Therefore, adequate instrumentation of the dam and the diaphragm was done and data observed. In this dissertation, a brief description of the site and construction of the diaphragms has been given. The available performance data have been analysed and it has been concluded that the concrete diaphragm walls are functioning satisfactorily.

Chapter 1

PROJECT DESCRIPTION

1.1. General

Damodar basin contains India's largest coal deposits in the lower basin and other minerals like mica, lime stone, bauxite and graphites in abundance in the upper valley. The adjacent basin Subarnrekha contains iron ore and other precious minerals like uranium and copper. All these mineral deposits have immense prospect for rapid industrialisation of the area which will require power and water in abundance. Damodar basin has limited hydro power potential most of which has already been tapped by D.V.C. but there is ample scope for thermal generation at low cost from the coal available in the basin. Damodar Valley Corporation has installed capacity [1] of 1035 MW thermal in addition to 104 MW hydel power in the grid. In this area, the Bihar State Electricity Board has also one thermal power station of 400-MW capacity under extension to 800 MW during the fifth Five-Year Plan and another upcoming thermal power station of 1000-MW capacity just 4 km downstream of this dam.

1.2. D.V.C. Proposals

Mr. Voordwin, a TVA expert, had recommended construction of a series of eight dams [2] for checking recurrent floods in lower Damodar basin at Aiyar, Bermo and Panchet Hill on river Damodar, Tilaiya, Balpahari and Maithon on its principal tributary Barakar, and at Konar on river Konar, and at Dania on Bokaro

Nala, its two other tributaries (Fig. 1.1).

1.2.1. The D.V.C. completed four dams, viz. Panchet Hill dam on river Damodar, Tilaiya and Maithon dams on river Barakar and Konar dam on river Konar. Rest of the four dams proposed were shelved.

1.3. Tenughat Dam Proposal

With decision to locate the fourth steel plant in public sector at Bokaro, for meeting its industrial water requirement, a dam at Aiyar on the main river was needed. But the site has a disadvantage of submerging approximately 22 million tonnes of metallurgical coal in addition to non-availability of good earth within the economic lead. Hence, the site had to be shifted to Tenughat which does not have these two shortcomings of the upper site [3].

1.3.1. Location: Tenughat dam site is located near village Tenu (23°44'N, 85°50'E, 73 E/14) about 24 km west of Bokaro Steel Plant. The nearest railway station is Gomia, about 8 km north of the site on South Eastern Railway.

1.3.2. Water Available: The damsite intercepts a catchment of 4430 km^2 (1730 sq. miles) and the dependable lean year flow of approximately 1850 million m^3 is available for storage. The dam will have a gross storage capacity of 1150 million m^3 at its full height at the end of stage II and a live storage capacity of 816 million m^3.

1.3.3. Salient features: The dam with a maximum height 50 m is rolled, zoned earth dam, 6460 m long at formation level

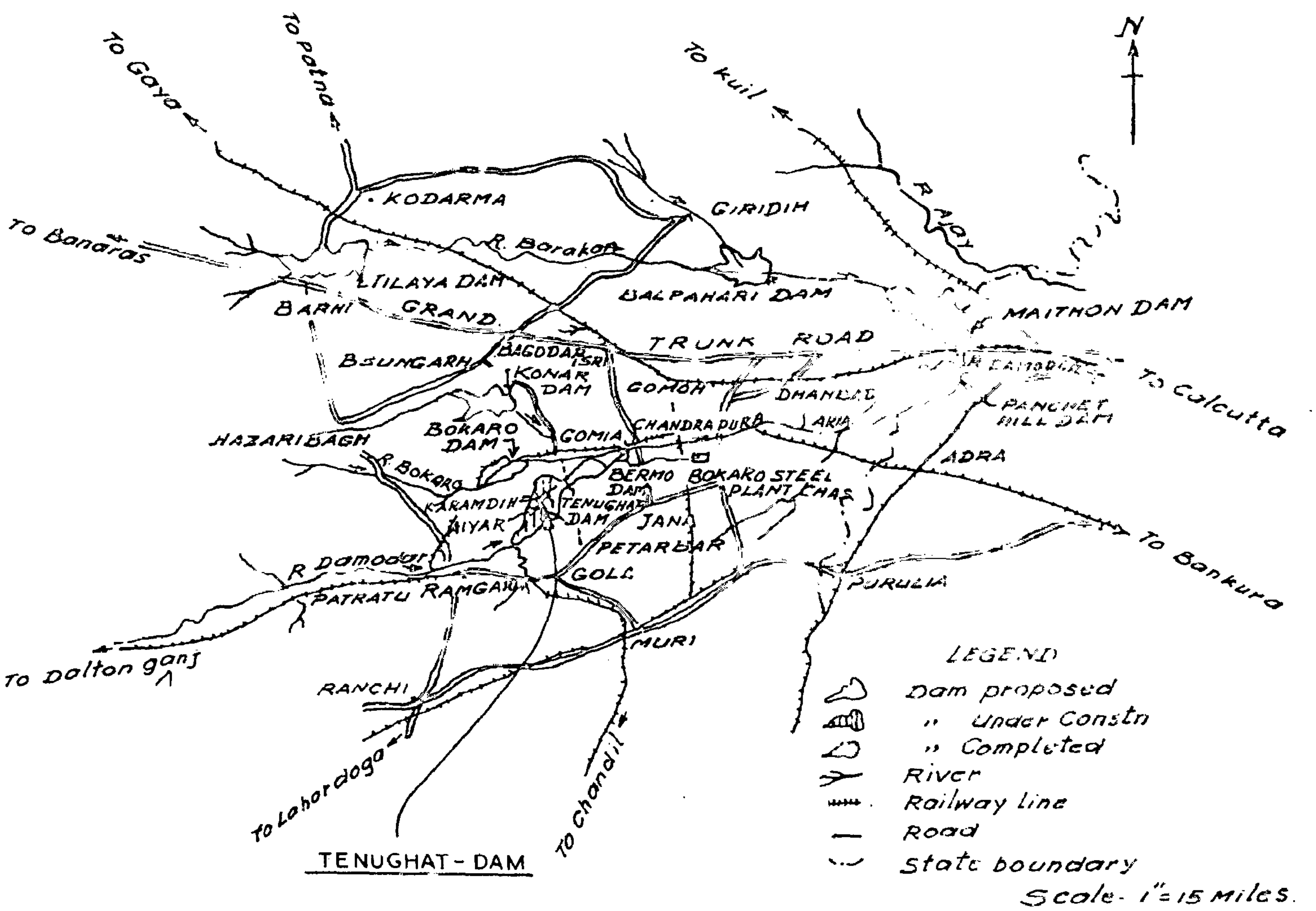
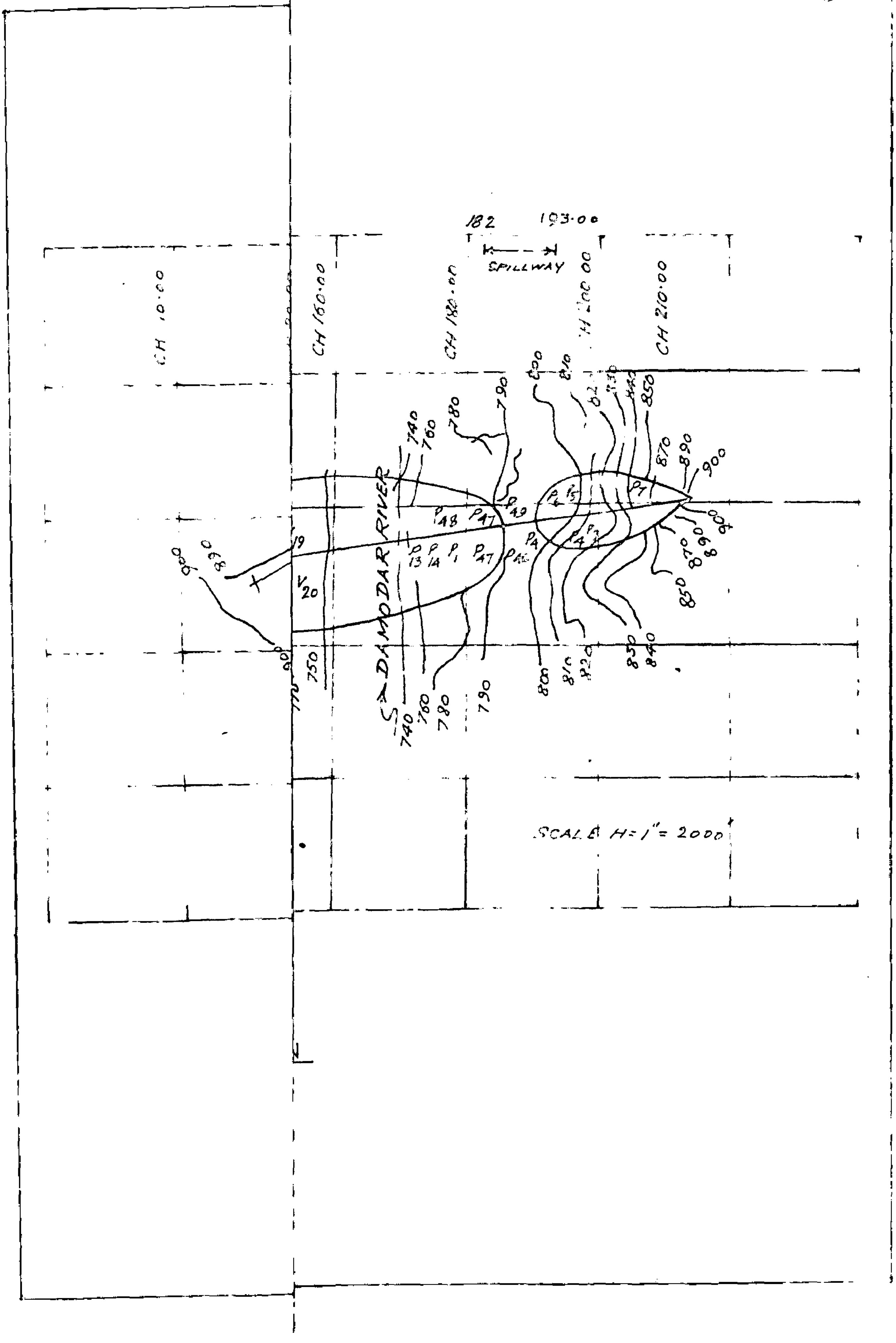


FIG. 1·1_LOCATION PLAN

273.00 m. The site is located in hilly tract and the valley is quite wide to meet the required storage at comparatively low height. The alignment is straight between stn. 3048 m to 6460 m and between chainage 0 to 3048 m follows the topography (Fig. 1.2). The masonry spillway 344.6 m in length with 10 nos. of crest gates of 15.1 x 12.4-m size, have spilling capacity of 18000 cumecs, is located between chainage 5597 m to 5941 m.

The earthen dam has a trizonal section with slanting central impervious core flanked by semipervious casing both on upstream and downstream with 3 m thick sand chimney filter, sandwiched between central core and downstream casing. The sand chimney is connected to a 1.5 m thick blanket filter laid below the downstream semi-pervious casing leading to a toe drain with suitable weight-and filter.

The cutoff below the earth dam on the left and right flanks consists of backfilled rolled impervious clay in trenches dug below the seat of the dam. The river bed material consists of highly pervious alluvial deposits of varying thickness from 5 m to 15 m of different grain sizes ranging from D_{10} = 0.23 mm to pieces of boulders at places. The permeability of river bed was so high that it would have cost millions of rupees only for pumping out water while providing conventional rolled backfilled earth as cutoff. So in place of a conventional open trench cutoff in the river bed, the cutoff consists of two rows of reinforced concrete diaphragm, 323 m in length, and upstream diaphragm extensions of 134 m on the left bank and 336m on the right bank, all embedded 1 m in hard bedrock.


182
193.00
SPILLWAY
CH 10.00
CH 160.00
CH 180.00
CH 210.00
DAMODAR RIVER
SCALE H=1"= 2000'

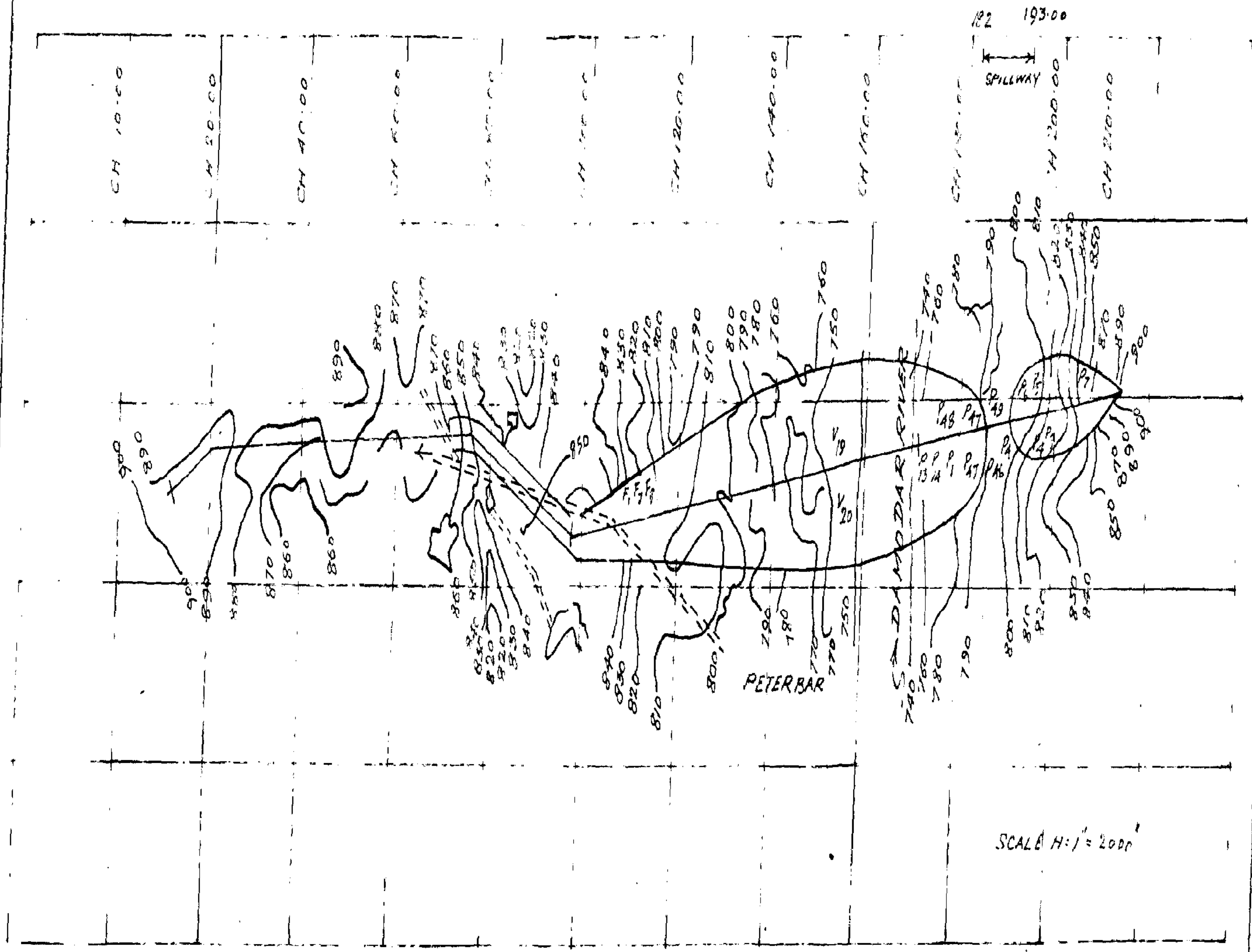


FIG. 1·2 TENUGHAT DAM PROJECT - SITE PLAN

Chapter 2

FOUNDATION INVESTIGATION : PROBLEMS

2.1. General

Detailed investigation of the present dam site was started in 1965 and had a target to supply water in December 1969 for commissioning of the 1st phase of Bokaro Steel Plant. The construction work was started on preliminary geological investigations on the 14th of March 1965. The preliminary investigations of the dam area indicated that the basement complex of the ancient gneisses [4] prevail in the south and they are overlain in the north by rocks belonging to lower Gondwana formations. The junction between the two, metamorphics of Archaean age and the sedimentaries of lower Gondwana formation, is a faulted one and this boundary fault was a long range fault running almost along the river upto large distances. The dam has to be constructed across this fault. Based on limited borehole data, rest of the foundation was considered to be without any problem, a set of design were prepared and the construction started. A number of problems cropped up thereafter [5].

2.2. Investigation Details

Therefore, it was necessary to have much more information regarding the foundation. Drill holes at every 152.5-m (500-ft) interval below the entire length of the earthen dam were taken up. In the spillway zone, 5 rows of drill holes at closer spacings varying from 12.0 m to 30 m were taken up. These investigations were further intensified in areas indicating abnormal

-ities. The intensities increased between stn 5597 m to 5982 m, i.e. at the spillway site and 48 numbers of holes of total depth 850 metres were drilled. In addition to this, 13 more holes of 270-m depth were drilled in the spill channel area to decipher the sub-surface conditon downstream of the spillway.

The natural soils overlying the bed rock were tested for strength parameter, the river bed deposits for their relative densities, the rock permeabilities below the entire dam cut-off including spillway and strength parameter of rock under the spillway. Based on resultant permeabilities derived from insitu water percolation test (Table 2.1) and interpretations of cores, a geological profile below the entire dam was prepared (Fig. 2.1). The cores of the drilling indicated that northern part ofthe dam lay on the Barakar and Bartron Measures, which consists of tightly-packed interbanded layers of medium-grain sand stone and carbonaceous shales. In the southern part, the dam which lay over the metamorphic suite of rocks consisting of garnetiferous biotite gneisses, hornblende biotite schist and amphibolited. It wasalso found that adjacent to the boundary fault between the metamorphics and the sedimentaries, the metamorphics have suffered deep weathering which generally follow the topography of the area, being deep under the valley and shallow under the ridge.

The foundation explorations so carried in the entire length of the dam and geological profile (Fig. 2.1) prepared indicated varying problems at chainages discussed in the following paragraphs.

2.2.1. <u>Suspected cross aquifer (2561 m - 2622 m)</u>

Conventional type open trench cut-off has been provided in

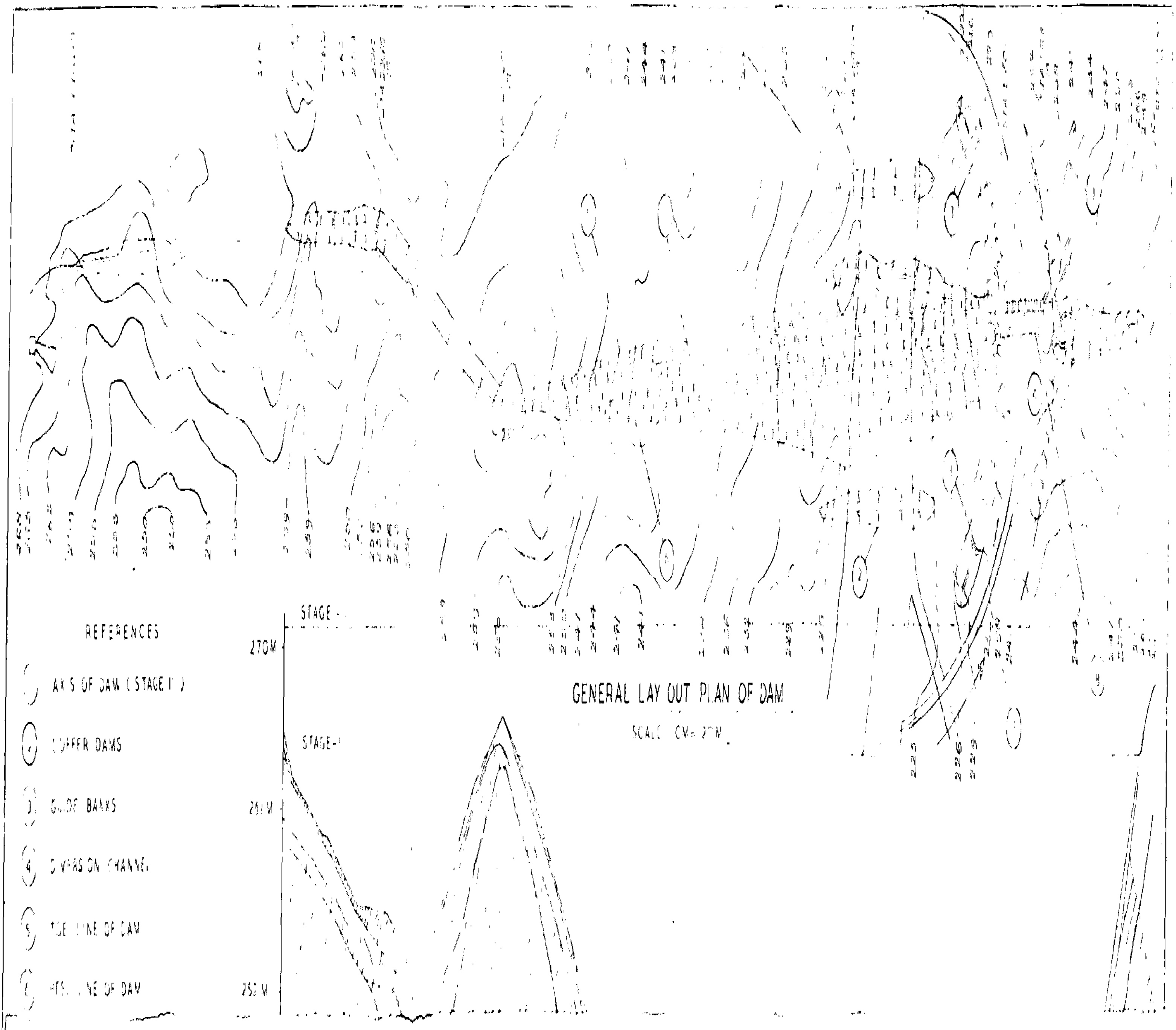

REFERENCES
STAGE-
270 M
GENERAL LAY OUT PLAN OF DAM
250 M

Table 2.3 - Foundation Permeability of Tenughat

Sl. No.	Chainage	G.L. in feet	Bottom level of key trench (ft)	H.R.P. in psi	Permeability in ft year below the bottom of the key trench: 0-10	10-20	20-30	30-40	40-50	50-55
1	2	3	4	5	6	7	8	9	10	11
1	78	842.86	850.00	17.00	155.00	170.00	170.00	170.00	110.00	40.00
2	108	855.49	842.84	11.50	9.00	4.34	2.59	53.80	2.30	1.00
3	113.47	327.00	812.00	24.00	960.00	900.00	625.50	563.00	286.00	150.00
4	113.50	327.00	812.00	24.00	960.00	616.00	289.50	164.00	-	-
5	113.72	327.00	812.00	24.00	95.00	81.00	82.00	85.00	82.00	-
6	114.50	823.00	808.00	25.00	710.00	475.00	445.00	329.00	-	-
7	115.50	814.25	799.25	29.00	320.50	377.50	322.00	250.00	125.00	-
8	116.50	805.02	789.02	33.04	475.00	487.50	412.00	302.00	-	-
9	117.87	805.00	787.64	33.08	235.00	235.00	372.00	275.00	355.00	43.00
10	118.00	805.00	786.64	33.04	235.00	235.00	172.00	124.50	-	-
11	123.00	809.48	784.78	31.00	326.50	250.00	250.00	5.30	4.50	-
12	128.00	809.84	794.66	31.40	170.00	170.00	309.50	160.50	20.00	--
13	133.00	803.07	789.38	34.30	355.00	625.00	307.00	512.25	310.00	-
14	138.00	788.06	777.44	40.60	250.00	180.00	180.00	104.50	2.33	-
15	143.00	780.44	763.80	44.16	111.50	113.00	82.00	2.00	-	-

(continued)

Table 2.1 - continued

1	2	3	4	5	6	7	8	9	10	11
16	148.00	761.52	741.57	52.4	460.00	256.00	40.00	10.00	5.00	
17	151.00	750.00	740.00	57.5	155.00	120.00	65.00	18.00		
18	153.00	745.61	725.01	59.00	55.00	8.00	-	12		
19	160.00	730.88	698.00	62.20	9.5	9.5	-			
20	161.75	725.00	692.75	63.8	14.25	19.25				-
21	165.05	735.00	692.75	63.80	17.50	17.50			-	-
22	172.00	743.00	727.40	60.04	96.50	3.00	-	-	-	-
23	176.00	763.00	742.19	50.00	23.25	2.40	-	-	-	-
24	178.75	786.00	766.40	42.00	31.50	-	-	-	-	-
25	180.50	708.82	777.21	36.20	92.00	61.50	2.00	-	-	-
26	185.67	708.82	777.21	36.20	-	-	-	-	-	-
27	188.00	778.50	-	63.80	-	-	-	-	-	-
28	192.50	779.00	-	-	-	-	-	-	-	-
29	201.00	807.20	783.00	34.70	15.00	60.00	50.00	50.00	10.00	-
30	203.00	809.19	790.90	33.80	12.00	100.00	20.00	30.00	10.00	-
31	205.00	834.96	820.00	22.60	600.00	180.00	20.00	30.00	10.00	-
32	206.30	835.00	805.00	22.00	58.00	58.00	58.00	58.00	58.00	-

this section. On backfilling the cut-off trench, percolation of water in pits dug downstream of the dam opposite station 2562 m was noticed. This created a doubt as this percolation was through an aquifer crossing the dam and opening out into the reservoir. To trace the existence of such an aquifer wells upstream and downstream at 3 and 4 locations, respectively, were dug and pumping-out tests were carried out, which indicated that while downstream wells could be dried up fully, there was no fall of levels in the upstream wells. Tracer Technique applied to these wells while pumping out, too did not indicateany connection from any of the upstream wells to those on downstream. For further checkup, the water in the existing wells in the area downstream and upstream were pumped out and water levels in the upstream wells recorded. These did not also show any fall or connection, whatsoever. The observations for water-level fluctuations in these wells were carried for about six months, which did not indicate any gradient, whatsoever, between the wells on the upstream and those on the downstream. It was, therefore, inferred that the water percolation noticed was due to the trapped rainwater. The presence of an aquifer crossing the dam was accordingly ruled out. It did not require any special treatment.

2.2.2. Boundary fault (Stn. 3448 m to 3555 m)

The boundary fault between metamorphics and the sedimentaries was completely traced and mapped to a length of approximately 457 m (1500 ft) both upstream and downstream by suitably planned vertical&inclined holes going upto 21.35-m (70-ft) depths, the examination of the cores of the drills, water-loss tests and surface examinations through drift pits.

In the metamorphics, the shattering effect was found extending upto stn 3488 m and the weathering effect upto stn 3887.5 m. The metamorphic rocks near the contact are highly decomposed, brecciated and fractured with development of sliken sides and gaugy material. Barakar sanddtone is highly recrystallised and brecciated near the contact. Sandstone and shales of Barakar and Barren Measures stages strike E-W and dip at 50° to 70° towards N.

The fault was treated by stage grouting. The area between chainage 3443 m and 3580 m was grouted in four stages of top 1.5 m, 3 m, 4.5 m and 6 m at pressure varying between 2.80 kg/cm^2 and 4.20 kg/cm^2 in a pattern 3-m equilateral triangles. The grouting brought the permeability from 10^{-3} cm/sec to less than 10^{-4} cm/sec. This was tested by post-water losses tests and drilling grouted rock mass with double core barrel which showed substantial grout recoveries in the cores, indicating satisfactory grout permeations.

A doubt, however, in regard to the reactivisation of the fault, due to earthquake arose. For an appraisal of these aspects of the problem, various reports in regard to seismicity of the area were studied, the most inmportant out of which is the report of Dr. J. B. Auden [5] (1948) who made particular studies of earthquakes in relation to Damodar basin. Dr. Auden expressed that Damodar basin was a part of stable peninsula and had not undergone faulting and folding since Mesozoic era, although it had been subjected to regional uplift. The basin feels only the fringe effects of earthquakes originating in seismically active areas. The faulting in Gondwanas are are believed as old fractures which have become part of the pattern of the peninsula. There is no evidence of recent activities along these faults.

Two of the recent great earthquakes, namely the one of the 15th January 1934, originating near Bihar-Nepal border, and the other of the 12th June 1897 in Assam, were in seismic zone and about 250 km away. No record of damage to structural installations such as buildings and bridges during these earthquakes in the area is available. All these lend support to the inference that the faults are not likely to reactivise. However, the Koyna earthquake of the 11th December 1967 recorded after filling of the reservoir, hasled to rethinking with regard to earth tremors occurring in peninsular shield. The recent observation of a couple of shocks at another dam site on the shield mass at Balimela dam in Koraput district of Orissa leaves no doubt that these areas cannot be assumed to be seismically inactive.

This led to investigation of tremors experienced in the past and measures to be taken. The matter was referred to the Director, School of Research & Training in Earthquake Engineering, University of Roorkee, to carry out investigationfor collection of data of past earthquake in the area around Tenughat. The report of Dr. Jai Krishna [6], Professor & Director, and Dr. L. S. Srivastava, Reader in Applied Geology in the School, dated July 1970, contains their recommendations. Extracts from the report are placed as Appendix I. One of the recommendations was to install a set of seismological stations in a triangular grid, this however has not been done so far.

2.2.3....

2.2.3. <u>Shear Zone (stn. 4060 m)</u>

During exploration of foundation at stn. 4060 m sticky gray materials were detected at a depth of 16.5 m below the N.S.L. This led to a suspicion of a shear plane crossing the dam. The extent of this shear plane along the key trench has to be known. So two holes at stn. 4063.06 m and 4056.94 m (10 ft both way) were drilled. The drilled hole at stn. 4063.06 (ch. 133.90) indicated presenceof clay while the other hole did not indicate the presence of shear zone. Further exploratory holes were drilled to establish the exact width of shear plane which indicated only 3-m width. Permeability tests carried through this zone indicated permeability from 2.5×10^{-4} cm/sec to 6.5×10^{-4} cm/sec. As a remedial measure, grouting from stn. 4030 m to stn. 4090 m with grout holes in 3-m equilateral triangle pattern was done and permeability cut down below 2×10^{-5} cm/sec. from 6.5×10^{-4} cm/sec.

2.2.4. <u>Artesian stratum (Stn. 4520 m)</u>

At stn. 4520 m, artesian water was detected from the drill holes. The artesian head and discharge were measured which were found to be 1.53 m and 30 cc/sec., respectively. In order to trace the width of the artesian area along the cut-off and its sources, additional drillings in the key trench upto 15-m depth below the bottom of the key trench were done and the rock permeabilities were determined. The rock permeabilities were found ranging from 4×10^{-5} cm/sec. to 4.6×10^{-4} cm/sec. upto a depth of 6 m from the bottom of the key trench; and below 6-m depth, the promeability dropped down to only 5×10^{-6} cm/sec. The examination of the cores of the drill holes, the permeabi-

-lity of rock strata as also the rock exposures around the key trench indicated that the source of artesian water were the interstices in the broken pegmatites whose fractures continued only upto 6 m below the bottom of the cutoff. The width upto which the artesian effect persisted along the key trench was found to be only 10 m starting from stn. 4520 m. The remedial measures consisted of grouting of broken pegmatites after balancing the artesian heads. The permeabilities were thus reduced to less than 3×10^{-5} cm/sec.

2.2.5. Weak Clay Tongue (Stn. 4760 m to 4945 m)

The existence of a weak clay tongue crossing the entire width of the dam between the above chainages was detected. This weak clay tongue was found protruding in the river bed adjacent to the left bank, upto a width of 61 m. The total width of the weak clay tongue on the downstream was 183 m which thinned out to only 15 m on the upstream end of the dam. The tongue was found to be varying in thickness from 1.5 m to 4.5 m and occurring at 1.5 to 4.25 m below the normal soil level. The densities and shear parameters of the weak clay were determined by laboratory tests on undisturbed samples collected from various locations and by vane shear tests in the field. Tests were also performed in the laboratory for determining consolidation characteristics of the tongue. Test results of two of such samples shown in Tables 2.2, 2.3, 2.4 indicated that the existing densities of the weak clay varied from 1.506 gm/c.c. to 1.85 gm/c.c. and the moisture content ranged from 29.23% to 33.4%.

In view of the fact that the weak clay tongue existed at depth varying from 1.5 m to 4.25 m from natural soil surface, it

Table 2.2

S. No.	Location	N.M.C. %	N.D.D. lbs/cft	Remoulded M.C.%	Remoulded D.D.	Shear Parameters Effective C	Ø	Undisturbed C	Ø
1	2	3	4	5	6	7	8	9	10
1	Ch.157.00, 200 ft d/s R.L. 730.5 to R.L.729.20	33.4	85.4	29.01	84.80	Crumbles		1.5	15°
2	Ch.159.00, 400 ft d/s R.L.729.00 to R.L. 728.00	29.23	90.00	29.3	90.0	Do.		0.75	15°

Table 2.3 - Consolidation Tests on Undisturbed Samples collected from Ch. 157.00, 200 ft d/s, R.L. 729.20 to 730.50

Applied pressure	Void Ratio	mm^2/gm	MV mm^2/sec	CV mm^2/sec	% Consolidation	γd gm/cc	Permeability K cm/sec
1	2	3	4	5	6	7	8
0.025	0.758	-	-	-	-	1.506	-
0.250	0.713	0.02	0.0117	0.189	2.670	1.546	2.21×10^{-7}
0.500	00.691	0.0088	0.0052	0.331	3.830	1.565	1.72×10^{-7}
1.000	0.661	0.0063	0.0030	0.233	5.550	1.59	8.40×10^{-8}
2.000	0.626	0.0033	0.002	0.198	7.550	1.628	3.90×10^{-8}
4.000	0.585	0.002	0.001	0.600	9.900	1.670	6.00×10^{-8}
8.000	0.540	0.0011	0.0006	0.326	12.450	1.720	1.90×10^{-8}
4.000	0.544	0.0001	-	-	-	1.715	-
2.000	0.548	0.0002	-	-	-	1.71	-
0.25	0.564	0.0009	0.00057	-	- -	1.69	-
0.025	0.580	0.0070	.004	-	-	1.677	-

Pre-consolidation Load P_c = 1.45 kg/cm^2

M_v = Coefficient of volume compressibility in mm^2/sec

C_v = Coefficient of consolidation in mm^2/sec

γd = Dry density in gm/cc

Table 2.4 - Consolidation Tests on Undisturbed Samples collected from Ch.159.00, 400 ft d/s, R.L.728.00 to 729.00

Applied pressure	Void Ratio	mm^2/gm	M_v mm^2/sec	C_v mm^2/sec	% Consolidation	γd gm/cc	Permeability K cm/sec
1	2	3	4	5	6	7	8
0.025	0.658	-	-	-	-	1.635	-
0.25	0.625	0.015	0.0092	0.116	2.00	1.67	1.06×10^{-7}
0.500	0.605	0.008	0.0050	0.103	3.190	1.69	5.15×10^{-8}
1.000	0.578	0.0054	0.0034	0.166	5.800	1.72	5.65×10^{-8}
2.000	0.546	0.003	0.002	0.690	6.700	1.755	1.38×10^{-7}
4.000	0.504	0.002	0.0013	0.2998	9.060	1.80	3.90×10^{-8}
8.000	0.464	0.001	0.00068	0.357	11.660	1.85	2.4×10^{-8}
4.000	0.468	0.0001	-	-	-	1.845	-
2.000	0.472	0.002	-	-	-	1.84	-
0.250	0.487	0.0009	0.0006	-	-	1.825	-
0.025	0.502	0.007	0.005	-	-	1.805	-

Table 2.5 - Relative Densities of River Bed Sand

Sl.No.	Location Ch.	Depth in m	N_c kg/cm^2	Relative Densities As per Terzaghi	Relative Densities As per Gibbs & Holtz
1		2.33	28	25	57
2	161.00	4.33	41	35	67
3		5.33	50	38	67
1		2.70	38	32	65
2		3.70	42	35	65
3	162.00	5.70	48	39	64
4		8.70	60	45	65
5		9.70	64	47	65
1		2.80	44	36	67
2		3.80	48	39	68
3		5.80	56	43	68
4	163.00	6.80	60	45	68
5		8.80	68	49	68
6		10.80	76	52	68
7		11.80	82	55	68

was neither economical nor feasible within the time limits to remove it out and backfill with suitable material. In order, therefore, to improve the density, the shear properties, it was necessary to drain out excess water from the tongue through consolidation process. The remedial measures adopted, therefore, consisted of 25-cm dia. vertical drainage holes drilled across the weak clay tongue at intervals of 3 m, 3.65 m, 5.5 m, depending upon the depth and drainage faces, and backfilling the drainage holes with selected sand filter and connecting them to the base of filter. Piezometers have been installed at selected intervals to watch thedrainage process as the load of dam over the weak claly buildsup. During first two years of successive partial loading by dam fill, it has been observed that the moisture content has been reduced from 33% to about 23%.

2.2.6. River bed sand

The river-bed deposit consists of loose sand of varying grain sizes inclusive of pebbles and boulders at places. The thickness of sand in river bed above bed rock varies from 1.5 m on the right bank, to 15 m on the left bank. The density characteristics of the sand were considered to be of primary importance and were investigated in detail. Direct determination of in-situ density of saturated sand at various depths is an intricate operation and requires specialised equipment for extraction of undisturbed samples. So, for assessing the relative densities of the natural sand deposits, standard Terzaghi's penetration tests were carried out at 60-m interval under the entire seat of the dam. Side-by-side, static penetration tests at selected locations near the points where Terzaghi's penetration tests were carried

..., were also done, for correlating the relative densities as obtainable from Gibbs and Holtz curve which were found to be much in excess to those obtainable from Terzaghi's (Table 2.5, Fig. 2.2). The possible reasons for the difference in the relative densities may be the large surcharges in static penetration. For a safer assessment, it was considered better to adopt relative densities as given by Terzaghi's standard penetration resistance curve. The samples of sand from different depths were also collected and analysed for their mechanical composition (Fig. 2.3) and permeabilities. The tests indicated that the sand densities were irregular and partly deficient. The relative densities vary from 25% to 55% (Table 2.5).

As the river bed sands have inadequate and irregular densities, to seat a dam without studying seismic effect on these loose deposits would have been unwise. Therefore, necessary investigation regarding partial or total liquefaction and undesirable settlement were carried out in the field and also at the School of Research & Training in Earthquake Engineering, Roorkee, on Tenughat sands both with and without surcharges.

2.2.6.1. Blast tests were carried out [7] at site in the bed of the river earmarked for the purpose divided in three plots (Fig. 2.4). Special gelatine (80%) alongwith electric detonator were installed at predetermined depth in a cased borehole. The hole was filled with sand and casing withdrawn before blasting. Both horizontal and vertical accelerations were measured with distance from the point of detonation. The surface settlement and pore pressures at 3-m, 6-m and 9-m depth were observed with distance from the point of detonation. Static and dynamic cone

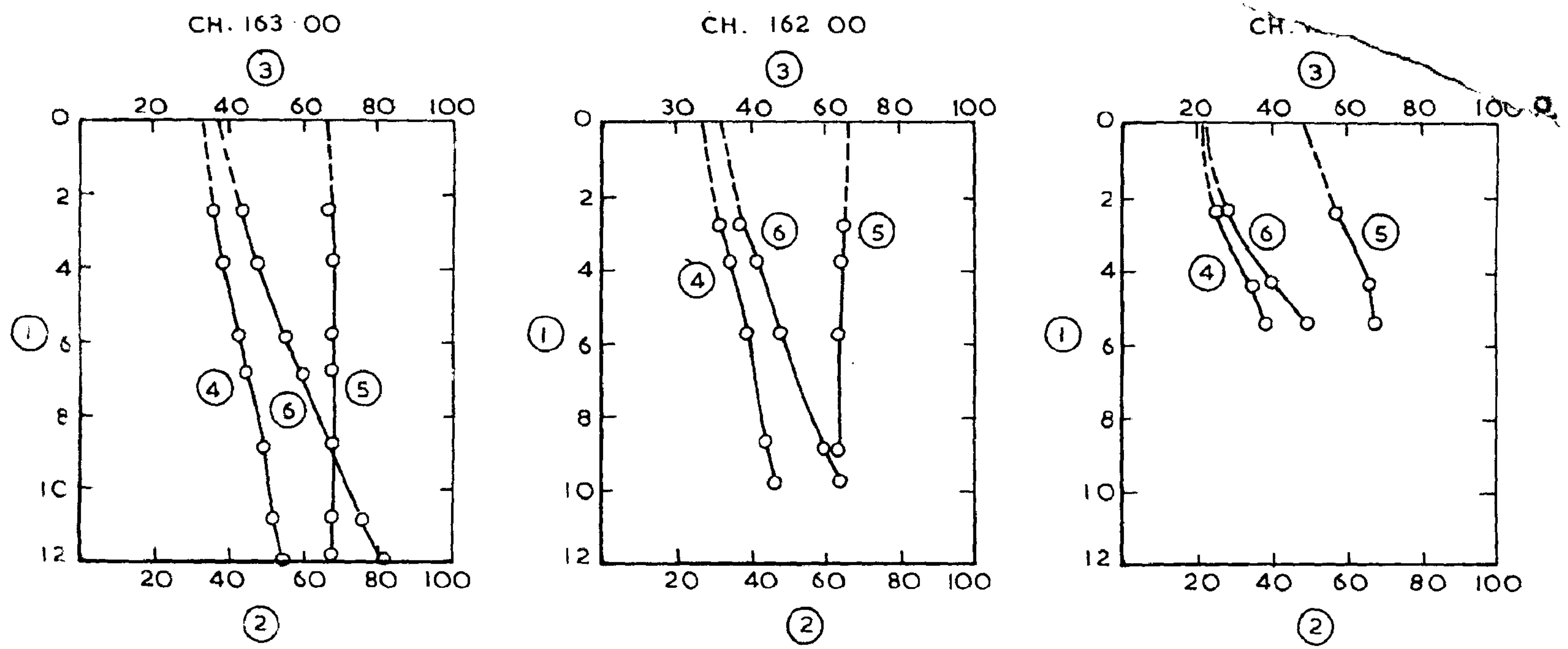


FIG. 2·2 STATIC-CONE-PENETRATION RESULTS AND RELATIVE DENSITIES.

① DEPTH IN METERS

② STATIC CONE-PENETRATION RESISTANCE IN KG/CM^2

③ RELATIVE DENSITY IN PERCENTAGE

④ RELATIVE DENSITY BY TERZOGHI

⑤ RELATIVE DENSITY BY GIBBS AND HOLIZ

⑥ CONE PENETRATION RESISTANCE

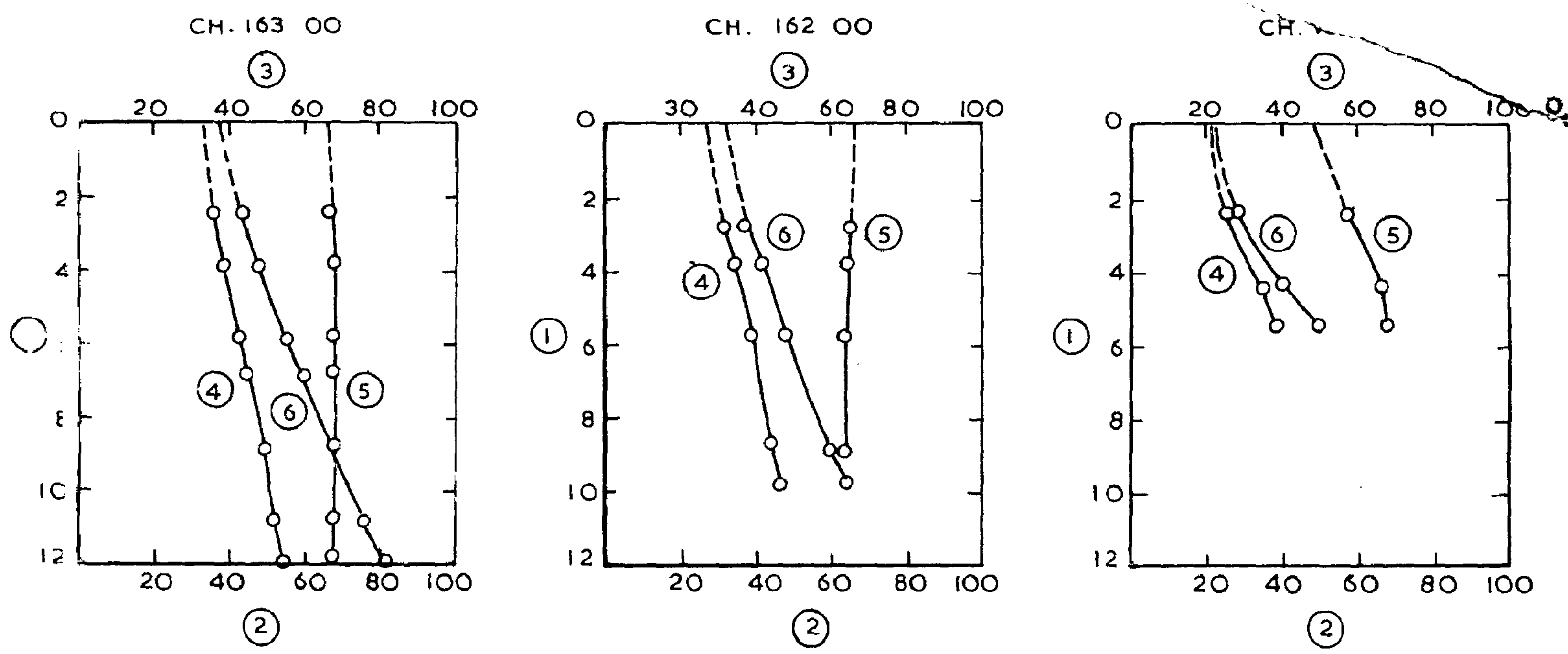


FIG. 2·2 STATIC-CONE-PENETRATION RESULTS AND RELATIVE DENSITIES.

① DEPTH IN METERS

② STATIC CONE-PENETRATION RESISTANCE IN KG/CM^2

③ RELATIVE DENSITY IN PERCENTAGE

④ RELATIVE DENSITY BY TERZOGHI

⑤ RELATIVE DENSITY BY GIBBS AND HOLIZ

⑥ CONE PENETRATION RESISTANCE

TENUGHAT SAND

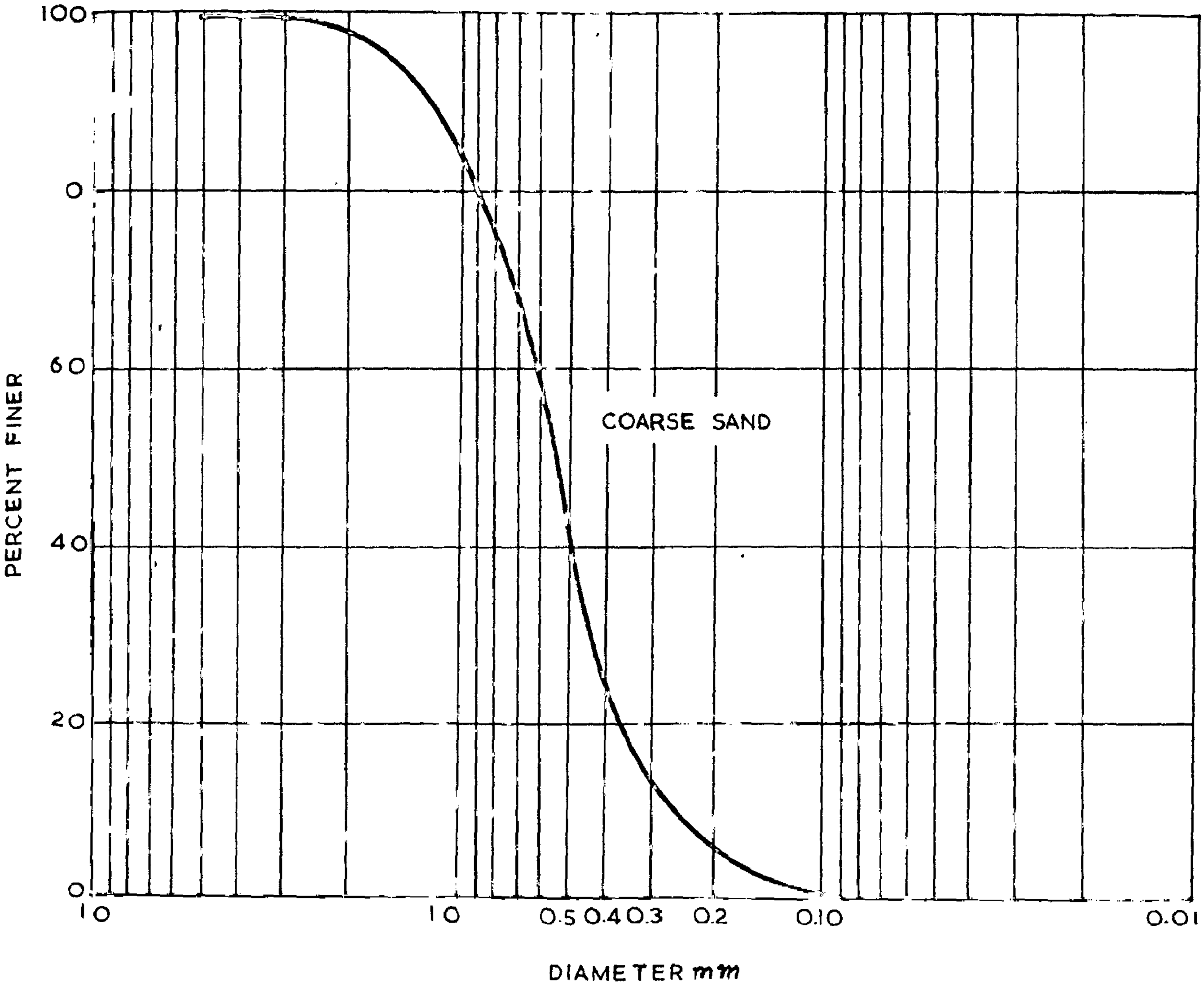
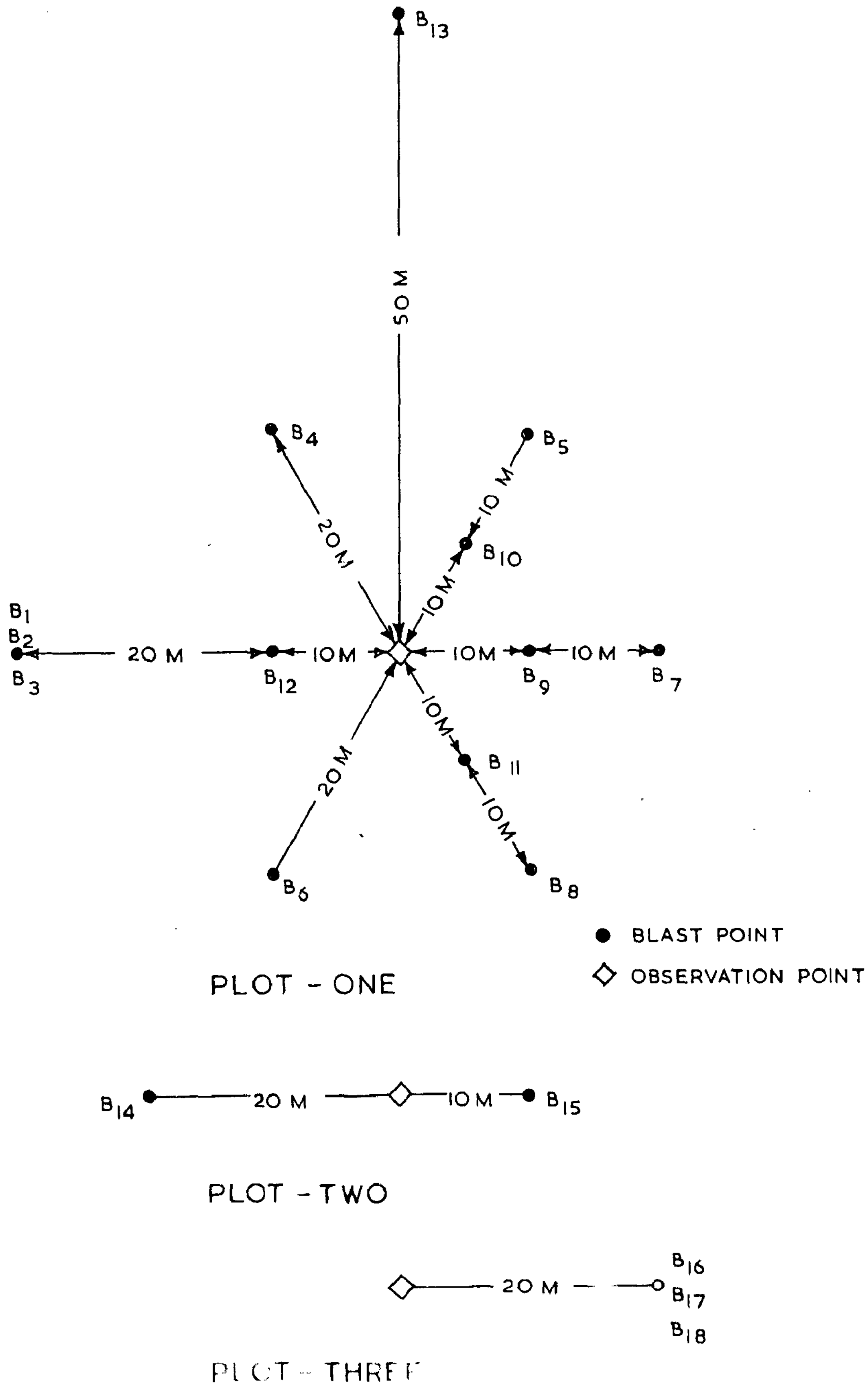


FIG. 2·3 GRAIN SIZE DISTRIBUTION



F 2·4 BLASTING - PLOT

penetration tests were also done before and after the blasts.

2.2.6.2. Details of Tests: Special gelatine (80%) was embedded at a predetermined depth (usually 6 m) below the bed and the fired. The measurement of surface acceleration both in the horizontal and vertical direction was made by embedding two Miller acceleration pickups at about 20 cm below the bed such that one would sense horizontal and the other vertical acceleration. The pickup is of inductance type in which the displacement of an iron core in a magnetic field is proportional to absolute acceleration of motion. Their signals were amplified by "Brush Universal Amplifier" and recorded on Brush Pen recorder. In a few cases, the acceleration was measured 3 m below the bed. In some tests the depth of blast was kept 4 m and 9 m also.

The site was divided into three plots as shown in Fig. 2.4. Depth of blast, size of blast, distance of acceleration pickup from the blast point and depth of pickup are indicated in column 3, 4, 5 and 6, respectively, in Table 2.6. Observed horizontal and vertical accelerations have been shown in column 7 and 8, respectively. In test B_1, B_2 and B_3 the charges were fired from the same location after lapse of a day or so. In plot 1, B_1 - B_{13} (Table 2.6) blasts were fired and only surface acceleration was measured.

In plot 2, B_{14} and B_{15} were fired and acceleration at 3-m depth was recorded. Acceleration pickups were attached to one end of 5-cm hollow pipe and lowered in boreholes supported by casing pipe. The bore was then filled with sand and casing withdrawn. Due to certain difficulty of working the pickup in saturated soil, only vertical acceleration at 3-m depth were obtained

Table 2.4 - Horizontal and Vertical Accelerations

Plot No.	Blast No.	Depth of charge m	Size of charge kg	Distance of Accln. pick ups. m	Depth of pick ups m	Horizontal Accln. g	Vertical Accln. g	Remarks
1	2	3	4	5	6	7	8	9
	B_1	6	1.25	30	0	0.161	0.321	
	B_2	6	1.0	30	0	0.182	0.258	
	B_3	6	1.0	30	0	0.156	0.372	
	B_4	6	1.25	20	0	0.288	1.12	Average
	B_5	4	1.0	20	0	0.310	1.23	H=0.169
	B_6	9	1.0	20	0	0.637	1.68	V=0.315
One	B_7	6	2	20	0	0.540	1.24	
	B_8	6	0.5	20	0	0.166	1.14	Results
	B_9	6	1.0	10	0	0.70	1.88	corrected
	B_{10}	4	1.0	10	0	0.447	1.52	for 1 kg
	B_{11}	9	1.0	10	0	1.36	4.0	charge
	B_{12}	6	0.5	10	0	0.46	1.17	
	B_{13}	6	1.0	50	0	0.0645	0.168	
Two	B_{14}	6	1.0	20	3	-	2.46	
	B_{15}	6	1.0	10	3	-	4.26	
Three	3 Blasts B_{16}, B_{17} B_{18} in quick succession	0	0.5 each	20	0	0.224 0.294	0.937 1.04	

0 = 20 cm.

in test B_{14} and B_{15}. No other data could be recorded.

In plot 3, series blasting was done, only three 0.5-kg charges (B_{16}, B_{17}, B_{18}) could be fired at an interval of 3 seconds and 5 seconds, and both horizontal and vertical acceleration recorded (column 7, 8, Table 2.6).

Settlement of ground surface was recorded on settlement plates embedded at shallow depth and the pore-pressure rise was observed on improvised piezometer along predetermined grids from blast points. The summary of results recorded are shown in Table 2.7, 2.8, 2.9, 2.10.

It was observed from these results that in most cases the blast had caused appreciable settlement of the sand in an area of 6-m radius. In case of three successive blasts of 0.5 kg each, vertical acceleration (α_V) went upto 1.04 g and α_H only 0.294g (Table 2.6). Against these maximum observed % effective stress was noted as 70.5% at a depth 6 m at a distance of 3 m from the blast point. The corresponding settlement came as -0.61cm to -2.13 cm at 6-m depth and distance 3 m to 6 m, respectively (Table 2.9). It was further observed from Table 2.7 that on four successive blasts, the % effective stress reached as high as 106% and that the sand was apparently forced to liquefy. But as Tenughat dam site is situated in isoseismal zone VI on M.M.Scale corresponding to which the maximum intensity of earthquake likely to be felt would be maximum of 10%. From these observations it was calculated that river sand will not liquefy and excessive settlement of foundation will not take place under anticipated intensity of tremors.

The field tests were verified by detailed laboratory tests[8]

Table 2.7 -

Plot No.	Blast	essure				
		(5.54)		At 9-m depth (8.54)		
		Pr.	% PP	0 Minutes	Eff. Pr.	% PP
1			11	12	13	14
I	B_1 (Re	2	64.4	2.18 m	8.92	35.6
	17. / 095	6	39.7	2.66 m	8.95	29.7
		3	20.9	0.546 m	8.75	6.25
		8	5.1	0.343 m	8.58	4.0
	B_2	9	78.5	4.32 m	8.88	49.8
	17.2. / 1105	3	32.5	3.04 m	8.93	34.0
		1	23.2	0.585 m	8.73	6.7
		6	28.2	0.216 m	8.58	2.52
	B_3	5	75.8	4.03 m	8.85	45.5
	17.2. / 1240	1	29.2	3.34 m	8.91	37.4
		1	21.4	0.649 m	8.73	7.4
		6	3.6	0.222 m	8.58	2.6
	B_4	6	10.5	0.022 m	8.64	0.25
	B_5	6	2.26	0 m		0
	B_6	6	5.2	0 m		0
	B_7	6	27.7	0.33 m	8.64	3.8
	B_8	6	1.25	0.0127 m	8.64	0.15
	B_9	6	42.2	2.20 m	8.64	25.5
	B_{10}	6	44.6	1.99 m	8.64	23.0
	B_{11}	6	32.0	1.805 m	8.64	20.9
	B_{12}					

Table 2.7 - Tenughat Blasting Test - Maximum Increase in Pore Pressure after Blasting of Different charges

Plot No.	Blast No.	Depth of charge m	Size of charge kg	Distance of piezometer from blast point in m	Rise in Pore Pressure: At 3-m depth (2.54) 0 Minutes	Eff. Pr.	% PP	At 6-m depth (5.54) 0 Minutes	Eff. Pr.	% PP	At 9-m depth (8.54) 0 Minutes	Eff. Pr.	% PP
1	2	3	4	5	6	7	8	9	10	11	12	13	14
I	B_1 (Repeat)			3	1.74 m	3.11	56.0	3.88 m	6.02	64.4	2.18 m	8.92	35.6
	17.2.67 0950 hrs	6	1	6	1.16 m	3.15	36.7	2.41 m	6.06	39.7	2.66 m	8.95	29.7
				15	0.305 m	2.94	10.4	1.22 m	5.83	20.9	0.546 m	8.75	6.25
				30	0.076 m	2.78	2.7	0.292 m	5.68	5.1	0.343 m	8.58	4.0
	B_2			3	2.08 m	3.06	68.00	4.70 m	5.99	78.5	4.32 m	8.88	49.8
	17.2.67 1105 hours	6	1	6	1.45 m	3.13	46.3	1.96 m	6.03	32.5	3.04 m	8.93	34.0
				15	0.432 m	2.92	14.8	1.35 m	5.81	23.2	0.585 m	8.73	6.7
				30	0.051 m	2.76	1.8	0.279 m	5.66	28.2	0.216 m	8.58	2.52
	B_3			3	2.10 m	3.03	69.0	4.51 m	5.95	75.8	4.03 m	8.85	45.5
	17.2.67 1240 hrs	6	1	6	1.470m	3.13	47.0	1.755 m	6.01	29.2	3.34 m	8.91	37.4
				15	0.483 m	2.92	16.5	1.245 m	5.81	21.4	0.649 m	8.73	7.4
				30	0.019 m	2.76	0.7	0.203 m	5.66	3.6	0.222 m	8.58	2.6
	B_4	6	1	20	0.267 m	2.85	9.4	0.615 m	5.86	10.5	0.022 m	8.64	0.25
	B_5	6	1	20	0.216 m	2.85	7.6	0.133 m	5.86	2.26	0 m		0
	B_6	9	1	20	0.137 m	2.85	4.8	0.305 m	5.86	5.2	0 m		0
	B_7	6	2	20	0.610 m	2.85	21.4	1.628 m	5.86	27.7	0.33 m	8.64	3.8
	B_8	6	1/2	20	0.0762 m	2.85	2.7	0.0732 m	5.86	1.25	0.0127 m	8.64	0.15
	B_9	6	1	10	1.230 m	2.85	43.0	2.48 m	5.86	42.2	2.20 m	8.64	25.5
	B_{10}	6	1	10	1.335 m	2.85	46.8	2.52 m	5.86	44.6	1.99 m	8.64	23.0
	B_{11}	9	1	10	0.813 m	2.85	28.5	1.00 m	5.86	32.0	1.805 m	8.64	20.9
	B_{12}												

(continued)

Table 2.7 (continued)

1	2	3	4	5	6	7	8	9	10	11	12	13	14
II	1	6	1	20									
	2	6	1	10									
	3	6	1	20									
	4	6	1	10									
III	3 blasts in quick succession with a time interval of 0.5 to 1 sec.	6	1/2 kg each	3	3.22 m	2.58	12.5	3.87 m	5.50	70.5	2.20 m	8.40	26.2
				6	1.435 m	2.58	55.5	2.44 m	5.50	44.5	2.52 m	8.40	30.0
				15	0.032 m	2.60	1.2	-0.05 m	5.50	0	0.953 m	8.40	11.3
	4 blasts	6	1 kg each	3									
				6	3.0 m	2.50	120.0	5.75 m	5.40	106.0	4.98 m	8.30	6.0
				15	0.94 m	2.50	-	8.20 m8	5.40	59.0	3.20 m	8.30 m	38.6

Eff. Pr.= Effective Pressure in metre height, $\bar{\sigma} = \sigma - p$

% PP = Pore pressure in % of effective stress

$$p_1 - p_2 = \bar{\sigma}_2 - \bar{\sigma}_1$$

$$\bar{\sigma}_1 = \sigma - p_1$$

$$\bar{\sigma}_2 = \sigma - p_2$$

Table 2.7 (continued)

1	2	3	4	5	6	7	8	9	10	11	12	13	14
II	1	6	1	20									
	2	6	1	10									
	3	6	1	20									
	4	6	1	10									
III	3 blasts in quick succession with a time interval of 0.5 to 1 sec.	6	1/2 kg each	3	3.22 m	2.58	12.5	3.87 m	5.50	70.5	2.20 m	8.40	26.2
				6	1.435 m	2.58	55.5	2.44 m	5.50	44.5	2.52 m	8.40	30.0
				15	0.032 m	2.60	1.2	-0.05 m	5.50	0	0.953 m	8.40	11.3
	4 blasts	6	1 kg each	3									
				6	3.0 m	2.50	<u>120.0</u>	5.75 m	5.40	<u>106.0</u>	4.98 m	8.30	6.0
				15	0.94 m	2.50	•	8.20 m8	5.40	59.0	3.20 m	8.30 m	38.6

Eff. Pr.= Effective Pressure in metre height, $\bar{\sigma} = \sigma - p$

% PP = Pore pressure in % of effective stress

$$p_1 - p_2 = \bar{\sigma}_2 - \bar{\sigma}_1$$

$$\bar{\sigma}_1 = \sigma - p_1$$

$$\bar{\sigma}_2 = \sigma - p_2$$

Table 2.8 - Horizontal and Vertical Acceleration

Blast No.	Charge Quantity	Depth m	Distance from Blasting Point m	Acceleration Depth of pickup	Horizontal Acceleration g	Vertical Acceleration g
1	2	3	4	5	6	7
			Repeat Blast			
$B_1(P_1)$	1	6	30	surface	0.2	0.4
$B_2(P_1)$	1	6	30	surface	0.2	0.25
$B_7(P_1)$	1	6	30	surface	0.16	0.4
			Depth vs Acceleration			
$B_5(P_1)$	1	4	20	surface	0.32	1.3
$B_4(P_1)$	1	6	20	surface	0.32	0.88
$B_6(P_1)$	1	9	20	surface	0.70	1.70
			Charge vs Acceleration			
$B_6(P_1)$	$\frac{1}{2}$	6	20	surface	0.17	1.1
$B_6(P_1)$	1	6	20	surface	0.32	0.88
$B_7(P_1)$	2	6	20	surface	0.5	1.4
			Distance vs Acceleration (charge 1/2 kg)			
$B_{12}(P_1)$	1/2	6	10	surface	0.5	1.3
$B_9(P_1)$	1/2	6	20	surface	0.12	1.1
			Distance vs Acceleration (Depth of charge 4 m)			
$B_{10}(P_1)$	1	4	10	surface	0.6	2.0
$B_5(P_1)$	1	4	20	surface	0.32	1.3
			Distance vs Acceleration (Depth of charge 6 m)			
$B_9(P_1)$	1	6	10	surface	0.8	2.0
$B_4(P_1)$	1	6	20	surface	0.32	0.8
$B_1(P_1)$	1	6	30	surface	0.2	0.4
$B_{13}(P_1)$	1	6	50	surface	0.06	0.1

(continued)

Tablo 2.8 (continued)

		Distance vs Accoleration (Depth of chargo 9 m)				
$B_{11}(P_1)$	1	9	10	Surface	10.5	3.0
$B_6(P_1)$	1	9	30	Surface	0.7	1.7
		Depthof pickup vs Acceleration				
$B_9(P_1)$	1	6	10	Surfaco	-	2.0
$B_4(P_1)$	1	6	10	3 m	-	4.4
		Depth of Pick vs Accoleration (Distance from tho blasting point 20 m)				
$B_1(P_1)$	1	6	20	Surfaco	-	0.89
$B_1(P_1)$	1	6	20	3 m	-	2.5
		Effect of cycles of Acceleration				
$B_8(P_1)$	1/2	6	20	Surface	0.17	1.1
$B_5(P_1)$	1/2	6	20	Surfaco	0.3	1.0
3 successive blasts	1/2 kg each	6	20	Surfaco	i) 0.06	0.33
					ii) 0.25	1.0
					iii) 0.3	1.0

Table 2.9 - Summary of Settlement Data

Plot No.	Blast No.	Charge kg	Depth m	Depth of settlement plate	Settlement at 3 m distance in cm	Settlement at 6m distance in cm.	Settlement at 15 m distance in cm.
1	2	3	4	5	6	7	8
I	B_1	1	6	Surface 6 m	8.85 15.3	3.0 -3.35	Nil
	B_2	1	6	Surface 6 m	5.8 8.25	2.75 2.44	0.92
	B_3	1	6	Surface 6 m	3.05 6.10	2.44 2.44	1.83
B	B_4	1	6	Surface	5.20	4.27	0.30
	B_5	1	4	Surface	6.10	2.74	1.83
	B_6	1	9	Surface	-0.61	-2.74	-3.36
	B_7	2	6	Surface	7.33	3.66	-0.91
	B_8	1/2	6	Surface	3.35	0.92	0.61
	B_9	1	6	Surface			
	B_{10}	1	4	Surface	8.25	3.66	1.22
	B_{11}	1	9	Surface	7.33	1.83	0.61
II	B_1	1	6	Surface			
III	4 cycles	1	6	Surface	9.46 8.55	4.58 7.02	-1.83
	3 cycles	$\frac{1}{2}$	6	Surface 6 m	-0.91 -0.61	-1.22 -2.13	-1.22

-ve sign shows heaving

Table 2.10 - Summary of the Static Penetration Data

Plot No.	Blast No.	Charge in kg	Depth (m)	3-m distance	6-m distance	15-m distance
1	B_1, B_2, B_3	1 kg each	6	0-3.25 m 22-31 kg/cm^2 Increase 41% 3.5-4.5 m 53-38 kg/cm^2 Decrease 28%	4-70 m 48-37 kg/cm^2 Decrease 23%	3.5-7 m 57-49 kg/cm^2 Decrease 14%
	B_4	Do.	6	3.5-9 m 42-26 kg/cm^2 Decrease 38%	2-8.5 m 46-32 kg/cm^2 Decrease 30%	1.8-7.5 m 21-45 kg/cm^2 Increase 115%
	B_5	Do.	4	2-6 m 50-35 kg/cm^2 Decrease 30%	3-7.5 m 26-40 kg/cm^2 Increase 56%	0-2.5 m 15-25 kg/cm^2 Increase 60% 2.5-5 m Increase 25% 4-9 m Dec. 25%
	B_6	Do.	9	0-6 m 16-35 kg/cm^2 Increase 120% 7-9.5 m 79-50 kg/cm^2 Decrease 37%	0-6 m 18-30 kg/cm^2 Increase 66%	4-6 m 63-42 kg/cm^2 Decrease 33%
	B_7	2 kg	6	0-3.26 m 21-37 kg/cm^2 Increase 76% 6-9.5 m 61-43 kg/cm^2 Decrease 30%	0-6 m Inc. about 30%	0-8 m 25-36 kg/cm^2 Increase 44%
2	3 successive blasts	1/2 kg	6	Practically no change.	2-2.5 m 39-23 kg/cm^2 Decrease 41% 5.5-8.5 m 16-23 kg/cm^2 Increase 44%	2.5-7.5 m 51-32 kg/cm^2 Decrease 37%

at the School of Research and Training in Earthquake Engineering, Roorkee, and possibility of liquefaction and total settlements expected were estimated. The field results were in conformity with the laboratory results. The conclusion drawn on the basis of the above field and laboratory tests were [8]

(i) That the Tenughat sands are not prone to liquefaction under anticipated ground motion.

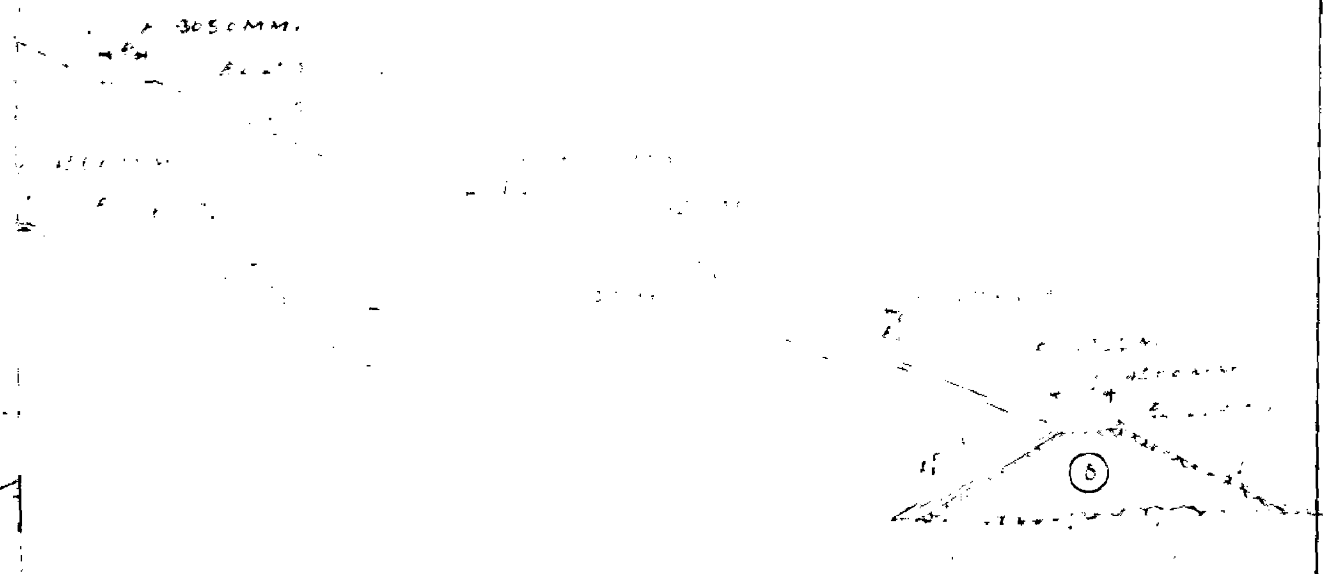
(ii). That the settlement of the sand are not excessive (0.02 m on anticipated ground motion of 0.1 g).

Hence no treatment of foundation is necessary. However, at the junction of the upstream and downstream toes with the bed, a flexible apron 3 m (Fig. 2.5) thick graded as inverted filter material in a length of 30 m have been provided. Now the foundation sand was only a problem for seepage and to provide an impermeable barrier across this depth was a task before the project authorities.

2.2.7. Underlying Old Stream Bed (Stn. 6070 m to 6375 m)

Detailed investigation between this length consisting of drilling and water-loss tests, indicated high permeabilities between stn 6100 to 6250 m. Further investigation brought out the existence of an underlying old stream bed crossing the dam. Hard rock strata is, however, found on an average of 3 m of the bottom of key trench. It has, therefore, been decided to grout from stn 6070 m to 6375 m upto 10 m below the bottom of cutoff.

Thus in the entire length of the dam there was practically no problem in providing the cutoff except in the river bed where concrete diaphragm wall was adopted as permanent cutoff.

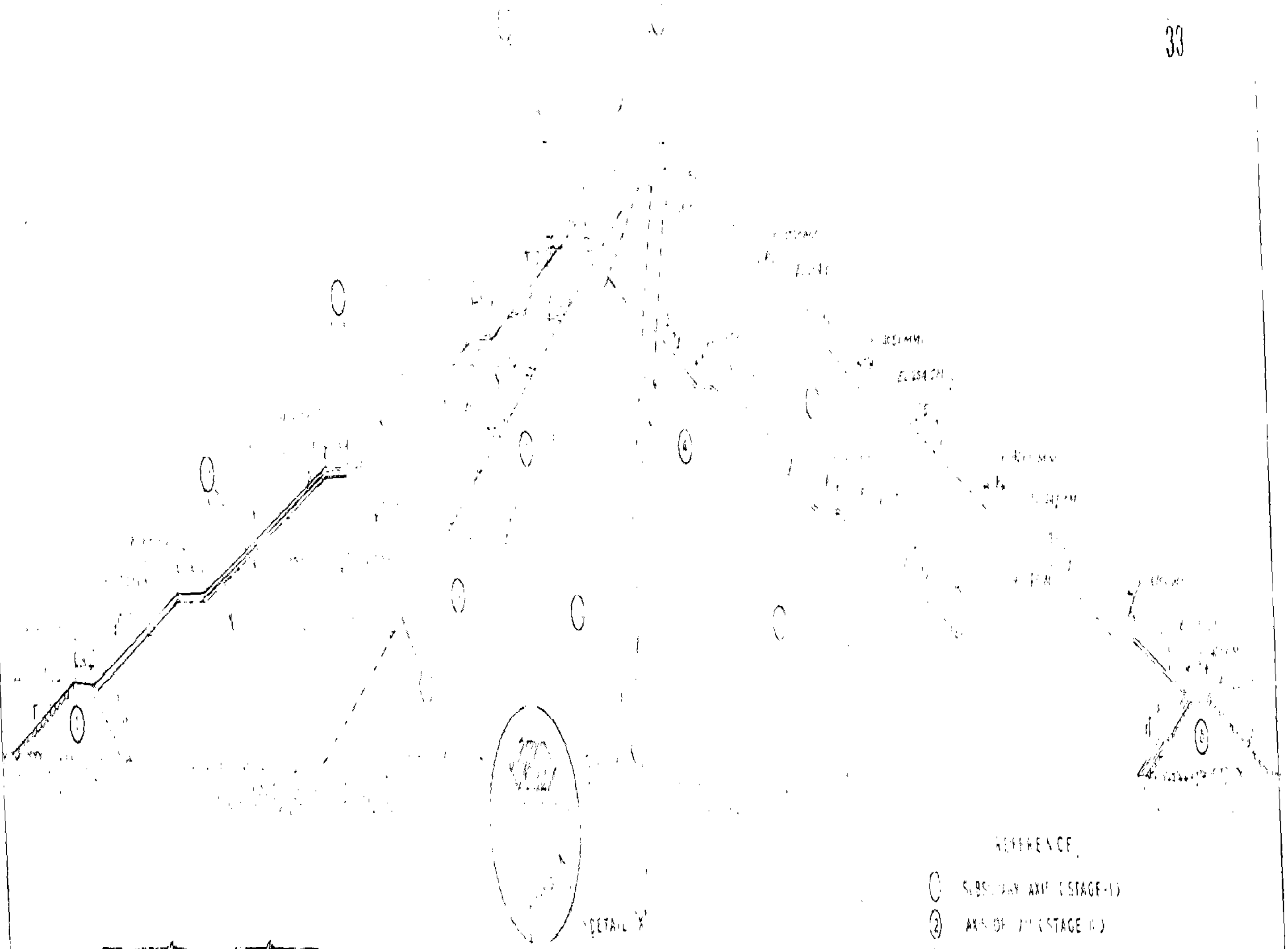


REFERENCE

1. SUBS[illegible] AXIS [illegible] STAGE [illegible]
2. AXIS OF [illegible] STAGE [illegible]
3. [illegible] RAF[illegible]ON FILTER
4. VERTICAL SAND CHIMNEY
5. [illegible]
6. [illegible]
7. [illegible] SAND FILTER LAYER
8. SAND FILTER TERMINATION LINE
9. [illegible]
10. [illegible]
11. [illegible]
12. [illegible]
13. [illegible]
14. [illegible]
15. [illegible] PANEL
16. [illegible]

[illegible] SECTION &
[illegible] OF [illegible] DIAPHRAGM CUT OFF

FIG. 2·5

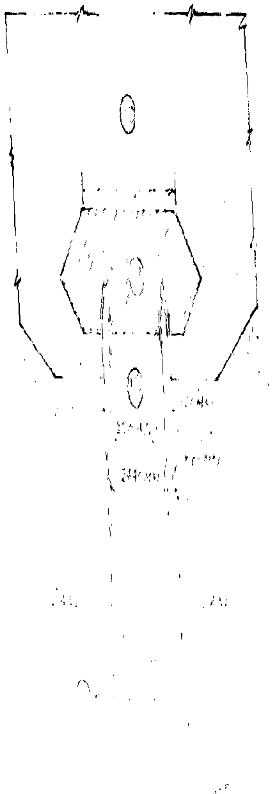


SECTION RIVER BED PORTION FROM STN 4938 M TO 5182 M

SCALE 1 CM 60 M

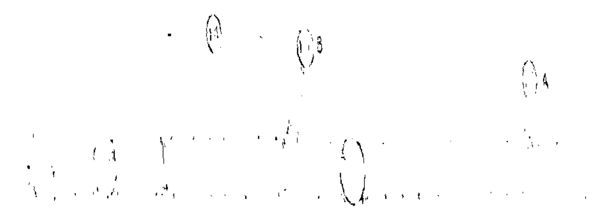
REFERENCE

1. SUBSIDIARY AXIS (STAGE-I)
2. AXIS OF ... (STAGE II)
3. ... SLOPE RIP RAP CUM ... FILTER
4. VERTICAL SAND CHIMNEY
5. TOWING
6. ROCK TOE
7. ... SAND FILTER LAYER
8. SAND FILTER TERMINATION LINE
9. ...
10. ...
11. ...
12. ... CLAY CAPPING
13. CONCRETE DIAPHRAGMS
14. ... PANEL
15. ... PANEL
16. ...



ELEVATION

SCALE 1 CM 125 M



PLAN OF CONCRETE DIAPHRAGMS

SCALE 1 CM 25 CM

... DIAPHRAGM ... SECTION & ... OF DIAPHRAGM CUT OFF

FIG. 2.5

Chapter 3

CHOICE OF CUTOFF

3.1. General

The safety of a dam with a foundation of deep alluvium is directly related to the effectiveness of the cutoff in the reduction or control of seepage under the dam. When the foundation alluvium is shallow, or where dewatering problems are not great, a positive cutoff in the form of an excavated open core trench backfilled with compacted impervious material is usually adopted. In situations requiring a deep cutoff, the compacted core trench method often becomes quite expensive and impracticable. specially when working period is restricted.

3.2. In case of Tenughat dam, where the river bed sand posed serious dewatering problem, different alternative methods for providing cutoff in the river bed were considered on the base of time and economy. These are -

(i) Steel sheet piling

(ii) Provision of upstream impervious blanket

(iii) Backfilled open trench

(iv) Grouting

(v) Slurry trench

(vi) Concrete cutoff walls

These alternatives were considered on the above-mentioned base of time and economy. In case of Tenughat, time was more important than cost as the target was to store water during July 1969 to October 1969 and supply to Bokaro Steel Project from December 1969.

3.2.1. Steel Sheet piling

Steel sheet piles cutoff have been provided in a number of dams, some of the examples being Fort Peck, Garrison, Oahe [9], Denison and Santiago Dam [10] in U.S.A. and Maithon, Panchet Hill [11] about 80 km downstream from this site where alluvial deposit is also of the same depth, i.e. 15 m. It is very difficult to make a sheet pile cutoff watertight. There are always chances of leakage through the joints and at contact with bed rock. Due to presence of pieces of boulders in the mass of bed sand at lower depth, and irregular bed rock surface, tearing and curling of steel sheet piles could not be ruled out. Again, to drive sheet pile without the fear of damage to interlocking and in the whole river bed in one season was difficult task at Tenughat dam. A study of effectiveness of sheet piles at Fort Peck, Garrison and Oahe dams as presented by Lane and Wohlt [9] reveals that 'sheet piling has been relatively ineffective for controlling underseepage'. At Kota dam [12] also it was not found to be effective. Hence this possibility was entirely ruled out.

3.2.2. Provision of upstream impervious blanket

Upstream blanketting is also a measure to control seepage. Its use alone or in combination with steel sheet piles was considered but was found to be unsuitable under circumstances. The lower valley dam on this river at Panchet Hill, where the cutoff is a combination of sheet piles and blanketting has the main purpose of flood control whereas in this project, the purpose is conservation for utilisation during lean months of flow. In Panchet Hill dam, a single row of sheet pile driven upto rock in foundation below river bed and extended 6 m above river bed in the core with a blanket spreading 150 m upstream of the dam [13] in the river bed. This arrangement was not workable at Tenughat as the whole work was to be completed within one working season of nearly 200 days.

3.2.3. Backfilled open trench

This is the commonly used method under many dams. It provides direct avenues to inspection and control of quality of work. As a result this type of cutoff provides a tight and effective barrier to seepage . The principal difficulty in construction of a rolled earth cutoff is frequently the dewatering of the excavation and holding down the water level until the trench is backfilled.

At Tenughat, this problem of dewatering and holding down the water level for backfilling the excavated trench was stupendous. On working out the cost of dewatering itself, it was coming more than the cost of cutoff finally adopted, the excavation and backfilling cost was in addition. A foreign firm offered to use the technique of freezing the water which would make the excavation and backfilling task much easier. The cost of this method was estimated to be lower than the conventional dewatering and backfilling open trench, but this technique has been tried successfully in cold climates and not in a tropical climate like that of Tenughat where day temperature during summer working season varies between 100°F and 120°F. So, completion of open trench cutoff in one working season was found uneconomical.

3.2.4. Grout Curtain

This method is providing a dependable cutoff and being frequently adopted nowadays in dams with large pervious deposits in river bed as permanent seepage barrier. Such grout curtains have been used in many dams, the notable examples being Serre Poncon Dam in France [14], High Aswan Dam in Egypt [15], Mission Dam in Canada [16], Mattmark Dam in Switzerland [17], and Kota Dam in India [12]. Serious doubts on the efficacy of cutoff containing even very small percentage of openings have been cast by Casagrande in his first Rankine lecture [18]. He has given examples of a membrane which has slits 1.5 mm (1/16 in.) wide spaced every 1.5 m (5 ft). This would correspond with an open-space ratio of 0.1%. For the case where base width of structure is 30.5 m (100 ft) and so is the depth of curtain, so that for a 1.5-m spacing the number of slits, n, is 20, the theoretical cutoff efficiency, i.e. the ratio of the reduction ofseepage discharge due to the cutoff to the rate offlow without the cutoff is 29%. In other words, 71% of the rate of flow without cutoff would still be going through these slits. If the same 0.1% of the area is divided into a larger number of slits, the cutoff efficiency drops down even lower, with n equal to 120 it is 6%. In some of the recent designs increasing reliance has been placed on the grout curtain, and the provision of the long impervious blanket has been omitted or curtailed in some dams in which this was provided. However, this grout curtain has mostly been found to be more successful in gravel and boulder strata, where clay-cement, clay, or clay chemical grout can be employed. At Kota, it was used purely as an additional safety feature. For dependability, it is necessary that the thickness of the grouted zone should not be small in relation to the head, and this tends to make the method expensive. At Tenughat where bed width was more than 300 m, both time and economy were against the technique, even with cement or bentonite grouting. In case of chemical grouting, the cost would have been four-to-five times that of the adopted method.

3.2.5. Slurry trench

The 'slurry trench' procedure with soil backfill had been used as a permanent cutoff under a dam first at Wanapum Dam [19, 20] in U.S.A. in 1958. There the foundation material consisted of an erratic deposit of sands, gravels, and cobbles with some open work gravel extending upto a depth of 27 m. The permeability of the various layers ranged roughly between 0.1 cm/sec. and 3 cm/sec. In case of Tenughat Dam, sand deposit varies from 5 m to 15 m only and permeability of sand 1 cm/sec. maximum to 0.1 cm/sec.

Global tenders were invited for providing the cutoff with design in October 1966 with time limit of completion upto June 1967. No tender was received for slurry trenching and the River Valley Department had no such earthmoving machine in hand to provide this type of cutoff though, it seems, this might have been the cheapest cutoff. The usual depth upto which this type of cut off can be provided is not limited and it depends on the equipment working. The usual equipment currently available indicate practical limits of about 27 m [21] with a dragline, about 30 m with a clamshel. Some trenching machines such as the Parsons 355 Trenchliner, can be equipped with a special ladder to excavate depths upto 14 m. As the deposit was only 15 m, the dragline could have worked and it would have proved cheaper. Lack of previous experience in this type of work was also a consideration in the decision.

3.2.6. Concrete Diaphragm Wall

Vertical concrete walls of various types have been installed under a few major earth dam foundations as seepage barriers (Table 4.1). When founded on rock over their whole length, concrete walls are perfectly satisfactory and provide almost complete imperviousness. The offer by the firm Messrs Rodio Foundation Engineering and Hazrat & Co. of Bombay under the Rodio-Marconi process was workable with both time and economy consideration and was adopted at Tenughat Dam.

Withthe decision to provide a double concrete diaphragm as permanent cutoff wall, the dam section as shown in Fig. 2.5 was adopted. The earthen dam as shown in the figure (2.5) has a tri-zonal section with a slanting central impervious core flanked by semipervious casing both on upstream and downstream with a 3 m (10 ft) thick sand chimney filter, sandwiched between central core and downstream impervious fill, leading to a toe drain with suitable weight and filter. In the river bed portion where the dam rests on pervious deposits, a rock toe has been provided both at toe and heel.

Chapter 4

DIAPHRAGM TECHNIQUE

4.1. General

Design of dams on pervious foundations presents many problems, and these tend to become much more complex when the pervious stratum is not well-compacted and is irregular in density. Where thickness of the deposit is large and of high permeability, the cost of providing a positive cutoff for sifting a dam becomes almost prohibitive. The diaphragm technique, which is of recent origin is a very useful method of tackling such problems under a dam or similar other structures.

4.2. Historical Development

The Engineering profession was greatly in need of some new technique which could enable deep trenching without providing support in presence of sub-soil water and without affecting the the stability of the surrounding soil. The principle for deep oil well drilling with thixotropic liquid support was well known to the Engineering profession. This thixotropic property was first utilised for excavation of a rectangular trench in Italy [22] in the year 1950. Since then there has been a continuous improvement in the technique and its use has also widened. The art of placing concrete under water with or without reinforcement by tremie is in practice since early forties. The concrete diaphragm wall technique is a combination of the trenching in presence of thixotropic liquids and concreting by tremie.

4.2.1. This technique was initiated for the construction of sub-way in Milan, Italy, by I.C.O.S. (Impresa Construzioni opre Specializzata) [22], a firm doing the specialised job.

4.2.2. The method by now had been widely used in France, Switzerland, Great Britain, Greece, Canada, U.S.A., Australia and many other countries in the world. The first use of this technique in U.S.A. was for a permanent structure at Kennewick Levee in 1952, and in India at Obra dam in 1966. The construction of this slurry trench was done by patented equipment. The pioneering firms undertaking this type of work are I.C.O.S., Milan, Italy; E.L.S.E., also Milan, Italy; Harza Engineering Co., Chicago, U.S.A.; Soletanche, France; Bonato, France; Rodio Foundation Engineering Ltd., Switzerland; McDougalt-Ireland Pvt. Ltd., Melbourne. In India, Rodio-Hazarat and Co., Bombay, and Willner and Co., Bangalore [23] undertake the work of this nature.

4.2.3. Many dams of low-to-moderate height seated on pervious foundation have been constructed with slurry trench cutoff within last two-and-a-half decades. One of the most recent and important examples is that of a 91 m (300 ft) high Bighorn Dam [24] in Canada on a 65-m (212-ft) deposit of sand, cobbles and boulders without any problems of maintenance so far. A listof some of the dams, founded on pervious deposits of low-to-moderate depths in which permanent cutoff had been provided by slurry-trench technique, and has served the purpose without any maintenance problem so far, is appended in the end of this chapter (Table 4.1)

4.3. Slurry-trench Technique

The slurry-trench technique of constructing a seepage cutoff consists of excavating a trench with vertical sides supported by bentonite slurry and backfilling this trench with selected soil material blended with bentonite slurry.

As the materials are removed from the trench, bentonite slurry is added at a rate such that the trench always remains filled. This slurry, slightly heavier than water, forms an impermeable filter cake on the trench sides and exerts sufficient pressure on the walls of the trenchto support them in thin vertical positions. After a sufficient length of trench has been excavated, backfilling operations can be started. The selected backfill may consist of part of the materials from trench excavation mixed with borrow materials to provide a well-graded backfill that will be impermeable, safe from piping, and will minimise settlement. Slurry is mixed with the backfill material primarily to reduce the permeability. It also aids in blending fine and coarse backfill materials and improves the backfill placement characteristics.

Since the backfilling operation progresses at the same rate as trench excavation, the slurry displaced by the backfill is used in the extension of the trench.

4.4. <u>Types of Diaphragm</u>

There is no broad classification for the type of diaphragm which has been constructed so far. It has been mentioned earlier, that it is practical innovation. Hence it can be broadly classified into two on the basis of construction procedure, viz. -

(i) Rigid type

(ii) Plastic type

4.4.1. Rigid type of diaphragms are concrete, both plain and reinforced cast insitu diaphragm. The rigid type of diaphragm are constructed in different manner. According to Edison [23] Group rigid diaphragmsof the following important types are known:

(i) Diaphragm consisting of tangential piles (bored without drilling mud) or Bonoto type with drilling mud) Fig. 4.1a.

(ii) Diaphragm with interlocked piles (Fig. 4.1b).

(iii) Diaphragm with overlapping elements (Fig. 4.1c).

(iv) Diaphragm with elongated elements either circular or straight aight (Plain, reinforced or plastic concrete) Fig. 4.1d.

(v) Special load bearing type: C. Veder [22] indicates that current trend is to provide separate load bearing element such as I.L.H. and double I-shaped for load bearing groups of diaphragms. These elements allow vertical loads, shearing forces and bending moments to be resisted. They also exhibit the same advantage as continuous diaphragms, such as absence of disturbance to the surrounding soil, minimum noise, the possibility of construction very close to the buildings, high load bearing capacity and great versatility (Fig. 4.1e).

4.4.2. Plastic Diaphragm: This type of diaphragms are provided by backfilling the slurry trenches with selected soil material blended with bentonite slurry. Most of the slurry trench cutoff provided in American dams are of this type. (Fig. 4.2)

4.5. Machinery used for Diaphragm Construction

The construction of diaphragms on dams and other structures is being done by patented machinery. They can be sub-divided

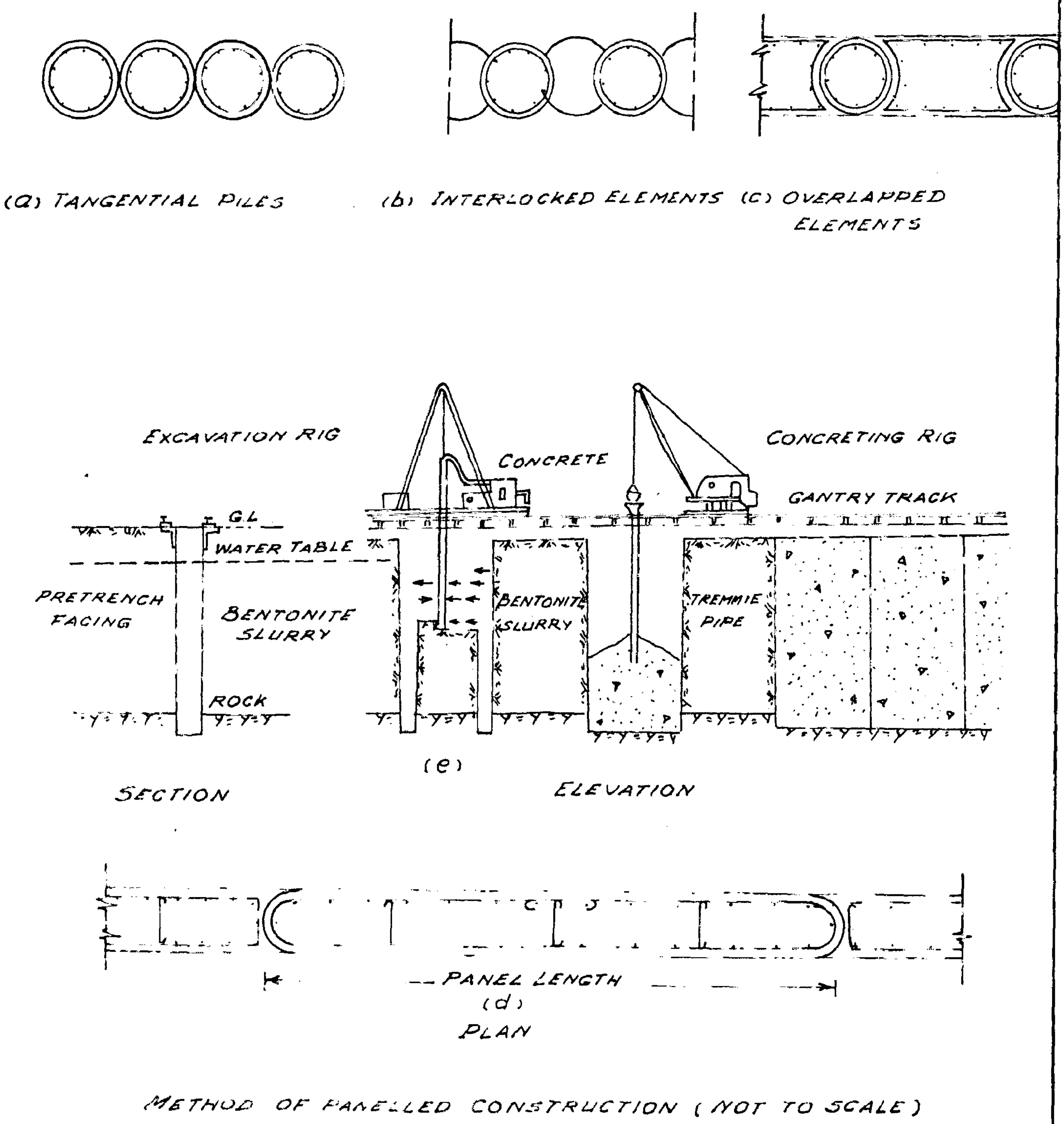


FIG 4·1 _ DEVELOPMENT OF SLURRY TRENCH CUT OFF IN EUROPE

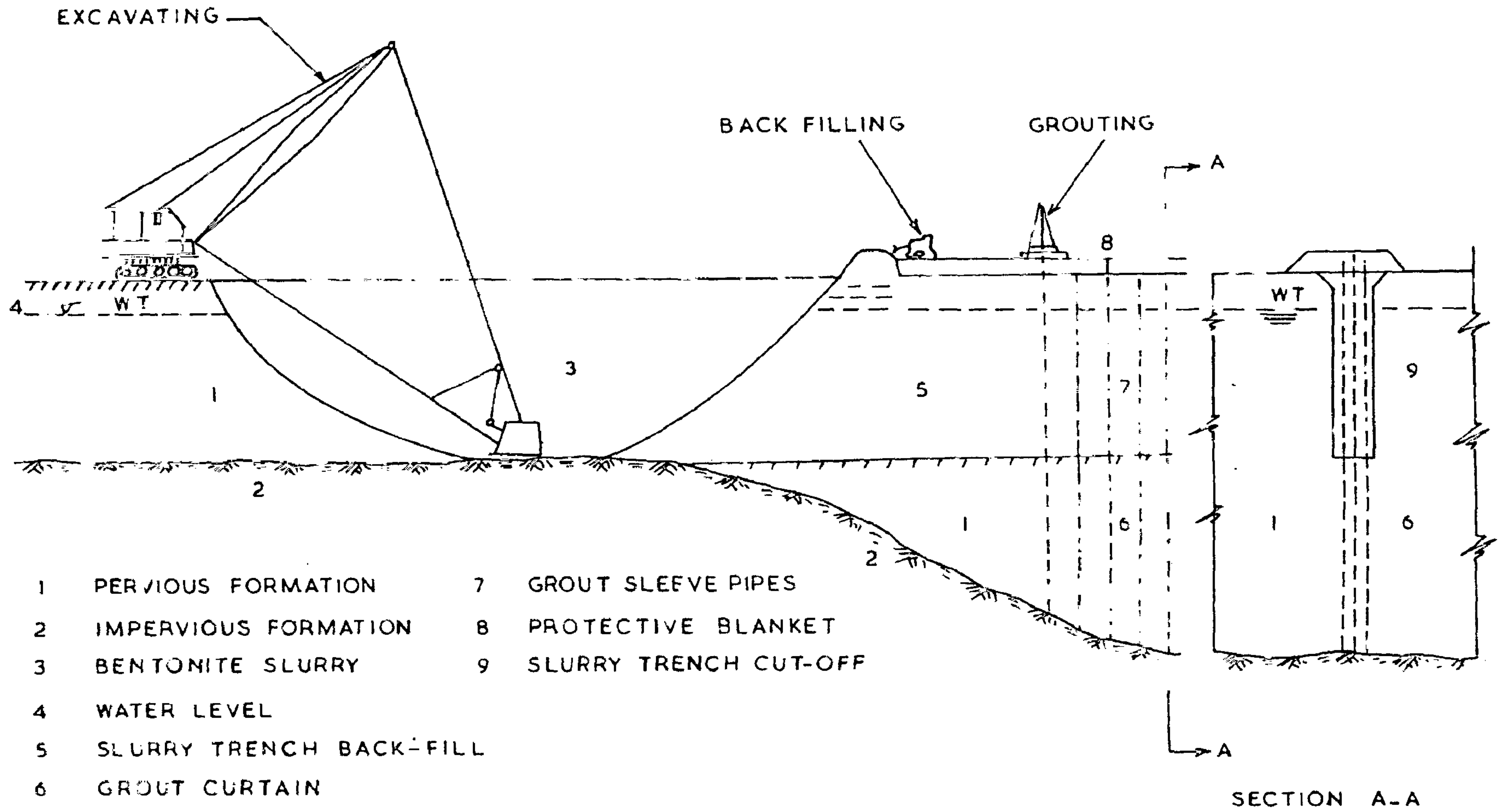


FIG. 4·2 – AMERICAN METHOD OF CONSTRUCTING SLURRY TRENCH CUT OFF WITH GROUT CURTAIN into two categories on continental basis, i.e. -

(i) European (Fig. 4.2)

(ii) American (Fig. 4.2)

4.5.1. European Type: The European type patents for the various techniques known in diaphragm fields are -

(i) I.C.O.S. (Italy)

(ii) Edison Group (Milano)

(iii) Titania System (Italy)

(iv) Rodio Marconi (France-Italy)

(v) E.L.S.E. (Italy)

A list of machines [23] used for excavating trenches is given at the end of the chapter (Table 4.2)

4.5.2. American type: The U.S.A. has also made headway in designing special technique for diaphragm wall construction. A number of patents relating to diaphragm walls have been issued. Many of them relate to special excavating tools and methods for excavating a slot or a trench section. One of the famous patents is Hayza Engineering Co., Chicago.

4.6. Construction Procedure

The principle of construction of the slurry trench is based on "Reverse mud circulation system" in which "rich active clay" or "bentonite" is used as a drilling mud to support the walls of the elements or boreholes. The reverse circulation system ensures continuous flow of bentonite in the element being excavated or drilled. The patented "trenching equipment" with a powerful open vane pump sucks out the drilled or excavated materials alongwith the bentonite slurry. The debris is discharged into slurry pits specially excavated nearby to receive the mud and cuttings. The bentonite slurry flows out to enter again the element being excavated through a system of drains. All the while, the level of bentonite slurry is maintained in the element or panel being operated so that supporting action is continuous. Whatever slurry is absrobed inside the voids of the surrounding material or used for forming a membrane along the walls is to be replenished. Till the excavation of the element is completed, the circulation of the drilling mud is to be continued. The sounding of the trench for its clearance from the loose materials are repeated and the diaphragm reinforcement cage (if reinforced) is lowered and concrete poured through the tremmie. The detailed construction of Tenughat dam diaphragm is inscribed in Chapter 5.

4.7. Factors Accounting for Stability of the Excavated Trench

Much work has been undertaken in recent years to determine the nature and extent of the forces which act on the vertical walls of the slurry supported trenches. Before sixties, there were no theoretical reasoning advanced towards the stabilising factor of the slurry trenches. In sixties, the factors and forces were experimentally verified in the laboratory and also observed in field. The factors which account for the stability of long trenches as suggested by various authors are -

(a) Hydrostatic pressureof the clayslurry.

(b) Passive resistance of the slurry.

(c) Resistance to deformation of the filter cake - Vedor (1961).

(d) Electro-Osmotic forces

(e) Factor preventing initial disturbances.

The prominent researchers in this field are Moradi (1962), Lorenz [26] (1963), Veder [22] (1963), Nash and Jono [27] (1963), Morgenstern and Amir-Tahmasseb [28] (1965), Piaskowsky [29] (1965), and Elson [30] (1968).

Recently, Tem [31] (1975) has also conducted experiments and verified recently the density and properties of the clay slurry at the Water Resources Development Training Centre, University of Roorkee, accounting for the stability of trenches in dry sands. For long trenches where the length may be several times the trench-depth, a formula for stability of the trench wall was first developed by Morgestern and Amir-Tahmasseb. Further, it was confirmed by Duguid [32] (The slurry trench cutoff for the Duncan Dam 1971). However, for narrow and deep trenches, where the length is only a fraction of depth, no rational mathematical analysis of the stability forces has yet been achieved. This is because of the short deep trench, there is an arching action around the excavation, as shown schematically in Fig. 4.3, which contributes towards a stable excavation. Full use of this arching action is made in a round hole, which is more stable than a flat-walled trench.

4.8. Functions of the Slurry (Bentonite)

It has been noted that during excavation of the wall panels, the functions o. the bentonite slurry were very similar to some of those mentioned in 4.7. In oil-well drilling, the mud used has also similar functions and these are -

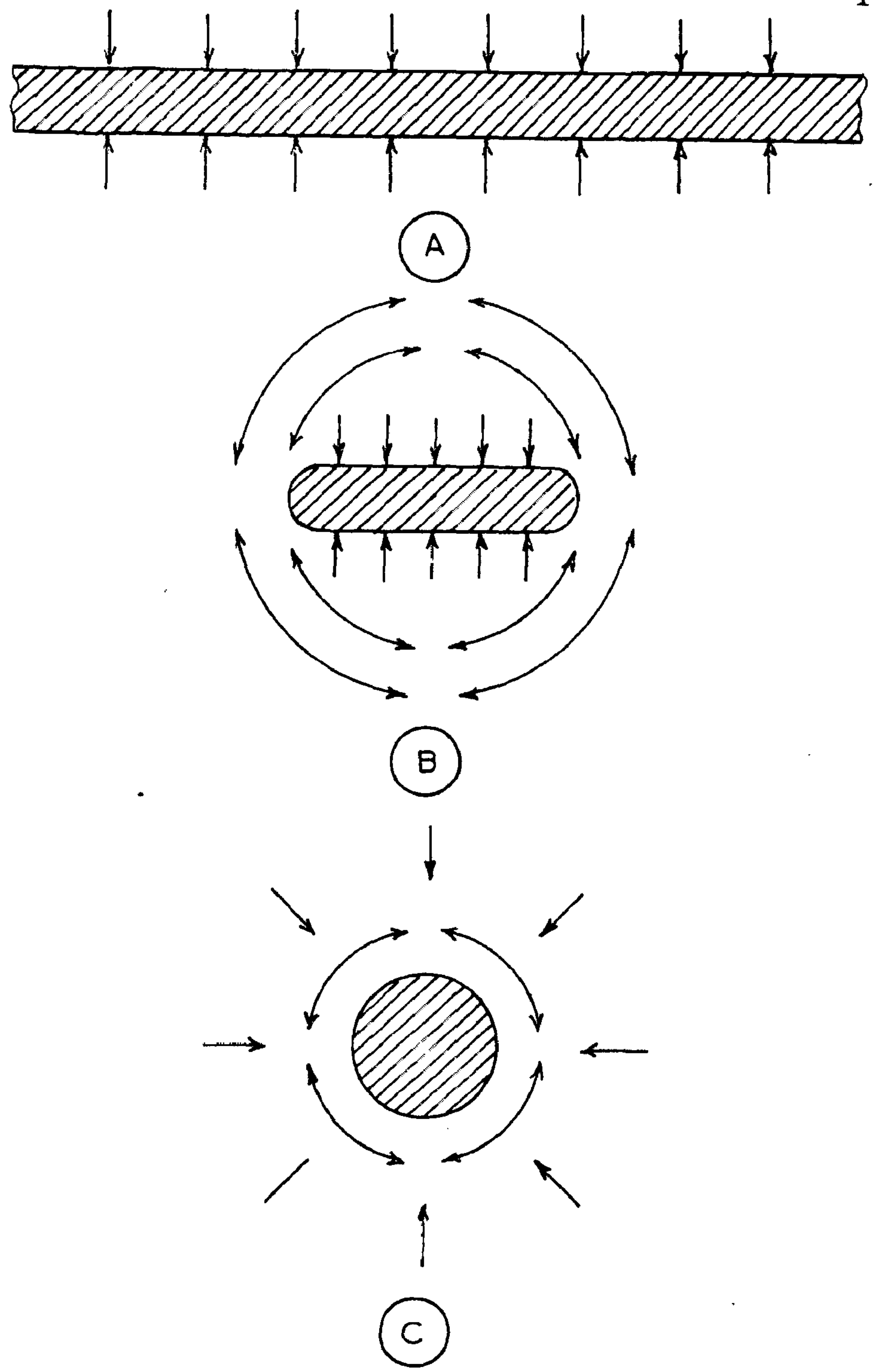


FIG 4·3 SCHEMATIC DIAGRAMS OF ARCHING ACTION

(A) STRAIGHT LONG SLURRY TRENCH EXCAVATION NO ARCHING ACTION AROUND WALL.

(B) SHORT STRAIGHT SECTION OF EXCAVATION SOME ARCHING AROUND ENDS.

(C) ROUND HOLE : FULL ARCHING ACTION.

(i) To prevent the trench wall from caving.

(ii) To deposit a thin filter cake of minimum permeability on the sides of the trench walls, preventing excessive loss of slurry, and also enabling the slurry pressure to balance the soil pressure.

(iii) To keep the cutting in suspension.

(iv) To remove the cuttings from excavation but release them readily in the side trenches after lifting from the panel trenches.

The viscosity of the slurry is an important factor in lifting cuttings, a higher viscosity giving better lifting properties. High densities account for better stability because of higher hydrostatic head, but reduce the rate of excavation.

4.9. Concrete Diaphragm Walls

Mainly the diaphragms are constructed of three type of concrete -

(i) Plain cement concrete

(ii) Reinforced cement concrete, and

(iii) Plastic concrete.

4.9.1. For diaphragms with no structural requirements, plain cement is used.

4.9.2. When the diaphragms are subjected to differential pressure, structural stability requires the use of reinforced concrete.

4.9.3. Stress-strain studies on plastic concrete indicate that plastic concrete of suitable mix, yields adequately before rupture either in compression or tension. Besides, its cohesiveness, im-

-permeability and fluidity in field is quite satisfactory. The plastic concrete with coarse aggregate also indicates good results. Therefore, use of plastic concrete under earth dams is increasing.

4.10. Placement of Concrete

The concrete in the trench is placed by either of two methods -

(i) Tremmie method

and (ii) Pumping method

4.10.1. Tremmie Method - The placement of concrete by tremmie is most common and has been described in Chapter 5.

4.10.2. Pumping Method:

Recently, the Americans have started to place the concrete by pumping methods, through pumps of the piston-displacement type. The method of placement of concrete by pumping ensures faster and continuous supply, but it needs richer mix with plasticizers and retarders. This placement technique is costlier than the tremmie method.

Table 4. 1 - Diaphragm Cutoff under Dams (Slurry)

Sl.No.	Project	Foundation Material	Trench Width	Maximum Head	Depth of Cutoff	Year of Construction
1	2	3	4	5	6	7
1	Kennewick levee, McNary Dam Project, Columbia River, U.S.A.	Sandy or Silty gravels with zones of open gravel K = 0.4 cm/sec.	1.89 m central core	4.92 m	7 m	1952
2	Wanapum Dam, Columbia River, U.S.A.	Sandy gravels and gravelly sands underlain by open work gravels K = 2.5 cm/sec. Av. K = 1 cm/sec.	3.28 m central core	27.8 m	58 m	1959-62
3	Mangla Closure Dam, Mangla Dam Project, Jhelum River, Pakistan.	Sandy gravel with cobbles and boulders; gap graded in range of fine gravel and coarse sand K = 0.4 cm/sec.	3.28 m central core	72.4 m	7 m	1964
4	Duncan Lake Dam, Duncan River, British Columbia, Canada.	Surface zone of sands and gravels over zone of silt to fine silty sand with silty clay K=(surface zone) = 1 cm/sec.	3.28 m upstream berm	32 m short term	18 m	1965-66
5	West Point Dam, Chattahoochee River, Georgia State, U.S.A.	Upper stratum of alluvial soil, alternating layers of clay, silt, sand, sand and gravel; K varies from 1.8 x .01 cm/sec to 3.5 x .00001 cm/sec. Lower stratum of residual soil, brown silty sand; K = 0.6 x .00001 cm/sec.	1.64 m upstream blanket	19.2 m	18 m	1966 Grouting sound rock below the trench upto 30 metres
6	Saylorville Dam, Des Moines River, Iowa, U.S.A.	Surface zone of impervious alluvial sandy clay. Pervious zone, medium-to-fine sand and gravelly coarse-to-fine sand, average K (gravelly sand) = 0.15 cm/sec.	2.52 m upstream berm	29.2 m short term	18 m	1969
7	Broko Pondo Project, Suriname River, Suriname, U.S.A.	Uniform fine-to-coarse sand with some gravel D_{10}=0.1 mm.	1.26 m	17.6 m	4.9 m	1959
8	Wells Dam, Columbia River, Washington State, U.S.A.	Pervious gravels	2.52 m Central core	21.3 m	24 m	1964
9	Yard Creek Lower Reservoir, New Jersey, U.S.A.	Sands gravels, cobbles, and boulders.	2.52 m central core	16.8 m	12 m	1964
10	Camanche Dam, Dike 2, Mokelumne River, Calif., U.S.A.	Alluvial deposits of varying nature. Av. K 7.5×10^{-3} cm/sec.	2.52 m	41 m	29 m	1966

(continued)

Table 4.1 - continued (Concrete Diaphragm Cutoff under Dams)

S.No.	Project	Foundation Material	Trench Width	Maximum Head	Depth of Cutoff	Year of Construction
1	2	3	4	5	6	7
11	Peneos Dam, Greece	Thick layer of alluvial deposits, sands, gravels of high permeability.	0.6 m	50 m	17.5 m	1965
12	Allegheny Dam, U.S.A.		0.6 m		55 m	1964
13	Obra Dam, India	Alluvial overburden with alternate bands of carbonaceous shales and limestones. $K = 1.3 \times 10^{-2}$ cm/sec to 8.6×10^{-2} cm/sec	0.6 m double row	30 m	24 m	1966-67
14	Tenughat Dam, India	Sand, gravels and boulders. K=0.75 cm/sec to 1.0 cm/sec.	0.6 m double row	50 m	15 m	1967
15	Bighorn Dam, Canada	Alluvial deposits of higher permeability with mass of rocks.	0.6 m	91 m	65 m	1969
16	Temporary Cutoff					
16	Ukai Dam, India	Sand, gravel and boulders	0.6 m double row	81 m	30 m	1968
17	Arroio Duro Dam, Brazil	Sandy materials	0.4 m	21 m	35 m	1964
18	Manicouagan 5, Canada (under cofferdam)	Pervious alluvial deposits and boulders	0.6 m	215 m arch dam	70 m	1964
19	Manicouagan 3, Canada	Pervious alluvium	0.6 m double row(piles)	107 m	105 m	
20	Feistritz Dam, Austria	Pervious river bed deposits $K = 2 \times 10^{-2}$ cm/sec	0.5 m.	116 m	47 m	1965-69

Table 4.2 - Machinery used for excavation of trenches

Sl. No.	Rigs	Weight	Horse power	Type of operation	Diaphragm width in in.	Pile dia. capacity in.	Max. depth of excavation (ft)
1	2	3	4	5	6	7	8
1	CIS-56	17.5	110	Percussion	24-60	24-60	1000
2	CIS-60R	19.5	122	Rotary or Percussion	24-60	24-60	1000
3	CIS-58	8.5	109	Percussion	24-48	24-48	500
4	CIS-61R	9.5	131	Rotary or Percussion	24-48	24-48	500
5	CIS-61RD	2.0	32	Rotary	10-28	10-28	165
6	CIS-61RA	3.2	20	Rotary	10-28	10-28	165
7	Tranchesol	4.5	30	Benato Grab	16-24	-	100
8	* RF 6 Rig	20.0	100	Percussion	8-40	24-60	120
9	ESIE	-	-	Grab or Rotary	15-35	-	80
10	Titanea	-	-	Rotary	13.5-40	50	91

* Used at Tenughat Dam

Chapter 5

CONSTRUCTION OF CUTOFF IN TENUGHAT DAM

5.1. General

Economy and time consideration as discussed in Chapter 3, led to the decision of providing concrete diaphragm walls as permanent cutoff in the Tenughat Dam. The tender quoted by Messrs Rodio Foundation Engineering and Hazarat and Co. of Bombay was accepted. The work was entrusted to the firm within the framework of the contract [33] as per 5.3.

5.2. As per construction schedule prepared before starting of the trenching operation, the entire cutoff in the river bed and accomplishment of the closure and diversion of the river bed to be done in two working seasons only, viz. 1966-67 and 1967-68. The closure required laying of 18 million m^3 of earthwork to come to a safe level against a diversion discharge of 10618 m^3 per second. Before the raising of the dam, R.C. capping of the concrete diaphragm had to be done and encased, in a 1.5 m thick plastic clay cover. It, therefore, became imperative to complete the entire cutoff across the river bed and the grout curtain below the cutoff in the working season 1966-67 and allow monsoon flood to pass over the diaphragm after providing protective cover. R.C. capping of diaphragm, grouting of sand between the two walls, 1.5-m plastic cover and closure of river bed was in the working schedule for season 1967-68.

5.3. The decision on the design and construction features were taken after careful consideration. The main points were -

1) Two reinforced concrete diaphragms walls, each 0.6 m thick,

6-m panel length and spaced 3 m apart were to be provided in the river bed; the upstream wall only was to extend on either side (33.6 m on right bank and 134 m on left bank).

2) The bottom of the diaphragm wall was to be keyed at least 1 metre into hard rock.

3) The joints in the two rows of diaphragms to be staggered to avoid any straight-line passage of water from upstream to downstream.

4) The rock below the upstream diaphragm to be grouted through holes formed into the upstream row to a depth of 12 m. The spacing of holes for grouting to be 3 m - c/c.

5) R.C. capping to be provided monolithically with the diaphragm projecting 1 metre above the river bed. The capping to be tapered at the top to facilitate penetration into plastic cover, and to minimise stress concentration.

6) Cross-walls at 30 m to 60 m apart, 0.6 m in thickness, to be provided to connect the two diaphragms for better rigidity of the structure.

7) The top of the diaphragm to be encased in plastic clay of 1.5-m minimum thickness from all sides. The sand upto 2-m depth between the two rows to be excavated and filled back with impervious clay before laying the plastic clay.

8) The sand between the diaphragm walls tobe grouted for better resistance against differential pressures and unequal settlement and to improve back-resistance.

9) The monsoon flood during 1967 to be allowed over the built-up diaphragm keeping it 1.5 m below river bed and providing due

protection.

5.4. Construction Procedure

Messrs Rodio-Hazarat and Co., Bombay, construct the cutoff under Rodio-Morconi process. This type of cutoff wall was first constructed in India by the same firm at the 30 m high Obra Dam [34,43] in Uttar Pradesh. The procedure adopted at this site was the same.

A working platform with its top elevation kept 3.5 m above the groundwater table, was constructed first. It had a top width of 18.25 metres with a view to accommodating the 2 metre deep and 6 metre wide bentonite slurry tanks on upstream side followed by the pretrenches for the two walls in next 3.75 m width, and a haul road for movement of men and equipment on the balance 8.5 m space on the downstream side. The purpose of keeping the top of the platform 3 metres higher than the soil-water table was to allow higher lateral pressure to be exerted against the trench wall than the water pressure for stability against caving and for imparting required gravitational head to the concrete to flow with ease.

A 70 cm wide pretrench was excavated manually all along the diaphragm alignment and was lined with 15 cm thick and 1 m deep reinforced concrete, meant to serve the twin purpose of guiding the trenching chisel during excavation of the trench and providing resistance against collapse of the top of the trench due to movement of machine etc. By the time pretrenches were completed, other construction facilities like the bentonite mixing plant, the concrete mixing plant and bentonite slurry tank were established and trenching for the construction of the diaphragm wall was started on the 26th February 1967. After filling the pretrench with thick bentonite slurry with density 1.4 to 1.45 the Calyx and percussion rigs were mounted on 40-kg rails laid along the pretrenches while the clamshell was operated by a mobile crane. Alluvium was excavated with the Calyx and clamshell rigs, whereas the percussion rigs with a chisel weighing 0.47 tonne was used for excavation in rock. The chisel of the percussion rigs used was 60-cm dia. corresponding to a width of wall with a 20.32-cm hole located centrally and connected to the drill stem to suck out the bentonite slurry containing chipped and broken rock, as part of the reverse circulation. The trench was always kept full with the bentonite slurry under motion to maintain a marsh cone viscosity inbetween 100 to 300. After the trench was excavated to the desired level, the bottom levels were checked by close sounding in the entire panel length and was thoroughly cleaned by the reverse circulation flow of the bentonite slurry. This operation was followed by lowering of the reinforcement cage (Fig. 5.1) firmly welded to an angle-iron frame, into the trench. With the angle frame, 7.62-cm dia. galvanised-iron pipe were also welded 3 metres apart for forming the required holes, in the upstream diaphragm wall for grouting of the bedrock strata.

The tremmie was placed in the middle of the panel, and concreting from the mixers was carried by dump trucks and discharged into the funnelled mouth of the tremmie. The bottom of the tremmie was kept always embedded in concrete poured by about 0.5 m till the entire panel was completed. The mix design for the concrete at

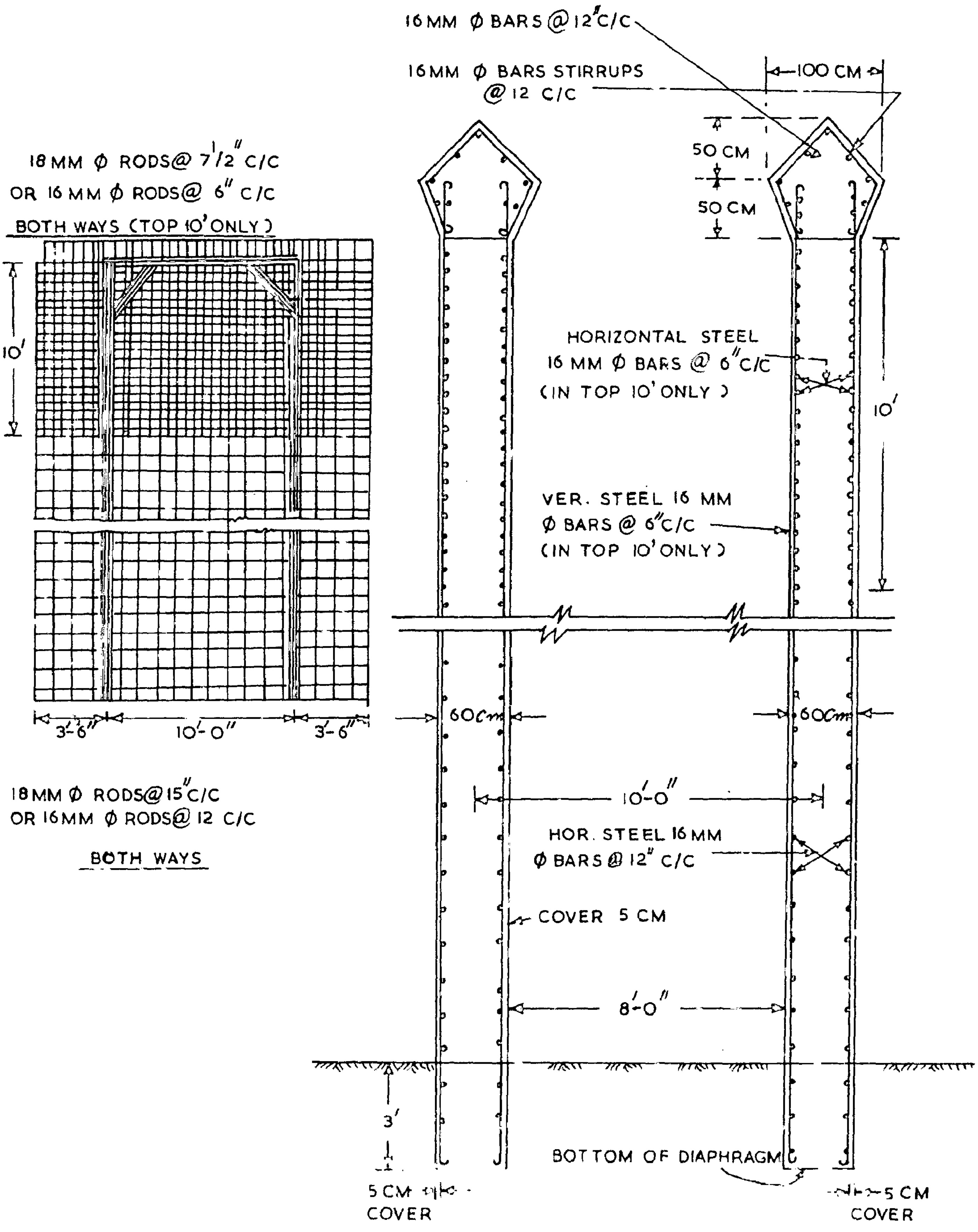


FIG. 5·1 REINFORCEMENT DETAILS IN DIAPHRAGM

Tonughat differed from that at Obra Dam or Ukai Dam. One cubic metre of concrete at Tonughat comprised -

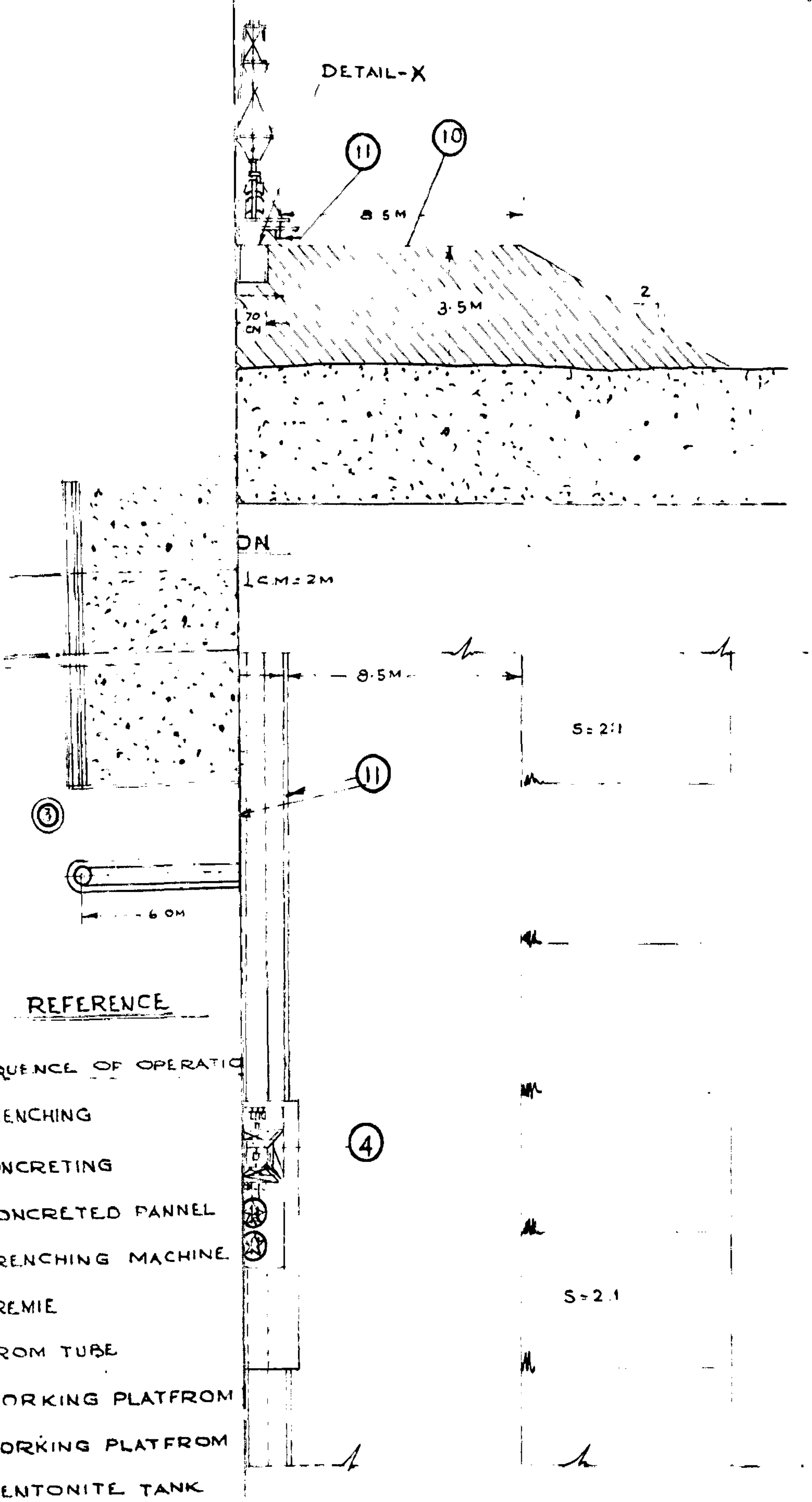
Ordinary portland cement	252 kg
River bed sand (F.M. 2.6)	669 kg
Coarse aggregate (18-mm)	995 kg
Water	252 kg
W/C	0.265
Slump	18 cm
28 days cube strength	178.5 kg/cm^2

(Concrete mix proportion used at Obra dam and Ukai dam are shown in Table 5.1 for comparison.) The bentonite slurry displaced during concreting operations flowed back into the tanks where it was rejuvenated for further use.

5.5. The sequencing of operations adopted for tenching and concreting panel-wise are shown in Fig. 5.2 and detailed soil slurry properties as shown in Table 5.2.

Trenching of panel was taken up first and two cast-steel form tubes 60-cm outer diameter were lowered in dead plumb, at the two ends before concreting was started. After the concrete achieved the initial set, the tubes were pulled out. The excavation and concreting of secondary panels was taken up only after 28 days so as to allow adequate time for the drilling and grouting following the construction of panels.

Top of diaphragm before June 1967 was kept 1.5 metre below the river bed to avoid diaphragm wall working as a protruding weir - launching apron of boulders was provided for protection against scour for a diversion flood of 10,600 m^3/sec. Trenching

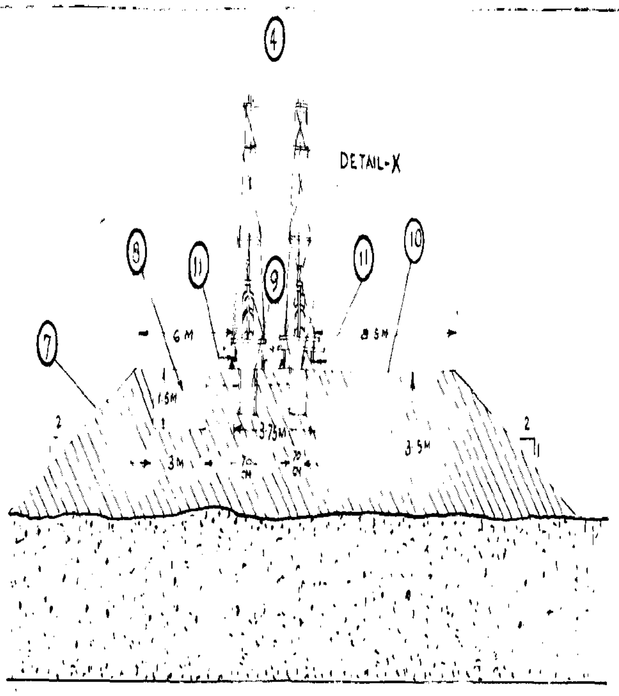


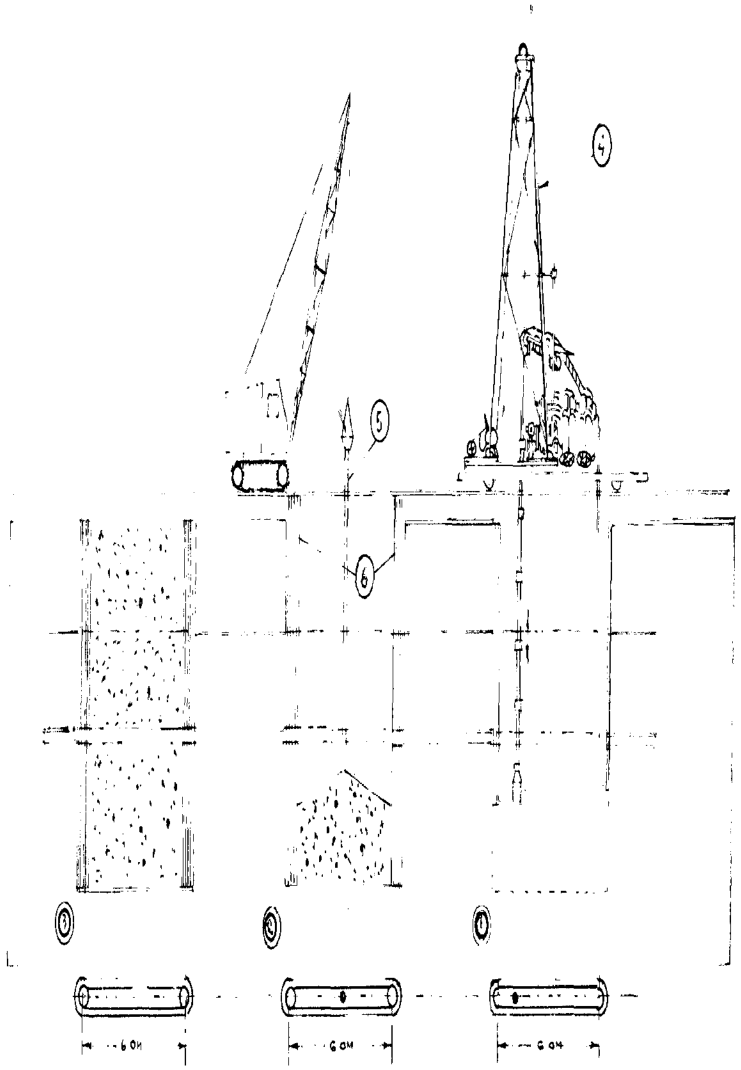
REFERENCE

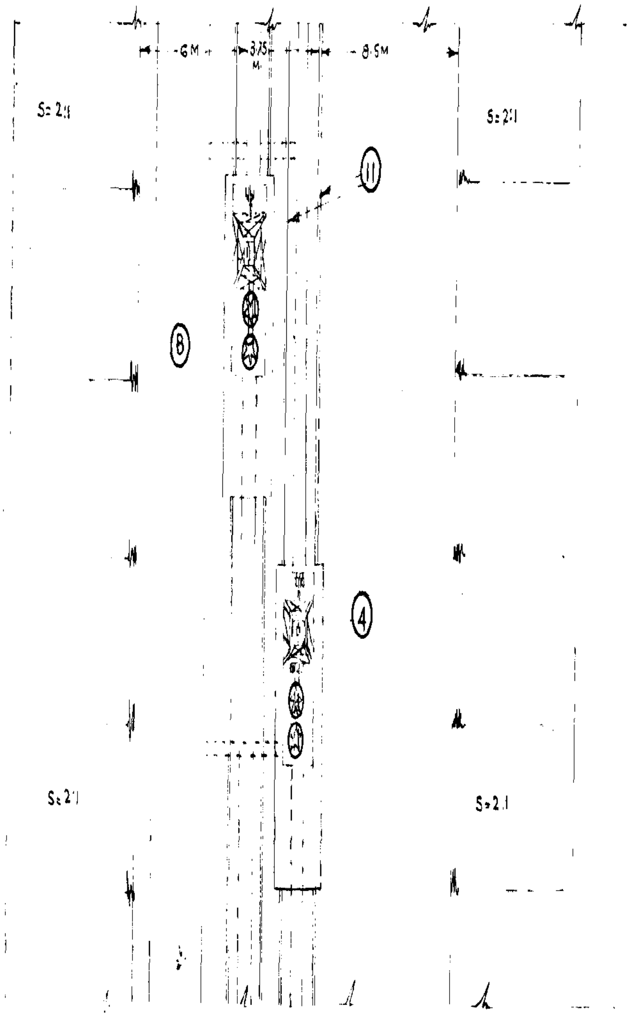
SEQUENCE OF OPERATIO

1. TRENCHING
2. CONCRETING
3. CONCRETED PANNEL
4. TRENCHING MACHINE
5. TREMIE
6. FROM TUBE
7. WORKING PLATFROM
7. WORKING PLATFROM
8. BENTONITE TANK
9. PRE TRENCHES
10. HAUL ROAD
11. RAILS

TAILS OF CONCRETE DIAPHRGM CUTT-OFF
NUGHAT EARTH DAM.







REFERENCE

SEQUENCE OF OPERATION

1. TRENCHING
2. CONCRETING
3. CONCRETED PANNEL
4. TRENCHING MACHINE
5. TREMIE
6. FROM TUBE
7. WORKING PLATFROM DETAILS
7. WORKING PLATFROM
8. BENTONITE TANK
9. PRE TRENCHES
10. HAUL ROAD
11. RAILS

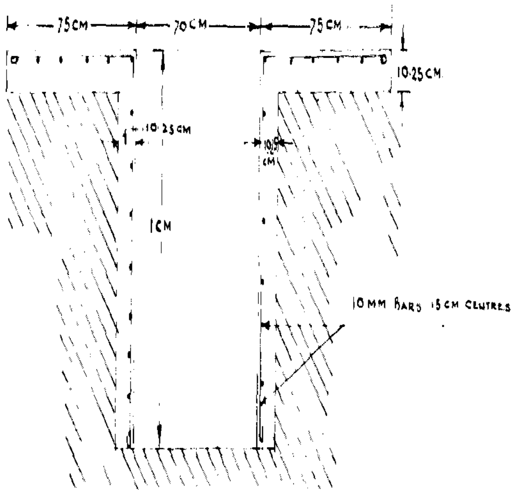


FIG [illegible] CONSTRUCTION DETAILS OF CONCRETE DIAPHRGM CUTT-OFF FOR TENUGHAT EARTH DAM.

Table 5.1 - Concrete Mix for Obra Dam (Uttar Pradesh) and Ukai Dam (Gujarat)

Obra Dam		Ukai Dam	
For (1: $1\frac{1}{2}$:3 mix)		(Proportion by weight for plastic concrete)	
Cement	1150 kg	Cement	1.7
Aggregate -			
3/4 in. to 1/2 in.	45 cft	Clay (Black Cotton, PI=25 to 30	3.0
1/2 in. to 3/8 in.	45 cft	Sand (F.M.=3.7 to 4.2	6.0
Sand	50 cft	Water	4.6
Water	50 litres/2 bag mix		
Slump	6 in. to 9 in.	Slump	25 cms
		90 days cube strength	25 kg/cm^2

Table 5.2 - Foundation Soil and Slurry Properties at Tenughat Dam

Soil Property		Slurry Property	
1. Type of soil	- Medium to coarse sand	1. Slurry (for Bentonite) -	
2. D_{10}	- 0.23 mm	LL	- 300 to 350
		PL	- 50 to 55
		PI	- above 250
3. D_{50}	- 0.55 mm		
4. Max. void ratio	- 0.79	2. a) Initial density -	1.4
5. Min. void ratio	- 0.49	b) Density in trench	1.20 to 1.25
6. Relative density	- 25% to 55%	3. Slurry level	- 3 m above sub-soil water level
7. Sp. Gravity	- 2.62		
8. F.M.	- 2.6		
9. Uniformity Cooff.	- 2.27		

Trench Dimensions

1. Length of panel	- 6 m
2. Width of panel	- 0.6 m
3. Penetration of slurry	- 15 cm to 25 cm
4. Depth of panel	- Upto 1 m in hard bedrock
5. Collapse of sides	- None
6. Any chemical added	- Nil

operations were started on 26th February and the first panel was cast on the 6th March 1967. The portion of the diaphragm in the river bed was completed by the end of May 1967, the launching apron was provided and working platform completely removed by 15th June. The extension of the upstream concrete wall in the flanks continued till middle of September 1967. The grout curtain below the upstream wall through 3.7-cm dia. holes placed at 3-m intervals was completed by December 1967. The grouting procedure envisaged grouting of top 1.5 m first so as to build up a roofing and ensure watertight joint between the bottom of the diaphragm and the bed rock. The grout intake in the top section was generally larger than in the lower sections.

The entire cutoff diaphragms in two rows for a length of 323 metres togetherwith cross-walls and single row in a length of 134 metres, making a total of 5780 sq. metres was thus built in 150 working days at a daily average of 38.5 m^2 of wall area. During the monsoon, peak floods varying from 3145 m^3/sec. to 4570 m^3/sec. passed over the built, but protected, diaphragm and soundings taken after receding of flood revealed that upstream apron had remained unaffected while the downstream had launched to a depth of 1.5 metre only. The width of the 1.5 m thick launching apron was kept 15 m on both sides. This arrangement stood the monsoon flows without any damage, whatsoever. This was an unique achievement since there were no past records of allowing floods to pass over diaphragms.

With the monsoon over, operations were again resumed after rebuilding the cofferdam and removal of launched apron. The top 60 cm of the built concrete diaphragm walls which does not ensure sound concrete, was chipped off, as provided for in the specifications, and further raising of the wall and placing of the cap was taken up as conventional R.C. work. Having completed this, the sand inbetween the diaphragms was removed upto a depth of 1.5 metre and backfilled with compacted impervious clay before the plastic clay encasement was built up.

The plastic clay consisted of clays having liquid limits higher than 40% which were compacted around the concrete diaphragm walls in 10-cm layers at moisture contents 5% more than O.M.C. The mechanical composition of a few samples is given in Table 5.3.

Table 5.3

Plastic clay used at Tenughat Dam

	Sand	Silt	Clay	OMC	MDD	L.L.	P.L.	P.I.
1.	57.2	8.65	34.15	16.2	112.2	41.5	18.95	22.55
2.	39.75	26.45	33.80	15.30	112.6	45.0	18.35	26.65
3.	35.50	38.80	25.70	15.8	112.2	47.0	18.30	28.70

The compaction of the plastic clay inside the wall was achieved manually and by pneumatic tampers till it rose to the top of wall after which normal compaction by roller was done. For this purpose, light rollers of 3-tonne capacity were used and compation achieved was of the order of 95% to 97%.

5.6. Grouting of Sand inbetween the Diaphragms

The results of permeability test of sand inbetween the diaphragms, carried out before and after grouting in the test slot, revealed that from point of view of reducing permeability of sand between the two diaphragms, this grouting was not very effective.

But grouting of the sand did help in improving the imperviousness of the diaphragm walls along the joints and would further help in adding structural stability to the two diaphragms which with grouted sand inbetween them would better resist unequal settlements and differential pressures. Initially, it was decided that the sand between the diaphragm should be grouted completely in three rows, the outer rows with cement bentonite and central row with bentonite chemical. Later on, the scheme was changed, and grouting was done only in two rows and that too with cement bentonite only, the outer row near downstream diaphragm was given up.

The grouting of the sand between the diaphragms was done with 3.75-cm (1-1/2-in.) dia. tubes by 'Solotanche' method. The holes were drilled by driving casings inside which the tube-a-manchette rubber sheath opened at pressures ranging from 9 kg/cm^2 to 11 kg/cm^2 and falls down to 4 kg/cm^2 at which grouting is being done. The amount of grout has been restricted to a prefixed quantity of 2.5 m^3/sleeve, the idea being to grout 25% of the voids in the sand mass. A mix of cement+15% bentonite by weight of cement was used for both the rows. As mentioned earlier, the grouting of the sand in this area was not essential for reduction in permeability but it was done to reduce the permeability to some extent as well as to add to structural stability. The diaphragms and combination provide permanent cutoff at Tenughat, as also at the earlier construction of Obra Dam. At Ukai Dam in Gujarat, the diaphragms were only used as cutoffs for pit excavation, and inbetween the diaphragms open trench excavation and filling back was done. This rather a conservative approach was justifiable due to the disaster potential of, and possible fail-

-ure for the large town of Surat situated downstream.

5.7. Inspection and Testing

As this type of cutoff wall had not been constructed previously in the country, except at Obra Dam which was not tested under full-reservoir head, the designers wished tobe fully assured about its effectiveness. There were some apprehensions about the watertightness of joints between the panels and between the diaphragm wall and bedrock due to the following reasons:

1) While trenching proceeds, and excavated materials are pumped out, there remains a chance of some of the heavier materials remaining at the bottom of the trench. This further gets aggravated due to some possible minor sand-wall collpses at lower elevations, which may remain as a thin sandwiched layer between the rock and the diaphragm concrete.

2) Since the diaphragm wall is built by flow of concrete through the tremmie kept in the centre of the 6-m panel, chances of sound concrete forming at the ends of the panel in the entire vertical length with the same density, as in the main wall, was rather doubtful.

3) Worst apprehension was about the jointing of the panels, particularly at the bottom where there is every likelihood of bottom sand collecting and forming into a smaller heave, which may form triangular wedge between the two panels. Hence, visual checking for the following were considered necessary and accordingly done -

(a) Joints between the panels

(b) Joints with the bed rocks

(c) Soundness of concrete in the entire depth of wall.

5.7.1. Panel Joints

In order to check the joints between the two panels and the bed rock, and inspection chamber was constructed at the left-bank end (Fig. 5.3) by constructing another cross-diaphragm upto bed rock. This gave a closed chamber of 3 m x 2.4 m, through which visual inspection of the entire depth of joints inbetween two panels and joint of the concrete with the bedrock was carried out. The depths of the concrete walls at this location, below the bed level, was 9 metres and at the time of inspection, the water level on the upstream of the diaphragm was 8 metres. Against, this head of water the top one metre did notreveal any seepage, whatsoever. From this point to the bottom there was some sweating only. The joint also appeared fairly regular and tight with a thin bentonite film of about 3-mm thickness firmly sticking with the wall. The concrete wall was sound and regular. The joint with the bottom rock was also sound and tight with the concrete firmly adhering to the rock. Two inclined holes intercepting the joints were also drilled. The joints were intercepted between 7.5 m to 9 m below the top. The cores revealed the joints abutting tight against each other, separated by a thin film of bentonite, about 3-mm, in thickness only.

5.7.2. Joints of the Diaphragm with Bed Rock

Apart from the visual inspection through the inspection chamber, the efficiency of the junction between the bottom of the diaphragm, and bed rock and the grout curtain below the upstream diaphragm, was checked by drilling nine 3.75-cm dia. test-holes, 0.5 metre downstream of upstream diaphragm wall, at locations which had taken large intake of grout. These tests have indicated permeability of less than 2 lugeons except into 3 metres in two cases where it went up to 7 lugeons.

5.7.3. Soundness of Concrete

Three numbers of vertical holes, in addition to those indicated in 5.7.2 were drilled making a total of 59 metres of core drilling through the diaphragm. The average core recovery in the inclined hole was 70% while the core recovery in the vertical holes varied from 90% to 100%. The test details of concrete diaphroms are shown in Fig. 5.3. There was no segregation of aggregates, honey-comb or any significant intrusion of bentonite except that near a few joints, the wall had to be rebuilt in a depth of 1.5 m. The density of concrete was found to be varying between 2.43 T/m^3 and 2.45 T/m^3.

5.8. Construction Observations

As there was no past experience of this nature of work, utmost caution was taken by both construction and quality-control wings of the project. A few observations made during the construction are discussed below.

5.8.1. At a few joints, concrete in the panel was not found acceptable in a depth of 1.5 m and width of 1.25 m from the top. This was however removed and rebuilt without any difficulty.

5.8.3. As the reinforcement cage was lowered into bentonite slurry filled trench, there was a possibility of a thin film of bentonite sticking inbetween the concrete and reinforcements, causing reduction in bond stress. However, when the top 60-cm of the wall was chipped off, it was found that concrete was firmly sticking to the reinforcement bars, and no visible bentonite film was

KEY PLAN

REFERENCE

KEY PLAN

1 RIVER [illegible]

2 DAM AXIS

3 COFFER DAMS

4 DIVERSION CHANNEL

5 EARTH DAM

6 CONCRETE DIAPHRAGM

PRE & POST GROUT WATER LOSS TEST

M PERCENTAGE CORE RECOVERY

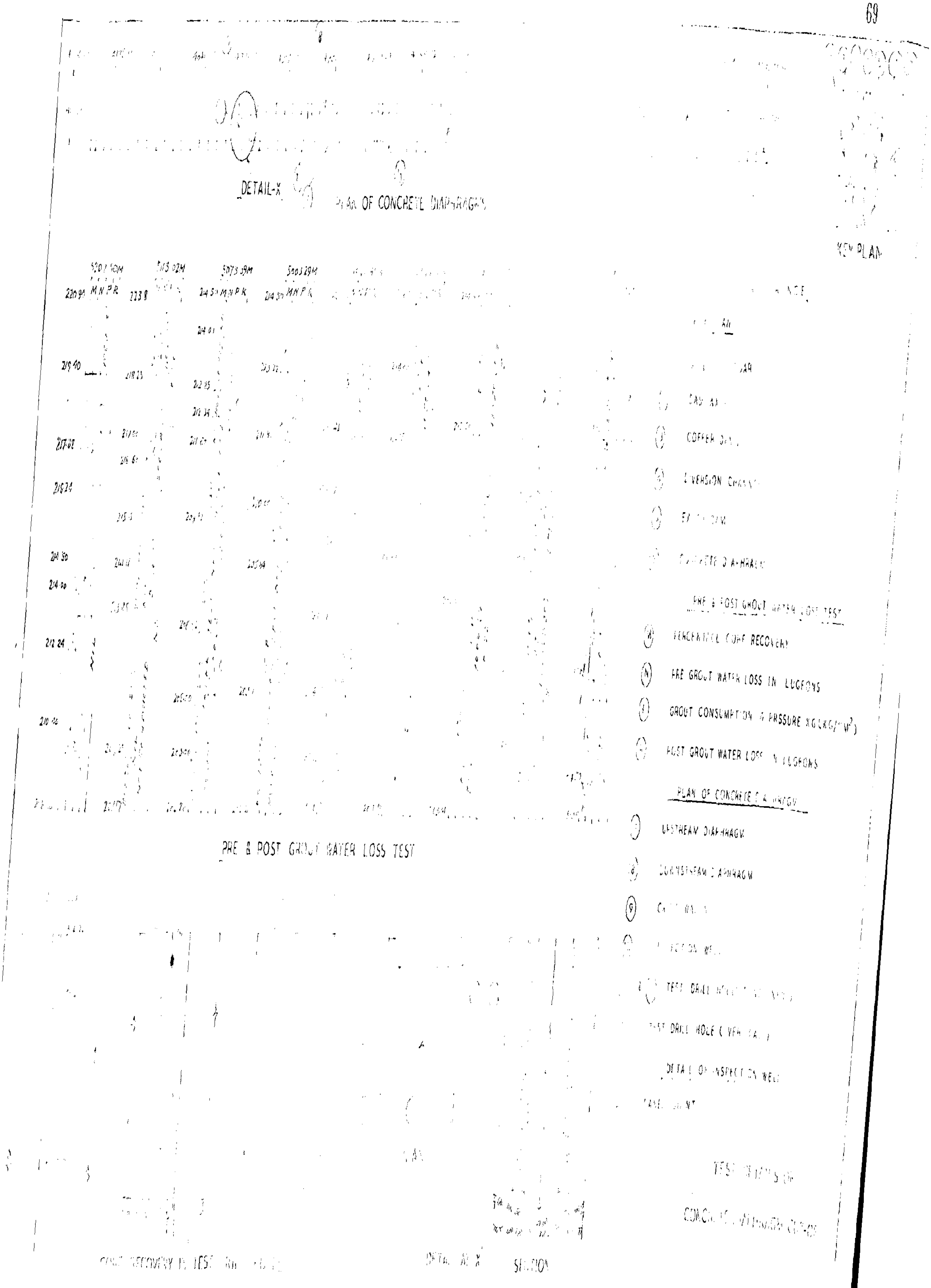
N PRE GROUT WATER LOSS IN LUGEONS

GROUT CONSUMPTION & PRESSURE KG (KG/CM^2)

POST GROUT WATER LOSS IN LUGEONS

PLAN OF CONCRETE DIAPHRAGM

UPSTREAM DIAPHRAGM


DETAIL-X
OF CONCRETE DIAPHRAGMS
KEY PLAN
PRE & POST GROUT WATER LOSS TEST
COFFER DAM
DIVERSION CHANNEL
PERCENTAGE CORE RECOVERY
PRE GROUT WATER LOSS IN LUGEONS
GROUT CONSUMPTION & PRESSURE
POST GROUT WATER LOSS IN LUGEONS
UPSTREAM DIAPHRAGM
DOWNSTREAM DIAPHRAGM
TEST DRILL HOLE
DETAIL OF INSPECTION WELL
SECTION

noticed. Possible reduction in bond stresses, the magnitude of which has yet to be determined by adequate research, could be made good by increasing bond area, through use of smaller-dia. bars or twisted bars.

5.8.3. A thin film (3-mm) of bentonite gets interposed in-between the panel joints. Though the joints have been found to be watertight, the efficiency of the joints could be further improved by forming keys between them and subsequently grouting.

5.8.4. To ensure a completely watertight joint with the bed rock, the reverse circulation of slurry and perfect sounding should be continued till all the loosened materials are removed. The joints should be properly grouted also.

5.8.5. During excavation of sand in the inspection chamber, it was found that the bentonite penetrated 15 cm to 25 cm. The penetration was more wherever there were gravels or pebbles. This would curtail substantially the permeability of the sand in the slurry-saturated zone.

5.9. Precautions

The construction of the concrete diaphragm wall is fully mechanised, hence chances of defects are minimised; however, the following precautions are needed.

5.9.1. While trenching operation was going, the difference between the slurry level and sub-soil water level was kept at not less than 3 m. This was found necessary to prevent collapsing of the trench.

5.9.2. The density of the slurry was checked constantly and was never allowed to drop below 1.2 in the circulation and slurry trench was always rejuvenated.

5.9.3. Reinforcement cage was lowered, after fully cleaning the bed rock by reverse circulation of bentonite slurry.

5.10. Cost ofthe Diaphragm

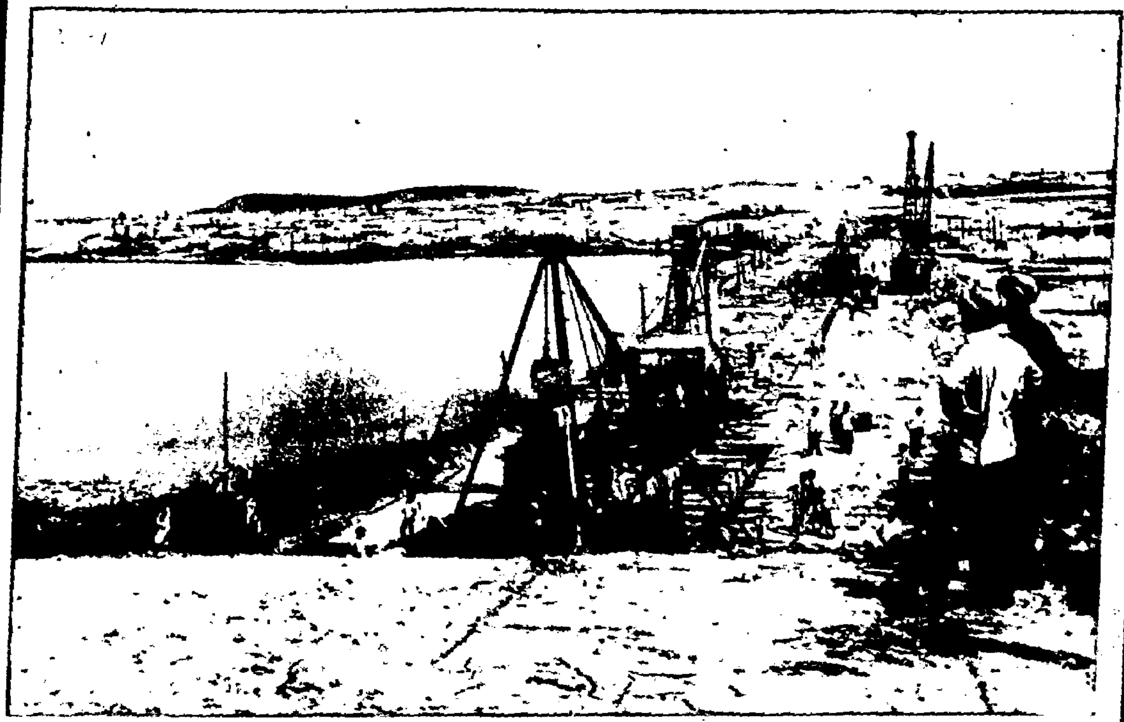
The cost of the completed diaphragm, including laying and removal of protection works for 1967 monsoon flood, dewatering to keep water table 3 m below surface level, grouting of sand and grout curtain below upstream diaphragm wall worked out to Rs. 775 per m^2. This was estimated to be 50% lower then the positive clay cutoff.

5.11. Recent Advancement in Construction

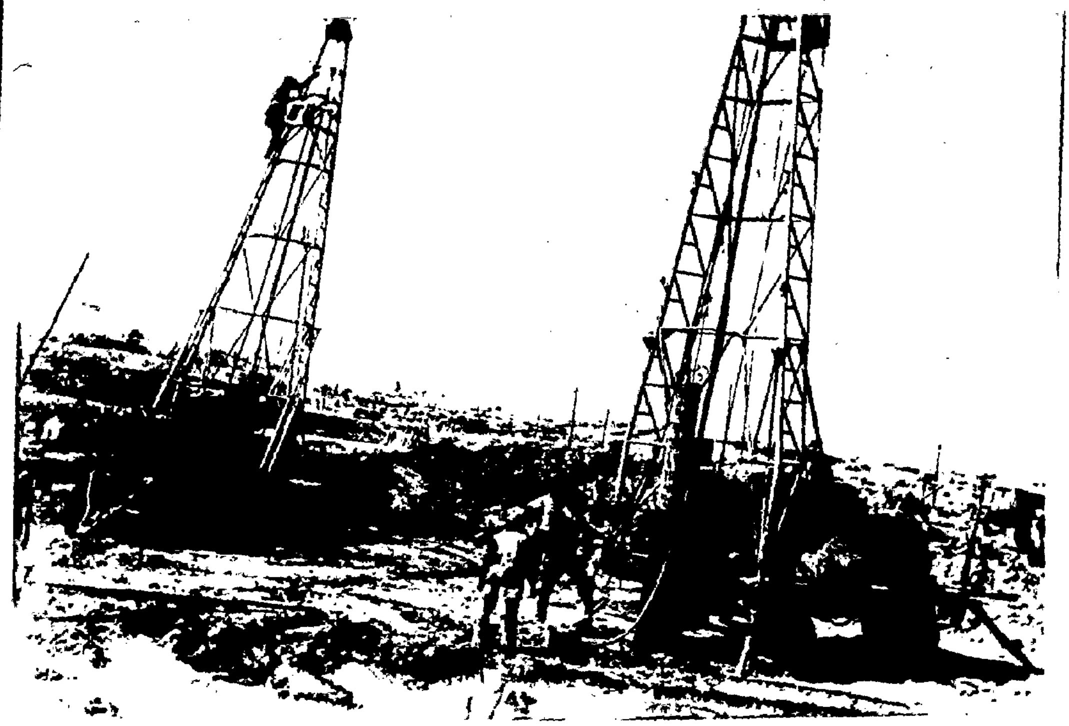
For the last two decades, the number of large earth and rockfill dams has increased sharply, as more challenging solutions were adopted to handle situations of increasing difficulty. The growing trend towards the construction of these dams was obviously due to the lack of other sites with more favourable geological conditions on the one hand, and to the advancement of engineering which provided reliable solutions to problems previously considered unsurmountable, on the other.

Deep deposits of alluvial soil in valleys, otherwise suited for locating earth dams, precluded their use due to lack of skill in providing an effective seepage barrier. But since the advent of grouting technique and other methods, these sites are being gradually developed. Concrete cutoff walls as impervious barrier had been used earlier on alluvial deposits but now it is being used for deep deposits also at relatively moderate depths. The recent construction of 65 m deep single-row concrete cutoff on 91 m high Bighorn [24] embankment dam (above stream bed), is a new landmark in this technique. At this site, the problem of removing deposit of unsuitable sand stone 11 m long and 5 m deep had also to be solved. This was done by using an explosive charge of 18 kg. The sand stone was broken up by two such charges and satisfactorily removed.

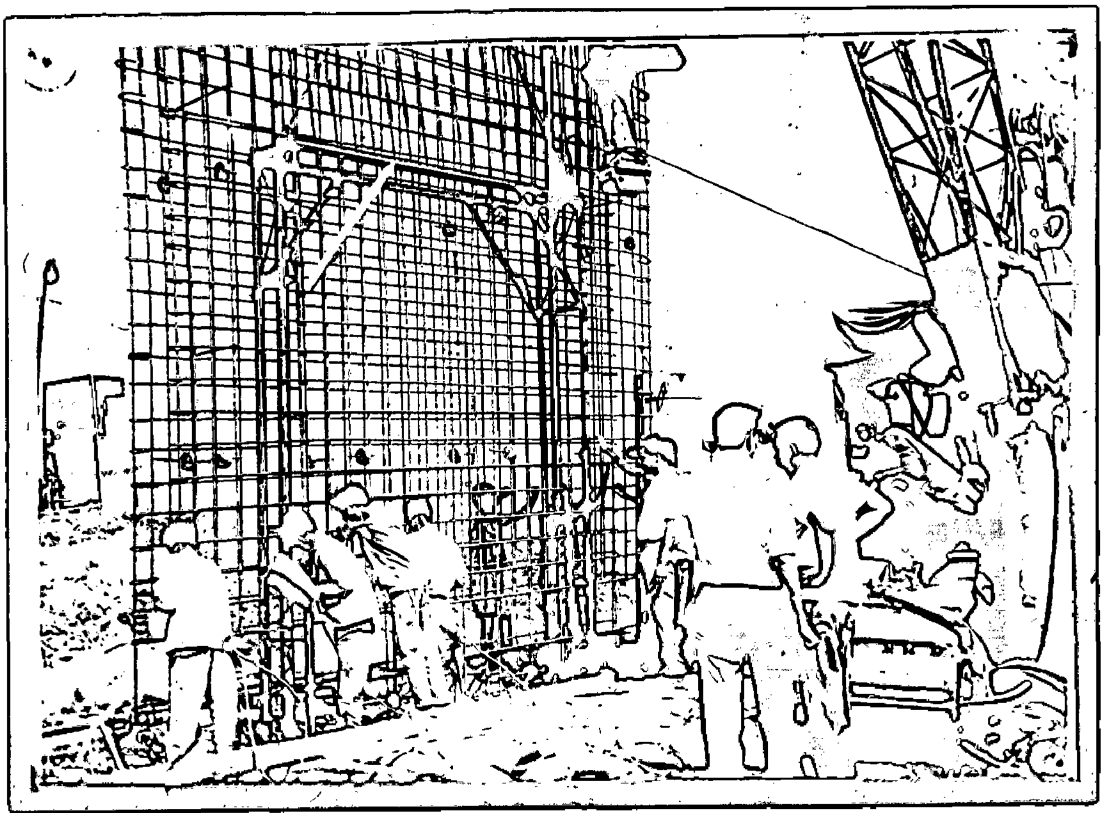
At this site, the stability of trench under such blast conditions has also been demonstrated. It was observed during the blast carried out at 17-m depth, that it ejected all the slurry from the top 9 m of the panel and threw it upto 15 m into the air. The trench, however, remained open and stable; however, fresh slurry was quickly introduced as a safety measure.



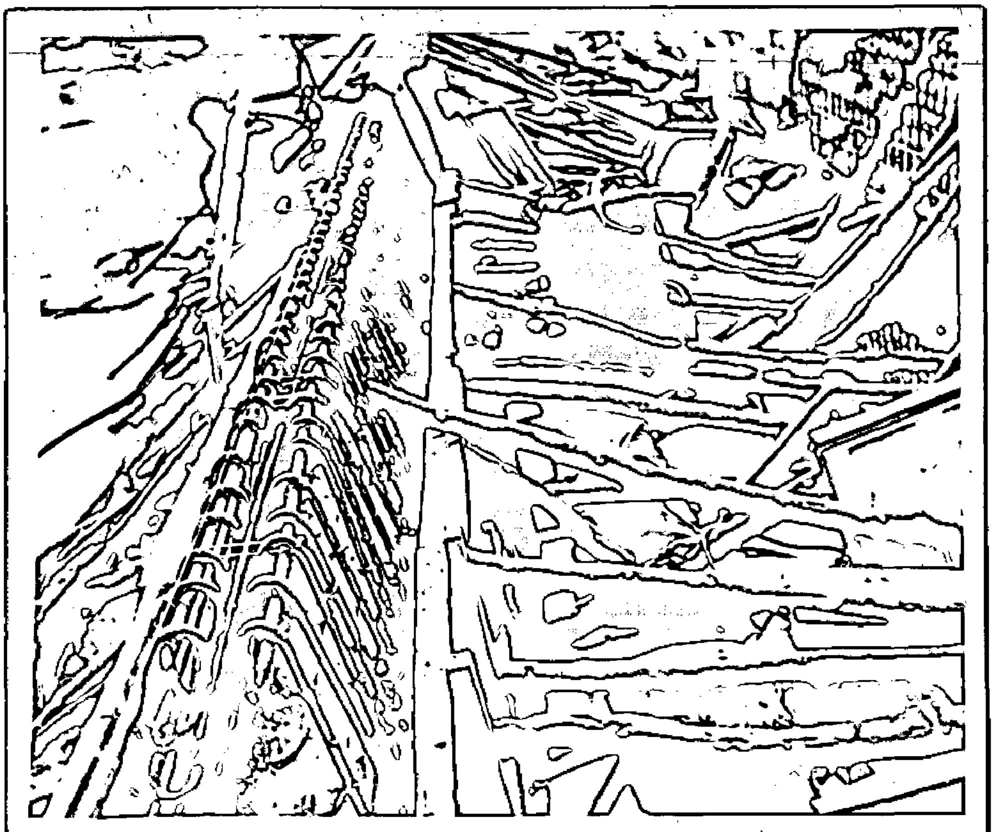
Trenching in Progress



Trenching in the Left Bank



Lowering of reinforcement cage into the trench



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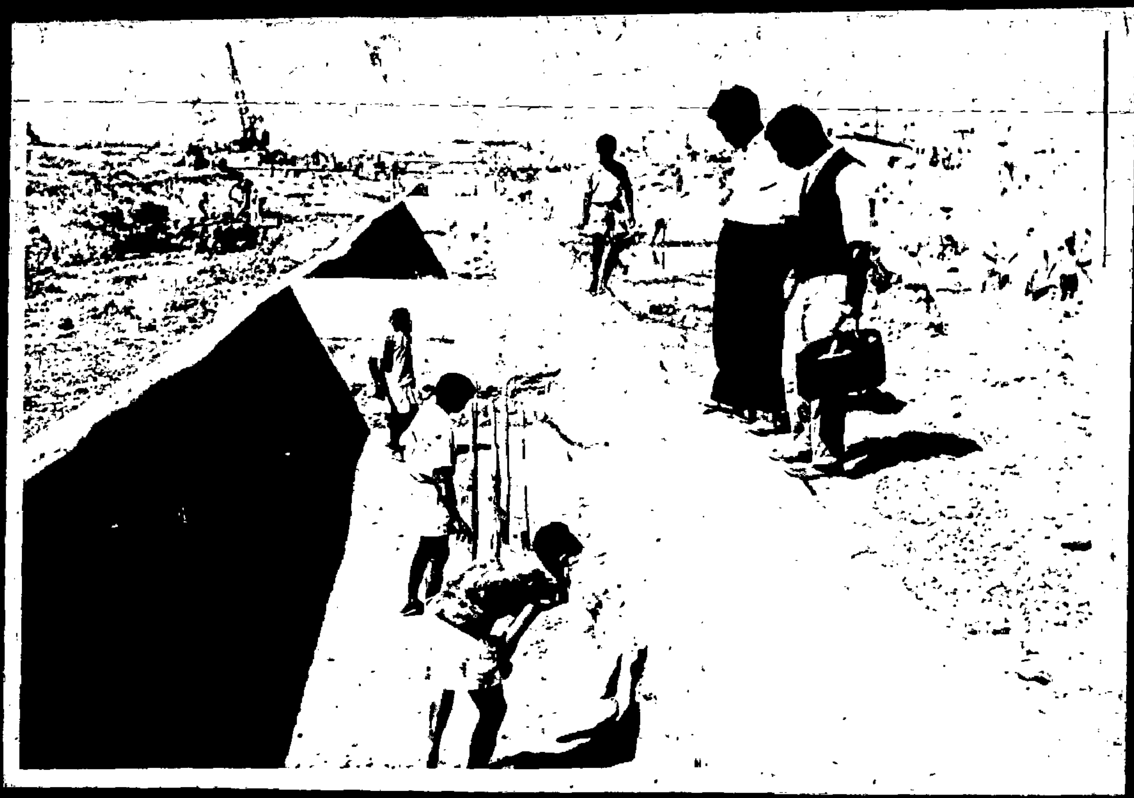
Concreting by Tremmie



Filling Plastic Clay in and around the Diaphragm

Jint Cleaning Chisel

Chapter 6

PERFORMANCE OF THE DIAPHRAGM

6.1. General

So far the concrete diaphragms used as permanent cutoffs under embankment dams are few in number and in the absence of adequate performance records may not be considered fully dependable. For projects involving considerable outlay on diaphragms or similar new and specialised jobs, no opportunity should be missed to study the behaviour of such structures by providing instruments and analysing the recorded data. For studying the behaviour of concrete diaphragm walls, instruments such as piezometers, slope indicators, cross-arm installations combined with horizontal movement devices, surface settlement pins, stress meters and strain meters (where possible) are to be installed and observed. Presently, two such dams, in which concrete diaphragm walls function as permanent cutoff are in operation in this country, i.e. Tenughat Dam [33] and Obra Dam [34] in which some of the above instruments had been installed and observations made regularly.

6.2. Instrumentation in Tenughat Dam

Adequate instrumentation has been done in the Tenughat dam, to observe settlement, pore-pressure variations during and after construction of the dam and slope indicator for measuring the deflection of the diaphragm under different stages of the dam construction and reservoir operations. The details of the instruments installed are as given below.

6.2.1. Piezometers tips

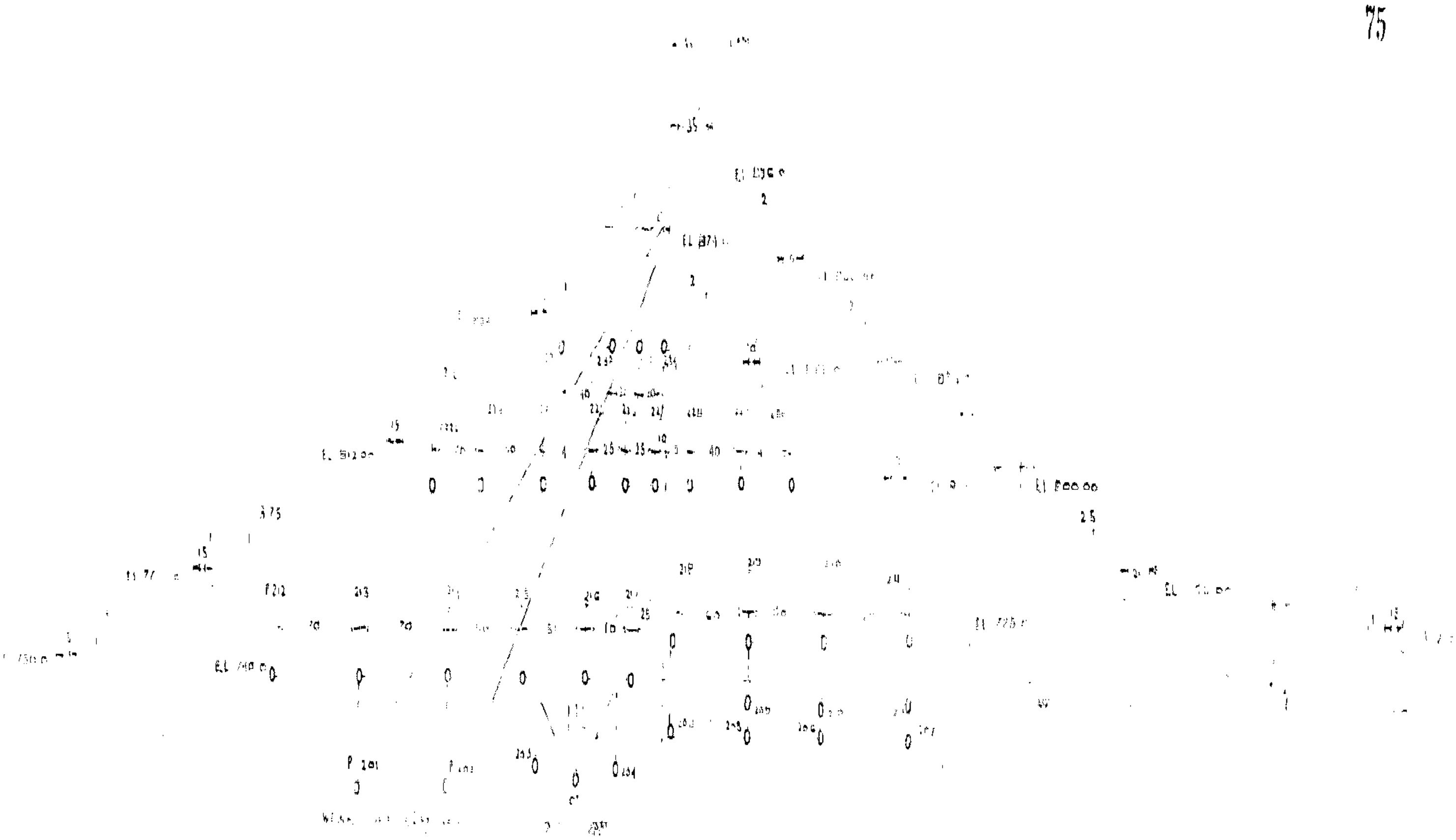
U.S.B.R. piezometers, both foundation and embankment type, have been provided at five sections in the dam at Chainage 124.00, 133.00, 160.00, 164.00 and 167.00. The terminal wells have not been completed so far at Ch. 124.00 and 133.00 and observations are made at the terminal wells at Ch. 160.00 and 164.00, respectively. There is no provision for a terminal well at Ch. 167.00 and the tips of Ch. 167.00 have been connected to the stand of terminal well at Ch. 164.00. The locations of these piezometer tips are shown in Fig. 6.1.

6.2.2. Slope indicator

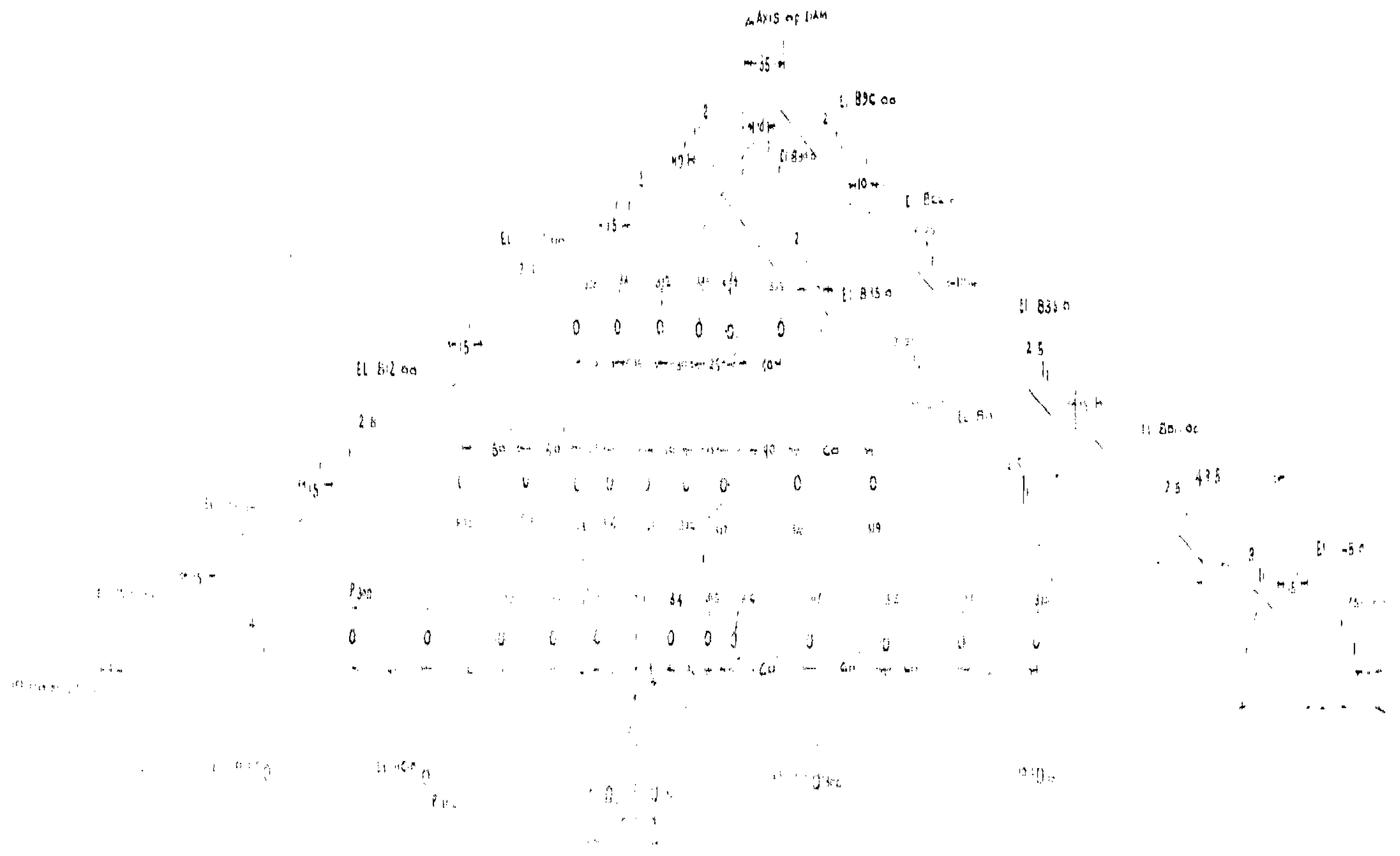
This has been installed at Ch. 158+96.5 panel L-7 of concrete diaphragm in the left bank of the river to record deflection of the same under lateral loadings, both before and after filling of the reservoir. Slope indicator is comparatively a new device adopted in this project to measure angular displacement of concrete diaphragm under lateral loadings. Abrief description of the apparatus which is based on finding null point in a balanced electrical circuit, is enclosed in the end together with a set of observations taken on 21st July 1967 to record initial deflection of the diaphragm from vertical plumb.

6.2.3. Cross arms combined with horizontal-movement measurement device have been installed at two sections but far away from the diaphragm walls.

6.2.4. Settlement plates: These have been installed at different chainages and elevations as indicated below -



ECTION ON STA. 16+00
SCALE



TION ON STA. 14+00

Settlement Plate Locations

Berm level	Chainages (odd)
812.00	137.00
872.00	171.00
804.00	129.00
	Chainages (even)
852.00	114.00
834.00	116.00

6.3. Observation of Data

The regular observations of data in respect of all these instruments installed have not been made. However, in case of piezometer tips [35] at section 164.00, data in case of numbers of foundation tips both upstream and downstream of the diaphragms is available. In case of piezometric tips of sections 160.00 and 167.00, the readings for upstream tips are only available which are not of much significance to study the performance of the diaphragm in absence of downstream tips data. The observations at Ch. 164.00 are taken regularly at daily intervals; however, for studying the performance random data at monthly intervals have been selected (Table 6.1) and studied.

6.4. Pore Pressure Study

Pore pressure observations from piezometer tips connected to terminal well at Ch. 164.00 were started from July 1970 and from then onwards records upto March 1975 are available. The pressures recorded by various foundation tips are tabulated in Table 6.1 and computations for working head, head loss due to up-

Table 6.1 - Piezometers Observations at Tenughat Dam

Instruments installed -	USBR Piezometers							
Chainage -	164.00 (stn. 5000.00 m)							
Dial elevation		751.10	751.63	752.17	746.77	747.31	747.86	748.39
Tip elevation		710.00	710.00	710.00	710.00	710.00	710.00	710.00
Distance from dam axis		396'-0"	237'-0"	92'-0"	77'-0"	66'-0"	60'-0" d/s	250'-0" d/s

Date	Lake-water Level	Tailwater Level	301	302	303	304	305	306	307	Total Head	Head loss due to u/s wall	Head loss due to d/s wall	% Loss due to u/s wall	% Loss due to d/s wall	Total % loss
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
7.7.70	744.40	732.60	-	-	-	-	-	-	-	-	-	-	-	-	-
8.8.70	784.20	732.06	783.10	781.63	783.17	760.77	-	-	-	-	-	-	-	-	-
9.9.70	818.20	736.85	811.10	809.63	811.17	770.77	-	-	-	81.35	43.43	-	-	-	-
8.10.70	792.50	735.15	788.10	787.63	792.17	762.77	-	-	-	57.35	29.73	-	-	-	-
9.11.70	789.90	734.01	785.10	785.63	789.19	759.77	732.31	732.85	745.39	55.89	30.13	27.46	53.80	49.20	103.00
10.12.70	792.95	734.20	789.10	785.63	792.17	761.77	725.31	723.85	-	58.75	31.18	36.46	53.00	61.20	114.20
11.1.71	793.65	733.65	789.10	787.63	793.17	761.77	728.31	726.85	-	60.10	31.88	33.46	53.00	55.75	108.75
11.2.71	796.95	734.40	790.10	789.63	796.17	763.77	727.31	727.85	-	62.55	33.18	36.46	53.00	58.40	111.40
12.3.71	797.20	734.20	790.10	789.63	796.17	763.77	729.31	729.81	-	63.09	33.03	34.06	53.08	54.70	107.78
11.4.71	792.70	733.40	790.10	788.63	791.17	761.77	733.31	732.85	-	59.30	30.93	28.46	53.20	48.18	101.38
10.5.71	790.40	734.35	790.10	786.63	789.17	760.77	733.31	735.85	-	56.05	29.63	27.06	52.80	48.90	101.70
10.6.71	780.35	734.48	781.10	780.63	781.17	758.77	731.31	734.85	-	46.87	21.58	17.46	46.00	37.30	93.30
12.7.71	807.00	734.72	795.10	795.63	799.17	762.77	738.31	727.85	-	72.28	44.23	24.46	61.30	33.95	95.25
14.8.71	843.65	736.40	841.10	840.63	842.17	795.77	737.31	734.85	-	107.25	47.88	58.46	44.60	54.40	99.00
7.9.71	843.15	735.65	843.10	842.63	843.17	797.77	737.31	728.85	-	107.50	45.38	60.46	42.25	56.20	98.45
10.10.71	779.00	734.00	776.10	790.63	777.17	762.77	736.31	732.85	-	45.00	16.23	26.40	36.60	58.85	95.45
10.11.71	772.20	732.45	773.10	779.63	774.17	755.77	735.31	735.05	-	39.75	16.43	20.46	41.50	51.60	92.10
11.12.71	778.10	732.35	773.10	777.63	777.17	755.77	736.31	728.85	-	45.75	22.33	19.46	48.80	41.00	89.80

(continued)

Table 6.1 - continued

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
11.1.72	785.05	732.45	781.10	782.63	784.17	758.97	743.31	735.80	-	52.60	26.08	15.66	49.40	29.80	79.20
10.2.72	791.28	732.60	787.10	789.63	792.17	760.77	738.31	734.85	-	58.68	30.51	22.46	51.25	33.35	84.60
8.3.72	794.36	732.65	778.10	792.60	794.17	763.77	740.31	729.85	-	61.71	31.12	23.46	50.40	38.00	88.40
9.4.72	794.95	730.70	790.10	793.63	794.17	763.77	740.31	731.85	-	64.25	31.18	23.46	48.5	36.55	85.05
10.5.72	794.60	732.65	790.10	792.63	794.17	761.77	740.31	732.85	-	61.95	32.83	21.46	55.20	34.70	87.90
9.6.72	792.90	732.90	789.10	791.63	792.17	758.77	737.31	727.85	-	60.00	34.13	21.46	56.75	35.80	92.75
14.7.72	795.30	732.40	786.10	794.03	797.17	766.17	746.31	742.85	-	62.90	29.13	19.86	46.40	31.60	77.00
12.8.72	832.60	732.60	824.00	823.63	824.17	776.77	737.31	729.85	-	108.40	61.43	39.46	56.65	36.40	93.05
14.9.72	846.20	733.06	820.10	845.63	805.17	778.77	742.31	739.8	-	113.14	67.43	36.46	59.50	32.30	91.80
10.10.72	843.85	731.00	809.10	842.63	780.17	803.77	736.31	736.85	-	112.85	40.08	67.46	35.50	59.75	95.25
9.11.72	838.20	729.80	808.10	837.63	769.17	806.77	738.31	738.85	-	108.40	31.43	68.46	29.00	63.25	92.25
10.12.72	839.60	727.55	826.10	837.63	839.17	806.77	738.31	739.85	-	112.05	32.83	68.46	29.3	61.00	90.30
8.1.73	839.85	726.06	825.10	838.63	839.17	808.77	738.31	738.85	-	113.79	31.08	70.46	27.20	62.00	89.00
11.2.73	840.75	724.90	825.10	840.63	840.17	811.77	739.50	739.85	-	115.83	28.98	72.27	25.10	62.40	87.50
10.3.73	840.55	724.68	826.10	840.63	840.17	811.77	739.31	723.85	-	115.67	28.78	72.46	25.10	62.40	87.50
14.4.73	840.20	724.28	825.10	837.63	837.17	823.77	735.37	734.85	-	115.92	16.43	88.40	14.2	76.40	90.60
11.5.73	839.20	723.70	825.10	836.63	836.17	812.77	736.31	736.85	-	115.50	26.43	76.46	22.95	66.25	89.20
10.6.73	823.00	724.65	814.10	822.63	822.17	804.77	734.31	734.85	-	98.35	18.23	70.40	18.58	71.50	90.08
9.7.73	829.09	729.58	815.10	826.63	827.17	806.77	734.31	735.85	-	103.51	22.32	72.46	21.60	70.00	91.60
10.8.73	842.43	730.08	826.10	839.63	838.17	816.77	733.31	733.85	-	112.37	25.68	83.46	22.85	74.25	97.10
11.9.73	840.15	732.30	826.10	838.63	838.17	818.77	746.31	736.85	-	107.85	21.38	82.46	19.85	76.50	96.35
13.10.73	855.80	736.90	837.10	853.63	853.17	827.17	738.31	731.85	-	118.90	28.63	88.86	24.00	74.75	98.75
12.11.73	840.90	731.60	825.10	839.63	840.17	821.77	736.31	733.85	-	109.30	19.13	87.46	17.50	79.80	97.30
9.12.73	841.45	730.72	825.10	838.63	840.17	822.77	736.31	735.85	-	110.73	18.60	86.46	16.90	77.20	94.10
9.1.74	842.10	730.42	828.10	841.63	841.17	829.91	736.31	736.85	-	111.68	12.19	93.60	10.92	84.00	94.92
10.2.74	840.80	726.10	827.10	838.63	840.17	828.77	736.31	730.85	-	114.70	12.03	92.46	10.50	79.50	90.00

(continued)

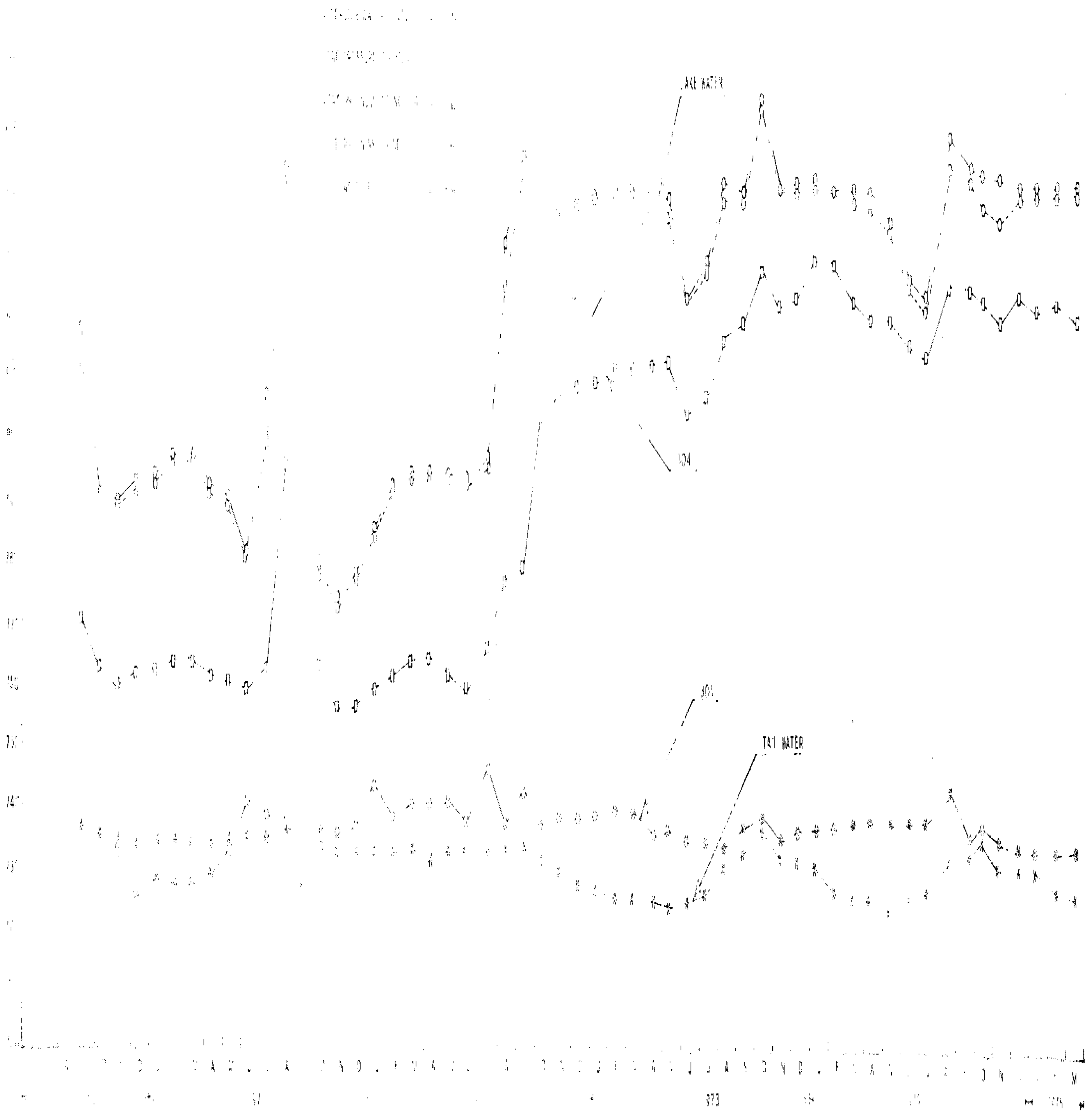
Table 6.1 - continued

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
11.3.74	840.70	725.72	826.10	839.63	838.17	822.77	737.31	726.85	-	114.98	17.93	85.46	15.60	74.30	09.90
8.4.74	840.15	725.16	825.10	836.63	837.17	819.77	737.31	724.85	-	114.99	20.38	82.46	17.75	71.60	89.35
11.5.74	835.40	723.85	822.10	834.63	834.17	819.77	737.17	731.85	-	111.55	15.63	82.60	14.07	74.00	85.07
12.6.74	826.35	725.60	818.10	824.63	824.17	815.77	738.31	737.85	-	100.75	10.58	77.46	10.55	77.50	88.05
9.7.74	823.45	725.58	818.10	820.63	820.17	813.77	737.31	734.85	-	97.87	9.68	76.46	10.20	78.25	88.45
17.8.74	849.45	731.32	826.10	843.67	844.17	825.77	742.31	721.85	-	118.13	23.68	82.46	20.05	70.70	90.75
21.9.74	845.00	732.30	827.10	841.63	842.17	824.77	735.31	721.85	-	112.70	20.23	89.46	18.00	79.50	97.50
12.10.74	843.85	734.68	drratic	822.63	838.17	822.77	737.31	735.85	-	109.17	21.08	85.46	19.35	78.25	97.60
8.11.74	843.10	731.34	erratic	820.63	835.17	819.77	735.31	735.85	-	111.76	23.33	84.46	20.90	75.50	96.00
12.12.74	842.35	730.65	erratic	835.63	840.17	824.77	733.31	735.85	-	111.70	17.58	91.46	15.90	81.75	97.65
9.1.75	842.10	730.18	erratic	839.63	840.17	821.77	733.31	720.85	-	111.92	20.33	88.46	88.15	79.00	97.15
12.2.75	842.05	727.12	erratic	841.63	840.17	822.77	733.31	722.85	-	114.93	19.28	89.46	16.80	77.80	94.60
15.3.75	842.00	726.95	erratic	841.63	840.17	819.77	731.31	726.85	-	115.05	22.23	88.46	19.35	77.00	96.35

-stream concrete diaphragm wall recorded by piezometer tip no. 304 inbetween the two rows of the concrete diaphragm walls, head loss due to the downstream piezometer located about 3 m downstream of downstream diaphragm and the % head loss by the two diaphragms and the total % loss. From Table 6.1 it is observed that the total head dissipation due to the combined two rows is minimum 77% and maximum 114%. (The observed values above 100% may be due to measurement error and/or time lag in the adjustment of pore-pressure in response to headwater and tailwater variations.) The pore-pressure dissipation due to upstream concrete diaphragm wall varies from 10.2% to 61% and that due to downstream concrete diaphragm wall, from 29% to 84%. A plot of all these pore pressures recorded by the piezometers, Lakewater elevation and tail-water level as ordinate and time as abscissa has been made (Fig. 6.2) which presents a pictorial view of pressure distrubution through the pervious foundation upstream and downstream of concrete diaphragm walls.

From Fig. 6.2, it is observed that the piezometer tips no. 303, which is located about 3 m upstream of the upstream concrete diaphragm wall records about 1 metre less head than the reservoir and the nature of the fluctuations recorded by this tip is similar to that of lakewater elevation. At times, such as November 1971, July 1972, this tip records slightly higher pore pressure than the reservoir level which seems abnormal but it may be due to the rise in reservoir level prior to the date of recording.

The piezometer tip no. 304 recording the pore pressure between the two rows, the curve is almost identical to the reservoir level and pressure dissipation during the construction stage recorded are higher than in the latter period when the reservoir


LAKE WATER
SO4
TAP WATER
1973

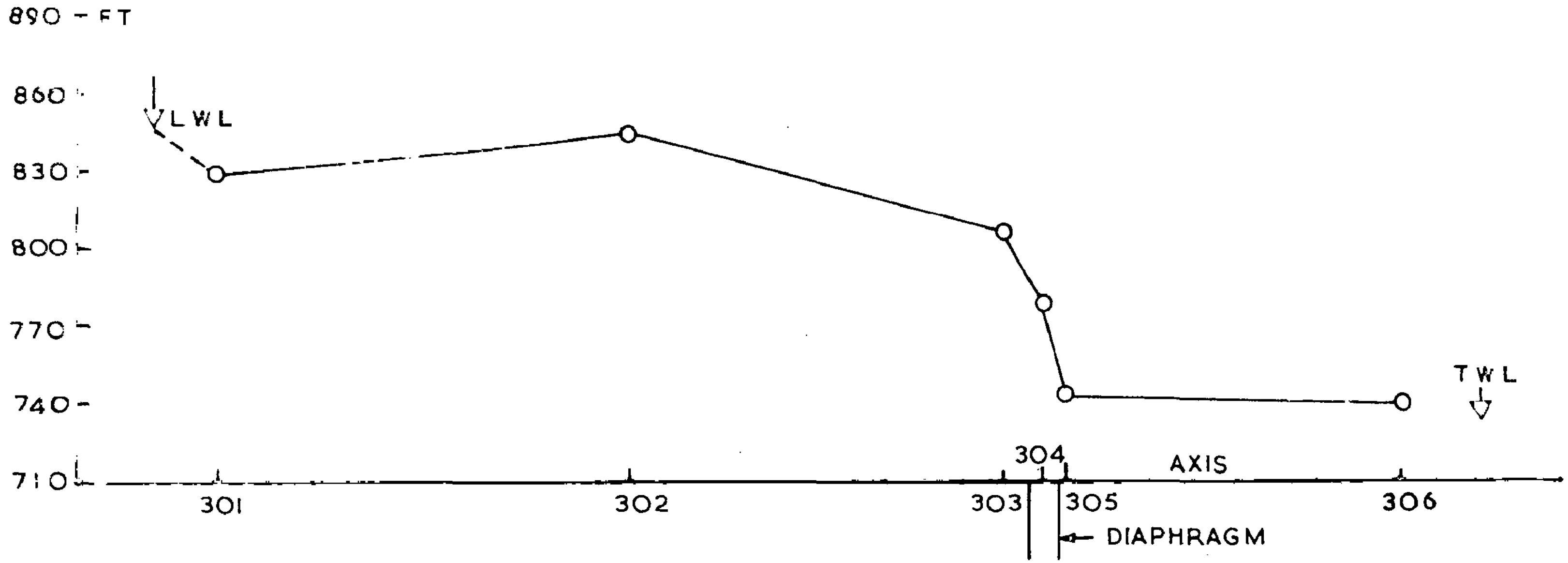
level is almost stationary during major period of the year showing only a few kicks. The pressure dissipation shown is lowest during July 1974 and highest during September 1972, since then the reservoir remains stationary for larger part of the year.

The piezometer tips no. 305, which is about 2 m downstream of downstream concrete wall records almost constant level, and most of the time recording higher pore pressure than the tail water except in the beginning upto May 1971 and during September 1971. The curve shows many kicks whereas tailwater rating is nearly a straight line.

Apart from the above plot, a plot of pore-pressure variation of different tips as ordinate and their horizontal location as abscissa (Fig. 6.3a,b,c) for different dates and years had been made, which shows clearly the pressure dissipation through the diaphragm. In this plot, reservoir level and tailwater level have also been shown. These curves for different dates are similar in nature.

Plots of % pore-pressure dissipation (Fig. 6.4) through the upstream concrete diaphragm wall and downstream concrete diaphragm wall as ordinate and time as abscissa and % pore-pressure dissipation of the combined two rows as ordinate and time as abscissa have been made which show the effectiveness of the two rows separately and the combined effect of the cutoff. The average % pore-pressure dissipation of the combined system is about 90% and during some intervals it is more than 100% which is due to recording of pressure lower than the tailwater level by tip no. 305. As mentioned earlier this could be due to transient conditions. There is no trend for reduction in % head loss over the

TENUGHAT DAM



LAKE AND TAIL WATER LEVEL & TIPS PRESSURE ON 14·9·71

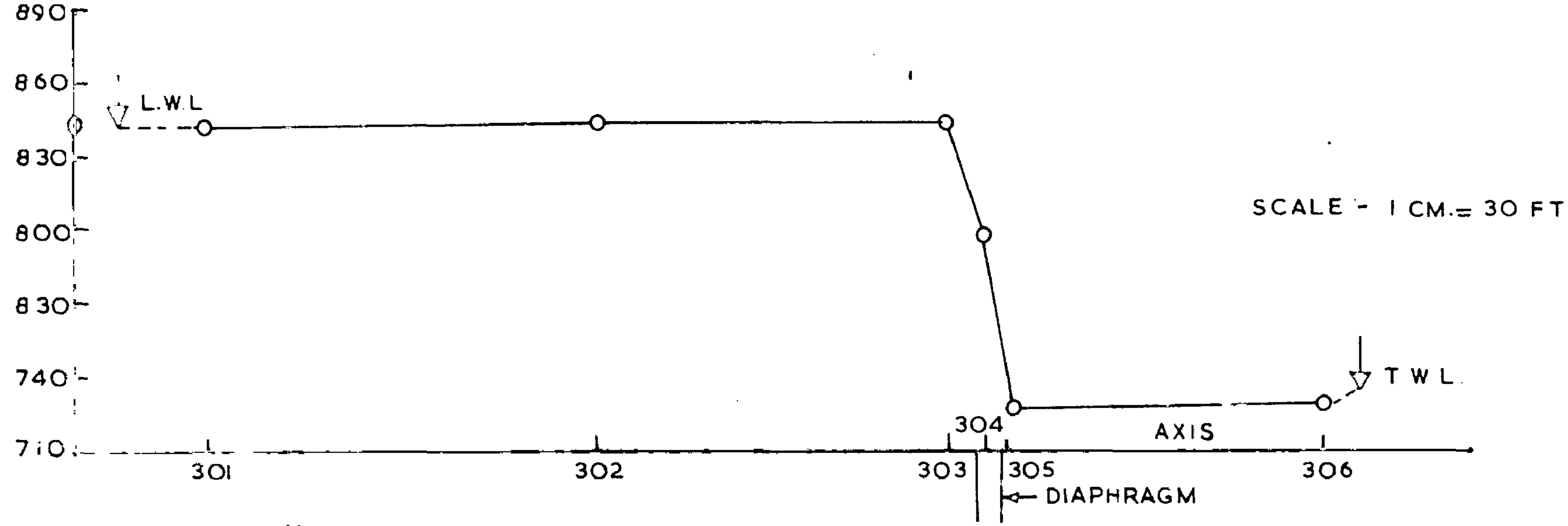
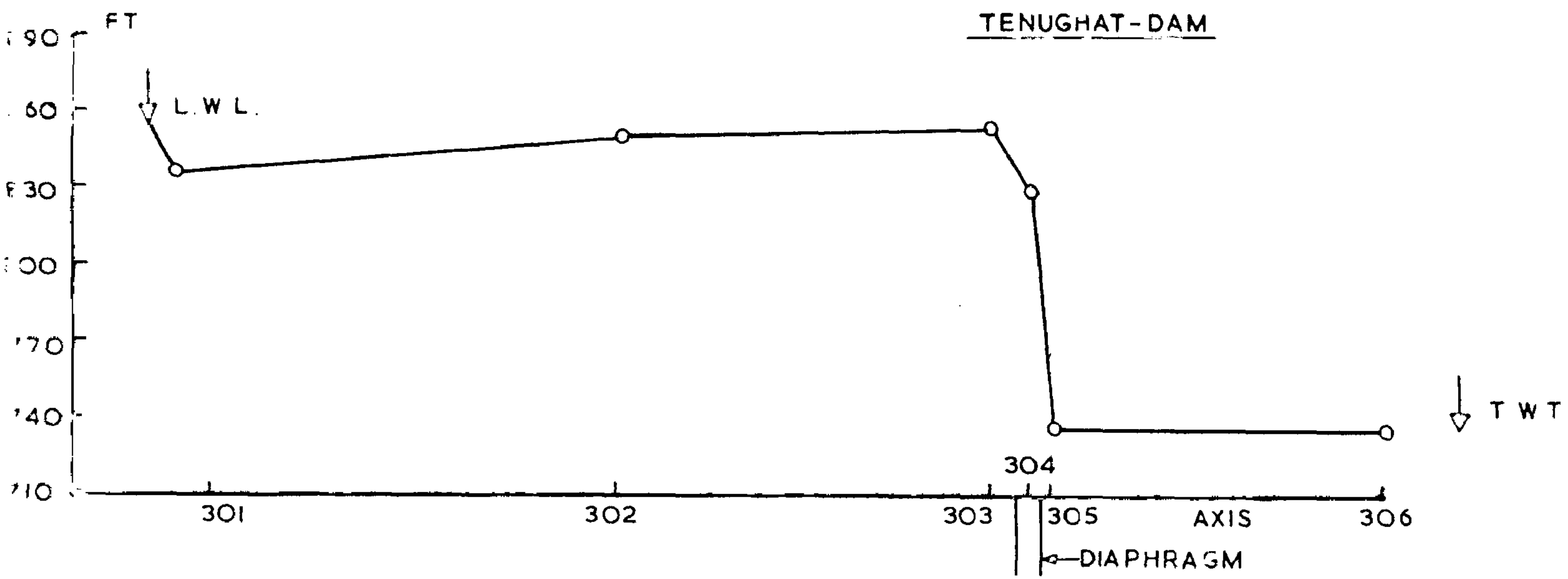
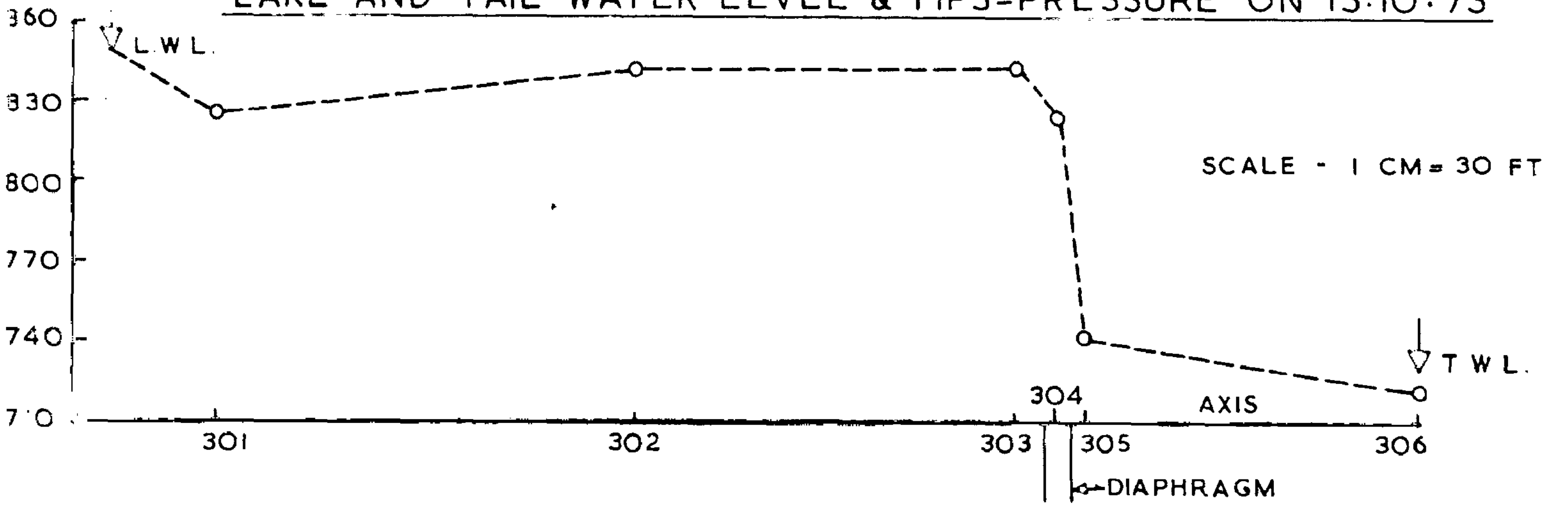


FIG. 6·3 (d) LAKE AND TAIL WATER LEVEL & TIPS PRESSURE ON 7·9·71

TENUGHAT-DAM



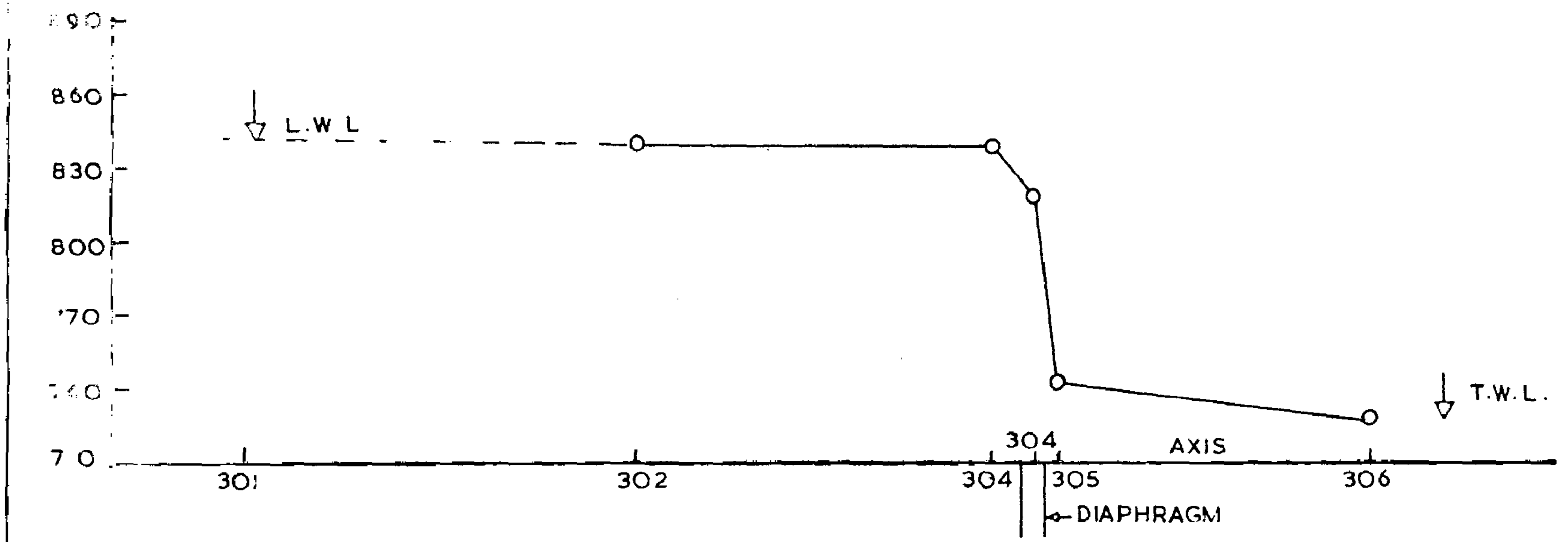
LAKE AND TAIL WATER LEVEL & TIPS-PRESSURE ON 15.10.73



SCALE - 1 CM = 30 FT

FIG. 6.3 (b) LAKE AND TAIL WATER LEVEL & TIPS PRESSURE ON 17.8.74

TENUGHAT DAM



LAKE AND TAIL WATER LEVEL & TIPS PRESSURE ON 15·3·75

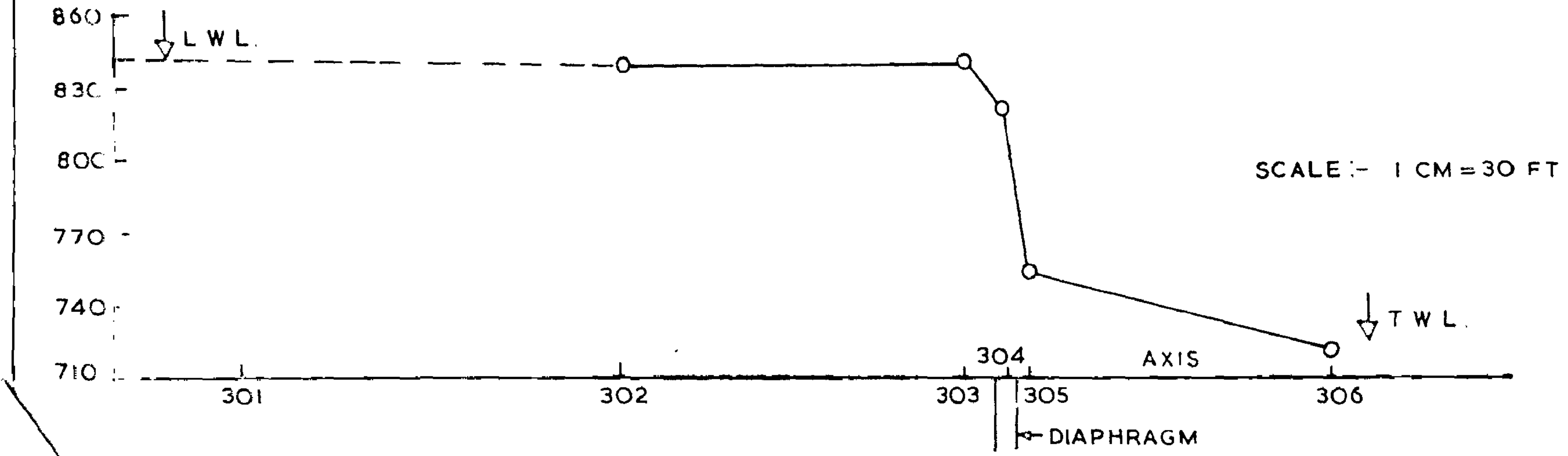


FIG. 6·3(c) LAKE AND TAIL WATER LEVEL & TIPS PRESSURE ON 12·2·75

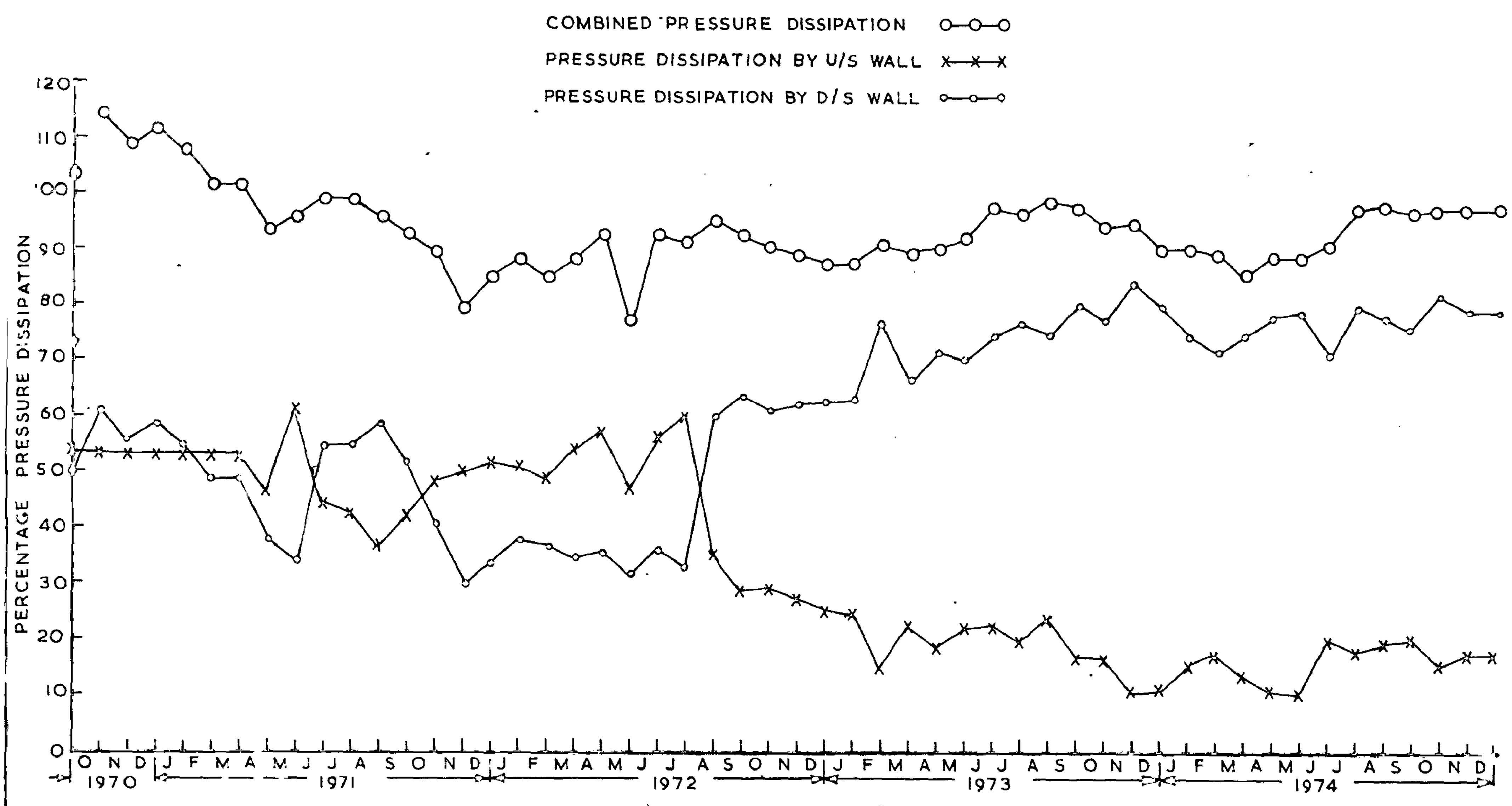


FIG. 6 4 TIME ~ % PORE-PRESSURE DISSIPATION TENUGHAT-DAM BIHAR period of observation - nearly 4 years. This shows that over this period there had been no loss of efficiency of the cutoff either due to cracking of the diaphragm walls or due to loss of gellification of the grouted sand between the two cutoffs.

6.5. Structural Performance of Diaphragm

The thin diaphragm wall was likely to get tilted due to differential pressures in the alluvium on the two sides of the wall. The slope indicator (USBR-type) installed in the left bank at Ch. 158+96.5 panel L-7 shows an initial tilt of 0°-30′ at a depth 13 m (42.5 ft initial readings) shown in Table 6.3. After about a year when the damfill had been raised to a height above the top of the diaphragm, readings were again taken. This has indicated a tilt of 1/100 seconds at the top of the diaphragm wall and 1/1000 seconds at a depth of 7 m. This shows that the diaphragms are structurally sound.

6.6. Seepage Observations

Relief wells have been provided in the dam which do not record any discharge so far. The surface downstream of the dam in the river section is completely dry which also confirms the effectiveness of the diaphragm as permanent cutoff.

6.7. Performance of Obra Dam Diaphragm Wall

Obra Dam has similar cutoff in the river section where the alluvium is also of similar characteristics. Pore-pressure observations through foundation tips (Table 6.2) have been recorded since October 1970. A plot of pore pressures observed by

Table 6.2 - Piezometer Data (Gora Dam)

Instrument installed - USBR (Foundation) Piezometers

Tips elevation Date	511.35 7	493.31 8	511.29 9	521.02 10	495.15 11	522.54 12	Lakewater Level
1	2	3	4	5	6	7	8
5.10.69	615.35	599.35	580.85	589.35	586.37	570.37	617.13
4.11.69	627.61	643.61	596.11	580.62	595.11	577.62	630.42
20.2.70	617.46	621.30	628.71	637.70	623.25	618.83	634.19
2.1.70	631.85	605.35	598.35	591.86	589.37	577.87	633.88
16.3.70	601.34	600.84	594.84	588.35	610.34	574.34	629.43
18.4.70	629.34	594.34	595.34	589.85	595.34	572.84	633.70
17.6.70	627.84	598.34	593.34	591.85	588.84	573.84	633.37
26.8.70	623.34	591.34	593.84	596.35	601.84	580.84	633.04
25.9.70	623.34	598.34	583.34	578.85	587.34	585.34	630.92
24.10.70	613.84	584.84	580.34	581.85	583.44	574.34	630.75
20.11.70	625.85	593.34	59834.	585.35	594.29	589.84	633.70
29.1.71	628.60	588.76	584.48	583.24	584.48	566.63	630.59
23.2.71	625.60	587.74	576.48	574.24	576.48	567.73	631.40
25.3.71	626.60	589.74	575.48	579.24	574.46	567.73	631.07
21.4.71	627.60	583.74	574.48	577.24	572.48	567.73	630.75
28.5.71	625.60	588.74	579.48	572.24	572.48	570.73	631.56

(continued)

Table 6.2 - continued

1	2	3	4	5	6	7	8
30.6.71	628.60	587.74	574.48	573.24	570.48	569.64	631.40
7.8.71	629.60	538.76	575.48	571.24	570.48	570.73	623.05
30.3.71	629.60	590.74	574.48	575.24	572.48	571.73	631.40
30.9.71	629.60	583.74	575.48	573.24	570.48	569.64	632.05
25.10.71	628.60	583.74	572.48	573.24	570.48	571.73	631.73
24.11.71	628.60	588.74	572.48	573.24	570.48	570.73	631.73
24.12.71	628.60	588.74	572.48	574.24	570.48	569.73	631.07
3.2.72	629.10	582.74	569.48	583.46	567.98	571.73	630.92
13.3.72	628.60	592.74	570.48	576.96	570.48	574.73	630.59
15.4.72	628.60	590.74	570.46	577.96	568.48	571.73	631.40
27.5.72	628.60	592.74	591.28	571.96	574.48	568.73	630.04
27.6.72	628.60	592.74	583.48	575.96	570.48	573.73	631.24
15.10.72	629.60	595.24	583.48	583.96	568.48	567.73	631.07
20.12.72	629.60	595.24	583.48	583.96	568.48	567.73	630.09
12.3.73	625.60	594.74	574.48	571.24	575.47	567.73	630.75
15.4.73	629.60	594.74	572.48	580.24	570.48	565.73	631.07
20.5.73	625.60	594.74	576.48	581.24	578.48	571.73	631.24

(continued)

Table 6.2 - continued

1	2	3	4	5	6	7	8
6.8.73	624.60	594.74	576.48	572.74	571.48	570.73	630.75
21.9.73	623.60	594.74	570.48	571.42	569.48	571.73	630.92
11.10.73	625.60	596.74	568.48	580.42	570.48	571.73	630.42
25.11.73	625.60	585.74	569.98	578.74	585.48	579.73	630.75
25.12.73	625.60	537.74	572.48	574.74	570.48	570.73	630.42
21.1.74	625.60	595.74	576.48	572.74	578.48	569.73	630.92
13.2.74	627.60	594.74	571.48	573.96	576.48	566.73	631.07
15.3.74	625.60	595.74	572.48	579.96	574.48	566.73	630.92
29.4.74	627.60	589.74	572.48	578.96	569.48	570.73	631.07
27.5.74	629.60	509.43	572.48	574.24	570.48	573.73	631.24
15.10.74	629.60	595.43	569.48	576.74	575.48	585.73	631.07

different tips has been made (Fig. 6.5) with time as abscissa and pressure as ordinate, which shows pressure variations through the foundation tips. In the beginning these readings show some erratic behaviour but since April 1970 the pressure readings are nearly steady. In case of tip no. 9, which is just downstream of the d/s diaphragm wall, a few readings are erratic upto November 1970, after that all the tips shows similar pressure dissipation as in case of Tenughat Dam. The pressure dissipation through the diaphragm cutoff as indicated by the tips has stabilised and is consistent.

Another plot (fig. 6.6a, b) has been made showing pressure recording of all the foundation tips on a particular date similar to Fig. 6.3 for Tenughat Dam. These indicate substantial pressure dissipation. The % pore-water dissipation could not be computed in absence of tailwater level.

The observations of the only two existing dams in our country in which concrete diaphragm walls have been provided as permanent cutoffs, indicate the performance of the cutoff is fully satisfactory.

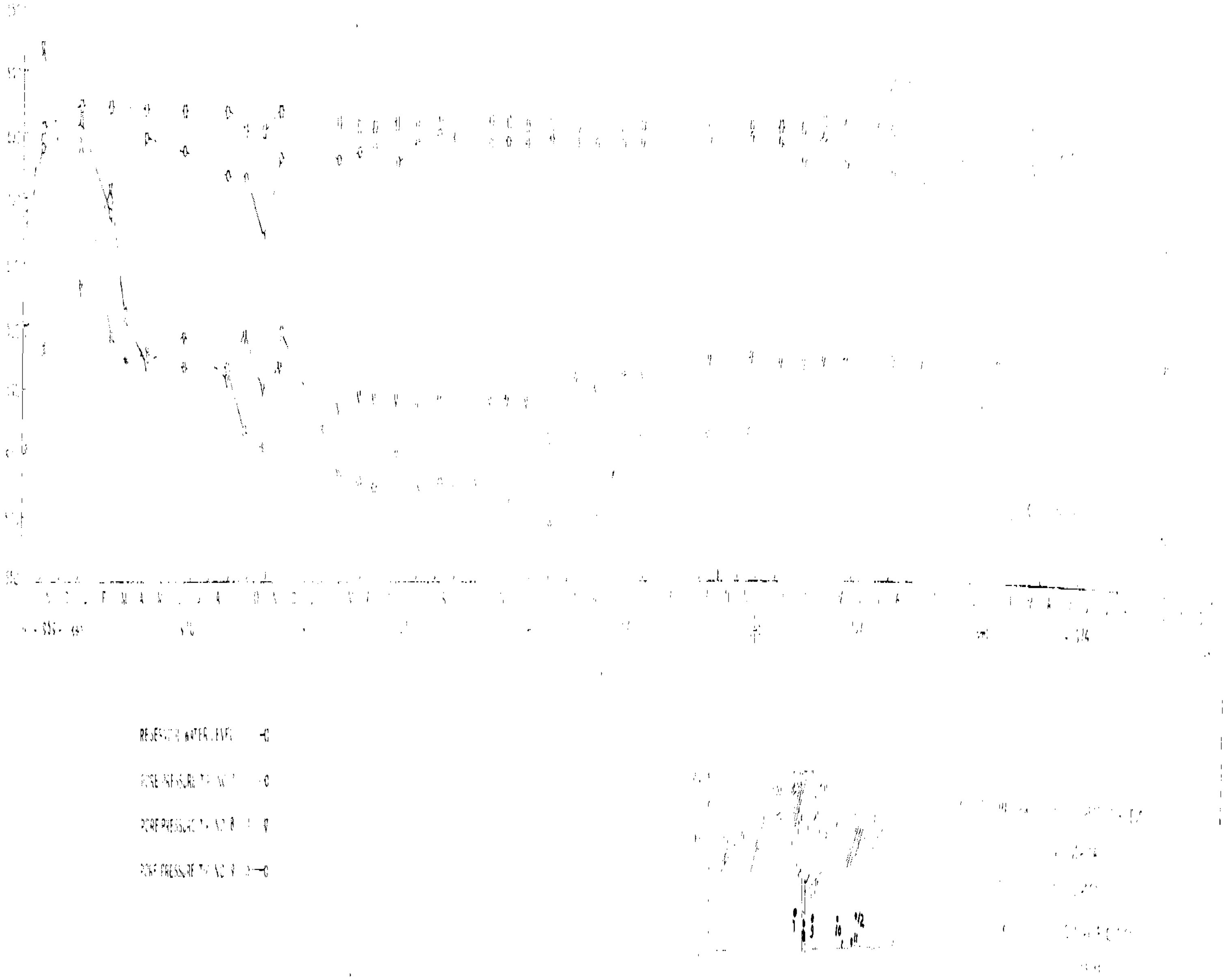
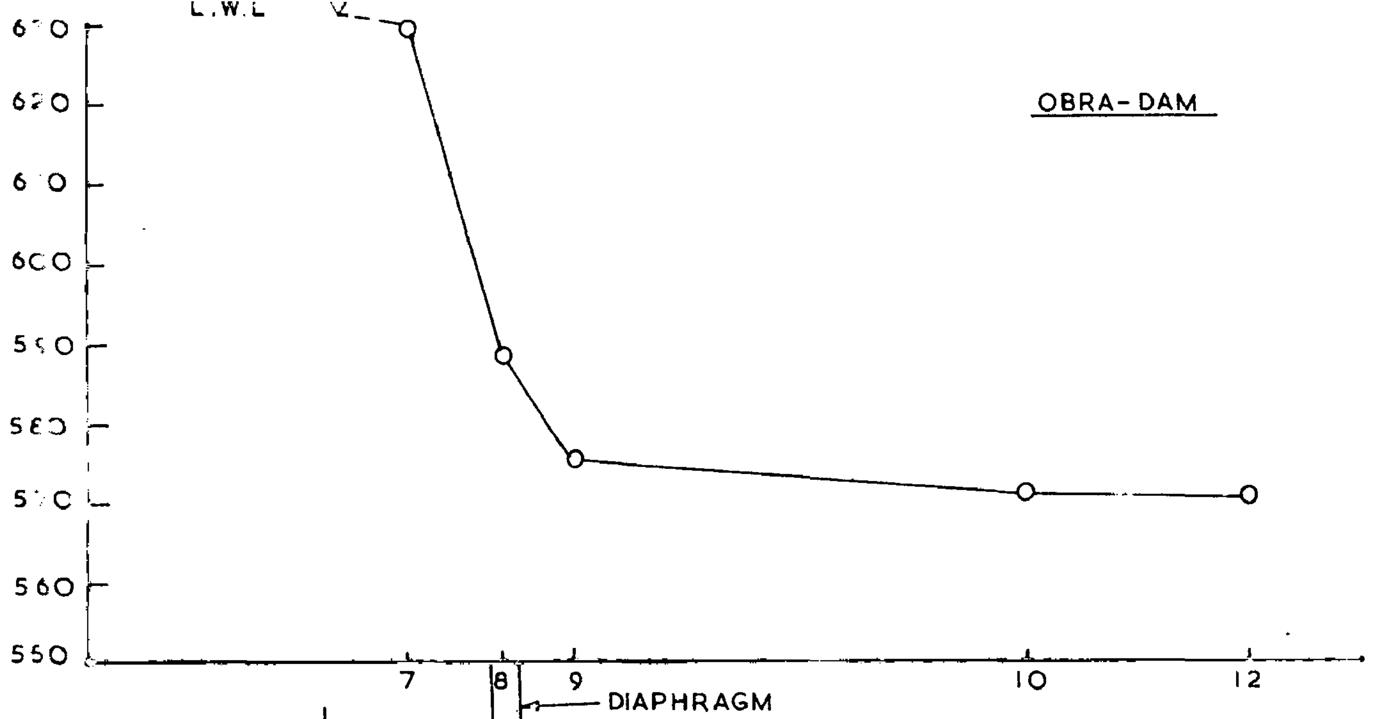

RESERVOIR WATER LEVEL
PORE PRESSURE
PORE PRESSURE NO. 8
PORE PRESSURE

FIG 5.5



LAKE WATER LEVEL & TIPS PRESSUR ON 7.8.71

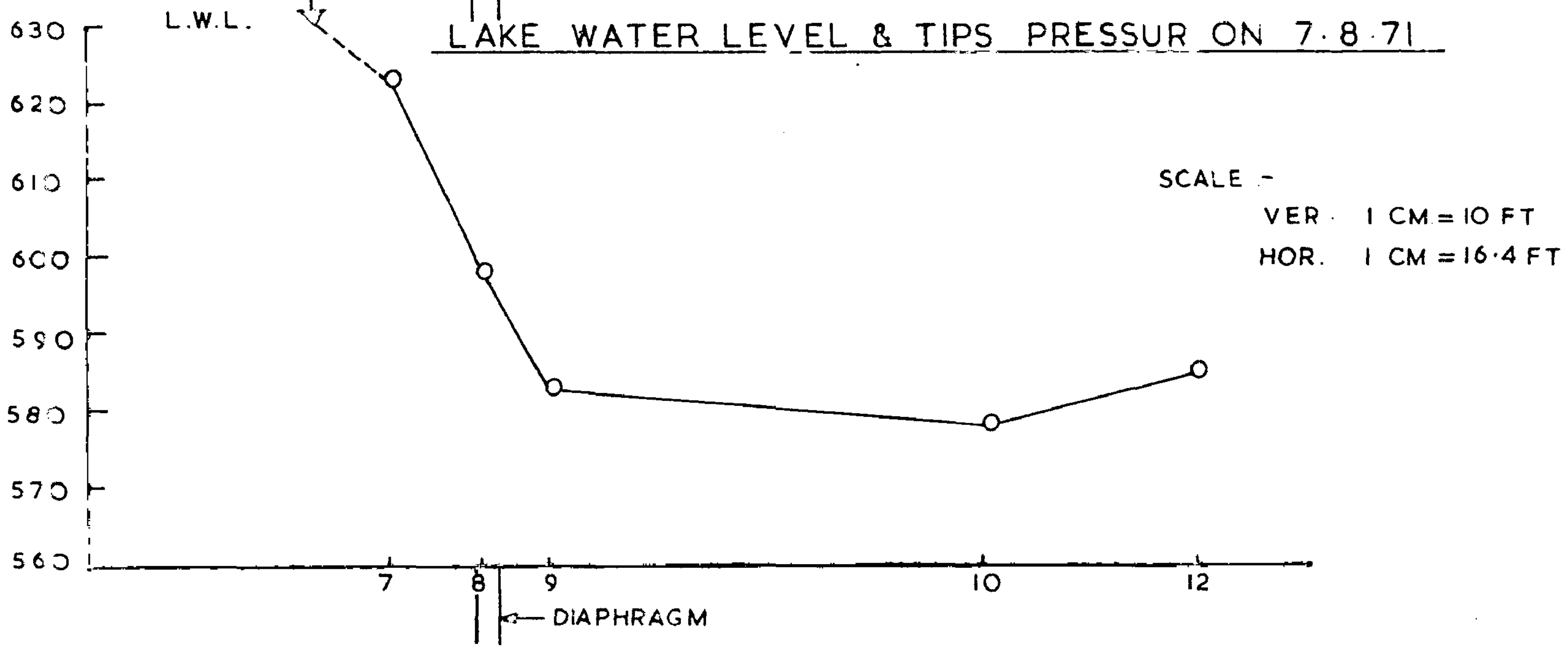
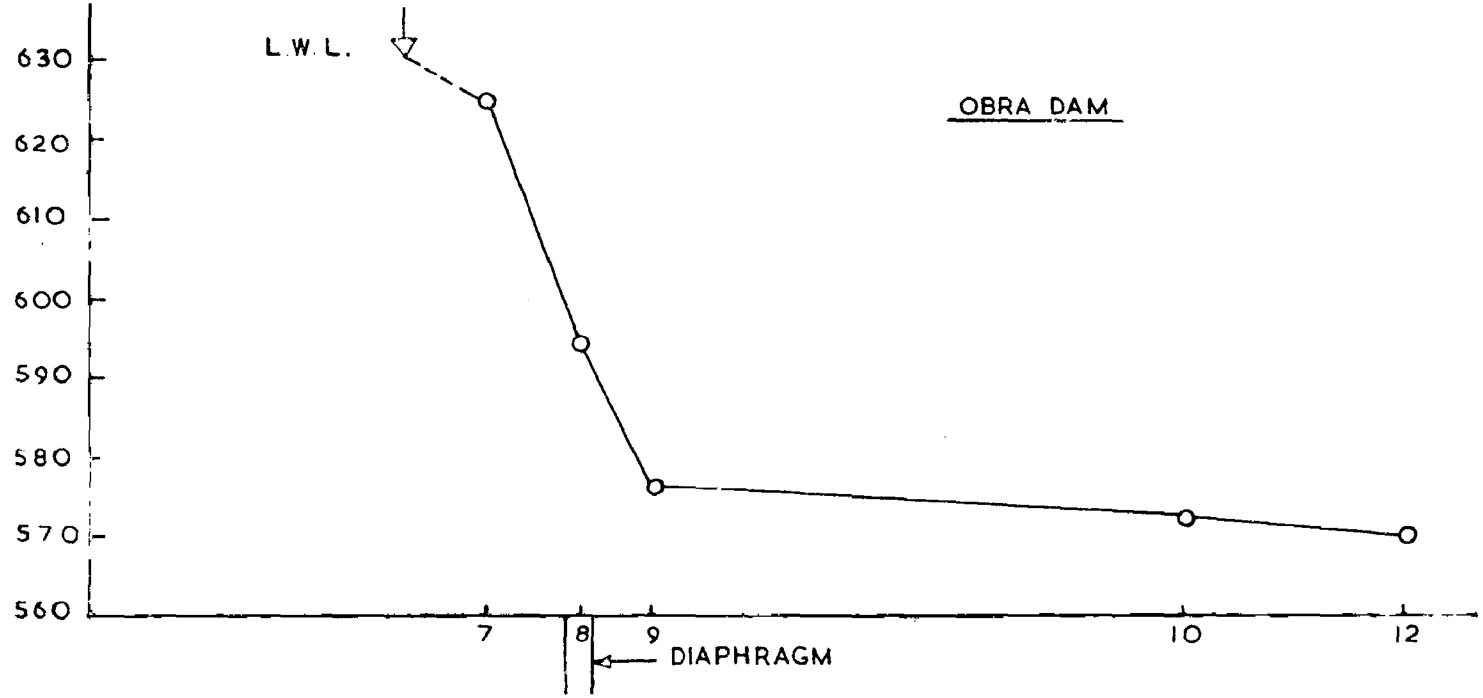
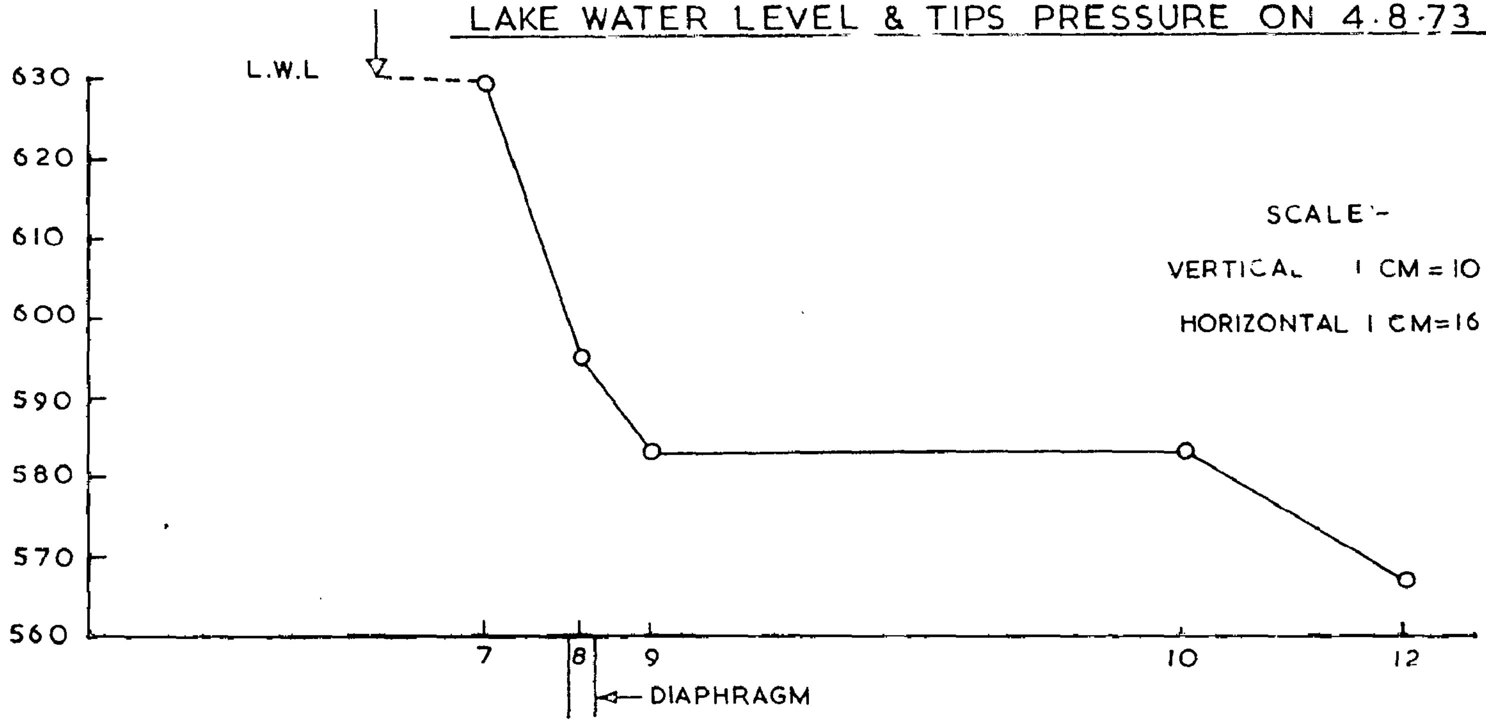


FIG. 6.6(d) LAKE WATER LEVEL & TIPS PRESSUR ON 25.9.70

OBRA DAM



LAKE WATER LEVEL & TIPS PRESSURE ON 4.8.73



SCALE:-
VERTICAL 1 CM = 10 FT.
HORIZONTAL 1 CM = 16.4 FT

FIG. 6.6(b) LAKE WATER LEVEL & TIPS PRESSURE ON 15.10.72

Slope Indicator

(Available from Slope-Indicator Co., Seattle, Washington, U.S.A.)

General Description

Slope indicator measures slide movements, displacements within earth dams, bending of sheet pile bulkheads and other types of lateral earth movements. Instrument is waterproof.

The instrument is normally lowered down on extruded aluminium or grooved plastic casing, the grooves controlling the orientation of the instrument in a predetermined direction. Inclination readings are taken at frequent intervals of depth and are subsequently converted into displacements. Consecutive readings at the same depth-intervals at periodic intervals of time are used to determine depth and rate of ground movement.

The special grooved casing, plastic or aluminium, can be installed in 4.5-in. or larger borings, or it may be built up with the embankments of earth dams or even epoxied to the face of sheet piling or beams. The plastic casing is available in 5-ft lengths and the aluminium casing is available in either 5-ft or 10-ft lengths.

Sensitivity = 1/1000, i.e. as little as 3 minutes, can can be detected, i.e. a lateral displacement of 1 in. in 100-ft depth can be noted.

Actually, displacement in thin shear zones of less than 1/16 in. has been detected earlier.

The instrument is so designed that the inclination by the plane defined by the four guide-wheels is directly proportional to the potentiometer dial reading when the circuit is in balance.

DATA

The slope indicator measures the inclination of the casing in an observation well at frequent intervals of depth. One set of readings is taken in one groove and additional set is taken in the groove on the opposite wall of the casing. The instrument is adjusted so that the inclination of the casing from the vertical may be computed by the following equation -

$$\tan i = \frac{D_n - D_s}{2K}$$

where, D_n = dial reading in northerly groove,

D_s = dial reading in southerly groove,

K = 1000 for series 200 instrument

= 2000 for series 200-B instrument

= 3600 for series 100 instrument.

CONCLUSIONS (on the basis of Table 6.3)

(i) At depth 42.5 ft.

$$\tan i = \frac{D_n - D_s}{2K} = \frac{596 - 560}{2 \times 2000} = \frac{36}{4000} = 9/1000 = 0.009$$

Hence $i = 0° 30'$ (This gives initial position of the slope indicator (tube) fixed in upstream concrete diaphragm.

(ii) At other depths, inclincation of tube can be similarly calculated.

(iii) Marked increase in worked out difference at Sl. 7, 13, 19, 22 indicate joints, where tube pieces have been coupled together, not being in one plumb, apparently.

Table 6.3 - Slope Indicator Readings (Tenughat Dam Project)

The Slope Indicator Co. Field Sheet
Slope Indicator Data
Tenughat Dam Project (Bihar)
Left Bank Ch. 158+96.5 Panel L-7
Date 21st July 1967. INSTR-200-B (Hence, value of K=2000)

Observation Well No. 1
Top El. of observation casing 743.34
Sheet 1 of First Reading.

S.No.	Depth ft	Diff.	A_z D+	A_z S-		Diff.	Change	Diff.	A_z D+	A_z D-		Diff.	Change
1	2	3	4	5	6	7	8	9	10	11	12	13	14
1	42.5		596	560	156	+36			606	551	157	+55	
2	41.0		599	556	155	+43			609	549	158	+60	
3	39.5		603	554	157	+49			609	546	155	+63	
4	37.0		606	551	157	+55			620	533	153	+87	
5	35.5		605	552	157	+53			622	534	156	+88	
6	34.0		603	552	157	+51			621	534	155	+87	
7	31.5		608	547	155	+61			609	547	156	+62	
8	30.0		608	548	156	+60			605	549	154	+56	
9	28.5		608	546	154	+62			603	561	154	+52	
10	26.0		615	540	155	+75			585	570	155	+15	
11	24.5		614	541	155	+73			585	570	155	+15	
12	23.0		613	542	156	+71			587	569	156	+18	

(continued)

1	2	3	4	5	6	7	8	9	10	11	12	13	14
13	20.5		630	526		+104			556	600		-44	
					156						154		
14	19.0		632	526		+106			553	601		-48	
					158						154		
15	17.5		629	527		+102			553	604		-51	
					156						157		
16	15.0		604	531		+ 53			566	592		-26	
					155						158		
17	13.5		603	552		+ 51			564	593		-29	
					155						157		
18	12.0		602	554		+ 48			563	593		-30	
					156						156		
19	9.5		580	576		+ 04			574	582		-08	
					156						155		
20	8.0		581	574		+ 07			574	581		-07	
					155						155		
21	6.5		581	574		+ 07			576	581		-05	
					155						157		
22	4.0		563	593		- 30			593	561		+32	
					155						154		
23	2.5		564	593		- 29			595	560		+35	
					157						155		
24	1.25		565	593		-28			599	558		+41	

<u>Note</u> - Dial readings indicated undercolumn 'N' have been taken when fixed wheels of the slope indicator were pointing towards north.

Chapter 7

C O N C L U S I O N S

On the basis of the study of the foregoing chapters, following conclusions are derived -

(i) The concrete diaphragm wall provided in Tenughat Dam as permanent cutoff is effective and working satisfactorily.

(ii) The upstream concrete diaphragm wall is not as effective as the downstream concrete diaphragm wall. This might be due to leakage through the joints between bed rock and the diaphragm wall or between panel joints.

(iii) A double-row of diaphragm with cross walls and sand in-between grouted, is structurally sound and more dependable than single diaphragm.

(iv) The floods may be allowed to pass over built-in diaphragm wall with suitable protective measures leading to possible reduction in the construction period - by one year, as had been achieved in the Tenughat Dam Project.

(v) The cost of the completed diaphragm walls including laying and removal of protections, dewatering, grouting of sand in-between the walls and grout curtain below the upstream wall works out to be Rs. 775 [33] per sq. metre in the period which is about 50% lower than the conventional open trench cutoff, in the same period.

Recommendations

Design of dams on loose pervious stratum presents many problems. In case when adequate equipment to dredge out the loose materials is available, the impervious zone can be taken down to bed rock. However, when the cost becomes prohibitive for provision of an open trench, alternative sites are searched. This may be the reason why so few dams have been constructed so far on not-too densesand. In such cases, the concrete diaphragm walls may prove to be economical by the methods described in Chapter 4. These methods are patent and there is scope of developing new indigenous technique which would further reduce the diaphragm costs. One such technique by jetting has been developed at the Central Building Research Institute, Roorkee, which is suitable for sand strata and for moderate depths of upto about 10 - 12 m. The technique has been successfully adopted in constructing three syphons in Sarda Sahayak Pariyojana in Uttar Pradesh and work on two more syphons is in progress on the same project.

In developed countries where most of the sites with rock foundation have already been utilised, sites with pervious foundation will have to be increasingly taken up. Therefore, it is recommended to adopt this technique as cutoff. The recommendations are -

1) If a diaphragm cutoff is adopted, a double row of diaphragm walls should be adopted as it will be more reliable.
2) Adequate instrumentation should be provided to havesufficient performance data which will raise the confidence for the future projects.
3) The performance at Tenughat, Obra and during execution at Ukai [38, 39] show the watertightness of the diaphragms. The measurements also indicate no loss of effectiveness with time. Thus, diaphragm should be used as permanent cutoffs,

and in the light of present experience the decision to use them as temporary cutoffs only for excavation of open trench seems over-conservative.

4) The construction of the diaphragms has not presented any serious problems at any Indian site. This is in contrast to grouting of alluvial deposits, which proved costly as well as troublesome at Girna Dam in Maharashtra [40].

5) To compare cost and performance slurry trenching with fill-back of soil material should be taken up on a suitable project.

Appendix - 1

Extract Report on collection of data of past earthquake occurrences in the area around Tenughat Dam site, District Hazaribagh (Bihar), and the seismicity of the region.

by

Dr. Jai Krishna & Dr. L. S. Srivastava

3.00. Data from past earthquakes in the various rift zones of the peninsular shield has shown that the maximum M.M. Intensity has varied from VII to VIII. Thus an M.M. Intensity of VII could be anticipated to occur in the region, and a higher M.M. Intensity VIII could be expected in immediate vicinity of the active faults or shear zone (within a distance of about 10 km on either side of the faults assuming the depth of focus at about 20 km). Thus in order to take adequate safeguards in the design and construction of important structures, it is recommended that a dynamic analysis should be carried out to check if the designs of structures prepared on the basis of the Code would withstand the strong ground motion of the type of the Koyna earthquake, suitable scaled down for various M.M. intensities.

3.01. As Tenughat Dam site lies very near to the bonding faults of the Gondwana basin of the Damodar rift, it is recommended that dynamic analysis be carried out tocheck if the Tenughat dam (Earth dam as well as the concrete spillway section) will withstand the strong motion as recorded during December 11, 1967 Koyna earthquake.

4.00. As the geology and tectonics of the Chotanagpur Plateau has not been studied in great detail, it is not possible to delineate the various active faults and shear zones. In order to have a better knowledge of the earthquake occurrence in the region, it is recommended that the Geological Survey of India be asked to prepare a tectnoic map of the Chotanagpur Platou on a scale of 1:233,400 (1 inch=4 miles). It would also be useful for future development of the valleys in this plateau that a net-work of seismological observatories be established with sensitive seismographs and accelerograph to demarcate the active belts and assess their seismic potentialities.

4.01. It is recommended that in this network of seismological observatories for the Damodar Valley in Chotanagpur Plateau, the main observatories be established at Tenughat Dam site and subsi-diary observatories be established at Plamau, Lohardaga, Ranchi, Hazaribagh, Giridih, Jamshedpur, Kodarma, Gaya and Monghyr and instruments be installed as per Recommendation for Seismic Instru-mentations for River Valley Projects (IS4967-1968) of the Indian Standards Institution. Studies of micro tremors at various sites along major faults and shear zone using sensitive seismographs are also recommended.

4.02. Data obtained from the above studies and installation of observatories, will help in a better assessment of the seismicity of the areaaround the whole Chotanagpur Plateau and lead to more efficient designs of structures to resist future earthquakes in the region.

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