Foundation Problems at Obra Dam

by

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Introduction

OBRA Dam is located across River Rihand about 33 km downstream of the Rihand Dam in District Mirzapur of U.P., India. The project comprises construction of 480 m long earth and rockfill dam with a height of 29·25 m in the deepest portion, a 218·76 m long overflow concrete flank spillway designed for a maximum flood of 13,880 cu m/sec; an integrated intake power house structure located adjacent to the spillway, and 1405 m length of earthen dykes on both flanks. The structure is basically a pick up dam for levelling of tail race fluctuations from Rihand Power House. The Obra plant will have an installed capacity of 99 MW with 3 numbers Kaplan turbine of 33 MW each.

Geology

The rocks at Obra Dam site belong to the Vindhyan and pre-Vindhyans formation comprising an extremely variable series of papery micaschists, phyllitic slates and shales, banded quartzites and siliceous limestones. In the vicinity of the dam site, the Vindhyans are represented only by Semri Series, consisting mainly of the basal sandstones, limestones and conglomerates. The limestones are homogeneous in nature and fine grained in texture, very hard and compact. The dips of bed rock at dam alignment are about 10^0 to 15^0 in an upstream direction. Numerous joint planes present in the rock detract the advantage inherent in an upstream dip of the bedding planes. The joints are open in many cases being susceptible to formation of solution cavities. It is apprehended from the nature of foundation that the problem of leakage through the foundation of the dam will be of serious proportions.

The dam has been sited in the downstream portion of the gorge *vide* layout plan Figure 1. The foundation strata at this point comprise alternate bands of carbonaceous shale (CS) and limestone (L) *vide* Figure 2. According to geological mapping there are two faults in the bed rock at distances of 145 to 215 m from the retaining wall as shown in drawing. The dam section at the deepest point, as adopted on

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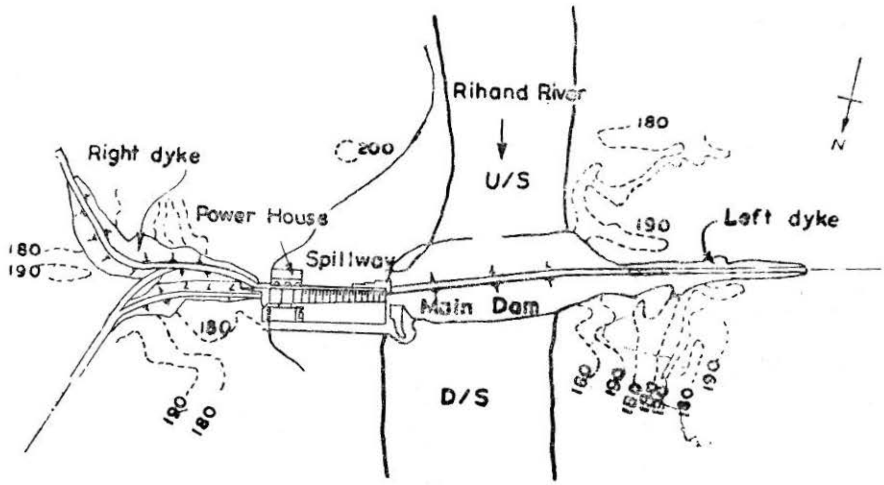


FIGURE 1 : General layout plan.

the basis embankment material properties and settlement considerations, is shown in Figure 3.

Investigations

Detailed sub-soil investigations indicated that Obra Dam foundations comprise 25 to 30 m thick overburden of sand overlying interbeddings of limestones and shales. In about 50 m length on the right flank adjacent to spillway abutment, however, the foundation sand overlies an intrusion of compressible clayey material. Foundation sand is poorly graded *vide* Figure 4 with mean size varying from 0.5 mm and 0.6 mm and uniformity coefficient from 2·5 to 8·0 (average 5·45) (1966). Dynamic and static cone penetration tests on foundation material indicated relative density varying from 30 to 70 percent. Permeability tests carried out in the river bed by pumpout method yielded a permeability value ranging from 14,500 to 29,000 $\times$ 10^{-6} cm/sec. The loose and pervious nature of foundation deposits presented the following problems :

(*i*) Possibility of liquefaction of foundation sand under seismic conditions.

(*ii*) Possibility of excessive settlement of foundation sand under seismic condition.

(*iii*) Provision against likelihood of excessive differential settlement in the portion of dam underlain by compressive soil.

(*iv*) Provision for controlling excessive seepage through the foundation and guarding against its detrimental effects.

These problems are discussed in detail in the following paragraphs.

(A) LIQUEFACTION AND SETTLEMENT OF FOUNDATION SAND

According to IS:1893-1962, the site of Obra Dam falls in Zone III, wherein a coefficient of 10 percent of gravity has been suggested for earth dams in this zone. This value was, therefore, adopted for test

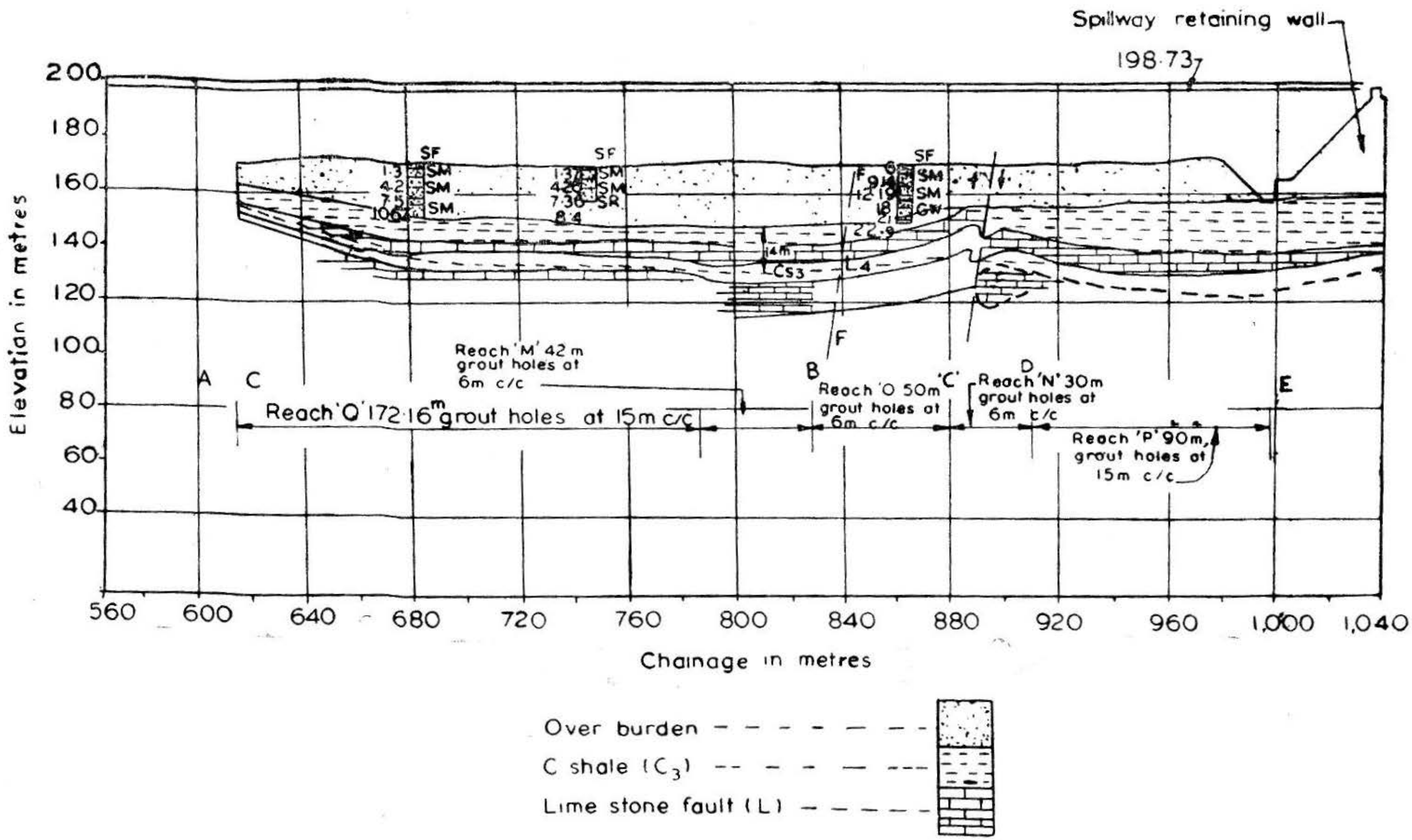
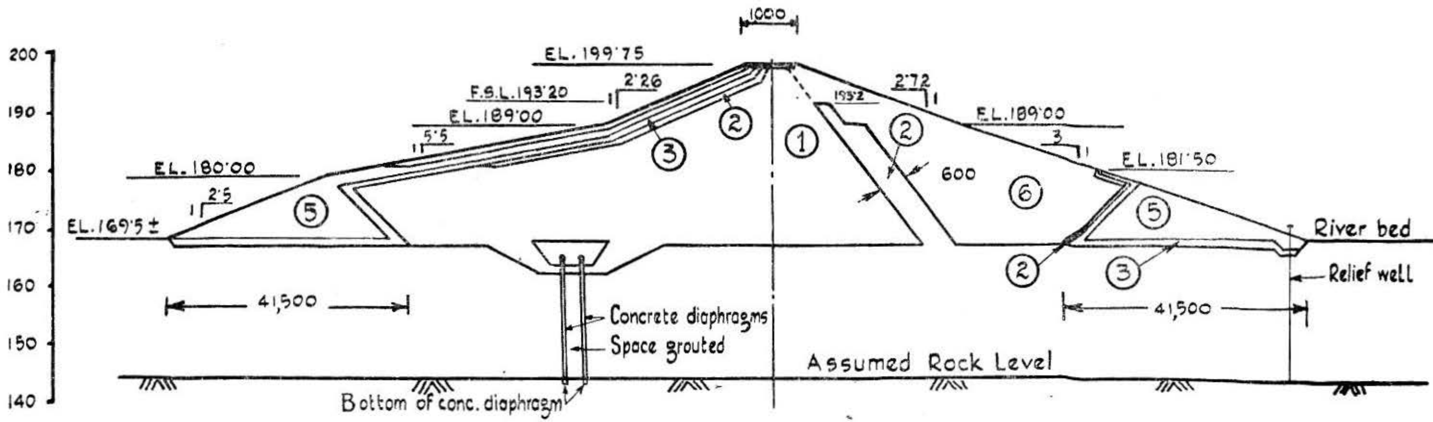


FIGURE 2 : L-Section along centre line of Obra Dam showing strata.

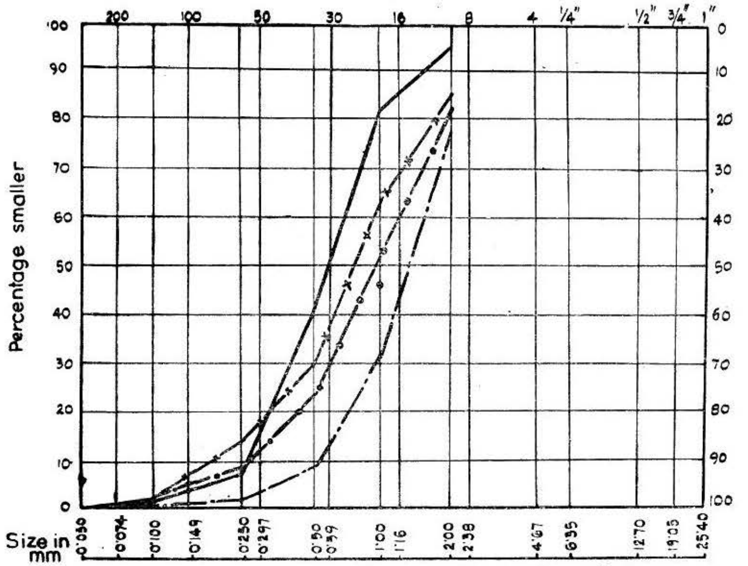


TYPICAL X-SECT'ON
Embankment zone

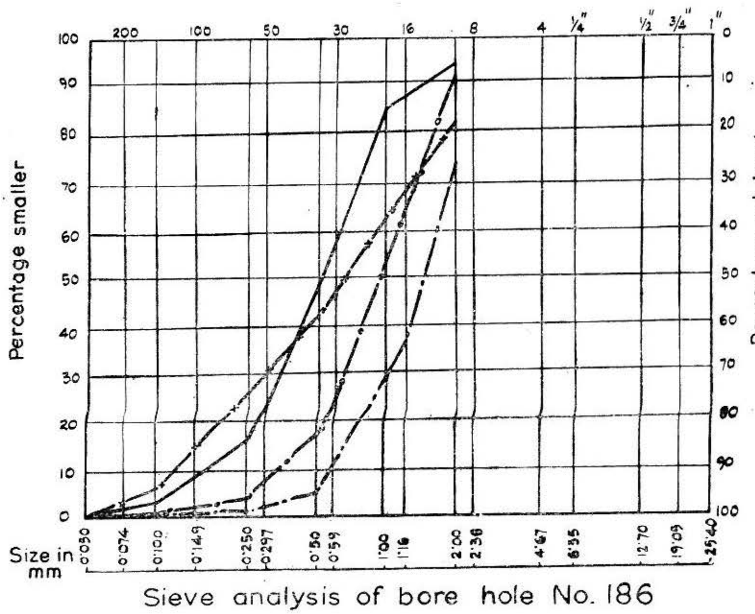
(LEVELS AND DIMENSIONS ARE IN METRIC UNITS)

1. Impervious Clay
2. Coarse River Sand (-) 6·3 mm
3. Crushed Rock 6·3 mm to 7·62 cm
4. Selected Rock 7·62 cm to 25·4 cm
5. Rock-fill
6. Random (Impervious, Semi-pervious)

FIGURE 3.



Sieve analysis of bore hole No. 184



Sieve analysis of bore hole No. 186

FIGURE 4 : Gradation curves.

studies and design. The nearest epicentre of past earthquake (Nepal-Bihar 1934) is at about 300 km from site and it is expected that only long period wave (1 second) will appear in the area. Period T, for Obra Dam has been calculated to be 0·71 second by formula :

$$T = 4{\cdot}30\, h \sqrt{\frac{p}{E}}$$

Where h = height of dam = 30 m approximate,
p = mass density = $1{\cdot}92$ gm/cm^3, and
E = modulus of elasticity of section material
= $6{\cdot}75 \times 10^6$ kg/m^2.

Assuming 20 percent damping of soils, the acceleration response from standard spectra is 0·066 g. Assuming a multiplying factor of 1·5 for standard spectrum, the acceleration for testing foundation for liquefaction has been taken as 0·1 g, *i.e.*, 10 percent g. Testing has been done for 5 percent and 20 percent g also to account for the extreme conditions.

The range of grain-size, relative density, uniformity coefficient of foundation-sand lay in critical range of liquefaction under seismic loading. Both *in situ* and laboratory tests were, therefore, done to investigate the likelihood of liquefaction and assess foundation settlement.

Field Tests

In situ experiments comprising blasting tests (1966) at the dam site, *viz.*, 1 to 3 kg of 50 percent gelatine charges at 6 m depth and recording observation by blasting at distances varying from 10 to 50 m from the observation point were carried out. Two Miller type acceleration pick-ups were embedded at the observation points for recording horizontal and vertical accelerations.

The values of maximum observed acceleration have been plotted in Figure 5. The curve approximately fits the observed points by a relationship

$$\alpha = C \frac{Q^{0\cdot97}}{d^{1\cdot25}}$$

where α = Acceleration in g.
Q = amount of charge in kg.
d = distance of charge point from observation point in metres, and
C = constant being equal to 27 and 75 for horizontal and vertical accelerations respectively.

Pore pressures and settlement observations were also taken for 2 kg blasts. The changes in pore pressures as percentage of initial inter-granular stress are given in Table I (for 2 kg blast).

TABLE I

Distance from centre of blast	2·5 m		5 m		10 m		15 m		30 m	
Depth of Piezo-meters (m)	In cm	In % of effective-ness	In cm	In % of effective-ness	In cm	In % of effective-ness	In cm	In % of effective-ness	In cm	In % of effective-ness
2·5	170	48	132	37	69	19	49	14	21	6·0
5·5	181	29	130	20·5	55	8·7	18	2·8	0	0
8·5	140	15·2	80	8·8	10	1·1	0	0	0	0

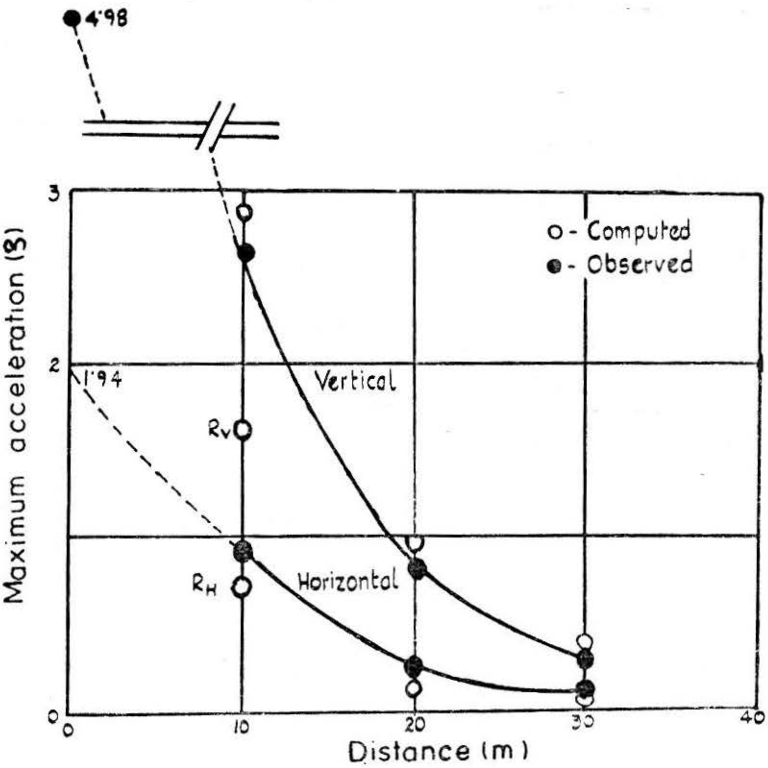


FIGURE 5: **Acceleration versus distance 2 kg blast**

Field tests indicated that the maximum observed pore pressure at a depth of 2·5 m and at a distance of 2·5 m from the blast point is 48 percent of initial intergranular stress and increases to about 60 percent when projected to the blast point. For complete liquefaction to occur, the increase in pore pressure should be equal to the intergranular stress. The likelihood of liquefaction was, therefore, not indicated.

The settlement of blast point of 2 kg for single impulsive load for 6 m depth was 17·5 cm (*vide* Figure 6). Assuming that depth of soil participating in settlement is (6 m+2 m), the percentage settlement $= \frac{17{\cdot}5 \times 100}{800} = 2{\cdot}2$ percent.

Laboratory Studies

In order to study the effect of number of cycles under different acceleration values, percentage settlement was obtained from laboratory studies (1965) on vibration table for cycles ranging from 100 to 2000 for values of α/g ranging up to 20 percent (*vide* Figure 7). The curve indicates that the following combination of acceleration and number of cycles gives 2·2 percent settlement:

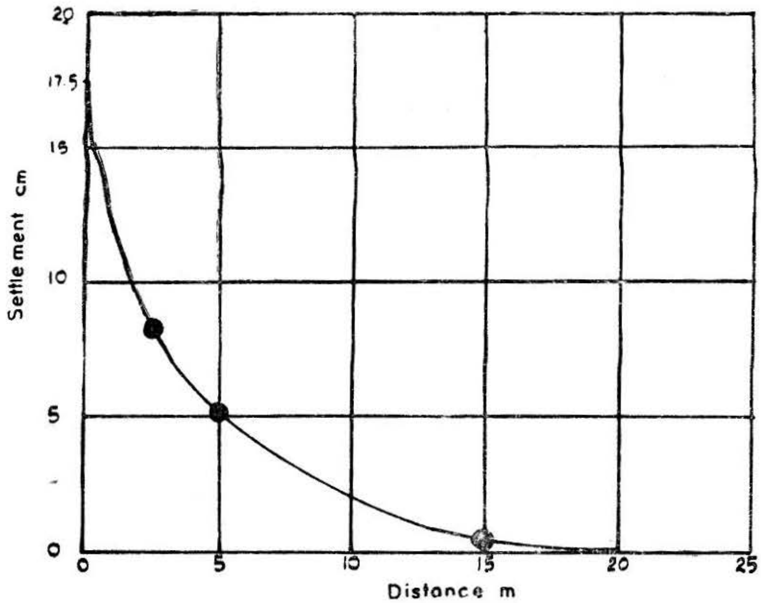


FIGURE 6 : Settlement versus distance 2 kg blast.

$\frac{\alpha}{g}$	*No. of cycles*
1·5%	100
0·75%	200
0·50%	500

Blasting test for 2 kg blast gives acceleration α horizontal$=1{\cdot}94\ g$ and vertical$=4{\cdot}98\ g$. Assuming vertical acceleration to be half effective as horizontal for causing settlement, the combined effect of the two works out to $4{\cdot}43\ g$.

The value of α/g and number of cycles given above have been plotted in Figure 8 alongwith field value of $\alpha/g=4{\cdot}43$, corresponding to one cycle.

The design acceleration for dam is 10 percent. The maximum acceleration at such a strata during an earthquake may reach 20 percent in extreme case. The number of cycles of higher acceleration are likely to be very small. Also the number of cycles of acceleration smaller than design acceleration is usually large. Based on this concept, the number of cycles to which the dam will be subjected in its life time were computed from the relationship given in Figure 8 and worked out in Table II.

It is observed from the curve No. 7 that 500 cycles of 5 percent yield a settlement of 5 percent for an initial relative density of 30 percent which was the lowest figure obtained in the field.

In order to compute settlement of foundation, percentage settlement of saturated sand was obtained from various values of initial relative density for acceleration of 5, 11 and 20 percent g subjected to different numbers of vibration cycles. The depth of deposit was kept 11-12 cm in each case in the model and settlement was measured by syphoning out water expelled from the saturated pores as a result of vibration. The relative density of the sand deposit was kept 30 percent.

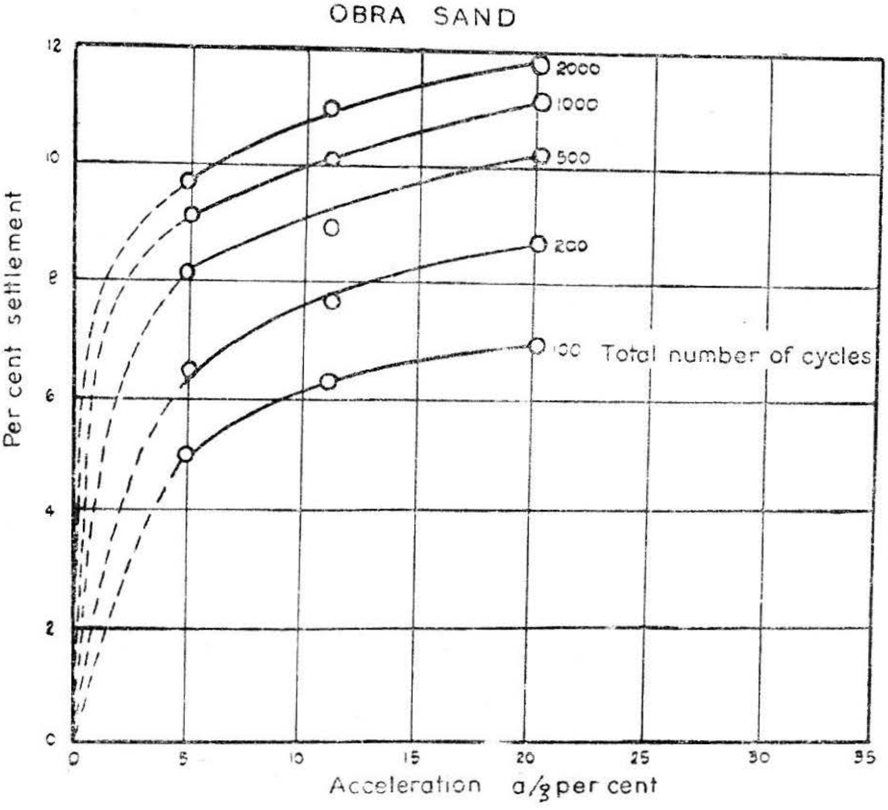


FIGURE 7: Acceleration versus settlement.

TABLE II

No. of earthquake	Maximum ground acceleration to which the dam may be subjected	Total No. of cycles of 5% equivalent horizontal acceleration as seen from Figure 8
1	20% *g*, 5 cycles+ 15% *g*, 10 cycles+ 10% *g*, 20 cycles+ 5% *g*, 50 cycles	(1+16+24+36+50)=127
2	15% *g*, 10 cycles+ 10% *g*, 20 cycles+ 5% *g*, 30 cycles	2(24+36+30)=180
3	10% *g*, 10 cycles+ 5% *g*, 20 cycles	3(18+20)=114
10	5% *g*, 10 cycles	10×10=100

Total No. of cycles of 5% *g*=520=say 500 cycles of 5% *g*

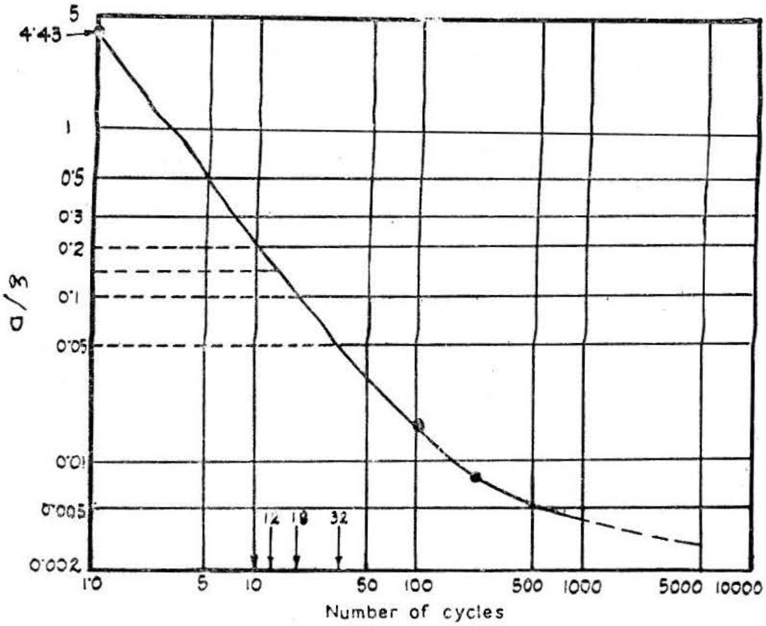


FIGURE 8 : **L/g versus number of cycles for 2·2 percent settlement.**

TABLE III

Initial relative density (percent)	Settlement as percent of thickness of deposit fcr α/g = 20·2 percent
30	15·0
40	13·0
50	9·0
60	6·5
70	4·5
80	2·25

The deposit was vibrated for a total time of 5 minutes, till the settlement was complete which imparted about 4·200 cycles of vibration (*vide* Figure 9). The test results giving total percent settlement for varying initial density are plotted in Figure 10. Typical values of percent settlement for a few initial relative density figures are given in Table III.

Tests on actual strata indicated that a relative density of 30, 50 and 70 percent were obtained at strata of 3·6 m, 12 m and 8·4 m thickness respectively. From the relationship between settlement and initial relative density given in Figure 10, it is indicated that :

(*i*) Maximum settlement of deposit with 50 percent initial relative density is about 2/3rd the maximum settlement with 30 percent initial relative density.

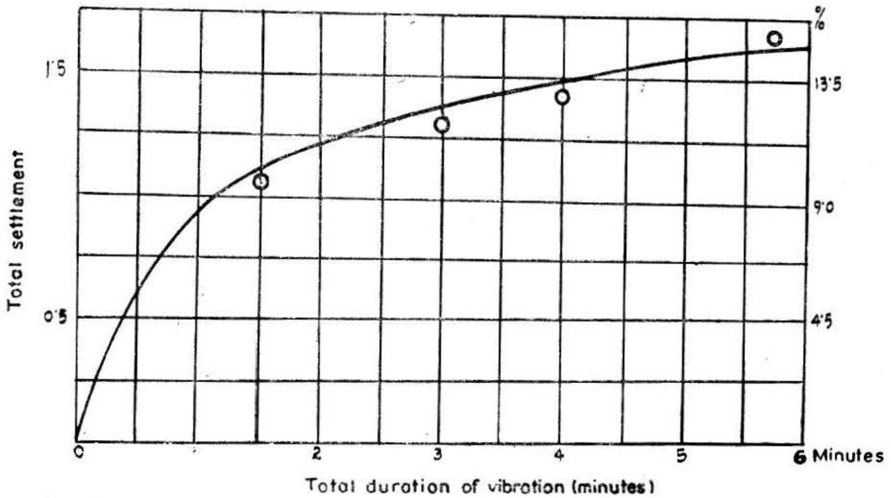


FIGURE 9 : Total settlement versus duration of vibrations.

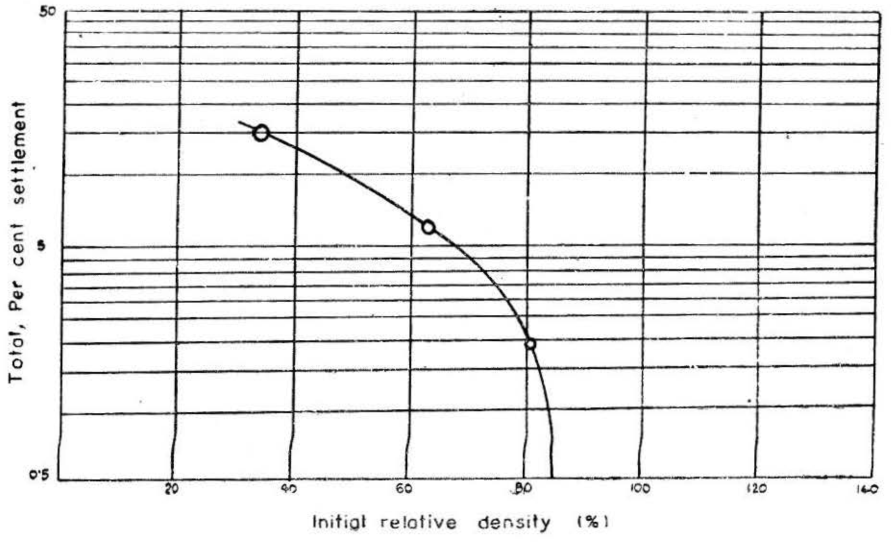


FIGURE 10 : Total percent settlement versus initial relative density of deposit.

(*ii*) Maximum settlement of deposit with 70 percent initial relative density is about 1/4th the maximum settlement with 30 percent initial relative density.

Total settlement of foundation deposit with 50 percent and 70 percent initial relative density with 5 percent acceleration and 500 cycles works out = 8 percent × 2/3 = 5 33 percent and = 8 percent × 1/4 = 2 percent respectively. Applying these percentages to different foundation strata, the total foundation settlement is obtained as 1·11 m. Extra allowance has been made in the free-board for this value of anticipated settlement.

Liquefaction

The laboratory tests described above were also utilised for investigating the possibility of liquefaction of the foundation mass when subjected to sinusoidal vibration. This was done by placing iron pieces on the surface of deposit of saturated sand contained in the tank resting on vibrating table. It was observed from the tests that vibration of the sand at various fre uencies and acceleration did not cause appreciable sinking of weights. It was, thus, indicated that the deposit did not liquefy.

Experiments were also carried out with impervious loads of fill section on saturated foundation deposit with and without clay cut-off extending to impervious boundary and under full reservoir conditions. The model was subjected to an acceleration of 10 percent g. No tendency to liquefaction was indicated and the results were thus, in conformity with the field observations.

The model studies, however, indicated development of zones of high pore pressure below the toe of the dam extending from 10 m height of dam to some distance beyond the toes of the dam, thus, indicating the necessity of strengthening the toes of the dam.

In order to safeguard against any adverse effects of excessive settlement of foundation, 5 m and 6 m thick graded filters have been provided on the upstream and downstream contact faces of impervious core respectively with a view to seal up any differential crack. In addition 1 m thick filter was provided in a length of 41·5 m below the upstream and downstream toes to prevent dislodgement of foundation soil particles arising from excessive pore pressure generated from foundation vibration.

Provision against Differential Settlement

The clayey overburden within the dam foundation on right flank covers a length of about 50 m along the dam axis, and tapers towards the downstream end with a thickness of about 5 m at the upstream end increasing to about 15 m at the downstream end. Laboratory tests conducted on representative samples from the material indicated ac and ϕ value of 0·33 kg/cm^3 and 18° respectively. Consolidation tests on samples indicated highly compressible nature of clay and the settlement computation indicated a value of about 1·4 m for settlement under the weight of the dam. The likely settlement on adjacent sandy foundation was worked out as 0·1 m and this indicated likelihood of excessive differential settlement resulting in transverse cracks in the dam.

The following proposals were considered to eliminate or provide against the adverse effects of cracking :

(*i*) Complete removal of compressible material in the entire width of the dam and extending dam foundation up to bed rock.

(*ii*) Partial removal of compressible material to bed in the width of central clay core.

(*iii*) Partial removal of compressible material in the entire length of 15 m adjacent to the spillway abutment and bonding the dam foundation to rock in the entire width of the section and negotiating the difference in the foundation levels on the rock and river bed by a slope of 2 : 1 through the deposit of compressible material.

The first proposal was very costly and the second of taking the central core to the rock level did not prove satisfactory since the side slopes would be supported on compressible soil and undergo excessive settlement leading to longitudinal cracking. The third was, therefore, finally adopted. The possibility of transverse cracking due to differential settlement could not be ruled out and was provided for by laying a 2·5 m thick filter layer of extra fine sand interposed between coarse sand layer on the upstream face. In addition to liberal provision of filter as above, the plastic behaviour of the fill was ensured by placing clayey soil of relatively higher plasticity (LL=22−34 & PI=12−16) and compacting at about 2 percent higher on the wet side of standard optimum moisture content. With these provisions, it was felt that differential cracking would initially tend to heal up due to built in flexibility within the embankment fill and would tend to get sealed by inflow of fine sand from the filter layer in the event of actual cracking, and would thus prevent any possibility of damage. The details are shown in Figure 11.

(B) SEEPAGE THROUGH FOUNDATION AND ITS TREATMENT

Seepage computations through pervious foundation material indicated seepage losses of about 0·566 cu m/sec (20 cusec) under maximum head excluding losses through the solution cavities in the foundation rock which would further increase this figure. In view of high order of seepage losses and their attendant adverse effects, it was decided to provide a complete cut-off in the foundation.

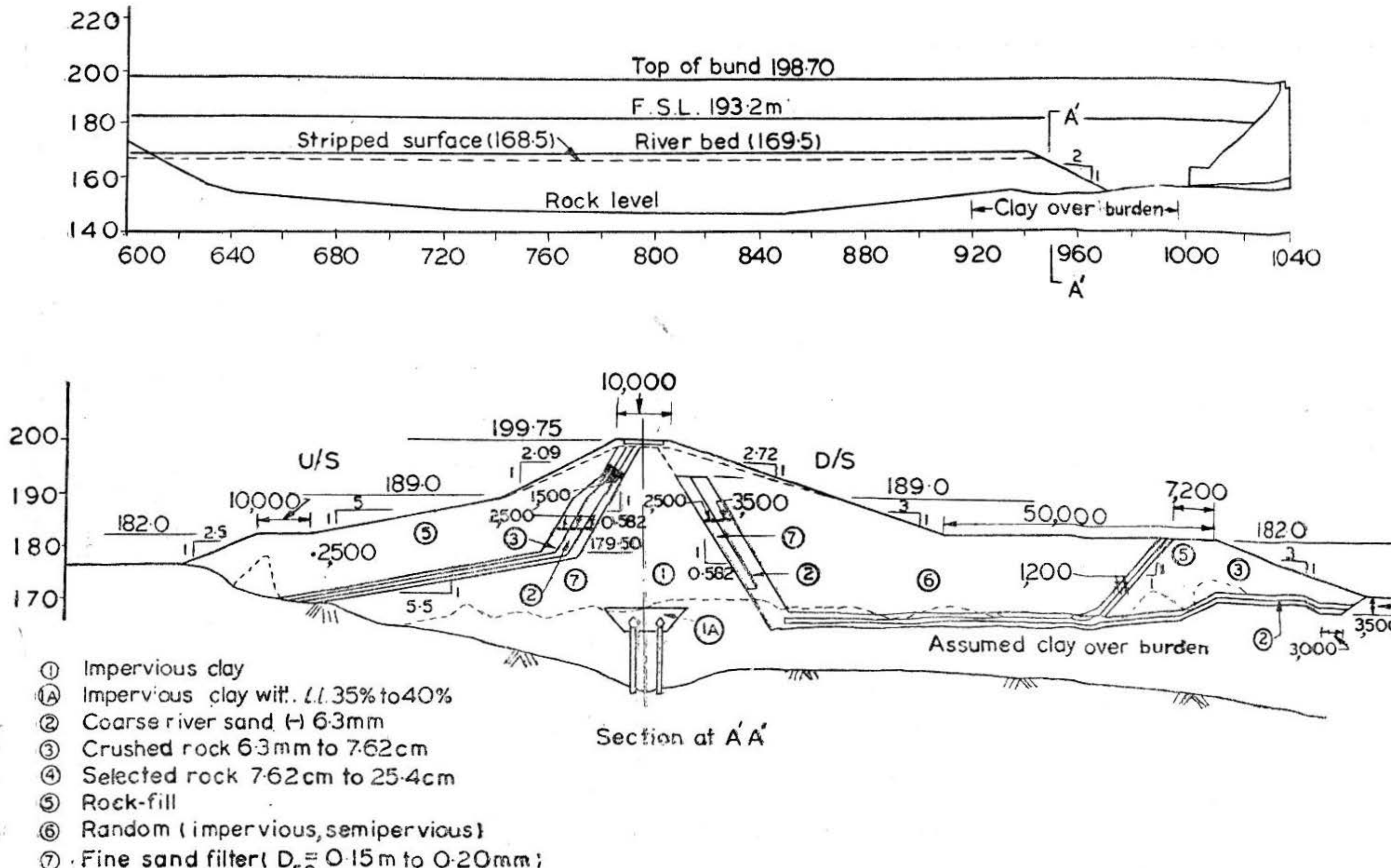
The following alternative proposals were considered for providing complete cut-off.

(1) *Clay Backfilled Trench with Chemical Grout*

A 9 m wide clay backfilled cut-off trench extending 9 m into the foundation with a 7·6 m wide curtain of clay chemical grout extending from bottom of trench to 3 m depth into the clayshale, and alternatively a mud wall type cut-off with gravel backfill were considered. These were more considered acceptable in view of difficult, time consuming and expensive dewatering operations. They also suffered from tight schedule of completing entire work in one working season.

(2) *Concrete Diaphragm*

In order to overcome the above difficulty, it was finally decided to provide two rows of plastic concrete diaphragm cut-off with intervening soil mass grouted. Alternatively two rows of steel sheet piles spaced 3 m apart were also considered but dropped due to wandering tendency and likely damage by break of the interlocks or tearing of steel. Doubts also exist as to the effectiveness of long piling because of the difficulty of keeping the pile straight and because long piles curl at the bottom if they, meet anything hard and thus they cannot be driven watertight. Besides the depth of cut-off was about twice the length in which the piles are manufactured and would necessitate end to end welding thereby making construction costlier. The proposal was not found acceptable due to very high cost of construction coupled with uncertain reliability under the circumstances.



Top of bund 198·70
F.S.L. 193·2m
Stripped surface (168·5)
River bed (169·5)
Rock level
Clay over burden
A'
220
200
180
160
140
600
640
680
720
760
800
840
880
920
960
1000
1040
10,000
199·75
U/S
D/S
189·0
189·0
182·0
182·0
7,200
50,000
10,000
2·09
2·72
2·5
5
5·5
3
1500
2500
2500
3500
0·582
179·50
1200
3500
3000
Assumed clay over burden
Section at A'A'
① Impervious clay
(1A) Impervious clay with L.L 35% to 40%
② Coarse river sand (–) 6·3mm
③ Crushed rock 6·3mm to 7·62cm
④ Selected rock 7·62cm to 25·4cm
⑤ Rock-fill
⑥ Random (impervious, semipervious)
⑦ Fine sand filter (D50 = 0·15m to 0·20mm)

FIGURE 11.

The cut-off formed by two concrete diaphragms and intermediate space grouted was found to be most economical, surer, and feasible and was, therefore, finally adopted. It was proposed to provide two diaphragms 0·6 m thick and 3 m apart, extending 2 m deep into foundation rock at the lower end and to grout the intervening space with cement, bentonite and chemical grouts.

Alignment and Design of Cut-off

The cut-off is aligned to act as continuous barrier through the length of dam and following clayshale bend *vide* Figure 12 and avoid cavernous limestone foundation. The dam axis has been kept straight and impervious clay blanket has been extended upstream of the core so as to envelope the grouted cut-off line all along and form a continuous watertight system.

Originally the diaphragms were proposed to be made of plastic concrete of cement-sand-clay mix. But, subsequently, it was felt that it would be difficult to produce any concrete so plastic as to take the deformations of the foundations. Also in order to lay the diaphragm continuous and to have sufficient strength ordinary 1 : 1½ : 3 cc, was adopted for diaphragm, with the following ingrdients:

(*i*)	Ordinary portland cement	1200 kg (24 bags)
(*ii*)	Aggregate 3/4 in. to ½ in.	1·24 cu m
	½ in. to 3/8 in.	1·24 cu m
(*iii*)	Sand	1·42 cu m
(*vi*)	Water-cement ratio	0·5
(*v*)	Slump	15·24 mm (6 in.) 22·86 mm (9 in.).

To make the diaphragm rigid, resistant to cracking (due to differential settlement of foundation during earthquake) reinforcement in steel cages 5-6 m length, 16 mm bars @ 325 mm c/c in both direction and at both faces of diaphragm were provided. Originally the reinforcement was to be welded fully on 3 m apart on both faces and tack welded at intermediate points to form a rigid cage so that adequate bond is developed in the presence of bentonite film around the surface. It was, however, dropped since actual test indicated that bond was being developed between reinforcement and concrete. Thereafter, only sufficient welding of each joint was done to make the frame rigid enough to keep in shape during handling.

Semi-elliptical reinforced diaphragm caps (Figure 11) were provided to effect uniform transference to stress from surrounding soil. A sub-zone of highly plastic clay (LL=35-40 percent) placed at moisture 5 percent above O.M.C. 4 to 7 m deep (about twice the anticipated settlement), 7 m wide at bottom with 1 : 1 side slope has been provided at the top of the caps so that the diaphragm can penetrate into plastic mass in case of excessive foundation settlement and there is no differential settlement of embankment. The diaphragm penetrates 2 m into the cut-off for breaking the seepage path, a minimum of 2 m being provided in view of the difficulty in compacting clay in cut-off trench on the sides of diaphragm which would be done by pneumatic tampers or manually.

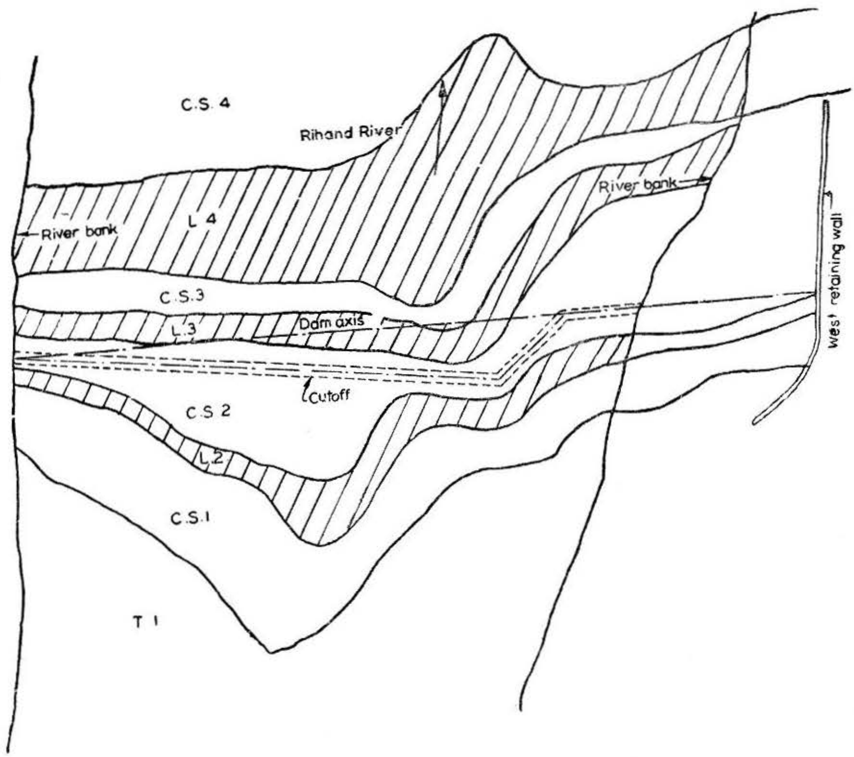


FIGURE 12 : Alignment of cut-off trench.

Diaphragm is keyed 1 m deep into clayshale at the bottom to eliminate leakage at the junction. Any limestone sequence encountered along the cut-off alignment is proposed to be grouted to guard against leakage through joints and solution cavities. The vertical joints between panels of diaphragms have been staggered for above reason and their edges made semi-cylindrical in alternate panels so as to provide a longer seepage path as compared to the straight vertical joint.

Overburden material in the river bed is loose up to a depth of 6 m. For effective economy, 1 m layer of loose material from the entire base of the dam, and 6 m in the cut-off trench portion has been removed to place the dam on dense sand layers.

Construction Techniques

The process consists in loosening the soil by a percussion tool having a diameter corresponding to the width of the trench and connected to a series of rigid tubes. The drilling mud is circulated from bottom to top by means of large suction pump. The percussion tool travels horizontally and vertically both. The trench is kept full of drilling mud which is continuously replenished with new mud to compensate for the soil removed and for any losses. The loosened material from the trench is carried to the surface by reversed circulation flow of the drilling mud. The drilling mud is then led to large tanks where the excavated material settles down. At the surface the soil is separated from the drilling mud discharged in heaps and the drilling mud is replenished with bentonite and returned to trench. The sides of the trench are maintained by the hydrostatic pressure of the drilling mud with which it is kept filled throughout the operations. The excavation proceed over the 5 to 10 m length of trench. Alternate trenches are dug first, leaving a full trench length unexcavated inbetween. When the excavation of alternate trench is complete, a forming tube is placed at each end over the entire depth so that the concrete trimmed into the trench, forms a semicylindrical surface at the contact with the tubes. The tubes are withdrawn when the concrete has taken an initial set leaving the ends of the trench concave towards the adjacent excavation. The entire length of cut-off is divided into primary and secondary panels, the secondary being left unexcavated till primary panels are concreted.

Grouting of Foundation

In view of highly jointed nature of foundation rock and presence of solution cavities, it was decided to grout the foundation rock all along the alignment of the diaphram cut-off to form a deep and permanent impermeable curtain. The entire reach was divided in 5 zones, *viz.*, P,N,O,M,Q (Figure 2). The spacing of holes in fault zones N, O, M, was 6 m, and in zone P, Q as 15 m. The grouting was to be done extending to 15 m depth or 1 m below L_3 band, whichever is greater. In fault zone N, O, M, it was extended below L_4 band. Apart from primary, some secondary and territory holes were also provided in zone N,M,O, where grout intake was excessive. The loss of water (lugeon values) in the various bands were determined in advance to decide mix required and assess the extent of work involved. The range of lugeon values is given in Table IV.

TABLE IV

Rock Band	Range of lugeon values	General indication
CS_2	3-14	Weathering and presence of fissures.
L_3	53-23	Small and large cavities.
CS_3	0-13	Small fissures.
L_4	0-325	Large and small cavities.
CS_4	0-1	Practically impermeable.

On the basis of above assessment it was decided that grouting with neat cement grout mix along may not be practicable as large and medium cavities cannot be grouted with non-viscous grouts. The grouting pattern adopted for cement and stable grout mixes used was :

Mix	*Ratio by weight*
(*i*) Cement/water	1/6, 1/4, 1/2, 1/1, 1·5/1, 2/1
(*ii*) Bentonite/cement	1/10, 2/10, 3/10, 4/10
(*iii*) Cement/(water=3 percent sodium silicate)	1/1

The grouting was done at a constant injection rate. The maximum pressures allowed were as below:

10 kg/sq cm	Up to 5 m below rock level
15 kg/sq cm	From 5 m to 15 m below rock level
20 kg/sq cm	15 m below rock level.

The grout intake pattern has indicated that approximately 500 tonnes of cement in the form of stable grouts had to be injected in one hole near the fault zone. There were eight other holes in which the consumption of cement was between 60 to 100 tonnes. The total rock drilling was 2462 m and total grout intake was 2414 tonnes, giving an average intake of 0·98 T/M. The water loss in test holes made in the various reaches showed the desired permeability of 300 m/year was achieved after grouting.

Grouting of Intervening Sand Between Diaphragms

The 3 m sand between the two diaphragm was grouted with suitable clay and chemical grout so as to make it impervious.

For grouting the sand, two series of holes were made in two rows at a distance of 1 m clear of internal face of diaphragm at 3 m c/c and were staggered. The first series of grout holes called primary at 6 m c/c were grouted by cement bentonite deflocculated grout so as to fill large pores of sand with this mix under specified pressure, whereas the secondary holes were grouted with chemical grout to fill the finer pores. Grouting was done by tube-a-manchette method patented by M/s Solentanche Company.

The cement bentonite mix was used at pressures not exceeding 10/15 kg/cm². Holes in which pressure rose above 15 kg/cm² were grouted with deflocculated bentonite grout. Generally the pressures were kept at the lower limit and were seldom allowed to cross this limit. Generally the grouting was done at pressures between 8 to 10 kg/cm² and only on a few occasions, when the pressure did not build up, regrouting with fresh batch had to be done. The grout mixes used were as below.

CEMENT BENTONITE MIX (PER M³ OF GROUT)

Constituents		*Properties*
Water	862 litres	Viscosity 28-31 seconds (Marsh cone value).
Cement	300 kg	Setting time 4 to 6 hours.
Bentonite	100 kg	

The order of mixing the different elements was : water, bentonite and cement.

Chemical grout mix or deflocculated bentonite grout mix had the following composition and properties.

DEFLOCCULATED BENTONITE MIX (PER M³ OF GROUT)

For secondary holes :—

Constituents	*Properties*	
Water 910 litres	Viscosity 36 sec (Marsh cone value)	
Bentonite 105 kg	Setting time 45 mts—1 hour	
Monosodium Phosphate 40 kg	Specific gravity	1·11
Sodium Silicate 35 litres	Needle penetration	135 gm
	Resistance Cohesion	10 gm/cm^2 —

The order of mixing was water plus bentonite, mix for two minutes, add phosphate and mix for two minutes, and add sodium silicate just before transferring to agitator tank.

The composition of the sheath grout used to form watertight seal around the sleeved tube in the grouting process comprised a mix of cement and bentonite in proportions of 2 : 1 with requisite quantity of water to render it in flowing conditions. This mix took about 48 to 72 hours for setting after which time the grouting operations were started.

The properties of the grout mixes used in the grouting were evaluated before hand. A close watch on these properties was kept to have a uniform grout of identical properties. Bentonite having liquid limit less than 200 was rejected. The variations in property of bentonite was from 200 to 350 percent. The percentages of bentonite and sodium silicate were varied as and when required to obtain the required viscosity and setting time.

Acceptance tests for checking the permeability of grouted sand were carried out @ 25 m c/c which indicated permeability values much less than the specified limit of 304·8 m per year.

Relief Measures

Additional measures comprising a line of relief wells have also been provided on the downstream side of the dam to relieve residual pressures which may be transmitted into the foundation through any joints left ungrouted, leaky panel joints or due to subsequent development of cracks in the foundation cut-off. The system comprised 450 mm diameter fully penetrating relief wells at 30 m apart.

Performance of the Dam

The reservoir created by constructing Obra Dam was filled for the first time to its full supply level in September 1969 and performance of the dam is quite satisfactory since then without any trouble of any type.

Conclusions

1. Obra sand is quite coarse and is not likely to undergo liquefaction at relative density of 30 percent and above.

2. The total settlement due to vibration as percentage of thickness of deposit varies with initial relative density of the deposit. The total settlement has been calculated as 1·11 m allowance for which has been made in the free-board. Liberal provision of filters has been made on both contact faces of impervious core for healing up differential cracks, and below the toes of the dam for preventing dislodgement of foundation sand in the zone of high pore pressures resulting from seismic vibrations.

3. Dam section in the portion of compressible foundation material has been provided with 2·5 m thick filter layer of extra fine sand interposed between coarse sand layer on the upstream slope. Besides clayey material of high plasticity has been placed in the dam section at a moisture content 20 percent on the wet side of the optimum to ensure built in deformability against tendency for differential settlement.

4. Adequate provision for reducing seepage has been made by providing diaphragm. Relief wells have been provided to relieve residual pressure, if any.

References

Compaction of Obra Dam Foundation Sands by Blasting—A Field Study, *T. M. No. 37*, U.P. Irrigation Research Institute, Roorkee, October 1966.

JAI KRISHNA and SHAMSHER PRAKASH (1966). Report on Blasting Tests at Obra Dam Site, *School of Research & Training in Earthquake Engineering*, University of Roorkee, Roorkee, October 1966.

JAI KRISHNA ; SHAMSHER PRAKASH and MATHUR, J.N. (1965). Study of Liquefaction of Sand with Particular Reference to Obra Dam, *School of Research & Training in Earthquake Engineering*, University of Roorkee, Roorkee, August 1965.