Properties of the Diversion Channel Soil and Problems of its Utilisation in the Earth Dam at Ukai

by

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Introduction

UKAI across the Tapi is the largest reservoir on the west coast of the country. Out of the total length of 4,927 m the earthen portion of the dam at Ukai covers about 4,000 m and its maximum height above the river bed is about 69 m. It is a zoned rolled fill embankment involving controlled placement of about 23.5 million cu m of earth. In addition other miscellaneous earth work to the extent of about 6 million cu m has been done. At F. R. L. the dam impounds 851×10^3 ha-m of water to extend irrigation facilities to 3,83,000 ha of culturable area. It will generate 193 MW of hydro power at 35 percent load factor and will also provide flood relief to the areas lower down including Surat City.

As a result of detailed layout studies, the entire masonry dam has been located on the left bank terrace and the river channel itself is plugged by an earthen dam. Low saddles on the left and right banks have also been filled with earth embankments. The layout of the various components of the dam is shown in Figure 1.

To facilitate the construction of the earth dam in the gorge, the river was diverted through an open cut diversion channel (perhaps the largest of its kind in the river valley projects of the world) on the left bank terrace which will eventually function as the spillway channel† involved handling of about 6510×10^3 m^3 of earth, most of which was silt deposited by river *action, and 453×10^3 m^3 of rock.*

The dam has recently been completed. For economy and convenience the huge quantity of over 6.51 million m^3 of soil available from the compulsory excavation of the diversion channel had obviously to be utilised suitably

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† The excavation of the diversion channel.

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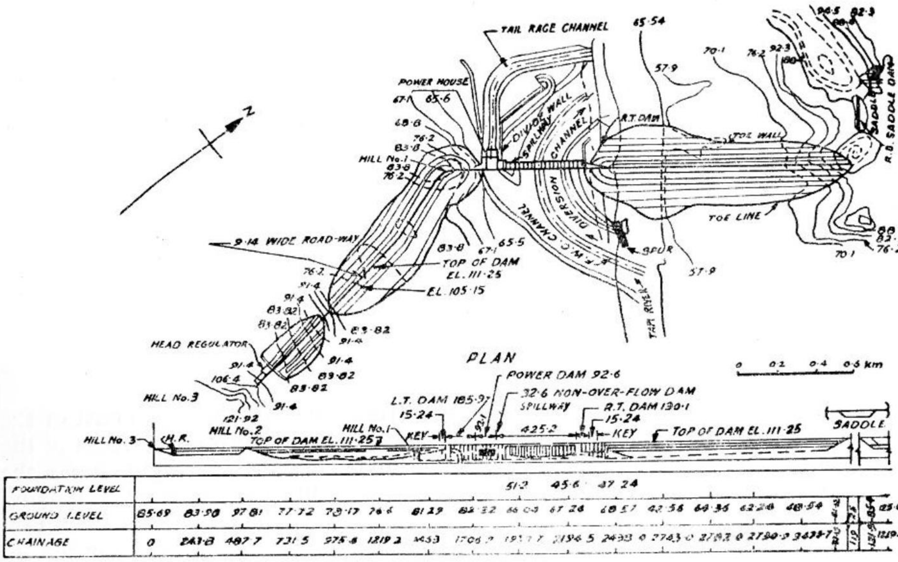


FIGURE 1 : Ukai Project : General layout of Ukai Dam.

in the earth dam. The physical and engineering characteristics of this material and its compactibility proved to be of special interest and significance and are discussed in detail.

After flowing through a hilly terrain the Tapi emerges into the plains of Gujarat near Ukai about 130 km upstream of its mouth. In its transitional reach near Ukai the river has particularly a uniform section with a bed width of about 300 to 360 m and steep banks. The river, during the high flood stages, has established its regime with more or less stable banks in its own alluvium. The river carries a heavy silt charge during floods and practically no silt during the fair weather. The variation of silt load with flood stages has caused the peculiar type of bank formation resulting in essentially silty overburdens at the dam site, extending to about 360 m on either bank. The deposits are occasionally interspersed by fine to medium sand layers and lenses which also signify a pattern involving depositions at varying flood stages.

The Diversional Channel

The huge quantity of earthwork 7.36 m^3 in the dam, in the river channel required the construction to spread over three seasons making it obligatory to divert the huge monsoon floods through a specially excavated channel which after numerous studies was located on the left bank to pass through the spillway portion. It had a bed width of 235 m and a depth of cutting averaging 20 m. The 1525 m long channel involved 6.51 million m^3 of overburden and 0.453 million m^3 of rock excavation.

The entire overburden in the channel excavation consisted of mostly feebly plastic to non-plastic silty soil and unless it was usefully utilised in appropriate zones of the various reaches of the earth dams, its very disposal would pose a difficult problem apart from adversely affecting the overall economy of construction of the dam. Its use in the earth dam, therefore, became more or less obligatory though, as explained later, the compactibility of the soil presented an intricate problem.

The design and phasing of construction of various reaches of the earth dam have been largely governed by the quantity and phasing of excavation of the diversion channel. Figure 2 shows the cross-section of the earth dam between hills 1 and 2 where the diversion channel silt has been abundantly used. It is conspicuously seen how the impervious core had to be kept rather excessively slanting in order to accommodate maximum quantity of the silt in the downstream random zone. With the silt used in the earth dam in the river channel and on the right bank, the total utilisation has been of the order of 90 percent of the total overburden excavation.

Characteristics of the Silt

Such an ambitious programme of utilisation naturally require intensive study regarding the geotechnical characteristics of the material. A number of bailer holes 559 mm dia and 18 to 20 m deep were therefore drilled at representative locations to collect samples. While collecting, the total sample at a particular depth was taken with the help of the bailer with a flap valve without allowing the fines to wash out. Samples were also collected from deep trial pits along the diversion channel. The results of tests on the samples indicated that the material was more or less uniformly grained clayey to sandy silt which could be classified in CL, ML, SM and occasionally CL—ML groups.

For various tests the values fall within the following brackets :—

(*i*) *Grain size*

(*a*) Clay	0 to 24%
(*b*) Silt	36 to 75%
(*c*) Sand	14 to 55%
(*d*) Gravel	0 to 10%
(*e*) d_{10}	0.001 to 0.03 mm
(*f*) d_{15}	0.002 to 0.006 mm
(*g*) d_{50}	0.02 to 0.25 mm
(*h*) $\frac{d_{60}}{d_{10}}$	4.44 to 36 (For sandy material).

(*ii*) *Atterberg Limits*

(*a*) Liquid Limit	24 to 49
(*b*) Plastic Limit	Nil to 29
(*c*) Plasticity index	Nil to 24

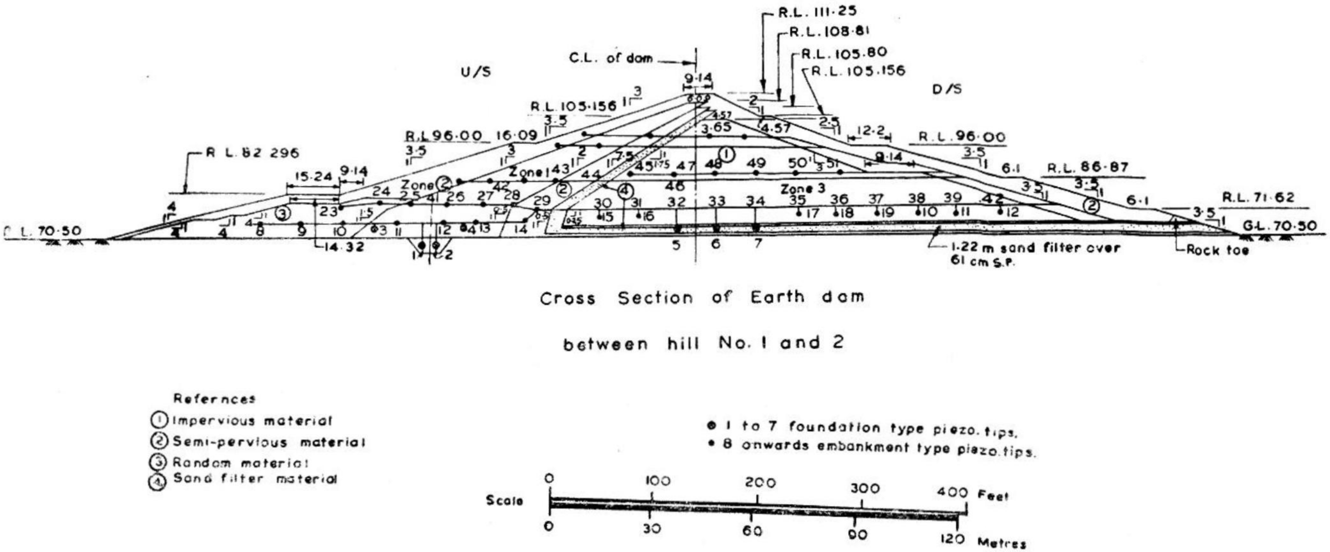


FIGURE 2 : Cross-section of earth dam between hill Nos. 1 & 2.

(*iii*) *Compaction Tests*

(*a*)	Standard Proctor Maximum Dry Density	1.57 to 1.8 gm/c.c.
(*b*)	Modified Proctor Maximum Dry Density	1.76 to 1.86 gm/c.c.
(*c*)	O. M. C. for (*a*)	18 to 24%
(*d*)	O. M. C. for (*b*)	14 to 18%

(*iv*) *Specific Gravity* 2.65 to 2.70

From these studies the silty material could be divided into (*i*) distinctly plastic CL, (*ii*) border-line mix soil CL—ML and ML and (*iii*) non-plastic ML, ML—SM and SM types. Quantitative surveys revealed that about 90 percent of the total soil fell in the last category.

Looking to the laboratory characteristics and the planning for large scale utilisation of the soil, studies had to be oriented to consider the following aspects :—

(*i*) The compactibility of the soil with respect to the optimum conditions and the best suited equipment.

(*ii*) Its in situ behaviour in the foundation of the earth dam during construction and after the filling of the reservoir, as the properties above indicated probability of loessic behaviour of the soil.

Field Compaction Tests

A programme of intensive field testing was, therefore, taken up. Initial studies indicated that the CL type of soil could be compacted to required standard with the help of normal sheeps-foot-rollers, though with some difficulty and after applying 2 to 4 more passes as compared to the normal 8 to 10 passes. However, the feebly plastic to non-plastic types proved to be the most difficult to compact. It was seen that though the laboratory compaction values were fairly high the field compaction could not be obtained to the required standard during initial tests with the normal sheeps-foot-rollers. Test embankments for trying various rollers, layer thickness, compactive efforts and moisture contents, etc., were therefore planned.

With a 23 cm thick loose layer, the standard sheeps-foot-roller (foot pressure 47 kg/cm^2) could give an average percentage compaction $\left(\frac{\text{Field dry density}}{\text{Standard proctor M.D.D.}} \times 100\right)$ of 88 to 92 percent (95 percent being obtained only in rare cases, and many cases falling even below 85 percent). It was seen that soil got locally sheared off presumably due to low bearing capacity. There was, therefore, no gradual compaction from down upwards and the "walking out" of the roller teeth was not observed. The foot pressure was, therefore, reduced to 23.5 kg/cm^2. This had however no material effect. Partial compaction was obtained mainly due to the passage of the roller drum over the soil. For further reducing the pressure the teeth of the sheeps-foot-roller drums were cut out and they were converted into plain drum rollers each drum weighing 4 tonnes when unballasted, to be drawn by crawler tractors. The layer thickness was reduced to 15 cm. The result was a little better and percentage compaction obtained was of the order of 91 to 94 percent with 12 coverages. Ordinary road rollers weighing 8 to 10 tonnes were also tried but the results were not satisfactory. A 4-tonne vibratory smooth roller with an equivalent dynamic loading of 16 tonnes was also tried considering that the non-plastic, less cohesive soil might respond to vibratory compaction. The roller however could not move properly in layers of 15 cm and above. The wheels tended to bog down or spin round without forward movement, indicating that much thinner layers would be necessary. As this was not practicable looking to the huge programme ahead, this roller was not tried further. The pneumatic tyred roller are considered to be very well suited for compacting silty soils and a 30-tonne pneumatic roller available on Dantiwada Project in the State was also tried out. With the layer thickness of 15 cm the compaction obtained was hardly 91 to 95 percent (the latter being rare) with 12 passes. Finally a low foot pressure roller was developed by welding 20 cm × 10 cm steel plates to the feet of an ordinary sheeps-foot-roller. This roller was termed "elephant foot roller" a name given by Prof. Sowers of the Georgia Institute of Technology. The pressure intensity could be thus brought down to 8.8 kg/cm^2. Three pressure intensities of 8.8 kg/cm^2, 10.3 kg/cm^2 and 12.3 kg/cm^2 were used for trials. This roller gave the best relative performance with a pressure intensity of 10.3 kg/cm^2, the percentage compaction achieved being 90 to 97 percent with 12 passes. The roller gave a uniform and consistent performance compared to other types where achievement of higher compaction up to 95 percent was occasional. The summary of typical rolling trials (not all) conducted is presented in Table I. As a result of these experiments, it was decided to use the elephant foot rollers for compacting the problematic material. The use of the silty material was limited to the downstream zones of the various reaches of the dam next to the inclined filter and on the upstream side of the core below the drawdown level where it would not be subjected to effects of rapid drawdowns. Even so, cover zones of granular murrum on either side were considered essential. A compaction of 90 to 92 percent of standard proctor maximum dry density achievable by 8 to 10 passes of elephant foot roller was considered initially satisfactory for the saddle dams where the material would have at least four years to consolidate under its own load before the dam was commissioned. Shear parameters were worked out for the purposes of design corresponding to the relatively low densities as could be expected to be achieved on the field.

Uniformly graded silty soil of ML and SM group are inherently less susceptible to adequate compaction. However, to confirm that the tendency was not attributable to other factors, chemical and petrographic tests were got carried out on a few samples. It was seen that R_2O_3 ($SiO_2 + Fe_2O_3 + Al_2O_3$) content was good (above 75 percent) and the loss of about 8 percent on ignition was mainly due to loss of CO_2 rather than of any organic matter. The minerals present were apophyllite, heulandite, agate, opal, jasper, quartz, calcite, magnetite, chalcedony, olivine and particles of ferrous rich kankar. It was suspected that apophyllite being fresh, flaky with well developed basal cleavage and smooth cleavage planes was perhaps

TABLE I

Summary of Typical Roll'ng Trials on Diversion Channel—Silty Soil with Different Types of Rollers.

Sl. No.	Type of Roller	M.D.D.	O.M.C.	Thickness of layer in cm	Density achieved after					
					8 Pass :		12 Pass :		16 Pass :	20 Pass
					% M.C.	D.D.	% M.C.	D.D.	% M.C.	D.D.
1	2	3	4	5	6	7	8	9	10	11
1.	Sheep-Foot-Roller (23.5 kg/cm²)	1.62	23.1	15	—	—	14.2	1.42	14.9	1.47
					—	—	13.6	1.46	13.6	1.47
		1.70	22.8	15	—	—	17.6	1.45	19.0	1.47
					—	—	17.6	1.44	17.6	1.45
		1.62	23.1	15	—	—	22.0	1.45	22.7	1.46
2.	Sheep-Foot-Roller (47 kg/cm²)	1.62	25.0	15	—	—	15.6	1.38	13.6	1.40
					—	—	14.3	1.36	14.9	1.42
		1.62	25.0	15	—	—	20.5	1.35	19.8	1.32
					—	—	19.8	1.34	21.2	1.38
					—	—	21.2	1.33	20.2	1.39
					—	—	22.0	1.32	19.8	1.38
3.	Elephant Foot Roller (8.8 kg/cm²)	1.62	24.8	15	—	—	15.6	1.51	15.6	1.53
		1.63	24.8	15	—	—	17.6	1.50	17.6	1.51
					—	—	17.6	1.51	19.8	1.52

TABLE I (*Contd.*)

1	2	3	4	5	6	7	8	9	10	12
4.	Elephant Foot Roller (10.28 kg/cm^2)	1.59	23.5	15	14.9	1.48	14.9	1.52	15.6	1.54
					14.9	1.48	15.6	1.53	17.6	1.56
		1.64	23.6	15	19.0	1.46	19.0	1.54	18.3	1.55
					19.6	1.47	19.8	1.51	18.3	1.41
		1.64	24.2	15	19.5	1.44	22.0	1.50	19.8	1.42
					20.5	1.47	20.5	1.53	21.2	1.55
5.	Elephant Foot Roller (12.3 kg/cm^2)	1.64	24.7	15	—	—	15.6	1.45	14.2	1.46
			—	—	—	—	15.6	1.43	16.2	1.46
		1.59	24.1	15	—	—	19.2	1.47	19.8	1.48
		1.64	24.7	15	—	—	19.8	1.50	21.2	1.50
							20.5	1.49	21.2	1.51
6.	Converted Plain Roller	1.64	24.2	15	15.6	1.43	16.5	1.47	15.6	1.49
					15.6	1.44	14.2	1.46	14.9	1.48
		1.64	23.6	15	16.2	1.47	18.3	1.48	18.3	1.56
		1.64	24.2	15	18.8	1.33	22.0	1.49	22.0	1.49
7.	Pneumatic Roller (30 tonnes)	1.59	22.9	15	18.2	1.44	17.0	1.45	17.0	1.42
		1.66	20.6	15	19.8	1.46	19.8	1.50	19.8	1.52
		1.66	20.6	15	22.0	1.47	21.2	1.47	21.2	1.49
		1.66	20.6	15	25.0	1.45	24.2	1.47	23.4	1.49
		1.64	23.4	15	24.2	1.46	24.2	1.46	24.2	1.46
8.	Pneumatic Roller (60 tonnes)	1.81	15.6	30	14.9	1.69	14.9	1.74	—	—
		1.67	21.4	30	19.8	1.55	19.8	1.59	—	—
		1.67	19.8	30	22.0	1.57	22.0	1.63	—	—
							(10 Passes)			
		1.69	21.0	38	19.0	1.59	19.0	1.67	—	—
		1.67	20.5	38	19.0	1.52	19.0	1.58	—	—

responsible for the soil not getting adequately compacted as the particles had a tendency to slide over one another. Smooth surfaced rounded to semi-rounded particles of amorphous agate and chalcedony would also prevent proper compaction. It was, however, seen that the percentage presence of all these minerals was so small that their effect on the total sample would hardly be significant. There was some mica (13.45 percent of the sandy fraction) also which perhaps affected the compaction, but this was also small as compared to the total sample and was not considered to be the root cause of the problem.

The only conclusion that could be drawn was that at Ukai, the non-plastic silty soils were more difficult to compact specially because of their practically non-cohesive nature and because of the large components of coarse silt and fine sand (which in certain cases exceed 70 percent). The shear strength of the material in the as laid loose condition (before rolling) is thus significantly low compared to other relatively plastic material with the result that even foot pressure intensity exceeding about 10.3 kg/cm^2 of the roller caused failures of the type that occurs in foundation of buildings where a low bearing capacity material is met with. Mobilisation of side restraint (adequate value of the coefficient of passive earth pressure) was poor with the result that compaction from bottom upwards could not be achieved as in a conventional fine grained material compacted by a sheeps-foot-roller. The solution to the problem therefore lay not only in restricting the foot pressure to relatively low values but also having adequate bearing area (as in the case of a footing). This objective was partially achieved by increasing the bearing area of the foot to 206.4 cm^2 against 45.15 cm^2 of a sheeps-foot-roller tooth. Even this change was not quite conducive to transmitting the intensities uniformly during the passage of the feet over the soil. Though the roller was successfully used in the initial stages of the work on a saddle, there were difficulties when even a slight improper blending of moisture (added on the fill) was noticed. The search for more suitable rollers was, therefore, continued.

The pneumatic tyred roller is ideally suited for rolling low strength silt of ML group but the one used for the diversion soil in initial compaction trials had perhaps too low a weight (30 tonnes only) and inadequate tyre pressure (not exceeding 4.2 kg/cm^2) to result in effective compaction of the soil. Heavy pneumatic tyred roller thus appeared to be the right choice for compacting this type of the material. Three such rollers were procured for Ukai in the year 1965. This could be loaded up to 100 tons and had the maximum tyre inflation capacity of 8.4 kg/cm^2. Each roller had four tyres which give a contact width of 55 cm each. Trials with this type of roller with 23 cm to 30 cm thick loose layers yielded good results. It was found that the non-plastic soil could be compacted even with 30 cm loose layers to 95 to 97 percent (or even a little more) of Proctor maximum dry density with about six to eight passes.

The results of trials with these rollers are also given in Table II. For the 30 cm loose layer a higher weight up to 80 tonnes was necessary for economy in number of passes. The ideal moisture content for compaction was found to be about 1 percent below the O.M.C. The rollers gave very good results if the fill moisture was more or less uniform.

If the addition of moisture in the field was however not uniform and if the loose layer thickness was of 30 cm or more the roller tended to bog-down at relatively wetter spots. To overcome this difficulty, the moisture was added in the borrow area (diversion channel) itself well in advance so that the material brought in the field had more or less uniform moisture content near about the O.M.C. As an additional precaution the roller weight was reduced to 60 tonnes. With this weight a 23 cm loose layer could be compacted on application of 6 to 8 passes, 30 cm and 37.5 cm layers required 10 to 12 and 12 to 14 passes, respectively. With 37.5 cm thickness, however, there were chances of lower part of the layer remaining loose and hence this was not continued further. The final practice adopted was to roll the soil in 30 cm lifts with 10 passes of the 60 tonne roller. The progress could then be boosted up very considerably and as much as 1.7 million m^3 of this soil could be laid and compacted in a single season.

Laboratory Tests for Assessing the Behaviour of the Soil in the Foundation of the Earth Dam

A part of the earth dam at Ukai rests on the terrace overburden of the silty soil. It was therefore very necessary to investigate the behaviour of the soil under the load of the dam and also on saturation after filling of the reservoir, particularly as the grain size and other physical characteristics indicated possibility of volume collapse on saturation due to the probable loessic behaviour of the soil.

Investigations necessary to judge the loessic behaviour of a soil would have to cover the following criteria :—

(*i*) Grain size distribution with a view to comparing the grain size curve with the curves of loessic soils.

(*ii*) U.S.B.R. criterion for the design of foundations on the fine grained soils.

For such soils U.S.B.R. recomends a schematic chart from which, merely by using the density results it can broadly be determined whether a treatment of foundation against sudden settlement on saturation is required. (Design of small dams U.S.B.R.).

(*iii*) The criterion of liquid limit and field density as suggested in the paper read by Huang Wen XI at the 5th International Conference on Soil Mechanics and Foundation Engineering at Paris. The criterion used in this paper is based on a comparison of the liquid limit of the soils with the water content required to saturate the soil in its natural state of compaction (i.e., field density). If this water content is greater than the liquid limit, it is quite natural that such a soil may lose shear strength partly or completely which may result in a settlement.

TABLE II

Statement showing the field densities and Proctor values of samples of Ukai Soil.

Sl. No.	Location	Depth in metres	Field density	N.M.C W	Proctor values M.D.D.	Proctor values C.M.D.	Wo-w	Field density $\times$ 100 / M.D.D in %	L.L.	Calculated S.M.C.
1	2	3	4	5	6	7	8	9	10	11
1.	1920 C.L.	1.52-1.68	1.57	22.7	1.62	23.1	0.4	97	37.7	26.5
2.	1935 C.L.	1.22-1.37	1.48	27.3	1.75	19.0	—8.1	24.5	44.0	30.2
3.	-do-	1.68-1.98	1.36	33.3	1.52	28.0	—5.3	90	36.4	36.3
4.	2256/228 u/s	2.44-3.05	1.55	20.5	1.7	21.6	1.1	91.2	38.8	27.5
5.	-do-	3.05-3.96	1.65	18.2	1.73	18.4	0.2	95	36.0	23.6
6.	-do-	3.96-4.88	1.61	23.0	1.65	22.4	—0.6	97	36.2	25.0
7.	2080/335 u/s	3.66-4.88	1.60	22.1	1.66	19.8	—2.3	98	32.5	24.1
8.	2286/603 u/s	0.92-2.28	1.65	21.8	1.75	17.4	—4.4	89	36.0	27.6
9.	2408/454 u/s	2.44-3.66	1.63	23.5	1.71	19.2	—4.3	96.5	35.0	24.0
10.	2042/228 d/s	2.44-3.66	1.60	22.2	1.74	19.9	—2.2	92	34.3	25.3
11.	2348 C.L.	3.05-3.20	1.38	31.7	1.52	25.8	—5.9	90.2	44.0	35.4
12.	-do-	3.35-3.50	1.56	34.2	1.57	25.0	—9.2	99	41.8	26.9
13.	2395 C.L.	2.28-2.44	1.45	26.3	1.64	23.6	—2.8	89	39.5	31.7
14.	2410 C.L.	1.98-2.13	1.44	25.0	1.61	25.0	0.0	90	38.1	32.0
15.	-do-	3.05-3.20	1.43	29.0	1.65	22.46	—6.6	84	27.0	32.6
16.	2425 C.L.	1.83-1.98	1.46	27.0	1.62	24.5	—2.8	90	36.5	31.5
17.	-do-	2.44-2.59	1.48	28.2	1.53	28.3	0.1	97	42.7	30.2

TABLE II—(*Contd.*)

1	2	3	4	5	6	7	8	9	10	11
18.	2455 C.L.	1.98-2.13	1.37	17.6	1.59	26.4	8.8	86	32.2	36.0
19.	-do-	3.35-3.50	1.49	21.2	1.73	23.0	1.8	86	31.5	30.0
20.	2470 C.L.	1.83-1.98	1.42	19.8	1.65	23.8	4.0	86	29.8	33.3
21.	2470	2.59-2.74	1.48	23.4	1.79	23.0	—0.4	83	24.4	30.4
22.	2313/244 d/s	4.57-6.10	1.43	12.90	1.67	21.60	8.7	85.6	31.6	32.7
23.	2327/198 d/s	4.57-6.10	1.39	14.20	1.58	23.60	9.4	87	29.2	34.9
24.	2347/198 d/s	4.57-6.10	1.41	12.30	1.68	22.0	9.7	84	32.0	33.7
25.	1920	0.75-0.92	1.38	32.7	1.44	30.6	—1.1	96	52.6	35.2
26.	1950	1.07-1.22	1.37	29.8	1.50	29.6	0.2	91.5	42.9	35.9
27.	1968	1.68-1.83	1.44	31.7	1.59	25.3	—6.4	91	41.8	32.2
28.	1968	1.07-1.22	1.40	28.2	1.44	31.6	3.4	97	63.0	34.2
29.	1983	2.44-2.59	1.39	39.8	1.55	26.7	—13.1	89	60.5	34.8
30.	2318/183 u/s	1.52-3.05	1.37	20.5	1.62	23.9	3.4	84.6	38.4	34.9
31.	2258/457 d/s	0.0-1.52	1.55	22.0	1.60	22.1	0.1	97.1	46.0	27.2
32.	-do-	1.52-3.05	1.51	21.2	1.58	24.8	3.6	95.5	26.0	29.1
33.	-do-	3.05-4.57	1.49	20.5	1.58	25.0	4.5	94.6	45.0	29.8
34.	-do-	4.57-6.10	1.56	21.2	1.58	27.6	6.4	99.1	30.3	26.8
35.	2285/92 u/s	3.05-4.57	1.57	22.7	1.81	15.4	—7.3	86.9	28.0	26.5
36.	-do-	4.57-6.10	1.58	23.4	1.71	18.4	—5.0	92.0	22.0	26.1
37.	2318/183 u/s	4.57-6.10	1.45	22.7	1.79	30.0	7.3	80.9	25.0	31.7

Note : Locations of samples are with respect to upper alignment.

Looking to the previously described characteristics of the soil it was felt that under loose state it might collapse under load and saturation. The experiments for above mentioned criteria were therefore carried out and results analysed.

(*i*) On Figure 3 are shown the curves for Franklin falls dam, exceptionally unstable sands, stable sand, sandy loess limits and Ukai silt curves. The curves for stable and unstable sands are given by Terzaghi in his book "Soil Mechanics in Engineering Practice" page 102, (Asian Edition). The diversion channel silt curves fall mainly within the limits of loessic soil near the unstable region.

(*ii*) *U.S.B.R. Graph* : The computed parameters from test results of 37 representative samples were plotted on a graph (Figure 4) having on abscissa the difference in O.M.C. and N.M.C. (+ve or −Ve as the case may be) and on ordinate the ratios of field density to proctor density expressed as percentages (Table II). It was observed 28 points fell in the "no treatment" zone and the other 9 points fell in the "treatment" zone. Out of these 9 samples 5 were from diversion channel and 4 were from the seat of the right transition dam.

(*iii*) The same 37 samples were examined for the criteria of liquid limit plotted against the field density. It was seen on the standard graph that out of 37 points, 27 fell below the standard graph and the other 10, above it (Figure 5). Out of these 10 samples in the treatment zone, 6 were from diversion channel and the remaining 4 were from the seat of the transition dam. Four of the above mentioned 6 were the same which fell in the treatment zone as per U.S.B.R. graph. Out of the remaining 4 samples 3 were those falling in the treatment zone in tne U.S.B.R. graph also.

The Saturation Moisture Contents (S.M.C.) and liquid limits were compared for 37 samples. It was found that the saturation moisture content in the field state was less than the corresponding liquid limit for 27 samples. Out of the remaining 10 samples, 7 fall in the treatment zone in the U.S.B.R. graph and all 10 fall in the treatment zone in the standard graph of liquid limit versus field density. Of these, 6 samples are from the diversion channel and the remaining 4 are from the seat of the right *transition dam.*

Consolidation Tests : For studying probable volume collapse on saturation standard consolidation tests were carried out on 17 samples under different conditions up to a maximum loading of 8.7 kg/cm^2. The tests were carried out in the standard way by first saturating the sample for 24 hours and then subjecting it to a stress of 1.1 kg/cm^2. The consolidation was observed for 24 hours and then the load was increased to 2.2 kg/cm^2. The experiments was continued with loads of 4 and 8.71 kg/cm^2.

The volume collapse study was also done by first observing consolition up to the required load under O.M.C. condition and then saturating the sample under this load and observing its sudden settlement if any during the next 24 hours.

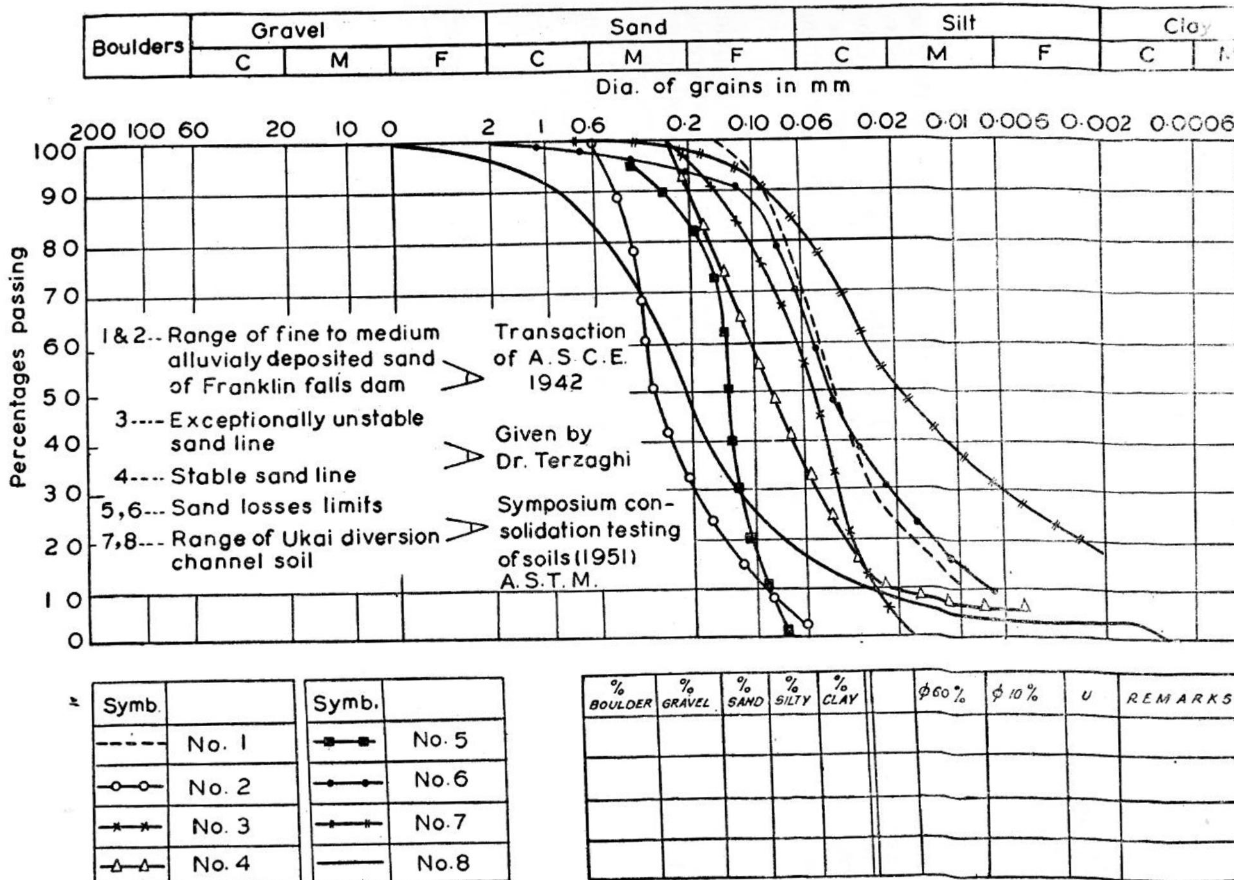
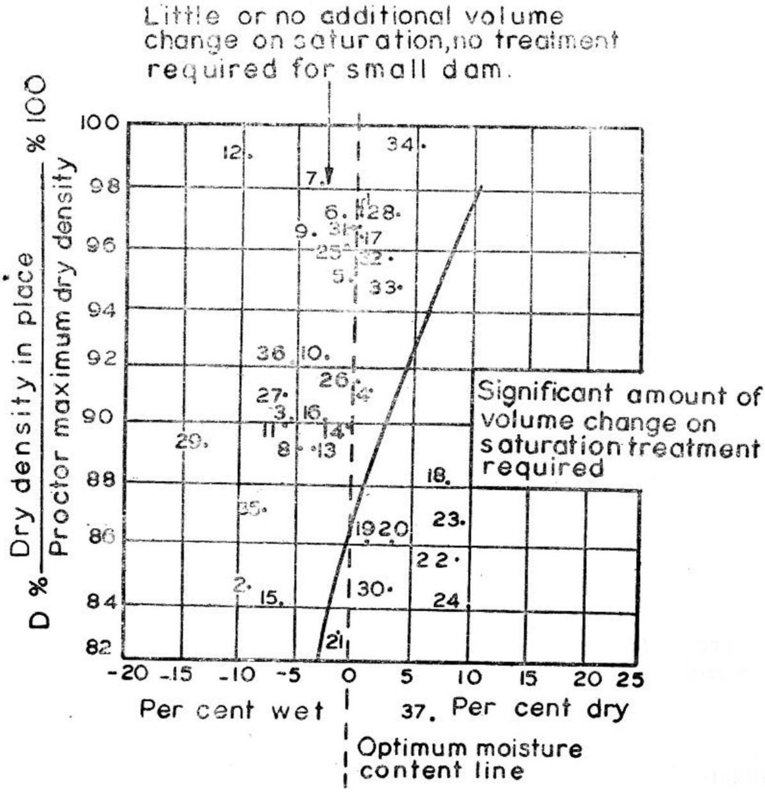


FIGURE 3 : Comparison of the Ukai silty soil (left bank) with other unstable silts and sands.



Note :–

1. The position of any point on this graph is a function of Proctor's maximum density and natural moisture content for a fixed value of field density.
2. Figures given in the plot above are based on table II.

FIGURE 4 : **U.S.B.R. Graph—Ukai Dam Project—U.S.B.R. foundation design criteria for relatively dry *fine grained soils.***

From the settlement values with time for two typical samples as observed under each load for 24 hours, it is seen that in both samples about 90 percent of the primary consolidation occurs in the first minute only, after applying the load. Subsequent consolidation is very small and is secondary one. It is found that the maximum consolidation varies from 3.2 to 13 percent at 8.7 kg/cm². Volume collapse study was done on 4 samples by saturating them at 2,4 and 8 tonnes and another 8 samples by saturating them at 4 tonnes. The volume collapse was found to vary from 0 to 0.464 percent only.

From all these studies it was concluded that there were hardly any chances of volume collapse and hence it was decided not to plan any special treatment for the soil.

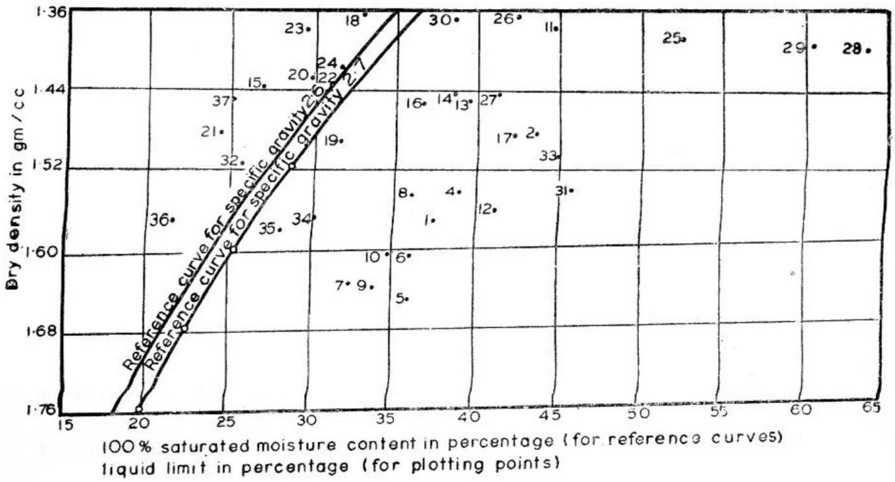


FIGURE 5 : Liquid limit criteria for settlement on saturation—Graph for liquid limit versus field density.

Compressibility and Pore Pressure Characteristic of the Soil in the Embankment

Before the earlier decision to permit the placement of the soil at about 92 percent proctor was taken, the compressibility and the hydro-consolidation properties of the compacted soil were carefully studied. The typical consolidation curves of samples remoulded at about 92 to 93 percent of Proctor density showed that under a load of 6.6 kg/cm² corresponding to a height of 36.5 m the total settlement would be of the order of 5.5 percent. While this is rather high, the average time for 90 percent consolidation is, however, only 62 secs. (86.4 secs. being the maximum for a clayey type). The average value of C_v for the mix type of soil works out to 0.0086 cm²/sec. More than 90 percent settlement would therefore be completed within a few months in the actual dam, assuming a height of raising of say 18.2 m in one season. The settlement would therefore not pose a serious problem.

For the maximum utilisation of the silty material, its use was also planned on the upstream below the lowest drawdown stage of the reservoir (R.L. 82.4 m). It was necessary to ensure that the semi-compacted silty soil did not undergo a relatively sudden settlement or hydro-consolidation on saturation. Tests were therefore conducted in the 7.5 cm diameter oedometers as well as in the large 20 cm diameter apparatus. The results showed that hydro-consolidation was only of the order 0.30 percent and would not present a problem.

Based on lower of the observed permeability values conservative design assumptions were made for construction pore pressures. Also, there was the problem of accommodating such a large quantity of the material in the dam and a flatter section on the downstream meant only additional handling and compaction costs. Stability of the saddle dam was, therefore, worked out with the assumption that the maximum pore pressure, generated (expressed as height of water column) would be of the order of $1H$, H being height of the dam above the slip circle. Later on calculations based on Hilf's method using the results of consolidation tests. With the rolling moisture contents considered at O.M.C. (conservatively) the pore pressure was computed as "0.5 H" m head of water and for a condition 3 percent dry of O.M.C. it was "$0.3H$" m (Appendix I). The permeability results observed over a season indicated K value between 0.2×10^{-6} cm/sec for impervious silt to 15×10^{-6} cm/sec for sandy silt indicating fairly early dissipation of pore pressures in most of the cases. The initial air content of the silt fill would be naturally high (which also could be seen during the deairing operations of the various piezometers embedded to study the pore pressure generation). It was, therefore, considered adequate to assume the pore pressure as "$0.5H$"m head of water instead of 1 H as previously assumed, H being the height of single raising in season. The design of the other reaches of the Ukai earth dam was prepared on this modified assumption. It was seen that about 1.13 million m^3 of earth work quantity could be saved due to steepening of the slopes as a result of the revised assumptions.

Because of appreciably slanting upstream core and the large random silty zone in the downstream part of the dam, it was decided to observe carefully and systematically the construction pore pressures by installing a battery of piezometers. The piezometers installed are shown in Figure 2. The observation of pore pressures was commenced in June 1966.

Figure 6 shows the pore pressure record of four representative piezometer tips in the silt (random) zone of the earth dam between hills 1 and 2, tips Nos. 33 and 34 in the tier at R.L. 79.4 m while Nos. 47 and 48 in the upper tier at R.L. 90 m. These selected tips are at locations where the pore pressures are not likely to be affected by filters, proximity of other zones, etc. It is seen that the pore pressure build up is very limited (about 1.5 m to 2.5 m of water head). The lower tier was embedded in 1966 and it shows continued dissipation after a little build up in the initial stage. In comparison to the height of fill above the tips, the pore pressures are only about 0.03 H to 0.04 H. The tips in the upper tier installed in mid. 1967 also show initial build up and subsequent dissipation. *The maximum rise is seen to be* about 0.25 H column of water. Small undulations in the patterns are due to the operation of pressure gauges in their lowest ranges of working, the build ups themselves being small. The accuracy of the gauges installed is not likely to be greater than 0.6 to 1 m of water head. Throughout the construction period the pore pressures have remained quite low presumably because the rate of construction was rather slow and in fact there was no raising of the embankment for about three seasons at the section where instruments were embedded. It can however be safely interpreted that unduly high pore pressures are not likely to be generated in this soil if the normal rate of construction is adopted and the design based on the construction pressures, derived by Hilf's method should adequately serve the purpose.

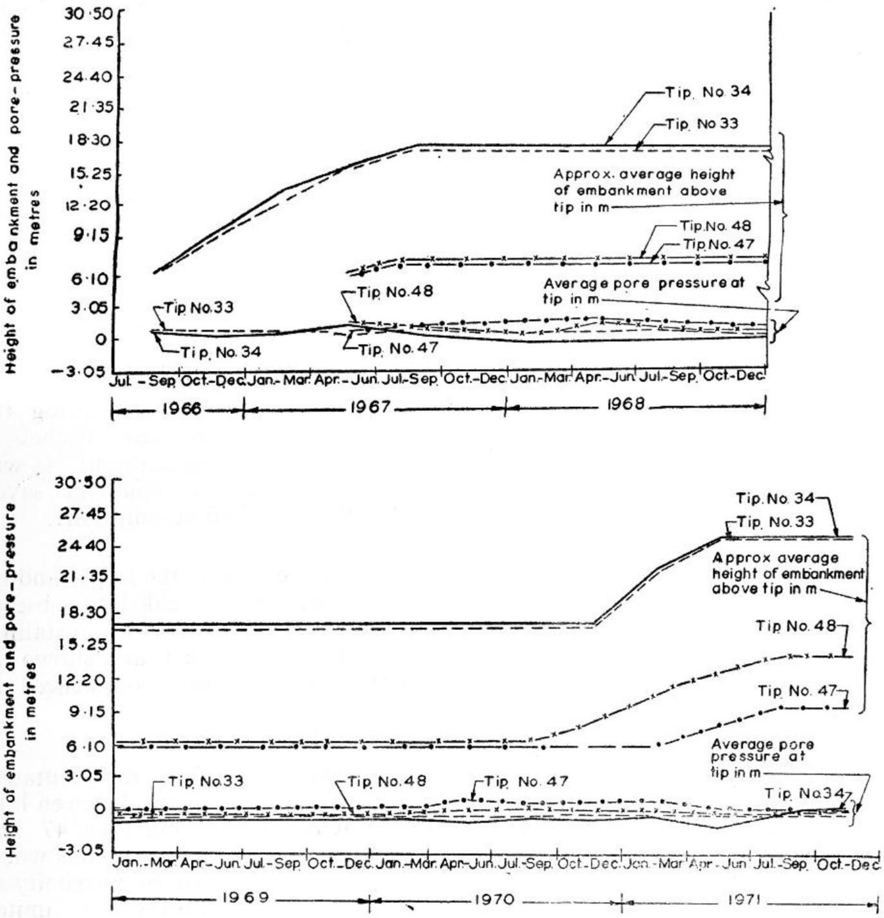


FIGURE 6 : Ukai Dam Project—Graph showing pore pressure as compared to height of embankment dam between hills 1 and 2, Ch. 1341 m random zone.

Tip No. 33 : 30 m downstream R.L. 79.4
Tip No. 34 : 42.06 m downstream R.L. 79.4
Tip No. 47 : 10.6 m upstream R.L. 89.90
Tip No. 48 : 5.5 m upstream R.L. 89.90

The development of construction pore pressure in the silty soil is also studied in the laboratory with a triaxial test known as "Ko-Test" (Bishop and Henkel) with zero lateral yield. Since the sophisticated equipment in which the lateral strain of the specimen can be accurately measured is not available in the laboratory an indirect method in which the coefficient of earth pressure at rest 'Ko' required to maintain practically a zero lateral yield over a given range of deviator stress was studied by trial and error. This value of 'Ko' was used for ensuring that the specimen did not yield laterally materially. The value used was 0.4 to 0.44. The major and minor principal stresses were raised in small increments allowing time for pore pressure responses. The results are given

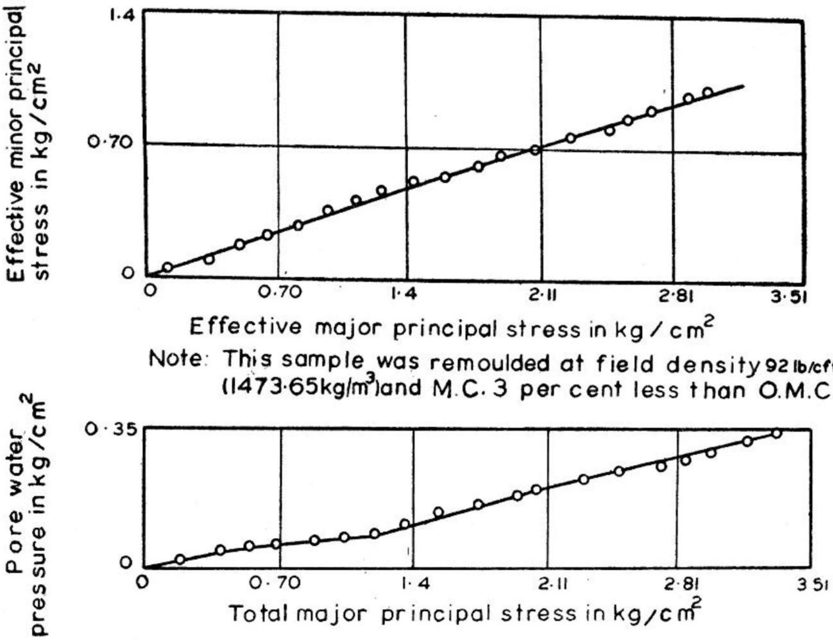


FIGURE 7 : Undrained Ko test (no lateral strain test) conducted on sample of non-plastic silty soil of left terrace.

in a plot in Figure 7 for the non-plastic soil. It is seen that for a principal stress of about 2.75 kg/ cm² (15.2 m raising) the pore pressure developed is 2.82 m of water, i.e., 0.185 H in terms of vertical water columns. The values correspond closely to the average values as observed. The construction pore pressure thus would not be a serious problem with the use of the diversion channel non-plastic soil.

Conclusion

The physical and engineering properties of the silty soil available from the diversion channel and existing below the seat of the dam had to be intensively studied because of the necessity of the obligatory use of the diversion channel silt in the dam for obvious economic reasons and also assessing the probable behaviour of the foundation soils particularly under the super imposed load and saturation. The studies revealed that :

(*i*) The silty soil can be used in the downstream zones in the full height and even in the upstream zone below the low water-level provided it is adequately compacted.

(*ii*) Satisfactory compaction of the soil can be achieved by the use of elephant foot type low foot pressure rollers or still better by the pneumatic tyred heavy rollers of 60-100 tonnes capacity.

(*iii*) Even in the in situ state, the soil in the left terrace is adequately stable and no volume collapse on saturation under load is apprehended.

(*iv*) The analysis shows moderate generation of pore pressure which is corroborated by the available data.

APPENDIX I

Computations of Pore Pressure for Left Bank Terrace Soils using Hilf's Formula

Hilf's equation for construction pore pressure is

$$U = \frac{Pa \times \triangle}{Va + 0.02\,Vw - \triangle}$$

Where, U=Pore water pressure in kg/cm^2

Pa=Atmospheric pressure in kg/cm^2

$\triangle = \triangle V \times 100$ percentage (settlement under load from oedometer test)

Va=Percentage volume of air measured in terms of original volume of soil.

Vw=Percentage volume of water measured in terms of original volume of soil.

For D.C. soil, compacted dry density

=1.52 gm/cc (after pneumatic roller)
rolling M.C.=21% (maximum)

and Sp. gr. =2.65 with these values,

Va=10.5%, Vw=32%

$\triangle$=3.7% (maximum) for mixed silty soil.

$$\therefore U = \frac{1.03 \times 3.7}{10.5 + 0.02 \times 32 - 3.7}$$

$$= \frac{3.811}{7.44} = 0.5 \text{ kg/cm}^2$$

i.e., 5.12 m of water.

In terms of height of fill this comes to $\frac{5.12}{10.67} = 0.479$

Say......0.5H

With rolling moisture content of 18%

Va=15.4%, Vw=27.1% and $\triangle$=3.7% (which will not change appreciably)

$$U = \frac{1.03 \times 3.7}{15.4 + 0.02 \times 27.1 - 3.7}$$

$$= \frac{3.811}{12.24} = 0.311 \text{ kg/cm}^2$$

3.11 m of water

$= \frac{3.11}{10.67} = 0.29\ H$ Say...0.3 H in terms of height of fill.

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