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SYNOPSIS: The foundation of the proposed Polavaram Earth and Rockfill Dam (India) has a sand deposit about 15 to 30 metre deep. The medium size sand is poorly to uniformly graded having relative density in the range of 40-45% and angle of internal friction 30°. Standard Penetration Tests have been conducted in 22 bore holes at site and the N-counts have been observed to range between 4 to 111. The liquefaction potential of deposit under maximum credible earthquake (MCE) for the site has been evaluated by analysing the data on the basis of prevailing emperical approaches and by the results of the experiments conducted on the sand in laboratory. The results of the analyses indicate that for the design earthquake the liquefaction would occur at some spots where N-counts are very low. The experiments have been conducted by depositing the sand, with the relative density close to that at site, in a box placed on a shaking table. The table was subjected to equivalent number of sinusoidal cycles corresponding to 75, 65 and 50% level of maximum acceleration of the time history of the design earthquake. As the dam provides an overburden pressure to the foundation, experiments have been conducted with loading on sand deposit in the shake table. The effect of frequency of motion has also been observed. The results of these experiments show that for the design acceleration level of 0.1g (65% of maximum acceleration) the maximum pore water pressure would be 31% of the effective overburden. The frequency of 3 cps seems the optimum frequency for generation of pore water pressure and at 8 cps this is less by about 20 percent. Thus the experiments indicate only partial loss of shear strength and not the total liquefaction of the foundation sand.

THE SITE AND SOIL DATA

The dam site is situated across the Godavari river about 2.5 km upstream of Polavaram village in west Godavari district, State of Andhra Pradesh. The proposed dam is an earth and rockfill type having maximum height of 60m (above river bed level 50m). The site plan, showing the axis of the dam and locations of the bore holes, is given in Fig.1.

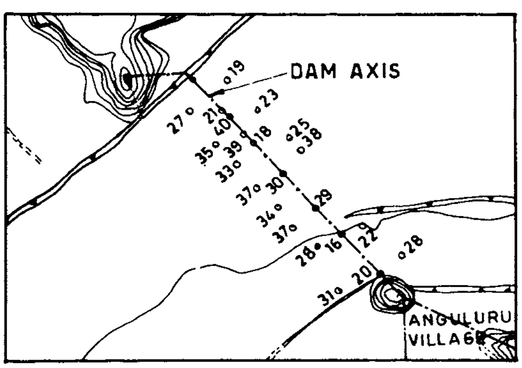

FIG.1_ PLAN SHOWING THE LOCATION OF THE BORES FOR POLAVARAM PROJECT.

The foundation of the dam consists of sand deposit, with clay lenses, of 15 to 30m thickness above the bed rock. The bore holes 22 nos. have been made for foundation investigations. For this, 8 nos. along the proposed axis of the dam, 6 nos. along the line 100 metre upstream and 8 nos. along the line 100 metre downstream of the axis of the dam are located (Fig.1). As the base width of the dam is about 320m the bore holes are located well within the foundation of the dam. In these bore holes,mainly sand and clay have been found, and at certain locations coarse sand with pebbles and river gravels are also present. The standard penetration tests (SPT) have been conducted in these bore holes at an average interval of 3m. The average grain size distribution of the sand is given in Fig. 2 and its geotechnical properties are listed in Table 1.

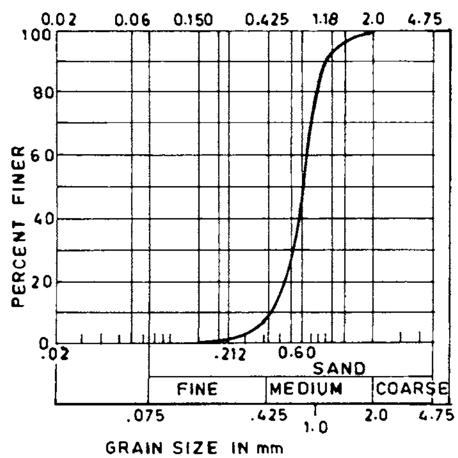

FIG. 2_ GRAIN SIZE DISTRIBUTION OF THE SAND

THE DESIGN EARTHQUAKE TIME HISTORY

The design earthquake data needed for this study have been taken from the report (1986) given for design earthquake parameters for the project site. The artificial accelerogram generated for the site and normalised to 1g is shown in Fig. 3. The maximum acceleration corresponding to maximum credible earthquake (MCE) is 0.16g for the site.

EVALUATION OF LIQUEFACTION POTENTIAL USING EMPERICAL APPROACHES

The approaches that have been used for evaluation of lique-

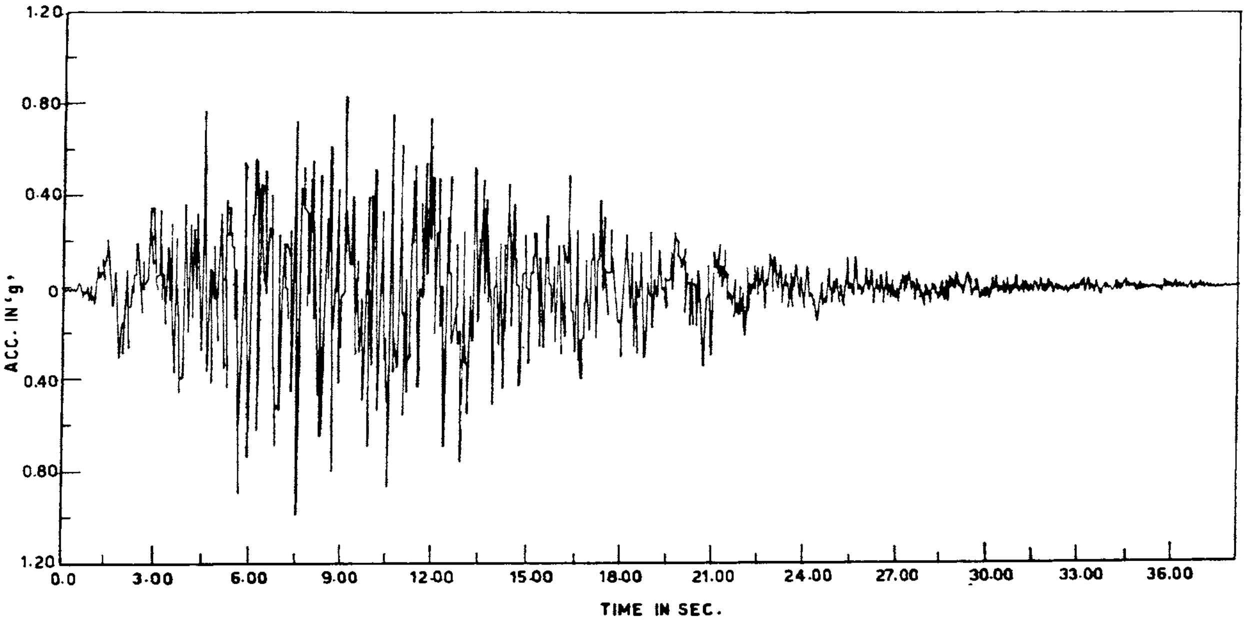

FIG. 3 _ ARTIFICIAL ACCELEROGRAM (NORMALISED TO 1g) FOR POLAVARAM SITE

TABLE 1. Soil Properties

	Data from site Engineers	Data from Laboratory Test at the University
I. Grain size properties		
a) Soil type	SP	SP
b) Effective size 'D_{10}'	0.12-0.77	0.45
c) Grain size 'D_{50}'	0.38-2.50	0.71
d) Max. void ratio 'e_{max}'	–	0.745
e) Min. void ratio 'e_{min}'	-	0.541
f) Specific gravity 'G'	2.69-2.72	2.65
g) Coefficient of Uniformity 'C_u'	2.0-11.1	1.78
h) Coefficient of curvature 'C_c'	0.5-10.9	1.0
II. Coefficient of Permeability	-	1.5×10^{-3} cm / sec
III. Stress-strain behaviour	-	Non-linear, hyperbolic
IV. Density		
a) Rel. density	-	43 percent
b) Submerged density	-	$1.0 T/m^3$
c) Saturated density	-	$2.0\ T/m^3$

faction potential of the site are: Seed and Idriss (1970), Chinese Building Code (1974), Seed et al. (1983) and Iwasaki (1986).

Seed and Idriss approach compares the average cyclic shear stresses to be induced by an earthquake with cyclic shear strength of the soil. The number of recommended equivalent cycles for an earthquake of magnitude 6.5 at 0.65 τ_{max} is 10. Hence for the site (earthquake magnitude 6.5) the number of stress cycles is taken as 10. In addition to this, the number of cycles as computed from the accelerogram of the site (Fig.3) and in accordance with the procedure suggested by Lee and Chan (1972) and Annaki and Lee (1975), the equivalent number of cycles for the site at 65 percent of peak acceleration level has come out to be 30. Therefore the cyclic shear strength of the soil has been calculated for 10 and 30 cycles. The typical plots of the stress induced and strength of the soil for borehole No. 16, 18 and 22 are given in Fig. 4, 5 and 6. It is seen from such plots for the site that in general the dynamic shear strength of the soil is higher than the average shear stress likely to be induced by the ground motion. Out of 21 boreholes considered there are only 4 boreholes which indicate that the dynamic shear strength of the soil at some depth is equal or less than the shear stress likely to be induced.

Chinese Building Code approach gives the critical value of SPT counts for different magnitudes of earthquake, below which the saturated sand deposit in a level ground is likely to liquefy. The value of $\bar{N}$ assigned for MMI VII earthquake

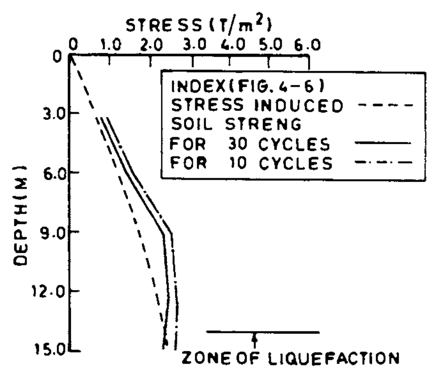

FIG. 4 _ EVALUATION OF LIQUEFACTION POTENTIAL AT B.H.16

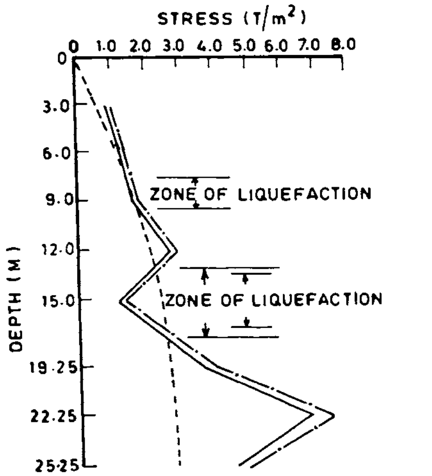

FIG.5_EVALUATION OF LIQUEFACTION POTENTIAL AT B.H.18

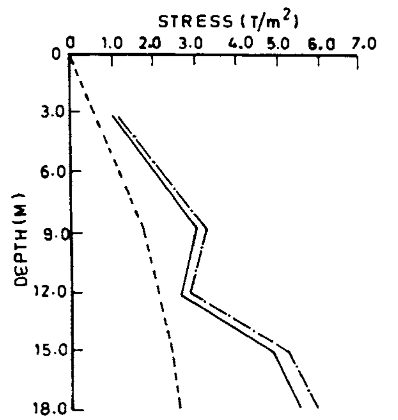

FIG.6_EVALUATION OF LIQUEFACTION POTENTIAL AT B.H.22

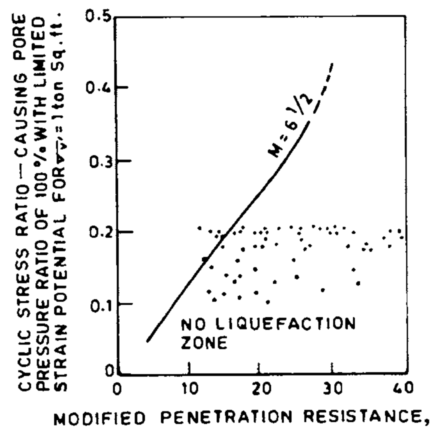

FIG.7_EVALUATION OF LIQUEFACTION POTENTIAL AT VARIOUS DEPTHS OF BORE HOLES OF POLAVARAM DAM SITE.

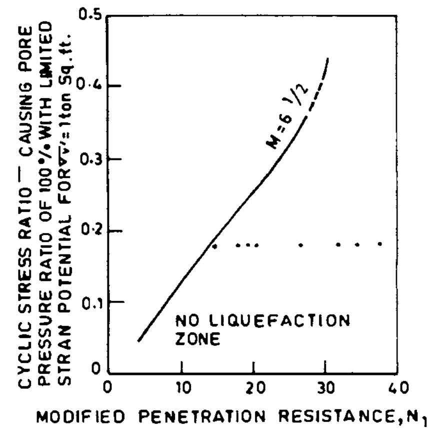

FIG.8_EVALUATION OF LIQUEFACTION POTENTIAL AT DEPTH 10.9M

is 6. The critical values of N calculated according to the given relation show that at two locations only N_{crit} is lower than N at the site. These corresponds to borehole No.16, 18 at 15m depth. Thus out of 64 N values considered the N_{crit} is close to or more than N at two locations only.

Seed et al. (1983) considered the N counts to be a better representative of the dynamic shear strength of the soil at site in comparison to that obtained by laboratory testing. They have collected a vast amount of data for different earthquakes corresponding to liquefiability or otherwise of the soil. Finally a plot for strength ratio vs Modified Penetration Resistance N_1 has been given. Acordingly for the Polavaram Dam site, the modified N values, have been calculated from the SPT values. For the cyclic stress ratio causing pore pressure ratio of 100 percent with limited strain potential for σ'_v = 1 Ton/sq.ft. vs these modified N values have been located as shown in Fig. 7, 8. As the plot given by Seed et al. corresponds to 1 ton/sq.ft. i.e. 10.9 t/m^2 vertical stress level i.e. for depth of 10.9 m, Fig. 8 gives the location of the points corresponding 10.9m depth in each borehole. It is seen that all the points lie in non-liquefiable zone. In Fig. 7, the points have been located irrespective of consideration of depth and it is seen that only 7 points out of 82 lie within the liquefiable zone.

Japanese State of Art (1986) gives an approach for evaluating liquefiability of a site simillar to that given by Seed and Idriss. The maximum acceleration level for a site is determined by the relation which takes into account epicentral distance and earthquake magnitude. For the site it works out to be 0.0576g. The earthquake load in the soil element induced by a seismic motion is determined taking into account the maximum acceleration level, and a reduction factor which varies with depth. The values of insitu resistance of the soil element to earthquake loading which are calculated show that out of 82 locations only at one location the liquefaction resistant factor is less than unity, thus indicating that the liquefaction is likely at this location only. However, this approach, like the other above considered emperical approaches, do not consider the effect of the loading of the site due to the construction of the earth dam.

EXPERIMENTAL STUDY

Results of the emperical approaches discussed above indicate that the site would not liquefy. But none of these approaches bring out the magnitude of reduction in shear strength of the soil during the earthquake. Therefore, in order to verify the emperical results and to assess the likely reduction in shear strength, experiments were conducted on the shake table, having a water tight tank of size 105 cm x 60 cm x 39 cm mounted on it. The sides of the tank are made up of rigid mild steel frame with 5mm thick steel panels. The soil samples have been prepared in this tank for carrying out the tests. The pore water pressure is measured with the help of a stand pipe piezometer. The number of sinusoidal cycles with a given frequency, are imparted to this tank with the help of motor and crank mechanism.

A series of experiments have been conducted in the laboratory. In the first set, 12 tests were conducted to study the effect of acceleration level on development of pore water pressure. In the second and third sets of tests; 22 (16 + 6) tests have been done to observe the effect of frequency on liquefaction potential at different acceleration levels. A 4th set of experiments with 16 number of tests was conducted to see the effect of relative density on liquefaction potential at different acceleration levels.

The design accelerogram as seen in, Fig.3, has a single peak of maximum accelration 0.16g. At 65 percent acceleration level i.e. at 0.10g, the number of equivalent cycles worked out to be 30. At 50 percent acceleration level i.e. at 0.08g the number of equivalent cycles worked out to be 83. Also for 75 percent acceleration level i.e. 0.12g the equivalent number of cycles is 19. Therefore, first set of experiments was conducted at shaking acceleration levels 0.08g, 0.10g and 0.12g with corresponding number of cycles for frequency 5 cps. The relative density at the site is 43 percent. An attempt was made to obtain the same relative density of the soil deposit in the shaking table tank. But the relative density was 39.22 percent. As results are expected to be on conservative side with this relative density, the further attempt to match the relative density was not done. The results of the first set of experiments are given in Fig.9. The experiments have been conducted with and without dead weight surcharge as indicated. It is seen from these plots that the maximum pore presure is obtained when there is no dead weight surcharge. The values of pore pressure and effective stress ratio (liquefaction potential) corresponding to acceleration levels 0.08g, 0.10g and 0.12g are 16.09 percent, 27.01 percent and 27.97 percent respectively. It is thus clearly seen that even at 0.12g acceleration level with the application of corresponding number of cycles, the maximum pore water pressure developed would be of the order of 1/3 of the effective soil pressure. The experimental results confirm the results of the various emperical approaches and also give the idea of the maximum pore water pressure that can be developed at the site.

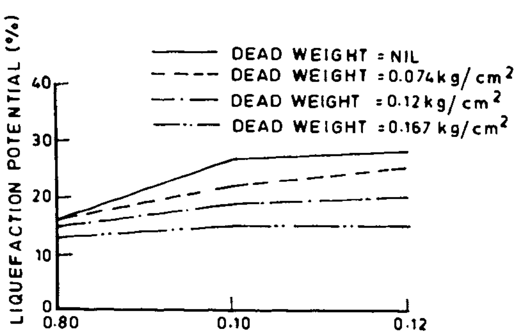

FIG.9 _ LIQUEFACTION POTENTIAL VS ACCELERATION

The second set of experiments, as mentioned above, was conducted to see the effect of frequency of vibrations for most of the acceleration levels (0.05g, 0.08g, 0.10g, and above). The frequency as worked out from the accelerogram is between 4 and 5 cycles per second. The experiments were conducted keeping the total number of cycles as 30 with these acceleration levels at soil deposit relative density 36.27 percent and frequency 8 cps, 5 cps and 3 cps. The records of these experiments are given in Fig. 10(a). A third set of experiment was also conducted in a similar way to see the effect of frequency of vibration at acceleration levels 0.08g, 0.10g and 0.12g with corresponding equivalent number of cycles at relative density 39.22 percent and frequency 8 cps and 3 cps. The results of these experiments are given in Fig. 10(b). It is seen from these plots (Fig.10) that the liquefaction potential at 3 cps is slightly higher than at frequency 5 cps, and at frequency 8 cps it is about 20 percent lower than in the other two.

The relative density in the first set of experiments, which were the main experiments for evaluation of liquefaction potential of the site, was 39.22 percent. The relative density at the site is of the order of 43 percent. Therefore, in order to have an idea of the conservatism of the results, fourth set of experiment was conducted with relative densities corresponding to 39.22 percent and higher values. The results of these experiments are shown in Fig. 11. It is seen from these plots that there is sharp reduction in liquefaction potential with increase in relative density. Estimating from the slopes of the plots it can be interpreted that liquefaction potential observed at 0.08g, 0.10g and 0.12g are about 5 percent, 8 percent and 8 percent on higher side.

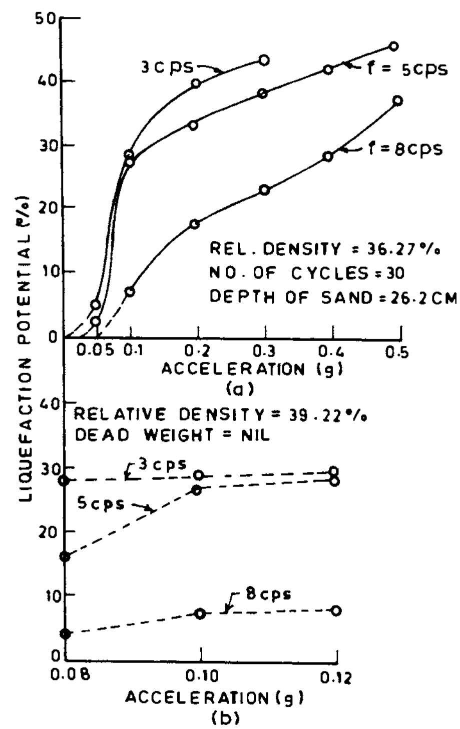

FIG. 10 _ EFFECT OF FREQUENCY ON LIQUEFACTION POTENTIAL

CONCLUSIONS

The emperical approaches indicate that the site as a whole would not liquefy and there could only be local liquefaction at some spots where the N count is very low. The Seed and Idriss approach indicates that out of 82 locations considered liquefaction is likely only at 6 locations. As per Chinese building code, out of 64 locations considered only at two locations the liquefaction is likely. According to the approach given by Seed et al. based on site data, it is seen that only 7 points out of 82 points lie within the liquefiable zones. Application of Japanese State of Art, indicates that out of all 82 locations only at one location the liquefaction potential resistance factor is less than unity.

The experiments conducted also indicate that the maximum pore water pressure development corresponding to 0.10g acceleration (65 percent of the maximum expected acceleration) could be only 31 percent of the effective over burden pressure. The development of this excess pore water pressure will not liquefy the soil but bring down the shear strength of the soil. The plots between acceleration levels and reduction in the value of angle of internal friction of the soil are given in Fig.12. For the average acceleration of 0.10g at site the angle of internal friction may be taken as reduced from 30° to 24°.

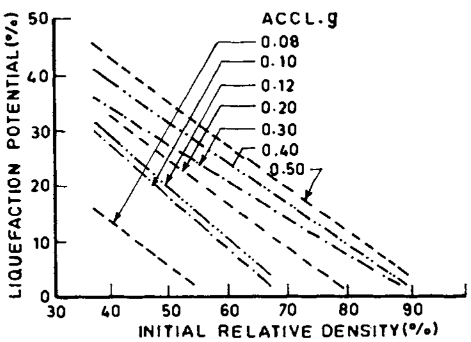

FIG.11_EFFECT OF INITIAL RELATIVE DENSITY ON LIQUEFACTION POTENTIAL AT FREQUENCY 5cps

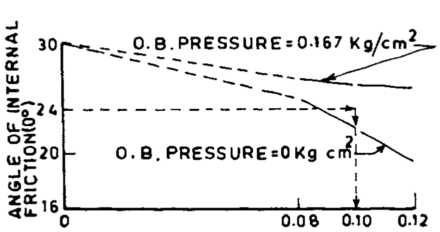

FIG.12_ANGLE OF INTERNAL FRICTION VS ACCELERATION

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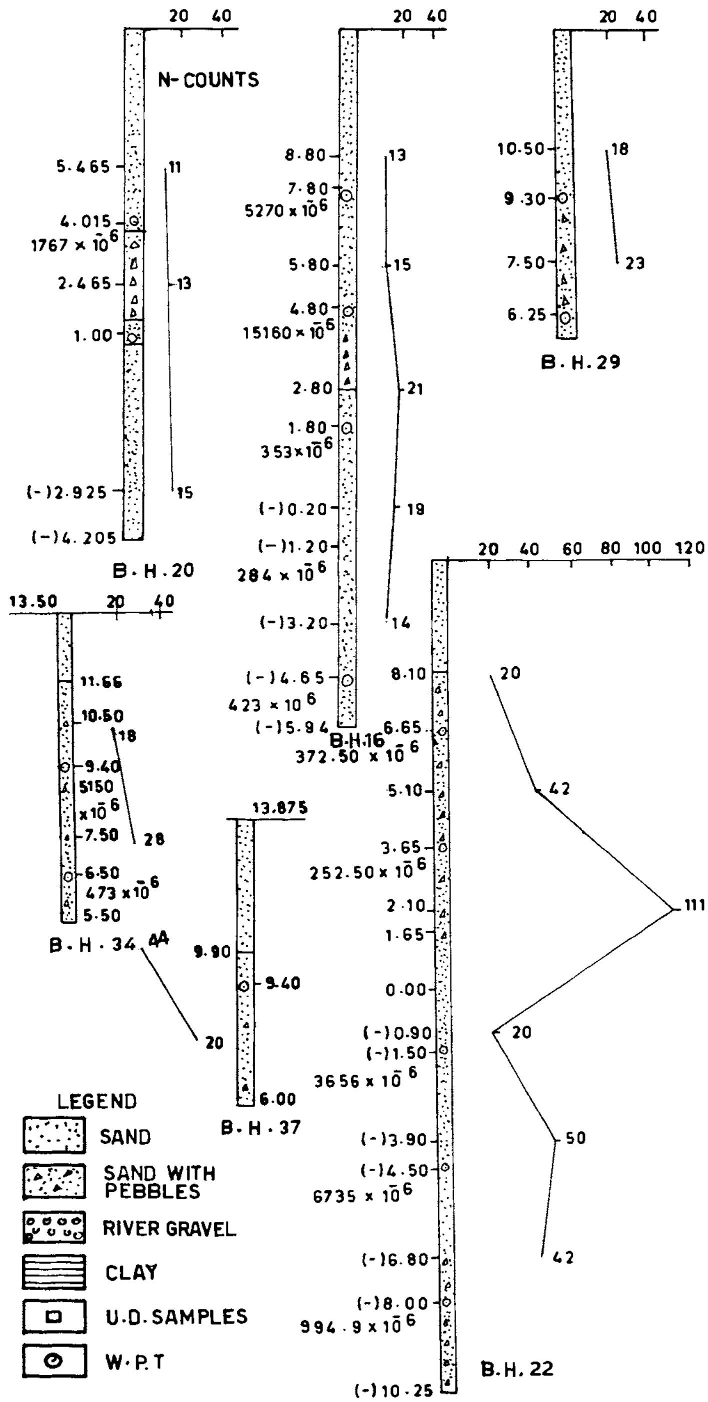
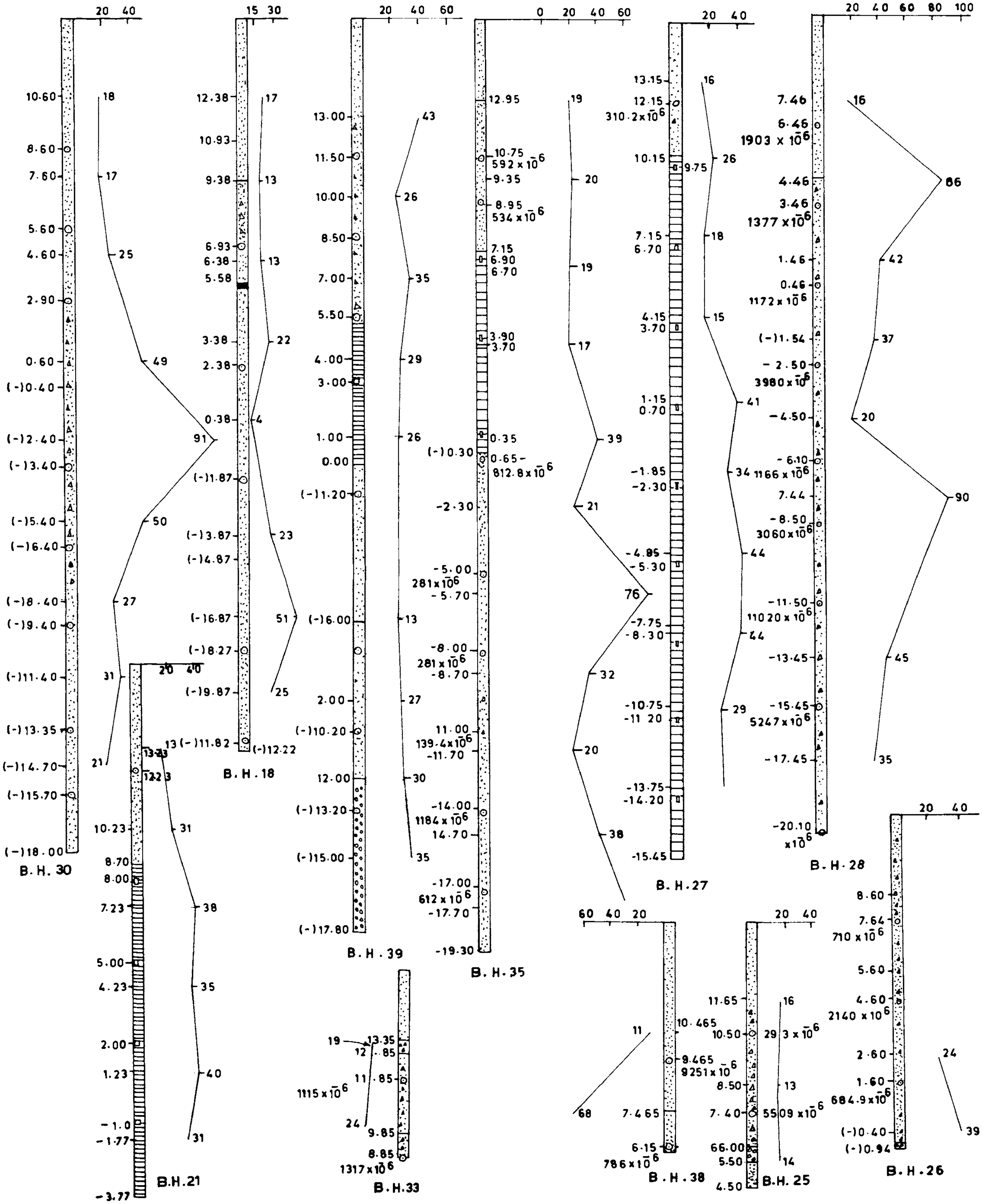
APPENDIX_ BORE HOLE DETAILS

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