EVALUATION OF LIQUEFACTION POTENTIAL OF RIVER ALLUVIUM AT DIAMER BASHA DAM SITE

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ABSTRACT: Liquefaction is rapid loss of shear strength in loose, saturated granular soils during an earthquake. This phenomenon is one of the most common causes of failure of civil engineering structures including dams and other water retaining structures. Keeping in view its devastating effects, the liquefaction potential of Diamer Basha dam site has been evaluated based on site geology and seismicity of the area. Different in-situ as well as laboratory testing data like standard penetration test (SPT), Atterberg's limits and Grain size analysis is collected and analyzed with their application to geotechnical earthquake engineering. The latest and evolving methods i.e. Seed and Idriss, Tsuchida´s liquefaction susceptibility method and Wang (1979) method have been employed to evaluate the liquefaction potential of granular and clayey silt deposits. The study revealed that the granular deposits of the river alluvium are prone to liquefaction as evaluated both by Seed & Idriss and Tsuchida´s liquefaction susceptibility method. However, the clayey silt river deposits according to Wang (1979) and Tsuchida methods are not susceptible to liquefaction.

Key words: Liquefaction, Earthquake engineering, Granular deposits, Liquefaction potential evaluation methods.

INTRODUCTION

The proposed Diamer Basha dam is located on river Indus 315 km upstream of the existing Tarbela dam and about 165 km below the town of Gilgit and 40 km downstream of Chilas. The project is located between longitude 73^o to 75^o E and latitude 35^o to 36^o N and is bounded by the districts of Hazara, Diamer, Gilgit and Chitral, lying respectively to the south, east, north and west. This region is seismically very active due to its position near collisional boundary of the Indian and Asian tectonic plates. The seismic hazard evaluation by NEAC (2004), the project consultants (NESPAK-ACE Joint Venture), during feasibility studies revealed that the Maximum Credible Earthquake (MCE) at the site was of magnitude 7.5 with Peak Ground Acceleration (PGA) of 0.40g. This paper is aimed to assess the liquefaction potential of river alluvium (as discussed in the following section) at Diamer Basha dam site considering the above mentioned values of seismic parameters as input in the liquefaction analysis.

Soil conditions at dam site: Seven boreholes were drilled in the river bed along the dam axis to investigate the engineering and geological properties of alluvial deposits present in the river valley. These boreholes were drilled during feasibility studies by the project consultants up to depths varying from 40 m to 65 m below the river bed level. Standard Penetration Tests were performed in these boreholes following ASTM D 1586 (2004). The variation SPT N blows versus depth is represented in Fig. 1. The subsurface profile along the dam axis is shown in Fig. 2.

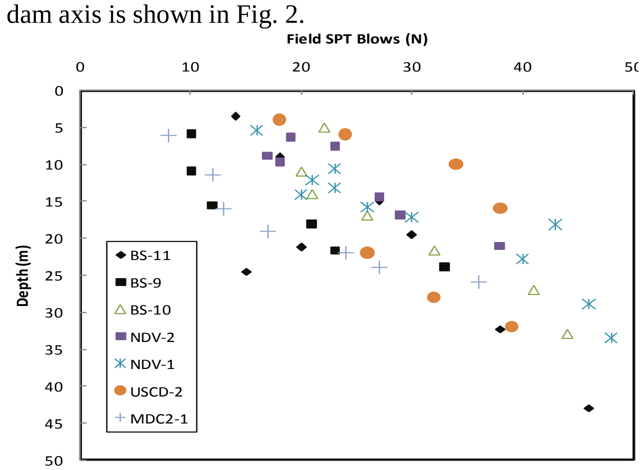

Fig. 1 SPT Profile of River Alluvium at Diamer Basha Dam.

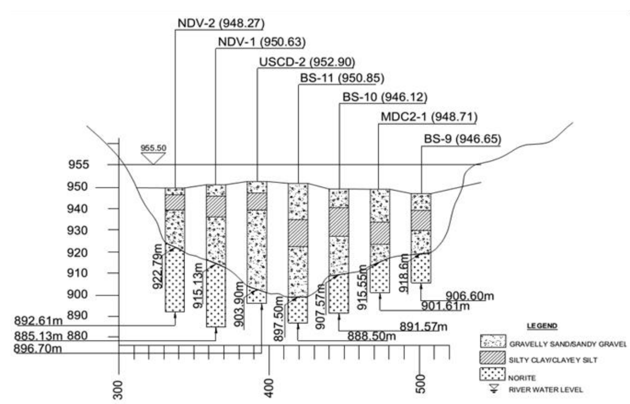

Fig. 2 Subsurface Exploration Details in River Alluvium

The analysis of borehole data indicates that the thickness of the river alluvium is about 60 m. The cross-section of river alluvium shows that granular deposits are the main and dominant deposits in the river bed whereas clayey silt deposit is a minor unit in the river alluvium. The alluvium is divided into three layers (i.e. layer-1, layer-2 and layer-3) for analysis. The top layer of the alluvium consists of sandy gravels/ gravelly sand having a thickness of about 15 m (Layer-1). Afterwards there is a layer of clayey silt deposit having a thickness of 10 m (Layer-2) followed by gravelly sand/ sandy gravel with its thickness varying between 20–30 m (Layer-3). The generalized subsurface profile showing three distinct soil layers is presented in Fig. 3.

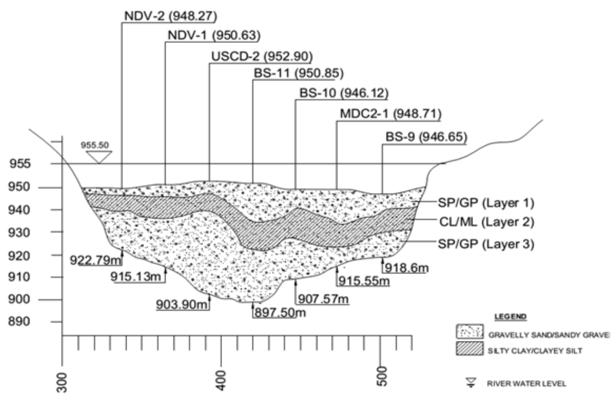

Fig. 3 Cross-Section Showing Detail of River Alluvium

Grain size analysis was carried out on the material obtained from these layers. The average gradation curves for Layer-1, Layer-2 and Layer-3 are presented in Fig. 4. It is clear from Fig.4 that Layer-1 and Layer-3 are sandy/gravelly deposit where as the Layer-2 consists of silt/clay. Based on the grain size analysis, it is evident that the amount of fines present in the granular deposits is less than 5%. Average bulk unit weight of samples procured from the boreholes fall in the range of 19.42 ~ 20.6 kN/m^3.

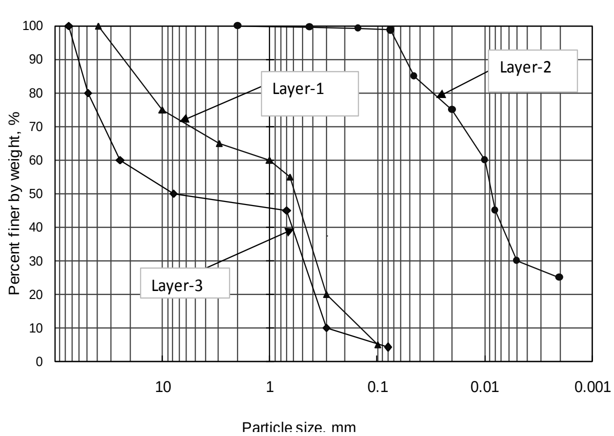

Fig. 4 Average gradation curves for Layer-1, 2 and 3

MATERIALS AND METHOD

There are many methods to assess the liquefaction potential of granular deposit, however in this research the following three methods have been used.

- Seed and Idriss Method
- Tsuchida Liquefaction Susceptibility Method
- Wang 1979 Criteria

RESULTS & DISCUSSION

Seed and Idriss method: This method of liquefaction analysis proposed by Seed and Idriss (1971) is also named as simplified procedure and is the most commonly used method to evaluate liquefaction potential of a soil using standard penetration test (SPT). In liquefaction analysis the standard penetration test (SPT) N_{60} value is corrected for the overburden soil pressure. When a correction is applied to N_{60} values to account for the vertical effective stress then this value are referred as $(N_1)_{60}$ values and is given by Eq. 1

$$(N_1)_{60} = C_N N_{60} = [\frac{100}{\sigma'_v}]^{0.5} N60 \quad (1)$$

Where $(N_1)_{60} =$ Standard Penetration test value (SPT N-value) corrected for both field testing procedure & overburden pressure.

C_N = Correction factor to account for the overburden pressure and is approximately equal to $\left[\frac{100}{\sigma'_{vo}}\right]^{0.5}$ where σ'_{vo} is the vertical effective stress in kPa.

N_{60} = SPT N-value corrected for field testing procedure and is calculated by using Eq. 2

$$N_{60} = C_b C_r C_s N \left[\frac{E_m}{60}\right] \quad (2)$$

C_b, C_r, C_s, E_m are the corrections which are made according to Table 1

The value of cyclic stress ratio CSR can be estimated from a simplified procedure as follows

$$\text{CSR}=\frac{\tau_{cyc}}{\sigma'_{vo}} = 0.65 r_d \left(\frac{\sigma_{vo}}{\sigma'_{vo}}\right)\left(\frac{a_{\max}}{g}\right) \quad (3)$$

Where a_{max} is the peak ground acceleration, g is the acceleration of gravity, σ_{vo} and σ'_{vo} are the total and effective overburden stress, r_d is the stress reduction factor.

Table-1 Correction to field SPT N-values (Youd & Idriss, 1997)

Factor	Equipment Variables	Value
Borehole diameter	65 – 115 mm	1.00
correction factor C_B	150 mm	1.05
	200 mm	1.15
Rod length	3 – 4 m	0.75
correction factor,	4 – 6 m	0.85
C_R	6 – 10 m	0.95
	> 10 m	1.00
Sampling method	Standard Sampler	1.00
correction factor,	Sampler without liner	1.20
C_S		
Energy Ratio, C_E	Donut Hammer	0.5 – 1.0
	Safety Hammer	0.6 – 1.2
	Automatic – Trip donut type Hammer	0.9 – 1.6

Subsequent refinements have been made in this method over the years. Seed et al. (1985) compared the corrected SPT resistance and cyclic stress ratio for clean sand and silty sand sites at which liquefaction was or was not observed in the earthquake of magnitude 7.5. Ishihara (1993) proposed that for CPT based liquefaction for silty sand (>5% fines) measured tip resistance is increased by a certain increment based on % age of fine present. Corrections for earthquake of different magnitude as well as for high confining pressure and sloping ground are made in this method.

$$CRR_{field} = CRR_{7.5} \times MSF \times k_{\sigma} \times k_{\alpha} \quad (4)$$

Where K_σ is correction for confining pressure and K_α is correction for initial static shear stress, MSF is magnitude scaling factor (for earthquakes other than 7.5). Seismic parameters are selected based on seismic hazard analyses by NEAC consultants (2004) and in accordance with recommendations of International Commission on Large Dams (ICOLD,1989).

This method compares the cyclic resistance ratio (CRR) of in-situ soil with the earthquake induced cyclic stress ratio (CSR) for a specified design earthquake. If the CSR caused by the earthquake is greater than the CRR of the in-situ soil, then liquefaction could occur during the earthquake and vice versa. The FOS against liquefaction is defined as follows:

$$\text{FOS} = \frac{CRR}{CSR} \quad (5)$$

The higher the FOS, the more is resistance of soil to liquefaction. Based on available SPT data, peak ground acceleration and maximum credible earthquake at dam site, the value of CRR and CSR are calculated at Diamer Basha dam site. The calculations are carried out by Seed and Idriss method and the factor of safety (FOS) versus depth for each borehole is presented graphically from Fig. 5 to Fig. 11. An average factor of safety for all the borehole is also calculated and is presented in Fig. 12. It can be inferred from the Fig. 12 that value of FOS is in the range of 0.29 ~ 0.69. The value of FOS is independent of depth; however, FOS increases with the increase in N values.

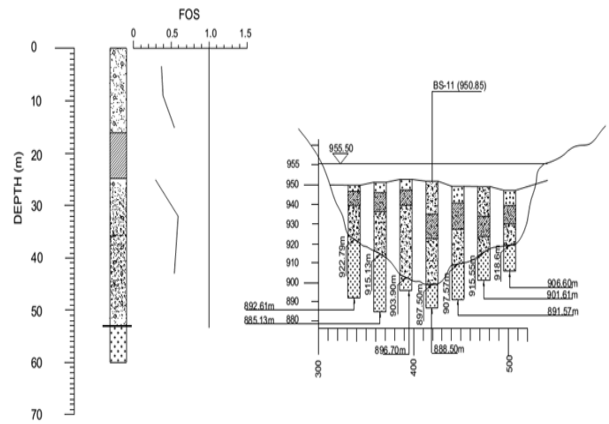

Fig. 5 FOS for Borehole No. BS-11

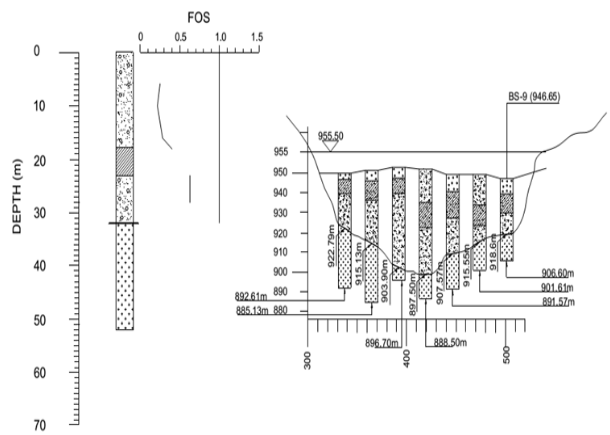

Fig. 6 FOS for Borehole No. BS-9

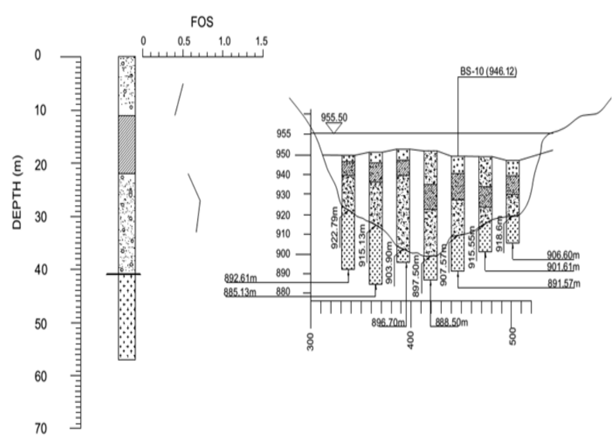

Fig. 7. FOS for Borehole No. BS-10

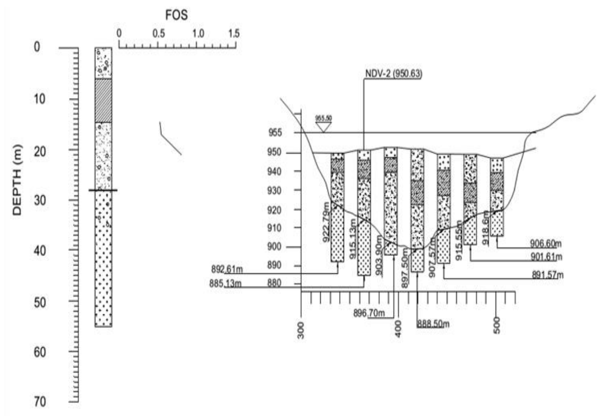

Fig. 8. FOS for Borehole No. NDV-2

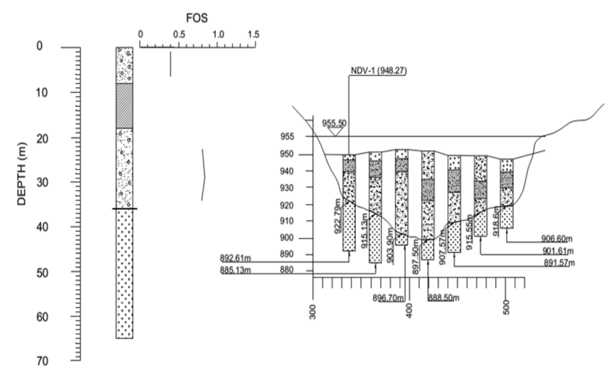

Fig. 9. FOS for Borehole No. NDV-1.

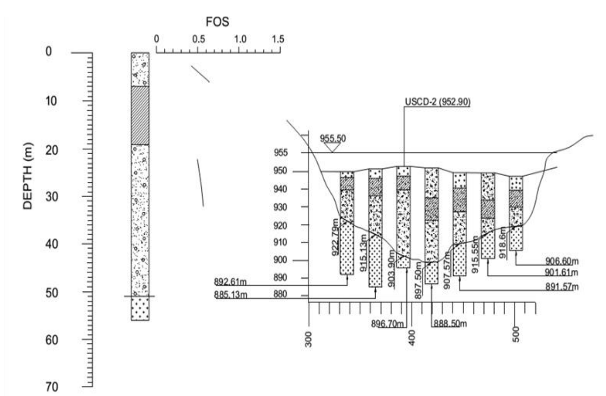

Fig. 10. FOS for Borehole No USCD-2.

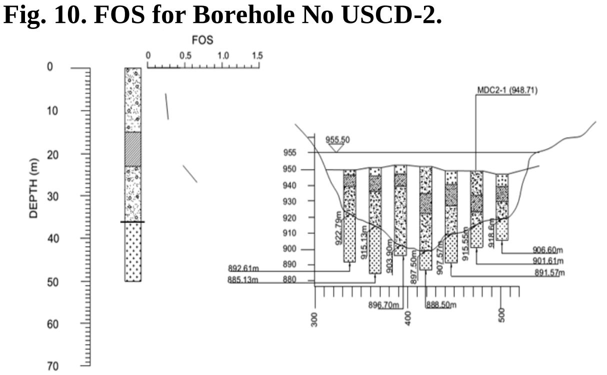

Fig. 11. FOS for Borehole No. MDC2-1

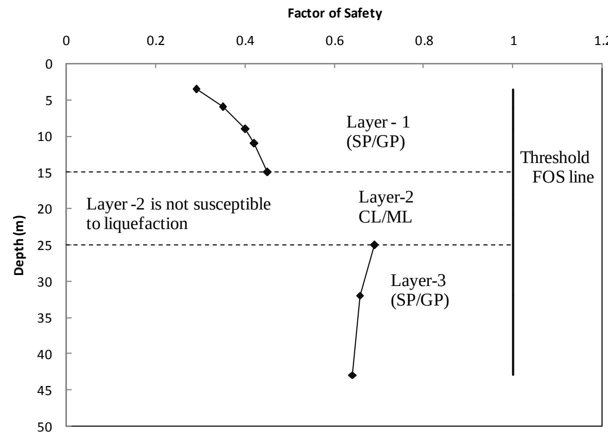

Fig. 12 Average FOS versus Depth

Tsuchida's Liquefaction Susceptibility Method: The type of soil most susceptible to liquefaction is one in which the resistance to deformation is mobilized by friction between particles. If other factors such as grain shape, relative density are equal, the frictional resistance of cohesion less soil decreases as the grain size of soil becomes smaller. Tsuchida (1971) proposed boundaries for the grain size distribution of soils susceptible to liquefaction. These boundaries were the result of sieve analyses on soils that did or did not liquefy during past earthquakes. The area within the two inner curves shown in the Fig. 13 represents sands and silty sands, the soil with the lowest resistance to liquefaction.

Soils with the gradation curve falling in the zones between the outer and inner curves are the potentially liquefiable soils. Soils with a higher percentage of gravel tend to mobilize higher strength during shearing and dissipate excess pore pressures more rapidly than sands. However, there are case histories indicating that liquefaction has occurred in loose gravelly soils during severe ground shaking or when the gravel layer is confined by an impervious layer (Ishihara, 1985).

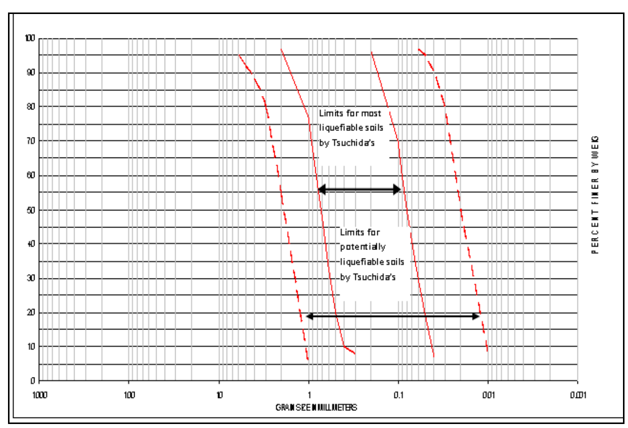

Fig. 13 Tsuchida's Liquefaction susceptibility Curves

Results of grain size distribution obtained from subsoil investigation are superimposed on Tsuchida curves as shown in Fig. 14 through 17. The result indicates that grain size analyses of river alluvium for Layer-1 and Layer-3 falls within limits of most liquefiable soils whereas clayey silt deposits (Layer-2) has higher percentage finer by weight in the grain size range of 75mm to 0.002 mm. Thus these deposits are out of the range of potentially liquefiable soils.

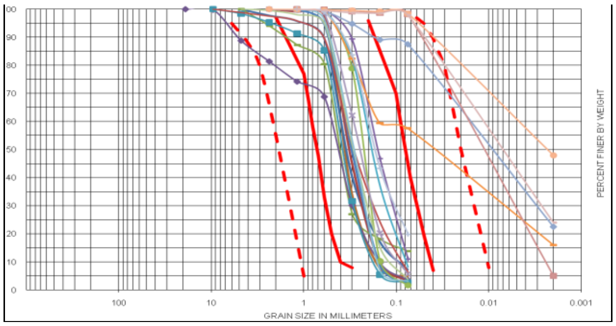

Fig. 14 Tsuchida's Liquefaction Susceptibility for Borehole No.9

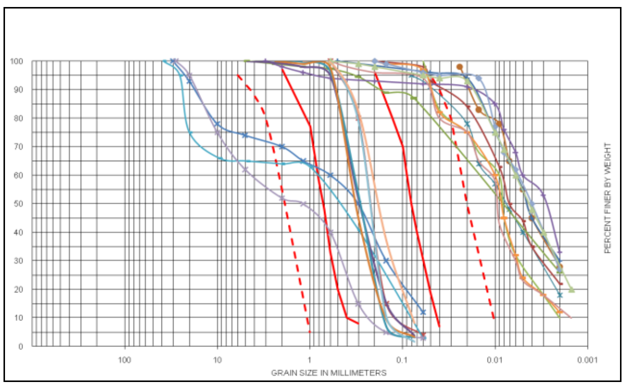

Fig. 15 Tsuchida's Liquefaction Susceptibility for Borehole No.10

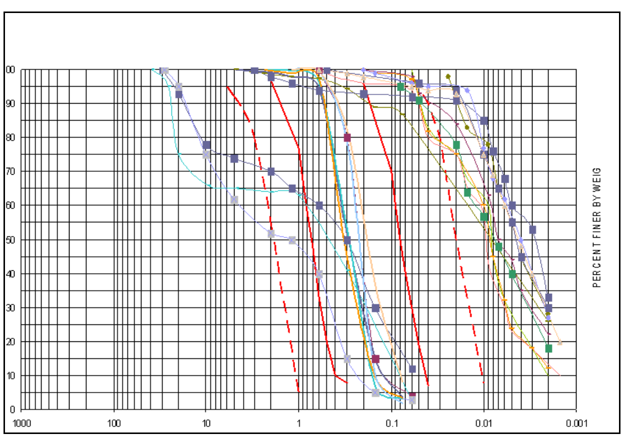

Fig. 16 Tsuchida's Liquefaction Susceptibility for Borehole No. NDV-1, NDV-2, USCD-2, MDC2-1

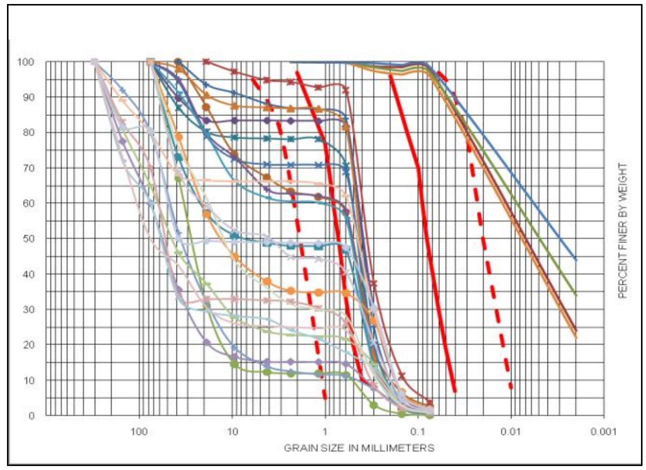

Fig. 17 Tsuchida's Liquefaction Susceptibility for Borehole No.11

Wang 1979 Criteria: Earlier studies on liquefaction were devoted to sands and it was considered that fine grained soils are non-liquefiable. However, the observations following the Haicheng (1975) and Tangshan (1976) earthquakes indicate that many cohesive soils had also liquefied. Liquefaction of non-plastic silts has been observed in the laboratory and in the field, indicating that plasticity characteristics rather than grain size alone influence the liquefaction susceptibility of fine grained soils. Coarse silts with bulky particle shape, which are non-plastic and cohesionless are fully susceptible to liquefaction. Finer silts with flaky plate like particles generally exhibit sufficient cohesion to inhibit liquefaction. Clays remain non-susceptible to liquefaction, although sensitive clays can behave similar to that of the liquefied soils.

Wang (1979) proposed the following four criteria which were subsequently adopted by Seed and Idriss (1982), the satisfaction of all of which would indicate that cohesive soils are susceptible to liquefaction:

a) Fraction finer than 0.005 mm ≤ 15%
b) Liquid Limit (LL) ≤ 35%
c) Natural water content ≥0.90 LL
d) Liquidity Index ≤ 0.75

Hence only those clays or silts that have a low plasticity (i.e., CL-ML or ML) and high water content will be susceptible to liquefaction. However, even if cohesive soil does not liquefy, there can still be the possibility of a significant undrained shear strength loss due to seismic shaking (Day, 2004). The laboratory test results of the samples obtained from the fine grained soil deposits are compared to the above mentioned method in Table 2. From this comparison; it is evident that none of the samples satisfies all of the four of above mentioned criteria.

Table-2 Wang Criteria (1979) for Soils Susceptible to Liquefaction.

Borehole No.	Depth (m)	% Finer than 0.005 ≤15%	Liquid Limit ≤ 35%	Water content ≥ 0.9LL	L.I. ≤ 0.75 $LI = \frac{w-PL}{PI}$
USCD-2	6.60-7.10	55	39	27.2<35.1	-0.07
USCD-2	7.10-8.60	43	36	28.4<32.4	0.37
USCD-2	13.20-13.80	45	37	28.6<33.3	0.07
USCD-2	15.39-16.00	40	32	23.2<28.8	0.02
BS-11	16.8-19.8	43	32.1	21.9<28.9	-0.10
BS-11	19.8-21.3	50	30	26.4<27	0.46
BS-11	21.6-22.9	40	35.4	32.6>31.9	0.76
BS-11	22.9-24.4	57	26.5	23.2<23.8	0.49
BS-10	9.13-10.7	20	28.2	18.6<25.4	-0.75
BS-10	10.7-11.2	48	36	31.3<32.4	0.60
BS-10	12.2-13.7	52	38.5	26.1<34.6	0.14
BS-10	13.7-14.3	43	32	26.7<28.8	0.39
BS-10	13.7-15.2	40	36.8	31.4<33.1	0.51
BS-10	15.2-16.8	44	36.3	23.1<32.7	0.02
BS-10	16.8-17.1	48	35.3	32.7>31.8	0.76
BS-10	16.8-18.3	40	32.5	22.3<29.3	0.11
BS-10	18.3-19.8	43	31	26.8<27.9	0.50
BS-10	19.8-21.3	40	35.5	29<31.95	0.42
BS-10	21.7-22.6	40	31	31.2>27.9	1.03
MDC2-1	21.08-21.69	35	33	25.9<29.7	0.29
MDC2-1	23.2-23.8	46	37	29.1<29.7	0.12
BS-9	18.3-18.8	40	29	26.2<26.9	0.55
BS-9	15.2-15.7	28	26	31.6>23.4	2.1
BS-9	21.3-21.8	30	28	27.6>25.2	0.92
NDV-1	8.61-10.16	65	36	31<32.4	0.43
NDV-1	12.16-12.61	30	31	26.6<27.9	0.10
NDV-2	7.66-8.11	50	28	-	-
NDV-2	8.84-9.30	48	30	-	-
NDV-2	9.88-10.33	47	28	-	-

Does not Satisfy the Criteria Satisfies the Criteria

None of the sample satisfies all four criteria so fine grained soils (Layer-2) present in the river alluvium are not susceptible to liquefaction.

Conclusions: Following conclusions have been drawn from this study:

- Factor of safety (FOS) in layer-1 is found to be 0.45 (average value) whereas in layer-3, the average value of FOS is determined as 0.60 which are less than 1. Hence by Seed and Idriss method, the granular river alluvium is prone to liquefaction.
- The grain size analysis of river sand falls within the limit of most liquefiable soils as defined by the Tsuchida's liquefaction susceptibility curves. These results also support the conclusions drawn from the application of Seed and Idriss method.
- The results of grain size analysis of clayey silt deposits (Layer-2) indicate that these samples have higher percentage of fines and thus fall out of range of potentially liquefiable soils as per Tsuchida's method.
- The results of classification tests obtained from the clayey silt deposits of Layer-2 have been compared to the limits provided in Wang, 1979 method. It is evident (Table 2) that none of samples fulfill all four requirements of the Wang criteria. Hence these deposits (Layer-2) are not considered susceptible to liquefaction.

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