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Liquefaction Susceptibility Evaluation of Hyderabad Soil

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ABSTRACT: *In this research paper, liquefaction susceptibility evaluation of the Hyderabad (in the vicinity of Raees Karan Khan Shoro Village) soil has been attempted using the simple assessment criteria (i.e. Chinese, Modified Chinese, and Bray's Criteria). The study involves assessment of earthquake-induced liquefaction potential using simple assessment criteria, wherein ordinary engineering properties of soil were utilised following the laboratory tests to ascertain the liquefaction susceptibility of the subject soil. The research thus conducted,showed that the subject soil was significantly liquefaction prone and will require soil improvement measures for it to be utilised as a ground for any engineering construction project.*

Keywords:- *Hyderabad, Liquefaction, Liquefaction Evaluation, Liquefaction Potential, Liquefaction Susceptibility, Chinese Criteria, Modified Chinese Criteria, Bray's Criteria.*

I. INTRODUCTION

Liquefaction is a type of geotechnical ground failure, which can be described as the loss of shear strength that occurs primarily in saturated cohesionless soils due to the increased pore water pressure caused by static (monotonic) or dynamic (i.e. shock, cyclic) loading conditions. The most primitive form of the dynamic (cyclic) loading is the seismic (earthquake) activity. The extensive earthquake-induced cyclic and or shock loading is caused by the aggressive shaking of the earth.This results in an instantaneous and abrupt increase in the pore water pressure of the saturated soil mass, leaving no time for the pore pressure to dissipate which results in significant reduction of the effective stress thus causing the soil mass to liquefy (i.e. behave as a liquid).

The liquefaction potential of the soil can be assessedand evaluated using various methodologies, among them the simple assessment criteria are an attempt to make the identification of the liquefiable soils quick and easy. The simple assessment criteria involve correlating various simple (ordinary) engineering properties of the soil with each other in an attempt to identify and recognise the soil as "liquefaction susceptible", "potentially liquefiable", or "non-liquefiable". Chinese, Modified Chinese, and Bray's Criteria are some of the widely used simple assessment criteria to check the soil for its liquefaction potential.

II. LIQUEFACTION – AN OVERVIEW

Saturated loose sandy (cohesionless) soilswhen introduced to cyclic loadingconditions developed by the upward propagation of shear waves from the low lying earthquake jolted hard strata, tend to settle and reconfigure the subject soil into a denser packing. However, the brief time duration of the cyclic stress application does not allow the subject soil to dissipate the pore water, as a result the volumetric contraction is hindered and the pore water pressure progressively builds up until it becomes equal to the total stress, thus lowering the effective stress to the ultimate zero. In such a situation the soil losses its shear strength / stiffness, this state is called liquefaction. Earthquake associated liquefaction has long been the cause of various structural failures. The phenomenon has been known to scientists and engineers from as early as 1918 when the first Calaveras Dam failed, wherein the liquefaction phenomenon was observed and the significance of pore pressure and effective stresses was realised for the first time [1].

However, it was not until 1964 that the importance of the subject was fully realised. The Alaskan (US) Earthquake, and Niigata (Japan) Earthquake of 1964 causedsevere damage to civil engineering structures and infrastructuremostly attributable to the liquefaction associated ground failures. Liquefaction associated damaged ranged from lateral spreading to sand boiling. Numerous cases of lateral spreading, ground oscillation, slope failures, foundation settlement, and buoyant rise of structures were reported during both the events[2]. The phenomenon required some serious attention from Geotechnical Engineers. Attempts were made by several of the renowned geotechnical scientists to help understand the phenomenon and the geotechnical failures associated with it. Casagrande and Castro made a very early attempt to define and explain the mechanism of statically induced liquefaction in soils. Later, the phenomenon was studied by Seed and his associates for the cyclic loading conditions. All these attempts were aimed to help understand the liquefaction mechanism for various soil types under different loading conditions. This resulted in the emergence of several different ideas regarding the parameters necessary for the proper identification of the liquefaction susceptible soils. However, of all the proposed ideas, the simple assessment criteria are a nascent rule to the understanding of the liquefaction phenomenon, as they require no any time consuming field tests, and are literally very easy to be utilised.

III. LIQUEFACTION SUSCEPTIBILITY AND THE SIMPLE ASSESSMENT CRITERIA

The liquefaction phenomenon has perplexed many scientists and engineers for quite some time, as it is often very difficult and costly to identify the liquefaction susceptible soils. Several different methods have been used by different researchers to identify the liquefaction susceptible soils. Most of them required the soil to be tested for its shear strength parameters using cyclic shear tests. However, the cyclic shear apparatus are costly and are not readily available all the time. It was thus very necessary to correlate the liquefaction potential of the soil to the ordinary engineering properties of the soil for the process of identification to be made easy. Following are some simple soil assessment criteria for the evaluation of the earthquake-induced liquefaction potential.

Liquefaction Evaluation using Particle Sizes

In the early years of the liquefaction study, it was a general understanding that the soils that were most prone to liquefaction associated damage had smaller grain size andless relative densityfor which they could not mobilise sufficient frictional resistance if and when subjected to liquefaction causing load conditions.[3]Tsuchidahaving studied the particle size distribution of a wide range of liquefied and non-liquefied alluvial and diluvial soils from past earthquakes proposed grain size curves separating liquefiable and non-liquefiable soils, as shown below.

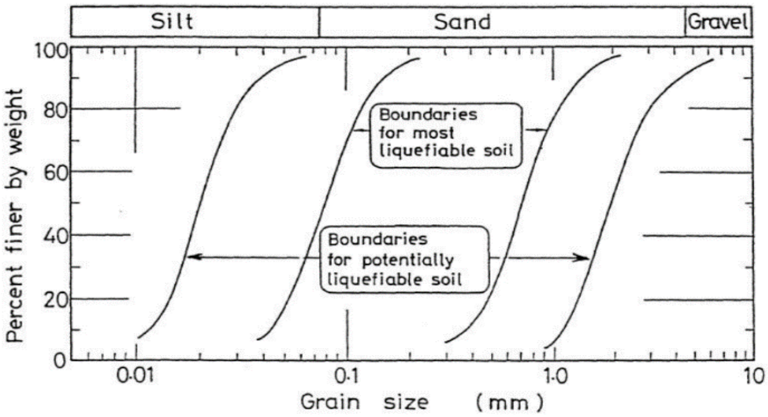


Fig 1 - Gradation curves marking the zones for liquefiable and potentially liquefiable soils. [3]

Soils falling within the boundaries of the two inner curves are most susceptible to liquefaction, whilst the soils falling with in the confinements of the extreme curves but having managed to stay out of the inner curves were moderately liquefaction susceptible. It was generally understood that the soils having larger particle size tend to mobilise greater shearing resistance and relieve themselves of the pore pressure much quicker than soils with smaller particle size if and when subjected to static or dynamic loading conditions.

Later[4], [5]Ishihara and Andrus concluded in the light of precedents from the earthquake case histories that the liquefaction had indeed occurred in soils with coarser grain size during the event of a seismic activity or when the coarse soil layer was confined within two fine soil layers.

The curves on the far ends of the grain size distribution chart denote the influence of the fines in resisting the reconfiguration of the soil into a denser packing if and when subjected to seismically induced shearing. Whilst, the fines with plasticity present in the soil, hinder the rearrangement of the soil particles during the seismically induced shearing, thus making it difficult for the soil to liquefy under such loading conditions. [4]Ishihara proposed that the low-plasticity fine soils tend to exhibit a similar behaviour as that of the cohesionless soils; it is thus that the soils with low plasticity fines are more prone to liquefaction-associated damage.

Chinese Criteria

[6]Wang researched extensively on the liquefied soilsfrom the 1975 Haicheng (China) Earthquake and 1976 Tangshan (China) Earthquakethataffected the eastern coast of China, and came closer to proposing a liquefaction evaluation criteria for the very first time which was later named as the Chinese Criteria. Wang's study did not include the observational data of the low plasticity soils which made the proposed Chinese Criteria less practical and unreliable. Later, [7]Seed et al.revised the criteria and proposed that a soil susceptible to liquefaction damage must possess or satisfy these following three conditions:

1. The % fines (<0.005mm) must be less than or at most be equal to the 15% of the soil mass.
2. The Liquid Limit (*LL*) of the suspected liquefiable soil must be less than or at most be equal to 35%.
3. And, that its water content (*w*) must not be less than 90% of Liquid Limit (*LL*).

The method was genuinely simple and did not require any laborious laboratory work as it involved assessment of liquefaction potential using ordinary engineering properties of the soil (i.e. water content, % fines, and Liquid Limit). The method is still in practice and is widely used to assess the saturated soils for their liquefaction potential; it is one of the most primitive methods. However, the method may fail to detect a range of potentially liquefiable soils, as it is too conservative and less reliable. It thus became imperative to seek out a method that were to address the problem of properly identifying the liquefaction susceptible soils that werebeing missedor were unable to be detected by the Chinese Criteria.

Table 1 – Chinese Criteria

Potentially Liquefiable Soil	
Fines (< 0.005 mm)	≤ 15%
Liquid Limit (*LL*)	≤ 35%
Natural Water Content (*w*)	≥ (0.9 × *LL*)

Modified Chinese Criteria

The need for a more reliable and a standard assessment criteria for the liquefaction susceptible soils made the researchers [8]to come up with an improved method which they called "Modified Chinese Criteria"to detect the liquefaction susceptible soils using ordinary engineering properties of the soil. The method is a moderate enhancement to the old Chinese Criteria for the proper identification of the liquefaction susceptible soils. The modified Chinese Criteria reduced the % fines particle size to 0.002 mm instead of the old 0.005 mm and that the Liquid Limit (*LL*) were to be found using the Casagrande Type Liquid Limit Apparatus (ASTM D-4318). The method involves the assessment of the soils for the liquefaction potential by correlating the % Fines (% Clay Content < 0.002mm) and the Liquid Limit (*LL*) to ascertain the liquefaction potential of the subject soil. The method is much better than the old Chinese Criteria, but it still requires further study, before it could be considered as a standard for the classification of the liquefiable soils.

Table 2 – Modified Chinese Criteria

		Liquid Limit □ 32%	**Liquid Limit □ 32%**
Clay Content (< 0.002 mm)	**< 10%**	Potentially Liquefiable Soil	Further Study Required for Non Plastic Clay Size Particles
	□ 10%	Further Study Required for Non Plastic Clay Size Particles	Non Liquefiable

Bray's Criteria

The use of grain size has been widely contested since the scale of unreliability of the Chinese Criteria method was determined after the 1989 Loma Prieta (US) Earthquake, 1999 Izmit (Turkey) Earthquake, 1999 Jiji (Taiwan) Earthquake. However, it was generally agreed that the Liquid Limit (*LL*) in combination with the Plasticity Index (*PI*) is a much better indicator of the liquefaction behaviour. [9]Bray et al. proposed a different perspective to the liquefaction potential study. The researcher suggested on using the ratio of water content (*w*)to Liquid Limit(*LL*) correlating it with the Plasticity Index (*PI*) of the soil to ascertain the liquefaction potential of the subject soil, the method offered better results than both the Chinese and Modified Chinese criteria.

Table 3 – Bray's Criteria

	w/LL **□ 0.85**	***w/LL*>** **0.8**
PI **□ 12**	Liquefaction Susceptible	-
12 <*PI* **□ 20**	-	Moderately Liquefaction Susceptible
***PI*>** **20**	Soil too Clayey for Liquefaction to occur, however ground deformations are inevitable during cyclic loading	

Relative Density

One more factor that influences the liquefaction potential is relative density (D_r). It is evident from the earthquake case histories and the laboratory test results thatfor a given soil, initial voids ratio (e) or relative density (D_r)are the prime factors that control the liquefaction behaviour. Saturated clean sandy soils and silty sands with relative density (D_r) of 0.5 or less are most susceptible to the liquefaction-associated damage. Whereas, medium compact and compact sandy soils have the dilative behaviour if and when subjected to the cyclic shearing stresses which results in the generation of negative pore water pressures thus increasing their capacity to resist shearing stresses. The relative density (D_r) value after which the soil is relatively safe from the liquefaction damage is about 0.75. However, the value is not a standard, as there are other very many factors involved in the liquefaction process[10].

Degree of Saturation

[11]Sherif et al.concluded from laboratory tests that the soils with less degree of saturation (S) possess greater resistance to liquefaction than the ones with higher degree of saturation. There is an ample amount of evidence to support the statement,as it is generally understood and believed that the liquefaction occurs because of pore-water pressure generation, which occurs in the voids, therefore less the soil saturation the lesser pore pressures will be generated during the application of the static or dynamic loads.

IV. METHODOLOGY

Atterberg's (Consistency) Limits

Liquid Limit

Soil specimen for the liquid limit test were collected from the representative samples by passing the soil from the #40 sieve. The specimenwere then checked for their liquid limits using Casagrande Type Liquid Limit Apparatus (ASTM D4318). Three trials were made for each sample (at 10-20, 20-30, 35-45 blows respectively)so as to obtain the blows vs. water content (w) plot. Thereafter, the water content (w) corresponding to the 25 blows was determined from the graph plot of the recorded test data.

Plastic Limit

Soil specimen for the Plastic Limit (*PL*) test were collected from the representative samples by passing the soil from the #40 sieve. The soil specimen were then checked for theirplastic limitby the rolling threads method, wherein the samples were checked by rolling the soil-water mix on the glass plate and rolled until the threads crumbled or cracks appeared on them. The crumbled soil thread samples were then oven dried to check for their water content (w). The process was repeated three to four times for each sample and the average corresponding water content was determined. Thus, the average water content found was the plastic limit of the particular soil sample.

Hydrometer Analysis

Material (soil specimen) passing through #200 Sieve was taken from each representative soil sample and measured to weight 50g individually. The soil specimen was then soaked in a dispersing agent i.e. Sodium Hexametaphosphate $(NaPO_3)_6$ for the duration of 24 hrs. The soil specimen was then mixed with distilled water and mixed thoroughly in the stainless steel blender cup using the electrical stirrer. The specimen was then shifted to the sedimentation cylinder where it was analysed for the recommended period of the test. The test was performed using ASTM Type 151H hydrometer.

Relative Density

The representative soil samples were checked for their relative density (D_r)using a 3000 cm^3 mould. Firstly, the soil specimen was filled in the mould using a funnel in its loosest state and the minimum dry density of the soil specimen was obtained. The soil specimen was then again filled in to the mould, but in its most compact state, and was shaken at the vibrating table only to ensure that the specimen was compact to its maximum capacity, and the maximum compact dry density of the soil sample was thus obtained. Finally, the soil specimen was checked for its saturated maximum density by adding the water in the soil specimen and then filling the mould to its maximum compact state. Every specimen was checked twice for the individual density (i.e. minimum density, maximum density, saturated density) determination; the average of the two trials was taken as the result. The final results were then replaced in the following equation to obtain the relative density (D_r) value of the soil specimen.

$$D_r = \left(\frac{\rho_f - \rho_{min}}{\rho_{max} - \rho_{min}}\right)\left(\frac{\rho_{max}}{\rho_f}\right)$$

Where ρ_f is the field density, ρ_{max} , ρ_{min} are the maximum and minimum dry densities respectively.

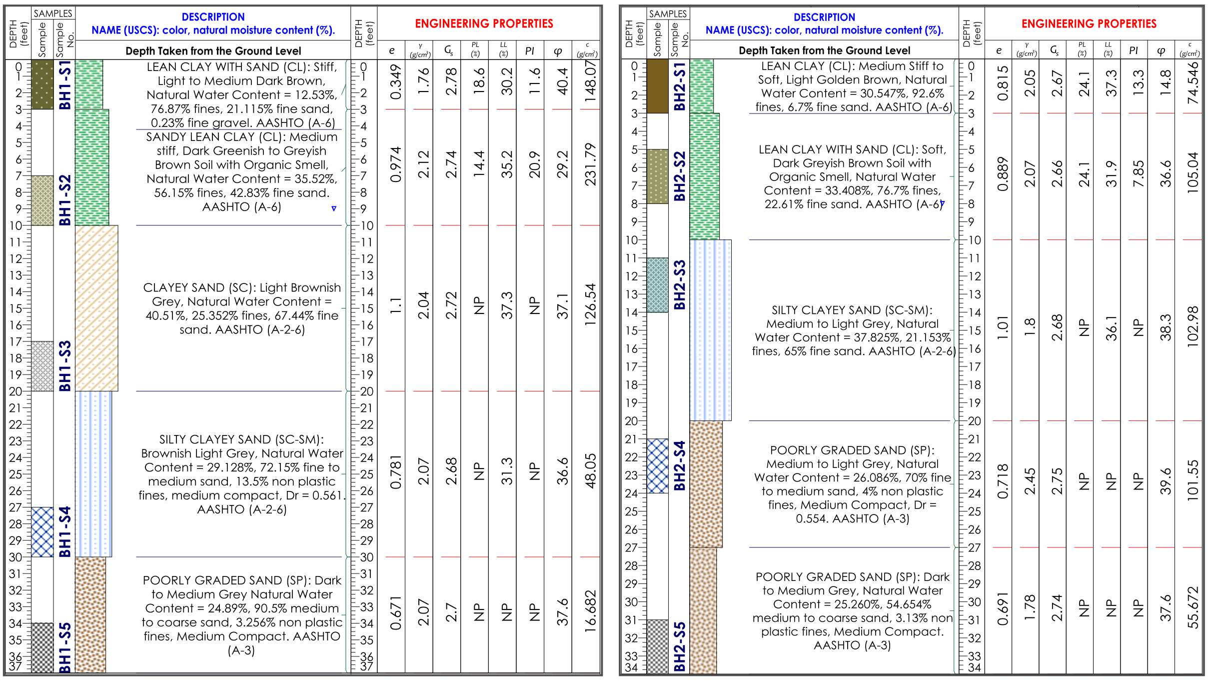
V. RESULTS

Table 4 – Laboratory Tests' Results for the Subject Soil

	Sample	**Depth**	**Water Content (*w*)**	***LL*[1] (%)**	***PL*[2] (%)**	***PI*[3] (%)**	***w/LL*[4]**	**D_r[5]**	**% Fines < 0.005**	**% Fines < 0.002**	**Unified Soil Classification**	
BOREHOLE-1	B1-S1	00'-03'	12.53%	30.25	18.62	11.63	0.414		76.87	0.2	CL	(Lean Clay with Sand)
	B1-S2	03'-10'	35.52%	35.22	14.35	20.87	1.009		56.15	0.83	CL	(Sandy Lean Clay)
	B1-S3	10'-20'	40.51%	37.34	-	-	1.085		25.35	-	SC	(Clayey Sand)
	B1-S4	20'-30'	29.13%	31.31	-	-	0.93	0.56	13.5	-	SC-SM	(Silty Clayey Sand)
	B1-S5	30'-37'	24.89%		-N.P.-		-		3.26	-	SP	(Poorly Graded Sand)
BOREHOLE-2	B2-S1	00'-03'	30.55%	37.34	24.06	13.28	0.818		92.58	11.14	CL	(Lean Clay)
	B2-S2	03'-10'	33.41%	31.91	24.05	7.85	1.047		76.71	3.4	CL	(Lean Clay with Sand)
	B2-S3	10'-20'	37.83%	36.06	-	-	1.049		21.15	0.66	SC-SM	(Silty Clayey Sand)
	B2-S4	20'-27'	26.09%		-N.P.-		-	0.55	4.08	-	SP	(Poorly Graded Sand)
	B2-S5	27'-34'	25.26%		-N.P.-		-		3.13	-	SP	(Poorly Graded Sand)

[1]*LL* denotes Liquid Limit.
[2]*PL* denotes Plastic Limit.
[3]*PI* denotes Plasticity Index.
[4]*w/LL* is the ratio of water content and Liquid Limit.
[5]$D_r$denotes relative density.

VI. BOREHOLE LOG



(A) (B)

Fig2 – (A) Borehole Log of Borehole 1. (B) Borehole Log of Borehole 2.

VII. DISCUSSION

Chinese Criteria [12]

Remarks: With % fines less than 15%, *LL* below the 35% guideline value, and the water content greater than 90% of the *LL*, the samples BH1-S4, and BH2-S4 satisfy all the three criterion proposed by Seed et al.[7].It can therefore be concludedthat the subject soil is potentially liquefiable.

Modified Chinese Criteria [8]

Remarks: Samples BH1-S4 and BH2-S4 have the clay content less than 10% and *LL* less than 32% which satisfies the Modified Chinese Criteria, thus indicating the potentially liquefiable behaviour of the saturated sandy strata represented by the aforementioned soil samples. It can therefore be concluded that the subject soil is potentially liquefiable.

Bray's Criteria [9]

Remarks:PI of samples BH1-S4 and BH2-S4 is less than 12 and ratio of water content to Liquid Limit (*w/LL*) is more than 0.85 which satisfies the criteria given / proposed by Bray et al.[9]. Therefore, the subject soil is substantially prone to liquefaction associated failure / damage.

Furthermore, the particle size distribution of the subject soil is well within the particle size range for the potentially liquefiable soils as suggested by H. Tsuchida[3], for the **Assessment of Liquefaction Potential based on Particle Size Distribution**. The subject soil can thus be classified as potentially liquefiable soil to most liquefiable soil.

The subject soil is of medium compact nature and does meet the lower limit criteria set for the non – liquefiable soil as described earlier in the **Relative Density Criteria**. It can thereforebe concluded that the subject soil is moderately liquefiable under the seismic loading conditions.

The subject soil is sandy and fully saturated beyond 10ft depth. It can therefore be concluded that the subject soil beyond the aforementioned depth is significantly liquefaction susceptible as described by Sherif et al.[11]in the **Degree of Saturation Criteria**.

VIII. CONCLUSION

The subject soil satisfies all the simple assessment criteria for the evaluation of earthquake-induced liquefaction potential, in light of the ample evidence provided it is suggested that the subject soil is eminentlypredisposed to liquefaction associated ground deformation failures if and when subjected to the seismicallyinduced shearing stresses. The soil is therefore unsuitable for any engineering related projects. It is therefore emphatically suggested that the subject soil be improved and treated before it could be utilised as a ground for any engineering construction.

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