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Liquefaction of granular soils with non-cohesive and cohesive fines

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ABSTRACT: Granular soil samples were reconstituted in the laboratory by mixing different amounts of an aeolian fine sand, a cohesionless silt and a highly plastic clay. A total of 8 different mixtures have been made. For each mixture, routine tests have been conducted to determine the grain size distribution, void ratio and Atterberg limits. Cyclic triaxial tests were conducted on each mixture to obtain the liquefaction resistance. The analysis of the test results leads to two main findings: 1) There are problems of relating the liquefaction resistance to individual physical parameters of grain size distribution, void ratio or Atterberg limits. Such problems are consistent with observations recorded in the existing literature. 2) The liquefaction resistance can be consistently related to the strength parameters of friction angle and cohesion in terms of effective stress. Such consistency stems from the fact that both liquefaction resistance and strength parameters reflect the joint effects of all the physical parameters.

1 INTRODUCTION

During earthquakes, the shaking of the ground may cause saturated granular soils to lose their resistance and behave like a liquid. This phenomenon is called soil liquefaction and will cause building settlement or tipping, sand boils, landslides and other failures. The modern study of soil liquefaction has been triggered by the numerous liquefaction-induced failures during the 1964 Niigata, Japan earthquake. Many ensuing studies were concentrated on clean sand with relatively insignificant amount of fines. As information accummulates, it becomes clear that many soil liquefaction failures occur in saturated granular soils with significant amount of fines (e.g. Zhou 1981 and Troncoso and Verdugo 1985). A review of the literature however shows there is confusion in trying to seek a key parameter to quantify the liquefaction resistance of such soils. The present study is an attempt to clarify the confusion through the use of strength parameters of cohesion (c') and friction angle (ϕ') in terms of effective stress.

2 LIQUEFACTION RESISTANCE OF GRANULAR SOILS WITH FINES

The published literature contains divergent opinions regarding the liquefaction potential of granular deposits containing fines, the first being the meaning of fines. The Chinese engineers are among the earliest ones to investigate the effects of fines. They consider fines as particles smaller than .005 mm in size. Engineers in North America and other countries including Japan, customarily consider fines as particles passing the #200 seive, i.e. smaller than .074 mm in size. The fines defined by the Chinese engineers are composed mainly of clay particles which provide cohesive strength during shearing. The North American definition, particularly when applied to tailings, considers fines as materials composing a large amount of silt that has no cohesion. During dynamic behaviour, therefore, granular soils containing these different fines, generally known by the common term of silty soil, will behave quite differently.

The different nature of fines has led to conflicting observations of the role fines play in the liquefaction resistance of granular soils. For materials with the same Standard Penetration Resistance or the same tip resistance of a cone penetration test, a number of researchers (Seed et al. 1983, Liu 1986 and Shibata and Teparaksa 1988) show that the liquefaction resistance increases with increasing fines content. Such an observation has led many practising engineers to have the impression that fines have a beneficial effect on liquefaction resistance. This impression is of course oversimplistic for it neglects the density or the void

Table 1 Characteristics of Different Sand-Silt Mixtures

Soil Characteristics	Sand 100%	Silt 10% Sand 90%	Silt 15% Sand 85%	Silt 30% Sand 70%	Silt 100%
D_{10} (mm)	0.11	0.075	0.027	0.01	0.002
D_{50} (mm)	0.22	0.21	0.20	0.17	0.017
Uniformity Coefficient D_{60}/D_{10}	2.27	3.2	8.5	20	11
Void Ratio after consolidation, e_c	0.739	0.707	0.736	0.636	0.777
Friction angle ϕ'_d deg.	32.6	21.7	16.3	13.9	17.7
Specific gravity of soil grain	2.65				2.72

Table 2 Characteristics of Different Sand-Clay Mixtures

Soil Characteristics	Sand 90% Clay 10%	Sand 70% Clay 30%	Sand 65% Clay 35%
Void Ratio after consolidation, e_c	0.718	0.610	0.609
D_{50} mm	0.205	0.170	0.160
D < 0.002 mm	9	27	30
Liquid Limit, %	16.3	21.1	23.4
Plastic Limit, %	16.0	12.6	11.4
Plasticity Index	0.3	8.5	12.0
Dynamic Friction Angle ϕ'_d deg.	26.1	19.0	16.7
Dynamic Cohesion C_d kPa	1.2	4.2	9.0

ratio of the soil. A granular soil with a certain fines content, having the same tip resistance as one with a greater fines content, has a significantly higher void ratio and hence a lower resistance. On the other hand, tests on tailings sand (Troncoso and Verdugo 1985) with different fines contents and reconstituted to the same void ratio, show that liquefaction resistance decreases with increasing fines content. By testing different mixtures of sand, silt and clay, Ishihara and Koseki (1989) show that the liquefaction resistance cannot be consistently related to fines content alone.

There have been attempts to relate the liquefaction resistance to the relative density of granular soils (DeAlba et al 1976). Such attempts produce consistent results only for clean sand. Difficulties arise when fines are present. According to ASTM Standards (D4253 and D4254, 1990), relative density can only be determined for granular soils with less than 15% by weight of non-cohesive fines (size less than .074 mm). Therefore, the relative density of silty sand with fines exceeding 15% or of cohesive nature, cannot be consistently determined. Consequently, it is not fruitful to use relative density for comparing liquefaction resistance of different granular soils as demonstrated by Lefebvre and Malenfant (1988).

Other criteria have been used to describe the liquefaction resistance of silty soils. Wang (1979) states that any silty soils containing less than 15% to 20% fines (grain size less than 0.005 mm) and with plasticity index less than 3, is possible to liquefy during strong earthquakes if the water content of the soil is higher than 90% of its liquid limit. This 3-parameter criterion specifies only whether the soil is possible to liquefy. There are no details of how the liquefaction resistance is related to these three parameters. Similarly, Liu and Xie (1984) suggest that silty soil with plastic index (PI) of 10 or greater will not liquefy. Ishihara and Koseki (1989) also support the use of PI based on laboratory study on mixtures of sand, silt and clay soils. Although their study shows a dependence of liquefaction resistance on PI, the study also shows a large scatter between these two quantities.

Based on the above review, it is clear that existing studies have not produced a satisfactory method for describing liquefaction resistance of silty soils. The aim of the present study is therefore directed towards establishing such a method. The study is based on laboratory tests on reconstituted soil samples of different composition.

3 EXPERIMENTAL STUDY

The soil samples studied in this test program were reconstituted by blending three different materials: (1) Chalk River sand, (2) Little Jackfish silt, and (3) New Liskeard clay. All the materials are from Ontario, Canada.

The Chalk River sand is an aeolian deposit of subangular medium fine sand. Its mean grain size (D_{50}) is 0.22 *mm* and its coefficient of uniformity is 2.27. The maximum and minimum void ratios are 0.894 and 0.541, respectively. At a relative density (D_r) of 40%, the angle of internal friction (ϕ') is equal to 32.6°.

The Little Jackfish silt comes from Northern Ontario. It has a mean grain size of 0.017 *mm* and a coefficient of uniformity of 11. The angle of internal friction (ϕ') varies with the void ratio, e. At e=0.550 and 0.777, ϕ'=34° and 17.7°, respectively.

The New Liskeard clay was extracted from the clay varves of the varved clay from the same site as described by Lo and Stermac (1965). It is a highly plastic material with a plasticity index of 50.

Eight different soil types were obtained by blending the materials in different proportions by weight. Five were by blending the sand and the silt while three were by blending the sand and the clay. The different soil combinations and pertinent information are shown in Table 1 and Table 2 and their grain size distributions are shown in Fig. 1a and Fig. 1b.

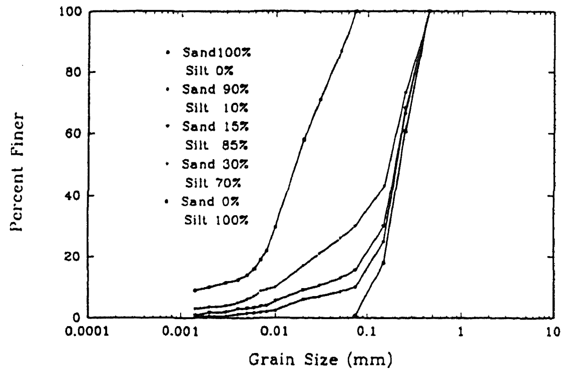

Fig. 1-a Grain Size Distribution of Sand and Silt Mixtures

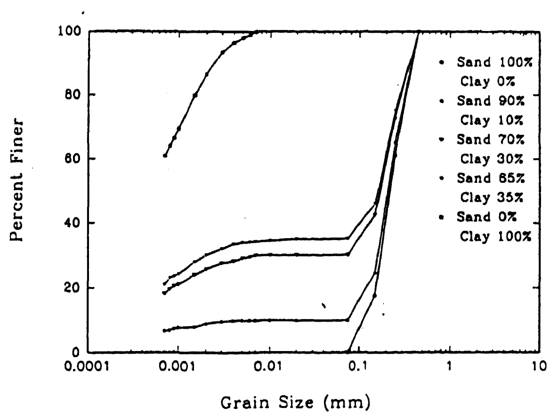

Fig.1-b Grain Size Distribution of Sand and Clay Mixtures

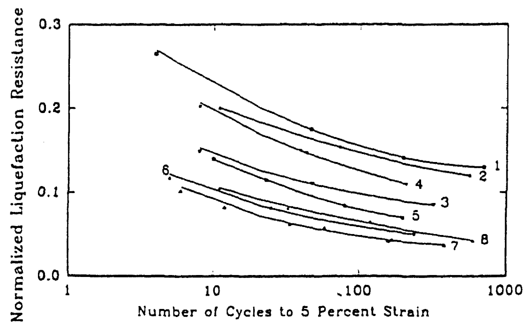

Fig. 2 Normalized Liquefaction Resistance Versus Number of Cycles

LEGEND

Curve	Composition			Initial void ratio	Void ratio after consolidation
	% of clay	% of silt	% of sand		
1	35	0	65	0.766	0.609
2	30	0	70	0.763	0.610
3	10	0	90	0.754	0.718
4	0	0	100	0.755	0.739
5	0	10	90	0.725	0.707
6	0	15	85	0.777	0.736
7	0	30	70	0.771	0.636
8	0	100	0	0.802	0.777

Samples were reconstituted using the moist tamping method. Layers of soil, each of 10 *mm* thick and at a moisture content of 4% were gently tamped to provide the specified density. Carbon dioxide was circulated through the sample to displace the air in the void. This was followed by circulating de-aired water to displace the carbon dioxide. A back pressure of 200 kPa was used to dissolve any trace of carbon dioxide that might remain in the void. A B value of 0.98 was achieved in most of the tests. An all-around consolidation pressure of 100 kPa was used for all the tests.

The samples were tested in an electro-magnetic cyclic triaxial device (Law et al. 1992). This device is similar to the more common pneumatic device except that a high frequency of cyclic loading can be applied. For the present study, however, the conventional low frequency of 1 Hz was used along with a sinusoidal load under the undrained condition. The axial deformation, axial load and excess pore pressure were monitored using an IBM compatible PC equipped with the proper data acquisition capability. Liquefaction resistance in this study is defined as the shear resistance associated with a certain number of load cycles, N, at which the pore pressure is equal to the initial consolidation pressure. For loose granular soil, this condition is accompanied by significant axial strain amplitude (5%) followed quickly by a collapse of the sample upon further straining. When the granular soil contains a significant amount of fines (defined in this experimental study as particles smaller than 0.074mm) the liquefaction resistance is the shear resistance applied for N cycles to produce an axial strain amplitude of 5%.

4 TEST RESULTS

Figure 2 summarizes the relationship between the normalized liquefaction resistance (τ_l/σ_v', where τ_l = liquefaction resistance and σ_v' = consolidation pressure) and number of load cycles (N) to reach liquefaction failure. There is a general decrease of liquefaction resistance with increase of N for both the sand-silt and the sand-clay mixtures.

4.1 *Sand-silt mixtures*

The normalized liquefaction resistance (τ_l/σ_v') for the sand-silt mixtures are replotted in Figure 3. At silt size less than 30%, there is a decrease of τ_l/σ_v' with increase in silt content. This is in agreement with some of the published observations (e.g.

Troncoso and Verdugo 1985). Beyond 30%, however, τ_l/σ_v' increases with increasing silt content. Plotted on the same figure is the uniformity coefficient (C_u) corresponding to the different mixtures. The lowest liquefaction resistance is associated with a C_u value exceeding 10. This is in contrast to what Liu and Xie (1984) stated that soil with $C_u > 10$ will be unlikely to liquefy.

Figure 3 shows one possible correlation between $\tau_l/\sigma_v'$and the dynamic friction angle, ϕ_d' which is measured from the slope of the effective stress path at and immediately after liquefaction failure. This dynamic effective friction angle is almost identical to the static effective angle of internal friction (ϕ') as demonstrated by Law and Ling (1992). The shape of the ϕ_d' curve is similar to those of the liquefaction resistance at different N. This suggests that a functional relationship between τ_l/σ_v' and ϕ_d' may exist.

To further investigate the potential of establishing a functional relationship between τ_l/σ_v' and ϕ_d', a series of tests was conducted on a sand (85%) - silt (15%) mixture at different void ratios. Figure 4 shows the test results indicating indeed there exists a simple relationship between τ_l/σ_v' and ϕ_d'. This relationship will be developed later in this paper.

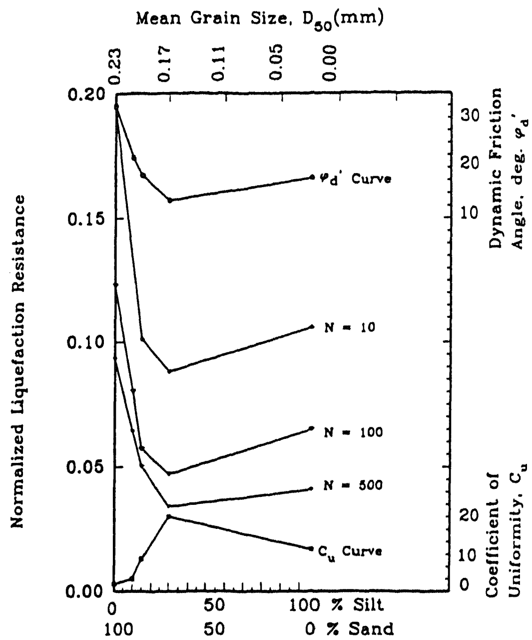

Fig.3 Normalized Liquefaction Resistance, Coefficient of Uniformity and Dynamic Friction Angle Versus Silt Content in Sand/Silt Mixtures

4.2 *Sand-clay mixtures*

The normalized liquefaction resistance (τ_l/σ_v') is replotted in Figure 5 for the sand-clay mixtures. The results show that τ_l/σ_v' decreases with increasing cohesive fines (clay) up to 10%. Beyond 10%, the trend is reversed with τ_l/σ_v' increases with increasing cohesive fines. The plasticity index (PI) associated with the mixtures are also shown on the same figure. As expected, the plasticity index increases monotonically with increasing cohesive fines. Therefore the trends of τ_l/σ_v' and PI are different. Consequently they cannot be related by a simple increasing function. This explains why Ishihara and Koseki (1989) produces a large scatter in attempting to relate τ_l/σ_v' with PI.

A better functional relationship can again be established between τ_l/σ_v' and the strength parameters of the soil mixtures: ϕ_d' and c_d' where ϕ_d' is again the dynamic effective friction angle and c_d' is the dynamic effective cohesion determined from the same dynamic test in a manner similar to that in the static test. The values of these parameters are listed in Table 2. Comparing these values with the liquefaction resistance in Figure 5 suggests that the liquefaction resistance can be related to the joint value of c_d' and ϕ_d' as shown in the next section.

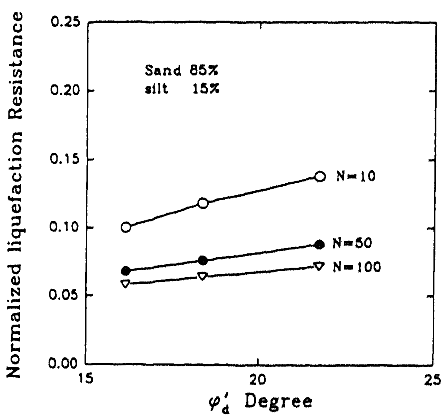

Fig. 4 Normalized Liquefaction Resistance Versus Dynamic Friction Angle φ_d'

5 DISCUSSION

The foregoing test results strongly suggest that the liquefaction resistance (τ_l/σ_v') can be functionally related to the strength parameters. Such relationships are developed herein separately for sand-silt mixtures and sand-clay mixtures.

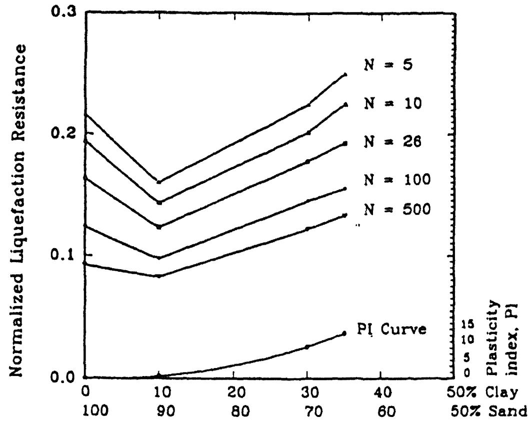

Fig. 5 Normalized Liquefaction Resistance Versus Clay Content in Clay/Sand Mixtures

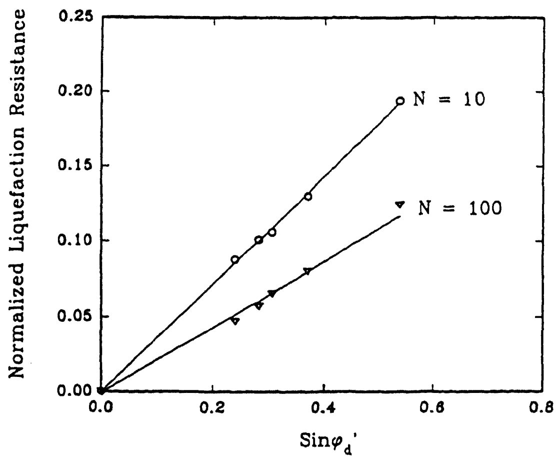

Fig. 6 Normalized Liquefaction Resistance Versus $\sin\varphi_d'$ for Sand/Silt Mixtures

5.1 *Relationship for sand-silt mixtures*

The sand-silt mixtures are a granular material whose strength can be characterized by a single parameter ϕ_d' or its modified form. The normalized liquefaction resistance (τ_l/σ_v') is therefore plotted with sin ϕ_d' in Figure 6 for different sand-silt mixtures and for N=10 and N=100. A linear relationship passing through the origin emerges between τ_l/σ_v' and sin ϕ' for a given N value. Hence

$$\tau_l/\sigma_v' = k_1 \sin \phi_d' \tag{1}$$

where k_1 is a function of N. In other words, τ_l/σ_v' for a given N is directly proportional to sin ϕ' for a granular deposit with non-cohesive fines.

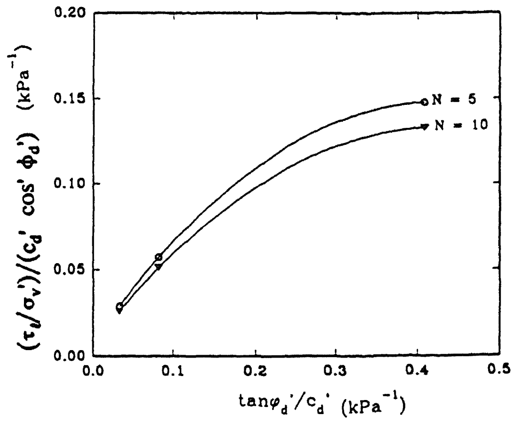

Fig. 7 Relationship between Liquefaction Strength and Dynamic Strength Parameters (c_d', φ_d') for Sand/Clay Mixtures

5.2 *Relationship for sand-clay mixtures*

The sand-clay mixtures are a cohesive-frictional material characterized by c_d' and ϕ_d'. A functional relationship can be developed between a modified form of τ_l/σ_v' and the joint effects of c_d' and ϕ_d'. One possible relationship is shown in Figure 7 in which $(\tau_l/\sigma_v')/(c_d' \cos \phi_d')$ from all the mixtures is plotted against tan ϕ_d'/c_d'. Smooth curves are obtained and hence:

$$(\tau_l/\sigma_v')/(c_d' \cos' \phi_d') = f \tag{2}$$

where f is a function of N and tan ϕ_d'/c_d'.

5.3 *Implications*

Since liquefaction resistance and the strength parameters, c_d' and ϕ_d', stem from the same failure condition for a given soil it is therefore reasonable that they should be functionally related as revealed by the test results. Such relationships in fact explain why problems exist in relating the liquefaction resistance with other parameters as noted earlier. Both the strength parameters and the liquefaction resistance reflect the joint effects of the physical parameters including void ratio, fines content, plasticity index and mineralogy. Attempt to relate τ_l/σ_v' with only one physical parameter, say fines content, therefore will be incomplete and will lead to erratic results. Theoretically, one can obtain satisfactory correlation only by consideration of all the physical parameters and this is fulfilled by correlating τ_l/σ_v' with the strength parameters.

6 CONCLUSIONS

The present study on the effects of non-cohesive and cohesive fines on the normalized liquefaction resistance (τ_l/σ_v') of a granular soil leads to two main conclusions.

1. There are difficulties in quantifying τ_l/σ_v' using a single physical parameter such as void ratio, fines content and plasticity index. The reason is that τ_l/σ_v' reflects the joint effects of all these parameters and therefore attempts to relate τ_l/σ_v' with only one of these parameters at a time will lead to unsatisfactory results.

2. Correlation between τ_l/σ_v' and strength parameters, c_d' and ϕ_d' appears to produce satisfactory results. The reason for such success is due to the fact that both τ_l/σ_v' and the strength parameters reflect the joint effects of all the physical parameters.

7 ACKNOWLEDGEMENTS

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