Liquefaction Susceptibility of Fine-Grained Soils

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Abstract—**The results of cyclic tests show that the Chinese criteria are not reliable for determining the liquefaction susceptibility of fine-grained soils. Soils that were observed to have liquefied in Adapazari during the Kocaeli earthquake did not typically meet the clay-size criterion of the Chinese criteria. It is not the amount of "clay-size" particles in the soil; rather it is the amount of clay minerals that best indicate a soil's susceptibility to liquefaction. Hence, a soil's plasticity index is a better indicator. Loose soils with PI $\leq$ 12 and w_c/LL > 0.85 were susceptible to liquefaction, and loose soils with 12 < PI < 20 and w_c/LL > 0.8 were systematically more resistant to liquefaction but still susceptible to cyclic mobility. Soils with PI > 20 tested at effective confining stresses < 50 kPa did not generate significant strain after the application of a large number of loading cycles, indicating that for these test conditions, they are not susceptible to liquefaction. Additionally, the results of this testing program provide relevant insights for the effects of confining pressure and initial static shear for fine-grained soils**.

Keywords—**Earthquake, Fine-grained soil, Liquefaction**

INTRODUCTION

The performance of fine-grained soils in Adapazari, Turkey during the 1999 Kocaeli earthquake offers an exceptional opportunity for understanding their susceptibility to liquefaction and response under cyclic loading. Following the earthquake, a comprehensive program of subsurface investigations through energy-measured SPT, CPTU (some seismic), and a battery of soil index testing was completed by Bray et al. [1]. Soils that were observed to have liquefied in Adapazari during the Kocaeli earthquake did not typically meet the Chinese criteria as defined by the state-of-the-practice paper by Youd et al. [2].

To investigate this further, a program of "undisturbed" sampling with a thin-walled hydraulic piston sampler with cyclic triaxial (CTX) testing was completed on the silty and clayey soils of Adapazari. These fine-grained soils typically contain more than about 70% fines (i.e. finer than the 200 sieve) with Plasticity Index (PI) ranging between 0 and 25. Because the liquefaction susceptibility of these types of soils is not well understood, the results of this testing program provide useful insights for projects encountering these soils.

PROCEDURES FOR EVALUATING THE LIQUEFACTION SUSCEPTIBILITY OF SOILS

Based on the data from sites where liquefaction was and was not observed after earthquakes in China, Wang established that any clayey soil containing less than 15% to 20% particles by weight smaller than 5 μm and having a water content (w_c) to liquid limit (LL) ratio greater than 0.9 is susceptible to liquefaction [3]. Based on these data, Seed and Idriss stated that "clayey soils" (i.e. plots above the A-line on the plasticity chart) could be susceptible to liquefaction <u>only if all three</u> of the following conditions are met: (1) Percent less than 5 μm < 15%, (2) LL < 35, and (3) w_c/LL > 0.9 [4]. Due to its origin, this standard is known as the "Chinese criteria."

Koester noted that the determination of LL by means of the fall cone used in China produced values that are about 4 points higher than those values determined by means of the Casagrande percussion device [5]. Hence, Koester recommended a slight reduction of the LL condition of the Chinese criteria before using it as a screening tool when the Casagrande method has been used. Similarly, Andrews and Martin reduced the LL condition for liquefaction susceptibility to < 32, and then used 2μm as the limit between silt-size and clay-size particles, with < 10% clay-size particles being necessary for a silty or clayey soil to be liquefiable [6]. In addition, they dropped the w_c/LL ratio as a condition in their liquefaction susceptibility criteria of silty soils.

After the 1989 Loma Prieta earthquake, Boulanger et al. noted that a deposit of clayey soil might have contributed to the surface deformations observed at the Moss Landing site [7]. They stated that indiscriminate use of the Chinese criteria as a substitute for detailed laboratory and in-situ testing should be avoided. Similarly, Perlea recommends laboratory testing as the best way to evaluate the liquefaction susceptibility and post-cyclic undrained strength of cohesive soils [8].

The location of a soil on the Casagrande plasticity chart and, or in combination with, the use of the "C" descriptor of the Unified Soil Classification System (USCS) have also been introduced as a tool to identify potentially liquefiable soils. For example, Youd recommends that natural soil deposits that have a "C" descriptor (e.g. CH, CL, SC, and GC) may be screened as nonliquefiable [9]. He finds that fine-grained soils that are possibly susceptible to liquefaction should have LL < 35 and plot below the A-line or have PI < 7. Seed et al. indicate that soils that have LL < 30 and PI < 10 are liquefiable, and those with 30 < LL < 40 and 10 < PI < 12 fall into an "uncertain range" [10]. Undisturbed samples of soils that plot in that region of the plasticity chart should be obtained for laboratory testing. Polito used a wider range of the plasticity chart, and according to his criteria, soils with LL < 25 and PI < 7 are liquefiable; soils that have 25 < LL < 35 and 7 < PI < 10 are potentially liquefiable; and soils with 35 < LL < 50 and 10 < PI < 15 are susceptible to cyclic mobility [11].

The current state-of-the-practice for liquefaction evaluations is established in Youd et al., which is based on the 1996 and 1998 NCEER/NSF workshops on evaluation of liquefaction resistance of soils [2]. Therein, the use of the "Chinese criteria" as defined above is recommended to evaluate the liquefaction susceptibility of both silts and clays.

SAMPLING AND TESTING PROCEDURES

Field sampling

"Undisturbed" soil samples were obtained using two different types of fixed piston samplers. The Geonor K-200 54-mm piston sampler was used initially. However, a majority of the specimens tested were obtained using the Dames and Moore hydraulic piston sampler, because this sampler induced less sample disturbance. It uses thin walled brass tubes of constant internal diameter, ID = 61.21 mm, and outside diameter, OD = 63.5 mm. The tubes are pushed into the soil a length of 45 cm using the pressure provided by the circulation water. The use of the brass tubes is advantageous in that the friction between the soil and the tube is reduced due to the smoother finish of the brass. Additionally, the area ratio, defined by Hvorslev [12] as $C_a = 100*(OD^2-ID^2)/ID^2$, is reduced from 11.4% for the Geonor steel tubes to 7.6% for the brass tubes. Following the extraction of the sample from the borehole, plastic caps were taped at the top and bottom, and the tube was placed upright in a box. The tubes were then carefully transported by car from Adapazari to Istanbul, where the soils were tested.

Testing equipment

All CTX tests in this study were performed using the Automated Triaxial Testing System in the geotechnical laboratory of ZETAS Corporation in Istanbul, Turkey [13]. This testing system is a modification and extension of the CKC electropneumatic cyclic triaxial testing system described by Chan [14]. A series of CTX tests were first performed on Monterey sand following the methodology described by Silver et al. [15], and the testing equipment and procedure were shown to produce results consistent with those previously published.

Specimen preparation and testing procedure

Trimming was avoided to minimize disturbance. The upper 5 to 10 cm of soil that was disturbed due to drilling was removed, and similarly, 5 to 10 cm of soil was removed from the bottom end of the tube. The sampling tubes were fixed onto the circular vise of a vertical extruder and cut at the desired length using a rotating tube cutter. To avoid radial deformation of the tube, two stiffening rings were placed above and below the location of the cut. Additionally, the tube cutter was turned slowly while applying gentle pressure. There was no visual evidence of reduction in diameter due to inward folding of the edges of the tube. The burrs produced by the cut were removed from the top end of the tube prior to extrusion.

The soil was pushed upwards in the same direction of movement during sampling. The upper portion of the exposed sample that was damaged during the deburring operation was removed using a wire saw. Once the desired length had been extruded (leading to height to diameter ratios between 2.1 and 2.5), the specimen was cut at the bottom using a wire saw and carefully placed over the bottom platen. Capillary stresses in these fine-grained soils were sufficient to temporarily maintain freestanding specimens. The top platen was set over the specimen and a 0.3 mm-thick latex membrane was placed around the specimen using a membrane stretcher. The membrane was sealed to the platens with O-rings, and a small vacuum was applied to provide confinement. The value of the vacuum depended on the confining pressure at which the specimen would be tested and was equal to 30 kPa for tests at confining pressures greater than 40 kPa and 15 kPa otherwise.

After application of the vacuum, the specimen height was measured with a dial gauge and its diameter was measured at the bottom, center, and top using a Pi tape. The average of the three diameter measurements was used for the computation of the effective diameter. Comparison of height measurements performed on a large number of specimens before and after the application of the initial vacuum show small variations in height (< 0.5%). The triaxial cell was then fully assembled and the chamber was filled with water. Sample saturation using differential vacuum was performed and followed by a new height measurement and installation into the triaxial testing equipment for backpressure saturation and cyclic loading. For all cases, B-values > 0.98 were achieved at relatively low backpressures (< 150 kPa). During consolidation of the specimens to the in-situ stress conditions, the change in void ratio (Δe) was measured and found to be minimal (i.e. $\Delta e/e_o < 0.03$), which indicates very good to excellent specimen quality according to the criteria used at NGI [16].

The cyclic triaxial test was then performed by applying stress controlled sinusoidal loading under undrained conditions at a frequency of typically 1 Hz. The loading amplitude was characterized in terms of the Cyclic Stress Ratio (CSR), which is defined as the

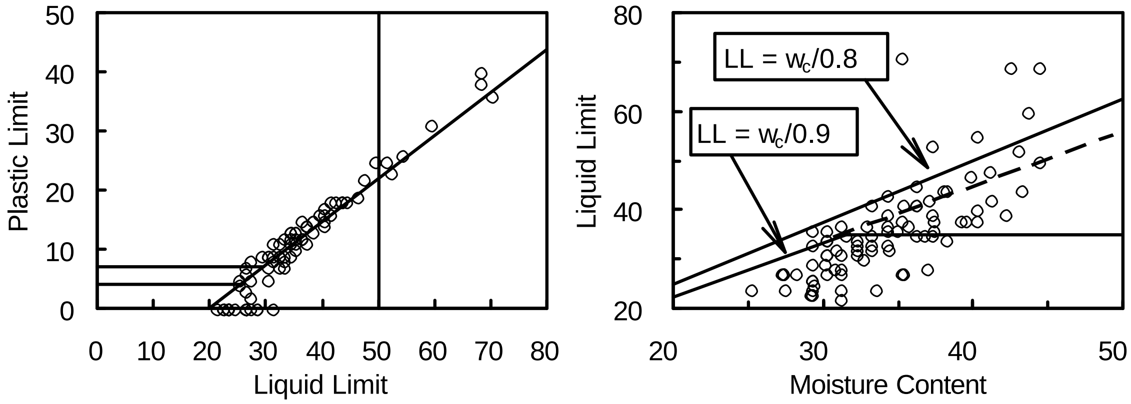

Fig. 1: (a) Location on the Casagrande plasticity chart of the soils tested in this study, (b) relationship between the LL and moisture content of the soils.

maximum peak shear stress divided by the initial effective mean consolidation stress, σ'_c. In general, one of three nominal CSRs (0.3, 0.4, and 0.5) was applied to a given specimen, and cyclic loading was applied until axial strains in excess of 4% were achieved. Post-liquefaction tests included reconsolidation or residual strength tests.

RESULTS

Soil Index Testing and Static Strength and Compressibility

The natural variability of the soil conditions in Adapazari allowed studying the effect of various factors, such as soil plasticity and initial water content, on the cyclic strength of silts, clayey silts, and silty clays. Overall, over 100 cyclic triaxial tests, 19 static strength tests, 24 consolidation tests (incremental loading and constant rate of strain), and numerous index tests (water content, Atterberg limits, and hydrometer) were performed on "undisturbed" soil specimens recovered at 7 different sites throughout the city.

Figure 1a presents the location on the plasticity chart of the specimens tested under cyclic triaxial conditions, and Figure 1b depicts the relationship between the LL and water content for these soils. All specimens have more than 35% of particles smaller than 75 microns (% fines). Many of the specimens had more than 15% of particles by weight greater than 5 microns. Tests were performed on soils with a wide variety of properties (but within a relatively narrow range of in situ penetration resistances) that cover a large range on the plasticity scale ($0 < PI < 40$).

Laboratory studies on the cyclic response of tailings materials by Ishihara [17] have shown that relative density "does not serve as a useful measure to consistently express the denseness and looseness of the materials." Instead, they found the cyclic resistance to correlate well with the void ratio, with a soil having a greater cyclic resistance at a lower void ratio. In this study, it was found that the soil's in situ penetration resistance was a useful index of the state of the soil, so that a soil's normalized Standard Penetration Test $(N_1)_{60}$ value in combination with its plasticity index were good indicators of the cyclic strength of fine-grained soils. The $(N_1)_{60}$ values of the soil specimens tested cyclically were typically between 3 and 8.

The results of consolidation tests were difficult to interpret for these low plasticity silts and clays. However, using the work method proposed by Becker et al. [18], supplemented with the conventional Casagrande method, the overconsolidation ratios of the upper soils of Adapazari were estimated to be generally between 2 and 4. Desiccation is the most likely cause for the slight overconsolidation of the soils in the upper few meters of Adapazari. Inhabitants of Adapazari stated that until flood control dams were built across the Sakarya River in the last half of the 20th century, flooding of the city was common, often occurring one or two times each year. The deposited soils were generally silty and cracked when exposed to sun and wind. A carbon sample taken from a sample at a depth of 4 m was dated to be less than 1000 years old, indicating that the upper soils in Adapazari were only a few hundreds of years old or less.

Static triaxial compression tests were performed using the recompression technique with both strain and stress controlled loading procedures. After an initial contractive response up to an axial strain of approximately 1%, the soils tested exhibited a pronounced dilative response upon continued shearing. The peak friction angle for these strain-hardening soils (based on the maximum stress obliquity) was measured to be typically between 38^o and 42^o.

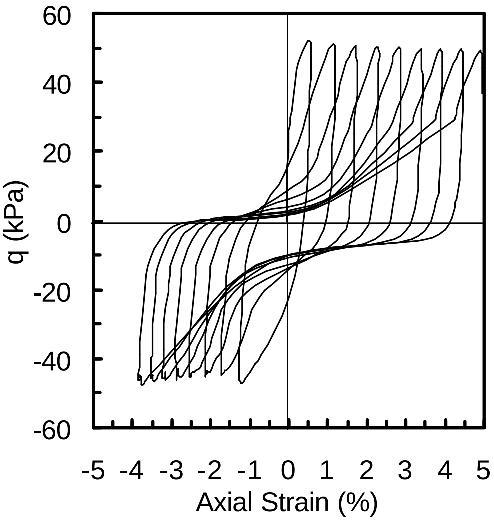

Fig. 2: Results of CTX test on non-plastic silt (LL=23) performed at σ_c'= 50 kPa and CSR = 0.48.

Isotropically Consolidated Cyclic Triaxial Tests

The calculation and comparison of the effective stress path (q vs. p') in the 1 Hz and 0.005 Hz cyclic tests provided evidence of the limitations of the measurements of pore water pressure changes on clayey silts during a 1 Hz cyclic triaxial test. Hence, it is often unclear if these soils reach a state of zero effective stress during undrained cyclic loading at 1 Hz. In this study, the results of particular tests will be presented in terms of the number of uniform stress cycles necessary to reach 3% single amplitude axial strain (typically extension). Results are similar if the 5% double amplitude axial strain criterion is used.

Fig. 2 shows the results of an undrained cyclic triaxial test on a non-plastic silt. At this relatively high CSR of 0.5 and low initial effective confining stress of 50 kPa, this loose silt specimen quickly generates positive pore pressures and reaches an axial strain of 3% in extension in 5 cycles of loading. Its dilative response after reaching a low effective confining stress is indicative of that observed for the other tests. The soil response of the silts in the upper soil strata of Adapazari is characterized as "cyclic mobility with limited strain potential."

Effects of Soil Plasticity

The effect of soil plasticity was studied by comparing the results of specimens of different plasticity tested at the same confining stress. Fig. 3 presents the results of tests performed at σ'_c = 40 and 50 kPa. A significant distinction of the cyclic strength for soil specimens with PI $\leq$ 12 could not be discerned, and the results are therefore grouped in two categories, soils with PI $\leq$ 12 and soils with 12 < PI < 20. It is noteworthy that most of the data from sites that liquefied in China on which the Chinese criteria is based have PI < 12 and w_c/LL > 0.85 [3]. Additionally, this is fairly consistent with Ishihara [17], who defines PI = 10 as the boundary after which a correction factor greater than one should be applied to the cyclic strength of soils as determined from in-situ methods. However, different from our results, Ishihara suggests the application of a linearly increasing correction factor to soils with 10 < PI < 50. In this study, specimens with PI > 20, and in particular when w_c/LL < 0.8, did not generate significant cyclic strains after a large number of cycles had been applied at low initial confining stress (σ'_c < 50 kPa).

The void ratio of the 26 specimens with PI $\leq$ 12 shown in Figure 3 ranges between 0.75 and 1.00, and on average e = 0.86. Given that the specimens tested are not of identical soils, it is difficult to assess the effect of density in terms of solely void ratio. However, the scatter of the data, particularly at 50 kPa, is sufficiently small to indicate that the possible effect of density on the cyclic resistance of the soil is negligible or compensated by another effect when the results are grouped in terms of PI. As stated previously, the normalized penetration resistances, $(N_1)_{60}$, of the soil deposits tested in this study are consistently between 3 and 8, indicating that the soils tested are generally in similar in situ states.

The first cycle of the test (fine line) and the cycle at which the specimen reaches 3% axial strain in extension (thick line) of four tests performed at a frequency of 1 Hz on specimens with PI ranging between 0 and 18 are shown in Fig. 4. All tests were performed at an initial confining stress of 50 kPa, and the specimens were loaded to a maximum peak deviator stress of approximately 40 kPa (CSR ~ 0.4). Upon inspection, some differences can be identified between each of the stress-strain curves. For example, specimens with lower plasticity appear to develop higher strains in extension during the first cycle of loading. Additionally, the number of cycles to reach 3% axial strain in extension increases as the plasticity index of the soil increases. The void ratio of the soil also increases as a soil's plasticity increases reaffirming that void ratio alone is not useful in characterizing the cyclic strength of these natural low plasticity silts and clays.

All of the specimens eventually undergo dilation during shearing, leading to the development of "banana loops." The shape of the stress-strain curve appears to be more symmetrical as the plasticity of the soil increases. The reduction of the soil's shear modulus as the deviator stress increases (or decreases) appears to be smoother as PI increases. The degree of equilibrium of pore water pressure throughout a specimen with lower PI will be higher for tests performed at the same loading rate. This may affect the shape of the stress-strain curve in soils with higher plasticity.

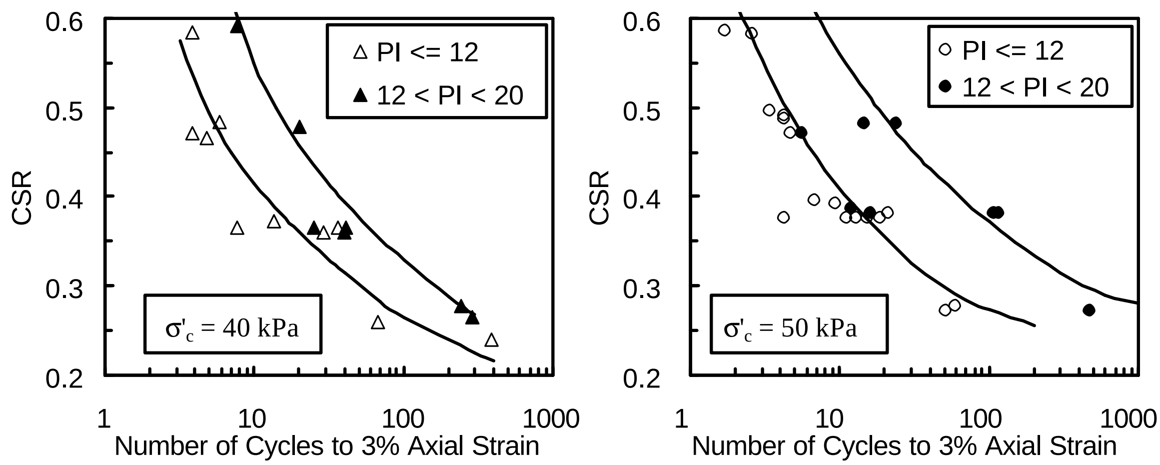

Fig. 3: CSR versus number of uniform cycles to reach 3% axial strain for isotropically consolidated specimens to (a) 40 kPa and (b) 50 kPa, that have PI ≤ 12 (empty symbols) and PI > 12 (filled in symbols). Note that soils with PI > 20 did not generate significant pore pressures or axial strain.

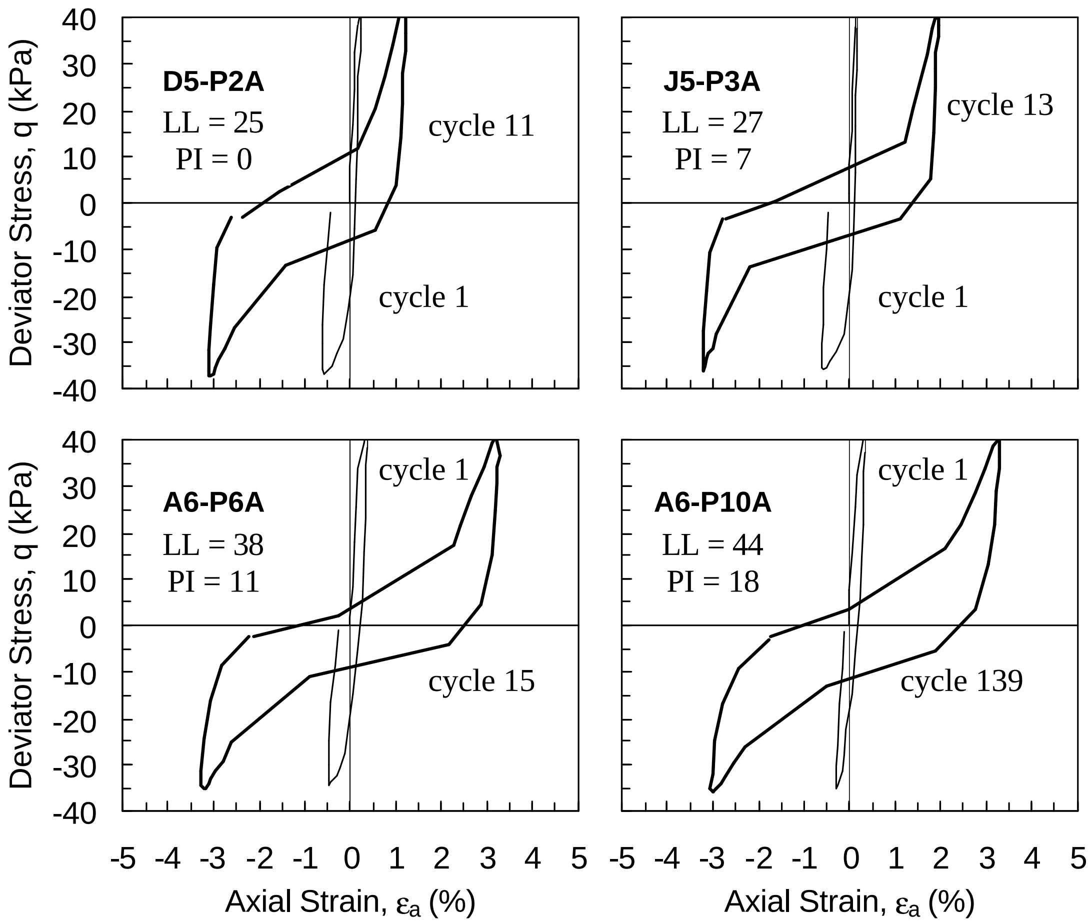

Fig. 4: Stress-strain relationship for the first cycle of loading and the cycle at which –3% axial strain is reached for four specimens of increasing plasticity. The tests were performed on soils initially consolidated to an effective stress of 50 kPa.

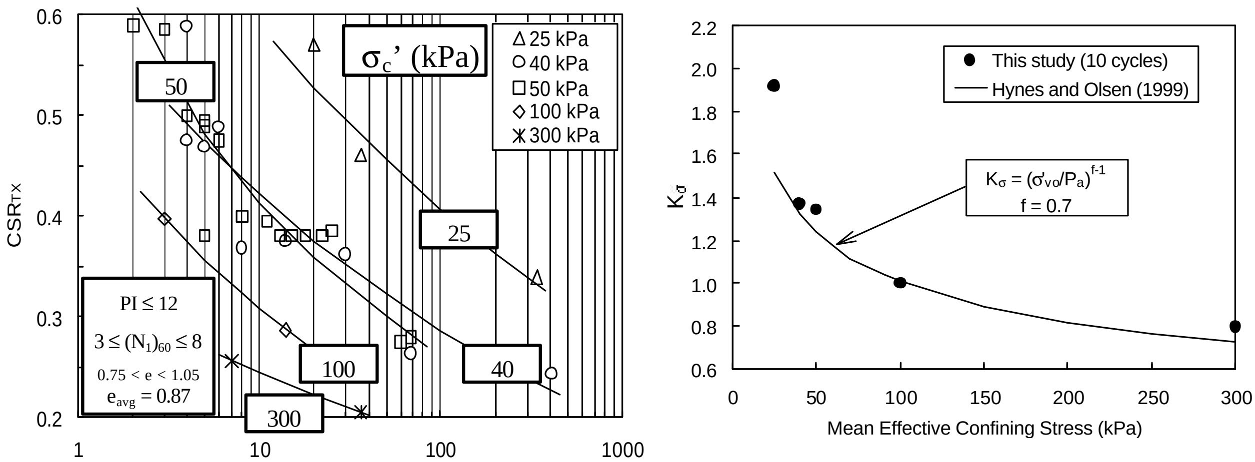

Fig. 5: (a) Number of cycles to reach 3% axial strain in extension versus applied cyclic stress ratio for specimens loaded to different hydrostatic initial effective consolidation stresses, and (b) normalized effect of confining pressure on liquefaction resistance at 10 cycles of loading for soils with low PI and penetration resistances.

The number of cycles required to reach 3% axial strain in extension (N_l) increases slightly from $N_l = 11$ at PI = 0, to $N_l = 13$ at PI = 7, to $N_l = 15$ at PI = 11, and then N_l increases abruptly to 139 (an order of magnitude) when the PI of the soil is 18. For this particular case, N_l increases by approximately 50% between a soil element with PI = 0 and another with PI = 11 (i.e. 11 cycles and 15 cycles, respectively). However, this effect is not always as significant as shown in Fig. 4 due to other variations in these natural soil specimens. For example, specimen I6-P6 (e = 1.05) has PI = 7 and $N_l = 5$ when subjected to a peak cyclic deviator stress of 50 kPa, while specimen I8-P1B (e = 0.81) is non-plastic and also required $N_l = 5$ when a similar peak deviator stress was applied. Although they have different PI, these two specimens required the same number of cycles to reach the same strain level. The non-plastic soil had a lower void ratio than the slightly plastic silt, and this may be a compensating effect. However, the strengths of the Adapazari silt specimens are consistently higher when the PI of the soil is greater than 12.

Effects of Initial Confining Pressure

To assess the effects of the weight of the structure on the soil's cyclic strength, tests were performed at different initial isotropic stresses. Liquefaction and ground failure were commonly observed adjacent to buildings and not in the open areas of the city. Liquefaction analyses indicated that a majority of the weak soil deposits at the sites of interest lie at depths generally between 2 and 5 m [1]. This considered, a value of $\sigma'_c = 25$ kPa was chosen as representative of these depths in the free field. Similarly, 40 kPa and 50 kPa were estimated as representative of the mean effective stress for soils under the corner of the mat foundation of typical 4 to 5 story structures in Adapazari. Additionally, a series of tests was performed on soils consolidated to higher stresses.

The CSR vs. the number of uniform stress cycles required to reach 3% single amplitude axial strain for a series of CTX tests performed on similar soils (PI ≤ 12) are shown in Fig. 5a. It is clear that soils at low effective confining stresses are more resistant to liquefaction than those at higher confining stresses. Hence, soils under the additional weight of the building would be more likely to liquefy and at lower CSRs or N_l than those in the free-field, and this experimental finding is consistent with the observations made by a number of survey teams following the Kocaeli earthquake.

The ratio of the CSR causing liquefaction in 10 cycles of loading at each effective confining pressure to that at an effective confining pressure of 100 kPa can be calculated from the data presented in Fig. 5a, and this is portrayed in Fig. 5b as a K_σ effect. Because the soils at low confining pressures (< 80 kPa) are actually slightly overconsolidated, the data from this study is not just expressing a K_σ effect. Instead, it is showing the combined effects of confining pressure and overconsolidation ratio. However, a comparison with K_σ data from other studies is insightful.

Examining Fig. 5b, the data from this study where OCR ~ 1 (i.e. > 80 kPa) compare favorably to those presented by Hynes and Olsen [19]. The equation for K_σ that was recommended by Youd et al. [2], which is given in Fig. 5b, captures the data well when OCR ~ 1. At lower confining pressures, the data from this study start to deviate from the trend of this equation, because the OCR of the soil is increasing as the confining pressure decreases, which is making the soil more resistant to liquefaction. Hence, it is important for these slightly overconsolidated shallow soils in the free field to capture the combined effects of confining pressure and overconsolidation. It is also clear that it is overly conservative to limit K_σ to a maximum of 1, because even at OCR ~ 1, these results coupled with the results of

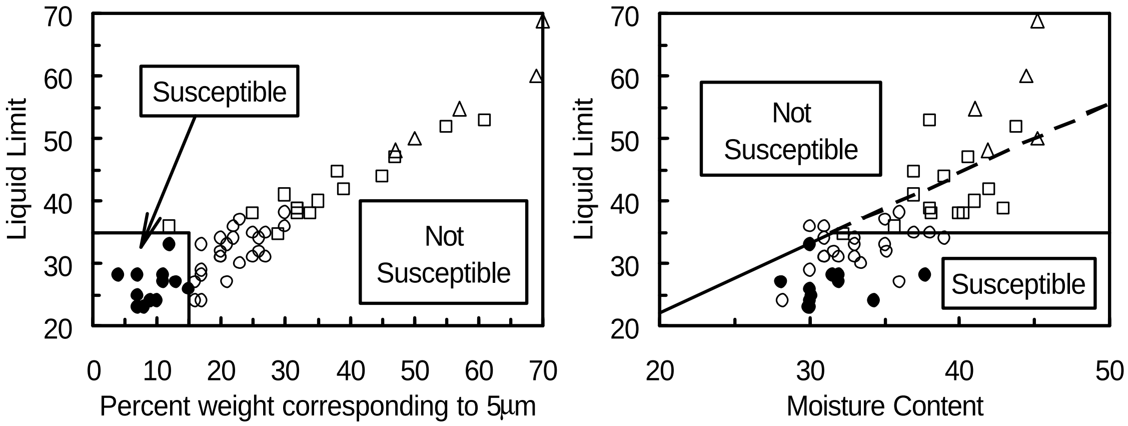

Fig. 6: Graphical representation of the Chinese criteria [2]. Test specimens susceptible to liquefaction are shown as circles; those with moderate susceptibility are shown as squares; and those not susceptible to liquefaction are shown as triangles. Only the filled symbols met all three conditions of the Chinese criteria.

Hynes and Olsen [19] indicate that soils at low confining pressures are more resistant to liquefaction.

RECOMMENDATIONS REGARDING LIQUEFACTION SUSCEPTIBILITY

The observations in Adapazari following the 1999 Kocaeli earthquake have shown that buildings on soil deposits that exhibit cyclic mobility in the laboratory can settle and tilt excessively and that surface manifestations of liquefaction such as sediment ejecta may also occur under conditions of cyclic mobility. Fig. 6 was developed to evaluate the effectiveness of the Chinese criteria [2] in the evaluation of the liquefaction/cyclic mobility susceptibility of the specimens tested in this study. In Fig. 6, circles are used to depict soils that underwent cyclic mobility in the laboratory and ground failure in the field. These soils generally had $PI \leq 12$. Squares are used for soils that are considered to be moderately susceptible to cyclic mobility ($12 < PI \leq 20$). Other soils with $PI > 20$ were not susceptible to cyclic mobility at the confining stresses used in this study, which represent the shallow soil deposits of Adapazari.

Out of 44 specimens that were found to be "susceptible" to $r_u = 100\%$, only 14 (32%) met all three conditions of the Chinese criteria (filled-in circles). Additionally, all of the soils with $12 < PI \leq 20$ reached at state of $r_u = 100\%$ at some point during the cyclic triaxial tests and deformed considerably under the applied cyclic stresses, and hence were designated as moderately susceptible to liquefaction. Contrary to this observation, the "Chinese Criteria" would not classify any of these intermediate PI soils as susceptible to liquefaction. All but eight specimens of soil with $PI \leq 12$ met the LL and w_c/LL conditions of the Chinese criteria. Of these eight, two specimens did not meet the w_c/LL condition, whereas eight did not meet the LL condition, yet they still underwent cyclic mobility in the cyclic triaxial test after a reasonable number of loading cycles.

Fig. 7 was prepared to evaluate the effectiveness of the Andrews and Martin criteria [6] for the evaluation of liquefaction susceptibility of silty soils. Of the 44 specimens that were found to be "susceptible" to cyclic mobility, only 11 (25%) met the two conditions of the criteria (filled circles). Fifteen other specimens have $LL < 32$ but 10% or more than 10% of particles smaller than 2 microns, and therefore fall into the category for which they recommend further studies to consider non-plastic clay-sized grains. More importantly, 12 other specimens were found to be susceptible to liquefaction, event though these criteria suggest that they are not susceptible.

These results indicate that the condition based on the amount of particles smaller than 5 µm, or for that matter 2µm as well, is a poor indicator of the soil's response and thus liquefaction susceptibility, and it should not be used. It is not the percent of clay-size particles that is important, but rather it is the percent of active clay minerals present in the soils. As warned by Casagrande as early as 1933 [20], use of particle size alone as a basis for classifying soil is not prudent. Moreover, use of the $LL < 35$ condition as an absolute standard is not prudent, because a number of soils with $LL > 35$ were found to be moderately susceptible to liquefaction. The w_c/LL condition appears to be more reliable, with soils that have $w_c/LL \geq 0.85$ as being susceptible to cyclic mobility or liquefaction and significant deformations.

Based on the results of this study, a soil deposit is considered to be susceptible to liquefaction or cyclic mobility if the ratio of the water content to liquid limit is equal to or greater than 0.85 ($w_c/LL \geq 0.85$), and the soil plasticity index is equal to or less than twelve ($PI \leq 12$). Soils with plasticity index greater than twelve and less than or equal to twenty ($12 < PI \leq 20$) and water content

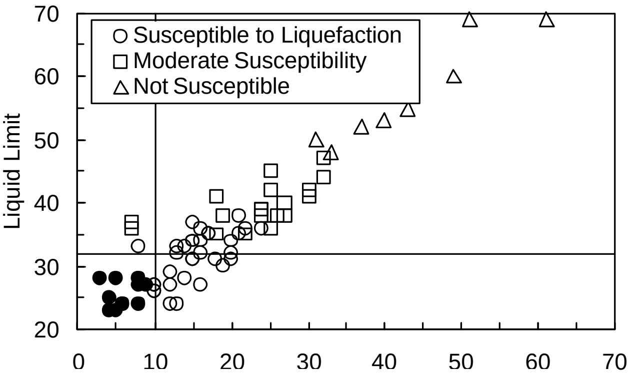

Fig. 7: Graphical representation of the Andrews and Martin criteria [6]. Filled-in circles correspond to specimens that meet the conditions of the criteria.

to liquid limit ratio greater to or equal than eighty hundreds ($w_c/LL \geq 0.8$) may be moderately susceptible to liquefaction or cyclic mobility, and they should be tested in the laboratory to assess the strain potential and liquefaction susceptibility under the loading conditions existing in the field. Soils with PI > 20 are considered too clayey to liquefy. However, structures founded on these soils, and for that matter, under any soil, may undergo significant deformations if the cyclic loads approach or exceed the dynamic strength of the soil.

ACKNOWLEDGMENTS

Financial support for the testing program was provided by the National Science Foundation under Grant CMS-011606. ZETAS Corporation provided the laboratory equipment and facilities in Istanbul, as well as the drilling equipment in Adapazari. Other support such as housing was provided by ZETAS Corporation and our Turkish colleagues, and this and all support is greatly appreciated.

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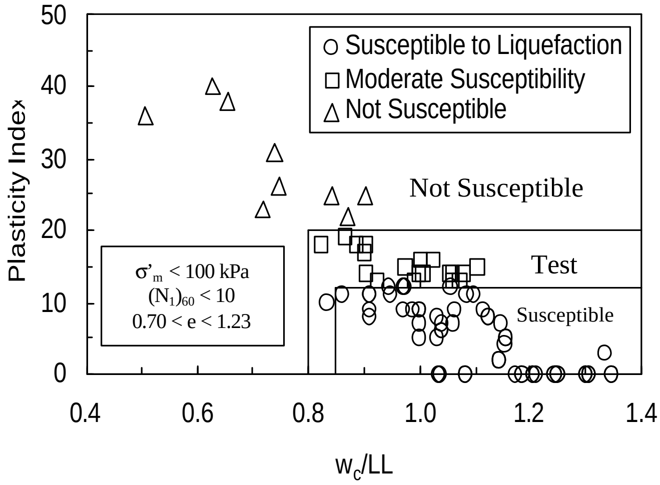

Fig. 8: Proposed criteria for the evaluation of the liquefaction susceptibility of fine-grained soils at low confining stresses.

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