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An assessment of the liquefaction susceptibility of Adapazari silt

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ABSTRACT: Liquefaction and cyclic softening of fine-grained soils occurred in Adapazari during the 1999 Kocaeli, Turkey earthquake. The results of a large number of cyclic triaxial tests performed on "undisturbed" specimens of silty and clayey soils obtained from throughout the city are discussed. The tests show that the current state-of-the-art method for the determination of liquefaction susceptibility of fine-grained soils, i.e. the Chinese criteria, is not reliable. Soils that liquefied often met the liquid limit and liquidity index conditions of the Chinese criteria, but did not typically meet the clay-size condition. Test results do not show a marked effect of soil plasticity on cyclic strength of soils with $(N_1)_{60} < 10$ when PI < 12. However, 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.

1 INTRODUCTION

The strong shaking produced by the 1999 Kocaeli earthquake caused widespread damage throughout the northwest region of Turkey. Hundreds of buildings in Adapazari, an industrial city located approximately 120 km east of Istanbul, and about 7 km north from the fault rupture, were severely damaged. Additionally, areas of extensive ground failure occurred as evidenced by building settlement, sliding, and tilting. Many of the affected buildings had sand boils in their vicinity, although often ejecta were not present. The soils underlying the buildings that experienced ground failure were found to be primarily fine-grained (Bray et al. 2001).

The use of state-of-the-art methods for screening and identification of potentially liquefiable soils (as defined by the recent consensus paper by Youd et al., 2001) in Adapazari did not identify the problematic soils without modifying the commonly accepted "Chinese criteria" for fine-grained soils (Sancio et al., 2002). This observation generated the need for testing "undisturbed" soil specimens retrieved from sites where surface manifestations of ground failure were observed to establish the threshold, if any, for liquefaction triggering of plastic and non-plastic fine-grained soils.

2 SCREENING AND IDENTIFICATION OF LIQUEFIABLE SOILS

In recent years much research has been devoted to understanding the liquefaction susceptibility of fine-grained soils. Due to space limitations, only a limited review of relevant studies is presented.

Based on the data from sites where liquefaction was and was not observed after earthquakes in China, Wang (1979) established that any clayey soil containing less than 15% to 20% particles by weight smaller than 5 μm and water content (w_c) to liquid limit (LL) ratio greater than 0.9 is susceptible to liquefaction. Based on these data, Seed and Idriss (1982) 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. Due to its origin, this standard is known in the literature as the "Chinese criteria."

Koester (1992) 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. 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 (2000) reduced the LL condition for liquefaction susceptibility to < 32. Andrews and Martin 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. Moreover, Andrews and Martin 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. (1998) noted that a deposit of clayey soil might have contributed to the surface deformations observed at the Moss Landing site. They concluded that indiscriminate use of the Chinese criteria as a substitute for detailed laboratory and in-situ testing should be avoided. Similarly, Perlea (2000) recommends laboratory testing as the best way to evaluate the liquefaction susceptibility and post-cyclic undrained strength of cohesive soils.

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. Youd (1998) recommends the Chinese criteria as a generally conservative predictive tool. Additionally, he follows a series of assumptions to state that natural soil deposits that have a "C" descriptor (e.g. CH, CL, SC, and GC) are screened as nonliquefiable. Possibly liquefiable fine-grained soils should have LL < 35 and plot below the A-line or have PI < 7. Seed et al. (2001) 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". Undisturbed samples of soils that plot in that region of the plasticity chart should be obtained for laboratory testing. Polito (2001) 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 finally soils with 35 < LL < 50 and 10 < PI < 15 are susceptible to cyclic mobility.

The current state-of-the-art and state-of-the-practice for liquefaction study is established in the summary report from the 1996 NCEER and 1998 NCEER/NSF workshops on evaluation of liquefaction resistance of soils (Youd et al., 2001). Therein, the use of the Chinese criteria is recommended to confirm the liquefaction susceptibility of silts and clays.

3 SAMPLING AND TESTING PROCEDURES

3.1 Field sampling

"Undisturbed" soil samples were obtained using two different types of fixed piston samplers. The Geonor K-200 54-mm piston sampler (Andresen and Kolstad, 1979) was used initially. With this device, samples are obtained using thin walled steel tubes of constant internal diameter, ID = 54 mm, and outside diameter, OD = 57 mm. The tubes were pushed a length between 45 cm and 50 cm using a hydraulic jack mounted on the drilling machine. About two-thirds of the specimens tested were obtained using the Dames and Moore hydraulic piston sampler. This sampler 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, and the area ratio, defined by Hvorslev (1949) as $C_a = 100*(OD^2-ID^2)/ID^2$, is reduced from 11.4% for the steel tubes to 7.6% for the brass tubes. These effects reduce the degree of disturbance of the soil sample. 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.

3.2 Testing equipment

All tests in this study were performed using the Automated Triaxial Testing System (Li et al. 1988) in the geotechnical laboratory of ZETAS Corporation in Istanbul, Turkey. This testing system is a modification and extension of the CKC electropneumatic cyclic triaxial testing system described by Chan (1982). The instrumentation of this equipment includes five sensors: an external load cell and external LVDT to monitor the axial load and vertical displacement, respectively and three differential pressure transducers to measure the chamber pressure, the effective stress, and the volume change. The system is controlled by a computer through the Georobot software (v. 5.2), and allows closed-loop stress controlled cyclic loading at low frequencies, and open-loop control at 1 Hz.

A series of cyclic triaxial tests were first performed on Monterey sand following the methodology described by Silver et al. (1976), and the testing equipment and procedure were shown to produce results consistent with those previously published.

3.3 Specimen preparation and testing procedure

Trimming was avoided to minimize disturbance. However, 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 vice of a vertical extruder and cut at the desired length using a rotating tube cutter. The length was chosen so that the sample would have a height to width ratio between 2.1 and 2.5. To avoid radial deformation of the tube and possible disturbance to the soil during the cutting operation, 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 the specimen was cut at the bottom using a wire saw and carefully placed over the bottom platen of the triaxial cell. 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, the top drainage line was connected 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 the applied vacuum had consolidated the specimen, its height was measured with a dial gauge and its diameter 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 ($\varepsilon < 0.5\%$). The triaxial cell was then fully assembled and the chamber was filled with water. Sample saturation using differential vacuum was performed using the procedure described by Riemer et al. (1999) and followed by a new height measurement and installation into the triaxial testing equipment for backpressure saturation and cyclic loading. During consolidation of the specimens to the initial confining stress, the volumetric strain was monitored and found to be < 1%. According to the criteria used at the NGI for the assessment of specimen quality (Andresen and Kolstad, 1979), a test specimen is "very good to excellent" when the volume change due to consolidation to in-situ conditions is < 1%.

The soil specimens were saturated at isotropic confining stresses (σ'_{bp}) slightly smaller than the final consolidation stress, σ'_c (e.g. σ'_{bp} = 30 kPa, for σ'_{bp} = 40 kPa) to avoid exceeding σ'_c during B-value measurements. For all cases, B > 0.98 was achieved at relatively low backpressures (< 150 kPa), and all measurements were performed with a stress increment of 50 kPa.

Following saturation and consolidation to the desired confining stress, the cyclic test was 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 for the triaxial test is defined as the maximum peak shear stress divided by the initial effective consolidation stress, σ'_c. In general, one of three nominal CSRs (0.3, 0.4, and 0.5) was applied to a given specimen. The cyclic loading was applied until axial strains in excess of 4% were achieved. Figure 1 presents typical results for a cyclic test performed on non-plastic silt.

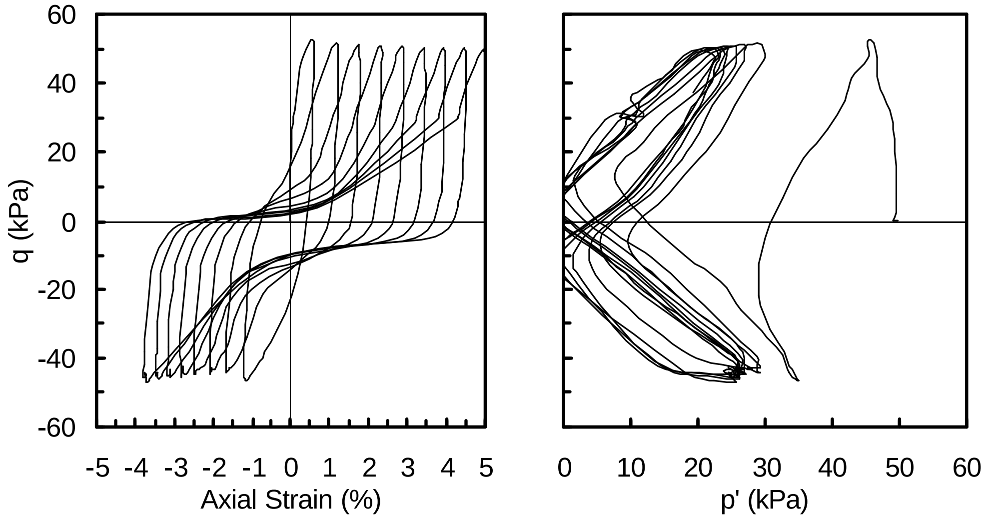

Figure 1 Typical results of an undrained cyclic triaxial test on non-plastic silt (LL = 23). This test was performed at σ'_c = 50 kPa and CSR = 0.48.

4 TESTING PROGRAM AND DISCUSSION OF TEST RESULTS

The primary objective of the testing program was to understand the causes of ground failure observed at building sites in Adapazari after the Kocaeli earthquake given that the in-situ tests showed that the foundation soils were predominantly silts and clays. Of interest was the fact that ground failure was prevalent adjacent to structures and less observed away from them (Bray et al. 2001).

The natural variability of the soil conditions in Adapazari allowed studying the effect of various factors, such as initial confining stress, plasticity, frequency of loading, and initial shear stress on the cyclic strength of silts, clayey silts, and clays. Overall, 100 cyclic triaxial tests and 20 static tests were performed on "undisturbed" soil specimens recovered at 7 different sites throughout the city. Figure 2a presents the location on the plasticity chart of the specimens tested under cyclic triaxial conditions, and Figure 2b depicts the relationship between the LL and water content for these soils. Tests were performed on soils with a wide variety of properties that cover a large range on the plasticity scale (0 < PI < 40). It should also be noted that all specimens with LL < 50 (PI ~ 20) have w_c/LL > 0.8. In this paper only the effect of soil plasticity and initial confining stress will be discussed for specimens tested under initial isotropic conditions using uniform cyclic loads at a frequency of 1 Hz.

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 and clays during a 1 Hz cyclic triaxial test. For this reason, it is 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 (compression or extension) axial strain. Results are similar if the 5% double amplitude axial strain criterion is used.

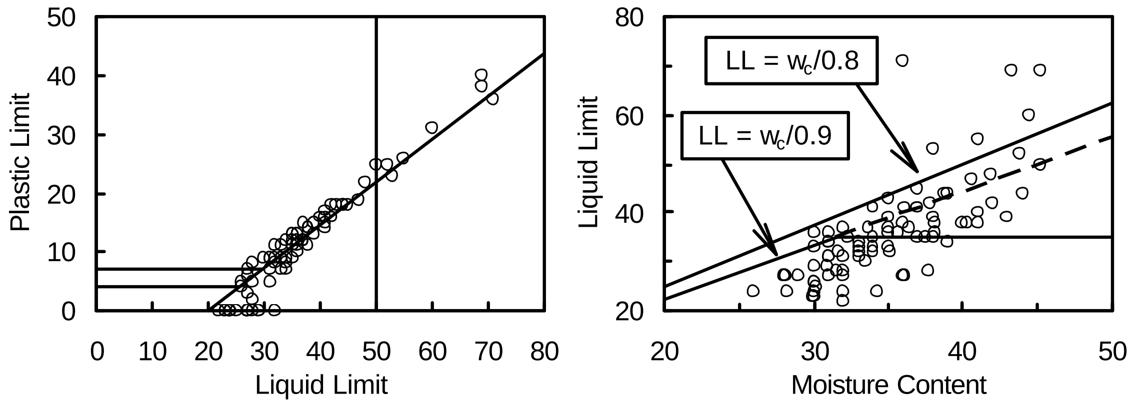

Figure 2 (a) Location on the Casagrande plasticity chart of the soils tested in this study, (b) relationship between the LL and moisture content of the soils.

4.1 **Effect of plasticity index on the cyclic strength**

The effect of soil plasticity was studied by comparing the results of specimens of different plasticity tested at the same confining stress. Figure 3a and 3b present the results of tests performed at σ'_c = 40 and 50 kPa, respectively. A distinction of the cyclic strength for soil specimens with PI ≤ 12 could not be discerned, and the results are therefore grouped in two categories, soils with PI ≤ 12 and soils with 12 < PI < 20. It is worth mentioning that most of the data from sites that liquefied in China on which the Chinese criteria (Wang, 1979) is based, have PI < 12 and w_c/LL > 0.85. Additionally, this is somewhat consistent with Ishihara (1996), 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 linearly increasing correction factors to soils with 10 < PI < 55. 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).

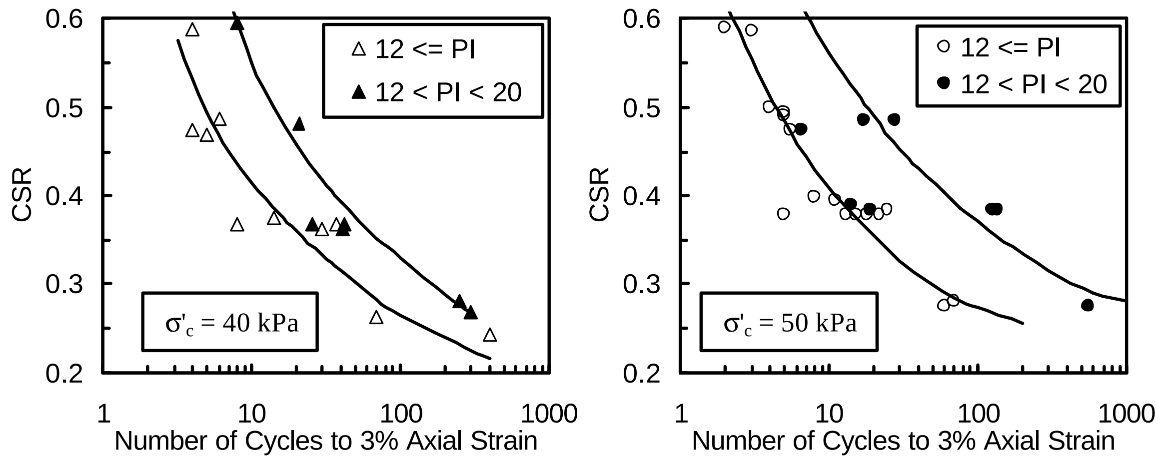

Figure 3 CSR versus number of uniform cycles to reach 3% axial strain for isotropically consolidated specimens to (a) 40 kPa and (b) 50 kPa, that have PI ≤ 12 (empty symbols) and PI > 12 , filled in symbols. Note that soils with PI > 20 did not generate significant pore pressures or axial strain.

The void ratio of the 26 specimens with PI ≤ 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 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. The normalized penetration resistances, $(N_1)_{60}$, in terms of the Standard Penetration Test (SPT) performed on the soil deposits tested in this study are consistently lower than 10.

4.2 Effect of initial isotropic confining stress

To assess the effect of the weight of the structure on the cyclic strength of the soil, a series of tests was performed at different initial isotropic stresses. Liquefaction analysis presented by Sancio et al. (2002) using state-of-the-art procedures delineated by Youd et al. (2001) show that the majority of the weak soil deposits at the sites of interest lie at depths generally between 2 and 5 m and have $(N_1)_{60} < 10$. This considered, a value of σ'_c = 25 kPa was chosen as representative of depths between 2 and 5 m 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 100 and 300 kPa.

The number of uniform stress cycles necessary to reach 3% single amplitude axial strain vs. the resultant CSR are shown in Figure 4a for specimens with PI ≤ 12 and $(N_1)_{60} < 10$. The proportionality of strength to initial confining stress can be noted from these results. From Figure 4b it is observed that in terms of CSR at 20 uniform cycles, specimens tested at σ'_c = 25 kPa are approximately 200% stronger than those tested at 100 kPa, and about 150% stronger than specimens tested at 40 to 50 kPa. The resulting K_σ correction factor, which results from the ratio of the CSR at a given confining stress by the CSR for σ'_c = 100 kPa at 20 cycles of loading, is presented in Figure 4b. As shown, the values obtained for the Adapazari soils with PI ≤ 12 is consistent with that recommended by Youd et al. (2001) for stress > 50 kPa when a value of f = 0.7 is used.

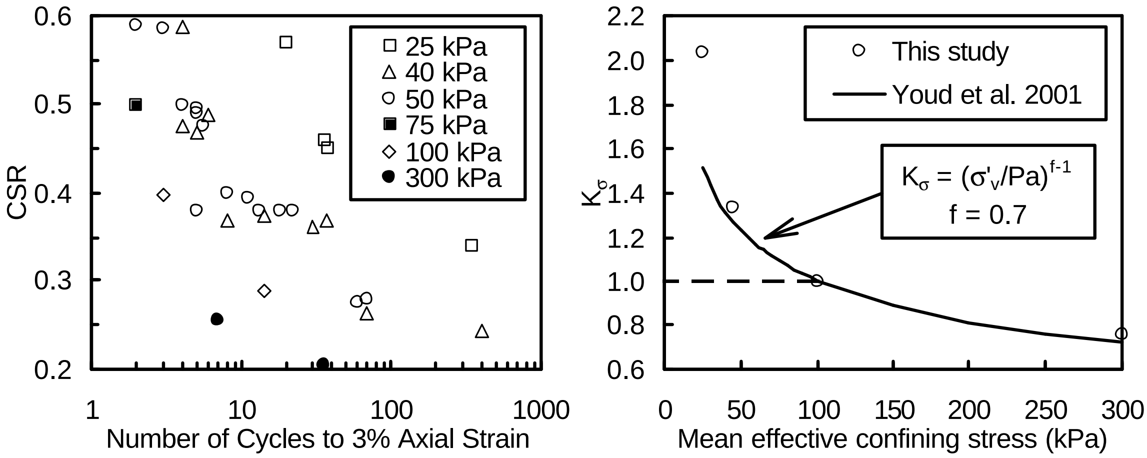

Figure 4 (a) Cyclic stress ratio (CSR) vs. number of cycles to reach 3% single amplitude axial strain for specimens of PI < 12 and $(N_1)_{60} < 10$, tested at 25, 40, 50, 75, 100 and 300 kPa, and (b) resulting confining pressure effect.

4.3 Evaluation of the Chinese criteria

Figure 5 has been developed to evaluate the effectiveness of the Chinese criteria in the evaluation of the liquefaction susceptibility of the specimens tested whose results are shown in Figures 3 and 4. In Figures 4a and 4b, circles are used to depict the properties of soils with PI ≤ 12, squares are used for soils with 12 < PI < 20, and triangles represent soils with PI > 20. If the Chinese criteria had been used, only about a fourth of the specimens (closed circles) would meet all three conditions. Conversely, all but four specimens of soil with PI ≤ 12 meet the LL and w_c/LL conditions of the Chinese criteria. Of these, two specimens do not meet the w_c/LL condition, and two others do not meet the LL condition, yet they still liquefied in the cyclic triaxial testing at a reasonable number of loading cycles. None of the soils with PI > 12 meet all three conditions of the Chinese criteria. These results indicate that the amount of particles smaller than the 5 or 2μm condition, is a poor indicator of liquefaction susceptibility of soils and should not be used.

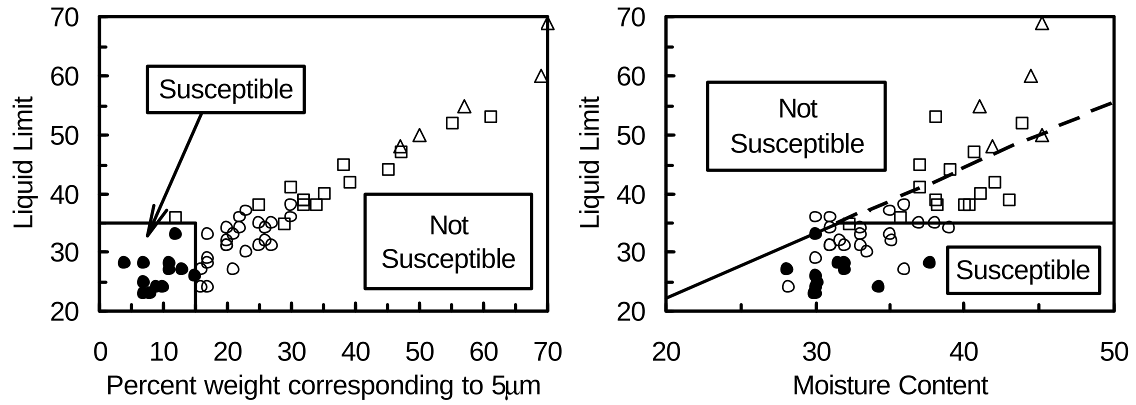

Figure 5 Graphical representation of the Chinese criteria. Specimens with PI ≤ 12 are shown as circles, specimens with 12 < PI < 20 as squares, and triangles represent soils with PI > 20. Filled in circles correspond to specimens that meet all three conditions of the Chinese criteria.

5 CONCLUSIONS

Based on a large number of cyclic triaxial tests performed on "undisturbed" specimens of Adapazari silt and silty clay of low penetration resistance [$(N_1)_{60}$ < 10], the following conclusions are drawn:

1. The Chinese criteria is not effective for the evaluation of liquefaction susceptibility of fine grained soils with PI ≤ 12, due primarily to the ineffectiveness of the condition on the amount of particles smaller the 5μm. Additionally, this testing program has shown that soils with PI > 12, which generally have LL > 35, can generate significant strains in a small number of cycles when a high CSR is applied.

2. Plasticity Index in combination with w_c/LL appear to be good indicators of soil liquefaction susceptibility. Fine grained soils from Adapazari with PI ≤ 12 and w_c/LL > 0.8 are susceptible to liquefaction, and soils of 12 < PI < 20 and w_c/LL > 0.8 should be tested in the laboratory to assess their liquefaction susceptibility.

3. A limited number of tests that were performed on specimens with PI > 20 with w_c/LL < 0.8, did not generate significant cyclic strains after a large number of cycles had been applied at initial confining stresses < 50 kPa.

4. The fact that ground failure in Adapazari was primarily observed adjacent to buildings may be attributed to, among other factors, the detrimental effect of an increase in confining stress on the cyclic strength when represented in terms of the cyclic stress ratio.

5. This testing program has shown that limiting $K_\sigma \leq 1.0$ may be overly conservative at low confining stress.

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