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Selman Sağlam
Middle East Technical University, Turkey

B. Sadık Bakır
Middle East Technical University, Turkey

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Selman Sağlam
Middle East Technical University,
Ankara, Turkey

B. Sadık Bakır
Middle East Technical University,
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ABSTRACT

Foundation failures observed over saturated silt-clay mixtures during past earthquakes clearly indicate the need for a profound understanding of the behavior of such soils under seismic loading. Although the mechanisms dominating the response of fine grained soils under seismic loading are known to be different from those of sandy soils, the behavior of low plasticity silt and clay is still under discussion. An experimental research program, still in progress, has been undertaken to investigate the cyclic behavior of low plasticity fine grained soils for developing useful guidelines for the assessment of seismic response. Samples of low plasticity silt, initially consolidated to stress levels above preconsolidation stress, have been tested systemically under monotonic and cyclic loading for isotropic and anisotropic stress conditions. To eliminate the sample variability inherent to the naturally deposited soils and to control the circumstances, the specimens were reconstituted by means of the slurry deposition technique in the study. The preliminary results from cyclic triaxial testing on reconstituted low plasticity silt specimens are presented. The liquefaction susceptibility of the silt was examined via comparisons to the existing empirical criteria in literature.

INTRODUCTION

Liquefaction of sands and associated damages to structures have long been recognized as a phenomenon to be the primary concern about the structures located over high seismicity regions. Occurrence of comparable damage at the sites underlain by silty soils during the earthquakes has led the researchers to focus on the seismic response of such soils, particularly in the last decade. The present study focuses on the cyclic response of silt subjected to different initial shear stress states, intended to simulate the response of the foundation soils beneath structures.

The difference between the seismic behaviors of isotropically and anisotropically consolidated soils has been one of the major interests of the researchers for the last three decades. Vaid and Chern (1983) suggested that the cyclic strength of loose sand was prone to decrease with increasing initial shear stress, while that of denser sand displayed the opposite trend. On the other side of the gradational scale, the cyclic strength of clay was reported to decrease with increasing initial shear stress (Hyodo et al., 1994; Lefebvre and Pfendler, 1996). Besides, Hyodo et al. (1994) reported that the pore pressure ratio at failure was observed to decrease with increasing initial shear stress. This is due to fact that, depending on the intensity of the sustained initial shear stress and hence the reduced cyclic strength, the development of pore pressure is limited. Although little research has been carried out on the cyclic characteristics of the silts, Soong et al. (2004) suggested that the cyclic strength of the materials having silt content ranging between 65 and 95% was observed to decrease with increasing initial shear stress as similar to the behavior of clay. Hyde et al. (2006), however, showed that increasing initial shear stress caused a tendency of increase in cyclic strength of silt after the ratio between the initial shear stress and mean normal effective stress had reached a value of 0.5. In this study, to investigate the influence of the initial shear stress on the behavior of silt, the reconstituted specimens were consolidated isotropically and anisotropically and were subjected to monotonic and load controlled cyclic traixial tests under undrained conditions.

The main purpose of the testing program is to provide a systematic and controlled study on the dynamic behavior of the fine grained soils. This study presents the details of the testing program and its primary conclusions, and in the regard with these findings, assessments related to liquefaction susceptibility of the silt are involved according to the current liquefaction susceptibility criteria of fine grained soils.

CURRENT PROCEDURES FOR EVALUATING LIQUEFACTION SUSCEPTIBILITY OF FINE GRAINED SOILS

The evaluation of liquefaction susceptibility of fine grained soils began with the criteria introduced on the basis of the data observed after large earthquakes in China (Wang, 1979). Seed and Idriss (1982) interpreted Wang's findings and stated that clayey soils meeting the conditions of (a) percent of particles less than 0.005 mm < 15%, (b) liquid limit (LL) < 35, and (c) the ratio of initial water content (w_c) to the LL (w_c/LL) > 0.9 could be sensitive to severe strength loss as a result of seismic loading. The conditions as a whole are known as the "Chinese Criteria".

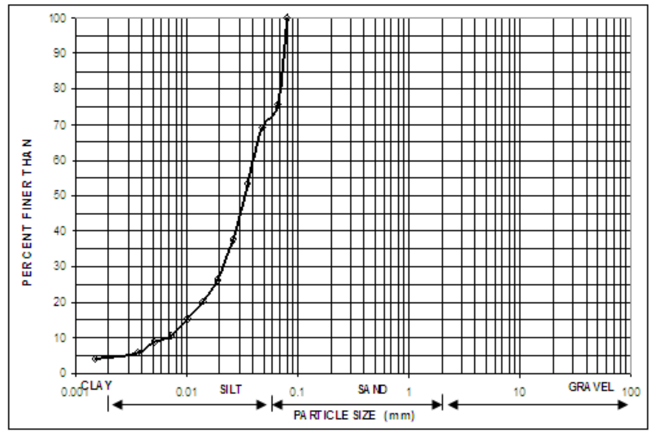
Andrews and Martin (2000) reviewed the observations of a few earthquake case histories, and discussed the properties of the soils that were documented as liquefied. Clay content and LL were regarded as the "key" parameters in liquefaction susceptibility evaluation of silty soils, and it was concluded that the soils are susceptible to liquefaction if they have LL < 32 and clay content < 10%, and are not susceptible if they have LL ≥ 32 and clay content ≥ 10% both. If the soil meets only one of the conditions mentioned above, further studies were suggested by Andrews and Martin.

Bray et al. (2004) proposed that fine grained soils are susceptible to liquefaction or cyclic mobility if the ratio of w_c/LL ≥ 0.85 and the plasticity index (PI) ≤ 12. Nevertheless, the soils satisfying the conditions of 0.8 ≤ w_c/LL ≤ 0.85 and 12 < PI ≤ 20 have been introduced as moderately susceptible to liquefaction or cyclic mobility.

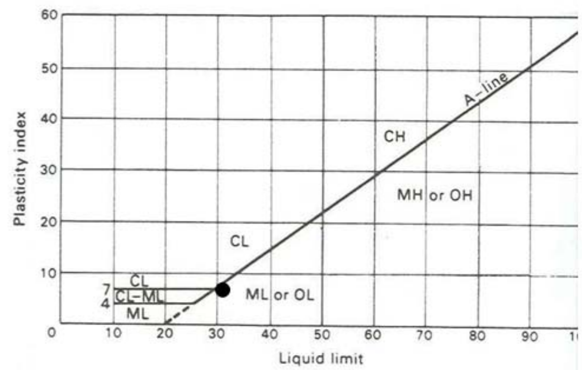
Boulanger and Idriss (2004) stated that the evaluation of liquefaction potential of the fine grained soils depended on the behavior characteristically dominated by either clay (clay-like behavior) or sand (sand-like behavior). Fine grained soils with PI < 7 have been classified as "sand-like" (i.e. susceptible to liquefaction), and soils with PI ≥ 7 have been classified as "clay-like". Boulanger and Idriss (2006) later proposed that if a soil plots on the plasticity chart as CL-ML the PI criterion may be reduced to PI ≥ 5, and the soils with PI values of 3-6 are better to be tested in-situ and in laboratory in addition to liquefaction correlations based on standard penetration test (SPT) and cone penetration test (CPT).

MATERIAL TESTED

The silt used in this research was supplied in powdered form from Balad, Iraq. The grain size distribution was determined using sieve and hydrometer analyses. As it can be seen in Fig. 1, the material consisted of 68.5% silt, 4.5% clay, and 27% fine sand. The specific gravity of the material is 2.69 and the Atterberg limits are as follows: LL=31; PL=24; PI=7. The liquid limit (LL) was determined by means of Casagrande cup. Therefore, LL value can be taken as 35 according to the corrections proposed by Koester (1992) when considering Chinese liquefaction assessment criteria. According to the Unified Soil Classification System (USCS), the material is classified as low plasticity silt (ML), and the plot on plasticity chart, which is shown by a dot in Fig. 1.b, is adjacent to the A-line.



(a)



(b)

Fig. 1. (a) Grain size distribution curve, (b) plot on plasticity chart.

LABORATORY TESTING

A series of laboratory tests comprising isotropically and anisotropically consolidated undrained monotonic and cyclic triaxial tests were conducted on reconstituted specimens. Test procedures and results are described in the following sections.

<u>Reconstitution Procedure</u>

Sample reconstitution technique has been reported to have great effect on the behavior of sands during testing (Mulilis et al., 1977; Tatsuoka et al., 1986; Yamamuro and Wood, 2004). However, the information related to the effect of reconstitution method on the behavior of fine grained soils is very limited. Accordingly, commonly utilized reconstitution methods were considered. The need of saturated specimens, and lack of large amount of material exposed restricts to utilization of most of the reconstitution techniques. The slurry deposition technique is, therefore, seemed as the most convenient method to obtain saturated silt specimens.

The material was initially mixed with de-aired water of an amount required to obtain an homogeneous slurry with a water content of about 2 to 3 times of the liquid limit. The slurry was then placed into a cubic box inside of which was covered with a woolen tissue. The box having dimensions of 19.5, 19.5 and 21 cm and little holes on the top and bottom cover was then mounted in a container having dimensions of 32, 32 and 25 cm. The container was filled up with de-aired water such that the box was remaining totally under the water. The slurry was deposited by consolidation under 40 kPa vertical pressure which was imposed by means of a pneumatic piston.

Undrained Monotonic Triaxial Tests

A series of undrained monotonic and cyclic triaxial tests were conducted over reconstituted samples to examine the static and cyclic response of the specimens, and to identify any conceivable relation between them. The specimens were extracted from the reconstitution box by means of the samplers of 36 mm in diameter and 100 mm in height. Then, they were trimmed to 71 mm height cylinders before being mounted on the device. Specimens were saturated under a back pressure up to 200 kPa, and a pore pressure coefficient (B) value of at least 0.95 was provided. Consolidation stresses applied before undrained shearing were chosen as to be representative of the stress conditions for a soil element beneath common shallow foundations. Undrained monotonic loading was applied with an axial strain rate of 0.07%/min, which was ascertained sufficiently slow (due to the method proposed by Germaine and Ladd, 1988) to ensure pore pressure equilibrium in a silty material. Stresses applied during monotonic compression tests performed are presented in Table 1.

Prior to testing, the specimens were first subjected to certain confining pressures (σ'_{3c}), and then the axial stress was increased incrementally by allowing drainage of the specimen until a particular stress state was reached. Variation of the axial stress applied at this stage imposed an initial sustained deviator stress ($\Delta\sigma_i$) of 0, 30, 50 and 60 kPa. To remove the influence of initial stress state, the deviator stress ($\Delta\sigma$) is normalized by initial mean effective stress (p'_i). Initial shear stress ratio, defined as q_s/p'_i where q_s is the initial shear stress ($\Delta\sigma_i/2$), ranged between 0 and 0.34. It should be noted that $q_s/p'_i=0$ condition refers to the isotropic consolidation.

The monotonic test results are presented in Figure 2 in the form of plots of mean effective stress (p') versus deviator stress ($\Delta\sigma$), where $p' = [\sigma'_1+\sigma'_3]/2$ and $\Delta\sigma = [\sigma_1-\sigma_3]$. After an initial contractive response up to an axial strain between 0.2% and 1.2%, the specimens exhibited a dilation upon continued shearing.

Table 1. Undrained monotonic triaxial tests

Test	σ'_{1c} (kPa)	σ'_{3c} (kPa)	q_s/p'_i	e_i	w (%)	w/LL
S1	50	50	0.00	0.76	26.60	0.86
S2	80	50	0.23	0.75	26.51	0.86
S3	100	50	0.33	0.73	25.74	0.83
S4	120	60	0.33	0.77	27.18	0.88
S5	80	80	0.00	0.78	27.51	0.89
S6	90	60	0.20	0.72	25.41	0.82

σ'_{1c} : Major effective principal stress
σ'_{3c} : Minor effective principal stress
q_s : Initial shear stress
p'_i : Initial mean effective stress
q_s/p'_i : Initial shear stress ratio
e_i : void ratio at the beginning of shearing
w (%) : Water content at the beginning of shearing
w/LL : The ratio between water content and liquid limit (LL)

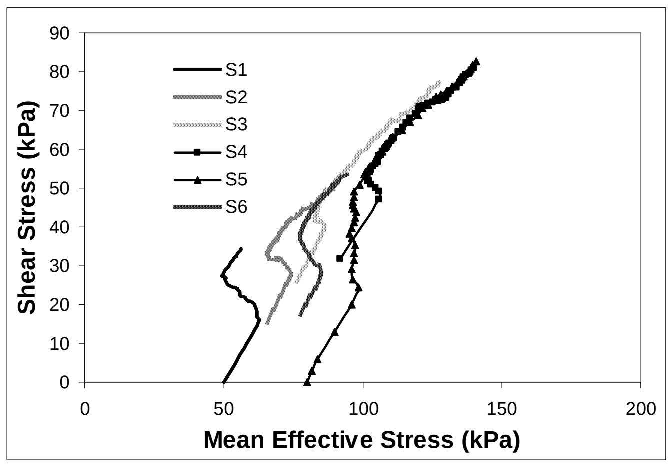


Fig. 2. Monotonic stress paths

Stress-strain behavior from undrained monotonic tests is shown in the from of plots given in Fig. 3. As it can be observed, there is a point of flexure at very low strains (0.1~0.6 %) for the tests. This was followed by hardening to the axial strains up to 10% without a peak is reached. It is observed that the undrained strength tends to increase with the increasing initial shear stress. Examining the stress paths and the measured pore water pressure, the maximum excess pore pressure generated during shearing is observed to decrease with increasing initial shear stress ratio (q_s/p'_i) as indicated in Fig. 4.

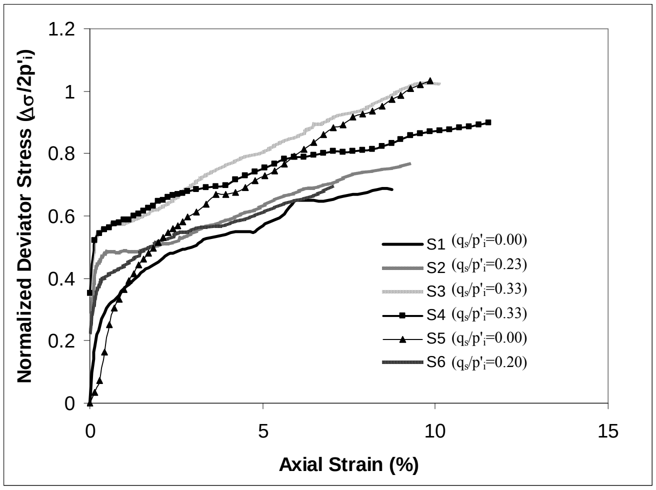


Fig. 3. Monotonic stress-strain behavior

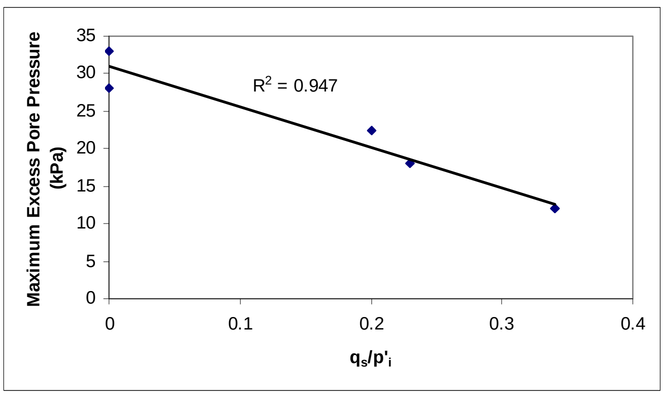


Fig. 4. Relationship between generated maximum excess pore pressure and initial shear stress ratio for monotonic tests

<u>Undrained Cyclic Triaxial Tests</u>

The specimens prepared using slurry deposition technique were isotropically and anisotropically consolidated under specific stresses to achieve prescribed stress conditions. The cyclic phase was then conducted in a load controlled manner with different triaxial cyclic stress ratios ($CSR_{tx}= \Delta\sigma_{cyc}/p'_i$, where $\Delta\sigma_{cyc}$ is single amplitude cyclic deviator stress) ranging between 0.3 and 0.72. Stresses applied during the cyclic tests are presented in Table 2. The void ratio following the initial consolidation phase is denoted by e_i. The frequency of the cyclic loading was set to 0.5 Hz, which is considered to be representative of the typical frequency range for earthquake loading. Corresponding number of cycles (N) needed to reach the axial strains of 3%, 5% and 10% in each test are displayed in Table 3. In the case of stress reversal double amplitude (DA) axial strains, and for non reversal stress conditions single amplitude (SA) axial strains were taken into consideration to calculate N.

Table 2. Undrained cyclic triaxial tests

Test	σ'_{1c} (kPa)	σ'_{3c} (kPa)	q_s/p'_i	CSR_{tx}	e_i	w/LL
C1	50	50	0.00	0.35	0.74	0.840
C2	50	50	0.00	0.30	0.74	0.843
C3	50	50	0.00	0.55	0.75	0.849
C4	50	50	0.00	0.60	0.75	0.851
C5	45	45	0.00	0.72	0.72	0.820
C6	75	50	0.20	0.32	0.74	0.835
C7	75	50	0.20	0.48	0.72	0.812
C8	75	50	0.20	0.64	0.75	0.846
C9	75	50	0.20	0.64	0.75	0.852
C10*	95	50	0.31	0.31	0.75	0.851
C11	95	50	0.31	0.48	0.75	0.848
C12	90	50	0.29	0.64	0.74	0.844
C13*	120	60	0.33	0.36	0.75	0.857
C14*	120	60	0.33	0.44	0.77	0.875
C15	120	60	0.33	0.53	0.74	0.839
C16	80	80	0.00	0.31	0.75	0.854
C17	80	80	0.00	0.41	0.74	0.838
C18	80	80	0.00	0.50	0.75	0.848
C19	80	80	0.00	0.59	0.74	0.842
C20	80	80	0.00	0.59	0.73	0.825
C21	80	80	0.00	0.59	0.73	0.833
C22	90	60	0.20	0.33	0.74	0.841
C23	90	60	0.20	0.50	0.75	0.850
C24	90	60	0.20	0.60	0.72	0.821
C25*	150	50	0.50	0.38	0.75	0.851
C26*	150	50	0.50	0.50	0.74	0.845

***** Number of cycles were calculated due to SA axial strains

σ'_{1c} : Major effective principal stress
σ'_{3c} : Minor effective principal stress
q_s : Initial shear stress
p'_i : Initial mean effective stress
q_s/p'_i : Initial shear stress ratio
e_i : void ratio at the beginning of shearing
w/LL : The ratio between water content and liquid limit (LL)

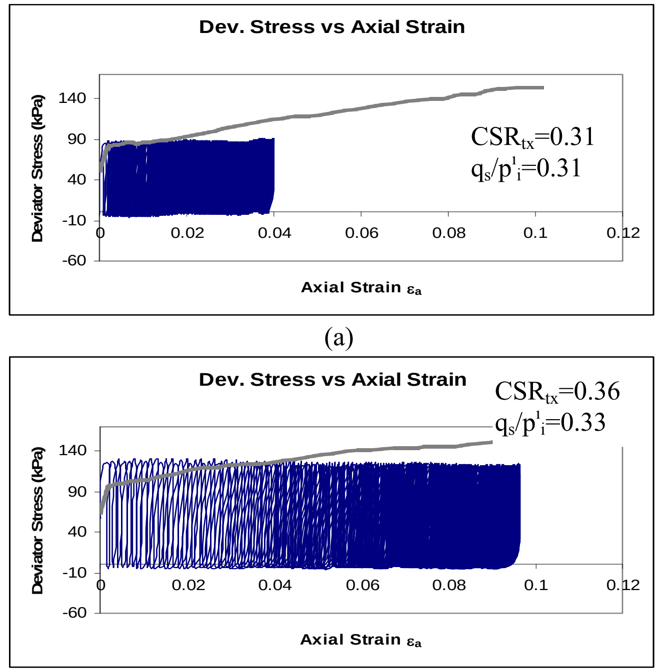
Cyclic response is observed to depend on whether the specimens are subjected to stress reversal or not during cyclic loading. In the case of no stress reversal, plastic strains accumulate with almost a constant rate for each cycle. As it can be seen in Fig. 5, the plastic strain accumulation rate tends to decrease after having reached the point where the peak cyclic stress becomes lower than the monotonic strength. The greater the ratio of the applied peak cyclic stress to the monotonic strength, the greater strain accumulation rate is observed, which is consistent with the observed trends cited in literature (Andersen et al., 1980; Yilmaz et al., 2004). No significant cyclic degradation was observed in stiffness of the specimens under loading without stress reversals although the axial strains exceeded 5%.

Table 3. Number of cycles and excess pore pressure ratio reached at the axial strains of 3%, 5%, and 10%.

	Number of Cycles (N)			Excess Pore Pressure Ratio, ru				
Test	3% Ax.St.	5% Ax.St.	10% Ax.St.	3% Ax.St.	5% Ax.St.	10% Ax.St.	Max. Ax.St. (%)	N at Max. Ax.St.
C1							0.6	220
C2	130	150		0.92	1.00		4.1	147
C3	1	7	21	0.16	0.63	1.13	12.0	49
C4	2	3	5	0.40	0.58	0.95	18.0	14
C5	13	25		0.50	0.73		4.4	25
C6							0.5	360
C7	12	21	34	0.46	0.57	0.71	13.0	45
C8	2	3	6	0.23	0.34	0.62	11.0	9
C9		1	5		0.31	0.54	12.0	5
C10*	110			0.48			4.0	217
C11	2	4		0.16	0.31		6.7	9
C12		1	4		0.28	0.47	10.7	5
C13*	22	54	300	0.39	0.48	0.55	10.0	300
C14*	4	7	12	0.29	0.33	0.52	30.0	23
C15	1	2	4	0.10	0.20	0.38	17.0	13
C16	50	56	68	0.62	0.72	0.92	10.0	68
C17	4	8	12	0.25	0.52	0.74	17.0	19
C18	2	3	4	0.15	0.27	0.37	21.0	19
C19		1	3		0.28	0.38	12.0	5
C20		1	2		0.27	0.47	14.5	4
C21		1	2		0.27	0.49	15.0	4
C22	110	134	154	0.67	0.74	0.80	13.0	160
C23	2	3	8	0.16	0.25	0.58	13.0	10
C24		1	2		0.17	0.32	23.0	10
C25*	43	116		0.24	0.29		5.5	147
C26*	5	10	35	0.15	0.21	0.27	30.0	83

* Number of cycles were calculated due to SA axial strains

In the case of stress reversal, the strain accumulation is predominant either in compression or in extension depending on the initial stress state. In the tests conducted with isotropically consolidated specimens, the incremental strains developed in each additional cycle are added to the maximum past strains in compression and extension respectively. The strain accumulation rate becomes more remarkable in extension with increasing N, which is attributed to lower strength of soils in extension.



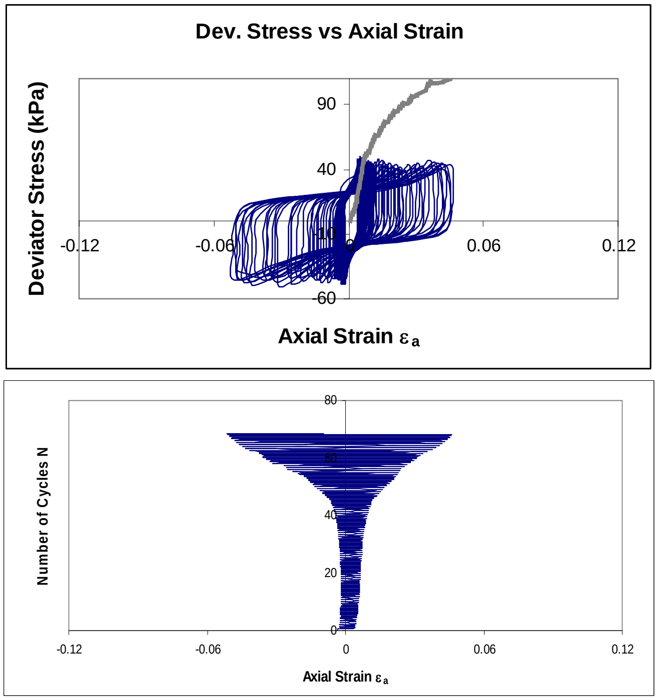
(b)

Fig. 5. Stress-strain behavior for the case of non-reversal stress conditions

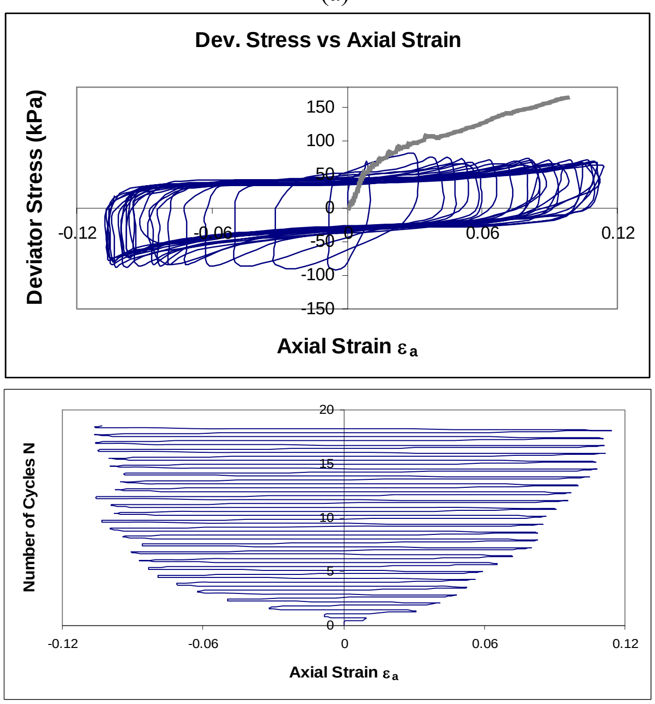
Figure 6 shows the representative test results of isotropically consolidated specimens subjected to different CSR_{tx} and p'_i values. As it can be seen in Fig. 6, axial strain accumulation rate increases for the specimen subjected to CSR_{tx} value of 0.31. There is no sudden increase in strain to be interpreted as flow liquefaction. At relatively high CSR_{tx} of 0.5, strain accumulation rate decreases with increasing N although the strain accumulates particularly in the earlier cycles of loading. In all isotropically consolidated cyclic tests, a high loading rate of 0.5 Hz was used. The r_u values were, however, observed to reach, and even exceed, 0.9 at different cycles. Due to the variations in the strains where r_u values build up in the order of 0.9, there is no common finding to be generalized or interpreted as a failure criterion in terms of either strain or pore water pressure. Although the excess pore water pressure reaching initial confining stress led to the loss in effective stress and thus to cyclic strain softening, the dilative tendency of the silt prevented excessive loss of strength.

In the tests with anisotropically consolidated specimens subjected to higher cyclic demands (higher CSR_{tx} values), the stress reversals also occurred. Figure 7 shows the representative test results of anisotropically consolidated specimens. The strains predominantly accumulate in the compression side of loading. Accumulation rate is increasing in relation with the quantity of peak stress exceeding monotonic strength and essentially with increasing CSR_{tx}. The pore water pressure ratio r_u generated in the specimen having an initial shear stress ratio q_s/p'_i of 0.2 approaches to 0.8 whereas that approaches to 0.5 for q_s/p'_i of about 0.3.

However, except C6, DA axial strains of the tests exceed 10% regardless of r_u values.

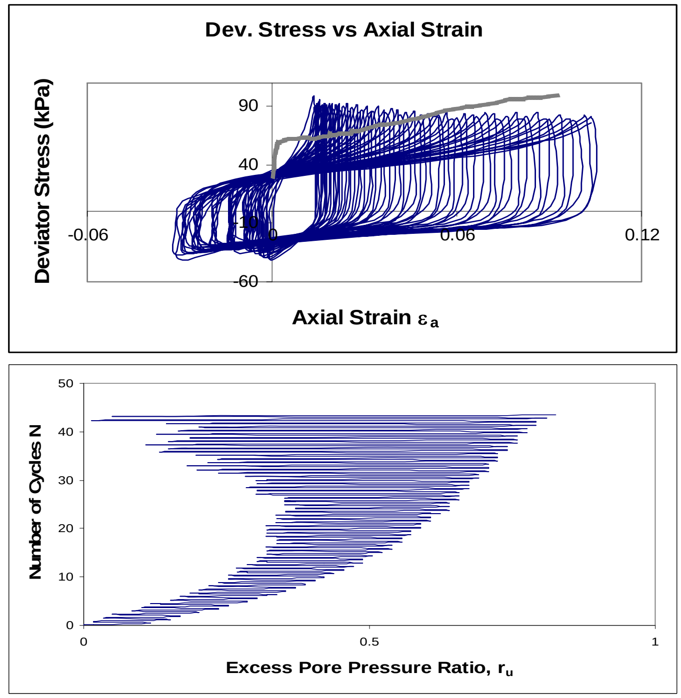


(a)

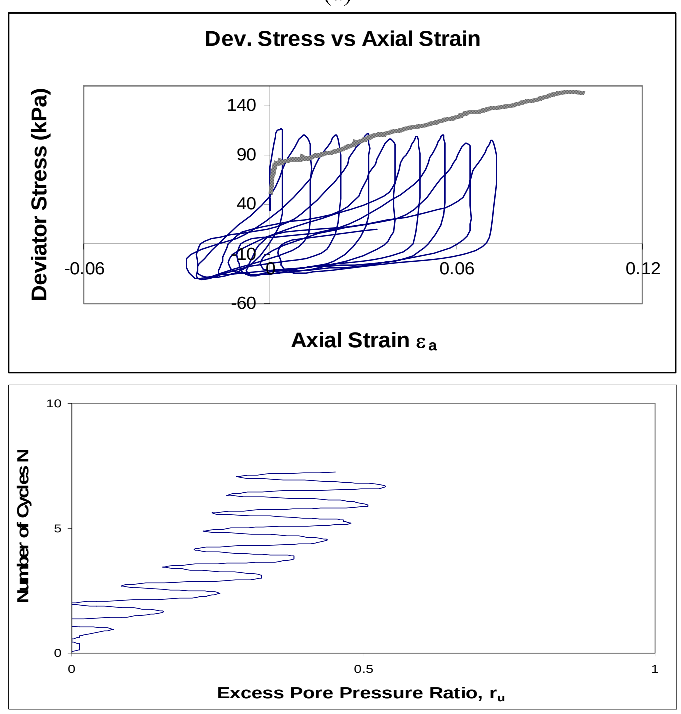


(b)

Fig. 6. Stress-strain behavior of the specimens subjected to(a) CSR_{tx} of 0.31 and p'$_i$ of 80 kPa, (b) CSR_{tx} of 0.50 and p'$_i$ of 80 kPa



(a)



(b)

Fig. 7. Stress-strain behavior and pore water pressure generation in the specimens subjected to (a) CSR_{tx} of 0.52 and q_s/p'_i of 0.2, (b) CSR_{tx} of 0.48 and q_s/p'_i of 0.31

EFFECT OF INITIAL SHEAR STRESS

The results of the cyclic tests are evaluated as a function of the number of cycles (N) needed to reach axial strains of 5% DA stress reversal and 5% SA for non-reversal cases. The relationship between CSR_{tx} and N required to achieve 5% axial strain is presented in Fig. 8. Isotropically consolidated specimens are observed to require the lowest cyclic deviator stress to reach 5% axial strain for a given N. Although the CSR_{tx} tends to increase as the q_s/p'_i increases from 0 to 0.5, the q_s/p'_i of 0.2 found to have a greater CSR_{tx} than that of 0.3 for a given N. The observed tendency of cyclic strength to increase with increasing initial shear stress is identical with the data for sands having relative densities greater than 50% as reported by Ishihara (1996).

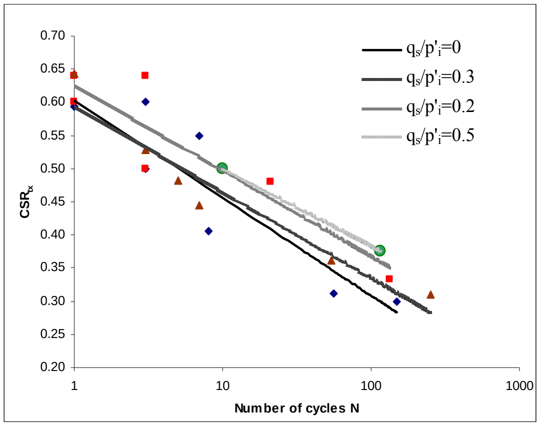


Fig. 8. Relationship between CSR_{tx} and number of cycles to reach 5% axial strain

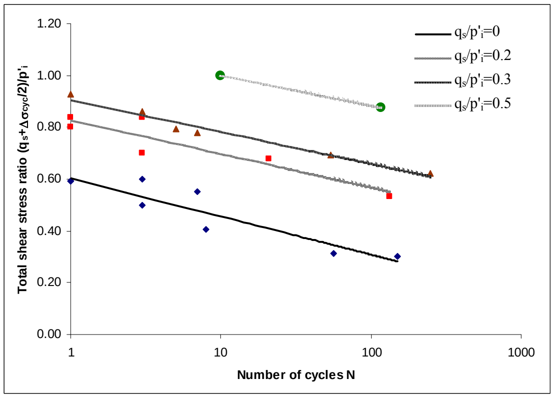


Fig. 9. Relationship between total shear stress ratio $(q_s+(\Delta\sigma_{cyc})/2)/p'_i$ and number of cycles to reach 5% axial strain

The increase in strength becomes more evident when the results are interpreted in terms of maximum total undrained shear stress $(q_s+[\Delta\sigma_{cyc}]/2)$. In Fig. 9, it is seen that the total shear stress ratio required to reach an axial strain of 5% increases with a given N. The specimens having q_s/p'_i of 0.3 are observed to have a greater total undrained shear stress ratio than those having 0.2.

LIQUEFACTION SUSCEPTIBILITY EXAMINATION OF THE SILT VIA EXISTING EMPIRICAL CRITERIA

Chinese criteria is represented graphically in Fig. 10 for the soils having 0.005 mm and smaller particle sizes less than 15%. The percent of particles less than 0.005 mm is about 9 % for the silt used in the study. The corresponding parameters for the tested 26 specimens of the reconstituted silt are plotted on the same figure. It is to be mentioned that the modification for LL suggested by Koester (1992) is applied for the silt. As it is observed in Fig. 10, the silt plots on the "not susceptible to liquefaction" side. The assessment of the criteria meets the test results when considering no observed flow liquefaction. However, this assessment can be regarded as somewhat conservative when considering the response of cyclic mobility that has frequently occurred during the cyclic tests. It is important to note that the points of the silt plot very near to the boundary defining potential susceptibility to liquefaction.

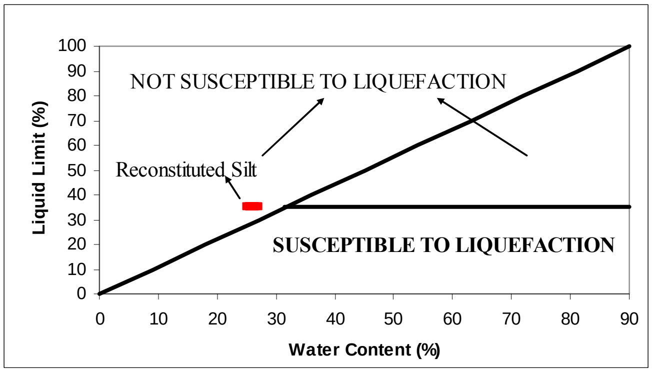


Fig. 10. Chinese criteria for the soils having 0.005 mm and smaller particle sizes less than 15 %.

The parameters of the reconstituted silt are also plotted on the graphical representation of the criteria proposed by Andrews and Martin (2000). As it is seen in Fig. 11, even though the silt plots in the area defined as "susceptible to liquefaction", the parameters are almost on the border of the boundary between "susceptible" and "not susceptible" sides.

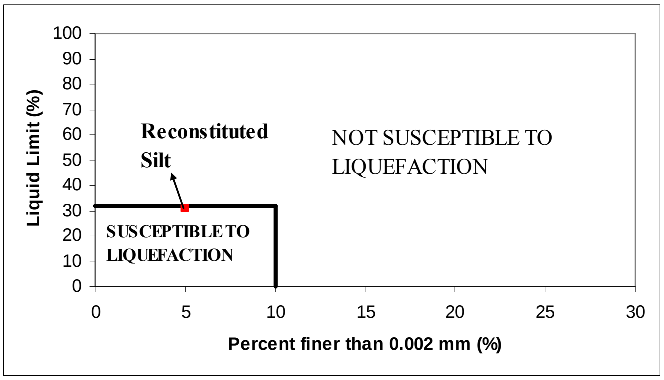


Fig. 11. Liquefaction susceptibility criteria proposed by Andrews and Martin (2000)

As similar to the above, the test parameters for the reconstituted silt are plotted with respect to the criteria proposed by Bray et al. (2004) (Fig. 12). The silt is classified as in the range from “moderately susceptible to liquefaction or cyclic mobility” to “susceptible to liquefaction or cyclic mobility”. This assessment is in conformity with the “cyclic mobility” observed for the reconstituted silt during cyclic triaxial tests. The area defined as “moderately susceptible to liquefaction or cyclic mobility” gives a room for evaluation of liquefaction susceptibility of borderline materials, especially for low plasticity silts. However, the criteria do not provide a clear distinction between the phenomena of “liquefaction” and “cyclic mobility”.

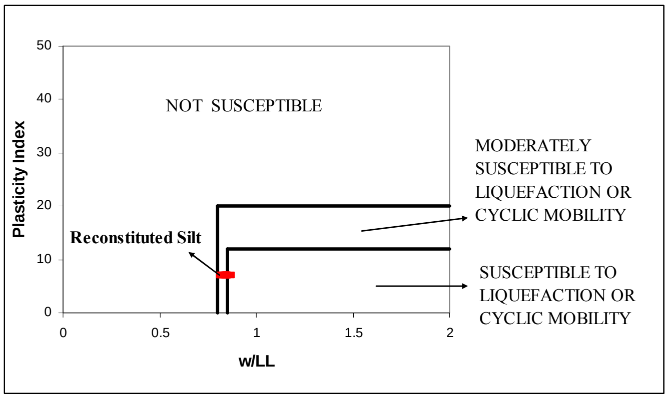


Fig. 12. Liquefaction susceptibility criteria proposed by Bray et al. (2004)

Boulanger and Idriss (2004) suggested that fine-grained soils of PI < 7 have been classified as “sand-like” (i.e. susceptible to liquefaction) and soils of PI ≥ 7 classified as “clay-like” materials. In accordance with this characterization, the silt of PI = 7 would be classified as “clay-like” material. However, it is expected to pay attention on the accuracy of the measured parameters which becomes quite critical during liquefaction susceptibility evaluation of such borderline materials.

CONCLUSIONS

Cyclic loading response of low plastic silt originated from Iraq has been investigated by means of a series of tests comprising isotropically and anisotropically consolidated undrained monotonic and cyclic triaxial tests. The specimens of the silt were obtained using the reconstitution method of slurry deposition technique. The slurry was deposited by consolidation under 40 kPa vertical pressure which was sufficient to obtain specimens able to stand freely under their own weight.

The monotonic tests were conducted to identify any conceivable relation between static and cyclic response of the specimens. The specimens were loaded with various axial stress levels to obtain initial shear stress ratios (q_s/p'_i) ranging between 0 and 0.33. These initial stress states were arranged to be the same with those of cyclically loaded specimens. In the tests with no stress reversal, the plastic strain accumulation rate tends to decrease after having reached the point where the peak cyclic stress becomes lower than the monotonic strength. Greater strain accumulation rate is observed with the increasing ratio of the applied peak cyclic stress to the monotonic strength.

The cyclic tests were conducted with the specimens consolidated under specific stresses to achieve prescribed initial stress conditions. Triaxial cyclic stress ratios (CSR_{tx}) ranging between 0.3 and 0.72 were utilized with the frequency of 0.5 Hz. Cyclic response is observed to depend on whether the specimens are subjected to stress reversal or not during cyclic loading. In the case of no stress reversal plastic strains are observed to accumulate with almost a constant rate for each cycle. In the tests conducted with isotropically consolidated specimens, axial strain accumulation rate increased gradually for lower CSR_{tx} values. On the other hand, for higher CSR_{tx} values, strain accumulation rate decreased with increasing N, although the strain accumulates remarkably in the earlier cycles of loading. The r_u values were observed to reach and/or exceed 0.9 at different cycles. In the tests with anisotropically consolidated specimens, accumulation rate increased in relation with the quantity of peak stress exceeding monotonic strength and essentially with increasing CSR_{tx}. Although the axial strains exceeded 10%, r_u generation is in the order of 0.5-0.8.

Due to the variations in the strains where r_u values build up in the order of 0.9, there is no common finding to be generalized or interpreted as a failure criterion in terms of either strain or pore water pressure. Although the excess pore water pressure reaching initial confining stress led loss in effective stress and thus cyclic strain softening, the dilative tendency of the silt prevented excessive strength loss.

The cyclic strength of the silt is observed to increase with increasing initial shear stress. The observed tendency is identical with the data for sands having relative densities greater than 50% as reported by Ishihara (1996).

Liquefaction susceptibility of the silt was examined by the existing criteria. According to the Chinese criteria, the silt falls into the category defined as “not susceptible to liquefaction” which is slightly conservative when considering the response of cyclic mobility which was frequently observed during cyclic tests. On the other hand, the parameters of the silt plot on the side of “susceptible to liquefaction” according to the criteria proposed by Andrews and Martin (2000). Therefore, it can be concluded that the criteria with sharp distinction between the conditions of liquefaction susceptibility can be conservative to some extent for borderline materials. The silt plots mostly in the area of “moderately susceptible to liquefaction or cyclic mobility” in the graphical representation of Bray et al. (2004) criteria. This assessment is in conformity with the cyclic mobility observed during the tests. Due to the diversity in evaluation of liquefaction susceptibility of such borderline materials, the accuracy of soil parameters becomes a critically important aspect of these screening tools.

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