SOME OBSERVATIONS RELATED TO LIQUEFACTION SUSCEPTIBILITY OF SILTY SOILS

Upul ATUKORALA[1], Dharma WIJEWICKREME[2] And Norman MCCAMMON[3]

SUMMARY

The liquefaction susceptibility of silty soils has not received the same level of emphasis as sandy soils. This paper presents some observations related to liquefaction susceptibility of silty soils based on the analysis of data from seven sites. The data includes cyclic triaxial and cyclic simple shear tests conducted on undisturbed samples of soils and index and grain size data on the same samples. The applicability of the commonly used Chinese criteria for the assessment of liquefaction susceptibility of fine-grained soils such as silts is examined using the laboratory test results. The characteristic cyclic loading behaviour of silty soils is compared with that of sandy soils, and parameters that are required for the assessment of deformations are identified and their anticipated range in values discussed.

INTRODUCTION

Deltaic deposits typically consist of fine-grained overbank and floodplain sediments, in some instances, interbedded with sandy soils. In regions of high seismic risk, the liquefaction susceptibility of these fine-grained soils is often a concern in the design of foundations. Although liquefaction susceptibility of sand and gravel has been investigated with an increased focus, the liquefaction potential and post-liquefaction behaviour of fine-grained soils such as silts has not been given the same level of emphasis.

The commonly accepted practice to assess the liquefaction resistance of sandy soils is to use the empirical liquefaction resistance charts[1,2] that have been developed based on penetration resistance measurements obtained using the SPT or CPT methods. These charts are generally applicable for soils containing up to 35% fines passing the US Sieve No. 200 (0.074 mm size). In soils containing significant amounts of fines, the accepted practice is to use the "Chinese Criteria" that have been established based on index properties and grain size data.

For projects warranting detailed assessment, it is common to undertake laboratory cyclic simple shear or cyclic triaxial tests on "undisturbed" soil samples to assess the liquefaction susceptibility and to obtain data on the post-cyclic stress-strain response of soil. In this regard, unlike in sand, relatively undisturbed samples of silt can generally be obtained for laboratory triaxial or simple shear testing.

It is generally believed that the simple shear test closely simulates the in-situ stress conditions both prior to and during earthquake shaking. However, due to unavailability of simple shear devices, cyclic triaxial tests are often carried out to determine the cyclic resistance of soils. It is common to apply a correction to the results of cyclic triaxial tests to account for the stress path effects either using site-specific correlations or published data.

Laboratory test results have indicated that the strain development and pore pressure generation characteristics of silty soils are generally different from those of sandy soils[3]. Typical results of pore pressure and strain development in silt and sand published by Singh (1996), plotted as a function of the number

1 Principal, Golder Associates Ltd., Vancouver, Canada
2 Principal, Golder Associates Ltd., Vancouver, Canada
3 Associate, Golder Associates Ltd., Vancouver, Canada

of cycles of loading, are shown in Figure-1. For a given level and intensity of cyclic loading, the following main differences are noted:

The rate of generation of cyclic loading-induced excess pore water pressure is generally slower in silt than in sand.

The developed maximum excess pore water pressure is less than the initial effective confining stress in silt whereas in sand it reaches 100% of the initial effective confining stress and remains at this level during subsequent loading.

The strain development in silt with the number of cycles is gradual. In sand, although strain development is initially slower than in silt, it increases rapidly after developing an excess pore pressure of about 70% of the initial effective confining stress and ultimately leading to liquefaction and softening of the sample.

The above differences in behaviour are important in geotechnical engineering practice, where one of the primary concerns is the ability of foundation soils to sustain the applied loads without undergoing bearing capacity failure and/or excessive deformations. Estimating foundation deformations in this regard require an understanding of the post-liquefaction stiffness and strength of soils. When the expected foundation movements are excessive, costly ground improvement measures have to be implemented to reduce, but not eliminate, the seismic loading-induced movements.

This paper presents laboratory cyclic shear test results on silty soils compiled from seven sites. The liquefaction susceptibility of the subject soils is assessed using the commonly used Chinese criteria and the results are compared with the outcome of laboratory test results on the same samples. Important differences in the cyclic loading behaviour of silty and sandy soils are identified and compared. Parameters that are important in the assessment of liquefaction-induced deformations in silty soils are identified and their anticipated ranges in magnitude are discussed.

CHINESE CRITERIA ON LIQUEFACTION SUSCEPTIBILITY OF FINE-GRAINED SOILS

According to the Chinese criteria4, which were originally published in 1979, a fine-grained soil is considered to be susceptible to liquefaction, when all of the following four criteria are satisfied:

1st	Liquid Limit (wl)	<	35%
2nd	Liquidity Index (Il)	>	0.75
3rd	Natural Water Content (wn)	>	0.9 x Liquid Limit
4th	Percent Passing 0.005 mm Sieve Size	<	15%

The Liquid Limit of a soil is an indirect measure of the water content or void ratio (density) that corresponds to a specified undrained shear strength5 of 2 to 2.5 kPa, while the Liquidity Index is an indirect measure of the sensitivity of the soil. The first three criteria, therefore, identify weak sensitive fine-grained soils. The fourth criterion, which is a measure of the clay content of the soil, eliminates the medium to high plastic soils that do not have the ability to undergo volume change during repeated application of cyclic loads. Since exclusively based on index properties and grain size data, the Chinese criteria are independent of the intensity and duration of the applied loads.

It has been observed that the cyclic resistance of fine-grained soils increases with the Plasticity Index6 depending on the nature of fines contained. In general, however, the increase observed is marginal or non-existent in soils with a Plasticity Index up to 10%.

DATA BASE ON SILTY SOILS

A data base has been developed over the past 8 to 10 years which consists of laboratory cyclic simple shear and triaxial test results compiled from seven sites. The sites correspond to various projects the authors were involved in where the liquefaction susceptibility of soils and the associated consequences were considered a major concern for the geotechnical design of foundations. In all cases, undisturbed Shelby tube soil samples taken from the soil strata of concern were subjected to anticipated in-situ and then cyclic stresses in the laboratory under controlled conditions. For ease of presentation of data and confidentiality reasons, the sites are labeled as Site-1 through Site-7.

Data from a total of twenty-two samples/tests were available from all of the sites. Five of the tests were conducted in the cyclic simple shear device and the remainder were conducted in the triaxial device.

The gradation envelope of the samples tested in the laboratory is shown in Figure-2. A majority of the samples consisted of 60 to 90% silt sizes (passing US Sieve Size 200) with 5 to 30% sand sizes and 10 to 30% clay sizes. As can be seen from Figure-2, all gradations generally fall within a relatively narrow band.

Table-1 summarizes the descriptions, depths and index properties of the soil samples tested from each of the seven sites:

Table 1: Summary of Soils Tested

Site	Soil Description	Depth (m)	Wn (%)	Wl (%)	Wp (%)	% Finer Than 0.005 mm	Test #
1	SILT, Trace Clay and Sand	14	37	39	29	13	1a
	SILT, Some Clay and Sand	25	38	34	23	22	1b
2	SILT, Some Sand & Clay	2	45	31	21	26	2a
	SILT, Some Sand & Clay	2	-	-	-	-	2b
3	SILT, Some Clay & Sand	11	30	28	18	42	3a
	SILT, Some Clay & Sand	11	-	-	-	-	3b
	SILT, Some Clay & Sand	11	30	28	18	46	3c
	SILT, Some Clay & Sand	11	-	-	-	-	3d
	SILT, Some Clay & Sand	11	-	-	-	-	3e
4	SILT, Some Clay, Trace Sand	9	28	20	27	22	4a
5	SILT Clayey	33	32	25	21	35	5a
	CLAY Silty	45	31	27	20	43	5b
6	SILT Sandy, Some Clay	6	43	53	43	23	6a
	SILT & SAND	6	41	49	41	21	6b
	SILT Sandy, Some Clay	6	38	48	38	24	6c
	SAND Silty, Trace Clay	11	35	40	35	6	6d
	SAND Silty, Trace Clay	11	-	-	-	-	6e
	SAND Silty, Trace Clay	11	-	-	-	-	6f
	SILT & SAND, Trace Clay	20	42	47	42	9	6g
	SILT & SAND, Trace Clay	20	-	-	-	-	6h
7	SILT Organic, Some Clay	8	94	87	50	42	7a
	SILT Organic, Some Clay	5	175	253	139	46	7b

A summary of the test variables and key results is presented in Table-2. In the case of triaxial tests (denoted by TX), the effective consolidation pressure refers to the isotropic consolidation stress. In the case of simple shear tests (denoted by SS), the effective consolidation pressure refers to the initial vertical effective stress. Similarly, the peak cyclic strain and post-cyclic peak strain refer to shear strains in the case of the simple shear tests and axial strains in the case of triaxial tests.

The cyclic stress ratios for testing were determined based on wave propagation analyses conducted using the computer program SHAKE or Seed simplified method of analysis assuming a representative horizontal ground surface acceleration. For Site-3 and Site-5, a number of samples were tested at varying cyclic stress ratios to determine the complete liquefaction resistance curve as a function of the number of cycles of loading. In general, however, the number of samples tested per site varied from one to three. Some of the tests were carried out for the purpose of confirming the available cyclic resistance of a specific soil layer.

Table 2: Summary of Test Variables and Key Results

Test #	Effective Consolidation Pressure (kPa)	Test Type	Cyclic Stress Ratio	Total Number of Cycles	Peak Cyclic Strain at N=15	Post Cyclic Su/p'	Post Cyclic Peak Strain (%)	Peak Excess Pore Pressure Ratio N=15
1a	150	SS	0.15	40	1.0	0.35	21	0.48
1b	315	SS	0.11	50	0.5	0.22	11	0.25
2a	135	TX	0.26	153	0.3	0.56	13	0.59
2b	135	TX	0.33	39	1.0	0.38	4	0.74
3a	225	TX	0.20	50	0.1	-	-	0.31
3b	225	TX	0.24	47	0.3	-	-	0.71
3c	225	TX	0.31	9	2.7	0.39	10	0.97
3d	225	TX	0.34	6	2.5	-	-	0.80
3e	225	TX	0.29	37	0.9	0.41	6	0.58
4a	105	SS	0.20	5	5.4	0.31	16	0.73
5a	255	SS	0.12	15	0.5	0.23	4	0.20
5b	320	SS	0.11	15	0.5	0.23	4	0.20
6a	65	TX	0.28	10	2.6	0.61	12	0.85
6b	65	TX	0.34	2	4.7	0.52	14	0.94
6c	65	TX	0.25	18	2.3	1.07	13	0.88
6d	105	TX	0.20	30	0.3	0.81	14	0.58
6e	105	TX	0.22	10	4.5	0.62	10	0.90
6f	105	TX	0.27	22	2.0	0.62	12	0.90
6g	200	TX	0.30	1	4.5	0.30	10	0.85
6h	200	TX	0.22	30	0.5	0.35	9	0.70
7a	50	TX	0.24	16	0.4	0.54	7	0.20
7b	50	TX	0.04	16	0.01	0.40	8	0.15

In all cases, sinusoidal cyclic loading was applied to the samples at a frequency of 0.1 to 0.2 Hz. The stress-strain or soils has been observed to be generally independent of the frequency of load application. A majority of the samples were subjected to post-cyclic monotonic loading to establish the stress-strain variations of soil that has liquefied or softened. In the case of triaxial testing, post-cyclic loading was applied in both the compression and extension modes to examine the effects of stress path.

Typical cyclic stress-strain and pore-pressure response variations observed for Sample #6a are shown in Figure-3. This particular test was carried out in two stages. In the first stage, the applied cyclic stress ratio of 0.19 was too low to cause liquefaction in 15 cycles of loading. Note that the strain development in the second stage with a stress ratio of 0.28 is more or less uniform with the cycles of loading and that the maximum pore pressure ratio developed at a cyclic strain of 2.5% is close to 0.9. Figure-4 shows the stress-strain variations observed during the cyclic and post-cyclic loading in the extension mode. The observed "S" shaped curves are typical of soils that have developed high excess pore pressures due to cyclic loading. The post-cyclic stress-strain curve consists of an initially soft/weak region up to a strain of about 2%, followed by a stiffer response as the sample tends to dilate as a result of further shearing. The stress-strain variations observed for Sample #6c where the post-cyclic loading was carried out in the compression mode are shown in Figure-5. As in the extension mode, the stress-strain curve approached the "S" shape with the development of excess pore pressures. However, in this case, the post-cyclic stress-strain response was both stiffer and stronger than in the extension mode.

ANALYSIS OF DATA

Liquefaction Susceptibility - Chinese Criteria vs Laboratory Results

The results of liquefaction susceptibility of the soils, assessed based on the Chinese criteria and laboratory test results, are summarized in Table-3.

Table 3: Summary of Liquefaction Assessment Results

Test #	Based on Chinese Criteria					Based on Laboratory Tests		
	1st Criterion	2nd Criterion	3rd Criterion	4th Criterion	Liquefiable?	Cyclic Strain	Excess PP Ratio	Liquefiable ?
1a	No	Yes	Yes	Yes	No	1.0	0.48	No
1b	Yes	Yes	Yes	No	No	0.5	0.25	No
2a	Yes	Yes	Yes	No	No	0.3	0.59	No
2b	Yes	Yes	Yes	No	No	1.0	0.74	No
3a	Yes	Yes	Yes	No	No	0.1	0.31	No
3b	Yes	Yes	Yes	No	No	0.3	0.71	No
3c	Yes	Yes	Yes	No	No	2.7	0.97	Yes
3d	Yes	Yes	Yes	No	No	2.5	0.80	Yes
3e	Yes	Yes	Yes	No	No	0.9	0.58	No
4a	Yes	Yes	Yes	No	No	5.4	0.73	Yes
5a	Yes	Yes	Yes	No	No	0.5	0.20	No
5b	Yes	Yes	Yes	No	No	0.5	0.20	No
6a	No	No	No	No	No	2.6	0.85	Yes
6b	No	No	No	No	No	4.7	0.94	Yes
6c	No	No	No	No	No	2.3	0.88	Yes
6d	No	No	No	No	No	0.3	0.58	No
6e	No	No	No	Yes	No	4.5	0.90	Yes
6f	No	No	No	Yes	No	2.0	0.90	Yes
6g	No	No	Yes	Yes	No	4.5	0.85	Yes
6h	No	No	Yes	Yes	No	0.5	0.70	No
7a	No	Yes	Yes	No	No	0.4	0.20	No
7b	No	No	No	No	No	0.01	0.15	No

It is interesting to note that according to the Chinese criteria, none of the soil samples would classify as susceptible to liquefaction. However, if only the first and second criteria are considered, eight samples (1b, 2a, 2b, 3a, 3b, 4a, 5a, 5b) out of the twenty-two samples would be classified as susceptible to liquefaction (see Figure 6). If only the first and fourth criteria are considered, six samples (1a, 6d, 6e, 6f, 6g, 6h) out of the twenty-two samples would be considered liquefiable (see Figure-7).

When assessing the results of laboratory tests, it is common to consider that samples that develop an excess pore pressure ratio of 1.0, or a peak single amplitude shear strain (simple shear tests) of 5%, or a peak double amplitude axial strain (triaxial tests) of 5%, to have liquefied. The data presented in Table-3 indicate that nine samples (3c, 3d, 4a, 6a, 6b, 6c, 6e, 6f and 6g) out of the twenty two samples tested satisfy either the pore pressure or the peak strain criterion, and have hence liquefied.

The above analysis of results indicates that the Chinese criteria may not be always be valid in the assessment of liquefaction susceptibility of silty soils. Being independent of the applied dynamic loads and being dependent only on the index properties and grain size data of soils, the Chinese criteria appear to provide inaccurate predictions particularly for soils in the geographic regions of medium to high seismicity.

CORRECTION OF TEST DATA

In order to reflect expected in-situ cyclic simple shear conditions, cyclic triaxial test results are corrected to account for stress path effects. The cyclic triaxial stress ratio (i.e. $\sigma dcy/ \sigma' 3c$) is the ratio of the maximum shear stress on an inclined plane to the isotropic confining pressure rather than the shear stress on the horizontal plane to the vertical initial effective stress (i.e. $\tau h / \sigma' v$) used in the cyclic simple shear test. The correction is normally applied to the cyclic stress ratio as:

$$\tau_h / \sigma'_v |_{SS} = C \bullet \sigma_{dcy} / \sigma'_{3c} |_{TX} \qquad [1]$$

For sand, it is reported that C varies from 0.6 to 0.9 depending on the lateral coefficient of earth pressure at rest7. The authors are unaware of any published data for silts. In various projects, the authors have used values ranging from 0.8 to 1.0 based on verbal communications with leading researchers. This range of values is within the variation of 0.8 to 0.85 established for fine to medium sands with a D50 of 0.15 to 0.2 mm based on comparison of simple shear and triaxial test results carried out in the laboratory8.

POST-CYCLIC STRESS-STRAIN-STRENGTH VARIATIONS FOR DEFORMATION ANALYSES

The cyclic loading-induced strains occur in the relatively flat segments of the "S" shaped stress-strain curves shown previously in Figures 4 and 5. This behaviour is typical for samples tested in both triaxial and simple shear devices. Ground deformations estimated from double integration of the ground surface acceleration time-histories recorded at the Wildlife[9] Site during the 1987 Superstition Hill earthquake confirm this aspect of behaviour of liquefied soils. Modeling of the softer response is therefore important when computing the liquefaction-induced patterns and magnitudes of deformations/strains.

For sandy soils, the accepted practice is to establish the post-liquefaction stress-strain response is to estimate the residual shear strength and limiting shear strain based on SPT N values. Often, the response is assumed to be bi-linear. For silty soils, however, no such data base exists and the geotechnical engineers have to resort to site-specific laboratory test results.

The results previously presented in Table-2 indicate that for silty soils, the post-cyclic residual shear strength and limiting shear strain are highly variable generally ranging from of 0.22 to 0.50 and 3 to 20%, respectively. Due to this wide range in variation, and in the absence of other data, we believe that site-specific post-liquefaction characteristics should be established for silty soils for projects requiring detailed stress-strain modelling of soil behaviour. It is however noted that the above observed range in residual shear strength and limiting shear strain values are within the range of values reported for fine to medium sand by other researchers7.

SUMMARY AND CONCLUSIONS

Chinese Criteria based on index test results and grain size data are often used to assess the liquefaction susceptibility of fine-grained soils. Laboratory test results compiled from seven different sites underlain by silty soils indicate that the Chinese criteria may not always be valid when assessing the liquefaction susceptibility of silty soils. Analysis of the data confirm that soils that do not satisfy all four of the Chinese criteria (to be classified as liquefiable) are capable of generating large excess pore pressures, soften, and undergo large strains when subjected to cyclic stress ratios that are representative of medium to high seismic shaking.

The cyclic behaviour of silty soils is seen to be different from that of sandy soils. The pore pressure and strain development in silty soils occur gradually with the increasing number of cycles, whereas in sandy soils, the pore pressure and strain development occurs rapidly during the last few cycles of loading. Therefore, sudden or "brittle" collapse of foundation soils appears to be unlikely in silty soils subjected to seismic shaking.

Similar to sandy soils, the cyclic stress-strain variations of silty soils tend to form "S" shaped curves with flat segments when they develop high excess pore water pressures and soften. The large liquefaction-induced oscillatory ground movements or strains occur as a result of these flat segments in the stress-strain curves. Accurate modelling of the post-cyclic stress-strain behaviour is important when assessing ground movements. For silty soils, the available data on post-cyclic stress-strain parameters is limited or non-existent. The data collected for this study indicate that the post-cyclic undrained shear strength and limiting shear strain for silty soils exhibit a high degree of variability and that site-specific laboratory testing should be carried out to establish these parameters until further data becomes available.

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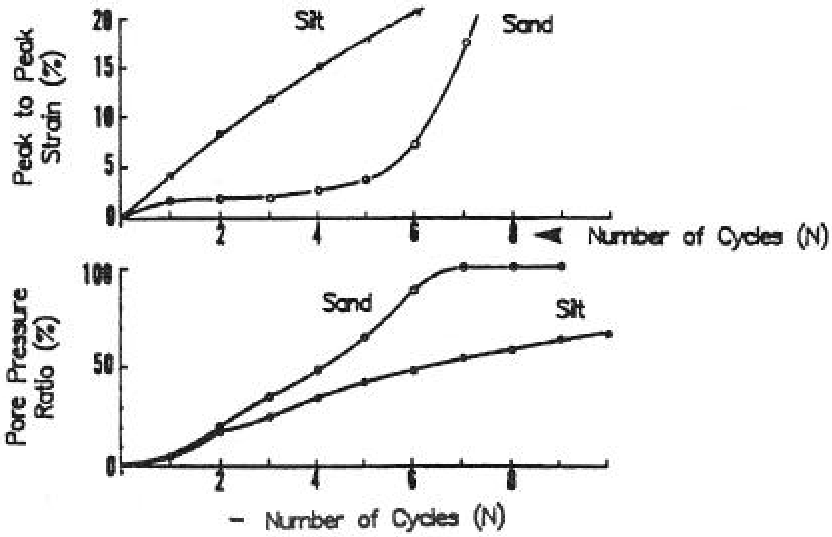


Figure 1.Comparison of Strain and Pore Preassuredevelopment in a typical Silt sample (after Singh, 1996)

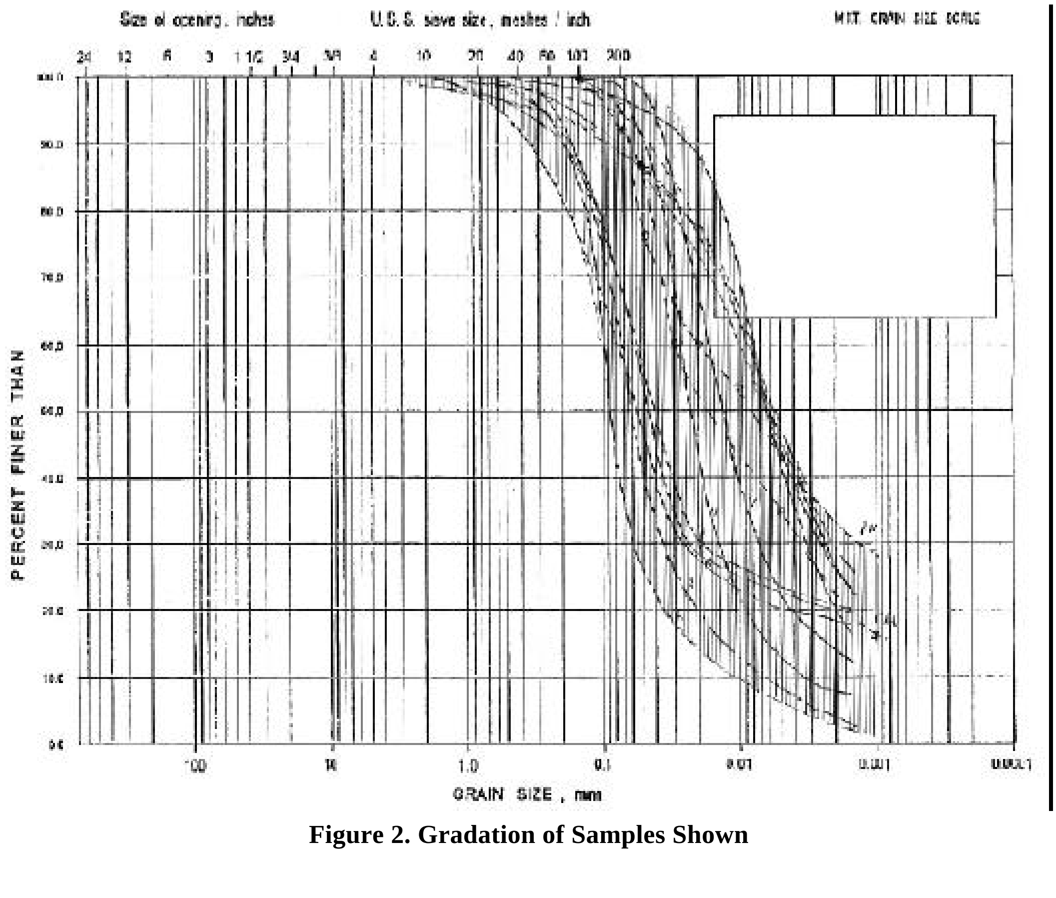


Figure 2. Gradation of Samples Shown

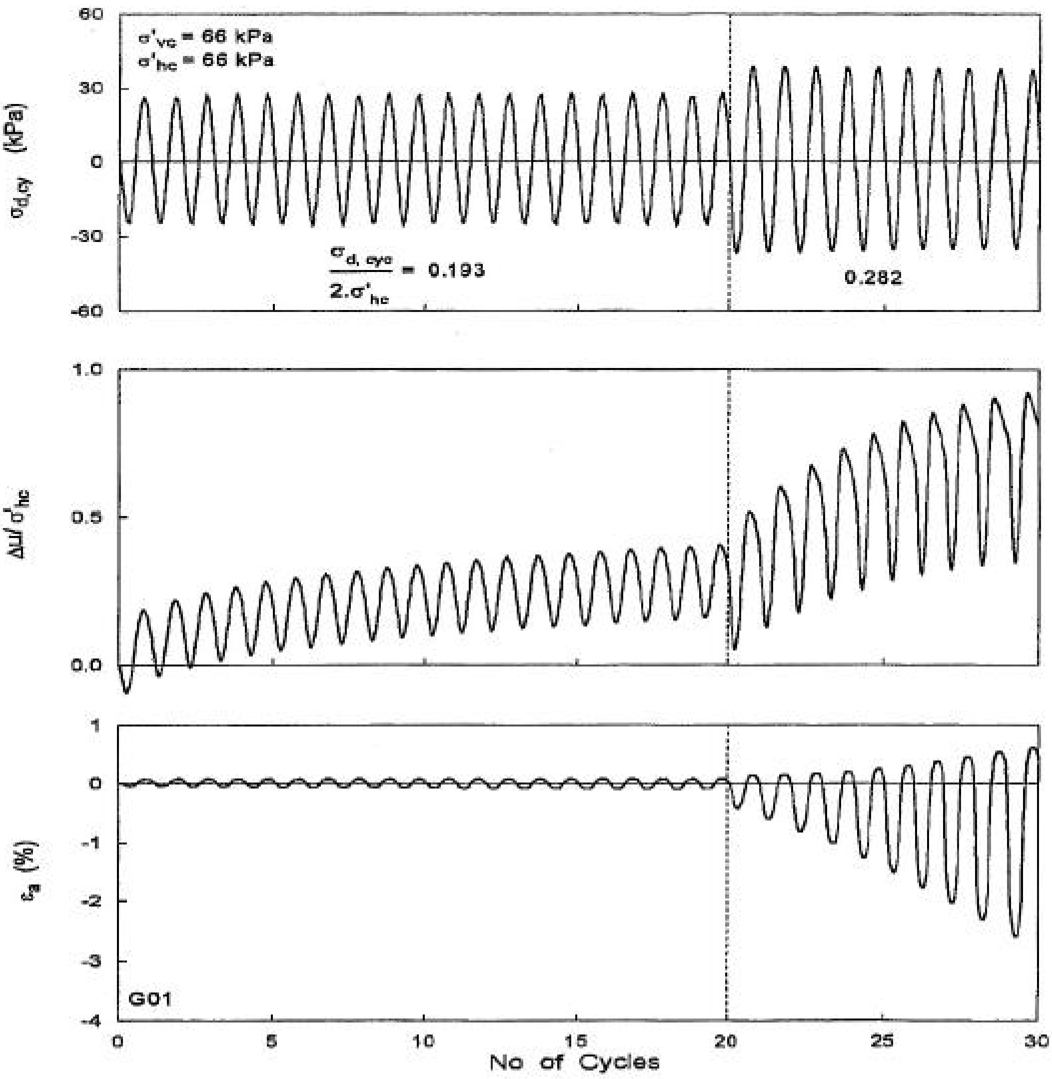
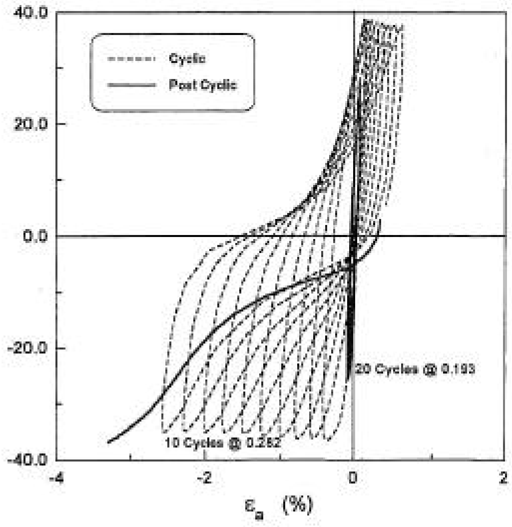


Figure 3. Results of Cyclic Testing of sample #6A



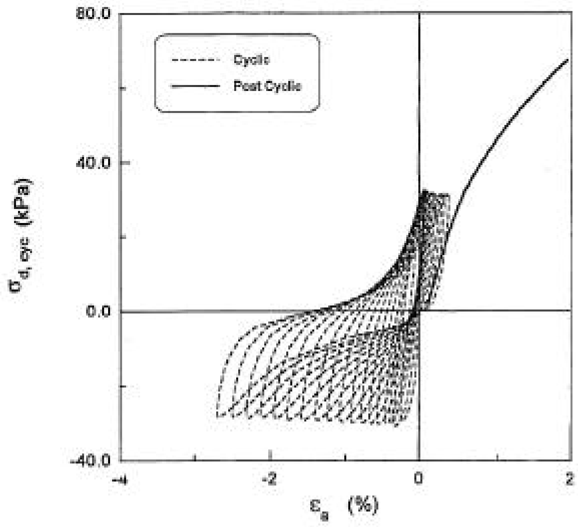


Figure 4. Cyclic and Post-Cylclic stress-strain Results for sample #6A

Figure . Cyclic and Post-Cylclic stress-strain Results for sample #6A