26 May 2010, 4:45 pm - 6:45 pm

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Fifth International Conference on
Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics
and Symposium in Honor of Professor I.M. Idriss
May 24-29, 2010 • San Diego, California

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ABSTRACT

Cyclic threshold shear strain, γ_t, is small cyclic shear strain amplitude above which soil properties significantly change with the number of cycles, N, and below which such changes are for all practical purposes negligible. To date, three cyclic threshold shear strains have been experimentally verified: for cyclic settlement (cyclic compression), for residual cyclic pore water pressure, and for cyclic stiffening. Subject of the paper is testing of fourth cyclic threshold shear strain for cyclic degradation, γ_{td}. When fully saturated soil is subjected in undrained conditions to moderate or large cyclic strain-controlled loading, its secant shear modulus, G_s, decreases with N. This is quantified for given cyclic shear strain amplitude, γ_c, by degradation index, $\delta = G_{sN} / G_{s1}$, where $G_{sN} = G_s$ at cycle N. Index δ and N are related via degradation parameter $t = -(\log\delta / \log N)$ which measures the rate of cyclic degradation. At $\gamma_c < \gamma_{td}$ there is no cyclic degradation and $t = 0$. If $\gamma_c > \gamma_{td}$ cyclic degradation takes place and $t > 0$. With a special simple shear device for small-strain testing the variation of t with γ_c was examined and γ_{td} evaluated for three clayey soils. Results show that γ_{td} increases with plasticity index, *PI*. For *PI*=12 γ_{td}=0.015%, for *PI*=26 γ_{td}=0.04, and for *PI*=47 clay γ_{td}=0.05%. Testing procedure and comparison to other types of γ_t are presented.

INTRODUCTION AND OBJECTIVES

When fully saturated soil is subjected in undrained conditions to cyclic strain-controlled loading with moderate to large cyclic shear strain amplitude, γ_c, its secant shear modulus, G_s, decreases with the number of cycles, N. This phenomenon of reduction of G_s with N is sketched in Fig. 1a and it is called cyclic degradation. If G_s at cycle N is denoted by G_{sN}, cyclic degradation can be quantified for given γ_c by degradation index:

$$\delta = G_{sN} / G_{s1} \ . \qquad (1)$$

Index δ describes relative decrease of G_s with N with respect to G_s in the first cycle, N=1. If δ is plotted versus N in a log-log scale, it has been shown that for many soils data points plot along a more or less straight line, in particular for clays. The slope of this line describes the rate of cyclic degradation with N which is called degradation parameter:

$$t = -\frac{\log\delta}{\log N} \ . \qquad (2)$$

The concept of degradation index δ and parameter t presented in Fig. 1a was originally introduced by Idriss et al. (1978) for normally consolidated clays, and it was subsequently employed for the characterization of cyclic degradation of sands, silts and overconsolidated clays by Vucetic and Dobry (1988), Tan and Vucetic (1989) and Vucetic (1992, 1994a).

If soil is subjected to cyclic strain-controlled loading with very small γ_c, smaller than a certain threshold value, it will not cyclically degrade. This means that there is γ_c below which modulus G_s remains practically constant with N, i.e., G_s=G_{sN}≈G_{s1}=const. Such a case of no degradation is sketched in Fig. 1b. The threshold cyclic shear strain amplitude below which degradation of fully saturated soil does not take place and above which it does is the threshold shear strain for cyclic degradation, denoted here as γ_{td}. The cyclic strain amplitude γ_{td} therefore represents boundary between two fundamentally different types of cyclic soil behavior. When soil is subjected to γ_c>γ_{td} its particle contacts and bonds are irreversibly disturbed resulting in permanent weakening of soil structure. Furthermore, when γ_c>γ_{td} residual cyclic pore water pressure may build up, softening the soil and causing additional reduction of G_s. When soil is subjected to γ_c<γ_{td} there are no such disturbances, and when cycling stops soil structure remains practically unchanged.

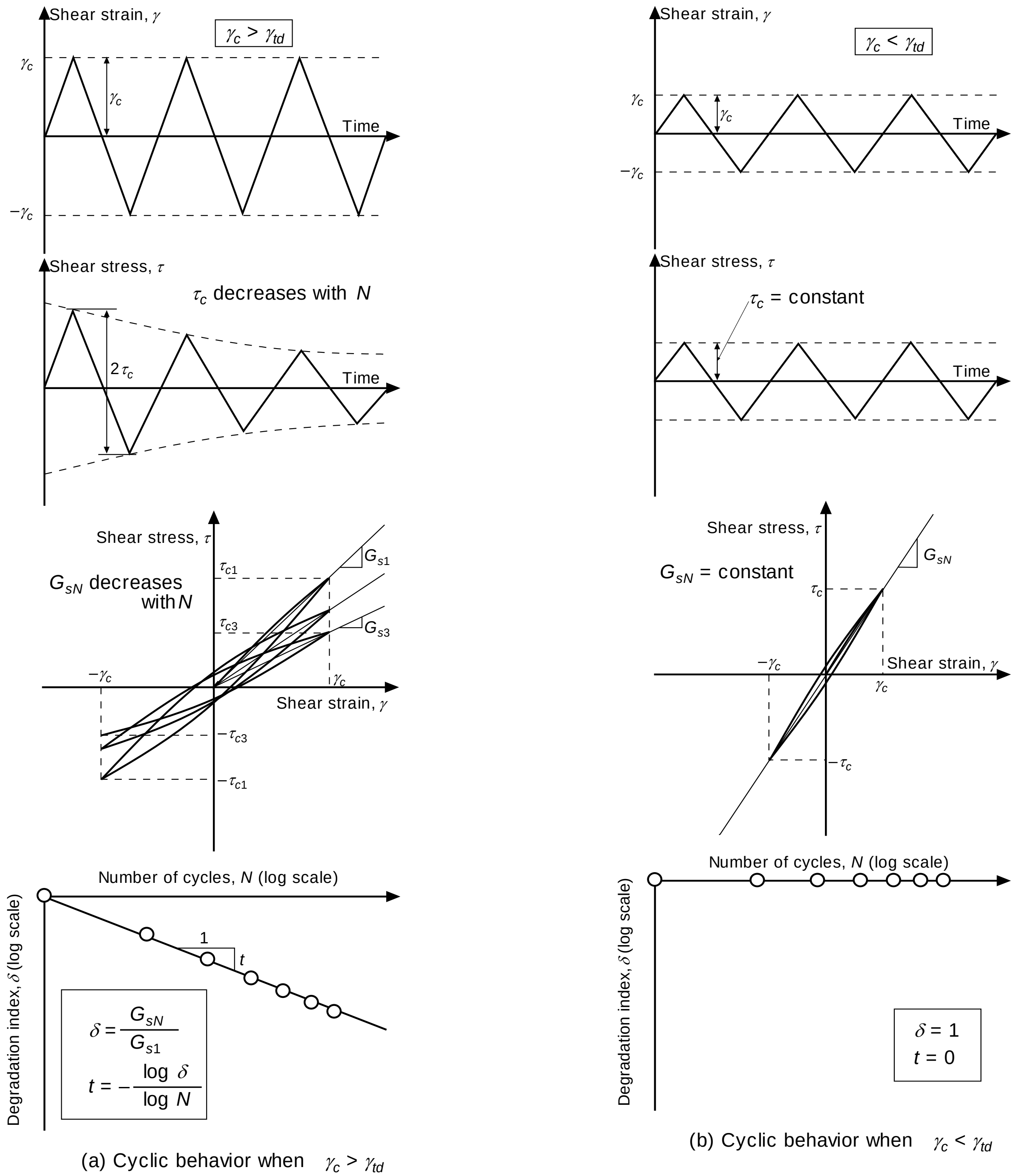


Fig. 1 Phenomenon of cyclic degradation and definition of parameters

The existence of cyclic threshold shear strain amplitude above which soil microstructure rapidly and significantly changes and below which the changes are for all practical purposes negligible was first identified experimentally for cyclic settlement of sands in simple shear device by Silver and Seed (1971) and Youd (1972). This threshold shear strain amplitude is known as the threshold shear strain for cyclic settlement, cyclic compression, or volume change, and it is denoted here as γ_{tv}. Later on, the threshold shear strain amplitude for residual cyclic pore water pressure buildup in sands was identified and precisely measured in triaxial tests by Dobry et al. (1982) and Dyvik et al. (1984). This threshold shear strain amplitude is known as the threshold shear strain for cyclic pore water pressure, denoted here as γ_{tp}. Dobry et al. (1982)

explained analytically and verified experimentally that for a given sand γ_{tv} and γ_{tp} are essentially the same, because pore water pressure buildup in undrained condition is caused by tendency of sand to decrease in volume. Subsequently, Dyvik et al. (1984) showed that γ_{tp} in sands is negligibly affected by consolidation stresses and density.

These studies on sands were followed by cyclic settlement and cyclic pore water pressure studies on clays that reveal existence of γ_{tv} and γ_{tp} in clays. They were conducted in simple shear devices by Ohara and Matsuda (1988), Chu and Vucetic (1992) and Hsu and Vucetic (2002, 2004, 2005), who showed that γ_{tv} and γ_{tp} in clays are significantly larger than in sands. Simultaneously, Kim et al. (1991) tested cyclic threshold shear strain for cyclic stiffening in drained conditions, which is denoted here as γ_{ts}. Kim et al. used the device that combines resonant column and torsional shear tests and defined γ_{ts} as γ_c at which shear modulus in 10th cycle is 2% larger than the modulus in the first cycle. They also found that γ_{ts} generally increases with the plasticity index of soil, *PI*, and just slightly increases with confining pressure. By doubling the confining pressure, γ_{ts} increased on average by only 10%.

Based on a synthesis of the above mentioned studies and other investigations containing direct or indirect evidence of existence of cyclic threshold shear strains conducted prior to 1994, it has been suggested that for given soil the magnitudes of all four threshold shear strains, i.e., γ_{tv}, γ_{tp}, γ_{ts} and γ_{td}, are essentially same or very similar (Vucetic 1992, 1994b). This conclusion was based on the notion that mechanism of all four cyclic threshold shear strains is associated with onset of permanent (residual) relative displacements between soil particles and their subsequent irreversible restructuring. Consequently, a unique correlation between all four types of cyclic threshold shear strain and soil's plasticity index, *PI*, was suggested (Vucetic, 1994b). This correlation exhibits a rather consistent increase of threshold shear strain with *PI*, regardless of the variation in confining pressure, *OCR*, and specimen fabric.

However, as the results on γ_{td} presented below show, the conclusion that all four types of cyclic threshold shear strain are more or less the same is somewhat simplistic. As shown below, γ_{ts} and γ_{td} are consistently smaller that γ_{tv} and γ_{tp}, which is an important conclusion of this paper.

By combining the concepts of cyclic threshold shear strain and cyclic degradation, Vucetic (1992, 1994a) suggested the relationship between γ_c and *t* such as sketched in Fig. 2. This relationship was then expanded into the practical chart shown in Fig. 3, which relates *t*, γ_c, *PI* and γ_{td}. These two relationships demonstrate that for a complete characterization of cyclic degradation γ_{td} should be known. The importance of the synthesis of cyclic degradation and cyclic threshold shear strain also transpires from substantial coverage of both phenomena in the textbooks by Kramer (1996) and Ishihara (1996). However, in spite of its fundamental nature, γ_{td} has not been studied systematically in the past. The research described in this paper and in the original thesis (Tabata, 2004) and report by the writers (Tabata and Vucetic, 2004) seems to be the first such effort focusing explicitly on γ_{td}.

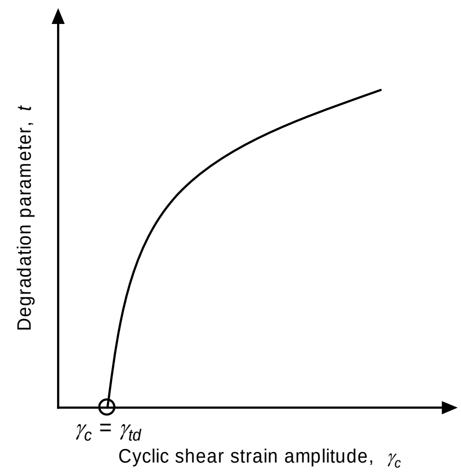


Fig. 2 Sketch of relationship between degradation parameter, t, and cyclic shear strain amplitude, γ_c (Vucetic, 1992)

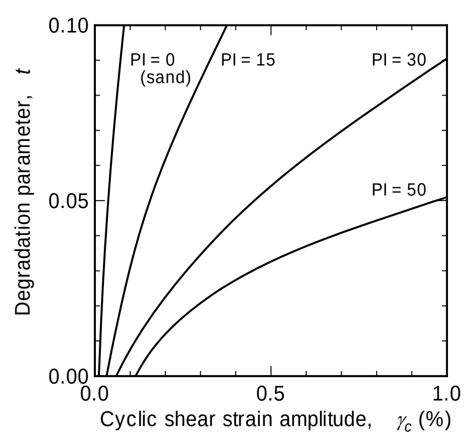


Fig. 3 Relationship between t, γ_c and PI for normally consolidated soils suggested by Vucetic (1992, 1994a)

The objectives of this paper are: (1) to provide values of γ_{td} for several clayey soils, (2) to compare γ_{td} and γ_{ts} because they both describe change of modulus G_s, (3) to compare γ_{td} to γ_{tv} and γ_{tp}, and (4) to describe practical experimental technique for determining γ_{td} in a single cyclic test. To meet these objectives, 3 small-strain cyclic tests were conducted on 3 different clayey soils yielding 3 values of γ_{td}. The tests were conducted in a unique Norwegian Geotechnical Institute (NGI) type of constant-volume equivalent-undrained direct simple shear (DSS) apparatus designed specifically for small-strain testing.

Table 1 Testing program and summary of results

Test name	Classification symbol	Plasticity index PI	Liquid limit w_L (%)	Void ratio e	Degree of saturation S_r (%)	Vertical consolidation stress σ_{vc} (kPa)	Step no.	Cyclic shear strain amplitude γ_c (%)	Degradation parameter t	Estimated threshold strain γ_{td} (%)
Obregon Park-7	ML	12	38	0.55	90	280	1	0.0011	0.000	0.015
							2	0.0030	0.003	
							3	0.010	0.003	
							4	0.031	0.043	
							5	0.10	0.050	
							6	0.31	0.098	
							7	1.04	0.176	
Halls Valley-1	CL	26	42	0.75	94	37	1	0.0024	0.000	0.040
							2	0.0083	0.000	
							3	0.025	0.000	
							4	0.099	0.037	
							5	0.30	0.077	
							6	0.83	0.104	
Halls Valley-3	CH	47	85	1.08	98	274	1	0.010	0.000	0.050
							2	0.096	0.043	
							3	0.30	0.054	
							4	3.48	0.292	

TESTING PROGRAM AND SOILS TESTED

Testing program is summarized in Table 1. The soils tested were low plasticity silt bordering with low plasticity clay (ML/CL), low plasticity clay (CL) and high plasticity clay (CH). Each test was conducted in 4 to 7 consecutive cyclic strain-controlled steps with constant γ_c in each step. In each subsequent step γ_c was slightly larger. In the initial steps γ_c was well below anticipated γ_{td}, while in the subsequent steps it was close to it or above it. Table 1 lists the classification properties, soil indices and conditions prior to cyclic shearing, γ_c applied in each cyclic step, degradation parameter t obtained in each cyclic step, and γ_{td} values eventually obtained. The vertical consolidation stress listed in the table, σ_{vc}, was total vertical stresses applied prior to cyclic shearing, i.e., vertical load applied on the top specimen cap divided by its area. As explained later, this total stress was essentially the same as effective stresses. The plasticity chart is presented in Fig. 4, showing how different the tested soils actually were. Grain size distributions can be found in Tabata and Vucetic (2004). The names of the tests correspond to the names of sites in Southern California from which soil samples were retrieved. A report by the writers (Tabata and Vucetic, 2002) describes many other cyclic properties of the same soils. All three specimens were trimmed from well-preserved, intact natural soil samples extruded from Shelby tubes

DESCRIPTION OF THE TESTING APPARATUS

The direct simple shear (DSS) testing has been used in many past investigations of cyclic soil properties, primarily because cyclic degradation, cyclic settlements and residual cyclic pore water pressures during earthquakes, machine foundation vibrations and traffic vibrations occur predominantly due to cyclic shear strains generated by shear waves propagating through soil deposits. The DSS test stress-strain conditions correspond quite well to those generated by such waves. Furthermore, the NGI-DSS testing and specimen trimming procedures were developed originally for highly sensitive Norwegian quick clays (Bjerrum and Landva, 1966), which makes them very reliable for investigation of less sensitive soils, such as those listed in Table 1.

NGI-DSS Constant-volume, Equivalent-undrained Testing Procedure

To appreciate the results of the NGI-DSS constant-volume equivalent-undrained testing presented below, the underlying concepts need to be fully understood. To evaluate the undrained stress-strain properties of fully saturated soils in the NGI type of DSS test, the shearing is conducted under constant-volume conditions. This is acceptable considering that in truly undrained test on fully saturated soil the volume of specimen is constant. While in NGI DSS test the specimen volume is maintained constant during the shearing, drainage of specimen is allowed (drains are open) and pore water pressure is consequently zero. The variation of the vertical stress required to maintain the volume of the specimen constant is then considered equivalent to the pore water pressure that would have developed in a truly undrained test. This means that in such NGI DSS test at all times during the shearing the specimen volume is constant, pore water pressure is zero, and the total vertical stress applied via the specimen top cap is actually the effective vertical stress. This constant-volume equivalent-undrained testing concept has been experimentally verified for the NGI-DSS conditions by Dyvik et al. (1987), while for the triaxial conditions it has been verified by Berre (1981) as reported in Vucetic and Lacasse (1984).

In the experiments described below, just like in any typical NGI-DSS test, specimens were confined in wire-reinforced rubber membrane. The role of such a membrane is to greatly restrict and nearly prevent radial deformations during consolidation and shearing while allowing vertical and shear deformations. During cyclic shearing pore water drains were open and the volume was maintained constant by just keeping the specimen height constant. This is acceptable considering that radial deformations are negligible if specimen is confined in a properly selected wire-reinforced rubber membrane. Such a constant-height procedure was verified as adequate by Iversen (1977) who compared tests with volume control and height control, and it is a standard procedure at NGI and other laboratories. In this study, in each test the same specimen height was maintained constant throughout all cyclic steps.

NGI-DSS Device for Small-strain Testing

The NGI-type of DSS device used in the present investigation was designed by Doroudian and Vucetic (1995) specifically for the testing of soils at very small cyclic shear strains. A general view of the device and specimen setup are presented in Fig. 5. Since its capabilities and limitations are described in Doroudian and Vucetic (1995; 1988) they are here just summarized. As shown in Fig 5b, the most unique feature of this device is that two parallel specimens of the same soil are sheared simultaneously. The device is thus named the dual-specimen direct simple shear device, abbreviated as DSDSS device. The configuration of two parallel specimens, in conjunction with very stiff components of the device and high-precision non-contact displacement transducer, enables almost complete elimination of the problems associated with the system's false deformations, compliance and friction. Another unique feature of the device is that horizontal cyclic load is applied manually directly to the middle cap between the two specimens. In this way, the vibrations that would have been otherwise introduced by electrical or hydraulic motors, which are unacceptable in small-strain testing, are avoided.

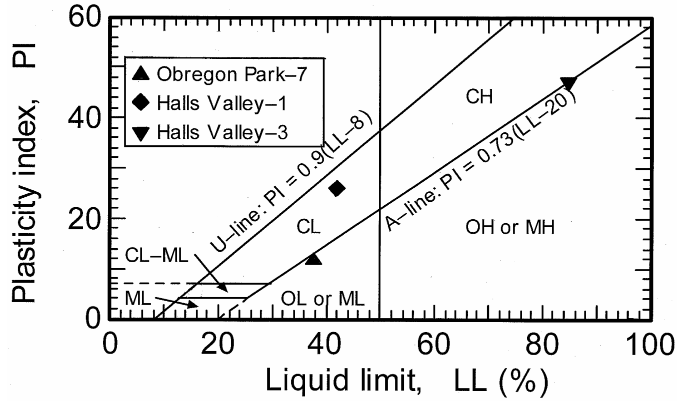


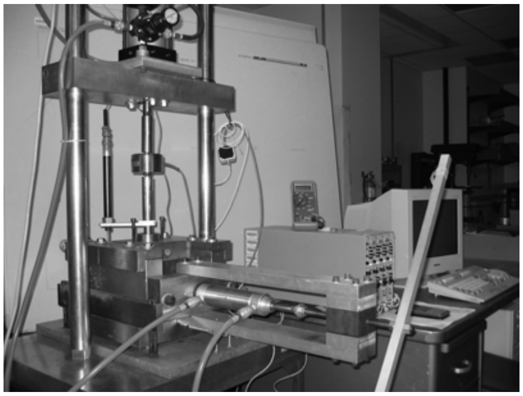
Fig. 4 Casagrande's Plasticity chart of soils

The DSDSS device accommodates cylindrical specimens 66 mm in diameter and 20 mm high. A slightly modified standard NGI trimming apparatus that can accommodate a set of two specimens instead of just one was used for the trimming of intact soil specimens. In this trimming apparatus the basic principles of preparation of high–quality NGI-DSS specimens are fully preserved. To date, the DSDSS device has been used successfully for investigation of several important small-strain cyclic soil properties because it enables application and precise measurement of very small shear strains and stresses in controlled manner. Some investigations with DSDSS device are described, for example, in publications by Lanzo et al. (1997), Vucetic, et al. (1998) and Vucetic and Tabata (2003).

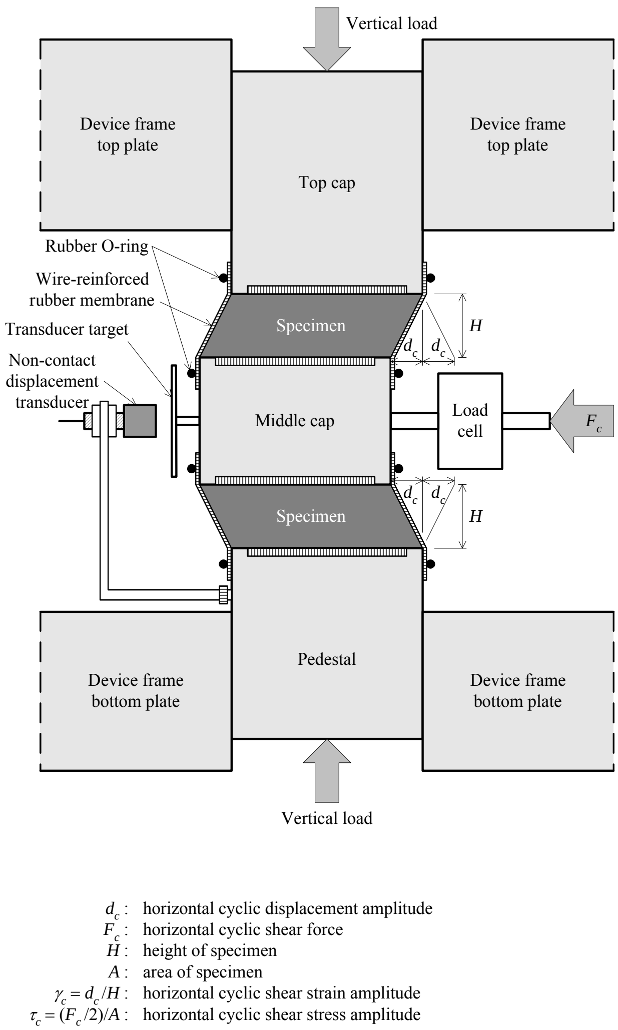
TESTING PROCEDURE AND DATA INTERPRETATION

Preparation and Consolidation of Specimens

The porous platens that are firmly mounted in the specimen pedestal, top cap and middle cap were first saturated by boiling all three components in water. The specimen pedestal was then secured in the trimming apparatus. The pair of specimens was then trimmed to their proper sizes and placed in the trimming apparatus between the pedestal, middle cap and top cap. The wire-reinforced rubber membranes were then pulled on and secured by O-rings. The entire setup was then carefully transported without disturbance into the DSDSS device and consolidated under desired σ_{vc}. Following completion of primary consolidation and considerable degree of secondary compression, the top cap and pedestal were firmly fixed to the top and bottom steel plates of the DSDSS device frame to ensure constant height of the specimens throughout shearing. The load and displacement transducers were then connected such as indicated in Fig. 5b.



(a) Outside view of DSDSS testing apparatus (specimen setup sketched below is behind steel plates and cannot be seen)



(b) Sketch of DSDSS apparatus specimen setup

Fig. 5 Dual-specimen direct simple shear (DSDSS) constant-volume apparatus for small-strain testing designed by Doroudian and Vucetic (1995)

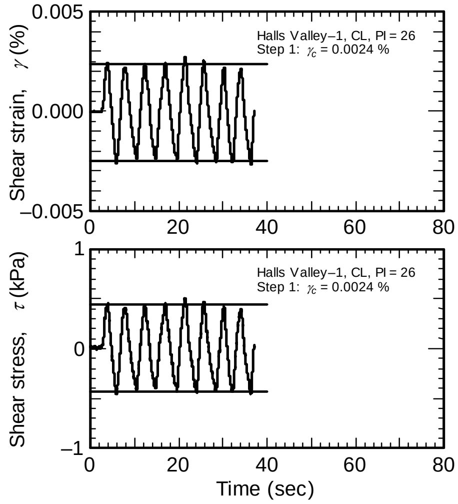
The specimens were then sheared in cyclic strain-controlled steps, such that in each step a series of uniform cyclic deformations was applied to the middle cap.

<u>Effect of Partial Saturation and Effective Consolidation Stresses on Cyclic Threshold Shear Strain Magnitude</u>

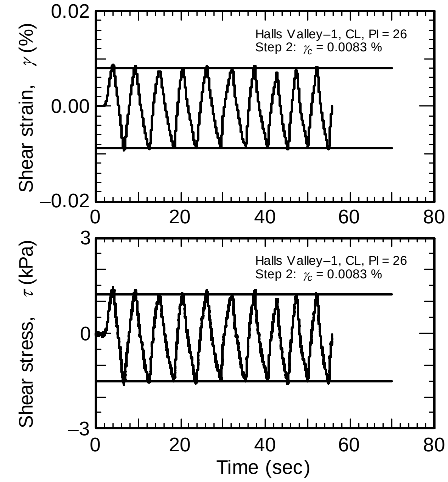
The subject of the paper are values of γ_{td} in fully saturated soils sheared in undrained conditions. However, as shown in Table 1, the specimens were not fully saturated. The degree of saturation, S_r, was 90% or higher, but not 100%. An explanation is therefore due as to why the test results can be considered equivalent to those that would have been obtained on the same fully saturated soils.

As already explained, in the constant-volume equivalent-undrained NGI-DSS and DSDSS tests on fully saturated specimens, during the shearing the drains are open, the pore water pressure is zero, and the applied total vertical stress via the top cap is actually the effective vertical stress in the soil. In partially saturated specimen tested under the same constant-volume equivalent-undrained conditions, prior to and during the shearing the pore water pressure is not zero but negative due to capillary tension. Consequently, the effective vertical stress in the soil is somewhat larger than the applied total vertical stress, and it is therefore important to asses how much larger it is and how the corresponding stress difference can influence the value of γ_{td}. This is discussed below in general terms based on the knowledge on capillary tension and behavior of unsaturated soils provided in textbooks by Mitchell and Soga (2005) and Fredlund and Rahardjo (1993), publications on the effect of S_r on shear moduli such as that by Wu et al. (1984), and information about the influence of confining stress on γ_{tv}, γ_{tp} and γ_{ts} provided in the papers already cited above.

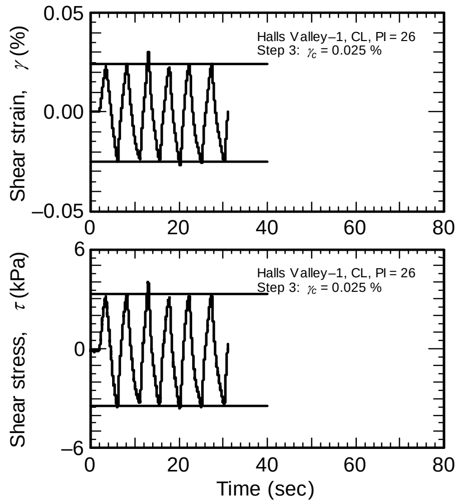
In sandy soils with relatively large particles and voids, capillary tension is generally small to negligible. In low-plasticity silts, such as in Obregon Park-7 soil, particles and voids are smaller, but not small enough to cause significant capillary tension. As opposed to that, capillary tension in clays may be considerable, especially in high-plasticity clays such as Halls Valley-3 clay. However, if clay is highly saturated and fully saturated porous platens eliminate capillary menisci at the specimen surfaces, effects of capillary tension should be small. Although exact capillary tension and resulting additional effective stresses in the tests are not known, because they were not measured, it can be concluded that they were relatively small in all three tests. This can be also concluded from study by Wu et al. (1984) who showed that for soil with grain size distribution similar to that of Obregon Park-7 soil the maximum shear modulus is practically the same over the range of S_r between 80% and 100%.



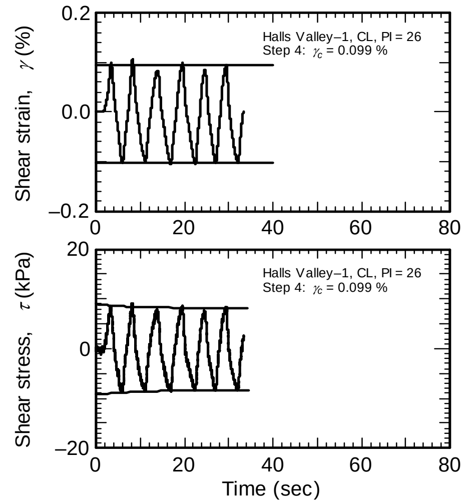
(a) Step 1: γ_c = 0.0024 %



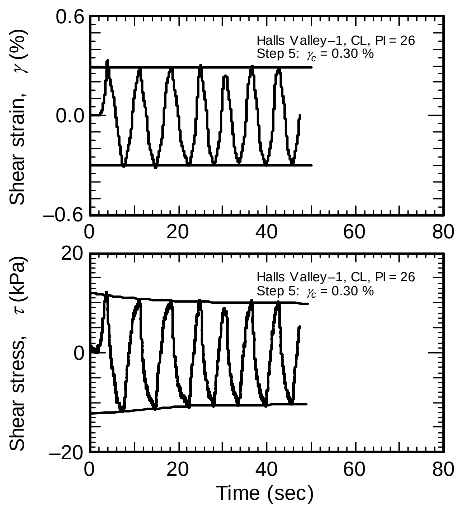
(b) Step 2: γ_c = 0.0083 %



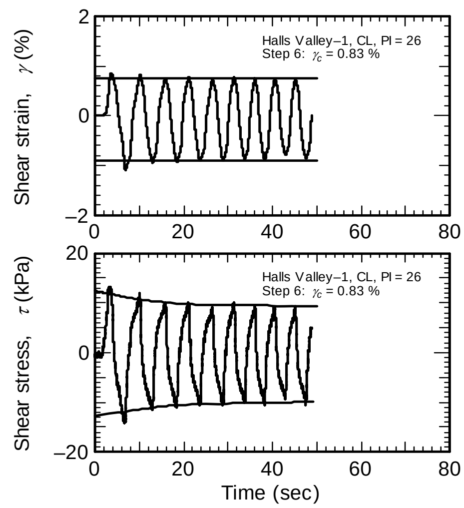
(c) Step 3: γ_c = 0.025 %



(d) Step 4: γ_c = 0.099 %



(e) Step 5: γ_c = 0.30 %



(f) Step 6: γ_c = 0.83 %

Fig. 6 Example of time histories of strain and stress (Test Halls Valley-1)

As far as the effect of the consolidation stress on the magnitude of different types of threshold shear strains is concerned, all available evidence shows that this effect is small to negligible. For the same soil practically the same threshold shear strain has been typically obtained for very different confining stresses. For example, Dyvik et al. (1984) revealed that γ_{tp} in sandy soils is practically unaffected by confining stress. The same conclusion plus the finding that γ_{tp} is also not affected by the type of sand and its fabric is vividly presented by Dobry in "Liquefaction of Soils During Earthquakes" (1985). Furthermore, Hsu and Vucetic (2004, 2005) found no visible effect of vertical stress on γ_{tv} and γ_{tp} in clays. Finally, as already mentioned above, Kim et al. (1991) demonstrated that γ_{ts} increases very little with the confining pressure.

From the above discussion it can be concluded that, (1) in the present testing effective vertical stresses during cyclic shearing were just slightly different than the applied total vertical stresses, and (2) since the effect of vertical stress on γ_{tv}, γ_{tp} and γ_{ts} is practically negligible, the same should be expected for γ_{td}. This means that γ_{td} values obtained in this study are practically the same as those that would have been obtained on the same fully saturated soils. Moreover, threshold shear strains are influenced so significantly by the type of soil, and so little by vertical stress, that the relationship between γ_{td} and *PI* presented at the end of the paper could not have been affected in any serious manner by relatively small variation of effective stresses caused by capillary tension.

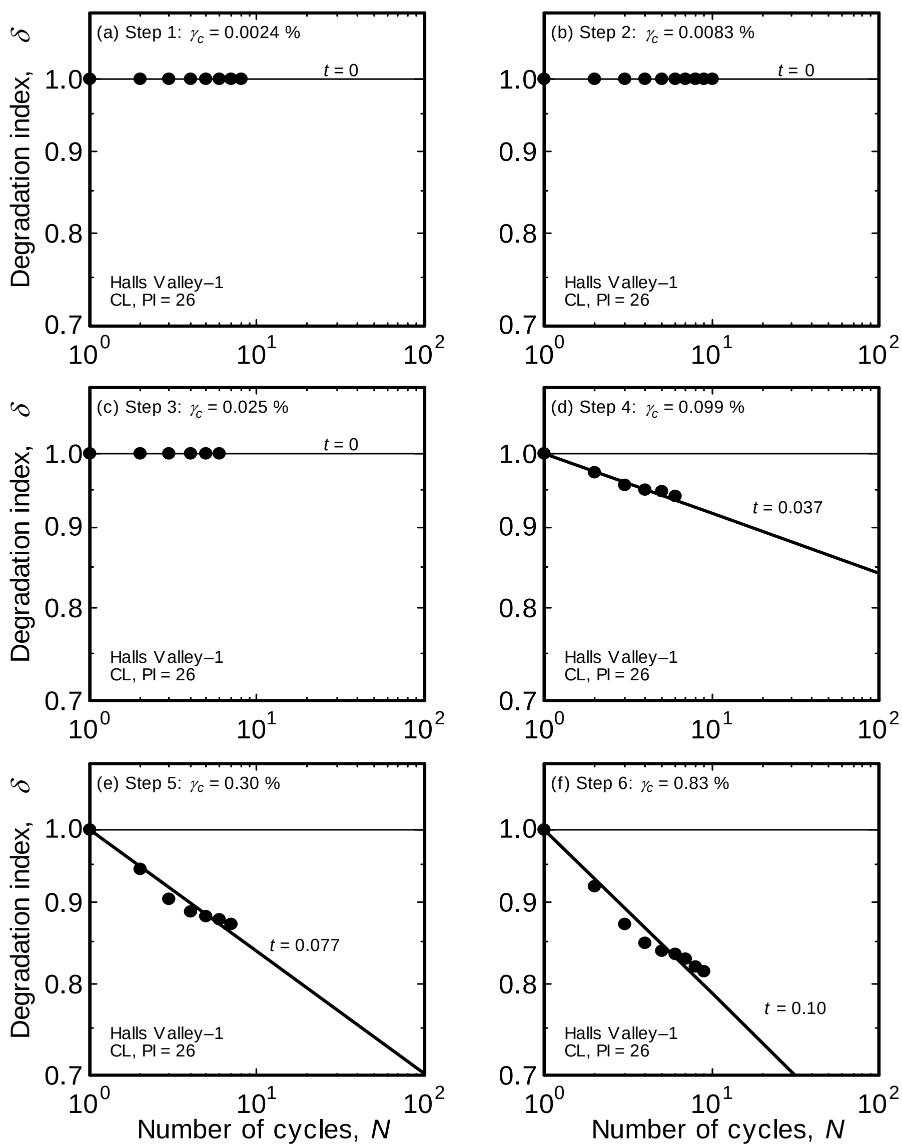


Fig. 7 Example of relationships between δ and N (Test Halls Valley-1)

Cyclic Shearing

Procedure for evaluation of γ_{td} in the DSDSS device is illustrated here on the results of test on low-plasticity Halls Valley-1 clay. Cyclic shearing was conducted in 6 cyclic strain-controlled steps. They are listed in Table 1 and displayed in Fig. 6. Amplitude γ_c was maintained constant in each step to the extent possible with the manually-controlled loading system. Cyclic strains were monitored on data acquisition system screen in real time and simultaneously the appropriate horizontal displacements were applied manually to the middle cap. In step 1, 8 triangular cycles of γ_c=0.0024% were applied, which is a very small strain indeed. The corresponding variations of cyclic strains and stresses (smaller than 0.5 kPa!) with time are presented in Fig. 6a, while the resulting variation of degradation index, δ, with the number of cycles, N, is presented in Fig. 7a. It can be seen clearly that during this initial step cyclic degradation did not take place, i.e., soil structure was not permanently changed. Consequently, at the beginning of step 2 the soil structure was essentially the same as at the beginning of step 1. In step 2, 10 triangular cycles of somewhat larger γ_c=0.0083% were applied. The cyclic behavior is presented in Figs. 6b and 7b, revealing that in this step too there was no cyclic degradation and accompanying soil disturbance. In step 3, 6 triangular cycles of even larger γ_c=0.025% were applied. Again, as shown in Figs. 6c and 7c, there was no detectable cyclic degradation. In the following step 4, 6 triangular cycles of γ_c=0.099% were applied. In this step the cyclic degradation finally took place. As shown in Fig. 6d, the cyclic stress amplitude, τ_c, decreased slightly with N. As shown in Fig. 7d, this resulted in a visible reduction of δ with N. The degradation after six cycles of γ_c=0.099% was around 6%, which means that δ was reduced from 1.0 to 0.94. Such a degradation yielded t=0.037. Accordingly, γ_{td} of Halls Valey-1 soil appears to be somewhere between γ_c=0.025% and 0.099%.

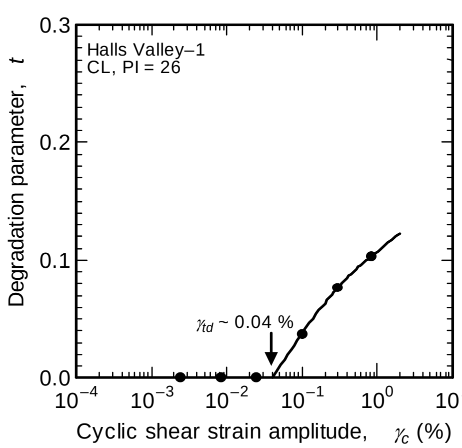
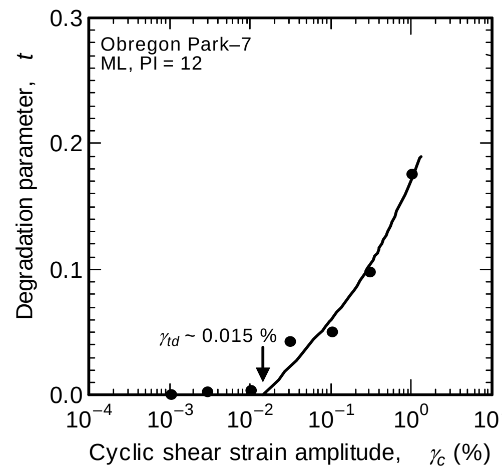


Fig. 8 Variation of t with γ_c and identification of γ_{td} in Test Halls Valley-1

The values of the degradation parameter, t, obtained in steps 1 through 4 are plotted against γ_c in Fig. 8, along with the t-γ_c data obtained in the subsequent steps 5 and 6. Steps 5 and 6 with even larger γ_c were conducted to confirm that γ_{td} is indeed smaller than 0.099%, and also to determine more precisely its value. In step 5, seven cycles of γ_c=0.30% were applied. As shown in Figs. 6e and 7e, τ_c and δ decreased more visibly and rapidly with N than in step 4, verifying that γ_{tv} is indeed smaller than 0.099%. The degradation after seven cycles of γ_c=0.30% was around 13%, which means that δ was reduced from 1.0 to 0.87, yielding t=0.077.



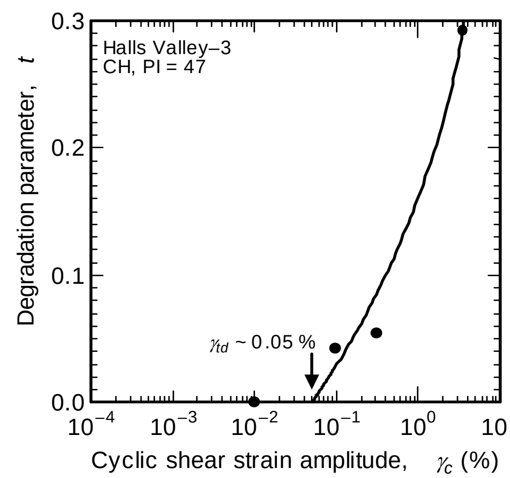


Fig. 9 Variation of t with γ_c and identification of γ_{td} in Tests Obregon Park-7 and Halls Valley-3

It is important to note that in the 2[nd] cycle of γ_c=0.30% in step 5, δ was reduced by 6%, which corresponds to the degradation achieved in 6 cycles of γ_c=0.099% in the previous step 4. This indicates that the degradation pattern in step 5 is practically the same as a pattern that would have been obtained on the fresh pair of specimens of the same soil not previously subjected to cyclic loading in step 4. In other words, cyclic degradation in step 4 was too small to alter appreciably the degradation pattern in step 5. In view of that, the actual value of γ_{td} can be obtained pretty accurately by extrapolating the trend of t-γ_c data points from steps 4 and 5 to t=0 axis. As shown in Fig. 8, such extrapolation yields $\gamma_{td} \approx 0.04\%$. In fact, the extrapolation trend in Fig. 8 is reinforced by additional data point from step 6, in which the largest γ_c=0.83% was applied. As shown in Figs. 6f and 7f, in step 6 the degradation was quite significant, yielding t=0.1. It must be mentioned that equivalent methodology of evaluating threshold shear strain from multi-step cyclic testing has been successfully used in the past by Dobry et al. (1982) and Ladd et al. (1989) for γ_{tp} in sandy soils, Hsu and Vucetic (2004, 2005) for γ_{tv} and γ_{tp} in cohesive soils, and Kim et al. (1991) for γ_{ts}.

Effects of Various Parameters on γ_{td}

The effects of σ_{vc}, specimen fabric and preparation, overconsolidation ratio, frequency and waveform of cyclic loading, and similar, were not investigated in the present study. Previous studies on other types of threshold shear strains indicate, however, that these effects are more or less negligible, especially in comparison to the effect of the type of soil. Nevertheless, these effects need be studied, perhaps in the manner Dyvik et al. (1984) studied some of them for γ_{tp} in sandy soils. In any case, in this study σ_{vc} varied from test to test between 37 and 280 kPa (see Table 1), frequency of cyclic loading was quite low as shown in Fig. 6, and in all tests the cyclic loading waveform was approximately triangular.

γ_{td} RESULTS AND THEIR COMPARISON TO PUBLISHED DATA

The t versus γ_c data points for other two soils are plotted in Fig. 9. Just like for Halls Valey-1 soil in Fig 8, the data trends are extrapolated to t=0 axis to estimate γ_{td}. Given the method of γ_{td} evaluation and relatively small number of data points, the extrapolations are done using certain engineering judgment instead of some rigorous mathematical data fitting routine. The resulting γ_{td} estimates are marked on the plots and listed in Table 1.

The γ_{td} values obtained in this study are compared to other types of γ_t in Fig. 10 in terms of their trend with *PI*. It can be seen that γ_{td} generally increases with *PI,* following the trend for other types of cyclic threshold shear strains. It can be also noticed that γ_{td} and γ_{ts} follow their own trend. This common trend of γ_{td} and γ_{ts} for cohesive, fine grained soils can be described by a relatively narrow data band presented in Fig. 11. It also appears from Fig. 10 that γ_{td} and γ_{ts} values are generally smaller and thus more conservative than those of γ_{tv} and γ_{tp}. The trend in Fig. 11 is therefore recommended for practical applications involving cyclic degradation and stiffening, instead of the general trend presented in Fig. 10.

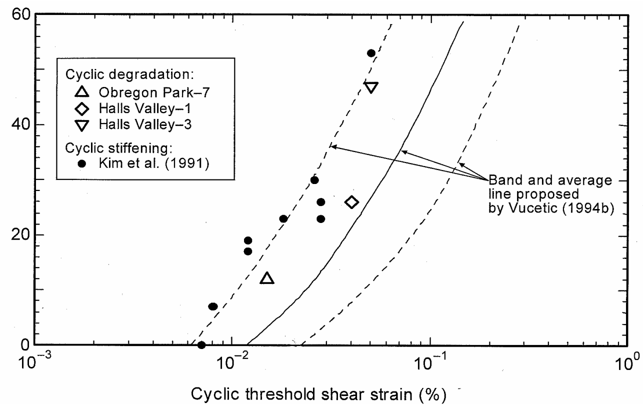


Fig. 10 Comparison of γ_{td} values for cyclic degradation obtained in DSDSS device with the trends obtained previously for other types of cyclic threshold shear strains

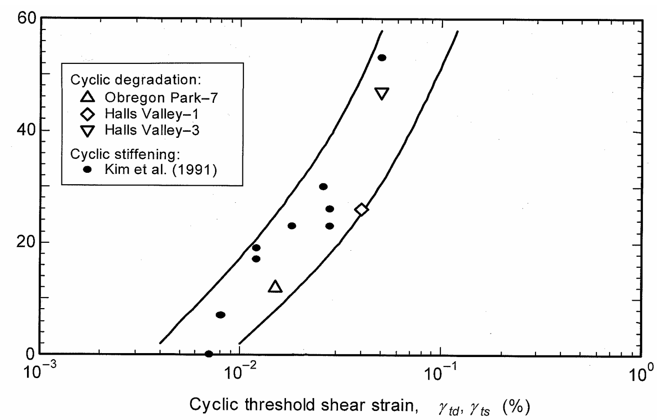


Fig. 11 Trend of γ_{td} for cyclic degradation and γ_{ts} for cyclic stiffening with PI

The reason why γ_{td} and γ_{ts} are smaller than γ_{tp} and γ_{tv} respectively cannot be readily explained. One possibility is that with currently available testing equipment and methods it is easier to identify the beginning of the reduction or increase of G_s with N than the onset of residual, permanent pore water pressures or cyclic settlements. The other reason may be that indeed, as data suggest, the reduction or increase of G_s with N starts somewhat below γ_{tp} and γ_{tv} respectively. That would mean that under fully saturated undrained or constant volume conditions soil stiffness can be reduced at γ_c that is smaller than minimum required for permanent, irreversible displacements of particles. Some particle bonds may brake or weaken under γ_c that is smaller than γ_{tp}, because relative displacements between particles required for the tendency towards volume change to occur may have to be larger than those corresponding to γ_{td}. In the case of cyclic stiffening when volume change is allowed, that would mean that soil stiffness may be increased due to γ_c that is too small to cause permanent, irreversible displacement of the particles. More research is apparently needed to explain why γ_{td} and γ_{ts} are smaller than γ_{tp} and γ_{tv}.

SUMMARY AND CONCLUSIONS

Knowing threshold shear strain for cyclic degradation, γ_{td}, is important in soil dynamics problems involving cyclic degradation of fully saturated soils in undrained conditions. The paper describes results of experimental investigation of γ_{td} for three different clayey soils ranging from low to high plasticity. The testing was conducted in a special NGI type of direct simple shear (DSS) device for small-strain testing. The type of test was cyclic multi-step strain-controlled constant-volume equivalent-undrained test. The following conclusions can be derived from the results and discussions presented:

1. The possibility of occurrence of cyclic degradation is controlled by a cyclic threshold shear strain, γ_{td}. If soil is subjected to cyclic strain amplitude, γ_c, which is smaller than γ_{td}, its secant shear modulus, G_s, will remain practically the same even after a large number of cycles, N. If $\gamma_c > \gamma_{td}$, G_s will consistently and relatively rapidly decrease with N.

2. Magnitude of γ_{td} can be obtained for a given soil from a single cyclic strain-controlled multi-step test conducted in a special NGI type of DSS device for small-strain testing.

3. For cohesive soils γ_{td} generally increases with plasticity index of the soil, *PI*. For low-plasticity silt bordering with low-plasticity clay having PI=12 γ_{td}=0.015%, for low plasticity clay having PI=26 γ_{td}=0.04, and for high-plasticity clay having PI=47 γ_{td}=0.05%.

4. The trend of γ_{td} with *PI* for cohesive soils is similar to the trend of the threshold shear strain for cyclic stiffening, γ_{ts}, published previously by others. Both sets of data plot within a relatively narrow band which is recommended for practical applications.

5. The results indicate that γ_{td} and γ_{ts} in cohesive soils are generally smaller than cyclic threshold shear strains for residual cyclic pore water pressure, γ_{tp}, and cyclic settlement, γ_{tv}. The reason for this difference needs to be investigated.

REFERENCES

Berre, T. [1981]. “Comparison of undrained and constant volume triaxial tests of plastic clay from Drammen” *Internal Report 56103-23, Norwegian Geotechnical Institute*, Oslo, Norway.

Bjerrum, L., and Landva, A. [1966]. “Direct Simple-Shear Test on a Norwegian Quick Clay.” *Géotechnique*, Vol. 16, No. 1, 1-20.

Chu, H.H. and Vucetic, M. [1992]. “Settlement of Compacted Clay in a Cyclic Direct Simple Shear Device.” *ASTM Geotechnical Testing Journal*, Vol. 15, No. 4, pp. 371-379.

Dobry, R., Ladd, R.S., Yokel, F.Y., Chung, R.M., and Powell, D. [1982]. “Prediction of Pore Water Pressure Buildup and Liquefaction of Sands During Earthquakes by the Cyclic Strain Method.” National Bureau of Standards Building Science Series 138, July 1982, 150 pp.

Doroudian, M., and Vucetic, M. [1995], “A direct simple shear device for measuring small-strain behavior”, *ASTM Geotech. Testing Journal*, GTJODJ, Vol. 18, No. 1, pp. 69-85.

Doroudian, M. and Vucetic, M. [1998], "Small-Strain Testing in an NGI-type Direct Simple Shear Device," *Proceedings of the 11th Danube-European Conference on Soil Mechanics and Geotechnical Engineering*, Porec, Croatia, Publisher A.A. Balkema, pp. 687-693.

Dyvik, R., Berre, T., Lacasse, S. and Raadim, B. [1987]. “Comparison of truly undrained and constant volume direct simple shear tests,” *Geotechnique*, vol. 37, no. 1, pp. 3-10.

Dyvik, R., Dobry, R., Thomas, G.E., and Pierce, W.G.[1984]. “Influence of Consolidation Shear Stresses and Relative Density on Threshold Strain and Pore Pressure During Cyclic Straining of Saturated Sands,” Miscellaneous Paper GL-84-15, Department of the Army, U.S. Army Corps of Engineers, Washington, D.C., pp. 73.

Fredlund, D. G. and Rahardjo, H. [1993]. “Soil Mechanics for Unsaturated Soils”, John Wiley & Sons, Inc., New York.

Hsu, C-C., and Vucetic, M. [2002]. “Dynamic and Cyclic Behavior of Soils Over the Wide Range of Shear Strains in NGI-type Simple Shear Testing Devices", *UCLA Research Report* No. ENG-02-228, Civil and Environmental Engineering Department, University of California, Los Angeles, Dec., 267 p.

Hsu, C-C. and Vucetic, M. [2004]. “Volumetric Threshold Shear Strain for Cyclic Settlement”, *ASCE Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 130, No. 1, pp. 58-70.

Hsu, C-C. and Vucetic, M. [2005]. “Threshold Shear Strain for Cyclic Pore water Pressure in Cohesive Soils”, conditionally accepted to *ASCE Journal of Geotechnical and Geoenvironmental Engineering*, Idriss, I.M., Dobry, R. and Singh, R.D. [1978]. “Nonlinear Behavior of Soft Clays During Cyclic Loading,” *Journal of the Geotechnical Engineering Division*, ASCE, Vol. 104, No. GT12, pp. 1427-1447.

Ishihara, K. [1996]. “Soil Behavior in Earthquake Geotechnics”, Clarendon Press, Oxford.

Iversen, K. [1977]. “Avsluttende forsok i forbindelse med automatisk hoydekontroll ved statiske CCV-forsok” *Internal Report 56204-6, Norwegian Geotechnical Institute*, Oslo, Norway.

Kim, D.-S., Stokoe, K.H., and Hudson, W.R. [1991]. “Deformational Characteristics of Soils at Small to Intermediate Strains from Cyclic Tests.” Research Report 1177-3, Center for Transportation Research, Bureau of Engineering Research, The University of Texas at Austin, pp. 142.

Kramer, S.L. [1996]. ”Geotechnical Earthquake Engineering”, Prentice-Hall, Inc., Upper Saddle River, New Jersey.

Ladd, R.S., Dobry, R., Dutko, P., Yokel, F.Y., and Chung, R.M. [1989]. “Pore-Water Pressure Buildup in Clean Sands Because of Cyclic Straining.” *Geotechnical Testing Journal*, GTJODJ, Vol. 12, No. 1, pp. 77-86.

Lanzo, G., Vucetic, M. and Doroudian, M. [1997]. “Reduction of Shear Modulus at Small Strains in Simple Shear”, *ASCE Journal of Geotechnical and Geoenvironmental Engineering,* Vol. 123, No. 11, pp.1035-1042.

“Liquefaction of Soils During Earthquakes.” [1985] National Research Council [NRC]. Committee on Earthquake Eng., *Report*, No. CETS-EE-001 Washington, D.C., pp. 240.

Mitchell, J. K. and Soga, K. [2005]. “Fundamentals of Soil behavior”, John Wiley & Sons, Inc., New York.

Ohara, S. and Matsuda, H. [1988]. “Study on Settlement of Saturated Clay Layer Induced by Cyclic Shear.” *Soils and Foundation*, Proc. JSSMFE, Vol. 28, No. 3, pp. 103-113.

Silver, M.L. and Seed, H.B. [1971]. “Volume Changes in Sands During Cyclic Loading.” *Journal of the Soil Mechanics and Foundations Division*, Proc. ASCE, Vol. 97, No. SM9, pp. 1171-1182.

Tabata, K., and Vucetic, M. [2002]. “Results of cyclic simple shear tests on fifteen soils conducted for PEARL project and other research purposes”, *UCLA Research Report* No. UCLA-ENG-02-233, Civil and Environmental Eng. Department, University of California, Los Angeles, Dec., 300 p.

Tabata, K. [2004]. “The Effects of the Rate of Loading and Frequency on the Behavior of Soils at Small Monotonic and Cyclic Shear Strains”, *Ph.D. Thesis,* University of California, Los Angeles, 289 p.

Tabata, K., and Vucetic, M. [2004]. “The Effects of the Rate of Loading and Frequency on the Behavior of Soils at Small Monotonic and Cyclic Shear Strains”, *UCLA Research Report* No. ENG-04-251, Civil and Environmental Eng. Department, University of California, Los Angeles, CA, November, 289 p.

Tan, K. and Vucetic, M. [1989]. "Behavior of Medium and Low Plasticity Clays Under Cyclic Simple Shear Conditions," *Proceedings of the 4th International Conference on Soil Dynamics and Earthquake Engineering,* Soil Dynamics and Liquefaction Volume, Mexico City, pp. 131-142.

Vucetic, M. [1992]. "Soil Properties and Seismic Response," *Proc. of the Tenth World Conf. on Earthquake Eng.,* Madrid, Spain, Publisher A.A. Balkema, Vol. III, July, pp. 1199-1204.

Vucetic, M. [1994a]. “Cyclic Characterization for Seismic Regions Based on PI”, *Proceedings, 13th International Conference on Soil Mechanics and Foundation Engineering*, New Delhi, India, Vol. 1, pp. 329-332.

Vucetic, M. [1994b]. “Cyclic Threshold Shear Strains in Soils”, *Journal of Geotechnical Engineering*, ASCE, Vol. 120, pp. 2208-2228.

Vucetic, M. and Dobry, R, [1988]. "Degradation of Marine Clays under Cyclic Loading,” *ASCE Journal of Geotechnical Engineering,* Vol. 114, No. 2, pp. 133-149.

Vucetic, M. and Lacasse, S., (1984). Closure to "Specimen Size Effect in Simple Shear Test," *Journal of Geotechnical Engineering, ASCE*, Vol. 110, No. 3, pp. 447-453.

Vucetic, M., Lanzo, G., and Doroudian, M. [1998]. “Damping at Small Strains in Cyclic Simple Shear Test”, *ASCE Journal of Geotechnical and Geoenvironmental Engineering,* Vol. 124, No.7, pp.585-594.

Vucetic, M. and Tabata, K. [2003]. “Influence of Soil Type on the Effect of Strain Rate on Small-strain Cyclic Shear Modulus”, *Soils and Foundation,* Vol. 43, No. 5, pp. 161-173.

Wu, S., Gray, D. H. and Richart, Jr., F. E. [1984]. “Capillary effects on dynamic modulus of sands and silts,” *ASCE Journal of Geotechnical Engineering,* Vol. 10, No. 9, pp. 1188-1203.

Youd, L.T. [1972]. “Compaction of Sands by Repeated Shear Straining.” *Journal of the Soil Mechanics and Foundations Division*, Proc. ASCE, Vol. 98, No. SM7, pp. 709-725.