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CYCLIC COMPRESSION OF COMPACTED CLAYEY SAND AT SMALL CYCLIC STRAINS

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ABSTRACT

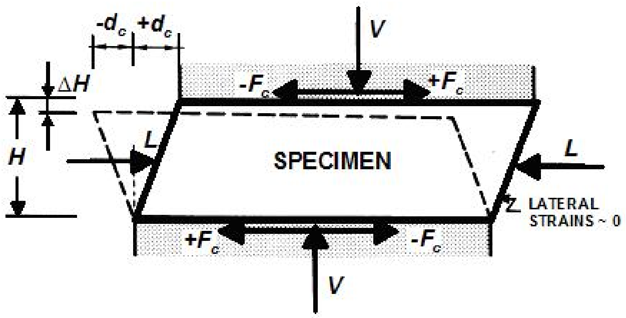
Ten drained cyclic strain-controlled direct simple shear tests were conducted on compacted low-plasticity clayey sand to measure its cyclic compression properties. The soil had 37 % fines, liquid limit of 28% and plasticity index 14. The relative compaction of specimens prior to consolidation and cyclic shearing was between 80 and 90 %. Cyclic compression is expressed as the accumulation of vertical strain with the number of cycles, N. Vertical strain recorded at the end of every cycle, ε_{vc}, increased with the cyclic shear strain amplitude, γ_c, and N. Such behavior is typical and has been obtained by others on other types of soils. Amplitude γ_c was relatively small, ranging between 0.008% and 0.24%. Such small cyclic strains are common in moderate and large earthquakes. The effects of the dry unit weight, γ_d, and corresponding void ratio, e, vertical consolidation stress, σ_{vc}, and certain aspects of the degree of saturation, S, on ε_{vc} are evaluated. The test results revealed that for the applied conditions ε_{vc} increases with σ_{vc} and e (decreases with γ_d) and is smaller if S is increased above approximately 90%. For this soil the cyclic threshold shear strain of about 0.02% was obtained. Simple mechanisms that most likely govern the cyclic compression of compacted soils are discussed.

INTRODUCTION AND OBJECTIVES

The volume of a deposit of dry sand, partially saturated sand, or partially saturated clayey soil will decrease if it is horizontally cyclically sheared with cyclic shear strain amplitudes larger than certain small cyclic threshold shear strain. Under overburden pressure, each cycle of horizontal shear straining will cause an increment of vertical strain. The volume change corresponding to the accumulation of such vertical strains and associated horizontal strains during cyclic loading is called the cyclic compression. The phenomena of cyclic compression have been traditionally investigated with the help of cyclic strain-controlled direct simple shear test. The test results are typically expressed in terms of the vertical strain measured at the end of each cycle, ε_{vc}, versus the number of shear strain cycles, N, of the cyclic shear strain amplitude, γ_c. Considering that in the standard direct simple shear test the radial (lateral) deformations, ε_r, of the circular specimen are negligibly small, ε_{vc} obtained in the direct simple shear test corresponds closely to the total volume change and hence to the cyclic compression.

The conditions of the soil specimen subjected to cyclic loading in direct simple shear (DSS) testing apparatus are sketched in Fig. 1 along with the definitions of relevant parameters. A sketch of typical results from a series of three cyclic strain-controlled DSS tests conducted at three different levels of γ_c is shown in Fig. 2. Vertical cyclic strain, ε_{vc}, is the residual cumulative strain at the end of each cycle, while ε_v is the vertical strain at any given time during the test. Figure 2b illustrates how ε_v fluctuates with time and ε_{vc} accumulates with time. Figure 2c shows a typical relationship between ε_{vc}, γ_c and N.

Figure 2 also shows that the volume of specimen will not change if amplitude γ_c is smaller than a certain threshold value, called the volumetric cyclic threshold shear strain, γ_{tv} (Vucetic, 1994; Hsu and Vucetic, 2004). Cyclic amplitude γ_{tv} represents a boundary between two fundamentally different types of volume change behavior. At cyclic shear strain amplitudes below γ_{tv} the soil essentially does not experience restructuring of its fabric and permanent cyclic compression even after many strain cycles. If γ_c is larger than γ_{tv} the restructuring of fabric takes place and the cyclic compression occurs and accumulates with each cycle of straining. In general, the magnitude of γ_{tv} increases as the plasticity index of the soil, PI, increases.



V = vertical consolidation load distributed over specimen area

L = lateral load generated by membrane confinement distributed over the specimen

F_c = cyclic horizontal shear force amplitude

H = initial height of specimen

A = area of specimen

d_c = cyclic horizontal displacement amplitude

ΔH = vertical cyclic settlement

γ_c = d_c/H = cyclic shear strain amplitude

τ_c = F_c/A = cyclic shear stress amplitude

ε_v = $\Delta H/H$ = vertical strain

Fig. 1. Loading conditions in cyclic simple shear test and definition of test parameters

From the typical pattern of cyclic compression exhibited in Fig. 2 and the results on cyclic compression published to date, which are briefly discussed below, and from the general understanding of soil microstructure and behavior of soils under monotonic and cyclic loads, it is not difficult to identify the most likely physical mechanisms governing the cyclic compression due to cyclic shear straining. As shown in Fig. 2b, during cyclic straining ε_v fluctuates as the soil element passes in each cycle through the phases of contraction and dilation. If during such cyclic straining the threshold shear strain is exceeded, in each cycle there will be an increment of permanent cyclic compression, $\Delta\varepsilon_{vc}$. This kind of behavior has been clearly explained for sands by Youd (1972). The most important mechanism of this behavior is apparently that individual soil particles, particle clusters and particle aggregates are displaced relative to each other in both loading and unloading direction and pushed into a denser state by existing vertical stress. In other words, the densification under the applied vertical stress is made possible by the disturbance of the soil structure caused by cyclic shearing. The degree of this shear disturbance, magnitude of vertical stress, shear resistance at particle contacts, and the available pore space to accommodate the densification must therefore be the key factors that govern the cyclic compression.

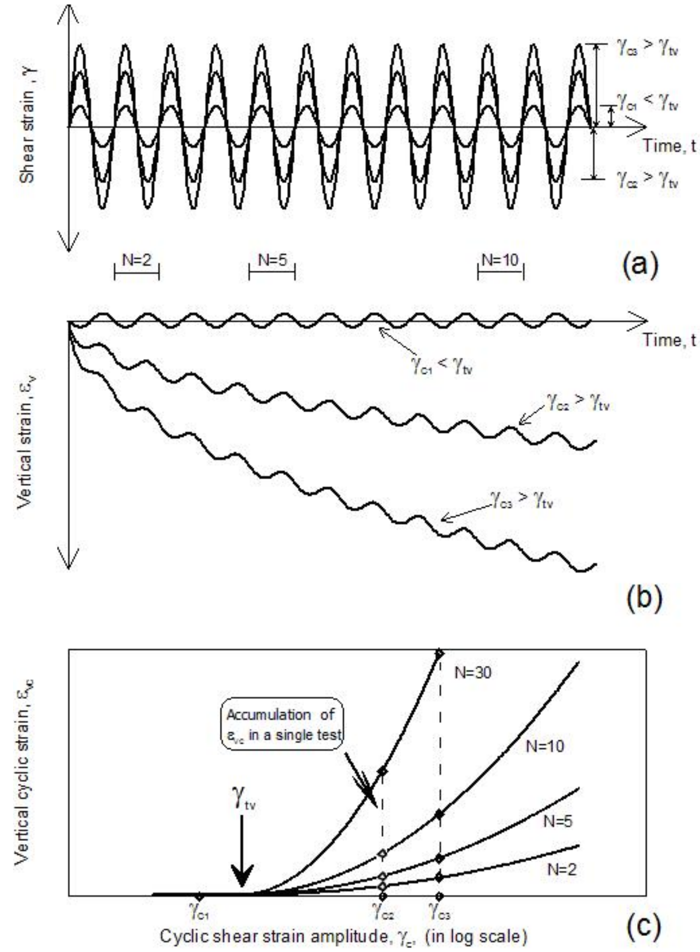


Fig. 2. Sketch of typical results of cyclic simple shear strain-controlled tests: (a) Strain-time histories of three cyclic strain-controlled compression tests; (b) Variation of vertical strain, ε_v, with time; (c) Relationship between vertical cyclic strain, ε_{vc}, cyclic shear strain amplitude, γ_c , and the number of cycles, N, obtained from the results of three cyclic strain-controlled compression tests.

The following basic trends should exist in the context of this relatively simple mechanism: (i) larger γ_c and N cause more soil disturbance and restructuring, which in turn facilitates larger densification under existing vertical stress; (ii) larger vertical stress, σ_{vc}, means that there is more stress to push particles into a denser state; (iii) as the volume of the voids (void ratio, e) increases and the associated dry unit weight, γ_d, decreases, there is more room to accommodate larger volume change; (iv) if the degree of saturation, S, increases, less void space will be occupied by compressible air and more by incompressible water, which will result in a structure that has smaller potential for volume change during cyclic shearing; (v) similarly, if S is high and close to the full saturation, densification will be governed in part by the ability of pore water to drain out during cyclic shearing; and (vi) under the same vertical stress and cyclic shear stresses, larger forces at particle contacts (including here also capillary tension which is related to the degree of saturation, S) will result in smaller cyclic strains and thus smaller cyclic compression.

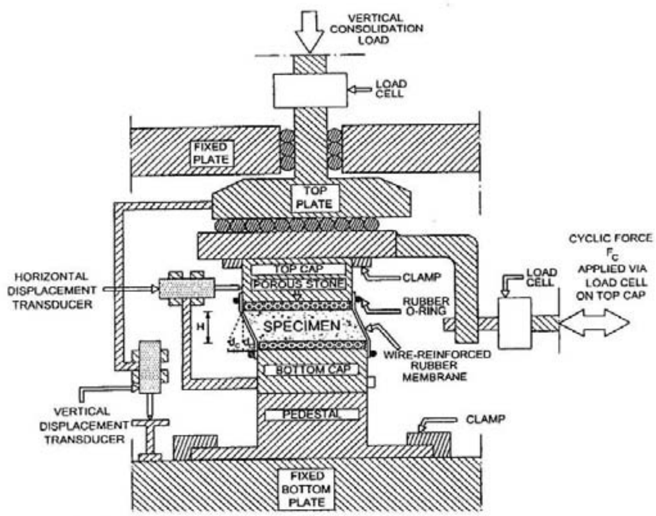
Accordingly, cyclic compression, ε_{vc}, should increase as γ_c, N, σ_{vc} and e increase (dry unit weight, γ_d, decrease), it should decrease if the stiffness of soil increases, and it should depend on S.

Many soil dynamics and geotechnical earthquake engineering problems involve the evaluation of cyclic compression. They include, for example, cyclic settlements of natural deposits during strong earthquakes, such as those discussed by Grantz et al. (1964) and Seed (1970), the earthquake-induced volume changes of earth fills, such as those encountered by the second and third writer in their engineering practice and those discussed by Stewart et al. (1996), and the ground movements and settlements caused by traffic vibrations, pile driving and the vibrations of machine foundations.

In spite of its importance, cyclic compression has not been studied on a wide enough variety of soils and for all relevant soil conditions. Available results in the literature include the results on clean sands by Silver and Seed (1971) and Youd (1972), results on a clayey sand by Pyke et al. (1975), results on a low plasticity compacted clay by Chu and Vucetic (1992), results on the variation of the volumetric cyclic threshold shear strain, γ_{tv}, for seven soils by Hsu and Vucetic (2004), results on four compacted soils by Whang et al. (2004) which include a low plasticity clay, two clayey sands and a silty sand, and recently completed study on clean sands by Duku et al. (2008). To this list can be added some other publications on volume change and cyclic compression due to horizontal cyclic shearing, such as that by Ohara and Matsuda (1988) who measured post-cyclic settlement of fully saturated clay. In that study the specimens were first cyclically loaded in undrained conditions and then allowed to consolidate in drained conditions.

Just four out of the eight papers listed above deal with the cyclic compression of compacted clayey soils. Pyke et al. (1975) present the cyclic compression results of a sandy clay for the relatively narrow range of γ_c between 0.1% and 0.4%, N=10 only, two vertical stresses, σ_{vc}, and two dry unit weights, γ_d. Chu and Vucetic (1992) tested cyclic compression of a low-plasticity compacted clay for the wide range of γ_c from approximately 0.01% to 3.0%, three different water contents, w, and associated degrees of saturation, S, but a single level of σ_{vc}. Hsu and Vucetic (2004) used multistage cyclic testing such that in each test γ_c increased in stages from approximately 0.01% to 1.0%. Such multistage testing is convenient for the determination of γ_{tv}, but its results cannot be used to study the cyclic compression within the context of the model presented in Fig. 2. Whang et al. (2004) investigated cyclic compression of compacted clayey soils for relatively wide ranges of relative compaction, RC, and degree of saturation, S, but for only one σ_{vc} and γ_c between 0.1% and 1%, with most of the tests conducted at relatively large γ_c= 0.4% and 1.0%.

From these publications the general trend of increasing ε_{vc} with increasing γ_c and N is rather clear. However, within this trend the quantitatively different pictures are obtained for different soils, so more compacted soils need to be tested over a wide range of strains, in particular in the range of smaller cyclic strains, to learn how ε_{vc} changes with γ_c and N for different type of soil. A general tendency of ε_{vc} increasing with void ratio, e, (decreasing with γ_d) is also rather evident from the above studies, but it has been reported for just a couple of soils and only for γ_c above 0.1%. On the other hand, from the above publications the trends of ε_{vc} with σ_{vc} and S are not clear, although some trends with S and water content have been reported. In conclusion, although the data base in the above publications covers various aspects of the cyclic compression of clayey soils, it also reveals that the subject is so complex that this data base is actually small. Accordingly, the subject of the cyclic compression of compacted clayey soils still needs to be extensively studied to fully understand it.



(a) Sketch of specimen setup

(b) View of the NGI-DSS cyclic testing apparatus

Fig. 3. NGI-DSS testing apparatus at UCLA Soil Dynamics Laboratory.

The investigation presented in this paper was conducted with this general goal in mind. One compacted clayey soil, a clayey sand, was tested in the range of γ_c from 0.008% to 0.24%, a range of different w and associated S, a range of γ_d and associated e, and at three different σ_{vc}. The results presented below therefore throw more light on the complex phenomenon of the cyclic compression of compacted clayey soils. In particular, the investigation described below was aimed at examining and explaining to the extent possible the following behavioral trends and characteristics of the soil tested: (1) cyclic compression relationship between ε_{vc}, γ_c and N over a range of smaller γ_c that are very relevant in many soil dynamics problems, (2) magnitude of γ_{tv}, (3) trends of ε_{vc} with σ_{vc} and γ_d, and (4) the effect of very high S on ε_{vc}.

In the present investigation the Norwegian Geotechnical Institute (NGI) type of direct simple shear (DSS) device and specimen setup were used, employing a circular specimen confined in wire-reinforced rubber membrane. It should be noted that in all of the past studies cited above, the cyclic compression tests were conducted using different versions of the NGI DSS device and test concepts.

It should be noted at the end of this introduction that in this paper only the behavior during cyclic loading is treated. After the end of cyclic loading clayey materials still undergo volume change due to consolidation and creep. These post-cyclic volume changes were not measured and are out of the scope of this paper.

TESTING APPARATUS AND PROCEDURE

Cyclic compression in soil deposits occurs typically as a result of the soil restructuring under vertical stress due to horizontal shear deformations. Such conditions can be simulated quite well on soil specimen in the NGI type of DSS device. The NGI-DSS device was originally introduced by Bjerrum and Landva (1966) for the monotonic-loading testing of highly sensitive, Norwegian quick clays. This makes the NGI-DSS device a very useful precise practical tool for the testing of less sensitive soils, such as the clayey compacted soil treated in this paper. Actually, the NGI-DSS device concept appears to be the concept of choice for the cyclic compression studies, as virtually all of such past studies have been conducted with such an apparatus, including all of the studies cited in this paper. The fact that so many researchers decided at different times and under different circumstances to eventually use the NGI-DSS concept means that it is the best concept for the cyclic compression investigations currently available.

A cross-section of the NGI-DSS specimen setup employed in this study is sketched in Fig. 3a and the apparatus itself is presented in Fig. 3b. The NGI-DSS specimen has the shape of short cylinder. The specimen is confined during the test in a wire-reinforced rubber membrane to greatly restrict (nearly prevent) radial deformations during consolidation and shearing, while allowing vertical and simple shear deformations. In the present study, prior to the cyclic shearing the specimens had a diameter of 66 mm and height between 21 and 23 mm. The specimens were prepared in the standard NGI trimming apparatus by compacting soil within the steel cutting ring employing a tamping procedure. During the compaction the steel ring was sitting on top of the porous stone that is embedded into the specimen bottom cap which is firmly fixed on the specimen pedestal. After the compaction, another specimen cap with the porous stone in it was placed on top of the specimen and secured, and then the steel cutting ring was slowly pulled off and simultaneously replaced by the wire-reinforced rubber membrane. The membrane was then secured to the specimen bottom and top caps with rubber O-rings. The porous stones were partially saturated just as the soil specimens. The complete specimen setup, consisting of the pedestal, bottom cap, specimen within the membrane, and the top cap, was then transferred into the NGI-DSS apparatus.

In the apparatus, each specimen was first consolidated by applying vertical stress in several increments until the prescribed total vertical stress, σ_{vc}, has been reached. Each specimen was then kept under σ_{vc} for a minimum of 12 hours, which was enough to reduce the rate of consolidation settlement to practically zero. To facilitate unobstructed volume change during the consolidation, the drains in the bottom and top caps connected to the porous stones were always open. After the completion of consolidation, each specimen was sheared in the cyclic strain-controlled mode with constant γ_c for a minimum of 30 cycles. The cyclic shearing was also conducted in drained conditions with the drains in the specimen caps open. The amplitude γ_c was applied and maintained constant with the help of a computer-controlled closed-loop servo-hydraulic system.

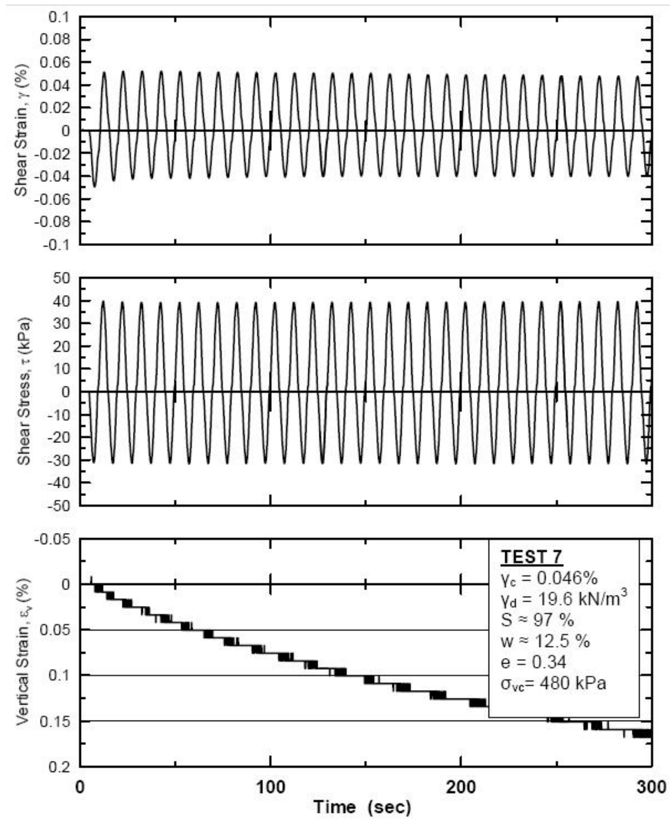


Fig. 4. Typical test results – Test 7
(values of γ_d , S and e are prior to cyclic shearing).

Table 1. Summary of specimen conditions

	Specimen conditions prior to consolidation					Specimen conditions prior to cyclic straining				
Cyclic test number	Dry unit weight γ_d kN/m^3	Relative compaction R.C. %	Void ratio e	Water content w %	Degree of saturation S %	Vertical stress σ_{vc} kPa	Dry unit weight γ_d kN/m^3	Increase in dry unit weight %	Void ratio e	Estimated degree of saturation* S %
1	17.7	86	0.48	14.9	82.8	192	18.8	6.2	0.40	99
2	18.6	90	0.42	12.9	82.9	192	19.2	3.2	0.37	93
3	18.4	89	0.43	12.5	78.3	192	19.0	3.3	0.38	88
4	18.6	90	0.42	12.2	78.6	192	19.1	2.7	0.38	87
5	18.6	90	0.42	12.9	82.9	192	19.2	3.2	0.37	93
6	18.4	89	0.43	12.5	78.3	48	18.7	1.6	0.41	82
7	18.4	89	0.43	12.5	78.3	480	19.6	6.5	0.34	97
8	18.4	89	0.43	10.1	62.9	192	18.7	1.6	0.41	66
9	16.4	80	0.60	15.4	68.4	192	18.0	9.8	0.46	89
10	17.3	84	0.52	14.0	72.1	192	18.0	4.0	0.46	81

***Assuming the water content prior to consolidation**

Table 2. Summary of cyclic compression data

			Vertical cyclic strain, ε_{vc} (%)				
Cyclic test Index	Vertical stress σ_{vc} kPa	Applied cyclic shear strain amplitude γ_c %	End of cycle 1	End of cycle 2	End of cycle 5	End of cycle 10	End of cycle 30
1	192	0.064	0.006	0.009	0.025	0.043	0.110
2	192	0.047	0.005	0.008	0.022	0.039	0.095
3	192	0.008	0.000	0.001	0.003	0.006	0.010
4	192	0.100	0.018	0.035	0.060	0.100	0.190
5	192	0.237	0.100	0.200	0.330	0.480	0.740
6	48	0.044	0.007	0.012	0.022	0.032	0.060
7	480	0.046	0.008	0.015	0.040	0.075	0.165
8	192	0.049	0.015	0.023	0.048	0.072	0.130
9	192	0.090	0.018	0.035	0.080	0.165	0.380
10	192	0.100	0.032	0.070	0.150	0.270	0.530

To evaluate the cyclic compression of the clayey sand in this study more accurately, typical magnitude of the radial strain, ε_r, of specimens that is caused by slight deformations of the NGI wire-reinforced rubber membrane under lateral loads was experimentally assessed. This was done in spite of the fact that ε_r is known to be negligibly small if the NGI membrane is properly selected (Vucetic and Lacasse, 1982; and Vucetic, 1984). Before the beginning of the cyclic testing program on the compacted clayey sand, ε_r was measured with the help of a π-tape prior, during and after the cyclic shearing of one dry sand and another compacted partially saturated clayey soil. In many measurements of ε_r that covered the same levels of γ_c, σ_{vc} and ε_{vc} that were applied in the tests on the clayey sand, and for even larger levels, the ratio of radial strain, ε_r, to vertical strain, ε_{vc}, was always smaller than $\varepsilon_r/\varepsilon_{vc}$=0.03, and typically around 0.01. Having such a small $\varepsilon_r/\varepsilon_{vc}$ ratio, it is evident that by neglecting the effect of ε_r a negligible error is made in evaluating ε_{vc}.

As shown in Fig. 3, the parameters were measured with electronic load cells and displacement transducers. The recorded data were processed with modern data acquisition system and computer software. The results of one of the tests, Test 7, which are rather typical of the entire investigation, are presented in Fig. 4.

SOIL TESTED AND LABORATORY TESTING PROGRAM

The soil tested was clayey sand sampled from a hillside fill located in northern Los Angeles County, California. The origin of the sand fill was sandstone from the Saugus formation. The grain size distribution is presented in Fig. 5. The fraction passing the #40 sieve had a liquid limit LL= 28% and the plasticity index PI=14. The soil contains 37 % fines passing through the #200 sieve (particle size ≤ 75 μm) and 14% clay (particle size ≤ 5 μm), suggesting that the behavior of this soil is significantly affected by the nature of the fines. The Unified Soil Classification System (USCS) symbol for this material is "SC."

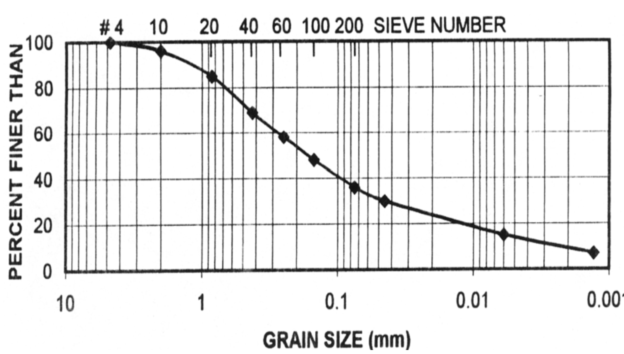


Fig. 5. Grain size distribution curve

The modified compaction curve obtained according to the ASTM D1557 procedure (ASTM, 1992) is shown on Fig. 6. At the optimum moisture content of 8.5 % the maximum dry unit weight, γ_d, was 20.6 kN/m^3, which corresponds to the maximum dry density of 2.1 Mg/m^3. The specific gravity was 2.69, leading to a degree of saturation at the maximum dry density of about 81 %. Figure 6 also shows the points representing the ten compacted specimens prior to the application of vertical stress. The associated levels of target densities corresponding to 80, 85, 90 and 95 % of the maximum dry density (% of relative compaction, RC) are also included for comparison.

The specimen conditions prior to consolidation (after compaction) and those prior to cyclic straining (after consolidation) are summarized in Table 1. In the testing program, three vertical stresses, σ_{vc}=48, 192 and 480 kPa, and the cyclic shear strain amplitudes, γ_c, between 0.008 and 0.24 % were applied. The cyclic compression, ε_{vc}, at the end of cycles 1, 2, 5, 10 and 30 are displayed in Table 2. Besides γ_c, N and σ_{vc}, the dry unit weight, γ_d (and corresponding void ratio, e), and the degree of saturation, S, were also varied.

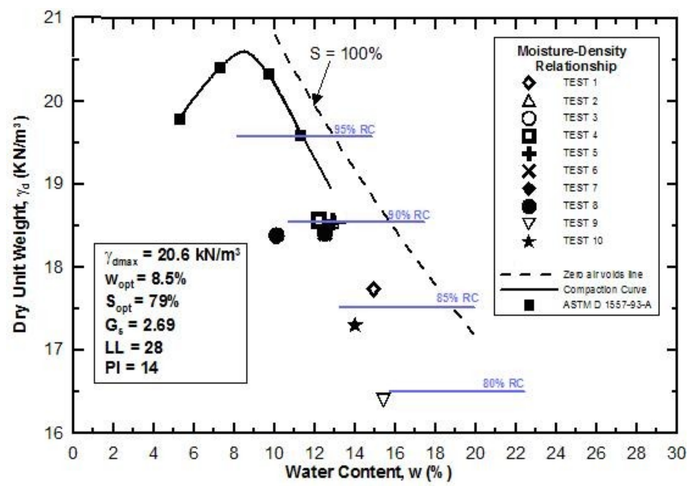


Fig. 6. Dry unit weight, γ_d, versus water content, w, chart with the modified compaction curve and data points corresponding to the conditions of specimens prior to the application of vertical consolidation stress.

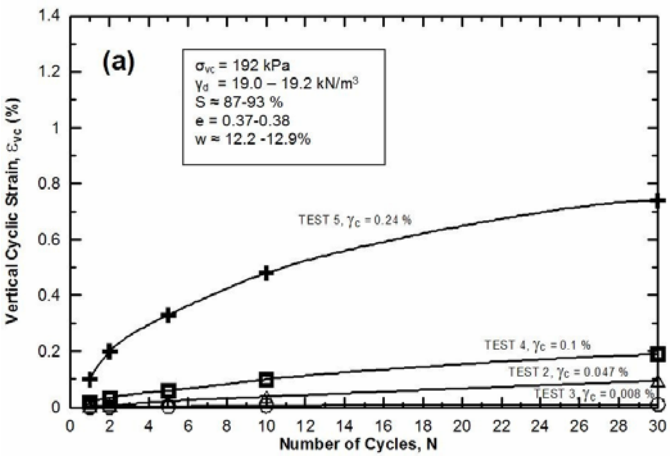
In the following chapter the cyclic compression results are compared and analyzed with respect to the specimen properties and conditions after the application of σ_{vc} and subsequent consolidation, i.e., the conditions prior to the cyclic straining. However, with the help of Table 1 and Fig. 6 the cyclic testing results can be compared straightforwardly to the specimen properties and conditions immediately after the compaction, i.e., the conditions prior to the application of σ_{vc}. Apart from the cyclic testing program, Table 1 and Fig 6 also enable the estimation of the magnitude of vertical stress required for a specific increase in the degree of compaction, which could is of interest to some experimentalists and practitioners.

It should be noted that the water contents prior to the cyclic shearing are not reported in Table 1, because they could not be determined. However, it is apparent that they were either the same as prior to the consolidation or somewhat smaller due to drainage during the consolidation. Consequently, the degrees of saturation, S, prior to the cyclic shearing could not be precisely determined either. Instead, the S values are calculated just approximately assuming the water contents prior to the consolidation and are thus reported in Table 2 as estimated.

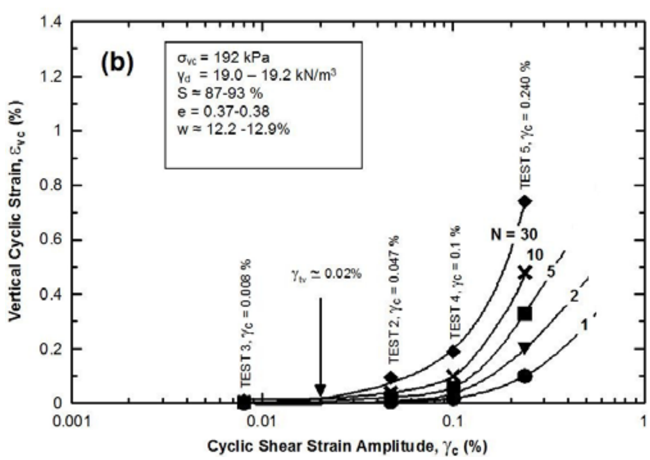
TEST RESULTS

The results of the tests are plotted in Figs. 7 through 10, displaying the effects of the cyclic shear strain amplitude, γ_c, number of cycles, N, dry unit weight, γ_d, and associated void ratio, e, normal stress, σ_{vc}, and the degree of saturation, S, on the cyclic compression, ε_{vc}.

Effects of the Cyclic Shear Strain Amplitude, γ_c, and the Number of Cycles, N



(a) Accumulation of ε_{vc} with N for different γ_c applied in different tests.



(b) ε_{vc} as a function of γ_c and N.

Fig. 7. Effect of γ_c and N on ε_{vc}.

The effects of γ_c and N on cyclic compression, ε_{vc}, are presented on Figures 7a and 7b. The figures include the results of Tests 2, 3, 4 and 5, conducted under the same vertical stress σ_{vc}=192 kPa. In these tests the dry unit weights, γ_d, prior to cyclic shearing were very similar, ranging between 19.0 and 19.2 kN/m^3, and the degrees of saturation varied relatively narrowly between $S \approx 87\%$ and $S \approx 93$ %. The two relationships show that the cyclic compression, ε_{vc}, consistently increases with γ_c and N. Such trends were expected because the same trends were obtained in all previous cyclic compression investigations.

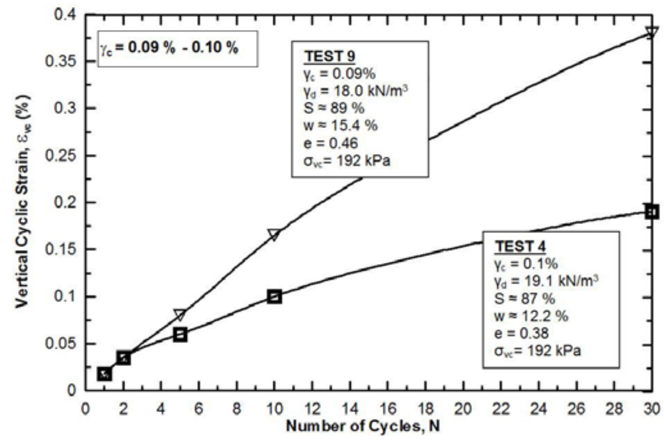


Fig. 8. Effect of dry unit weight, γ_d, on ε_{vc}.

Based on the data plotted on Figure 7b, a threshold shear strain, γ_{tv}, of about 0.02 % can be interpreted. This interpretation is consistent with the correlations between γ_{tv} and *PI* suggested by Vucetic (1994) and Hsu and Vucetic (2004).

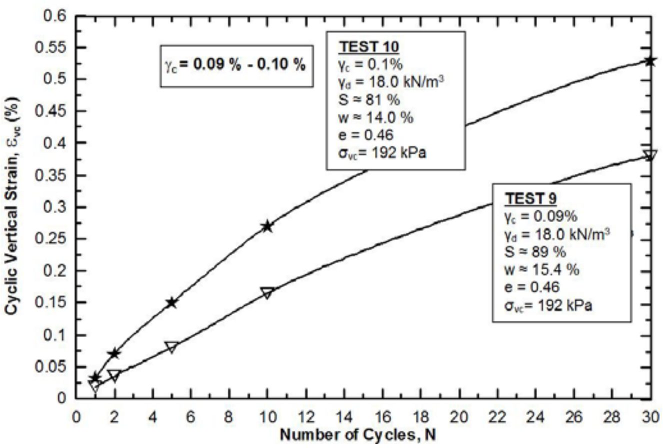
Effect of Dry Unit Weight, γ_d

The effect of dry unit weight, γ_d, and corresponding void ratio, e, on cyclic compression, ε_{vc}, is presented on Fig. 8 with the help of the results of cyclic Tests 4 and 9. Prior to the cyclic straining the Test 4 specimen had γ_d =19.1 kN/m^3 compared to γ_d =18.0 kN/m^3 for Test 9. Prior to the application of vertical stress and consolidation, the dry densities of the same specimens in Tests 4 and 9 were 18.6 kN/m^3 and 16.4 kN/m^3 respectively (see Table 1), corresponding to the relative compactions of 90 and 80 %, respectively. The shear strain amplitudes, γ_c =0.10 and 0.09 %, and the degrees of saturation, S =87.1% and 89.2 %, in these two tests were essentially the same. Both specimens were consolidated and cyclically tested under the same vertical stress σ_{vc}= 192 kPa.

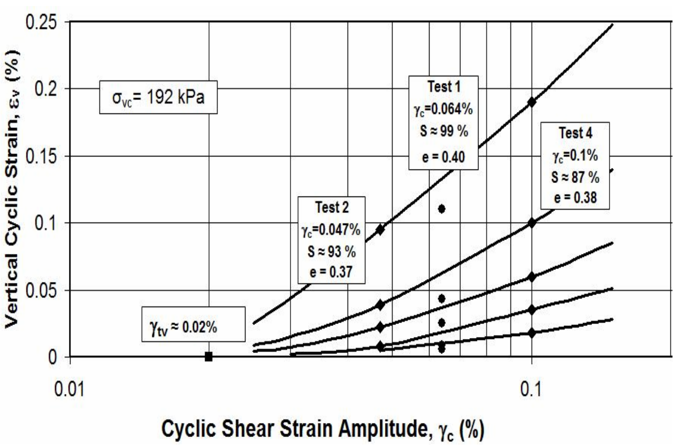
As shown on Figure 8, the cyclic compression of the two specimens was very similar in the first two cycles. However, at higher N the cyclic compression diverged significantly, with the looser sample experiencing twice as much cyclic compression after 30 cycles of straining than the denser sample. These results are in agreement with the results by Whang et al., (2004).

Effect of the High Degree of Saturation

The effects of the degree of saturation, S, on ε_{vc} presented here are applicable only for S approximately larger than 80%. They are also somewhat approximate because the values of S prior to the cyclic shearing could not be precisely determined. Before the relevant test results are presented it has to be noted that capillary tension and associated soil suction that are in part governed by S can influence cyclic compression. However, the capillary tension and its effects can be large at smaller S and are typically quite small at high levels of S. The capillary tension effects were not investigated in this study.



(a) Comparison of the results of Tests 9 and 10.

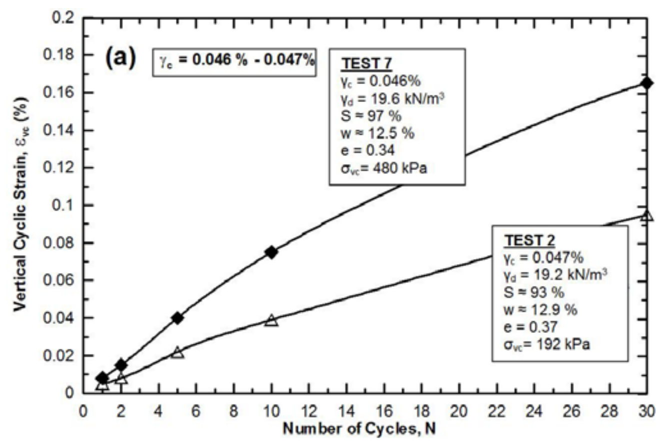


(b) Comparison of the results of Test 1 to Tests 2 and 4 in the context of the cyclic compression model in Fig. 7b.

Fig. 9. Effect of the degree of saturation, S, on ε_{vc} at high levels of S.

In Test 9 S was approximately 89% and in Test 10 around 81%, while the vertical stress was the same, σ_{vc}= 192 kPa, dry unit weights prior to cyclic shearing were identical, γ_d = 18.0 kN/m^3, and the cyclic shear strain amplitudes were practically the same, γ_c =0.09 % and 0.10 %. The results of these two tests are presented on Fig. 9a. As shown on the figure and reported in Table 2, the specimen in Test 9 having higher S exhibited around 30 to 50% smaller cyclic settlements. Similarly, if the results of Test 1 on specimen with very high S≈99% are compared to the results of Tests 2 and 4 on specimens with lower S (S ≈ 93% and S ≈ 87 %) in the context of the cyclic compression model presented in Fig. 7b, it can be noticed that ε_{vc} values from Test 1 consistently plot lower than the model would predict. This is shown in Fig. 9b. The same effect was observed in the laboratory investigations by Chu and Vucetic (1992) and Hsu and Vucetic (2002). In both of these investigations it was noticed that very high S impedes the rate of cyclic compression.

The exact mechanism responsible for such an influence of S on ε_{vc} is not clear and has yet to be fully investigated. One mechanism is clear though, and that is if the saturation is very high, such as S≈99% in test 1, soil volume can be reduced only if the water is drained out of the voids. The water drainage takes time and depends on the soil's permeability, which means that the cyclic compression in nearly fully saturated soils occurs slower and hence is smaller in a given number of cycles. The mechanism of smaller cyclic compression in soils with somewhat smaller degree of saturation around 90% or so, may be explained as follows.



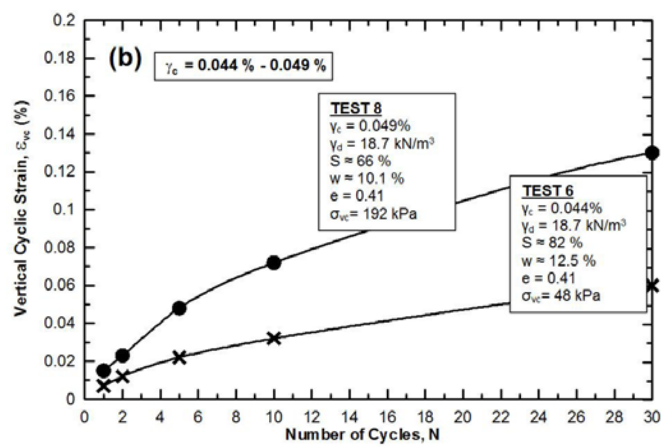


Fig. 10. Effect of vertical stress, σ_{vc}, on ε_{vc}.

In partially saturated specimens the volume reduction due to cyclic compression causes simultaneous buildup of the pressure of air and water occupying the voids. If the initial degree of saturation is high and the volume of air is so small that air bubbles are trapped in the pore water, the cyclic compaction may significantly increase the pore air pressure because it has no exit and relief. Consequently, the pore water pressure also significantly increases. During continuous cyclic compression the volume of the compressed air bubbles may eventually become so small and the air pressure so high that the air starts dissolving into the pore water. At this stage of cyclic compaction the soil is transforming from partially saturated to a nearly fully saturated. Consequently, the cyclic compression in such a state, and even before that at high saturation, can occur only if the pore water drains of out of the specimen. For example, Hsu and Vucetic (2002) observed that the rate of cyclic compression of silty and clayey soils decreases markedly when during cyclic compression S becomes larger than approximately 90%. That would mean that when S is larger than approximately 90% water needs to drain out of the specimen to allow further cyclic compaction. Such a drainage of water was actually observed during the tests. Although the writers believe that the rate of cyclic compression must be smaller when water needs to drain out to allow soil densification, the entire mechanism needs to be systematically investigated before any final conclusions are drawn.

Effect of Vertical Stress, σ_{vc}

The effect of total vertical stress, σ_{vc}, on cyclic compression, ε_{vc}, is presented on Fig. 10a using the results of Tests 2 and 7. The vertical stress applied in Test 2 was 192 kPa whereas in Test 7 it was 480 kPa. In these two tests the cyclic shear strain amplitudes, γ_c=0.047 and 0.046%, dry unit weights prior to cyclic shearing, γ_d=19.2 and 19.6 kN/m^3, and the approximate degrees of saturation, S≈93 and 97 %, were very similar. As shown on the figure, the specimen subjected to two and a half times larger vertical stress exhibited nearly two times larger cyclic compression, and that was in spite of having somewhat higher values of γ_d and S.

The effect of σ_{vc} on ε_{vc} is also displayed on Fig. 10b, where the results of Tests 6 and 8 are compared. The vertical stress applied in Test 6 was 48 kPa whereas in Test 8 it was 192 kPa. In these two tests the cyclic shear strain amplitudes, γ_c =0.044 and 0.049%, were almost the same, while the initial dry unit weights prior to cyclic shearing were the same, γ_d =18.7 kN/m^3. However, the degrees of saturation prior to cyclic shearing, S, were different. In Test 6, S was approximately 82%, while in Test 8 it was approximately 66%. This means that in Fig. 10b the effects of σ_{vc} and S are combined. As shown on the figure, the specimen having four times larger vertical stress and a lower degree of saturation exhibits about 2.2 times larger cyclic settlements. What are in this case the relative contributions of σ_{vc} and S on ε_{vc} could only be speculated. However, based on the results in Fig. 10a and the fact that S in Test 8 is only 16% lower than in Test 6, and that S in test 6 is below 90%, the writers believe that the four times larger vertical stress in Test 8 is primary reason for larger ε_{vc}.

SUMMARY AND CONCLUSIONS

Ten NGI-type cyclic strain-controlled direct simple shear tests were conducted on compacted specimens of low-plasticity clayey sand to measure the effects of various test parameters on its cyclic compression properties. The soil had 37 % fines, liquid limit LL=28% and plasticity index PI=14. The effects of the cyclic shear strain amplitude, γ_c, number of cycles, N, dry unit weight, γ_d, and corresponding void ratio, e, vertical consolidation stress, σ_{vc}, and some aspects of the degree of saturation, S, on cyclic compression, ε_{vc}, were examined. The results show that ε_{vc} increases with increasing γ_c and N, which has been consistently observed and reported earlier by others. The relationship between ε_{vc}, γ_c and N indicates that the volumetric cyclic threshold shear strain, γ_{tv}, for this material is about 0.02 %, which is consistent with correlations between γ_{tv} and PI presented elsewhere. The results from limited number of tests revealed that ε_{vc} increases as σ_{vc} and void ratio prior to cyclic shearing, e, increase (as pre-shear γ_d decreases). These trends with σ_{vc} and e, which are quite logical, have to be studied further because, surprisingly, they have not been systematically investigated in the past.

The test results suggest a simple mechanism for cyclic compression of unsaturated soils. While during shear straining soil particles and their aggregates are displaced relative to each other and their bonds are weakened, the applied vertical stress is pushing them into a denser state. Larger γ_c and N cause more soil restructuring and disturbance, making it easier for σ_{vc} to produce larger ε_{vc}. Larger σ_{vc}, on the other hand, can more effectively produce larger densification. Higher void ratio, e, and corresponding lower γ_d mean larger volume of voids that can accommodate larger densification. Similarly, at very large S above approximately 90%, the volume of air in the voids that is easily compressed is very small while the volume of practically incompressible water is large. In such a state, for the cyclic compression to take place the water needs to be drained out of the specimen. Since the water drainage takes time, cyclic compression will be smaller and dependent on soil's permeability if S is very high.

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NOTATION

The following symbols are used in this paper
e = void ratio,
G_s = specific gravity,
LL = Atterberg liquid limit,
N = cycle number or number of strain cycles,
PI = Atterberg plasticity index,
S = degree of saturation,
S_{opt} = optimum degree of saturation in modified compaction test,
w = water content,
w_{opt} = optimum water content in modified compaction test,
$\Delta\varepsilon_{vc}$ = vertical cyclic strain increment in one cycle,
ε_v = vertical strain,
ε_{vc} = accumulated vertical cyclic strain taken at the end of strain cycle,
γ = shear strain,
γ_c = cyclic shear strain amplitude,
γ_d = dry unit weight,
γ_{dmax} = maximum dry unit weight in modified compaction test,
γ_{tv} = volumetric cyclic threshold shear strain,
σ_{vc} = total vertical stress prior to and during cyclic shearing,
τ = shear stress, and
τ_c = cyclic shear stress amplitude.