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EARTHQUAKE-INDUCED SETTLEMENT AS A RESULT OF DENSIFICATION, MEASURED IN LABORATORY TESTS

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SUMMARY

Permanent volumetric strain induced by earthquake-induced densification can produce significant ground settlement that can cause damage to the overlying structures.

In order to study this strain, laboratory tests were performed in the direct-shear device on a Greek sand. On saturated samples, cyclic tests were performed for three different soil densities and two OCR values, under conditions of constant volume. In these tests, a sufficient number of cycles of constant cyclic shear stress was applied for liquefaction to develop. Following liquefaction, the volume change as a result of the dissipation of the excess pore pressures was measured. In addition, under dry conditions, cyclic tests were performed for two different initial soil densities, the same as for the tests under constant volume, and two OCR values. In these tests, a constant cyclic shear strain was applied, and the permanent volumetric strain that accumulated with number of cycles was measured.

For saturated soils the methods proposed by Tokimatsu and Seed (1987) and by Ishihara and Yoshimine (1992) are often used to evaluate this volumetric strain. For dry soils the method proposed by Tokimatsu-Seed (1987) is often used. The results obtained from the laboratory tests were compared in the case of saturated soils with the predictions of the methods by Tokimatsu-Seed and Ishihara-Yoshimine and in the case of dry soils with the predictions of the method by Tokimatsu-Seed, in terms of the soil density and the OCR value. Cases of reasonable agreement and cases of disagreement were observed.

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1. INTRODUCTION

It is generally recognized that large earthquake-induced ground deformations can cause damage of civil engineering structures (e.g. European Prestandard, 1994, Stamatopoulos et al, 2001). In level, or approximately level ground, earthquake-induced settlement of sand is caused by permanent change in volume. Cost-effective methods usually calculate this one-dimensional seismic settlement using the following procedure: (a) the soil is divided into layers of approximately uniform (i) characteristics, (ii) applied static and (iii) applied dynamic stresses, (b) the volumetric strain of every individual layer is calculated separately, (c) the corresponding settlement of every individual layer is estimated by multiplying the strain by the height of the layer and (d) the total settlement is estimated by adding the settlements.

In order to study this volumetric strain, cyclic tests were performed in the direct-shear device on a Greek sand. On saturated samples, tests were performed for three different soil densities and two different OCR values. In these tests, following liquefaction, the volume change as a result of the dissipation of the excess pore pressures was measured. Under dry conditions, tests were performed for two different initial soil densities, the same as for the tests under constant volume, and two different OCR values. The permanent volumetric strain that accumulated with the number of cycles was measured.

Different state-of-the-art-methods are often used in the earthquake design to estimate the volumetric strain in each sub-layer for sands above and below the water table once the distribution of maximum earthquake-induced shear stress versus depth is known. The methods proposed by Tokimatsu and Seed (1987) and Ishihara and Yoshimine (1992) have widespread acceptance (e.g. Seco e Pito, 2003).

Below, first the stress conditions relevant to the present study, the relevant laboratory tests and the state-of-the-art methods predicting the earthquake-induced volumetric strain are described. Then the current testing program and its results are given. Finally, the test results are compared with the predictions of the state-of -the-art methods.

2. STRESS CONDITIONS, RELEVANT TESTS, AND STATE-OF-THE-ART-METHODS PREDICTING THE VOLUMETRIC STRAIN

Conditions of level ground

In level ground and approximately homogeneous soil, the geometry can be approximated as one-dimensional: Only variations with depth, z, are considered. Prior to the application of cyclic loading, the shear stresses σ_{xz}, σ_{yz} and σ_{xy} , where x and y are horizontal mutually perpendicular directions, are about zero. The effective normal vertical stress, denoted as $\sigma'_{v\text{-}o}$, equals the overburden effective pressure. The ratio of the effective horizontal stress, denoted as $\sigma'_{h\text{-}o}$, to $\sigma'_{v\text{-}o}$, is given by the coefficient of earth pressure at rest, K_o. The factor K_o is usually less than unity, and depends on the type of soil and its stress history. As a result of an earthquake, dynamic loading is applied primary in the horizontal direction, as shear stress $\sigma_{xy.}$

Relevant laboratory tests

The response of cyclic loading under level ground can be simulated in the laboratory by cyclic tests using various devices and procedures. These procedures can be classified as of two types: (a) tests with isotropic consolidation and oscillation of the vertical stress about the isortopic state (e.g. Seed and Peacock, 1971), and (b) tests where samples are subjected to one-dimensional consolidation and oscillation of the horizontal shear stress $\tau=\sigma_{xy}$ about a zero mean stress value (e.g. Seed and Peacock, 1971). In tests (b), the ratio of the effective horizontal to the effective vertical stress is the coefficient K_o. Tests (a) are performed in the triaxial chamber, while tests (b) can be performed in the direct-shear device, the simple-shear device, or the torsional shear device with zero lateral strain. Only K_o-consolidated cyclic tests where the cyclic stress is applied horizontally simulate the initial state of stress for level ground, as well as the primary direction of loading under earthquakes. It is inferred that these laboratory tests are relevant in the present study.

Under earthquake loading, non-uniform cycles of shear stress are applied. These irregular dynamic cyclic shear stresses can be converted for testing purposes to uniform cycles. Seed (1979) proposes the following simplified procedure: The equivalent uniform cyclic (that is, harmonic) shear stress τ_{cyc} has magnitude equal to $\{0.65\tau_{max}\}$, where τ_{max} is the maximum applied shear stress during the earthquake. The number of equivalent uniform cycles is determined from the earthquake magnitude according to table 1.

Cyclic stress ratio, cyclic strain, permanent strain

When harmonic shear horizontal loading is applied on K_o-consolidated soil about a zero mean shear stress value, we define a cycle of loading as the complete change of the shear stress σ_{xy} (i) from zero to τ_{cyc} , (ii) from τ_{cyc} to $-\tau_{cyc}$ and (iii) from $-\tau_{cyc}$ back to zero. Cyclic stress ratio SR is defined as the ratio of the cyclic shear stress τ_{cyc} to the effective vertical stress prior to the application of cyclic loading, $\sigma'_{v\text{-}o}$. Cyclic shear strain during a loading cycle, γ_{cyc}, is defined as the maximum value of shear strain attained. Double-amplitude cyclic shear strain during a loading cycle is defined as the sum of the absolute value of the maximum and minimum value of shear strain attained. Permanent strain, or pore water pressure, is the strain, or pore water pressure that accumulates at the end of each cycle of loading.

State-of-the-art method predicting the earthquake-induced volumetric strain above the water table

Cyclic simple-shear laboratory tests with zero initial shear stress performed by different researchers (e.g. Silver and Seed, 1971) illustrate that permanent volumetric strain of dry sand induced by cyclic loading depends on the following: (a) the exerted cyclic shear strain, γ_{cyc}, (b) the soil density and (c) the number of cycles applied. It is always compressive and increases with cycle number at a decreasing rate. Similarly, Tokimatsu and Seed (1987) suggest to calculate the volumetric strain of dry soil layers during an earthquake by using: (a) the cyclic shear strain γ_{cyc}, (b) the corrected blow count from the Standard Penetration Test (SPT), N_1, (e.g. European Prestandard, 1994), determining sand density and (c) the magnitude of the earthquake exerted, as expressed by the "equivalent" number of cycles applied, of $\tau_{cyc}=0.65\tau_{max}$.

On the basis of γ_{cyc} and the corrected blow count number N_1, the permanent volumetric strain for 15 uniform loading-unloading-reloading cycles, $\varepsilon_{vol\text{-}N=15}$, can be estimated from figure 1 (Tokimatsu and Seed, 1987). For a cycle number other than 15, or for earthquakes with a magnitude other than 7.5, the corrective coefficients of table 1 are recommended. Finally, to simulate the effect of the multi-directional motion applied on-site but not in a laboratory, it is recommended to double volumetric strain $\varepsilon_{vol\text{-}N=15}$.

State-of-the-art methods predicting the earthquake-induced volumetric strain below the water table

Cyclic soil strength

As a result of constant-volume horizontal harmonic loading with zero initial shear stress on K_o-consolidated samples, permanent pore pressure and cyclic shear strain build up with cycle number, while due to one-dimensional symmetry, considerable permanent horizontal strain does not accumulate (e.g. Ishihara, 1993, Stamatopoulos and Whitman, 1987). Cyclic strength for N cycles of harmonic loading, SR_N, is defined as the value of the cyclic stress ratio SR causing liquefaction in N uniform cycles. Liquefaction is the state where the effective stress becomes very small and the cyclic shear strain very large. In a more functional definition this state is defined in the present work, similarly to Ishihara (1993), as when the double-amplitude cyclic shear strain exceeds 5%. For the loose samples, this value corresponds to dramatic loss of strength, while for dense samples to excessive deformation for civil engineering design purposes. Finally, the factor of safety against liquefaction, is defined as the ratio SR_N/SR.

In liquefaction analyses, a reference earthquake of magnitude M=7.5, that corresponds to 15 cycles of harmonic loading is often used (e.g. European Prestandard, 1994, Seed et al, 1983). For this reason, the cyclic strength SR_{15} is used below as the index of cyclic soil strength.

Volumetric strain

On the basis of a combination of laboratory tests and on-site measurements, Tokimatsu and Seed (1987) correlate volumetric strain of clean sand after liquefaction in an earthquake with a magnitude of M=7.5, with the sand density, expressed with the corrected blow count value in a standard penetration, N_1, and the applied cyclic stress ratio SR. They propose the relationship given in Fig. 2a. It is observed that volumetric strain depends mainly on the sand density, while the effect of the exerted cyclic stress ratio is small. Fig 2a allows to assess the volumetric stain of sand in earthquakes with a magnitude of M=7.5, in conjunction with the cyclic shearing stress.

An alternative method predicting volumetric strain of saturated sand has been suggested by Ishihara and Yoshimine (1992). Volumetric strain is assessed from (a) the sand density, e.g. in accordance with the value N_1 in the standard penetration test, and (b) the factor of safety against liquefaction, defined previously, using the curve of Fig. 2b.

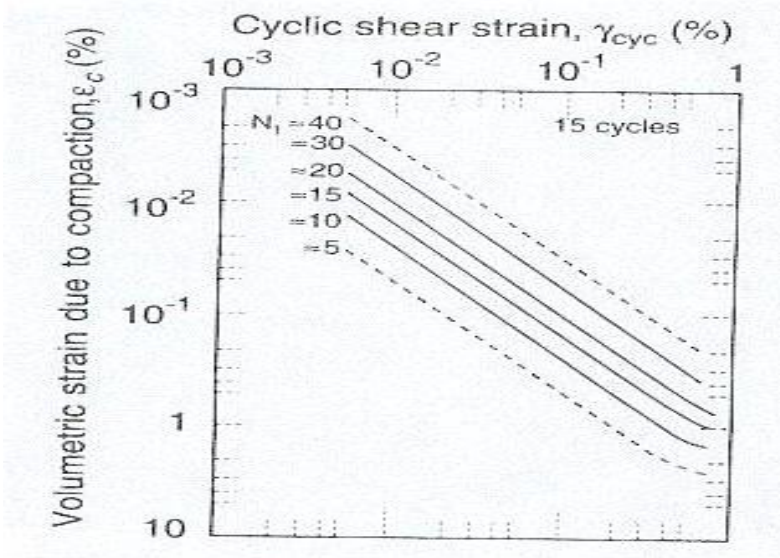


Fig. 1 Permanent volumetric strain of dry sands for an earthquake of magnitude 7.5 (or equivalently for 15 cycles of cyclic loading) in terms of the cyclic shear strain, and the corrected N value of the SPT. (Tokimatsu and Seed, 1987).

(a) (b)

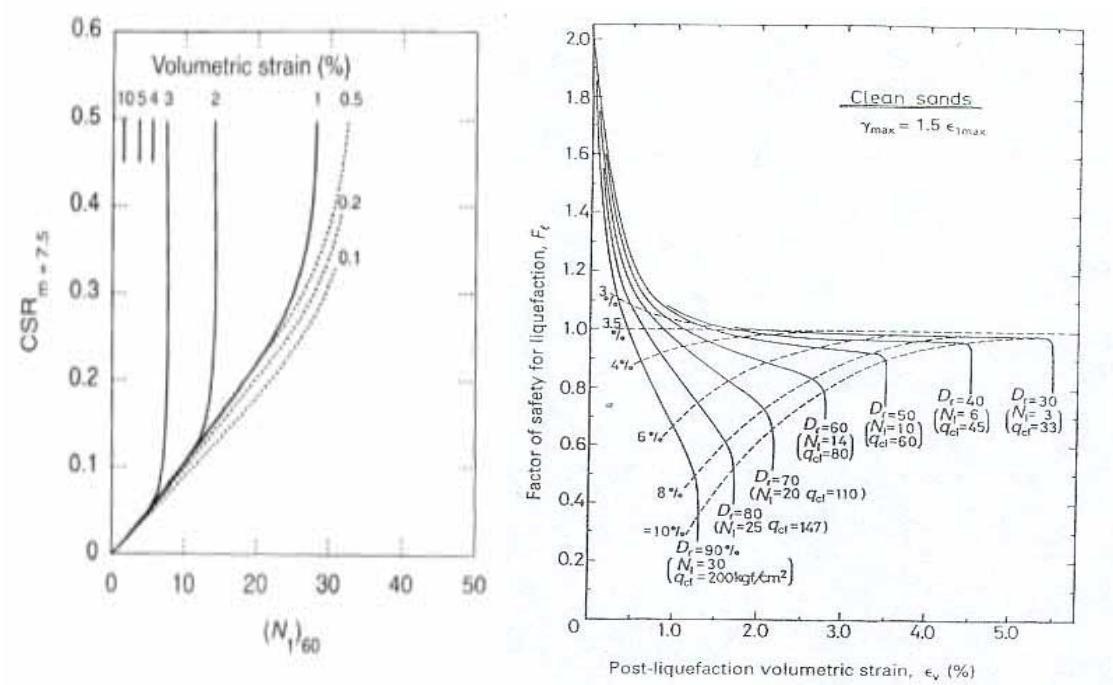


Fig. 2. Permanent volumetric strain of saturated sands (a) according to the Tokimatsu and Seed, (1987) method for an earthquake of magnitude 7.5 and (b) according to Ishihara and Yoshimine, 1992) in terms of the factor of safety against liquefaction.

Table 1. The effect the magnitude of an earthquake in volumetric deformation for dry sand. (By Tosimatsu and Seed, 1987)

Earthquake Magnitude (M)	5.25	6	6.75	7.5	8.5
Equivalent number of cycles (N)	2	5	10	15	26
$\varepsilon_{vol.M} / \varepsilon_{vol\ -N=15}$	0.4	0.6	0.85	1.0	1 .25

3. CURRENT TESTING PROGRAM

Sand tested

The sand tested was prepared by putting together similar specimens from various borings and depths from the region Schinia, near Athens, Greece. Its grain size distribution is given in Fig. 3. Content of fines is 11% and the plasticity practically zero. Specific gravity of grains is 27.1kN/m^3.

Tests performed

The sand was subjected to cyclic loading oscillating in the direct-shear device, either under constant-volume, or under (complete) drainage. In addition, monotonic constant-volume and drained tests were performed in the same direct-shear device. The device used in all tests was a Wykeham Farrance apparatus in compliance with British Standard 1377. The modification described e.g. by Saada and Townsend (1980) was used in the device to evaluate the undrained and cyclic soil strengths. The concept is to maintain a constant specimen volume during shear by adjusting the vertical stress. Equivalent pore pressures are determined based upon magnitude of vertical stress changes.

Cyclic direct-shear constant-volume tests were performed on saturated samples with zero initial shear stress. Cycles of cyclic shear stress were applied until liquefaction, defined as the condition of cyclic double-amplitude shear strain equal to 5%. After the condition of liquefaction, volume change was measured, as the effective stress increased to its initial value prior to the application of cyclic loading.

Cyclic direct-shear drained tests were performed on (approximately) dry samples with zero initial shear stress value. Forty cycles of constant cyclic shear strain were applied. The induced shear stress and volumetric strain were measured.

Test specimens were prepared at three densities that correspond to the loose, medium and dense states. The initial dry densities γ_{d-o} were 14.0, 16.4, 17.5 kN/m^3. The corresponding initial void ratios e_o were 0.93, 0.68, 0.54. All specimens had a height of 2cm and an area of 6X6cm^2. They were prepared by the method of moist placement (described e.g. by Ishihara, 1993).

Two types of stress paths were applied prior to the application of cyclic loading, described in table 2. In stress path {s-1}, prior to the application of cyclic loading the OCR value equals 1 and the vertical effective stress was 294kPa. In stress path {s-2} the OCR value equals 3 and the vertical effective stress was 294 kPa.

Table 2. Different consolidation procedures used in the present study.

Symbol	Description
{s-1}	Increase of the vertical stress from 0 to 294kPa and allow for consolidation
{s-2}	Increase of the vertical stress from 0 to 882kPa and allow for consolidation. Then unload to 294kPa and allow for consolidation.
{s-3}	Increase of the vertical stress from 0 to 147kPa and allow for consolidation

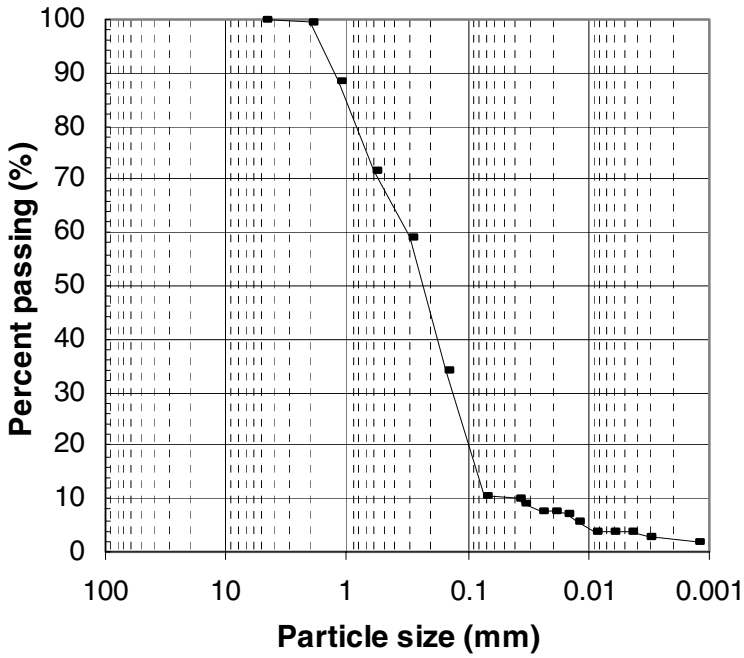


Fig. 3. Grain size distribution of the sand tested.

At least four cyclic constant-volume tests were performed on samples with consolidation following the stress paths {s-1} and {s-2}, at all three initial void ratios. Cyclic drained tests were performed also on samples with consolidation stress paths {s-1} and {s-2} and at the initial void ratios e_o of 0.93 and 0.68. Monotonic constant-volume and drained tests were performed on samples with consolidation following the stress paths {s-1} and {s-3}, described in table 2, and at all initial void ratios. Finally, oedometer tests were performed to measure the consolidation characteristics at the initial sample preparation densities, both during virgin loading and under unloading.

4. MEASURED PROPERIES OF THE SAND

Compressibility

Table 3 gives the void ratio of the sand, at the end of different states of consolidation, as measured in the oedometer.

Table 3. The void ratio after consolidation and the cyclic soil strength SR_{15} , in terms the stress path during consolidation, and the sample preparation void ratio, e_o.

	Consolidation Case {s-1}		Consolidation Case {s-2}	
e_o	e	SR_{15}	e	SR_{15}
0.93	0.71	0.08	0.66	0.19
0.68	0.64	0.13	0.57	0.30
0.54	0.51	0.21	0.49	0.38

Strength under monotonic undrained loading

At the monotonic constant-volume tests, the large-strain (or steady-state) shear stress τ_{ss} and effective vertical stress $\sigma'_{v\text{-}ss}$ were measured. Both were measured at a shear displacement of 9mm (corresponding shear strain about 45%), assuming that the vertical and shear force exerted during the tests is applied at the failure plane of the soil specimen, that is horizontal. Fig. 4 gives the final measured vertical effective stress $\sigma'_{v\text{-}ss}$ of the two sands in terms of the void ratio, e. Correlations are linear in the semi-log plot. The stress $\sigma'_{v\text{-}ss}$ seems to be uniquely related to the void ratio, and this relationship does not depend on the applied vertical stress prior to shearing, $\sigma'_{v\text{-}o}$. This is consistent with the theory of the critical state, where the steady-state effective confining stress for similar modes of failure is uniquely related to the void ratio, and this relationship does not depend on other factors such as the initial confining stress. Thus, we can write

$$e = \alpha_1 + \beta_1 \log (\sigma'_{v\text{-}ss}) \qquad (1)$$

The parameters α_1 and β_1 obtained by linear regression were 1.01 and –0.19 respectively. The coefficient of correlation R^2 equals 0.93. The stress is in kPa.

The large-strain friction angle

The large-strain friction angle, φ_{ss}, of the sand was estimated from monotonic drained tests and was found equal to 30°. This angle was consistent with the friction angle estimated from the large-strain effective stress ratio ($\tau_{ss}/\sigma'_{v\text{-}ss}$) measured in the constant-volume monotonic tests.

Liquefaction and volume change during consolidation after liquefaction

Figs 5 presents the number of cycles to liquefaction in terms of the applied cyclic stress ratio SR, for each state prior to cyclic loading, for all constant-volume cyclic tests performed. From these figures, the cyclic soil strength SR_{15}, defined previously, was obtained in terms of soil density and OCR value. It was obtained using regression of the SR value versus the logarithm of the number of cycles to liquefaction. It is given in table 3.

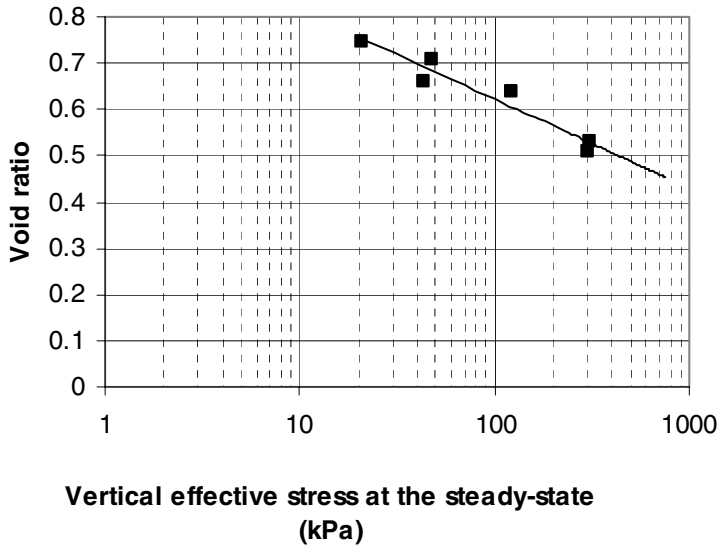
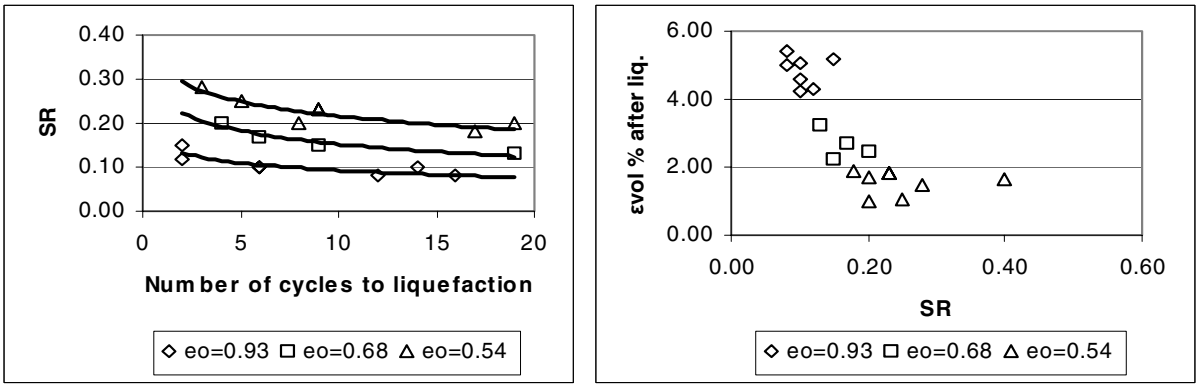


Fig. 4. The vertical effective stress at the steady-state (large-strain) versus the void ratio for the sand tested.

(a)



(b)

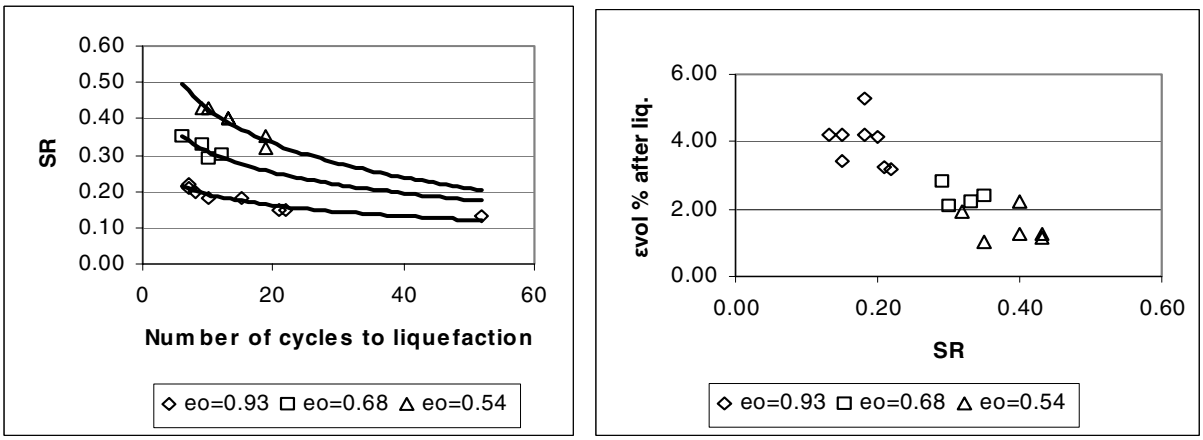
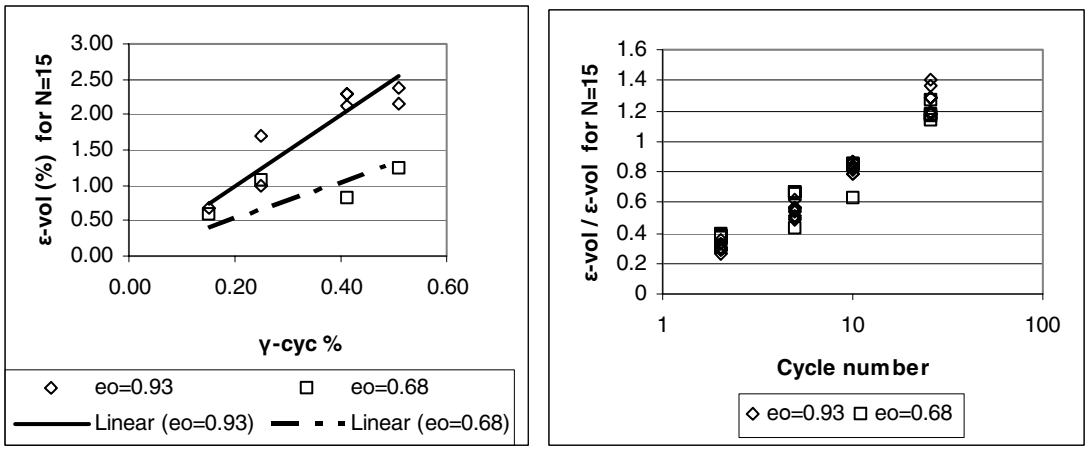


Fig. 5. Cyclic undrained rests. The measured liquefaction curves and the measured volumetric strain after liquefaction in terms of sand density for tests where the consolidation stress paths (a) {s-1} and (b) {s-2} are applied.

(a)



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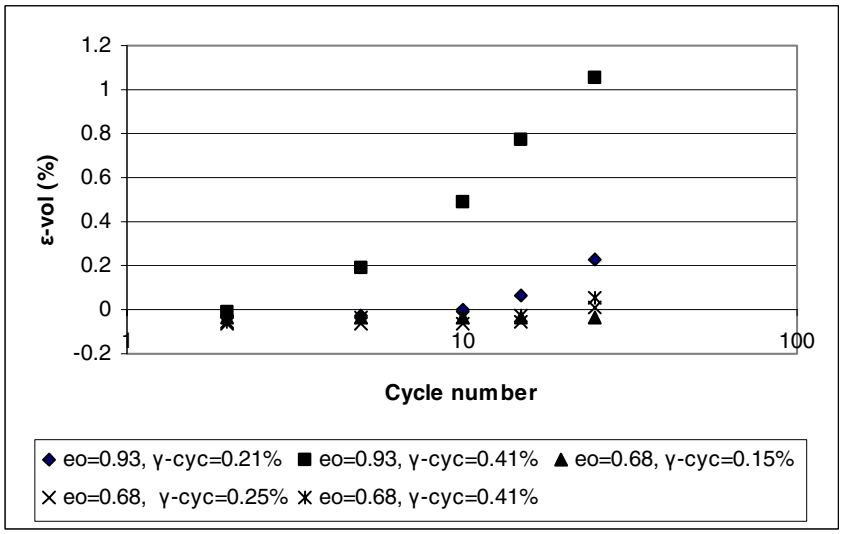

Fig. 6. Cyclic drained rests. (a) Tests with OCR=1. Measured permanent volumetric strain after 15 cycles in terms of the applied cyclic shear strain and measured effect of cycle number on the volumetric strain. (b) Tests with OCR=3. Measured volumetric strain with cycle number.

Figs 5 also present the corresponding volumetric change after liquefaction for all constant-volume cyclic tests performed, in terms of the applied stress ratio SR, the void ratio prior to consolidation e_o, and the stress path during consolidation. It can be observed that volumetric strain after liquefaction is about independent of SR, but depends on the density. For tests with OCR=3, the volumetric strain for similar e_o decreases, but not by much.

Volume change during cyclic drained loading

In the cyclic drained tests on samples with OCR=1, similarly to the Tokimatsu-Seed method assumptions, compressive permanent volumetric volumetric accumulated with cycle number. The measured cyclic stress ratio was more-or-less constant during the tests and varied in all tests performed from 0.16 and 0.33. Fig. 6a presents the measured permanent volumetric strain after 15 cycles for all tests performed in terms of the applied cyclic shear strain. Fig. 6a presents the

measured increase of the permanent residual volumetric strain with the number of cycles of loading-unloading-reloading.

In the cyclic drained tests on samples with OCR=3, contrary to the Tokimatsu-Seed method assumptions, dilative permanent volumetric strain accumulated in the first cycles of applied cyclic shear strain. Then, compressive permanent volumetric accumulated. The magnitude of the dilative permanent volumetric strain was larger for the denser samples. The measured cyclic stress ratio was more-or-less constant during the tests and varied in all tests performed from 0.20 and 0.37. Fig. 6b presents the measured permanent volumetric strain with the number of cycles of loading-unloading-reloading of all tests performed.

5. COMPARISON OF TEST RESULTS WITH THE STATE-OF THE ART METHODS

Saturated conditions

The volumetric strain following liquefaction predicted by the methods of (a) Tokimatsu-Seed (T-S) and (b) Ishihara-Yoshimine (I-Y) of the tests performed varies in terms of the initial void ratio and the stress path during consolidation of the samples. Table 4 gives the predicted values of volumetric strain after liquefaction, estimated by the methods of (a) T-S and (b) I-Y of all tests performed, in terms of the initial void ratio e_o and the consolidation procedure of the samples. The prediction of the T-S method was based on the N_1 value that corresponds to the measured soil strength SR_{15} of table 3, according to Seed et al (1983). In addition, the factor SR, was taken equal to SR_{15}. The reason is that the measured volumetric strain corresponds to a factor of safety against liquefaction equal to 1 for an earthquake of magnitude 7.5. The predictions of the I-Y method were obtained from N_1, estimated similarly. The factor of safety against liquefaction of the tests performed was taken one.

Fig. 7 compares the measured to predicted results of all tests. Table 5 gives the statistics of the ratio of predicted by measured values. It can be observed that:
- (i) The results of all tests performed are predicted better by the T-S method, than by the I-Y method.
- (ii) For specimens with an OCR value of 1, the volumetric strain is predicted reasonably by the T-S method. The ratio between the predictions and the measurements varies between 0.7 and 1.4 and has an average value of 0.9. The ratio generally decreases as the soil density decreases.
- (iii) For specimens with on OCR value of 3, the average ratio between the predictions by the T-S method and the measurements is 0.6. This indicates that the decrease in the volumetric strain induced by preloading is less than that corresponding on the increase of the cyclic soil strength, or equivalently the N_1 value. This is possibly a result of the fact that the volumetric strain depends only on soil density, while liquefaction susceptibly is affected additionally by the horizontal stress that increases with the OCR value (e.g. Seed and Peacock, 1971).

Table 4. Volumetric strain after liquefaction and under dry conditions in terms of the initial density and the stress path during consolidation predicted by the methods of (i)Tokimatsu-Seed (T-S) and (ii) Ishihara-Yoshimine (I-Y).

Case				Estimated value ε_{vol} (%) for saturated conditions		Estimated value ε_{vol} (%) for dry conditions		
e_o	Stress path	SR_{15}	N_1	With the method T- S	With the method I-Y	γ_{cyc}= 0.10%	γ_{cyc}= 0.30%	γ_{cyc}= 0.50%
0.93	{s-1}	0.08	4.50	4.00	1.80	0.60	1.70	2.40
0.68	{s-1}	0.13	7.50	2.60	1.50	0.35	1.00	1.60
0.54	{s-1}	0.21	14.00	1.40	1.15	0.17	0.50	0.80
0.93	{s-2}	0.19	13.00	1.50	1.20	0.19	0.60	0.90
0.68	{s-2}	0.30	19.00	1.20	0.90	0.11	0.32	0.52
0.54	{s-2}	0.38	21.50	1.10	0.80	0.09	0.28	0.48

Dry conditions

The volumetric strain under dry conditions predicted by the method of Tokimatsu-Seed (T-S) of the tests performed varies for given applied cyclic shear strain in terms of the initial void ratio and the stress path during consolidation of the samples. Table 4 gives the values of volumetric strain after 15 cycles of drained loading predicted by the T-S method in terms of the applied cyclic strain, for the initial void ratios e_o and consolidation procedures of all states studied. The cyclic soil strength and the corresponding value of N_1 of the sand for each initial density and stress path, needed in the predictions, is taken according to table 3, as described above for the saturated condition.

According to the previous discussion, and as the tests with OCR=3 have a qualitatively different response than the predictions of the T-S method, only tests with OCR=1 will be compared. Fig. 8a compares the measured to predicted by the T-S method volumetric strain after 15 cycles of all drained tests with OCR=1. Fig. 8b compares the measured to predicted effect of cycle number on the volumetric strain. Table 5 gives the statistics of the ratio of predicted by measured values. It can be observed that:

- (i) The volumetric strain after 15 cycles is predicted reasonably well by the T-S method. The ratio between the predictions by the T-S method and the measurements varies between 0.7 and 1.5, with an average value of 1.1.
- (ii) The effect of cycle number is predicted reasonably well by the factors of table 1. The ratio between the predicted and measured values varies from 1.5 to 0.8 with an average value of 1.1. More scatter exists for N=2. This is, possibly, a result of the low accuracy of measurements after only 2 cycles.

Table 5. Comparison of predicted and measured volumetric strain. Ratio of predictions and measurements.

(a) Maximum, minimum and average values.

Case	**Maximum** OCR=1, OCR=3	**Minimum** OCR=1, OCR=3	**Average** OCR=1, OCR=3
Cyclic undrained tests			
Predicted by T-S method / Measured	1.41, 1.10	0.74, 0.28	0.92, 0.56
Predicted by I-Y method / Measured	1.16, 0.80	0.33, 0.23	0.55, 0.42
Cyclic drained tests (OCR=1)			
Predicted $\varepsilon_{vol\text{-}N=15}$ / Measured	1.60, -	0.74, -	1.06, -
Predicted effect of cycle number / Measured	1.55, -	0.77, -	1.06, -

(b). Average values in terms of soil density

Case	**e_o=0.93** OCR=1, OCR=3	**e_o=0.68** OCR=1, OCR=3	**e_o=0.54** OCR=1, OCR=3
Cyclic undrained tests			
Predicted by T-S method / Measured	0.84, 0.39	1.00, 0.51	0.95, 0.82
Predicted by I-Y method / Measured	0.38, 0.31	0.58, 0.38	0.78, 0.60
Cyclic drained tests (OCR=1)			
Predicted εvol(N=15) / Measured	1.07, -	1.03, -	-
Predicted effect of cycle number / Measured	1.03, -	1.12, -	-

(a) (b)

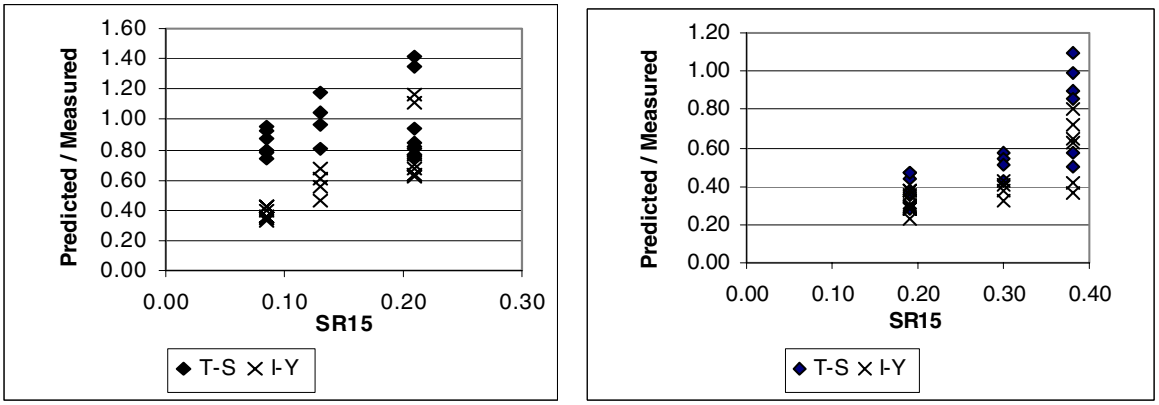

Fig. 7. Cyclic undrained rests. Comparison of predicted by the T-S and I-Y methods with the measured volumetric strain following liquefaction. (a) Tests with OCR=1, (b) tests with OCR=3.

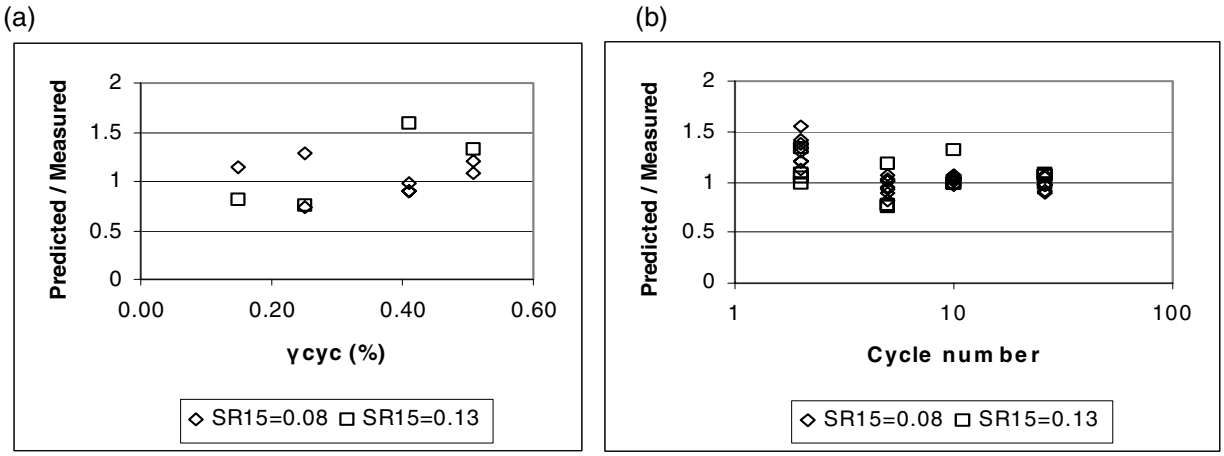

Fig. 8. Cyclic drained rests with OCR=1. Comparison of predicted by the T-S method and measured (a) permanent volumetric strain after 15 cycles and (b) effect of cycle number on the permanent volumetric strain.

6. CONCLUSIONS

Direct-shear cyclic drained and undrained tests were performed for three different soil densities and two OCR values. In the drained tests, the accumulation of permanent volumertic strain with cycle number was recorded. In the undrained tests, the consolidation settlement after liquefaction was measured.

Under saturated conditions, the Tokimatsu-Seed (T-S) method predicts better the measured volumetric strain of all tests performed than the Ishihara-Yoshimine (I-Y) method. In addition, for samples with OCR=1, the measured volumetric strain after liquefaction was predicted with reasonable accuracy with the T-S method. For samples with OCR=3, the measured volumetric strain after liquefaction was underpredicted with the T-S method by a factor of about 0.6. This indicates that the decrease in the volumetric strain induced by preloading is smaller than the increase of the cyclic soil strength.

Under dry conditions and samples with OCR=1, the measured volumetric strain was similar to that predicted by the Tokimatsu-Seed (T-S) method. Under dry conditions and samples with OCR=3, the measured response showed considerable dilation at least at the first cycles of loading, that is not qualitatively predicted with the T-S method.

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