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Volumetric Compression Characteristics of Unsaturated Sandy Soils under Cyclic Shear for Estimation of Settlement

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ABSTRACT

A method is presented for assessing the amount of settlements at the surface of the unsaturated backfill, by using the relationships between shear stresses, shear strains and volumetric strains, obtained from stress-controlled torsional cyclic shear tests under drained conditions. The method is based on the cumulative damage theory in which shear stress time histories based on seismic response analysis of the ground are taken into consideration. Backfill settlements due to the Niigata-ken Chuetsu-oki Earthquake in Japan (2007) are calculated by using the proposed method. The calculated amounts of the settlements were estimated to have good agreement with the observed data.

Introduction

Niigata-ken Chuetsu-oki Earthquake that occurred on July 16, 2007 caused settlement ranging from few centimeters to 1m and more in a wide range of backfilled ground of Kashiwazaki-Kariwa nuclear power plant. Especially, large settlement exceeding 1m occurred around the reactor building and the turbine building, and uneven settlement of the buildings and foundation of peripheral devices caused damage to the devices (Sakai et al. 2009, Sato et al. 2009).

Most of the backfilled ground of the power plant was sandy soil. Therefore, it was presumed that liquefaction caused sand boil and settlement at the places having high ground water level around revetment. This kind of settlement arising from liquefaction is extensively known (Ishihara 1999, Sawada et al. 2006). On the other hand, with regard to settlement at the places with low ground water level, sand boil etc. was also not found at almost all points except the revetment side. Therefore, it was presumed that settlement primarily occurred due to unsaturated backfilled ground shallower than the ground water level. Most of the backfilled ground in the power plant was in unsaturated state. Such case has been rarely reported so far.
The valuable study about seismic compression of the unsaturated soil were performed by Stewart et al. (2004) , who used laboratory testing to investigate under the drainage condition.

In the present study, based on the hypothesis that unsaturated sandy soil ground was subject to cyclic shear under exhaust and drainage conditions, we would attempt to explain the seismic

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settlement phenomenon that really occurred. We mainly covered the following three points in the present study, (1) quantification of volumetric shrinkage due to cyclic shear that is the main cause of settlement occurring in the unsaturated sandy soil ground at the time of earthquake, (2) evaluation of settlement amount, (3) verification of evaluation methods.

Settlement of the backfilled ground after earthquake in the power plant

Figure.1 shows an example of settlement distribution measured in the backfilled ground in the South-East side of Unit 1 reactor building (#1R/B) after 2007 Niigata-ken Chuetsu-oki Earthquake. Settlement measured at the points 12-20 m away from the building was 30-40 cm after the earthquake. Settlement rapidly increased in the vicinity of building, and it was about 57 cm at a place that was about 3 m away from the building, and it was about 87 cm alongside the wall. Based on the observation results of ground water level at these points, ground water level was found in the vicinity of border (about G.L.-25m) of the backfilled ground and the reactor installation ground, and we could verify that the backfilled soil was in unsaturated state.

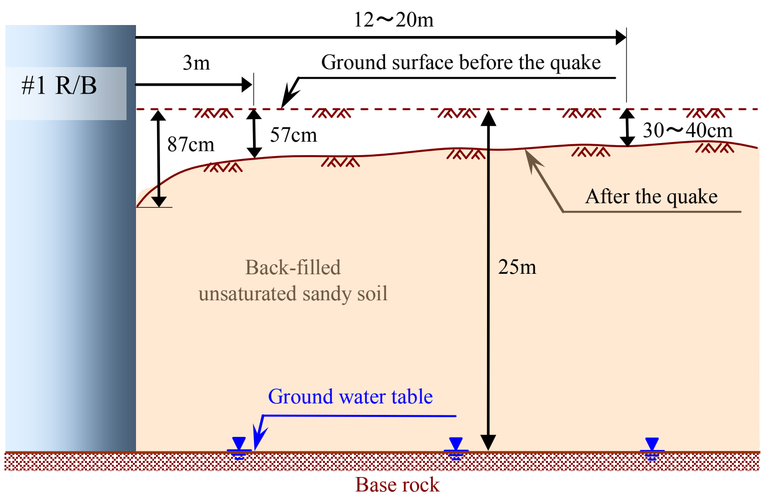


Figure.1 Settlement of ground surface in the vicinity of Unit 1 reactor building (#1 R/B)

Volumetric compression characteristics based on cyclic shear test

Test conditions and test cases

We collected sample from the surface of the backfilled ground at the point located about 15m away from the external wall of the South East side of Unit 1 reactor building, and on the hollow cylindrical specimen made with the target dry density 1.6g/cm^3 and moisture content 20% (external diameter 70 mm, internal diameter 30 mm, height 70 mm), and we applied cyclic shear using torsional shear tester equipped with double cell equipment, and measured the amount of change (shrinkage) in volume.

As for the average confining pressure σ'_m in the torsional cyclic shear test, considering the layer thickness (25m) of the backfilled ground, we used three types of confining pressure, namely σ'_m=50, 100, and 300 kPa. Initial stress state is isotropic consolidation state. We changed the shear stress ratio $SR_d = \tau_d / \sigma'_m$ in four stages with respect to each confining pressure, and we conducted loading test by applying cyclic shear load based on the sine wave (frequency 0.1Hz) with the stress control approach in all 12 cases. Cyclic shear process is performed in exhaust drainage condition. Table-1 shows confining pressure in each test case, and dry density and maximum shear stress ratio after consolidation. Besides, in the same table, we have also shown the test cases where we applied the shear load of irregular wave form simulating the observed ground motion described later.

Table.1 Test condition of torsional cyclic shear test

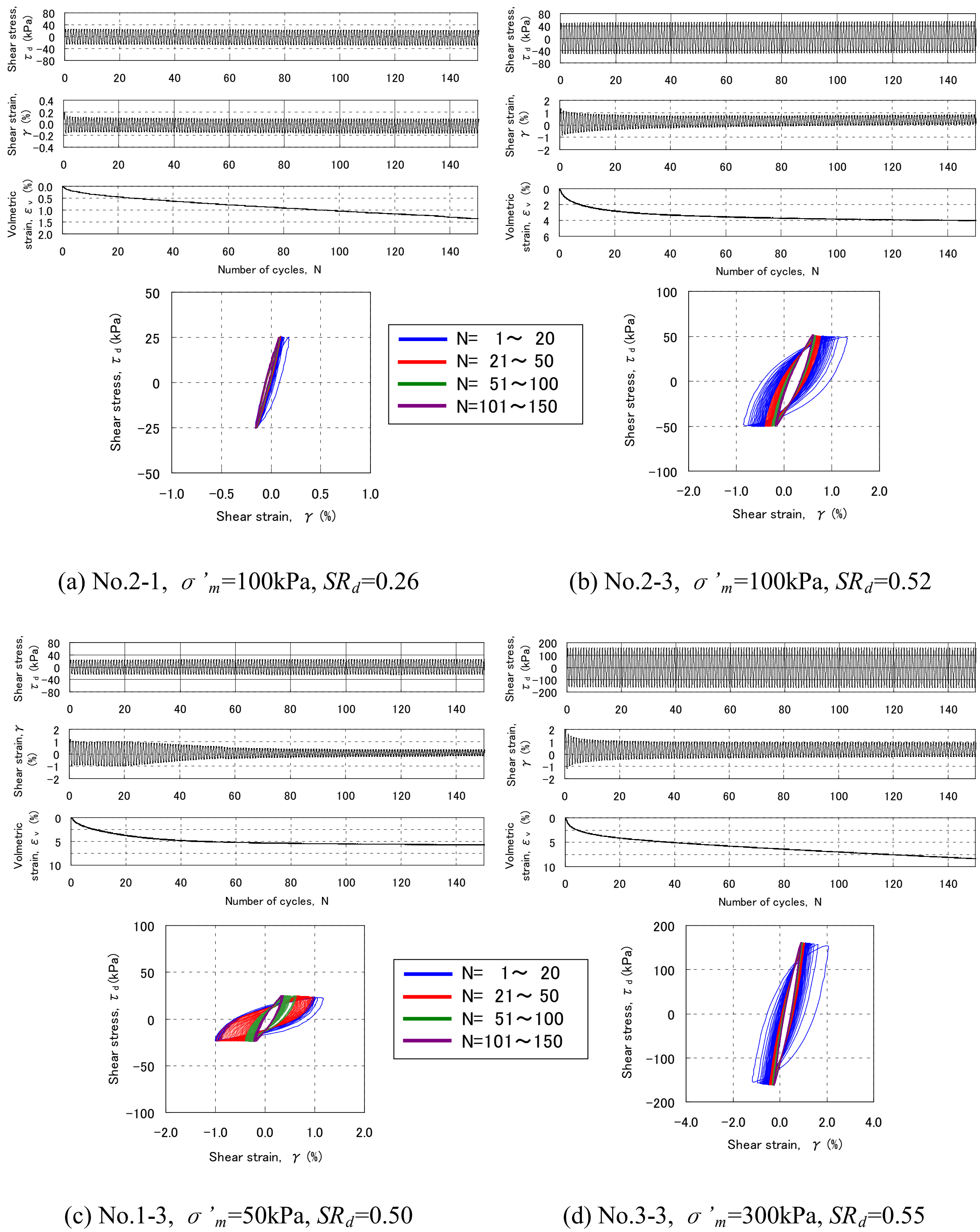
Shear load type	Test number	Confining pressure σ'_m (kPa)	Dry density after consolidation ρ_d (g/cm^3)	Shear stress ratio $SR_d = \tau_d / \sigma'_m$
Sine wave (0.1Hz)	1-1	50	1.673	0.23
	1-2		1.667	0.35
	1-3		1.708	0.50
	1-4		1.665	0.58
	2-1	100	1.686	0.26
	2-2		1.718	0.39
	2-3		1.706	0.52
	2-4		1.706	0.60
	3-1	300	1.833	0.21
	3-2		1.832	0.42
	3-3		1.815	0.55
	3-4		1.797	0.65
Irregular wave	R-1	50	1.627	0.72
	R-2	100	1.688	0.83
	R-3	300	1.767	0.68
	R-4	463	1.799	0.65

Initial state : Dry density ρ_{d0} =1.60g/cm^3, Moisture content w_0 =20%

Volumetric compression characteristics based on cyclic shear test

Figure.2 shows 4 cases (test number 2-1, 2-3, 1-3, and 3-3) of torsional cyclic shear test results. Test number 2-1 and 2-3 are the cases that showed maximum shear stress ratio $SR_d = \tau_d / \sigma'_m$ of 0.26 and 0.52 under the condition of average confining pressure σ'_m = 100kPa. In the case of test number 2-1 where the maximum shear stress ratio is 0.26, changes in shear strain due to cyclic load is not significant. However, volumetric strain tends to increase slightly on the compression side. Against this, in the case of test number 2-3 where the shear stress ratio is 0.52, shear strain is high in the initial loading phase, and with the increase in the number of cycles, it tends to gradually decrease and converge in a constant value. In the initial loading phase, volumetric strain rapidly increases on the compression side with the occurrence of a large shear strain, and after converging into a constant value, shear strain shows a constant increment.

This is because of volumetric shrinkage of the specific due to repetitive loading, resulting in the increase of shear rigidity. Even in the relation of shear stress and shear strain, it is found that secant line gradient of the hysteresis curve gradually increases with the increase in the number of cycles. It is needless to say that we didn't find this kind of situation in the cyclic shear test of saturated sandy soil that leads to liquefaction in the undrained condition. This is perhaps because of the cure phenomenon of rigidity arising from the decline of void ratio due to cyclic shear on unsaturated sandy soil under exhaust drainage condition.



(a) No.2-1, σ'_m=100kPa, SR_d=0.26

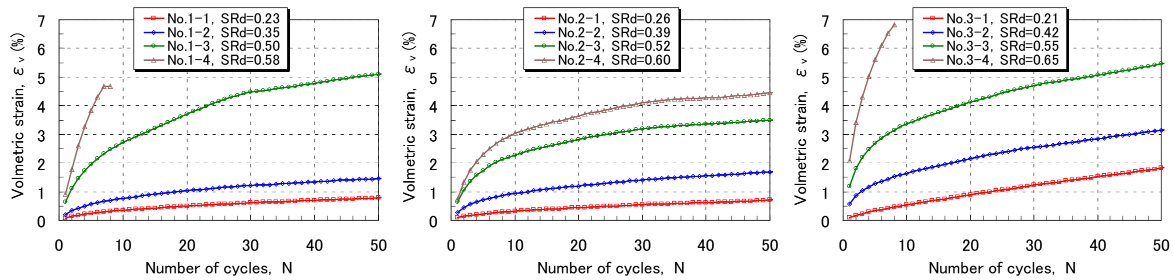
(b) No.2-3, σ'_m=100kPa, SR_d=0.52

(c) No.1-3, σ'_m=50kPa, SR_d=0.50

(d) No.3-3, σ'_m=300kPa, SR_d=0.55

Figure.2 Examples of the torsional cyclic shear test results for unsaturated sand

Test number 1-3 and 3-3 were conducted under the condition of average confining pressure σ'_m = 50kPa and 300kPa respectively, and they showed the maximum shear stress ratio τ_d / σ'_m 0.5 and above. Case of test number 2-3 carried out at average confining pressure σ'_m = 100kPa and the maximum shear stress ratio are almost same. Therefore, we would compare the test results of these three cases. When the maximum shear stress ratio is almost same, smaller is the average confining pressure, higher is the number of cycles required until large shear strain occurred during the initial loading decreases. Even when we look at the rise in secant line gradient of the hysteresis curve in the relation between shear strain and shear stress, it is evident that more number of cycles are required when the shear modulus has increased.



(a) σ'_m=50kPa (b) σ'_m=100kPa (c) σ'_m=300kPa

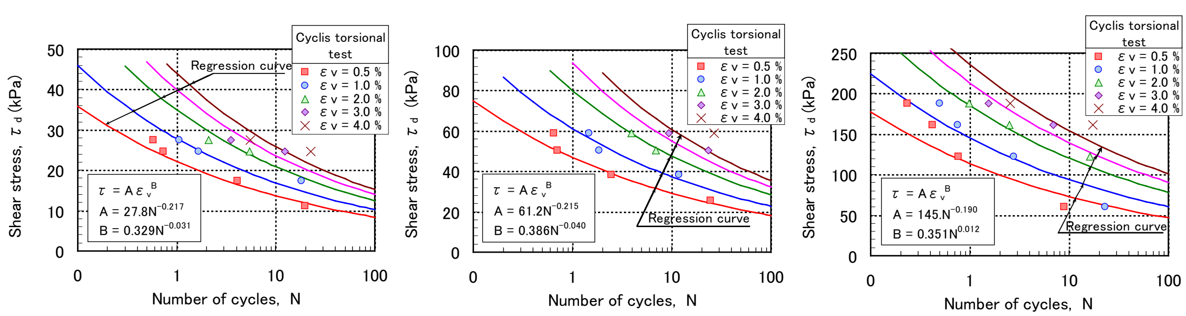
Figure.3 Relationship between volumetric strain and number of cycles

Figure.3 shows the trend of volumetric strain up to 50 cycles for each confining stress. From these, it is clear that under all confining stress conditions, volumetric strain increases with the increase in the number of cycles and shear stress ratio ($SR_d = \tau_d / \sigma'_m$).

Evaluation of settlement

Setting equivalent volumetric strain curves

In the evaluation of settlement, firstly, we would set an empirical formula of volumetric strain characteristics obtained from the torsional cyclic shear test.



(a) σ'_m=50kPa (b) σ'_m=100kPa (c) σ'_m=300kPa

Figure.4 Relationship between shear stress and number of cycles

With regard to torsional cyclic shear test results shown in Figure.3, Figure.4 shows the relation between the number of cycles (N) and shear stress (τ_d) such that volumetric strain (ε_v) becomes equal (hereinafter referred to as equivalent volumetric strain curves) for each confining stress. This figure shows an empirical formula (1) that approximates the equivalent volumetric strain curves with exponential function. After introducing confining stress dependence formula related to α and rearranging the equivalent volumetric strain curves, we get formula (2).

$$\tau_d = A \cdot \varepsilon_v{}^B \ , \quad A = \alpha \cdot N^a \ , \quad B = \beta \cdot N^b \tag{1}$$

$$\varepsilon_v = \left[\frac{\tau_d}{(0.464\sigma'_m + 9.81)N^{-0.207}} \right]^{\frac{1}{0.355N^{-0.0199}}} \tag{2}$$

Here, ε_v: volumetric strain (%), τ_d: shear stress (kPa), σ'_m: average confining stress (kPa), N: number of cycles, and α, a, β, b : regression coefficients.

Evaluation of settlement

With respect to shear stress determined from the response analysis based on the ground motion observed on the foundation of no. 1 nuclear reactor building, we would apply equivalent volumetric strain curves of the unsaturated ground, and we would calculate volumetric strain of the unsaturated ground based on the concept of cumulative damage.

Table.3 shows the ground model of seismic response analysis. This ground model defines the model of unsaturated backfill soil layer 25m, and its low-level Nishiyama mudstone layer that is the bedrock of the nuclear reactor building. With T.P.-284.0m(G.L.-289.0m) that shows shear wave velocity (Vs) of 700 m/s and above as the basis, with regard to shallower Nishiyama mudstone layer and backfilled soil layer, we considered shear strain (γ) dependence characteristics of shear modulus (G) and damping constant (h). In addition, the density dependency did not consider, because the wet density (= 1.9 g/cm^3) of the backfill soil layer of the targeted area for examination was approximately constant (Sakai et al. 2009, Sato et al. 2009). For the ground motion entered in the formula, we took the acceleration time series observed on the foundation of no. 1 nuclear reactor building (G.L.-37.5m= T.P.-32.5m), and for this wave, we entered E+F at the same depth of ground model shown in Table.3 (E: incident wave, F: reflective wave).

For the calculation of volumetric strain ε_v based on the cumulative damage theory, we used the following approach using the method of Tateyama et al. (1999) described below was used.

a) Obtain the time history waveform of shear stress at the depth based on the results of seismic response analysis.
b) For shear stress time history waveform, determine shear stress τ_{d1}, τ_{d2} … for every half amplitude (half pulse) based on the zero crossing method.
c) Calculate the volumetric strain consecutively at the designated depth based on the cumulative damage theory using the shear stress of half pulse and equivalent volumetric strain curves formula (2).
d) Multiply later thickness with the volumetric strain of respective depth and calculate surface settlement as from its total value.

Table.3 Ground model for seismic response analysis

Layer	T.P. (m)	ρ_t (g/cm^3)	V_s (m/s)	$G/G_0 \sim \gamma$, $h \sim \gamma$
Backfill	+5.0 / -20.0	1.90	100～350	$G/G_0=1/(1+9.01\ \gamma^{0.77})$ $h=29.4\gamma/(\gamma+0.088)+1$
Nishiyama	-32.5	←input	540	$G/G_0=1/(1+2.19\ \gamma^{1.0})$ $h=8.50\ \gamma^{0.749}$
	-62.0			
	-77.0	1.72	590	
	-103.0		620	
	-128.0		650	
	-192.0		670	
	-284.0		690	
Bedrock		1.72	730	—

Figure.5 shows the maximum response value of acceleration, shear stress and shear strain of the backfilled ground based on the seismic response analysis. Depth distribution of the maximum volumetric strain shown in Figure.6 is calculated using the aforementioned steps a) through d).

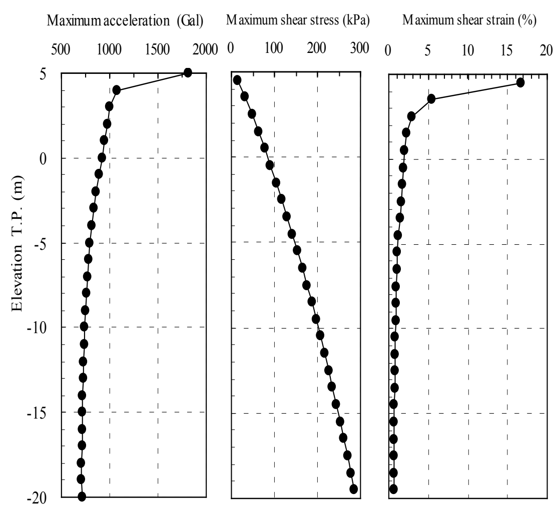


Figure.5 Results of seismic response analysis

Figure.7 shows the time history of ground surface settlement along with the acceleration waveform of input earthquake motion. The calculated value of ground surface settlement is about 50 cm, and it is more or less consistent with settlement 30 – 40 cm actually measured in the general parts of ground around no. 1 nuclear reactor building. Besides, large vibration appearing around 13 seconds of input earthquake motion has caused about 80% of the overall settlement.

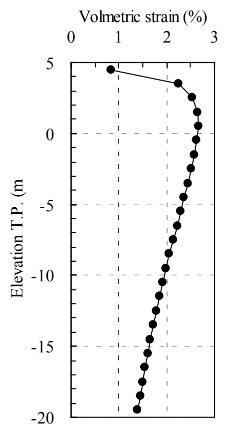


Figure.6 Depth distribution of the maximum volumetric strain

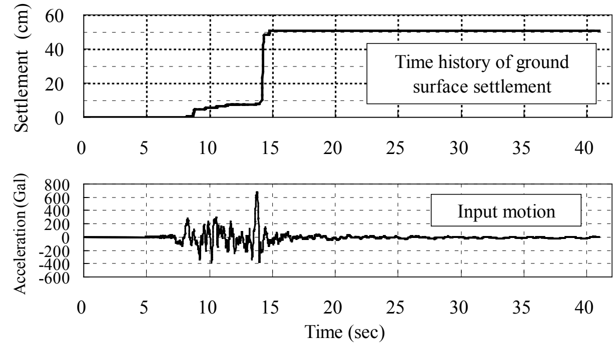


Figure.7 Time history of ground surface settlement by analysis

Verifying the applicability of settlement evaluation methods

With regard to applicability of the aforementioned evaluation methods of settlement, we measured the volumetric strain that occur by directly applying irregular shear load having large-amplitude pulse on the specimen (hereinafter referred to as irregular wave test), and verified the measurement results by comparing with the evaluation results. Irregular wave test was conducted under 4 different confining stress (50, 100, 300, 456kPa) conditions. We applied irregular shear load of each of 4 points calculated with seismic response analysis. Figure.8 shows an example of irregular wave test result. In all stress results shear strain and volumetric strain occur according to the fluctuation of shear stress amplitude. Especially, occurrence of a large volumetric strain due to large amplitude pulse is very unique. Figure.9 shows the time history of volumetric strain evaluated with the proposed methods. Evaluation results could more or less reproduce the features of volumetric strain time history in the irregular wave test. We presume that we could verify the applicability of settlement evaluation method.

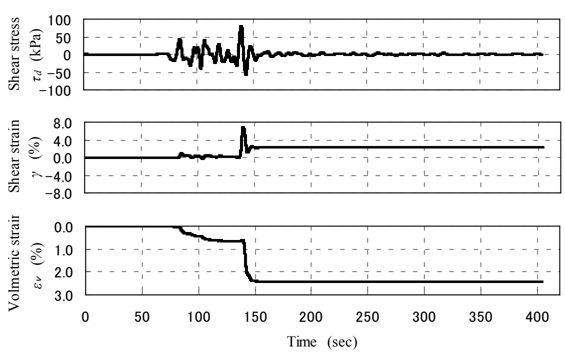


Figure.8 Example of the irregular wave test (σ'_m=100kPa)

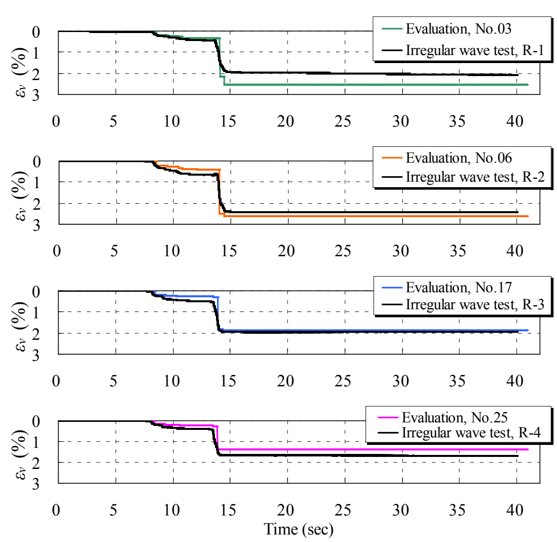


Figure.9 Evaluation by proposed method versus irregular wave test

Conclusions

(1) From the torsional cyclic shear test under exhaust drainage condition, we could quantitatively understand the volumetric compression characteristics of unsaturated sandy soil subjected to the cyclic shear stress loading.
(2) For the evaluation of the ground surface settlement, it is necessary to establish stress-dependent characteristics of volumetric compression (equivalent volumetric strain curves) based on the torsional cyclic shear test.
(3) Estimated settlement, based on the cumulative damage analysis by using equivalent volumetric strain curves and shear stress time history due to the seismic response analysis, approximately matched with the observed data at the unsaturated backfill in the power plant due to 2007 Niigata-ken Chuetsu-oki earthquake.
(4) Volumetric shrinkage evaluated based on cumulative damage analysis was almost same as the result of cyclic shear test using irregular wave. From this result, we presume that we could verify the applicability of the evaluation method.

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