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SETTLEMENT EVALUATION OF GRAVEL AND SAND DUE TO EARTHQUAKE

EVALUATION DE TASSEMENTS DES GRAVIERS ET DES SABLES CAUSES PAR TREMBLEMENT DE TERRE

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SYNOPSIS : Undrained cyclic triaxial tests were conducted on undisturbed gravelly-soil samples obtained by the in-situ freezing method, and on reconstituted gravelly-soil samples, to investigate the volume change characteristics after undrained cyclic shear loading. The tests showed that the volume change characteristics are dependent on the maximum shear strain of the specimen, relative density and difference between the maximum void ratio and minimum void ratio, irrespective of the effective confining pressure, grain size distributions and whether or not the samples are intact. On the basis of these test results, a method for evaluating residual settlement of dense gravelly or dense sandy ground induced by earthquake shaking is proposed. Moreover, by comparing calculated results by the method with the results of large-scale shaking-table tests, it was shown that the proposed method permits a rough estimate of residual settlements of ground induced by earthquake shaking.

INTRODUCTION

Accurate evaluation of the mechanical properties of dense gravelly soils is needed more than ever, because important structures are being constructed on dense gravelly ground. In assessment of seismic stability, the dense gravelly-soil layer, which can be a bearing strata for very important structures, merely shows cyclic mobility during strong earthquake motion, whereas the loose-sand layer shows liquefaction. However, even dense gravelly ground will subside after earthquakes because of dissipation of the residual excess pore-water pressure generated by the earthquake shaking. Therefore, it is very important to evaluate the residual settlement of the ground in order to secure the safety of the structure.

In this research, we investigated volume change characteristics after undrained cyclic shear loading for gravelly soil samples obtained by in-situ freeze sampling, or for reconstituted samples, which leads to a proposal of a method for evaluating residual settlement of dense gravelly and sandy ground induced by earthquake shaking.

CYCLIC TRIAXIAL TESTS FOR GRAVELLY SOIL

Field tests were performed at three sites: T, KJ and K, where in-situ freeze sampling and dynamic penetration tests in adjacent boreholes were performed. The soil profile, grain size distributions, test procedure and results of undrained cyclic shear tests at these three sites have already been described in other literature (Tanaka et al. 1991a). The physical and mechanical properties of the gravelly soils are listed in Table 1.

Although the lateral surfaces of the undisturbed gravelly-soil specimens were fairly smooth, some specimens were smoothed further by a cutting machine in a refrigerator in order to minimize impreferable effect of so-called membrane compliance. A detailed description of the lateral surface treatment is available in other literature (Tanaka et al. 1991b).

Table 1 Some test results of the three gravelly soils

Site and samples		Physical properties: Mean grain size (mm)	Maximum grain size (mm)	Fines content (%)	Undrained cyclic strength, R_s
K-site		15.0~30.0	100~150	less than 2.0	1.13
T-site	Upper layer	3.0~20.0	50~200	less than 5.1	0.42
	Lower layer	8.0~20.0		less than 8.1	0.33
KJ-site		3.0~7.0	40~60	less than 3.9	1.27

It has already been noted that in undrained cyclic triaxial tests, gravelly soils tend to deform less uniformly than sands, resulting in considerable constriction (Lee and Fitton 1968, Tanaka et al. 1986). According to Tanaka et al. (1986), constriction of gravelly-soil samples becomes marked when double amplitude of axial strain ε_{DA} is above 3%. Therefore, we assumed that the gravelly-soil specimens deform uniformly up to ε_{DA}=3%. However, for values of ε_{DA} above 3%, only those specimens which were rather uniformly deformed were used to obtain the empirical relationships described later.

RESIDUAL VOLUMETRIC STRAIN BY TRIAXIAL TEST

Sasaki et al. (1982), Tatsuoka et al. (1984) and Kokusho et al. (1984) investigated the volumetric strain in sands after cyclic shear loading using a torsional simple shear test apparatus or a cyclic triaxial test apparatus and found that the product of the volumetric strain, ε_{vr}, and relative density, D_r, is uniquely related to the maximum amplitude of the strain, irrespective of the effective overburden pressure. Therefore, the relationships between ε_{vr} and the maximum amplitude of axial strain, ε_{DAmax} divided by D_r are investigated for the three gravelly soils as shown in Fig. 1, where one should note that relative density and all the strains are expressed as percentage in this paper. The results for the gravelly-soil layer at T-site indicate that the relationship is not greatly affected by the differences in effective confining pressures and lateral-surface treatment. The same results also

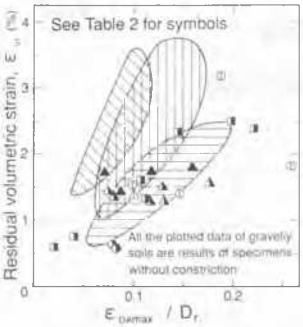

Fig.1 ε_{vr} vs. ε_{DAmax} / D_r relation

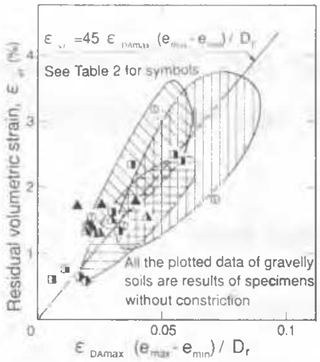

Fig.2 ε_{vr} vs. ε_{DAmax} (e_{max} - e_{min}) / Dr relation

Table 2 Symbols in Figs. 1, 2, 5, 6, 7 and 8

Sample description			Effective confining pressure, (kPa)	Undisturbed sample: Non-treated	Undisturbed sample: Treated*1	Reconstituted sample (non-treated)
Gravelly soils	T-site	Upper layer	176			
			441 or 490		—	—
		Lower layer	118		—	
	KJ-site		157	—		
	K-site		98		—	
Sands (Kokusho et al. 1988)	Toyoura sand		98	—		
	Tonegawa sand		29, 49, 98 or 196			
	Narita sand		49, 98 or 196			

*1 Lateral surfaces of specimens were made further smoother by a cutting machine in a refrigerator.

show that the relationship is little affected by whether or not the sample is intact. The test results for sands are also plotted on the same graph. The values of ε_{vr} for gravelly soils are similar to that for Toyoura sand, but the values of ε_{vr} for the other two sands are larger than those of gravelly soils.

The difference between gravelly soils and sands shown in Fig. 1 is probably attributable to the different compressibility of the samples due to the difference in grain size distributions. The different compressibility of the samples due to the difference in grain size distributions is sometimes expressed by the difference between maximum and minimum void ratios, e_{max} - e_{min}. Generally, compressibility of samples with same relative density is inclined to increase with an increase in the value of e_{max} - e_{min}. Therefore, in this paper, e_{max} - e_{min}, was selected as an index expressing the different grain size distributions. Figure 2 shows the relationship between $\varepsilon_{DAmax}(e_{max}-e_{min})/D_r$ and ε_{vr}. There seems to be a unique relationship between ε_{vr} and $\varepsilon_{DAmax}(e_{max}-e_{min})/D_r$, regardless of the grain size distributions. The solid line in Fig. 2 showing the average relationship is expressed as follows :

$$\varepsilon_{vr} = 45\ \varepsilon_{DAmax}(e_{max} - e_{min})/D_r \qquad (1)$$

COMPARISON BETWEEN RESULTS OF TRIAXIAL TESTS AND SIMPLE SHEAR TESTS

Figure 3 shows results of the simple shear test (Sasaki, et al. 1982 ; Tatsuoka, et al. 1984 ; Ishihara, et al. 1992) together with the average relationship of the triaxial test results, expressed by Eq.(1). The vertical axis corresponds to $\varepsilon_{vr} D_r/(e_{max} - e_{min})$; the horizontal axis is marked with ε_{DAmax} together with the maximum double amplitude of shear strain, γ_{DAmax} . The conversion from ε_{DAmax} into γ_{DAmax}, was performed by assuming that the value of γ_{DAmax} equals

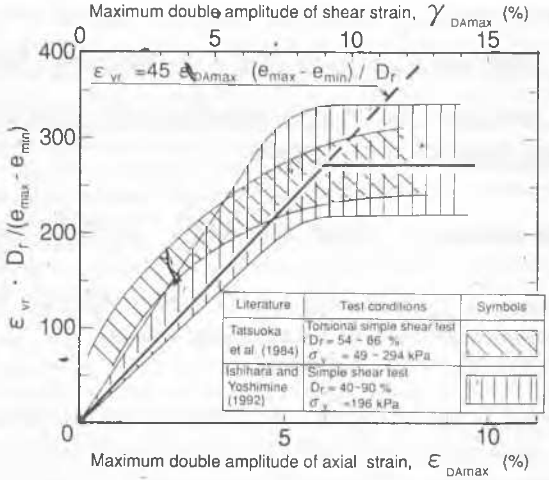

Fig.3 Comparison of triaxial test results and simple shear test results

1.5 times as much as the value of ε_{DAmax}. Figure 3 indicates that the relationship based on simple shear tests approximates the average relationship based on the triaxial test results expressed by Eq.(1) in the region of $\gamma_{DAmax} = 0 \sim 9$ %. However, for a value of γ_{DAmax} above 9 % (ε_{DAmax} above 6%), Eq.(1) tends to overestimate the residual volumetric strain. This might be attributed to the type of shear test apparatus. In cyclic triaxial tests, the directions of principal strains during shearing coincide with that during reconsolidation after shearing, whereas that is not the case in simple shear tests. Therefore, the decrease in stiffness of the samples during cyclic triaxial loading seems to bring direct increase in compressibility during reconsolidation even in the region of ε_{DAmax} above 6 % . This paper focuses on estimating the settlement of level ground caused by shear waves, which propagate upward. Therefore, the results by triaxial tests should be modified considering the results from the simple shear apparatus. The thick solid line in Fig. 3 instead of the dashed line , which corresponds to Eq.(1) should be used to predict the residual volumetric strain. The thick solid line is expressed by the following equation.

$$\frac{\varepsilon_{vr} \times D_r}{e_{max} - e_{min}} = \begin{cases} 45\ \varepsilon_{DAmax} & (0 \le \varepsilon_{DAmax} \le 6\%) \\ 270 & (6\% \le \varepsilon_{DAmax}) \end{cases} \qquad (2)$$

PARAMETERS REQUIRED TO ESTIMATE RESIDUAL VOLUMETRIC STRAIN BY STRESS RATIOS

In order to calculate the residual volumetric strain, ε_{vr}, using Eq.(2), it is necessary to determine the value of ε_{DAmax}. Since, it is generally said that the value of shear strain is much more affected by methods of dynamic response analysis than the value of shear stress, it is difficult to obtain the accurate value of shear strain by simple calculation. In this paper, empirical equations are derived to calculate the value of ε_{vr} by stress ratios as described in the following.

Figure 4 shows the relationships between the double amplitude of axial strain, ε_{DA} and number of loading cycles, N_c, in undrained cyclic triaxial tests. The value of ε_{DA} increases with the number of loading cycles. The plotted data in Fig.4 indicate that the value of ε_{DA} seems to increase linearly from about ε_{DA} = 2%. Therefore, the value of ε_{DA} above 2% can be expressed easily using two different numbers of loading cycles. For the values of ε_{DA} less than 2 %, we assumed that the relationship between ε_{DA} and N_c can be approximated using a

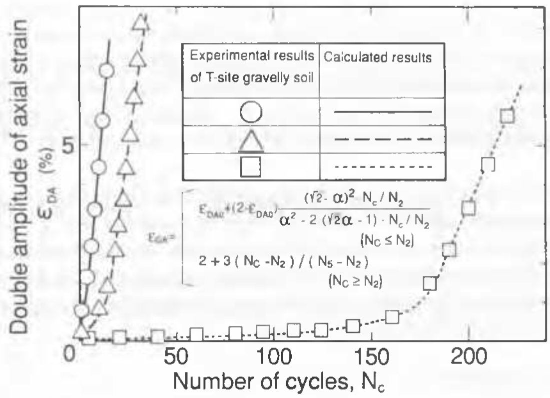

Fig.4 Comparison between experimental and calculated results

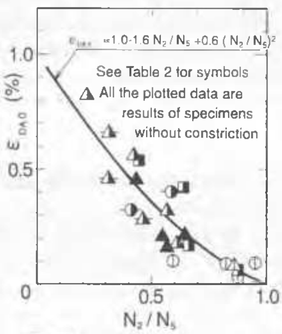

Fig.5 ε_{DA0} vs. N_2 / N_5 relation

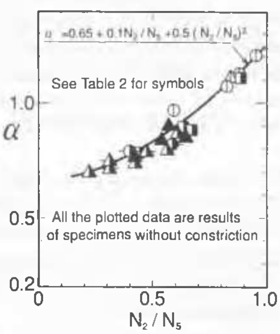

Fig.6 α vs. N_2 / N_5 relation

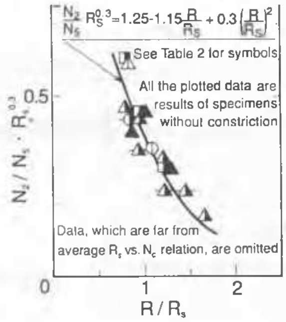

Fig.7 $N_2 / N_5 \cdot R_s$ vs. R / R_s relation

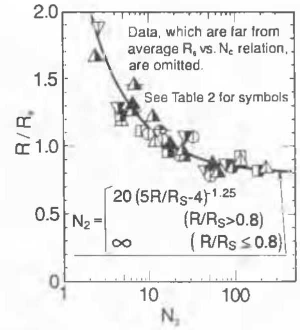

Fig.8 R / R_s vs. N_2 relation

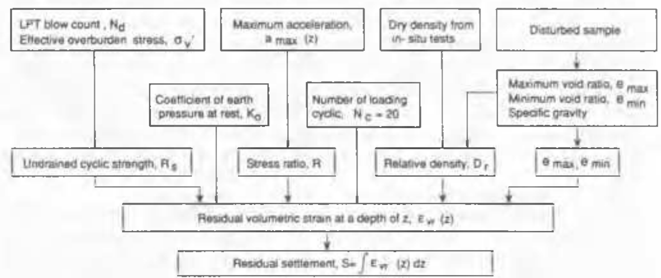

Fig.9 Procedure to calculate residual settlement of dense gravelly or dense sandy deposits

coordinates-transformed hyperbolic function. Finally, the relationships between ε_{DA} and N_c are expressed by the equation shown in Fig.4. In Fig.4, N_2, N_5, ε_{DA0} and α mean as follows :

N_2, N_5 : Numbers of cycles at $\varepsilon_{DA} = 2\%$ and 5%, respectively

ε_{DA0} : Apparent intersection at $N_c = 0$ of ε_{DA} vs. N_c relationship

α : Parameter determining shape of ε_{DA} vs. N_c curve

From a geometrical consideration, the value of ε_{DA0} and α might be related to the value of N_2/N_5. Expectedly, both the values of parameters ε_{DA0} and α in the equation shown in Fig.4 are closely related to the value N_2/N_5 as shown in Figs.5 and 6. The average relationships in Figs.5 and 6 can be expressed by the empirical equations shown in Figs.5 and 6.

The results calculated by the empirical equations mentioned above are drawn as lines in Fig.4. Figure 4 shows that the equations can be used to calculate ε_{DA} accurately from N_c, N_2 and N_5.

Moreover, the values of the parameters N_2 / N_5 and N_2 are closely related to stress ratio, R, which is applied to a specimen and undrained cyclic strength, R_s, as shown in Figs. 7 and 8. Though the equations in Figs.7 and 8 are derived empirically, the equations can express the average relationships, irrespective of grain size distributions, effective confining pressure and whether or not the samples are intact. In Figs.7 and 8, R and R_s mean as follows :

R : Stress ratio ($\sigma_d / 2\sigma_c'$) applied to a soil specimen in cyclic triaxial test

σ_d : Single amplitude of axial stress

σ_c' : Effective confining pressure

R_s : Undrained cyclic strength, which means stress ratio required to cause ε_{DA}=2% in 20 loading cycles

PROCEDURE FOR CALCULATING SETTLEMENT OF GROUND INDUCED BY EARTHQUAKE

Fig.9 shows the procedure for calculating settlement of gravelly ground induced by earthquake shaking. According to this procedure, it is necessary to evaluate the value of R_s. We have already proposed the following equation to evaluate the value of R_s of gravelly soils by the penetration resistance of the Large Penetration Test (LPT) (Tanaka et al. 1991a ; Tanaka et al. 1992).

$$R_s = 0.15 + 0.0059 \{N_d (\sigma_v' / P_1)^{-1}\}^{1.3} \qquad (3)$$

where, N_d : LPT blow count

σ_v' : Effective overburden pressure

P_1 : Standard pressure (= 98 kPa)

Using Eq.(3), undrained cyclic strength, R_s can be briefly evaluated by LPT, N_d-value without the need for undisturbed sampling.

When the value of the coefficient of earth pressure at rest, K_0, is not 1.0, R_s should be corrected by the following equation.

$$R_s^* = (1+2K_0)/3 \cdot R_s \qquad (4)$$

In Fig.9, R is calculated from the maximum acceleration. In this paper, we use the following equation which proposed by Seed(1979).

$$R = 0.65\, a_{max}(Z)\, \sigma_v / (g / \sigma_v') \qquad (5)$$

where, $a_{max}(Z)$: Maximum acceleration at a depth of Z

σ_v : Overburden pressure in terms of total stress

g : Acceleration of gravity

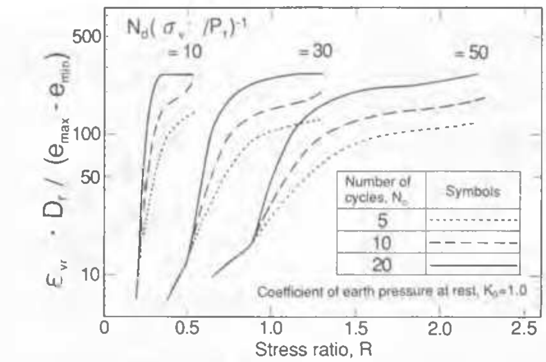

Fig.10 Relationship between $\varepsilon_{vr} \cdot D_r / (e_{max} - e_{min})$ vs. R

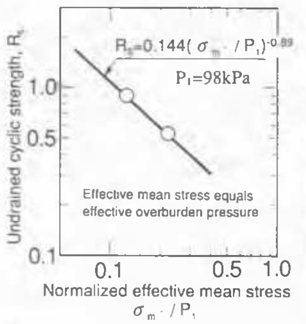

Fig.11 Undrained cyclic strength of dense Tonegawa sand

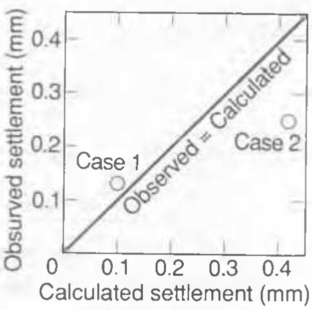

Fig.12 Observed settlement vs. calculated settlement

Using the procedure shown in Fig.9, one can conduct simplified estimation of the settlement of dense gravelly or sandy ground after an earthquake event. For example, Fig.10 shows the relationship between R and $\varepsilon_{DAmax}(e_{max} - e_{min})/D_r$ when K_0=1.

COMPARISON BETWEEN CALCULATED AND MEASURED SETTLEMENTS

The settlement calculated by the above-mentioned method was compared with results of shaking-table tests to investigate the propriety of the proposed method. Observed data of saturated dense sand deposits were obtained by large-scale shaking-table tests, the results of which are available in other literature (Tohma, et al. 1988).

A cylindrical soil container was used for the model ground in the shaking tests. The height of the container was 3.0 m. The shaking table tests were conducted by applying irregular time histories of accelerations, having the time history scaled down to one-fifth of the actual time history of accelerations recorded during El Centro earthquake, to the base of shaking table. The maximum base accelerations of Case1 and Case2 are 84 gal and 206 gal, respectively. The model ground was divided into six soil layers. The residual volumetric strain, ε_{vr} of the individual soil layer calculated using Eq.(2) and the equations in Figs.4, 5, 6,7,8 in which the undrained cyclic strength and maximum acceleration measured at each layer were used. Summing up the decrease of thickness of individual soil layer, total settlement of the model ground was calculated.

Tonegawa sand (mean grain size : 0.34mm, uniformity coefficient : 1.95) of D_r = 86 % was used as the model ground. The undrained cyclic strength, R_s was obtained by conducting undrained cyclic triaxial tests on the undisturbed samples carefully sampled from the model ground after the shaking table tests. Figure 11 shows the relationship between the undrained cyclic strength and effective mean stress, indicating that R_s decreases with an increase in effective mean stress. Therefore, the undrained cyclic strength of each layer were determined using empirical equation shown in Fig.11. Assuming K_0 = 0.4, the corrected undrained cyclic strength, R_s* is calculated by Eq.(4).

Figure 12 shows the comparison between the observed settlements and the calculated settlements, which are common results for N_c= 5, 10 and 20. Though the degree of agreement between the calculated settlements and the observed settlements is not so good, it can be said that the proposed method permits a rough estimate of residual settlements of ground induced by earthquake shaking.

CONCLUSIONS

Thus proposed is a method for evaluating earthquake-induced residual settlement of dense gravelly or sandy ground. It has been shown that the proposed method permits a rough estimate of residual settlements of ground induced by earthquake shaking by comparing the calculated results with the results of large-scale shaking-table tests.

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