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A FUNDAMENTAL STUDY ON SOIL LABORATORY TESTING METHOD FOR NONLINEAR SEISMIC GROUND RESPONSE ANALYSIS

J. IZAWA[(1)] and Y. MURONO[(2)]

[(1)] *Senior Researcher, Soil Dynamics and Earthquake Engineering Laboratory, Center for Railway Earthquake Engineering Research, Railway Technical Research Institute, izawa@rtri.or.jp*

[(2)] *Director, Earthquake Engineering Research, Railway Technical Research Institute, murono@rtri.or.jp*

Abstract

The authors have proposed a new testing method for obtaining appropriate parameters of deformation characteristics of soils necessary for time-domain nonlinear seismic ground response analysis, in which Level 2 earthquake is considered. The proposed method is composed of two types of testing method: a strain-controlled 1 cycle stage shear test and a strain-controlled constant strain cyclic shear test. The latter test can also provide information of soil liquefaction, which can be used for an assessment of soil liquefaction potential based on the theory of the accumulated dissipation energy. Trial tests by the proposed method using Toyoura sand were conducted and applicability of the method was examined.

Keywords: Ground Response Analysis, Deformation Characteristics of soils, Cyclic shear test

1. Introduction

Deformation characteristics of soil used in seismic ground response analysis are very important factors for seismic design of infrastructures, because inertia force and ground displacement acting on the structures are greatly dependent on the results of ground response analysis. Two relationships, i.e. G/G_0-γ and h-γ relationships, are typically used as parameters indicating deformation characteristics of soils, where G is shear stiffness; G_0, initial shear stiffness; h, hysteresis damping; and γ, shear strain. For determining these parameters, cyclic shear tests are conducted using a tri-axial compression test apparatus, a torsion shear test apparatus and so on. As a loading method, “the stage loading testing method” is generally adopted. At one loading stage in this test, a soil specimen is repeatedly sheared 11 times at a particular shear stress level, and G and h are determined as a secant shear stiffness and an area of a 10^{th} loop of shear stress and shear strain relationship, i.e. τ-γ relationship, respectively. After the loading stage, excess pore water pressure in a soil specimen generated during 11 times cyclic shear is dissipated, i.e, a soil specimen is consolidated, and the same loading test is conducted at a next shear stress level. This testing method was developed in order to mainly determine the parameters for small-to-medium shear strain level ($10^{-5}<\gamma<10^{-3}$) in the late 1960s, when the equivalent linear response analysis was a major tool for ground response analysis against a relatively small-scale earthquake. On the contrary, the parameters for large shear strain level is necessary for the current seismic design of structures against a large-scale earthquake, such as a level 2 which is defined as an assumed the maximum seismic motion at construction site of structure in the Japanese Design Standards for Railway Structures -Seismic Design- (Railway Technical Research Institute). Also in the standard, the use of the time-domain nonlinear seismic ground response analysis is recommended for a level 2 earthquake. Some researchers, therefore, pointed out some problems of the conventional stage loading testing method (e,g. Silver and Park).

The authors propose a new testing method to determine the appropriate G/G_0-γ and h-γ relationships necessary for the time-domain nonlinear seismic ground response analysis against a large-scale earthquake. This paper describes the outline of the proposed method, and the results of trial tests and analysis.

2. Outline of the proposed testing method

2.1 The conventional testing method

As mentioned above, G/G_0-γ and h-γ relationships are determined from a 10th loop of τ-γ relationship in 11 times cycle loading at a constant shear stress under undrained condition in the conventional stage loading method. This loading stage is repeated at various shear stress levels after a soil specimen is consolidated at each loading stage. The major problems of the conventional method are summarized as follows:

- Shear stiffness, G, and hysteresis damping, h, vary during 11 times cycles due to excess pore water pressure, especially at large shear strain levels.
- Density of a soil specimen is different at each loading stage due to consolidation.
- It is difficult to control the target strain level because the test is conducted under stress controlled, in which shear strain may rapidly increase during 11 times cyclic loading especially at large shear strain level.
- Tri-axial test apparatus cannot give an accurate loop of τ-γ relationship because the apparatus cannot simulate pure shear deformation of a soil specimen.

The above problems were not important for a seismic design against a relatively small-scale earthquake because shear strain level of the surface ground was relatively small. Recently, they have become obviously important in the current seismic design in which a level 2 earthquake is considered.

2.2 The proposed method

Deformation characteristics of soils depend on shear strain, γ and pore water pressure, u as mathematically indicated in the equation (1).

$$dG = \frac{\partial G}{\partial \gamma} d\gamma + \frac{\partial G}{\partial u} du \tag{1}$$

Since both of them have non-linearity respectively and it is impossible to consider their effects on deformation characteristics separately, an effective stress analysis must be conducted in order to strictly calculate ground response against an earthquake. It is, however, very cumbersome to make an effective stress analysis in practice. On the other hand, accuracy of a total stress analysis is acceptable for seismic design of infrastructures. The aim of the proposed method, therefore, is to determine the deformation characteristics necessary for the time-domain nonlinear seismic ground response analysis based on the total stress analysis. Furthermore, soil liquefaction of the surface layer is also very important phenomena for seismic stability of the structures. The proposed method therefore makes an assessment of soil liquefaction potential possible. Firstly, the proposed method aims to determine the deformation characteristics dependent only on strain level, in which effect of excess pore water pressure is eliminated as much as possible. The G/G_0-γ and h-γ curve relationships dependent only on strain level is defined as “Master curves”. Secondly, effect of excess pore water pressure is considered later as necessary. A conceptual scheme of the proposed method is shown in Fig.1.The method is composed of two different test series: the strain controlled 1 cycle stage shear test and a cyclic shear test under constant strain. These tests are basically conducted with the torsion shear test apparatus or the simple shear test apparatus in order to simulate pure shear deformation. Details of the respective tests are as follows.

(1) Strain controlled 1 cycle stage shear test (1ST)

In the strain controlled 1 cycle stage shear test(1ST), 1 cyclic shear test is repeatedly conducted under the controlled strain while gradually increasing strain level at each loading stage without consolidation after each loading stage. The purpose of doing this test is to determine a wide strain range of G/G_0-γ and h-γ relationships eliminatory the effect of pore water pressure as much as possible. In addition, only conducting the monotonic loading test may be sufficient for small strain level in some situation, where hysteresis damping is negligible.

(2) Cyclic shear test under constant strain (CST)

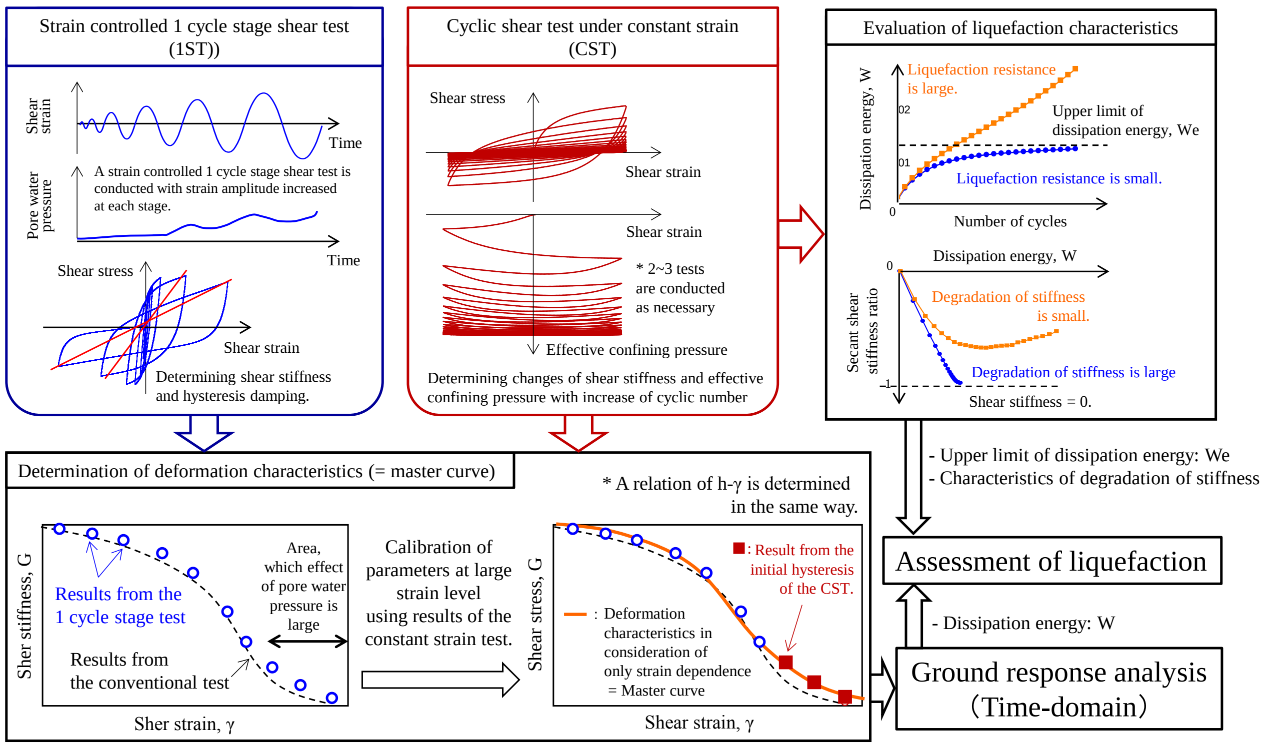


Fig. 1 Concept of the proposed testing method.

The 1ST may give a wide strain range of G/G_0-γ and h-γ relationships without effect of pore water pressure, i.e. master curves, to some extent. Effect of excess pore water pressure, however, would be large for large strain level. To obtain the more accurate master curves for large strain level, a few cyclic shear tests under constant strain (CST) are conducted using another soil specimen, and G and h are determined from the initial loop of τ-γ relationship. By calibrating G/G_0-γ and h-γ relationships at large strain level with the results of CST, the master curve is determined. Additionally, change in G/G_0-γ and h-γ relationships only due to excess pore water pressure at a particular shear strain level can be obtained from the CST. This information can be effectively used for evaluating the effect of pore water pressure. Furthermore, accumulated dissipation energy, *W*, can be calculated by the equation (2).

$$W = \int \tau(\gamma) d\gamma \tag{2}$$

Assessment of soil liquefaction potential based on the theory of accumulated dissipation energy (Kazama et. al) can be done using *W* and degradation of shear stiffness of liquefiable layer.

3. Trial tests

3.1 Outline of the test

Table 1 Test conditions of Toyoura sand in each shear test.

(a) Dr=60%

	Test type	1 cycle stage test	Constant strain test Strain amplitude 0.1%	 0.4%	 1.0%	Conventional test Method	Monotoni lodaing CUB
Before loading	Dry density (g/cm^3)	1.517	1.517	1.506	1.504	1.509	1.509
	Void ratio, e	0.749	0.749	0.762	0.764	0.759	0.758
	Relative density (%)	65.4	65.4	62.2	61.6	62.9	63.1
After loading	Dry density (g/cm^3)	1.512	1.512	1.510	1.509	1.573	—
	Void ratio, e	0.755	0.755	0.757	0.758	0.686	—
	Relative density (%)	64.0	64.0	63.4	63.2	80.9	—

※ Values of the conventional test is values after the final loading stage.

(b) Dr=80%

	Test type	1 cycle stage test	Constant strain test Strain amplitude 0.1%	 0.4%	 1.0%	Conventional test method	Monotoni lodaing CUB
Before loading	Dry density (g/cm^3)	1.560	1.567	1.559	1.559	1.563	1.560
	Void ratio, e	0.701	0.693	0.702	0.702	0.697	0.701
	Relative density (%)	77.3	79.2	77.0	77.0	78.1	77.3
After loading	Dry density (g/cm^3)	1.560	1.566	1.568	1.570	1.605	—
	Void ratio, e	0.701	0.694	0.691	0.690	0.653	—
	Relative density (%)	77.2	78.9	79.6	79.9	89.0	—

In order to verify the applicability of the proposed test method, a series of trial tests were conducted using Toyoura sand(G_s=2.645, D_{50}=0.190mm, e_{max}=0.973, e_{min}=0.609, U_c=0.682) for two cases of relative density, namely 60% and 80% respectively. The conventional stage loading tests and the monotonic loading tests were also conducted for comparison. The torsion shear test apparatus was used for all of the tests. Confining pressure was 100kPa in isotropic condition (back pressure=200kPa), and the size of the soil specimen was 70mm in the outer diameter, 30mm in the inner diameter and 70mm in the height. In the 1STs, shear strain time histories with triangle waves were applied to the specimens at the strain velocity of 0.1%/min. In the CSTs, constant strain of 0.1%, 0.4% and 2.0% were applied to the specimens at the strain velocity of 0.1%/min. The conventional stage loading tests used shear stress time history of sin wave shape with 10Hz. The monotonic loading tests were conducted at the strain velocity of 0.1%/min. All of the tests were conducted under undrained condition.

3.2 Test results

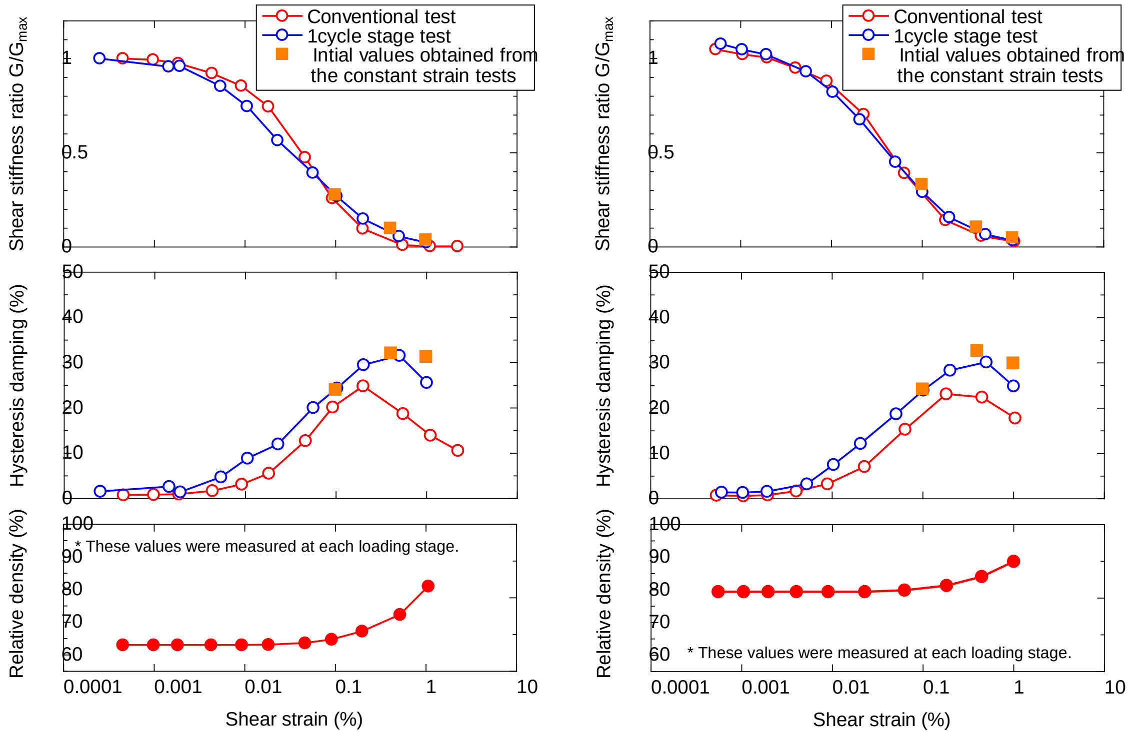
(1) Test conditions

Densities of the soil specimens before and after loadings at each test are indicated in Table 1. All the specimens before loading were prepared so as to have almost the same relative density, whose errors were limited within 5%. On the contrary, relative density of the conventional tests after loading increased as compared with those before loading: relative densities in the cases of Dr=60% and 80% changed from 62.9% to 80.9, and from 78.1% to 89%, respectively. This result clearly shows that the soil specimens with adequate density may not be used and accurate deformation characteristics may not be obtained in the conventional stage tests especially at large strain level.

(2) Deformation characteristics

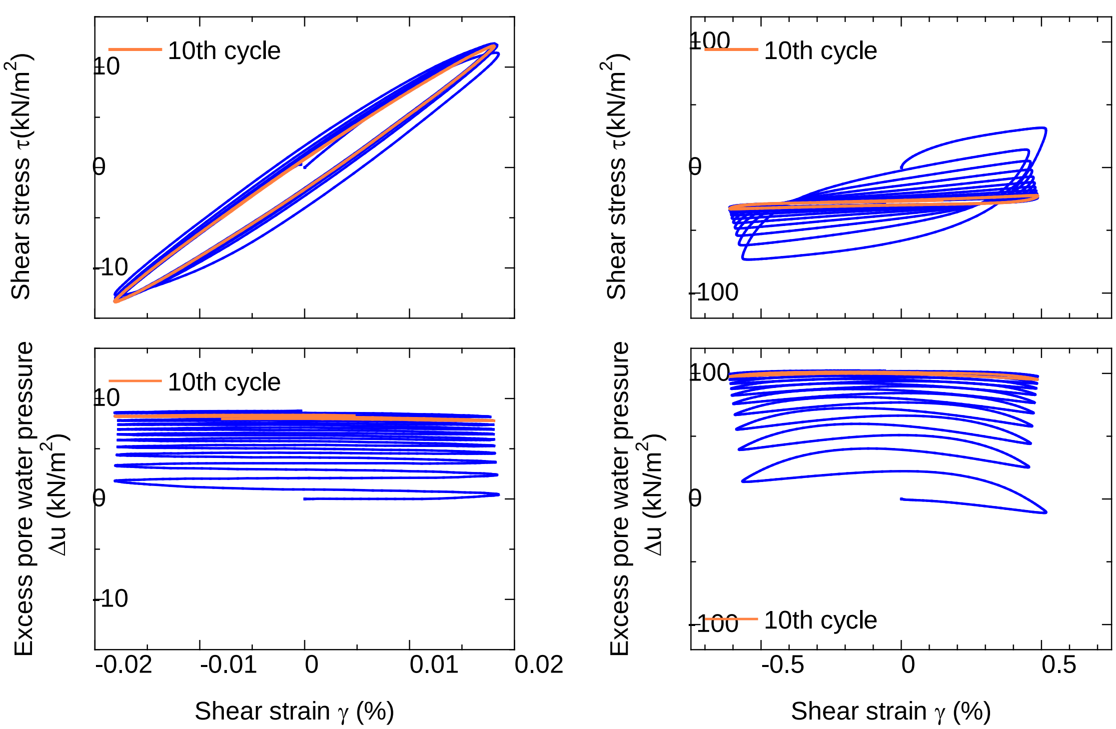
Fig. 2 shows G/G_0-γ and h-γ relationships obtained from the proposed tests and the conventional stage tests for two cases of Toyoura sands with relative density of 60% and 80%. The G/G_0 and h of the initial loop of τ-γ relationship obtained from CSTs were almost the same with those obtained from 1CTs. This means that the CST can give the G/G_0-γ and h-γ curve relationships without effect of excess pore water pressure, i.e. "Master curves". On the other hand, the G/G_0-γ relationship of Toyoura sand with Dr=60% obtained from the conventional stage test was different from the master curve: larger at the medium strain level and smaller at the large strain level than the master curve. In the case of Dr=80%, the difference of them are relatively small. According to the results, problems of the conventional test method seem to be significant for loose sand layer. Regarding the hysteresis damping, the conventional stage tests gave small damping as compared to those of the master curve, regardless of density of the soil specimen.

Fig. 3(a) and (b) show relationships between the shear stress and the shear strain at the strain level of 0.017% and 0.20%. At the medium strain level (0.017%), change in shear stiffness during cyclic loading due to excess pore water pressure was relatively small, but the area of hysteresis tended to gradually decrease with increase of



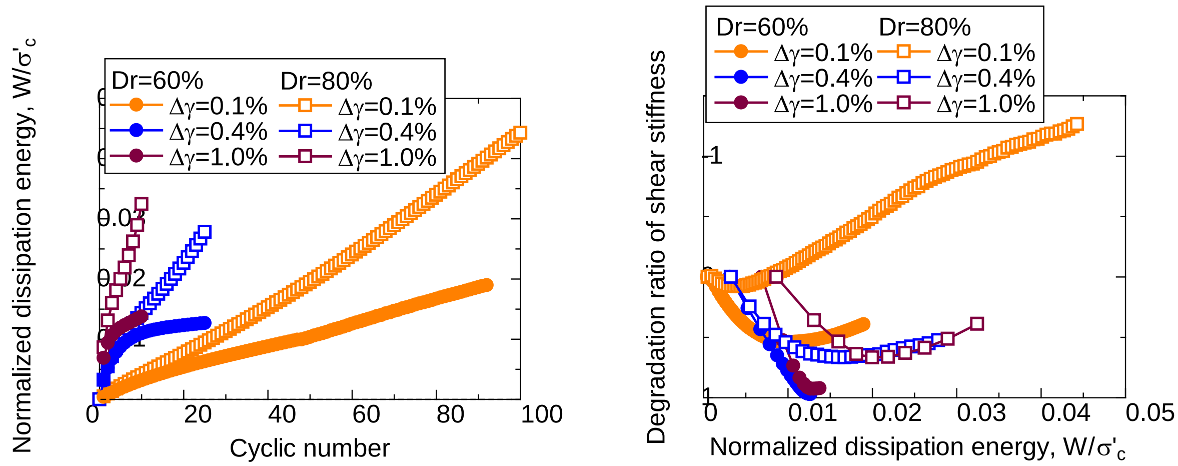
(a) Dr=60% (b) Dr=80%

Fig. 2 Comparison of deformation characteristics obtained from the convsentional test and those from the proposed tests.



(a) Shear stress ratio = 11kPa (b) Shear strain amplitude = 0.2%

Fig. 3 Shear stress – shear strain relationship of Toyoura sand with Dr=60% obtained from the conventional cyclic loading test



(a) W/σ’$_c$ ~ Cyclic number relationship (b) Degradation ratio ~ W/σ’$_c$ relationship

Fig. 4 Results obtained from constant strain cyclic loading tests

the cyclic number. On the other hand, at the large strain level (0.20%), shear stiffness greatly decreased with increase of the cyclic number due to excess pore water pressure. This result clearly shows that the conventional stage test, in which cyclic shears are applied 11 times, cannot give accurate deformation characteristics of soils.

(3) Liquefaction characteristics

Fig. 4(a) shows the relationships between the normalized accumulated dissipation energy, W/σ’$_c$, and the cyclic number, obtained from the cyclic shear tests under constant strain, where σ’$_c$ is confining pressure in the tests. The results of Toyoura sand with Dr=60% at γ=0.4% and 1.0% showed the clear upper limit at around W/σ’$_c$=0.01, which means that soil liquefaction may occur if the dissipation energy accumulated in the soil layer reaches to W/σ’$_c$=0.01 approximately. On the other hand, the upper limit was not observed for the case of γ=0.1%. It might be inferred from the results that soil liquefaction may not occur even if the accumulate dissipation energy reaches to 0.01 against a small-scale earthquake, for which strain level of the surface ground may be small. Similarly, Toyoura sand with Dr=80% did not show any upper limits at all the strain levels, which means that possibility of soil liquefaction is very low. This trend is corresponding to the past experiences. Fig. 4(b) shows the relationships between the degradation ratio of shear stiffness and the normalized accumulated dissipation energy. This shows that Toyoura sand with Dr=60% may lose its stiffness due to liquefaction. On the other hand, Toyoura sand with Dr=80% can maintain approximately 30% of its shear stiffness even if a large number of shear cycles may be applied during an earthquake. In this way, the CST can provide us with very valuable information on soil liquefaction, and may make more accurate assessment of soil liquefaction possible.

4. Trial analysis

4.1 Outline of the analysis

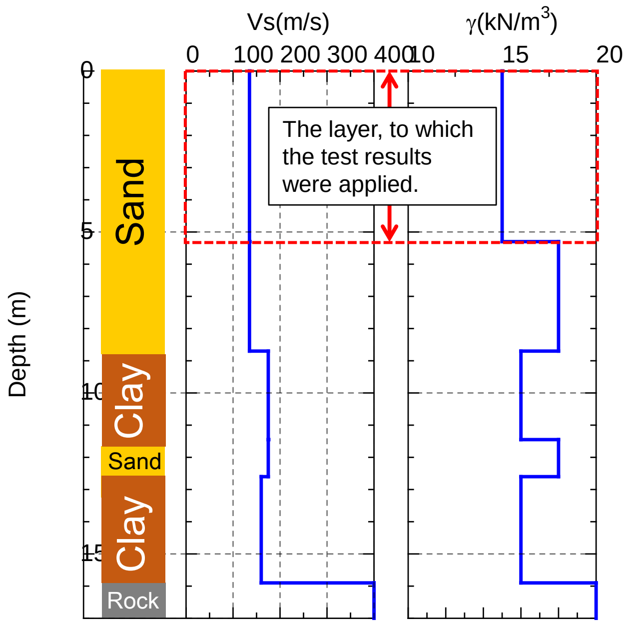


Fig. 5 Model surface ground

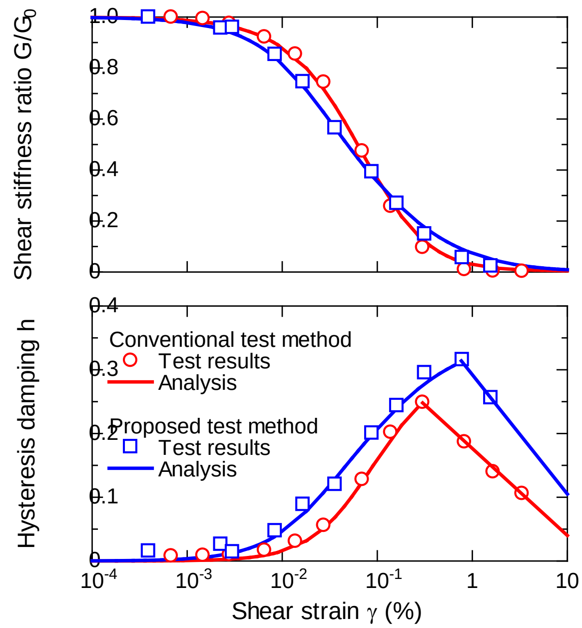


Fig. 6 Deformation characteristics used in the analyses

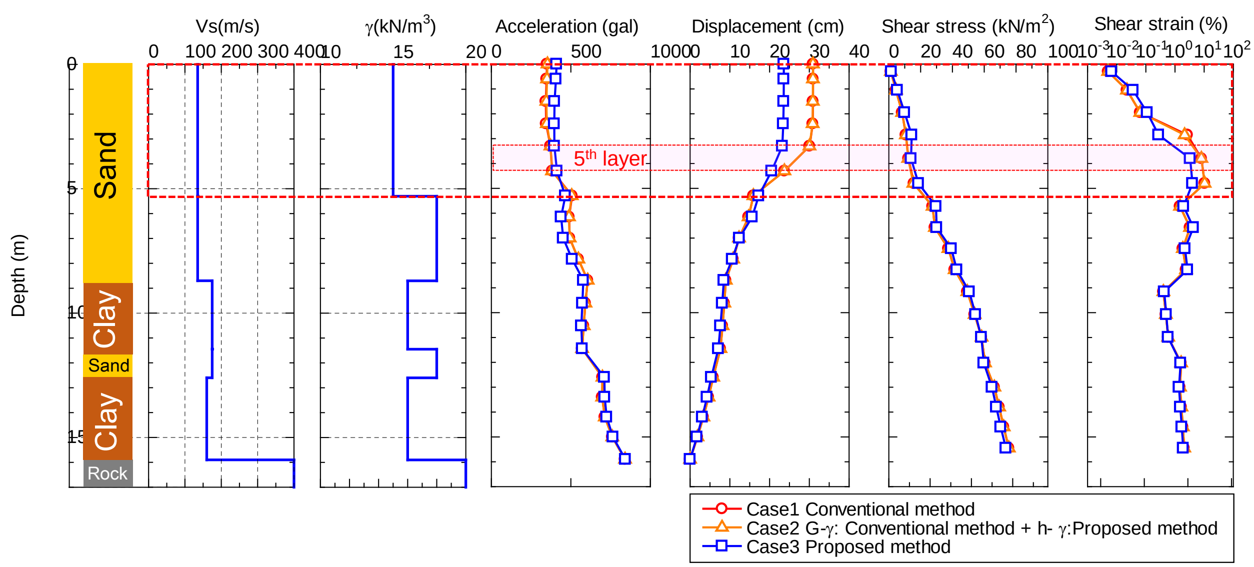


Fig. 7 Distribution of the maximum response values obtained from ground response analyses

In order to clarify the effect of difference of the testing method on the results of ground response analysis, trial analyses were conducted using the results of the trial tests described in the previous chapter. The model ground used in the trial analysis is shown in Fig. 5. Nonlinear deformation characteristics of the soils were modeled by the GHE-S model (Murono et. al.), which is generally used in the seismic design of Japanese railway structures. The test results were applied only to the upper layer as shown in Fig 5. Parameters for GHE-S model were determined so that G/G_0-γ and h-γ relationships modeled as the GHE-S model correspond to those of the test results as shown in Fig. 6. For the other layers, the standard parameters for the GHE-S model (Nogami et. al.) were applied. Trial analyses were conducted for the three cases: Case 1 which used G/G_0-γ and h-γ curves of the conventional stage test, Case 2 which replace only the h-γ curve with that obtained from the proposed method, and Case 3 which used the results of the proposed method. The level 2 spectrum II earthquake used for the seismic design of Japanese railway structures was applied to all of the models.

4.2 Analytical results

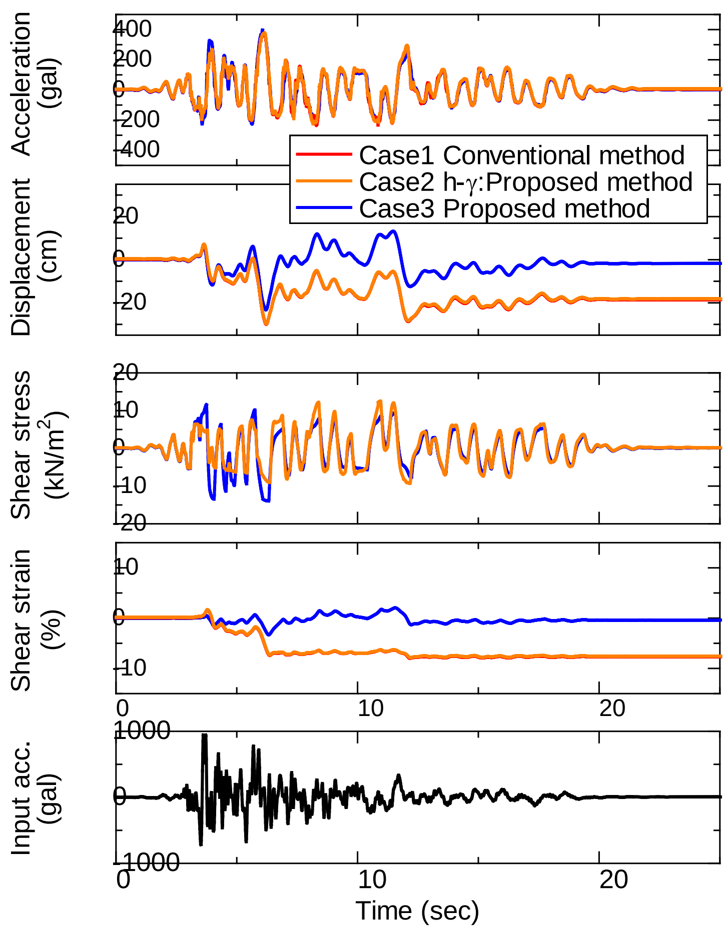


Fig. 8 Time histories of response of 5th layer

(1) Ground response analysis

Fig. 7 shows the distributions of the maximum response values obtained from the ground response analyses. The layer, in which test results were used, showed large shear strain with over 1.0%. The results of Case 1 and Case 2 are almost the same, which means that such difference in hysteresis damping as shown in Fig. 6 seems to have a small effect on seismic behavior of the surface ground against a large-scale earthquake. On the other hand, the result of Case 3 showed smaller displacement and larger response acceleration than those of Case 1 and Case 2 at the upper layer because the layer had relatively large shear stiffness as compared with those of Case 1 and Case 2. Fig.8 shows the time histories of response acceleration, displacement, shear stress and shear strain of the 5th layer indicated in Fig. 7. Shear stresses of Case1 and 2 at the 5th layer show the clear upper limit after around 3.25 seconds, which means that the layer cannot transfer large shear stress, i.e., large acceleration due to small shear stiffness. The differences in the shear stiffness between Case 1&2 and Case 3 seem to be small at large strain level as indicated in Fig. 6. The stiffness at large strain of Case 3 was, however, almost twice as large as that of Case 1&2, for example, G/G_0 = 0.068 for Case 1&2 and G/G_0 = 0.124 for Case 3at shear strain of 0.5%. Differences of the response acceleration spectrums at the ground surface were also large as shown in Fig. 9. These results clearly indicated that determination of shear stiffness at large strain level is most important for evaluating the seismic ground response against large-scale earthquakes. For determining accurate deformation characteristics, a testing method should be carefully established.

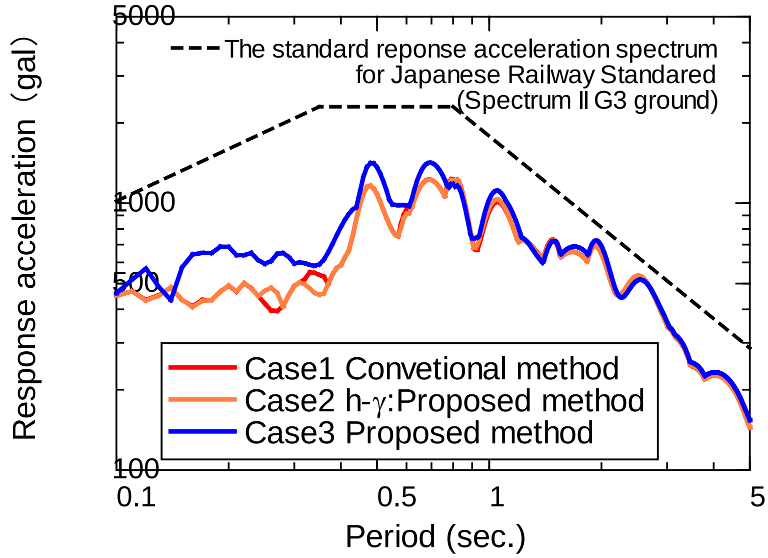


Fig. 9 Comparison of acceleration response spectrums at the ground surface

Table 2 Comparison of assessment of liquefaction

Depth of Lower surface (m)	Effective vertical stress (kPa)	Max. shear stress (kPa)	Max. shear strain (%)	Proposed method (Energy method)		Conventional method (FL method)		
				W/σ'_c	Fs(=We/W)	R	L	F_L
0.6	1.50	1.59	0.0067	0.000128	99.2	0.0900	1.06	**0.0849**
1.5	5.25	5.55	0.0376	0.000939	13.5	0.0900	1.06	**0.0852**
2.4	9.75	10.2	0.114	0.00310	4.09	0.0900	1.05	**0.0860**
3.3	14.3	14.7	0.275	0.00809	1.57	0.0900	1.03	**0.0872**
4.3	20.5	14.3	3.42	0.118	**0.107**	0.0890	0.695	**0.128**
5.3	28.5	19.0	4.08	0.137	**0.0924**	0.0890	0.668	**0.133**

(2) Assessment of soil liquefaction

An assessment of soil liquefaction potential for the model ground of Toyoura sand with Dr=60% was conducted by comparing the accumulated dissipation energy, *W*, obtained from the ground response analysis and the limit accumulated dissipation energy, W_e, obtained from the CSTs. The limit dissipation energy was set as W/σ'_c=0.0127, which was determined as the results of the CST at shear strain of 0.4% and 1.0%. The assessment using the conventional FL method was also conducted for comparison. Table 2 shows the results of the assessments of soil liquefaction potential. The lower 2 layers showed almost the same factors of safety against soil liquefaction regardless of the assessment methods. On the other hand, the proposed method gave very low liquefaction potential as compared to that obtained by the conventional FL method for the upper 5 layers. Unfortunately, there is no telling which result is correct at present. If the reliability of the proposed method is verified, more reasonable assessment of soil liquefaction potential can be made. In the future, we'd like to enhance the reliability of the proposed method by comparing the results of model test, hybrid ground response test and investigations with the analytical results.

5. Conclusions

The authors have proposed a new testing method to determine the appropriate G/G_0-γ and h-γ relationships necessary for the time-domain nonlinear seismic ground response analysis against a large-scale earthquake. Trial tests and analyses were conducted to verify the applicability of the proposed method. As a result, the following knowledges were obtained.

1. The proposed testing method, which is composed of the strain controlled 1 cycle stage shear test (1ST) and the cyclic shear test under constant strain (CST), can provide the master curves, which can represent G/G_0-γ and h-γ curve excluding effect of pore water pressure for large strain level as much as possible. Additionally, the CST can give valuable information on soil liquefaction based on the theory of accumulated dissipation energy.
2. The conventional stage test tends to give smaller shear stiffness at large strain lever and smaller hysteresis damping at wide strain length as compared to the master curves which can be obtained by the proposed method, because shear stiffness greatly decreases with increase of cyclic number due to excess pore water pressure during 11 times cyclic shearing. In addition, soil specimens with adequate density may not be used and accurate deformation characteristics may not be obtained especially at large strain level due to consolidation after each loading stage.
3. Determination of shear stiffness at large strain level is very important for evaluating the seismic ground response against a large-scale earthquake. For determining accurate deformation characteristics, a testing method should be carefully established.

Assessment of soil liquefaction based on the theory of accumulated dissipation energy can be made by conducting the CST, and may give more reasonable liquefaction potential. It is necessary to verify the reliability of the method in the future.

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