ABSTRACT

Based on a series of shaking table model tests, it was found that the effects of 1) subsoil and backfill deformation, 2) failure plane formation in backfill, and 3) pullout resistance mobilized by the reinforcements on the seismic behaviors of the geosynthetic reinforced soil retaining walls (GRS walls) were significant. These effects cannot be taken into account in the conventional pseudo-static based limit equilibrium analyses or Newmark’s rigid sliding block analogy, which are usually adopted as the seismic design procedure. Therefore, this study attempts to develop a simplified procedure to evaluate earthquake-induced residual displacement of GRS walls by reflecting the knowledge on the seismic behaviors of GRS walls obtained from the shaking table model tests.

In the proposed method, 1) the deformation characteristics of subsoil and backfill are modeled based on the model test results and 2) the effect of failure plane formation is considered by using residual soil strength after the failure plane formation while the peak soil strength is used before the failure plane formation, and 3) the effect of the pullout resistance mobilized by the reinforcement is also introduced by evaluating the pullout resistance based on the results from the pullout tests of the reinforcements. By using the proposed method, simulations were performed on the shaking table model test results conducted under a wide variety of testing conditions and good agreements between the calculated and measured displacements were observed.

Key words: failure plane, geosynthetic retaining wall, Newmark’s method, residual displacement, shaking table test (IGC: E12/E14/E8)

INTRODUCTION

Case history of recent large earthquake revealed good seismic performance of the geosynthetic-reinforced soil retaining walls (GRS walls) with full-height rigid facing (e.g., Tatsuoka et al., 1998; Koseki et al., 2006). For example, after the 1995 Hyogo ken Nanbu-Earthquake, the GRS wall constructed to support the railway embankment at Tanata site could be used after minor retrofitting works although many conventional concrete retaining walls (e.g., gravity, cantilever and leaning type retaining walls) needed reconstruction works due to their severe damage. However, these different seismic performances cannot be properly evaluated based on the conventional pseudo-static limit-equilibrium approaches.

On the other hand, there is a trend in Japan to shift the design procedure to performance-based design from conventional pseudo-static limit-equilibrium approaches (Koseki et al., 2007) because it is difficult to evaluate ductile seismic performance of the GRS walls based on the factor of safety obtained from the pseudo-static limit equilibrium design approaches, which are not directly related to the residual state of the retaining walls after the earthquake.

In the performance-based design, the seismic performance of the retaining walls can be typically evaluated by comparing the residual displacements with allowable values. Therefore, the performance-based design approach is more suitable for considering the different seismic performances between the GRS walls and conventional retaining walls, while more detailed investigations on the seismic behavior of the GRS walls are required so as to evaluate their seismic performance accurately.

In relation to the above, many experimental and analytical studies on seismic behavior of the GRS walls have been carried out so as to investigate into the mechanisms of their ductile seismic behaviors.

Watanabe et al. (2003) have revealed through a series of shaking table model tests that mobilized tensile resistance of the reinforcement which increased with the accumulation of the wall displacements could resist effectively against the seismic thrusts (i.e., the inertia force...
and dynamic earth pressure). They also found that the rapid increase in the wall displacement of the conventional type retaining wall was triggered by local bearing capacity failure of the subsoil beneath the toe of the retaining wall. It should be noted that increase of the wall displacements following the bearing capacity failure was not observed with the GRS walls.

Experimental studies on the GRS walls with segmental wall facing panels have also been carried out (e.g., Bathurst et al., 2002; Matsuo et al., 1998; Ling et al., 2005). The effects of the facing inclination and shear resistance of the facing interface (i.e., strong connection of each segmental panel) on the seismic performance of GRS walls were highlighted through their experiments.

The effects of the material properties (e.g., pullout resistance and rupture strength of the reinforcements) of the geosynthetics reinforcements on the seismic performance and failure mode of the GRS walls have also been investigated (e.g., Izawa et al., 2002; Nakajima et al., 2008a). Izawa et al. (2002) found out through the series of the centrifuge model tests on the GRS walls subjected to the sinusoidal excitation that the failure plane would not form in the backfill in which the reinforcements were installed (reinforced backfill) in the case of the GRS walls with rigid reinforcements while the failure plane could be formed within the reinforced backfill in the case of soft reinforcements. In addition to the deformation mode, they pointed out that the pullout resistance of the reinforcements should be appropriately taken into account in developing the displacement evaluation method. Similar considerations were also discussed by Nakajima et al. (2008a) based on the series of shaking table model tests which were conducted under the gravitational field using the irregular excitation.

Based on the knowledge obtained from the above experimental observations, studies to develop a displacement evaluation procedure for GRS walls were also carried out.

Horii et al. (1998) proposed a displacement evaluation method based on the Newmark's sliding block approach (Newmark, 1965), while the effect of the shear deformation of the reinforced backfill was also taken into account. This approach has been adopted as a procedure to evaluate the earthquake-induced residual displacement under large seismic load in the current railway design guideline in Japan (RTRI, 2007).

Because the concept of Newmark's method (NM method) is simple, there have been many studies to adopt the NM method for the evaluation of the earthquake-induced residual displacement of GRS walls (Matsuo et al., 1998; Cai and Bathurst, 1996; Huang and Wang, 2005). However, it should be noted that the sliding or rotating mass and its subsoil are assumed to be rigid in the NM method. In addition, the soil strength mobilized in the subsoil and backfill layers is assumed to be constant in the above analyses. In actual cases, on the other hand, the subsoil layer would deform during an earthquake and the strength mobilized along a failure plane in the backfill layer would exhibit strain-softening behavior from peak to residual strength.

The other approach which has also been attempted is to apply the finite element (FE) method to the displacement prediction for the GRS walls (e.g., Fujii et al., 2006). This approach will be more suitable for simulating the seismic behavior of the GRS walls especially at relatively small displacement levels, while the reinforced soil behaves as a continuum although the modeling of the geosynthetic-reinforcements will become complicated. However, there seems to be a limitation of the FE approaches for the displacement prediction of the GRS walls because the FE analyses considering the effects strain localization in the backfill layer which would result into the failure plane formation and strain softening behavior have not yet been fully developed although seismic behavior of the GRS walls are affected by these phenomena. As far as the authors know, there does not exist any procedure to evaluate earthquake-induced displacement which considers the deformation characteristics of the GRS walls.

In view of the above, Koseki et al. (2004) proposed a simplified displacement evaluation method for the GRS walls by combining the NM method with considering the effect of the shear deformation of reinforced backfill and subsoil layer while the effect of the strain softening behavior with the failure plane formation in the backfill layer that is under dense condition in general was also taken into account. However, the shear deformation characteristics of the reinforced backfill have not yet been investigated in detail.

This study, therefore, attempts to develop a simplified displacement evaluation method for the GRS walls by considering their seismic behaviors observed in the shaking table model tests and examine its applicability.

PROCEDURE AND GENERAL RESULTS OF SHAKING TABLE MODEL TESTS

Model Test Procedure

The model tests were conducted using a shaking table at Railway Technical Research Institute in Japan. The size of the soil container was 260 cm long, 60 cm wide and 140 cm high. The cross sections of the GRS wall models with three different reinforcement arrangements used in the series of shaking table model tests are shown in Fig. 1.

Two types of geogrid reinforcement models were used in the model tests. As shown in Fig. 2, a geogrid model made of lattice-shaped phosphor bronze strips with a thickness of 0.1 mm and a width of 3 mm for each strip was used. Particles of Toyoura sand were glued on the surface of the reinforcements so as to mobilize the frictional resistances effectively. For the other type of geogrid model, a polyester mesh sheet having a thickness of 0.6 mm and a mesh size of 3 mm was used. The former and latter geogrid models will be called hereafter as PBI and PE, respectively.

After the facing of model retaining wall was placed on a subsoil layer, a backfill layer was prepared. The geogrid
reinforcement models were installed at a vertical distance of 50 mm and they were fixed with the wall facing by soldering (for reinforcement PB1) or using screws (for reinforcement PE) as schematically illustrated in Fig. 2. Above the backfill layer, a surcharge of 1 kPa was applied by placing lead shots. It simulated dead load that is considered in the design for Japanese railway embankment (RTRI, 2007), while the value was reduced at one-tenth so as to reflect the geometric similarity between the models and our target prototype scale.

Both the subsoil and backfill layers consisted of air-dried Toyoura sand \( (D_{50} = 0.23 \text{ mm}, \ G_s = 2.648, \ e_{\text{max}} = 0.977, \text{ and } e_{\text{min}} = 0.609) \) which was prepared by air pluviation using a sand hopper while the target relative density was set equal to 90%. This dense model ground corresponded to well-compacted sand or gravel used for actual construction of railway or road embankments that were supported by the retaining walls.

It shall be emphasized that the backfill in the actual field is not the air dried conditions but unsaturated conditions. Although it is out of the present study, there are several aspects to be considered in discussing differences between the dynamic earth pressure mobilized by the air-dried and unsaturated backfill. First, the effect of suction which will be mobilized around the soil particles in the unsaturated backfill will reduce the dynamic earth pressure. Second, larger unit weight of the unsaturated soil will increase the dynamic earth pressure a little. It should be highlighted that the well drainage system is highly important for the backfill soil in practice so as not to be in the fully saturated condition, which will otherwise cause a drastic increase of the soil pressure.

The typical cross section of the model wall and ground is shown in Fig. 3 where the model of gravity type retaining wall is also indicated so as to highlight different failure patterns of the GRS wall model and gravity type retaining wall in the later part of this paper. The model test conditions that will be referred to in this study are summarized in Table 1. Case names in Table 1 will be used to refer to each of the shaking table model tests hereafter.

Seismic loads were applied to the models by shaking the soil container horizontally with sinusoidal or irregular excitations while the maximum base acceleration was gradually increased at increments of about 50 gals (sinusoidal excitations) or 100 gals (irregular excitations). A typical time history of the irregular excitations is
Fig. 3. Typical cross section of model retaining wall and ground (unit in cm)

Table 1. Summary of test conditions

<table>
<thead>
<tr>
<th>Test name</th>
<th>Type of wall (Fig. 1)</th>
<th>Reinforcement (Fig. 2)</th>
<th>Subsoil thickness</th>
<th>Seismic loading</th>
<th>Shaking conditions (Step/Large)</th>
<th>References</th>
</tr>
</thead>
<tbody>
<tr>
<td>HR1</td>
<td>Type 1</td>
<td>PB1</td>
<td>20 cm</td>
<td>Irregular</td>
<td>Step shaking</td>
<td>Watanabe et al. (2003)</td>
</tr>
<tr>
<td>HR2</td>
<td>Type 2</td>
<td>PB1</td>
<td>20 cm</td>
<td>Irregular</td>
<td>Step shaking</td>
<td>Watanabe et al. (2003)</td>
</tr>
<tr>
<td>HR2_50</td>
<td>Type 2</td>
<td>PB1</td>
<td>5 cm</td>
<td>Sinusoidal</td>
<td>Step shaking</td>
<td>Koseki et al. (2004)</td>
</tr>
<tr>
<td>HR3</td>
<td>Type 3</td>
<td>PB1</td>
<td>20 cm</td>
<td>Irregular</td>
<td>Step shaking</td>
<td>Watanabe et al. (2003)</td>
</tr>
<tr>
<td>HR2_PB</td>
<td>Type 2</td>
<td>PB1</td>
<td>20 cm</td>
<td>Irregular</td>
<td>Large shaking</td>
<td>Nakajima et al. (2007)</td>
</tr>
<tr>
<td>HR2_PE</td>
<td>Type 2</td>
<td>PE</td>
<td>20 cm</td>
<td>Irregular</td>
<td>Large shaking</td>
<td>Nakajima et al. (2007)</td>
</tr>
</tbody>
</table>

Fig. 4. Typical time history of irregular shaking

shown in Fig. 4. The predominant frequency of the irregular excitations was about 5 Hz, and 20 cycles of loadings at a frequency of 5 Hz were applied in the case of sinusoidal excitations. In the tests HR2_PB and HR2_PE, the shaking patterns were changed from the other tests so as to simulate the actual large earthquake. In these tests, an irregular shaking with a maximum acceleration of about 900 gals was applied from the beginning. By starting from the shaking with the large amplitude, seismic behaviors without the effects of shaking history can be evaluated from the test results. In the second and third shaking for tests HR2_PB and HR2_PE, irregular shaking with the maximum amplitude of 1200 and 1500 gals were also applied so as to investigate into the seismic behaviors of the GRS walls under the extremely large earthquake loadings. Finally, the shaking with the maximum amplitude of the 1500 gals was applied again in the forth shaking for these tests. Detailed test procedures are described in Watanabe et al. (2003) and Nakajima et al. (2009b).

Model Test Result

Residual displacements after each shaking step are plotted versus the maximum base acceleration in Figs. 5(a) and (b), while the horizontal sliding displacement \( d \) measured at a height of 40 mm from the base of the retaining wall model and tilting angle \( \theta \) of the wall facing are noted as the residual displacements in this study. In these figures, the model test results on the gravity type retaining wall (test HGir) are also plotted for reference, while the timings of the full formation of failure plane (i.e., the failure plane reached the surface of the backfill layer) are also indicated by vertical arrows. The timings of the failure plane formation in the other tests will be shown in Fig. 14. These displacements shown in Figs. 5(a) and (b) were evaluated based on the measurements with displacement transducers installed at the top and bottom of the wall facing.

As clearly shown in the figures, GRS wall models showed higher seismic performance than the gravity type retaining walls. Between the tests on the gravity type retaining wall and GRS walls, different deformation modes of the subsoil and backfill were also observed as shown in Figs. 6(a) and (b). Location of the failure plane indicated in Fig. 6(a) was evaluated based on the investigation on the deformation of colored sand layers from the apparent glasses.

Different patterns of failure plane formation on tests HR1, HR2 and HR3 are discussed by Watanabe et al. (2003). As for the tests HR2_PB and HR2_PE, failure
planes developed in the same manner with the test HR2 and HR2_50 (Nakajima et al., 2007). In these tests with the same reinforcement arrangements, two or three failure planes were observed. One failure plane appeared from the bottom of the reinforced backfill while the development was stopped by the middle reinforcement (45 cm in length). The second one, which also developed from the bottom of the backfill, was restricted by the uppermost reinforcement. The last one was the failure plane which reached the ground surface by passing just outside the uppermost reinforcement, while the second one was difficult to be visible as indicated in Fig. 6(a).

As also observed in Fig. 6(a), the reinforced backfill, where the reinforcement having a length of 20 cm was installed, was subjected to shear deformation, which would induce the tilting of the wall facing. On the other hand, the tilting of the gravity type retaining wall was induced by the subsoil deformation which was possibly caused by the overturning moment acting on the subsoil (Nakajima et al., 2009b).

The effect of the failure plane formation in the backfill layer was also significant. As clearly shown in Figs. 5(a) and (b), the increment of the wall displacement increased significantly after the failure plane formation with both the gravity type and GRS walls. In the tests on the gravity type retaining wall, the failure plane formed suddenly. On the other hand, certain amounts of the displacement accumulation were needed for the full formation of the failure plane in the tests on the GRS walls.

The effect of different geogrid reinforcement arrangements on the seismic performance of the GRS walls was also observed in tests HR1, HR2 and HR3. In terms of the accumulation of the sliding displacement as shown in Fig. 5(a), tests HR1 and HR2 showed almost similar tendencies while the accumulation of the sliding displacement in test HR3 was less than those in tests HR1 and HR2. On the other hand, test HR2 showed the highest seismic performance especially in terms of the tilting mode of deformation among the other tests. This behavior indicates that, in test HR2, tilting of the wall facing could be restricted effectively by the pullout resistance which was mobilized by the uppermost reinforcement when the wall moved outward.

In addition, the effect of the subsoil thickness on the...
sliding displacement of the GRS walls could be also observed in tests HR2 and HR2_50 as shown in Fig. 5(a). Even though the shaking conditions in test HR2 with sinusoidal shakings were severer than in test HR2_50 with irregular shakings, the sliding displacement of the walls in HR2 with thicker subsoil before the failure plane formation was larger than that in test HR2_50. This behavior could be explained by considering the effect of subsoil shear deformation as also schematically illustrated in Fig. 7, which will be discussed in the later part of this paper.

A time history of the sliding displacement in test HR2_50 is shown in Fig. 8 where the time histories of the sliding displacement in each shaking step were connected together. The sliding displacement increased even at the shaking step of 350 gals, while the values of evaluated threshold acceleration of the GRS wall model were larger than 350 gals (Table 4). In the conventional NM’s method, however, displacement increment would not be observed during the shaking steps at lower acceleration levels than the threshold acceleration. Moreover, the sliding displacement showed nonlinear accumulation at the shaking step of 350 gals, while the displacement would increase linearly in the conventional NM method. On the other hand, the sliding displacement increased rapidly with the linear accumulation after the failure plane formation in the backfill layer whose behavior seems to be modeled using NM method.

Important aspects observed from the series of the shaking table model tests can be summarized as follows.

1) The reinforced backfill showed shear mode of deformation which possibly induced the tilting of the wall facing.
2) Wall displacements increased rapidly after the formation of the failure plane in the backfill.
3) Failure plane in the backfill developed gradually in the tests on the GRS walls although the failure plane was completed at once in the tests on the conventional type retaining walls (i.e., progressive failure plane formation was more significant in the backfill of the GRS walls as compared with the conventional type retaining walls).
4) The extension of the uppermost reinforcement was effective in reducing the tilting displacement of the walls.

Considering these aspects, a simplified procedure to evaluate earthquake-induced residual displacements of GRS walls is developed in this study.

**DEVELOPMENT OF SIMPLIFIED PROCEDURE**

**Outline of Proposed Method**

The outline of the proposed method is summarized in Table 2. The basic concepts to consider the subsoil and

<table>
<thead>
<tr>
<th>Step 1</th>
<th>Step 2</th>
<th>Step 3</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Sliding displacement</strong></td>
<td><strong>Tilting angle</strong></td>
<td><strong>Before formation of failure plane</strong></td>
</tr>
<tr>
<td>Shear deformation of subsoil</td>
<td>Shear deformation of reinforced backfill</td>
<td>Shear deformation of subsoil</td>
</tr>
<tr>
<td>Newmark’s sliding block method using peak strength</td>
<td>Newmark’s rotating block method using peak strength</td>
<td>Newmark’s sliding block method using residual strength</td>
</tr>
<tr>
<td><strong>Before formation of failure plane</strong></td>
<td><strong>Formation of failure plane</strong></td>
<td><strong>After formation of failure plane</strong></td>
</tr>
<tr>
<td></td>
<td>Formation of failure plane is judged by the maximum shear strain in unreinforced backfill layer computed from the sliding displacement and tilting angle in Step 1.</td>
<td>Shear deformation of subsoil</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Newmark’s sliding block method using residual strength</td>
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<tr>
<td></td>
<td></td>
<td>Shear deformation of reinforced backfill</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Newmark’s rotating block method using residual strength</td>
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</tbody>
</table>
backfill deformation developed by Koseki et al. (2004) for the GRS walls and Nakajima et al. (2009b) for the conventional type retaining walls are also adopted in this study as schematically illustrated in Fig. 7. In the proposed method, the sliding displacement and the tilting angle of the wall facing are considered as the representative residual displacements while their combined effect (Okamura and Matsuo, 2000) is only taken into account in computing the maximum shear strain in the unreinforced backfill which is used as an index to judge the failure plane formation.

As schematically illustrated in Fig. 7, it is assumed in this study that sliding of the GRS wall is induced by the subsoil shear deformation while the tilting of the wall facing is induced by the shear deformation of the reinforced backfill. This assumption follows the experimental observations which have been discussed above. The displacement increment of the GRS wall as a rigid mass, which is considered in the NM method, is also taken into account as well.

The proposed method consists of three steps as summarized in Table 2: displacement calculation before the failure plane formation (Step 1), judgment of the failure plane formation (Step 2) and displacement calculation after the failure plane formation (Step 3).

In Step 1, wall displacements before the failure plane formation are evaluated while the effect of the shear deformation of the subsoil and the reinforced backfill is considered as well as the wall movement as a rigid mass which is evaluated using the NM method. The peak soil strength is used for the displacement calculation in Step 1 because the failure plane is not formed at this step.

In Step 2, the maximum shear strain in the unreinforced backfill layer is evaluated using the calculated sliding displacement and tilting angle in Step 1. It is assumed that failure plane will be formed when the value of the maximum shear strain exceeds a threshold value. The maximum shear strain in the unreinforced backfill is evaluated using the calculated sliding displacement and tilting angle of the GRS walls in Step 1 as schematically illustrated in Fig. 9. The affected area of the unreinforced backfill determined by the angle of the failure plane is assumed to deform as one macro element. For simplicity, it is also assumed that the volume of the affected area will be kept constant (i.e., \( e_h = -e_v \)). However, it should be emphasized that in the case of loosely compacted backfill which would show the negative dilation during the shear deformation, the displacement evaluation will result into less-conservative estimation because the timings of the failure plane formation would be delayed due to the underestimation of the vertical strain (i.e., \( e_v > e_h \) in case \( e_{vol} = e_v + e_h > 0 \)).

In Step 3, wall displacements after the failure plane formation are calculated while the residual soil strength is used so as to consider the effect of the failure plane formation.

It should be noted that observations in the shaking table model tests, which have been discussed in the former part of this paper, are reflected in the proposed method.

First, the effect of the shear deformation of the subsoil and reinforced backfill is considered while it cannot be taken into account in the NM method. Second, the effect of the failure plane formation is also considered by introducing the maximum shear strain in the unreinforced backfill as an index to judge the failure plane formation. Third, the rapid increase of the wall displacement after the failure plane formation can be considered by using the residual soil strength in Step 3. The detailed procedures to evaluate the wall displacement in each step will be presented hereafter.

**Shear Deformation of Subsoil (Steps 1 and 3)**

As summarized in Table 2 and Fig. 7, the proposed method assumes that shear deformation of subsoil induces partly the sliding of the GRS walls.

In the case of conventional type retaining wall, shear deformation characteristics of subsoil beneath the footing were investigated using the results from shaking table model tests and laboratory soil testing (Koseki et al., 2004; Nakajima et al., 2009b). In these attempts, directly measured force-displacement relationships at the bottom of the footing (see Fig. 10) are referred to for the modeling. Shear deformation characteristics of subsoil was modeled as the relationship between shear stress ratio \( SR \) and shear strain \( \gamma \) of subsoil, while the definition of \( SR \) and \( \gamma \) are also schematically illustrated in Fig. 10. In the case of GRS walls, however, it is difficult to properly measure the force acting at the boundary between the subsoil and reinforced backfill. Therefore, a lumped mass
model as shown in Fig. 11 is adopted to evaluate time history of shear force at the boundary between the subsoil and reinforced backfill based on the shaking table model tests, as has been used in similar analyses (Elgamal et al., 1997; Koga and Matsuo, 1990; Zeghal, 1994).

In the lumped mass model, the total soil mass is divided into a number of rigid masses at the middle height of the accelerometers. The shear stress $\tau_i$ (kPa) acting on a soil mass $m_i$ at a distance $z_i$ (m) from the top surface is then simply evaluated by integrating the inertia forces as expressed by Eq. (1).

$$
\tau_i = \sum_{i=1}^{n} m_i \sigma_i \Delta h_i
$$

where $h_i$ (m) is the height of the $i$th soil mass and $\rho$ (kN/m$^3$) is mass density of the soil.

In the evaluation, shear stress ratio $SR (= \tau/\sigma)$ acting at the bottom of the reinforced backfill is obtained by normalizing the shear stress $\tau$ with the average normal stress $\sigma$, while the effect of the vertical inertia is neglected because its amplitude is found to be negligibly small as compared with the horizontal one in the present model test results.

Shear strain $\gamma$ of the subsoil is evaluated by normalizing the sliding displacement of the wall facing at the subsoil surface with the thickness $h$ of the subsoil, while the possible effect of the slippage between the subsoil and the reinforced backfill is not extracted in this evaluation. Based on the relevant image analyses (Nakajima et al., 2008b), it was found that the effect of the slippage became significant after the failure plane formation.

The relationship between shear stress ratio and shear strain in test HR1 is shown in Fig. 12. This relationship can be considered as the averaged shear deformation characteristics of the subsoil beneath the reinforced backfill. In the following analysis, a polynomial fitting curve of the outer envelope of this $SR-\gamma$ relationship modeled as Eq. (2) and indicated in Fig. 12, is used to express the shear deformation characteristics of the subsoil.

$$
\gamma = 0.1632(SR)^3 + 0.0348(SR)^2 + 0.0212(SR)
$$

Usage of the outer envelope in computing the sliding displacement due to the shear deformation of subsoil is based on the observation that the maximum value of $SR$ affects predominantly the amplitude of the residual shear strain (i.e., effect of the cyclic loading on the residual shear strain of dense soils is negligibly small). Based on the relevant hollow cylindrical torsional shear tests on dense Toyoura sand (Nakajima et al., 2009b), the strain induced by cyclic loading has been found to be negligible, while further investigations are required if the subsoil consists of other materials (e.g., soft clay or highly crushable sand and gravel).

**Shear Deformation of Reinforced Backfill (Steps 1 and 3)**

In the proposed method, the reinforced backfill is assumed to be one macro-element which is subjected to shear deformation as schematically illustrated in Fig. 7. Following the same procedure as employed for the modeling of subsoil shear deformation, the relationship between shear stress ratio acting at the bottom of the reinforced backfill $SR$ and shear strain $\theta$ of the reinforced backfill can be obtained as shown in Fig. 13, where the tilting angle of the wall facing is assumed to be equal to the shear strain $\theta$ of the reinforced backfill. As for the shear stress ratio, the values at the bottom of the rein-
forced backfill evaluated using the lumped mass model as explained before are used. The outer envelope of the SR-\( \theta \) relationship is fitted by a polynomial curve of the outer envelope expressed as Eq. (3).

\[
\theta = 0.08(SR)^2 - 0.0083(SR)
\]

It is further requested to develop a rational procedure to model SR-\( \theta \) relationship of the reinforced backfill, because shear deformation characteristics of the reinforced backfill would be affected by the material properties of the geogrid and its vertical spacing as well as the deformation characteristics of the backfill material itself. Another attempt and further proposal for modeling the deformation characteristics of the reinforced backfill are also discussed in the later part of this paper.

**Judgment of Failure Plane Formation (Step 2)**

As explained in the outline of the proposed method, the maximum shear strain \( \gamma_{\text{max}} \) in the unreinforced backfill is used as the index to judge the failure plane formation because good correlation between the values of the maximum shear strain in the backfill layer and the timing of the failure plane formation was observed in the analysis on the relevant shaking table model tests (Nakajima et al., 2006; Nakajima et al., 2009b).

The relationships between the values of \( \gamma_{\text{max}} \) evaluated from the above-mentioned procedure (Fig. 9) and the maximum base acceleration are shown in Fig. 14 while the measured wall displacements and observed angles of failure plane are used in evaluating \( \gamma_{\text{max}} \). Both the timings of initiation and full formation (i.e., failure plane reaches the surface of the backfill) of failure plane based on the observation during the model tests are also indicated in the figure, while notation F is only indicated in tests HR1, HR2 and HR3. The failure plane started to form when the value of \( \gamma_{\text{max}} \) exceeded about 5%, while it was fully formed when the value reached about 10% as indicated in Fig. 14.

Based on the above, it is assumed in the proposed method that the failure plane formation begins when the value of \( \gamma_{\text{max}} \) exceeds 5%. Note also that the value of \( \gamma_{\text{max}} \) (=5%) employed as the threshold condition for the initiation of failure plane formation corresponds to the state of the full formation of the failure plane in the case of the conventional type retaining walls (Nakajima et al., 2009b). This is because the backfill of GRS walls tends to show progressive failure, while such behavior was less significant in the case of conventional type retaining walls. Therefore, it should be noted that the employed assumption yields a conservative displacement evaluation because effect of progressive failure is rather simplified than the observed behaviors.

Similar investigation on the threshold conditions of GRS walls has been also discussed by Izawa et al. (2008). They proposed a shear strain of 3% as the threshold conditions for the initiation of failure plane formation based on the results from centrifuge tilting model tests with a backfill layer consisting of Toyoura sand having relative density of about 80%. It should be noted that soil density difference in Izawa’s series of tests with relative density of 80% and the author’s tests with relative density of 90% cannot explain the difference of the timings of failure plane formations. Therefore, such difference in the threshold values for the failure plane formation is possibly because Izawa et al. (2008) did not consider the effect of sliding displacement of the wall (i.e., horizontal strain \( e_h \)) in the evaluation of \( \gamma_{\text{max}} \), which can be known by referring to their definition of \( \gamma_{\text{max}} \) \( (\gamma_{\text{max}} = 2\theta) \), where \( \theta \) is inclination of the bottom facing panel.

**Evaluation of Threshold Acceleration and Moment (Steps 1 and 3)**

In the proposed method, displacement increments due to the movement of the reinforced backfill as a rigid mass are also taken into account by adopting the NM method for evaluating both the sliding displacement and tilting angle, where the values of threshold acceleration and moment need to be evaluated. The peak soil strength is used in evaluating the threshold values in Step 1, while the
residual one is used in Step 3 so as to consider the effect of failure plane formation on seismic behavior of GRS wall. The threshold values are evaluated based on pseudo-static limit-equilibrium analyses considering the force equilibria as adopted in the relevant design guidelines in Japan (RTRI, 2007; JRA, 1999).

The force equilibria considered in the pseudo-static analyses are shown in Figs. 15(a) and (b). As shown in Fig. 15(a), it is assumed that the driving force $F_d$, which will induce the sliding displacement, is the sum of the horizontal component of the dynamic earth pressure and the inertia force of the reinforced backfill, while the resistant force $F_r$, which will resist against the wall sliding, is the shear resistance mobilized at the boundary between the bottom of the reinforced backfill and subsoil. The basic equations to evaluate $F_d$ and $F_r$ can be expressed as Eqs. (4) to (6). The value of threshold acceleration against the sliding $\delta_{\text{res}}$ is evaluated as the acceleration when the values of $F_d$ and $F_r$ become equal to each other (i.e., the factor of safety against sliding becomes unity).

$$F_d = P_s \times \cos \delta_s + \left(\frac{\alpha}{g}\right) \times W \quad (4)$$

$$F_r = R \times \sin \delta_s \quad (5)$$

$$R = P_s \times \sin \delta_s + W \quad \cos \delta_s \quad (6)$$

The pseudo-static approach proposed by Mononobe and Matsuo (1929) and Okabe (1927) is adopted to evaluate dynamic earth pressure (MO method). Based on results from relevant plane strain compression tests as shown in Fig. 16, the values of peak and residual soil strength are set equal to 51 and 43 degrees, respectively.

Following the same approach as employed for the sliding movement, the value of threshold moment $M_d$ and resistant moment $M_r$. The center of the rotation is assumed to be the center of the reinforced backfill. The actual center of rotation moved during the shaking. For evaluating the center of rotation, precise measurement on the subsoil reaction and dynamic earth pressure acting behind the reinforced backfill are required, while it is difficult to conduct such measurements without any disturbance on the boundary condition. In the simulation, therefore, it is assumed for the simplicity that the center of rotation is kept constant at the center of the reinforced backfill as indicated in Fig. 15(b). As summarized in Fig. 15(b), the value of $M_d$ is evaluated as the sum of the moment induced by the horizontal component of dynamic earth pressure and the inertia force of the reinforced backfill and wall facing. The value of $M_r$ is also evaluated as the sum of the moment induced by subsoil reaction $R$ and vertical component of the dynamic earth pressure. The equations to evaluate the values of $M_d$ and $M_r$ can be expressed as Eqs. (7) and (8), while threshold moment $M_{\text{res}}$ is evaluated as the moment when the values of $M_d$ and $M_r$ become equal to each other.

$$M_d = P_s \times h_p \times \cos \delta_s + \left(\frac{\alpha}{g}\right) \times W \times X = P_s \times \left(\frac{R}{2}\right) \quad (7)$$

$$M_r = W \times X + R \times B \times (d_k - 0.5) \times \sin \delta_s \quad (8)$$

In these calculations, the application points of the resultant forces of dynamic earth pressure $P_s$ and subsoil reaction $R$ need to be determined. For simplicity, it is assumed that the earth pressure is distributed hydrostatically (i.e., $h_p = H/3$ as indicated in Fig. 15(b)) for all the cases. As for the application point of the resultant force of subsoil reaction $R$, it was also assumed that the resultant force would act at the center of the wall facing because the stress concentration of the subsoil reaction would be most significant at the location beneath the wall facing as reported by Ling et al. (2005) although the subsoil reaction could not be directly measured in the present series of the model tests.

The relationships between the values of safety factor $F_s$ against the wall sliding and tilting (i.e., $F_s/F_s$) and $M_r/M_d$)
are typically shown in Fig. 17. As also indicated in Fig. 17, the threshold values are thus evaluated as the acceleration and moment when the value of $F_s$ is equal to unity.

**Evaluation of Pullout Resistance Mobilized by Extended Geogrid**

As discussed in the general model test results, the pullout resistance mobilized by the extended reinforcements worked effectively to reduce the wall displacements. In developing the proposed method, pullout tests of the geogrid models were also carried out so as to evaluate the pullout resistances. The tests on the geogrid models used in the present model tests were carried out by using the apparatus at Tokyo University of Science, Japan. The test apparatus is shown in Fig. 18. The geogrid models, which were the same as the ones employed for the shaking table model tests, were placed in the model ground consisting of air-dried Toyoura sand having a relative density of about 90%. Tests were conducted under the overburden pressures of 5 and 10 kPa, which correspond to the overburden pressure at the middle and full depths of the backfill, respectively.

Test conditions for the series of pullout tests are summarized in Table 3, while the test result on the other type of phosphor bronze geogrid model (PB2, Nojiri et al., 2007) is also included although the value of the overburden pressure and the shape of the geogrid model (as shown in Fig. 19) were slightly different from those in the other tests which correspond to the model tests. The relationships between pullout resistance and pullout displacement are shown in Fig. 19. In test PB1, the peak value of pullout resistance could not be measured because rupture of the reinforcement occurred at outside of backfill soil (Nakajima et al., 2007). On the other hand, the rupture of the reinforcement was not observed in the series of shaking table model tests. Therefore, for the evaluation of frictional resistance on the reinforcement

<table>
<thead>
<tr>
<th>Test name</th>
<th>Geogrid models</th>
<th>Overburden pressure</th>
<th>References</th>
</tr>
</thead>
<tbody>
<tr>
<td>PE $\sigma_v = 10$ kPa</td>
<td>PE</td>
<td>10 kPa</td>
<td>Nakajima et al. (2008)</td>
</tr>
<tr>
<td>PE $\sigma_v = 5$ kPa</td>
<td>PE</td>
<td>5 kPa</td>
<td>Nakajima et al. (2008)</td>
</tr>
<tr>
<td>PB1</td>
<td>PB1</td>
<td>5 kPa</td>
<td>Nakajima et al. (2008)</td>
</tr>
<tr>
<td>PB2</td>
<td>PB2</td>
<td>3.5 kPa</td>
<td>Nojiri et al. (2007)</td>
</tr>
</tbody>
</table>

**Fig. 17.** Typical result of pseudo-static limit-equilibrium analysis (HR1)

**Fig. 18.** Test apparatus for pullout test of geogrid model (unit in mm, Nojiri et al., 2007)

**Fig. 19.** Relationships between pullout displacements and pullout resistances
PB, test result PB2 conducted by Nojiri et al. (2007) was referred in which the rupture of the reinforcement had not occurred because the thickness of the reinforcement was 0.2 mm in test PB2 while the thickness was 0.1 mm in test PB1.

The pullout resistances were measured using the load cell installed at the loading shaft, while the pullout displacements were measured using the displacement transducers with the wires which were connected to the geogrid models at the horizontal distances of about 50, 200 and 400 mm from the wall facing as schematically illustrated in Fig. 18.

The frictional angle $\delta_R$ between the geogrid models and backfill was calculated using Eq. (9) from the observed peak pullout resistance.

$$T_{\text{peak}} = 2 \times A \times (\sigma_v \times \tan \delta_R + c)$$

where $A$ is the area covered by the geogrid models, $T_{\text{peak}}$ is the peak value of the pullout resistance and $c$ is cohesion of the backfill soil ($c = 0$ in this study).

The measured values of the frictional angles $\delta_R$ (= 38 degrees for PB1 and 42 degrees for PE) will be used in the following simulations on the model tests where the extended reinforcements were installed.

RESULTS AND DISCUSSION

Summary of Analytical Conditions

The proposed method is applied to simulate the shaking table model tests conducted under different conditions (Table 1). The parameters used in the simulation are summarized in Table 4. It should be noted that the effective depth of subsoil $h_e$, which is defined as the affected depth that was predominantly subjected to shear deformation, was assumed to be 5.0 cm (full depth of the subsoil) in test HR2_50 and 6.6 cm (one-third of the subsoil thickness) in the other tests. The values of $h_e$ was empirically determined in the relevant simulation on the conventional type retaining wall (Nakajima et al., 2009b). In addition, results of the image analysis also supported these empirical setting (Nakajima et al., 2008b). The image analysis revealed that the subsoil deformation of the model test having a subsoil thickness of 20 cm was clearly observed down to the depth of about 5 to 6 cm from the surface of the subsoil. Based on the above discussion and analysis, the effective depth of 6.6 cm for the subsoil with a thickness of 20 cm was assigned by adopting the empirically determined value for the conservative assumptions, while further investigations are needed for the other subsoil conditions. Results of sensitivity analyses on the values of $h_e$ will be also presented in the following discussion.

In each simulation, the displacements of the model GRS walls were computed as follows (see Table 2 as well). In Step 1, shear stress ratio $SR$ needs to be evaluated based on the procedures as schematically shown in Fig. 20, while the MO method with the peak soil strength was used to evaluate the dynamic earth pressure. The increments of sliding displacement $d_s = \gamma h_e$ and tilting angle $\theta$ due to shear deformation of subsoil and reinforced backfill, respectively, are evaluated by substituting the value of $SR$ into the Eqs. (2) and (3).

The displacement increments due to the wall movements as a rigid mass, were also evaluated from the NM method using the threshold values with the peak soil strength. These increments are evaluated from the following equations and they are added to the above subsoil

---

**Table 4. Summary of the parameters used in the simulations**

<table>
<thead>
<tr>
<th>Case name</th>
<th>Type of wall</th>
<th>$\xi$ (degree)</th>
<th>$h_e$ (mm)</th>
<th>$\delta_R$*** (degree)</th>
<th>$\alpha_{thres}(p)*$ (gal)</th>
<th>$\alpha_{thres}(r)*$ (gal)</th>
<th>$M_{thres}(p)*$ (Nm/m)</th>
<th>$M_{thres}(r)*$ (Nm/m)</th>
</tr>
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<tr>
<td>HR1</td>
<td>Type 1</td>
<td>51</td>
<td>66</td>
<td>—</td>
<td>751</td>
<td>399</td>
<td>364</td>
<td>286</td>
</tr>
<tr>
<td>HR2</td>
<td>Type 2</td>
<td>40</td>
<td>66</td>
<td>38</td>
<td>1127</td>
<td>650</td>
<td>740</td>
<td>562</td>
</tr>
<tr>
<td>HR2_50</td>
<td>Type 2</td>
<td>40</td>
<td>50</td>
<td>38</td>
<td>1127</td>
<td>650</td>
<td>740</td>
<td>562</td>
</tr>
<tr>
<td>HR2_PB</td>
<td>Type 2</td>
<td>40</td>
<td>66</td>
<td>38</td>
<td>1127</td>
<td>650</td>
<td>740</td>
<td>562</td>
</tr>
<tr>
<td>HR2_PE</td>
<td>Type 2</td>
<td>40</td>
<td>66</td>
<td>42</td>
<td>1244</td>
<td>891</td>
<td>798</td>
<td>720</td>
</tr>
<tr>
<td>HR3</td>
<td>Type 3</td>
<td>30</td>
<td>66</td>
<td>—</td>
<td>764</td>
<td>457</td>
<td>**</td>
<td>**</td>
</tr>
</tbody>
</table>

* (p) and (r) denotes the peak and residual state.
** Factor of safety (Mr/Md) did not become unity
*** Evaluated from pullout tests and Eq. (9)
DISPLACEMENT OF GRS WALLS

Fig. 21. Results of simulation on test HR1

Fig. 22. Results of simulation on test HR3

where \( d_s \) is the input ground acceleration, \( M_d \) is driving moment evaluated from Eq. (7), \( \alpha_{thres} \) is threshold acceleration, \( M_{thres} \) is threshold moment and \( I \) is the moment of inertia around the center of the reinforced backfill.

In Step 2, maximum shear strain \( \gamma_{max} \) in the unreinforced backfill layer is evaluated to judge the failure plane formation. When the value of \( \gamma_{max} \) exceeds 5\%, the failure plane will form in the backfill layer and the calculation will shift to Step 3 where the residual soil strength will be used in evaluating the value of \( SR \) and threshold values for NM method. Otherwise, displacements in the next time increment are evaluated following the procedure in Step 1.

The parameters used in the simulation are summarized in Table 4. The effects of pullout resistance \( T \) mobilized by the extended reinforcement in tests HR2, HR2_50, HR2_PB and HR2_PE are also taken into account in evaluating the values of \( \alpha_{thres} \) and \( M_{thres} \). As indicated in the following Eqs. (12) to (14), resistant force and resistant moment mobilized by the extended uppermost reinforcement (see Fig. 1) were added to the values of test HR1. On the other hand, the effect of the middle reinforcement was not considered by reflecting the experimental observation that the mobilized resistance of the middle reinforcement measured in the shaking table model tests was considerably small as compared with that mobilized by the uppermost one (Watanabe et al., 2003).

\[
F = R \times \sin \delta_s + T \quad (12)
\]
\[
M_i = W \times X_s + R \times B \times (d_i - 0.5) \times \sin \delta_i + T \times h_R \quad (13)
\]
\[
T = 2 \times A \times (\alpha \times \tan \delta_R) \quad (14)
\]

where \( h_R \) is the vertical distance of the uppermost reinforcement from the center of the rotation.

Note also that the measured angles of the failure plane are used in the following analysis because the evaluated angle of the failure plane based on the pseudo-static limit-equilibrium approaches did not correspond well to the measured ones.

Results of Analysis for Tests HR1 and HR3 (without Extended Reinforcement)

Results of simulation on tests HR1 and HR3, where the extended reinforcements were not installed, are summarized in Figs. 21 and 22. Both of the calculated sliding displacement and tilting angle for these two tests show reasonable agreements with the measured ones although the calculated sliding displacements are larger than the measured ones under extremely high acceleration levels (e.g., \( \alpha_{max} \) is larger than 1 G). Such overestimation is due possibly to the application of the NM sliding block concept without considering the damping effect, which will be caused by the shear deformation of the reinforced
backfill. The shear deformation of the reinforced backfill, which causes the tilting of the wall facing, was more significant especially under the high acceleration levels and it will result into the absorption of the energy to cause otherwise the sliding movements of the reinforced backfill. Such effect is not taken into account in the calculation of the sliding displacements using the NM method because the reinforced backfill was assumed to be a rigid mass. Further research is required so as to improve the overestimation of the sliding displacement under extremely high seismic loading levels by considering the above behavior.

The calculated displacements for test HR1 using the conventional NM method with the peak and residual soil strengths are also shown in Fig. 21. The calculated sliding displacements using the NM method with the peak soil strength is much smaller than the measured ones, while the ones using the residual soil strengths are much larger than the measured ones. The calculated tilting angles are much smaller than the measured ones even though the mobilized soil strength is assumed to be the residual one throughout the shakings. These results indicate the significance of the shear deformation of the reinforced backfill in evaluating the tilting displacement of the wall facing of the GRS walls.

In Figs. 21 and 22, results of sensitivity analyses on the values of $h_e$ are also indicated. The value of $h_e$ was changed from 33 to 132 mm (i.e., half to twice as compared with the original simulation). Results of displacement evaluation were found to be highly dependant on the value of $h_e$. In the series of simulation on the model tests, the value of $h_e$ could be evaluated reasonably based on the experimental observation and image analyses, while another approach (e.g., FEM analyses) will be required for the further improvement.

Results of Analysis for Tests HR2 and HR2-50 (with Extended Reinforcements)

Result of simulation on tests HR2, where the extended reinforcements were installed, is shown in Fig. 23. The calculated sliding and tilting displacement without consideration of the effect of pullout resistance by the extended reinforcements are much larger than the measured ones. In contrast, the calculated displacements considering the effect of the pullout resistance show reasonable agreements with the measured displacements.

The same tendencies are also observed in the simulation on test HR2-50. In the simulation on test HR2-50 shown in Fig. 24, the effect of the pullout resistance became significant when the base acceleration exceeded about 700 gals whose value was almost equal to the simulation on the test HR2 as shown in Fig. 23.

As also shown in Fig. 24, the calculation considering the pullout resistance can simulate the measured displacements while the ones without considering the effect of pullout resistance are much larger than the measured ones. These results indicate the importance of the pullout resistance mobilized by the extended reinforcements in evaluating the earthquake-induced residual displacement.

Further investigation on the pullout resistance-displacement characteristics under the actual field conditions is required.
It was found from the above analysis that the proposed method could reasonably evaluate the wall displacement of the model tests which were subjected to stepwise shakings although the actual earthquakes are not such stepwise type shakings. Therefore, the simulation on tests HR2_PB and HR2_PE, which were subjected to the large amplitude shaking from the beginning (i.e., $a_{max}$ for the initial shaking was about 0.9 G), have been also carried out. Results of simulations on tests HR2_PB and HR2_PE are summarized in Figs. 25 and 26. The calculated displacements are much larger than the measured ones even though the effects of the pullout resistances are taken into account. However, it should be noted that the proposed method could simulate the increments of the wall displacement except for those by the initial large shaking.

Shear deformation characteristics of the reinforced backfill and subsoil obtained from test HR2_PB are shown in Figs. 27 and 28 where the polynomial fitting curves used in the above simulations (Eqs. (2) and (3)) are also indicated in these figures. As indicated in Table 1, the value of effective depth of subsoil was assumed to be same in the set of simulation by referring to the other model test results. Shear deformation characteristics obtained from test HR2_PB are largely different from the one obtained from test HR1. One of the reasons would be the effect of shaking histories and preceding deformation (Nakajima et al., 2009a). Model ground in tests HR2_PB

**Fig. 25. Results of simulation on test PB**

**Fig. 26. Results of simulation on test PE**

**Fig. 27. Shear deformation characteristics of subsoil in test HR2_PB**

**Fig. 28. Shear deformation characteristics of reinforced backfill in test HR2_PB**

Results of Analysis for Tests HR2_PB and HR2_PE

It was found from the above analysis that the proposed method could reasonably evaluate the wall displacement of the model tests which were subjected to stepwise shakings although the actual earthquakes are not such stepwise type shakings. Therefore, the simulation on tests
and HR2_PE had not been subjected to the preceding deformation while that of test HR1, which was used for the modeling of shear deformation characteristics in the simulation, had been subjected to the preceding shakings. It should be noted that the difference of shear deformation characteristics in tests HR2_PB and HR1 could be caused by possible restraint effect of the extended reinforcements. However, such effect seems to be less significant as compared with the shaking histories because the outer envelope of \( SR-\theta \) relationship in test HR2 was not largely different from the one in test HR1 as indicated in Fig. 28.

The results of modified calculation on test HR2_PB using the shear deformation characteristics obtained from the same test, which are expressed as Eqs. (15) and (16), for the simulation on the initial shaking are also indicated in Fig. 25. In the set of simulations, the effect of shear deformation (Eqs. (2) and (3)) and extended reinforcements (Eqs. (12) to (14)) were modeled separately although it would be possible to model them as the one deformation characteristics by using the records in tests HR2 or HR2_50. But the authors attempted to model them separately for aiming for the further development of the proposed method as discussed in the later part of this paper.

The calculated displacements using the modified shear deformation characteristics for the initial shaking showed more reasonable agreements with the measured ones than the original simulations.

\[
\gamma = 0.1632(SR)^3 + 0.0348(SR)^2 - 0.0212(SR) \quad (15)
\]

\[
\theta = 0.0071(SR)^3 + 0.0023(SR)^2 - 0.0013(SR) \quad (16)
\]

Similar differences between the shaking table model tests subjected to stepwise shaking and large amplitude shaking from the beginning (i.e., like tests HR2_PB and HR2_PE) are also observed in the simulation on conventional type retaining walls (Nakajima et al., 2009b). Further study on effects of the shaking history on seismic behavior of the retaining wall is required using both laboratory soil testing and model test results.

**Requirements for Further Improvements**

It was found that residual wall displacements could be reasonably evaluated based on the simulations using the proposed method. However, following issues shall be investigated for further improvements.

First, a procedure to evaluate the shear deformation characteristics of reinforced backfill shall be developed. In this study, shear stress was evaluated for the modeling of shear deformation characteristics of reinforced backfill based on the lumped mass model using the recorded acceleration time history and wall displacements in the shaking table model tests. Therefore, studies on the modeling of shear deformation characteristics of the reinforced backfill are required for further developments of the proposed method.

In this study, it was also attempted to develop a procedure to obtain shear deformation characteristics of the reinforced backfill using the laboratory hollow cylindrical torsional shear (HCTS) tests as schematically illustrated in Fig. 29. Shear deformation of the reinforced backfill was simulated using the hollow cylindrical specimens reinforced with geogrid models while the vertical spacing of the geogrid models were set equal to the backfill of the shaking table model tests (i.e., 5 cm).

To simulate the conditions of the shaking table model tests, air-dried Toyoura sand specimens with the relative density of about 90% were sheared while horizontal and vertical stresses were set equal to 30 and 60 kPa, respectively. Although the value of horizontal confining stress in the HCTS was larger than the one in the backfill of the model test (i.e., about 5 kPa at the bottom of backfill), the effect of confining stress on the deformation and strength characteristics of Toyoura sand under such low confining stresses were experimentally proven to be negligible by Tatsuoka et al. (1986).

Shear deformation characteristics obtained from the HCTS test was compared with the one of test HR1 in Fig. 30. The laboratory test results showed reasonable agreements with the measured ones in test HR1 especially at the beginning of the shearing, while the laboratory tests did not show continuous increase in the shear stress ratio as observed in test HR1. This discrepancy is possibly because of the effect of the wall facing in the shaking table model tests. It was reported by Tatsuoka et al. (1989) that mobilization of the tensile force of the geogrid reinforcement, which will improve the shear deformation characteristics of reinforced backfill, was significantly affected by the facing rigidity. Since there was no effect of the facing rigidity in the HCTS tests, the \( SR-\theta \) relationships obtained in this test HCTS showed different trends from the one in test HR1.

Usage of the small scale model test with monotonic or cyclic loading, which can simulate the effect of the wall facing, will be more appropriate for further investigation into the shear deformation characteristics of the reinforced backfill.

In addition to the modeling of the shear deformation characteristics of the reinforced backfill, further investigation on the effects of pullout resistances is also important. In the above analysis, only the maximum pullout resistances were taken into account in evaluating the
threshold values for the NM method. However, possible restraint effects of the pullout resistance on the shear deformation of the reinforced backfill were not taken into account. In considering the above mentioned restraint effect, pre-peak behavior of the pullout resistances shall be properly modeled. Overestimation of the tilting displacement of the model tests with the extended reinforcements which is typically shown in Fig. 23, shall be further improved by considering the above mentioned restraint effect.

A generalized procedure to set the effective depth of the subsoil for the shear deformation should be also developed for the further applications of the proposed method to the practice although the effective depth of the subsoil in the shear deformation was evaluated from the observation and relevant hollow cylindrical torsional shear tests in this study. Effective depth of the subsoil is thought to be affected by the footing width, subsoil deformation characteristics and shear stress ratio $SR$ acting on the subsoil. The series of the finite element analyses shall be conducted in further study so as to make it as a basis for developing the methodology to set the effective depth of the subsoil. The applicability of the proposed method to the actual cases will be examined through the simulations on the damaged and non-damaged retaining walls for the recent large earthquake.

It should be emphasized that the scale effect is also important in adopting the proposed method to the prototype scale retaining walls. The effects of the grain size effects on strain softening behavior and stress-dependant deformation characteristics of the backfill and subsoil can be highlighted as the typical scale effect in this case. As for the grain size effect, required shear displacements along the failure plane to reach the residual state from the peak state increased proportionally with the increase of the particle size (Yoshida et al., 1997). In the proposed method, however, it is assumed for conservative displacement evaluation that the mobilized soil strength will reduce to residual value immediately after the failure plane formation. This conservative assumption may result into the overestimation of the displacement in the case of prototype scale because the grain size of the backfill in the prototype will be larger than the ones used in the model tests.

In the proposed method, shear deformation characteristics of the subsoil and reinforced backfill are modeled based on the model test result where the confining stress was almost equal to one tenth as compared with the prototype scale. Therefore, the effects of stress dependent deformation characteristics of geo-materials cannot be considered if the proposed procedure was applied to the actual retaining walls. To solve this problem, it is also attempted in this study to propose a method to evaluate shear deformation characteristics of the reinforced backfill by conducting the laboratory hollow cylindrical torsional shear test (Fig. 29), while it has not yet been developed. Small scale model tests with the application of sufficient confining stress will make it possible to evaluate the deformation characteristics considering the effect of stress dependant deformation characteristics of geo-materials for further improvement.

CONCLUSIONS

Based on the seismic behaviors observed in the shaking table model tests, a simplified procedure to evaluate residual wall displacements of the GRS wall with full-height rigid facing was developed by considering the following aspects.

1) Shear deformation of the subsoil and reinforced backfill
2) Shear deformation and failure plane formation in the unreinforced backfill which is associated with the strain softening behavior
3) Effects of the pullout resistance mobilized by the extended reinforcements

By using the proposed method, simulations of the model tests were carried out. Reasonable agreements between the results from a series of model tests and simulations were obtained in terms of the wall displacements as well as the timing of the failure plane formation in the backfill layer.

Further investigations on the modeling of the shear deformation characteristics of the reinforced backfill and pullout resistances mobilized by the extended reinforcements will be required to improve the proposed method.

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NOTATIONS

- $A$: area covered by the geogrid
- $a_{base}$: base acceleration
- $a_{ground}$: ground acceleration
- $a_{max}$: the maximum acceleration
- $\alpha_{thres}(p)$: threshold acceleration obtained by using $\phi_{peak}$
- $\alpha_{thres}(f)$: threshold acceleration obtained by using $\phi_{res}$
- $B$: width of reinforced backfill
- $d$: application point of subsoil reaction from edge of reinforced backfill
- $d_s$: sliding displacement of wall
- $d_{sa}$: normalized distance of application point of subsoil reaction from edge of reinforced backfill
- $\delta_k$: frictional angle between the geogrid model and backfill
- $\delta_f$: frictional angle between the bottom of reinforced backfill and subsoil
- $\delta_s$: frictional angle at the edge of the reinforced backfill
- $e_h$: horizontal strain of affected area of unreinforced backfill
- $e_v$: vertical strain of affected area of unreinforced backfill
- $F_d$: driving force
- $F_r$: resistant force
- $\phi_{mob}$: mobilized angle of internal friction
- $\phi_{peak}$: peak angle of internal friction
- $\phi_{res}$: residual angle of internal friction
- $\gamma$: shear strain of subsoil
- $\gamma_{max}$: the maximum shear strain of affected area of backfill
- $\gamma_s$: shear strain of affected area of backfill
- $H$: total height of wall
- $h$: thickness of subsoil
- $h_k$: vertical distance of the uppermost reinforcement from the center of rotation
- $h_e$: effective depth of subsoil
- $I$: moment of inertia around center of rotation
- $L$: length of affected area of unreinforced backfill
- $M_d$: driving moment around the center of the reinforced backfill
- $M_r$: resistant moment around the center of the reinforced backfill
- $P_d$: dynamic earth pressure acting on wall facing
- $\theta$: tilting angle of wall
- $R$: resultant force of subsoil
- $SR$: shear stress ratio ($= \tau/\sigma$)
- $\sigma$: normal stress
- $\sigma_s$: major principle stress
- $\sigma_f$: minor principle stress
- $T_{peak}$: peak pullout resistance
- $\tau$: shear stress
- $W$: weight of retaining wall including reinforced backfill
- $X_s$: horizontal distance of center of gravity from center of reinforced backfill
- $\xi$: angle of failure plane from horizontal direction
- $Y_s$: vertical distance of center of gravity from center of reinforced backfill

REFERENCES