EVALUATION OF EXTENT OF DAMAGE TO GEOGRID REINFORCED SOIL WALLS SUBJECTED TO EARTHQUAKES

JUN IZAWA and JIRO KUWANO

ABSTRACT

The aim of this study is to establish a simple method for evaluating the extent of damage to geogrid reinforced soil walls (GRSWs) subjected to earthquakes. Centrifuge tilting and shaking table tests were conducted to investigate the seismic behaviour of GRSWs, with special focus on the effects of the tensile stiffness of the geogrids, the pullout characteristics and the backfill materials. As a result, it was found that GRSWs showed large shear deformation in the reinforced area after shaking, that such deformation was influenced by the tensile stiffness of the geogrids, the pullout resistance and the deformation modulus of the backfill material, and that finally slip lines appeared. However, the GRSWs maintained adequate seismic stability owing to the pullout resistance of the geogrids, even after the formation of slip lines. It is considered that such slip lines appeared due to the failure of the backfill material. Since the maximum shear strain occurring in the backfill can be roughly estimated from the inclination of the facing panels, using a simple plastic theory, it is possible to evaluate whether the backfill has reached its peak state or not. The formation of slip lines observed in the centrifuge model tests could be well explained by this method. Finally, the method is proposed to estimate the failure sections in the GRSWs using a Two Wedge analysis.

Key words: earthquake, geosynthetics, reinforced soil, retaining wall, slip line (IGC: E8/H2)

INTRODUCTION

Earth structures reinforced with geosynthetics are widely used and contribute towards the improvement of the stability of many kinds of earth structures. In particular, steep embankments and soil walls can be constructed more easily and more economically with geogrids than with other conventional methods. Besides these two advantages, it is well known that geogrid reinforced soil walls (GRSWs) have a much better seismic performance compared to that of other conventional retaining walls. Tatsuoka et al. (1996) reported damage to some types of retaining walls after the 1995 Hyogo-ken Nambu Earthquake. According to their report, the GRSWs having a full-height rigid facing wall, used for railways, showed very small displacement, although they were located in areas which experienced severe shaking, where many conventional gravity types of retaining walls fully collapsed. Nishimura et al. (1996) also investigated the damage to GRSWs having divided facing panels after the earthquake. They reported that the GRSWs subjected to a seismic intensity of 6 or more showed no apparent failure, and maintained adequate stability during/after the earthquake. Due to these experiences, the total number of GRSWs being constructed has been increasing significantly since the Kobe earthquake, even though construction works in general in Japan have been decreasing year by year due to economic problems (Koseki et al., 2006).

However, it is natural that GRSWs show some extent of deformation during/after earthquakes, because their reinforcing effect can be mobilized after a certain amount of deformation to the GRSWs themselves. In particular, there is a possibility that the deformation of GRSWs could be significant due to bad weather conditions, inadequate construction techniques and so on. In fact, some GRSWs showed relatively serious deformation during/after the 2004 Niigata-ken Chuetsu Earthquake. One of the possible reasons for such severe damage is the heavy rain which hit the same area right before the earthquake and caused saturation to most of the backfill (Yoshida et al., 2005; The Japanese Geotechnical Society, 2007). It is obvious that damaged GRSWs with some deformation have a factor of safety greater than one under ordinary conditions. However, the factor of safety against the next event, e.g., a large earthquake or heavy rain, is not clear. In such cases, it is necessary to evaluate the residual stability of the damaged GRSWs and to assess the necessity of repairing or reconstructing them for the quick recovery of infrastructures. Such assessments should usually be made based only on surface deformation, such as wall displacement or the settlement at the crest, without a de-
tialed investigation into the damaged GRSWs. At present, there is only one empirical method for assessing the failure risk of reinforced cuttings (e.g., Japan Highway Public Corporation, 2002). Thus, the purpose of this study is to establish a simple and reasonable method for evaluating the extent of damage to GRSWs subjected to earthquakes based on surface deformation.

For the above purpose, it is essential to know the failure and the deformation behaviour of GRSWs during/after earthquakes in detail. Many studies have been undertaken on the seismic stability of GRSWs, focusing on the effects of the slope, the surface gradient, the spacing, the kind of facing, the length and the type of reinforcement(s) (e.g., Koga et al., 1988; Koga and Washida, 1992; Sakaguchi et al., 1992; Sato et al., 1995; Takahashi et al., 1999; Watanabe et al., 2003; El-Eman and Bathurst, 2004, 2005, 2007). However, they did not mention the extent of damage to the geogrid reinforced soil walls subjected to earthquakes. In addition, little attention has been paid to the effect of the properties of geogrids (e.g., tensile strength, stiffness and shape) or to the soil–geogrid interaction on the seismic stability of GRSWs, although the seismic behaviour of GRSWs may be significantly influenced by them. Therefore, in order to observe the seismic behaviour of GRSWs and to investigate the effect of the properties of geogrids and the soil–geogrid interaction, a series of centrifuge shaking table tests was conducted using some kinds of model geogrids and backfill material. In addition, centrifuge tilting table tests were conducted with the same centrifuge models to observe in detail the failure and the deformation behaviour of GRSWs subjected to a horizontal body force.

It is necessary to define some criteria for evaluating the extent of damage to GRSWs, which has to be evaluated based only on the surface deformation of the GRSWs, as mentioned above. Since GRSWs are composed of backfill material and geogrids, the failure or the extent of damage can be determined by ruptures or pullouts of the geogrids, the failure of the backfill material and so on. However, the meaning of damage to the backfill should be clarified, because GRSWs are soil structures that are mainly composed of backfill material. Thus, damage to the backfill is selected as an index to evaluate the extent of damage to the GRSWs, and the relationship between the surface deformation of GRSWs and damage to the backfill is discussed based on the test results. Finally, this paper proposes a simple and rational method, using surface deformation, to evaluate the extent of damage to GRSWs subjected to earthquakes.

**CENTRIFUGE MODEL TESTS**

The purpose of centrifuge model tests is to determine the relationship between the surface deformation of GRSWs and the extent of damage to the GRSWs brought about by earthquakes. Although shaking table tests are usually conducted to establish the seismic behaviour of soil structures, it is difficult to observe the deformation in detail during shaking, because a seismic phenomenon happens too quickly, especially in centrifugal gravity fields. Besides centrifuge shaking table tests, centrifuge tilting table tests were also conducted on the same centrifuge models used in the centrifuge shaking table tests. Centrifuge tilting table tests can apply a horizontal body force to the model statically, which is the same condition as that assumed in the pseudo static analysis used for ordinary as seismic designs. And, the deformation of the model can be observed in more detail than in centrifuge shaking table tests. This advantage can greatly help to determine the relationship between the surface deformation of GRSWs and the degree of damage to GRSWs subjected to a horizontal body force.

The seismic behaviour of GRSWs is probably affected by the types of geogrids and backfill material used. Therefore, the effects of such materials are also discussed by conducting some centrifuge tests with two types of model geogrids and three types of backfill material. Particular focus is placed on the stiffness and the pullout characteristics of geogrids and on the particle size of the backfill. Model setups and a test procedure will be presented below in brief, but details of the tests may be found in Izawa (2008).

**Backfill Material and Model Geogrids**

Toyoura sand and Silica sand Nos. 5 and 3 were selected for the centrifuge tests in order to establish the effect of particle size, because it is well known that particle size affects the mechanical properties of backfill (e.g., Yoshiida and Tatsuoka, 1990). These materials are mainly composed of silica, but have different particle sizes, which are larger in the order of Toyoura sand, Silica sand No. 5 and Silica sand No. 3. Particle size distribution curves are shown in Fig. 1. Each average particle size, \( D_{50} \), is 0.190, 0.520 and 1.40 mm, respectively. All the materials were used with a relative density of 80% in the centrifuge models. The physical and the mechanical properties of the soils are summarized in Table 1. The internal frictional angle, \( \phi' \), was determined from the peak deviator stress, \( (\sigma_1-\sigma_{3})_{\text{peak}} \), and the confining pressure, \( \sigma_3' \), was obtained from drained triaxial tests under confined pressure.

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**Fig. 1.** Particle size distributions of Toyoura sand and Silica sand Nos. 5 and 3.
Table 1. Properties of backfill materials

<table>
<thead>
<tr>
<th></th>
<th>Toyoura</th>
<th>Silica No. 5</th>
<th>Silica No. 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average particle size: (D_{50}) (mm)</td>
<td>0.190</td>
<td>0.520</td>
<td>1.40</td>
</tr>
<tr>
<td>Maximum void ratio: (e_{	ext{max}})</td>
<td>0.973</td>
<td>1.107</td>
<td>1.009</td>
</tr>
<tr>
<td>Minimum void ratio: (e_{	ext{min}})</td>
<td>0.609</td>
<td>0.720</td>
<td>0.697</td>
</tr>
<tr>
<td>Void ratio at (D_r=80%: e)</td>
<td>0.682</td>
<td>0.797</td>
<td>0.759</td>
</tr>
<tr>
<td>Dry unit weight at (D_r=80%: \gamma) (kN/m³)</td>
<td>15.4</td>
<td>14.3</td>
<td>14.5</td>
</tr>
<tr>
<td>Internal friction angle at (D_r=80%: \phi) (°)</td>
<td>40.4</td>
<td>45.0</td>
<td>46.0</td>
</tr>
<tr>
<td>Deformation modulus at (D_r=80%: E_{50}) (MN/m²)</td>
<td>(0.944\sigma_c^{0.728})</td>
<td>(0.985\sigma_c^{0.728})</td>
<td>(3.329\sigma_c^{1.124})</td>
</tr>
</tbody>
</table>

\(^{*}\sigma_c: \) confining pressure (kPa)

Table 2. Properties of model geogrids

<table>
<thead>
<tr>
<th>Name</th>
<th>CS</th>
<th>CS2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thickness (mm)</td>
<td>0.5</td>
<td>1.0</td>
</tr>
<tr>
<td>Tensile strength (kN/m)</td>
<td>7.01</td>
<td>20.2</td>
</tr>
<tr>
<td>Tensile strain at rupture (%)</td>
<td>4.21</td>
<td>4.72</td>
</tr>
<tr>
<td>Tangent tensile stiffness (kN/m)</td>
<td>197</td>
<td>557</td>
</tr>
<tr>
<td>Secant tensile stiffness (kN/m)</td>
<td>166</td>
<td>428</td>
</tr>
</tbody>
</table>

Fig. 2. Shape of model geogrids

Fig. 3. Pullout resistance vs Pullout displacement

Fig. 4. Pullout strength vs Vertical stress

levels, \(\sigma_v\), of 49, 98 and 147 kPa. \(E_{50}\) was determined as the secant Young’s modulus at half of the peak deviator stress. The approximate equations listed in Table 1 were determined from the relationship between \(E_{50}\) and \(\sigma_c\). Clearly, backfill soil with a larger particle size has a larger internal friction angle and a higher deformation modulus.

Two types of model geogrids, referred to as CS and CS2, were used. Type CS was made of polycarbonate plates with a thickness of 0.5 mm. Type CS2 was made of polycarbonate plates with a thickness of 1.0 mm to produce a higher tensile stiffness than and the same surface friction as Type CS. The two types of geogrids had the same shape, as shown in Fig. 2. The shape and the materials were selected to obtain sufficient tensile stiffness and pullout resistance. The tensile properties of the geogrids are summarized in Table 2; they were obtained from tensile tests. The tensile stiffness of Type CS2 was about 2.5 times higher than that of Type CS. Additionally, pullout tests were conducted to investigate the pullout characteristics between each type of backfill material and each geogrid. Figures 3(a) and (b) show typical pullout behaviour at vertical stress levels, \(\sigma_v\), of about 50 kPa and 80 kPa, respectively. The pullout strengths are plotted against \(\sigma_v\) in Fig. 4, where the pullout strength was determined as the peak pullout resistance or the pullout resistance at the maximum pullout displacement of 20 mm. The apparent cohesion, \(c_p\), and the pullout friction angle, \(\phi_p\), which are the interception and the gradient of the pullout strength and the vertical stress relation, respectively, were obtained from Fig. 4, as is summarized.
Table 3. Pullout characteristics

<table>
<thead>
<tr>
<th>Model geogrid</th>
<th>CS</th>
<th>CS2</th>
<th>CS2</th>
<th>CS2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Backfill material</td>
<td>Toyoura silica</td>
<td>No. 5</td>
<td>Silica</td>
<td>No. 3</td>
</tr>
</tbody>
</table>

| Apparent cohesion: \(c_p\) (kPa) | 0.930 | 7.90 | 16.7 | 30.2 |
| Pullout friction angle: \(\phi_p\) (°) | 21.4 | 40.5 | 44.5 | 46.1 |
| Pullout strength ratio: \(\tan \phi_p / \tan \phi\) | 0.451 | 0.983 | 0.983 | 1.003 |

in Table 3. The pullout friction angle of Type CS with Toyoura sand was almost half of that of Type CS2. This difference is attributed to the difference in thickness, because the surface friction between them is the same. The apparent pullout cohesion and the pullout friction angles of Type CS2 increased in the order of the particle size of the three types of sand. However, the pullout strength ratio, \(\tan \phi_p / \tan \phi\), was almost the same for all three of them.

Similarity for Centrifuge Modelling

The similarity for model geogrids should be carefully considered in centrifuge model tests on GRSWs. Many researchers have reported studies on the similarities (e.g., Springman et al., 1992; Zonberg et al., 1997; Viswanadham and Konig, 2004; Viswanadham and Mahajan, 2007). It is difficult to fulfill the geometric properties between the model and the real geogrid. It is more important to satisfy the mechanical and the soil-geogrid interaction properties than to make just a miniature. The first requirement is to use a model geogrid with \(1/N\) reduced stiffness compared to a real geogrid, where \(N\) indicates the centrifugal acceleration. To satisfy the second similarity, the model geogrid must have the same friction angle as that of a real geogrid.

Kuwano et al. (1999) investigated some mechanical and pullout characteristics of 11 kinds of geogrids used in Japan. They reported that the tensile stiffness of real geogrids ranged mainly from 200 kN/m to 3000 kN/m, although there was a very stiff geogrid with a tensile stiffness of about 11950 kN/m. The levels of tensile stiffness for CS and CS2 were 166 kN/m and 428 kN/m, respectively, which correspond to about 8000 kN/m and 21400 kN/m in centrifugal acceleration of 50 G. Although the model geogrids were somewhat stiff, such high stiffness and large strength were necessary to obtain the pullout characteristics in the pullout tests. In fact, as indicated in Fig. 3, even CS2 ruptured in the pullout test conducted in this study at a vertical pressure of 80 kPa. Past studies have revealed that a high pullout resistance is effective in the seismic stability of reinforced soil structures (e.g., Huang et al., 2008a, b). Thus, relatively high stiffness is necessary to model geogrids reasonably in centrifuge modeling. Kuwano et al. (1999) also reported, in regards to the pullout friction angle, that pullout strength ratios \(\tan \phi_p / \tan \phi\) were about 0.5 to 1.5. As indicated in Table 3, although the pullout strength ratios for CS seem to be a little bit lower, CS2 had sufficient pullout resistance for all three types of backfill material. Therefore, the geogrids seem to be modeled reasonably well in this study.

Outline of the Tests and Model Setup

Tokyo Tech Mark 3 Centrifuge (Takemura et al., 1999) was used for both the centrifuge tilting and the shaking table tests. Four cases were conducted with two types of model geogrids and three types of backfill material, as listed in Table 4. The first and second parts of each case name indicate the geogrid type and the backfill material, respectively. All the tests were conducted at a centrifugal acceleration of 50 G. The model GRSWs had a divided facing panel system, which is usually used for roads in Japan. Model setups for the centrifuge tilting and shaking table tests are shown in Figs. 5(a) and (b), respectively. The models were made by the air pluviation method to achieve a relative density of 80% in each
strong box. Although a laminar container is recommended for use in dynamic centrifuge model tests, especially in the case of a level ground, it is not suitable for model tests with asymmetric models, such as retaining walls and slopes. If a laminar container were used for the shaking table tests on GRSWs, the walls would deform toward the backfill. This is because the backfill does not receive any reaction force from the laminar wall, while it does receive reaction force from the facing and the relatively stiff reinforced zone. Residual deformation toward the backfill is not realistic. Thus, we used a rigid container with cushions on both sides of the box. For the cushions, 10-mm thick urethane with a stiffness of 210 kN/m² was selected. The height of the models was 150 mm, which corresponded to 7.5 m in prototype scale. Five layers of 90-mm long geogrids were laid in the backfill at 30-mm intervals, where the same geogrids were used along height to clarify the behaviour of the model and the effect of the material properties. Pre-cast concrete facing panels, which are commonly used in Japan, were modelled with polycarbonate plates with a thickness of 5 mm and triangular channels, as shown in Fig. 5. One geogrid was rigidly attached to the mid-height of one facing panel with epoxy adhesive. They were not fixed to each other and could rotate relatively freely. Surface displacements were measured with displacement transducers. In the centrifuge shaking table tests, accelerometers were set in the backfill to measure the response acceleration. Pre-failure deformation in the backfill was observed from digital images obtained from a CCD-TV camera. Optical targets were set on the glass side to form a grid with an interval of 15 * 15 mm and 10 * 10 mm for CS-T and the other cases, respectively. As optical targets, small nails, 5 mm * 5 mm in length, and an OHP sheet were used. The displacement of the optical targets, which were set on the glass side wall, was determined from the images using the image processing technique (Tayler et al., 1998). The displacement vectors and the strain distributions of the models were calculated using the results of the image analysis.

**Centrifuge Shaking Table Tests**

The centrifuge shaking table tests were conducted with a Horizontal-Vertical 2D shaker for the Tokyo Tech Mark 3 Centrifuge (Takehara et al., 2002). Although this shaker can operate horizontal and vertical accelerations independently, only the horizontal acceleration was applied to the GRSWs in this study. As indicated in Fig. 6, 20 cycles of sinusoidal waves at a frequency of 100 Hz (2 Hz in the prototype) were applied to the GRSWs during the first three shaking steps. Maximum accelerations were approximately 5 G, 15 G and 20 G for the 1st, 2nd and 3rd shaking steps, respectively. In the following steps, sinusoidal waves with 40 cycles and a maximum acceleration of 20 G were applied to the model several times until the model showed significantly large deformation.

**SEISMIC BEHAVIOUR OF GRSWS**

**Behaviour of GRSWs Subjected to Pseudo Static Loading**

Figure 7 shows the relationship between the horizontal displacement at the top facing panel and horizontal seismic coefficient $k_h$, which is calculated by the following equation.

$$ k_h = \tan \eta \quad (1) $$

where $\eta$ is the tilting angle. All cases showed almost the same behaviour, as follows. The GRSWs did not show any deformation during the spinning up of the centrifuge and at the early tilting stage. Then, the GRSWs began to deform at a particular seismic coefficient. After that, the deformation rate rapidly increased and the GRSWs fully collapsed. Failure points were determined as the horizontal displacement began to increase infinitely, as shown in Fig. 7. For example, in the case of CS2-T, the GRSW be-
gan to deform at about $k_h = 0.38$, and showed very large displacement due to the sliding of the reinforced area at about $k_h = 0.45$. In comparison with CS-T, CS2-T showed higher stability. This is because the CS2 type could restrict the deformation of the reinforced area owing to its higher tensile stiffness against the pseudo static horizontal force. CS2-S5 and S3 show relatively high stability compared to that of CS2-T, because backfill material with a larger particle size has a higher internal friction angle and a higher deformation modulus. These results clearly show that GRSWs having geogrids with higher tensile stiffness and backfill material with a larger particle size can show higher seismic stability.

Figures 8(a)–(d) show the distributions of maximum shear strain $\gamma_{\text{max}}$ around the time the GRSWs failed. The strain levels were calculated from the displacement of the optical targets set on the model. The concentrated area of $\gamma_{\text{max}}$ is considered to denote a slip line in these figures. Thus, it is found that clear slip lines with a two-wedge shape appeared in the cases of CS-T, CS2-T and CS2-S5 when the GRSWs failed, as shown in Fig. 7. Furthermore, these slip lines were not observed until just before the failure. For example, in the case of CS2-T, although there is no concentrated area at $k_h = 0.445$ ($\eta = 24.0^\circ$), a clear slip line can be seen at $k_h = 0.455$ ($\eta = 24.5^\circ$) at which time the GRSW almost collapsed. In other words, the GRSWs failed immediately after the formation of slip lines. That is, failure of GRSWs can be defined as the moment when the backfill material reaches failure with the formation of slip lines. On the contrary, such a clear slip line was not observed in the case of CS2-S3, as is shown in Fig. 8(d). In other words, the GRSW of CS2-S3 failed without showing a clear slip line at a higher horizontal seismic coefficient.

It is thought that the formation of such slip lines results from the failure of the backfill material due to the deformation of the reinforced area. In order to grasp the deformation mode before failure, distributions of horizontal displacement along the height are indicated in Fig. 9, which were displacements of the targets next to the panels. The displacement of the lowest target and the inclination of the horizontal displacement distribution denote sliding displacement and shear deformation, respectively, as indicated in Fig. 9. Almost the same deformation mode was observed in all cases. The GRSWs showed a relatively large shear deformation. In addition, such shear deformation was significant in the lower part. That is, it is thought that the backfill material collapsed to form slip lines due to the shear deformation. Then, the facing panel inclinations in the lower part, $\beta_u$, are plotted against the horizontal seismic coefficient in Fig. 10. The failure points shown in Fig. 7 are also indicated in Fig. 10. This figure clearly shows that the slip lines appeared and the GRSWs collapsed when the facing panel inclination in the lower part reached about $3.0^\circ$ for CS-T, CS2-T and CS2-S5. In the case of CS2-S3, although clear slip lines were not found, $k_h$ for a $\beta_u$ of $3^\circ$ corresponds to the $k_h$ at failure, as shown in Fig. 7.
DAMAGE IN GRSWs

In summary, the GRSWs show large shear deformation in the lower part for pseudo static horizontal loading. When the facing panel inclinations, due to such shear deformation, reached about 3.0°, the backfill material failed and slip lines appeared. The GRSWs collapsed immediately after the formation of slip lines. As mentioned earlier, the centrifuge tilting table tests can apply horizontal body force to the GRSWs as an earthquake load, whereby the loading conditions are the same as those modelled in designs. The failure modes observed in this study were almost the same as those assumed in the designs. This means that the concept of the present design regarding failure is valid. Although tilting table tests cannot simulate the real seismic behaviour of earth structures, they are useful for observing the behaviour of the GRSWs subjected to the same horizontal body force as that assumed in pseudo static method. We will discuss a comparison between the results of the centrifuge tilting table tests and the analysis with the pseudo static method later.

**Behaviour of GRSWs during Earthquakes**

Figure 11 shows the time histories for the horizontal displacement at the middle height of the GRSWs in the second shaking step together with the input acceleration time history in the case of CS-T. It has been confirmed that almost the same input motion could be applied in each test. Similar deformation behaviour was observed in all the cases. Namely, horizontal displacement gradually increased with shaking, but the GRSWs became stable again after the shaking ceased. In addition, the deformation rate gradually decreased during the shaking. This is probably because the tensile stress was effectively generated in the geogrids due to the deformation of the reinforced area, and the reinforced area could receive additional confining pressure. This is typical deformation behaviour for GRSWs, and such behaviour results in higher seismic stability of GRSWs in comparison to the other conventional earth structures. In addition, no ruptures were observed in any of the cases.

Similar to the results of the centrifuge tilting table tests, Fig. 11 shows that higher seismic stability could be obtained in the case with stiffer geogrids and a larger particle size of the backfill to some extent. However, the effect of the maximum input acceleration as well as the shaking duration should be considered when comparing the seismic stability of GRSWs, because GRSWs deform gradually with shaking, as described above, and the maximum acceleration alone does not characterize the input motion. In order to consider both effects, acceleration power \( I_E \) is used here as follows:

\[
I_E = \int_0^T a^2 dt
\]

where \( a \) is the input acceleration and \( T \) is the shaking duration. This index indicates the total power of the input seismic wave like arias intensity (e.g., Kayen and Mitchell, 1997). Figure 12 shows the relationship between the residual horizontal displacement after each shaking step and the cumulative acceleration power. By using the acceleration power, the effect of the properties of the geogrids and the backfill material on the seismic stability of the GRSWs can be clearly seen.

Figure 13 shows the vertical distributions of the horizontal displacement of the GRSWs along the height. Similar to the results of the centrifuge tilting table tests, the shear deformation in the lower part was significant. Thus, facing panel inclinations in the lower part, \( \theta_a \), are plotted against the cumulative acceleration power in Fig. 14. For CS2-T and CS2-S5, the facing panel inclinations exceeded 3.0° after shaking steps 3 and 2, respectively. Thus, it is assumed that slip lines had already appeared. In order to confirm this, the maximum shear strain distributions for CS2-T and CS2-S5, after some shaking steps,
are shown in Figs. 15 and 16, respectively. In the case of CS2-T, a very clear slip line can be seen in the backfill after Step 8 (Fig. 15(e)). However, it is found that $\gamma_{\text{max}}$ began to concentrate around the clear slip line observed in Fig. 15(b) after Step 3. Therefore, it is thought that the backfill material had already failed and a slip line had begun to appear during shaking in Step 3. Also, for CS2-S5, $\gamma_{\text{max}}$ began to concentrate around the slip line during shaking in Step 2 (Fig. 16(b)), which can be clearly seen during the shaking in Step 13 (Fig. 16(e)). That is, the slip line had already begun to appear during the shaking in Step 2, simultaneously with the failure of the backfill material. This result indicates that the slip lines appeared when the inclination of the facing panels in the lower part had reached about 3.0° for CS2-T and CS2-S5, similar to the results of the centrifuge tilting table tests. In addition, the failure mode was very similar to that in the centrifuge tilting table tests. On the other hand, no clear slip line appeared in CS2-S3, although the facing panel inclination was over 10 degrees, as shown in Fig. 17, which shows the distributions of maximum shear strain after the shaking in Step 13. Unfortunately, a similar statement cannot be made about CS-T, because clear maximum shear strain distributions could not be obtained for it, probably due to a measurement problem.

It is thought that the seismic stability of GRSWs after the formation of a slip line depends on the pullout resistance, while the stiffness of geogrids and backfill material are mainly effective before the formation of a slip line. Figure 18 shows an enlarged view of Fig. 12. CS2-T and CS2-S5 showed almost the same horizontal displacement before the shaking in Step 3. After that, larger displacement could be seen with the growth of a slip line in CS2-T. As mentioned earlier, CS2-S5 had a larger pullout resistance than CS2-T. This result supports the concept that the seismic stability of GRSWs with a slip line depends on the pullout resistance.

It should be noted that the GRSWs did not collapse in the centrifuge shaking table tests, even after the appearance of slip lines. In the centrifuge tilting table tests, the development of slip lines proceeded rapidly, because the pseudo static horizontal force kept acting on the GRSWs. Thus, the GRSWs failed immediately after the formation of the slip lines. On the other hand, in the centrifuge shaking table tests, the slip lines were generated gradually with the shaking, because the seismic horizontal force varied instantaneously and the models were not always subjected to the peak accelerations. In addition, the reinforcing effect of the geogrids could be effectively obtained with the increase in the gradual deformation of the reinforced area in the centrifuge shaking table tests, and the GRSWs were able to reach a new stable state after each shaking step. Therefore, the GRSWs in the centrifuge shaking table tests were able to sustain sufficient stability,
even though the deformations were larger than those in the centrifuge tilting table tests, as shown in Figs. 9 and 13.

As mentioned earlier, the model geogrids used in this study were modeled well on tensile stiffness and pullout resistance. The tensile strength of the model geogrids might be higher than those of practical geogrids used for GRSWs with a height of 7.5 m. If an ideal geogrid with lower stiffness were used in the centrifuge model tests, it is thought that similar deformation and failure behaviour would be observed in the test series. This is because the strain at failure is generally larger than 10\%, and the backfill will fail before the rupture of the geogrids.

CRITICAL INCLINATION OF FACING PANELS OF GRSWS

As mentioned above, the formation of slip lines is considered to be one of the most important factors which can indicate the degree of damage to GRSWs. It is very important to deduce the formation of a slip line in the backfill in order to evaluate the extent of damage to the GRSWs after earthquakes. The centrifuge test results conducted in this study clearly indicate that slip lines may appear due to the failure of the backfill material when the inclination of the facing panels of the GRSWs reaches a particular value. Such an inclination is defined as the critical facing panel inclination, $\theta_{cr}$. Here, the relationship between the inclination of the facing panels and the extent of damage to the backfill is discussed in order to relate $\theta_{cr}$ and the failure of the backfill. Bransby and Milligan (1975) proposed a simple method to evaluate the $\gamma_{max}$ occurring in the backfill behind a sheet pile wall using the facing panel inclination of the wall with the following equation:

$$\gamma_{max} = 2\theta_w \sec \nu$$  \hspace{1cm} (3)

where $\gamma_{max}$ is the maximum shear strain in the backfill, $\theta_w$ is the facing panel inclination of the wall and $\nu$ is the dilatancy angle of the backfill material. This relation was derived from a simple plastic theory (Bransby and Milligan, 1975). Furthermore, they confirmed that the proposed equation could provide a good agreement with the test results conducted by Bransby (1972). This equation is very useful for determining the $\gamma_{max}$ occurring in the backfill, because the facing panel inclination and the properties of the backfill material are the only information required. However, it is not a daily practice to determine the dilatancy angle of the backfill material from element tests. Figure 19 shows the relationship between the facing panel inclination and the maximum shear strain calculated by the proposed equation with some possible dilatancy angles. The results show that the effect of the dilatancy angle on the calculated maximum shear strain is small enough to be considered negligible. Thus, the proposed equation can be simplified as follows, assuming that the dilatancy angle is zero:

$$\gamma_{max} = 2\theta_w$$  \hspace{1cm} (4)
Using this equation, the maximum shear strain induced in the backfill of GRSWs may be approximately determined from the inclination of the facing panels.

In both the centrifuge tilting and the shaking table tests, the slip lines appeared when the facing panel inclination reached about 3.0° in the case of CS-T, CS2-T and CS2-S5. At that time, the maximum shear strain occurring in the backfill was estimated to be about 6.0° using Eq. (4). Figure 20 shows the relationships between the deviator stress and the maximum shear strain for Toyoura sand and Silica sand Nos. 5 and 3. They were obtained from drained triaxial compression tests at the confining pressure of 98 kPa, which is almost the same pressure as the vertical pressure in the lower part of the GRSWs used in this study. As shown in this figure, Toyoura sand and Silica sand No. 5 show their peak values at almost the same maximum shear strain. CS-T also showed the slip line at almost the same facing panel inclination, which had the same backfill material but a different geogrids from those of CS2-T. On the other hand, the GRSW of CS2-S3 did not show a clear slip line, as silica sand No. 3 did not show a clear peak in its stress-strain relationship, as shown in Fig. 20.

As shown in Eq. (5), $\theta_c$ can be determined only with the properties of the backfill. Test results confirmed the fact that slip lines appeared at almost the same inclination of the facing panel in cases of CS-T and CS2-T, which had the same backfill but different geogrids. This is because the formation of a slip line is attributed to the failure of the backfill. It is noted that the seismic stability of GRSWs are affected by the properties of the geogrids before the formation of a slip line, and the deformation of GRSWs can be effectively confirmed using geogrids with high tensile strength and a large pullout resistance.

The extent of damage to geogrid reinforced soil walls subjected to earthquakes can be evaluated by deducing the formation of a slip line in the backfill with Eq. (5). In addition, it is possible to use $\theta_c$ as the limit value in performance based designs.

**LOCATION OF SLIP LINE IN GRSWS**

The formation of slip lines in GRSWs can be evaluated with the proposed equation and the facing inclination of the GRSWs if the stress-strain relationship of the backfill material is known. Furthermore, if the location of the slip lines can be obtained, this information will be very useful for evaluating the residual stability of the damaged GRSWs. It can produce quite useful information for quick assessments of the necessity for repair or reconstruction. Therefore, a method for evaluating the locations of slip lines is discussed in the following.

**Two Wedge Analysis**

The Two Wedge method is widely used for evaluating the seismic stability of GRSWs (e.g., Cai and Bathurst, 1996; Ling et al., 1997), which gives the horizontal seismic coefficients at failure and the locations of the slip lines. Since the GRSWs showed the Two Wedge type of failure sections in the centrifuge tests, the Two Wedge method is applicable to seismic stability analyses of GRSWs.

Figures 21(a) and (b) show the failure sections of GRSWs assumed in the Two Wedge analysis and the force equilibrium between Wedge A and Wedge B, respectively. In the analysis, the minimum factor of safety against the sliding of Wedge B is determined by changing
\[ \Sigma F = k_h W_B \cos \omega_B + (1 - k_v) W_B \sin \omega_B + P_{AE} \cos (\phi_w - \omega_B) \]

where \( k_h \) and \( k_v \) are the horizontal and the vertical seismic coefficient, \( W_B \) is the weight of the Wedge B, \( P_{AE} \) is the dynamic active earth force acting on Wedge B and \( \phi_w \) is the friction angle between Wedge A and Wedge B. The dynamic active earth pressure is calculated using the Mononobe-Okabe earth pressure theory (Mononobe, 1924), and \( \phi_w \) is defined as being equal to \( \phi^* \) in this analysis. The resistance force, \( \Sigma R \), contains the shear resistance of the backfill soil and the pullout resistance, as follows:

\[ \Sigma R = (-k_h W_B \sin \omega_B + (1 - k_v) W_B \cos \omega_B + P_{AE} \sin (\delta - \omega_B) + R_p \sin \omega_B) \tan \phi + R_P \cos \omega_B + c_l B. \]

where \( R_p \) is the pullout resistance, \( c \) is the cohesion of the backfill material and \( h_b \) is the length of the slip line of Wedge B. Accordingly, a factor of safety against the sliding of Wedge B, \( F_s \), can be expressed as follows:

\[ F_s = \frac{\Sigma R}{\Sigma F}. \]

**Results and Discussion**

Figure 22 shows the relationship between the factors of safety against sliding and the horizontal seismic coefficient at failure. In this analysis, peak pullout resistances were used, as listed in Table 3. The horizontal seismic coefficients at failure, obtained in the centrifuge tilting table tests, are also indicated in the figure. It is clear that Two Wedge analyses, with the peak pullout resistances, produce too high of a safety factor. Figures 23(a)-(d) show the calculated failure sections. There are very deep calculated slip lines behind the reinforced area, although the slip lines in the reinforced area are almost the same as those in the test results. In the Mononobe-Okabe theory, the inclination of a slip line behind the wall increases with the seismic coefficient. An unrealistically large sliding block was observed behind the reinforced area, when the reinforced soil walls did not collapse against the very high seismic coefficient in the calculation with the peak pullout resistance. It is probably because excessively large pullout resistances were used for the calculations. As shown in Fig. 24, a large pullout displacement is necessary to obtain the peak pullout resistance for all geogrids. For example, a pullout displacement of about 6 mm is necessary for obtaining the peak pullout resistance in case CS2-T. Such pullout displacement is too large for the model GRSWs, because the horizontal displacement at the lower facing panel could be estimated to be about 1 mm when the GRSW showed clear slip lines and failed in the centrifuge tilting table tests. That is, it is considered that the pullout resistance at a pullout displacement of about 1 mm was mobilized at the failure of the GRSW as a whole, as indicated in Fig. 24. Then, the mobilized pullout resistances at failure were evaluated properly based on the results of the pullout test results and Two Wedge analyses were carried out again. The pullout parameters used for this analysis are summarized in Table 5.

Figure 25 shows the relationship between the horizontal seismic coefficients and the factors of safety obtained from the Two Wedge analyses with the mobilized pullout resistances. It is clear that the mobilized pullout resistances were able to yield give reasonably good results. These results reveal that the Two Wedge method can adequately simulate the force equilibrium in GRSWs subjected to pseudo static loading, which is used in sease-
Fig. 23. Failure sections obtained from Two Wedge analysis with peak pullout resistance

Fig. 24. Peak and mobilized pullout resistance

Table 5. Pullout characteristics used for Two-Wedge analysis with mobilized pullout resistance

<table>
<thead>
<tr>
<th>Geogrid</th>
<th>CS</th>
<th>CS2</th>
<th>CS2</th>
<th>CS2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Back fill soil</td>
<td>Toyoura</td>
<td>Silica 3</td>
<td>Silica 5</td>
<td></td>
</tr>
<tr>
<td>Internal friction angle:</td>
<td>(\phi') (deg.)</td>
<td>41.0</td>
<td>46.0</td>
<td>51.0</td>
</tr>
<tr>
<td>Apparent pullout cohesion:</td>
<td>(c_p) (kPa)</td>
<td>0.221</td>
<td>15.2</td>
<td>16.6</td>
</tr>
<tr>
<td>Pullout friction angle:</td>
<td>(\phi_p) (deg.)</td>
<td>11.0</td>
<td>13.0</td>
<td>17.7</td>
</tr>
<tr>
<td>Pullout strength ratio:</td>
<td>(\tan \phi_p / \tan \phi')</td>
<td>0.224</td>
<td>0.265</td>
<td>0.309</td>
</tr>
</tbody>
</table>

Fig. 25. Relationship between safety factor against sliding and horizontal seismic coefficients at failure with mobilized resistance

As mentioned before, in another series of tests, a GRSW with extremely soft geogrids (Izawa and Kuwano, 2010), whose tangential and secant tensile stiffness was about 300 kN/m and 100 kN/m in prototype, respectively, failed when the backfill failed with slip lines, and it was far before the deformation with which a peak pullout resistance was mobilized.

CONCLUSIONS

This paper has discussed how to evaluate the extent of damage to geogrid reinforced soil walls (GRSWs) subjected to earthquakes. For this purpose, centrifuge tilting and shaking table tests were conducted, with special focus on the effects of the tensile stiffness of the geogrids, the pullout characteristics and the particle size of the backfill material. As a result, the following conclusions were obtained.

(1) GRSWs subjected to pseudo static horizontal loading show large shear deformation in the lower part of the reinforced area. Such deformation can be influenced by the tensile stiffness of the geogrids and the deformation modulus of the backfill material. That is, large seismic stability can be obtained by using stiffer geogrids and backfill material. Due to such deformation, clear slip lines appear in and behind the rein-
which was derived from a simple plastic theory. When the maximum shear strain in the backfill, calculated by the equation, reaches the maximum shear strain at the peak state, $\gamma_{\text{peak}}$, the backfill material fails and slip lines appear simultaneously, which develops with deformation due to earthquakes. Therefore, the critical facing panel inclination of GRSWs, $\theta_{\text{cr}}$, can be given as follows:

$$\theta_{\text{cr}} = \frac{1}{2} \gamma_{\text{peak}}.$$

Using this equation, the formation of a slip line in GRSWs can be evaluated with only the facing panel inclination of the GRSW. As a slip line appears due to the failure of the backfill, the formation of a slip line can be evaluated with only the properties of the backfill.

(4) The location of a slip line can be obtained by the Two Wedge method. In the analysis, however, the mobilized pullout resistance should be used instead of the peak pullout resistance. The former is the pullout resistance at the displacement inducing a failure of the backfill; it is achieved at a much smaller displacement than the latter. The results obtained from the Two Wedge analysis help to evaluate the remaining stability of GRSWs after an earthquake.

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**NOTATIONS**

- $k_h$: Horizontal seismic coefficient
- $k_v$: Vertical seismic coefficient
- $\eta$: Tilting angle (°)
- $\psi$: Dilatancy angle (°)
- $\phi'$: Internal friction angle of soil (°)
- $\tau_p$: Pullout resistance (kPa)
- $\phi_p$: Pullout internal friction angle (°)
- $c_p$: Pullout apparent cohesion (kPa)
- $\theta_w$: Surface inclination of the wall (%)
- $\theta_{cr}$: Critical surface inclination (%)
- $\gamma_{\text{max}}$: Maximum shear strain
- $\gamma_{\text{peak}}$: Maximum shear strain at peak state of backfill

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