Use of Geosynthetics to Improve Seismic Performance of Earth Structures

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ABSTRACT After reviewing seismic performance of earth structures based on case histories in Japan and relevant model test results, advantages of using geosynthetics in improving their seismic performance are demonstrated. As one of successful applications, geosynthetics reinforced soil retaining walls are highlighted, focusing on several influential factors such as facing rigidity, arrangement and properties of reinforcements, and backfill and subsoil conditions. In addition, further applications of the reinforcement method using geosynthetics are introduced, which include combination with other reinforcement methods, application to bridge abutments and piers, and application to ballasted railway tracks.

INTRODUCTION

Table 1 summarizes the peak values of horizontal ground accelerations (PGAs) and velocities (PGVs) that were recorded during recent major earthquakes in Japan. After the 1995 Hyogoken-nanbu earthquake, the availability of strong motion data recorded near the epicenter was improved significantly. Therefore, some of them approached or exceeded 800 gals and/or 100 kines.

<table>
<thead>
<tr>
<th>Earthquake</th>
<th>Station</th>
<th>PGA (gal)</th>
<th>PGV (kine)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hyogoken-Nanbu (1995)</td>
<td>JMA Kobe (NS)</td>
<td>818</td>
<td>97</td>
</tr>
<tr>
<td></td>
<td>JR Takatori (EW)</td>
<td>645</td>
<td>136</td>
</tr>
<tr>
<td>Tokachi-Oki (2003)</td>
<td>K-net Hiroo (EW)</td>
<td>970</td>
<td>47</td>
</tr>
<tr>
<td>Niigataken-Chuetsu (2004)</td>
<td>JMA Kawaguchi</td>
<td>1676</td>
<td>146</td>
</tr>
<tr>
<td></td>
<td>(EW)</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Kik-net Ichinoseki-Nishi (EW)</td>
<td>1433</td>
<td>62</td>
</tr>
</tbody>
</table>

Fig.1 Distribution of earthquake epicenters with M>4.0 and depth<50 km recorded from 1990 to 2000 (modified after JMA, 2010).
On the other hand, earth structures, such as embankments as schematically shown in Figs. 2a&b, have been widely employed to construct highways, railways, river dikes and housing lots. In addition, in order to reduce the area to be occupied by the construction of embankments and thus the volume of fill material, retaining walls (RWs) have also been frequently adopted, as schematically shown in Figs. 2c&d.

If we convert the horizontal seismic inertia into pseudo-static force as schematically shown in Fig. 3a, the direction of apparent gravity will be inclined. Then, the driving moment to trigger the sliding failure along a potential failure plane will be increased, as shown in Fig. 3b. Under such circumstances, adding reinforcements in the embankment with their tensile forces mobilized effectively will increase the resisting moment, as shown in Fig. 3c.

**Fig.2** Schematic illustrations of embankments and retaining walls.

**Fig.3** Schematic illustrations on effects of horizontal inertia of embankment and tensile force mobilized in reinforcements.
In view of the above, by addressing the following questions in this paper, attempts are made to share Japanese experiences on the use of geosynthetic-reinforcement to improve seismic performance of earth structures:

Q1: How different are the seismic performances of earth structures with/without geosynthetic-reinforcement?
Q2: What are the influential factors in improving effectively the seismic performance using geosynthetics?
Q3: How can we extend the application of geosynthetic-reinforcement technologies to other structures?

In order to answer the above questions, the paper begins with a review of seismic performance of earth structures in Japan. Next, influential factors in improving seismic performance of retaining walls using geosynthetics are discussed. Some of further applications of geosynthetic-reinforcement are briefly reviewed as well, which are followed by conclusions.

SEISMIC PERFORMANCE OF EARTH STRUCTURES

In order to answer the question 1 raised in INTRODUCTION, the following two sub-topics are reviewed in this chapter, while updating the summary made by Koseki et al. (2007):

- How earth structures with/without geosynthetic-reinforcement behaved in case histories, and
- model tests?

Case Histories in Japan

Herein, case histories from the following five recent major earthquakes in Japan are reviewed:
- 1995 Hyogoken-nanbu (Kobe) earthquake
- 2004 Niigataken-chuetsu earthquake
- 2007 Noto-hanto earthquake
- 2007 Niigataken-chuetsu-oki earthquake
- 2008 Iwate-Miyagi-nairiku earthquake

1995 Hyogoken-nanbu (Kobe) earthquake

Figure 4 shows damage to RWs without reinforcement that were located in severely shaken area by the January 17, 1995 Hyogoken-nanbu earthquake. Conventional type RWs without foundation, such as cantilever, gravity and leaning-type ones, suffered overall tilting and/or failure of the wall body. Most of them had to be removed and reconstructed after the earthquake.
In contrast to the above, one geosynthetic-reinforced soil (GRS) RW with full-height rigid facing, which was also located in the severely shaken area, survived with minor residual lateral displacements of about 10 to 20 cm that are measured relative to the neighboring culvert box structure (Fig. 5). The standard procedures for staged construction of this type of GRS RWs with a full-height rigid facing is illustrated in Fig. 6.

On the other side of the culvert structure, as shown in Fig. 7, a cantilever-type RW with bored-pile foundation suffered similar amounts of residual lateral displacement, suggesting that this wall and the previous GRS RW without foundation exhibited almost the same seismic resistance.


**2004 Niigataken-chuetsu earthquake earthquake**

Figure 8 shows damage to highway RW and railway embankment by the October 23, 2004 Niigataken-chuetsu earthquake (Tatsuoka et al. 2006, Koseki et al., 2006a). Although the original structures were without reinforcement, the highway was reconstructed using GRS RW with segmental facing panels made of pre-cast concrete (Fig. 9a), while the railway on the down slope side was reconstructed using GRS RW with a full-height rigid facing and rock bolts (Fig. 9b). Such decisions were made considering ground conditions, construction time and available backfill material, while adopting the same concept that the reconstructed earth structures shall exhibit improved seismic performance. Numerical verification of such improvement was made by Shinoda et al. (2009) on the railway embankment.
Fig. 8  Failure of highway gravity-type RW and railway embankment at Tenno caused by 2004 Niigataken-chuetsu earthquake (Koseki et al., 2006a).

Fig. 9  Reconstruction of failed a) highway RW (Koseki et al., 2006a) and b) railway embankment (Morishima et al., 2005) at Tenno using GRS RWs.

Figures 10 and 11 show the collapse of an embankment for newly-developed housing estates and its reconstruction using GRS RW with segmental facing panels made of metal mesh.

These panels enable the vegetation to cover the facing (see Fig. 16a for typical outlook with vegetation).

2007 Noto-hanto earthquake Figure 12 shows damage to an embankment for Noto toll road by March 25, 2007 Noto-hanto earthquake. The embankment had been constructed by filling a valley. In this case, the fill material was weathered tuff, which flowed down the valley for a distance exceeding 100 meters. Note also that, based on the survey conducted after the earthquake, the ground water level was found within the fill.

As schematically shown in Fig. 13, the collapsed embankment was reconstructed using GRS RW, while ensuring the drainage of ground and surface water. The waste soil that had originally been a part of the collapsed embankment was re-used after lime-treatment for the construction of the upper fill (Ishikawa Pref., 2007).
Fig. 12  Failure of an embankment of Noto toll road (at site No. 32) caused by 2007 Noto-hanto earthquake.

*Upper unreinforced part: re-use of collapsed fill with lime-treatment
Lower reinforced part: use of borrow sand

Fig. 13  a) Schematic illustration on reconstruction of failed embankments for Noto toll road (Ishikawa Pref., 2007), and b) typical reconstruction work using GRS RW (at site No. 9).
On the other hand, as typically shown in Fig. 14, the embankments of Anamizu road that connects to the north end of the Noto toll road could survive the earthquake with minor damage. Such good performance may be attributed to the use of lime-treatment for the fill material, while ensuring the drainage of ground water by installing non-woven geosynthetic sheets.

Fig. 14 Minor damage to embankment for Anamizu road caused by 2007 Noto-hanto earthquake.

2007 Niigataken-chuetsu-oki earthquake Figure 15 shows damage to a RW for a municipal road constructed on a slope by the July 16, 2007 Niigataken-chuetsu-oki earthquake.

In contrast, a GRS RW with segmental facing panels made of metal mesh that was located at the foot of the slope could survive the earthquake as shown in Fig. 16.

Fig. 15 Failure of RW for municipal road at Agewa caused by 2007 Niigataken-chuetsu-oki earthquake.

Fig. 16 GRS RW with at Agewa with segmental facing panels made of metal mesh.

2008 Iwate-Miyagi-nairiku earthquake Figure 17 shows an anchored cut-slope at a dam construction site. The anchors with a length of 20 to 40 meters had been installed as a countermeasure against landslide movement. By the June 14, 2008 Iwate-Miyagi-nairiku earthquake, most of the tendons were broken at their free sections, and the upper parts of the broken tendons were ejected.

Fig. 17 Failure of anchor tendons for cut-slope at construction site of Isawa dam caused by 2008 Iwate-Miyagi-nairiku earthquake.
On the other hand, a GRS RW with segmental facing panels made of metal mesh that was located on the other side of the slope survived the earthquake with no damage as shown in Fig. 18.

Model Tests

Herein, focusing on RWs, results from relevant model tests on their different seismic performances with/without geosynthetic-reinforcement are reviewed.

Test conditions and procedures A series of relatively small-scale 1-g model shaking tests was conducted on six different types of retaining walls resting on level ground as shown in Fig. 19. The wall models were about 50 cm high and the subsoil and backfill were modeled by very dense dry sand layers. They were subjected to several sequential horizontal excitations as typically shown in Fig. 20 in 0.1 g increments. Refer to Watanabe et al. (2003) for the detailed test conditions.

Fig. 18 GRS RW with segmental facing panels made of metal mesh at construction site of Isawa dam (by courtesy of Mr. S. Hamaya).

Fig. 19 RW models on level ground (Watanabe et al. 2003).

Fig. 20 Typical excitation time history (Watanabe et al. 2003).
**Test results** Figure 21 compares the cumulative horizontal displacements near the top of each RW model. The seismic coefficient plotted in the horizontal axis is defined as the peak base acceleration during each shaking step that is normalized with the gravitational acceleration. Up to seismic coefficient of about 0.35, no significant difference could be observed. However, under higher seismic loads, the residual wall displacements accumulated rapidly with the conventional RWs, i.e., cantilever, gravity and leaning-type ones. In contrast, the GRS RWs with a full-height rigid facing exhibited more ductile behavior, in particular with the ones having extended reinforcements (types 2&3, Figs. 19e&f).

The reason for the less ductile behavior of the conventional RWs can be understood from Fig. 22. The subgrade reaction at the toe of base footing of the gravity-type wall increased sharply with the accumulation of wall top displacement. It suddenly decreased, however, after showing a peak state, suggesting a local failure due to loss of bearing capacity. On the other hand, the subgrade reaction at the heel of the base footing decreased in the beginning, followed by a slight increase with the occurrence of the local failure at the toe.

In case of GRS RWs, as shown in Fig. 23, the tensile forces in the reinforcements measured at three different heights increased with the accumulation of the wall top displacement. Such a response of GRS RWs is the key feature for their good performance under high seismic loads. It should be noted that the mobilization of tensile load was concentrated to the uppermost long reinforcement for the type 2 GRS RW, which could effectively resist against the overturning of the facing. Due attentions should be paid on such concentration of tensile force.
It should be noted that, as mentioned above, the RW models with/without reinforcement exhibited similar wall displacements up to around 5 mm during the shaking steps at relatively low excitation levels (Fig. 21). This behavior is possibly affected by the shear deformation of subsoil layer. When such deformation occurs, as schematically illustrated in Fig. 24, the RW would suffer residual horizontal displacement without any slippage at the interface between the base of the RW and the underlying subsoil layer. Under the same subsoil conditions, as were the cases with the present model tests, the amount of such residual displacement would not depend largely on the difference in the RW types.

Fig. 24. Schematic illustration of wall displacement induced by shear deformation of subsoil layer.

Note also that, in case of GRS RWs, not only the subsoil deformation but also the shear deformation of reinforced backfill was observed in model tests as typically shown in Fig. 25. In evaluating the residual displacements of GRS RWs, such effects of shear deformation of reinforced backfill should be considered properly. However, in many of the relevant design guidelines, the reinforced backfill has been modeled as a rigid body that would not undergo any shear deformation (Koseki et al., 2006b).

INFLUENTIAL FACTORS IN IMPROVING SEISMIC PERFORMANCE OF RETAINING WALLS USING GEOSYNTHETICS

In order to answer the question 2 raised in INTRODUCTION, the following four sub-topics on the seismic performance of GRS RWs are reviewed in this chapter:

- How their seismic performance is affected by facing rigidity,
- arrangement of reinforcement,
- properties of reinforcement, and
- backfill and subsoil conditions?

Facing Rigidity

Tatsuoka (1993) investigated and discussed in detail the effects of facing rigidity on the stability of GRS RWs, as summarized briefly below.

a) As the facing becomes more rigid, the earth pressure acting on the back face of facing increases. This large earth pressure confines the backfill soil immediately behind the facing, which decreases the deformation and increases the ultimate stability of the wall.

b) A large degree of flexibility is not necessarily a preferable property for completed GRS RWs, although this property is required to accommodate possible large deformation of the subsoil so that a deep foundation becomes unnecessary.

The above summary a) suggests also that the geosynthetic-reinforcement technique is not a method to reduce earth pressure exerted from the backfill soil. Rather, this technique takes advantage of the tensile force mobilized in reinforcements to resist against the earth...
pressure (and inertia force of facing in case of earthquakes).

Regarding the above summary b), one of the possible compromises are that walls are made as much as flexible during construction, while they are made stiff enough before they are open to service. The staged construction procedures to cast-in-place the full-height rigid facing at the last stage as shown in Fig. 6 enable us to control the flexibility in such a manner, while they require in general longer construction period than GRS RWs with pre-cast segmental types of facing.

It should be noted that, in case of GRS RWs with segmental types of facing, local instability of facing due to failure at the connection between the facing and the reinforcement may lead easily to overall failure. Such a failure mode was observed in the 1999 Chi-chi earthquake in Taiwan as typically shown in Fig. 26a.

Note also that, in this particular case shown in Fig. 26, the vertical spacing of reinforcements was 80 cm, which exceeded the value recommended by relevant design guidelines. At the time of the earthquake, therefore, insufficient number of reinforcements may have been subjected to excessive tensile force, causing local rupture at their connection with the facing, and/or the overall facing rigidity may have been too small to resist against the earthquake loads, causing excessive deformation of the stacked faceings and pull-out of connecting pins, as schematically illustrated in Fig. 26b.

On the other hand, in case of GRS RWs with full-height rigid facing, even if local rupture or failure takes place, the tensile force mobilized in the reinforcements would be re-distributed more easily, since the rigid facing is supported simultaneously by many layers of reinforcements. This feature would enhance their redundancy against overall failure.

GRS RWs with full-height rigid facing that were constructed in Japan following the procedures shown in Fig. 6 have performed well under not only working loads but also large earthquake loads (e.g., Fig. 5). Therefore, this particular type of GRS RWs has been adopted for constructing important permanent earth structures to support such as bullet train tracks and highways (Tamura, 2006). As summarized in Fig. 27, their application in terms of total wall length exceeded 120 km as of June, 2010.

Hereafter, focusing on GRS RWs with full-height rigid facing, effects of other influential factors are discussed.
Arrangement of Reinforcement

In the model tests presented previously (Fig. 19), three types of reinforcement arrangement were employed:
- R1 or type 1 having relatively short reinforcements with equal length,
- R2 or type 2 having partially extended reinforcements, and
- R3 or type 3 having relatively long reinforcements with equal length.

Among the above three types, as can be seen from Fig. 21, the type 3 wall exhibited the smallest amounts of residual wall top displacement. In addition, though the total length of reinforcement of the type 2 wall was about 80 % as large as that of the type 3 wall, their seismic performances in terms of the wall top displacement were similar to each other. Such good performance of the type 2 wall confirms that partial extension of upper reinforcements (preferably, connection with another wall on the opposite side as illustrated in Fig. 27) improves the seismic stability significantly, since the upper reinforcements can resist more effectively against the overturning mode of failure.

It should be noted that, as shown in Fig. 25, the extended uppermost reinforcement in the type 2 wall prevented full formation of a failure plane (marked as A in Fig. 25) in the backfill as well, which passed through the end of the other extended reinforcement at the middle height.

In turn, as mentioned previously on Fig. 24, the tensile force mobilized in the reinforcements concentrated into the extended uppermost reinforcement of the type 2 wall, implying that this reinforcement was the key to exhibit the above good performance.

Note also that, with the type 2 and 3 walls, the calculated values of critical seismic coefficient to induce a factor of safety equal to unity in pseudo-static limit-equilibrium stability analysis against overturning failure were different from each other (0.55 and 0.70, respectively). Refer to Watanabe et al. (2003) for the detailed conditions of calculation. In spite of such difference, the two walls exhibited similar seismic performances. This is possibly affected by the shear deformation of reinforced backfill (Fig. 25), which is not considered in evaluating the above critical seismic coefficients.

Properties of Reinforcement

As an extension of the model tests presented previously (Fig. 19), another series of 1-g model shaking tests was conducted by Nakajima et al. (2007a), where two kinds of geosynthetic-reinforcement models as shown in Fig. 28 were employed.

As summarized in Table 2, their tensile stiffness per single strip evaluated in direct tension tests was higher with the PB (phosphor bronze) model. On the other hand, their tensile stiffness per unit width of the grid was higher with the PE (polyester) model, since this model consisted of much larger number of strips than the PB model.

In addition, as shown also in Table 2, their ultimate pull-out resistance per unit width of the grid was higher with the PE model, due possibly to its mesh size (= 3*3 mm) that could confine approximately 10 particles of the sand material (with a mean diameter of 0.23 mm) that was
used to prepare the backfill in the model tests. In contrast, the PB model had a much larger mesh size (= 50*95 mm), and it exhibited larger pull-out resistance per unit width of the grid at small levels of pull-out displacement (up to about 1 mm, Fig. 30), due possibly to the effects of sand particles that were pasted on the surface of the reinforcement to mobilize the frictional resistance.

Despite the above differences in the reinforcement properties, the observed cumulative tilting angles and base sliding displacements of the GRS RW models (type 2, Fig. 19e) employing the two types of reinforcements, respectively, were in general similar to each other, as shown in Fig. 29. Note that, in these model tests, the peak base acceleration was 0.9 g in the first shaking step, which was increased in 0.3 and 0.4 g increments in the second and third shaking steps, respectively. Further, the fourth shaking step was conducted by using the same base acceleration as in the third one.

**Fig.28**  Geosynthetic-reinforcement models (Nakajima et al., 2007a).

**TABLE 2**  Properties of model reinforcements (modified from Nakajima et al., 2007a)

<table>
<thead>
<tr>
<th>Property</th>
<th>Secant tensile stiffness at T=Tmax/2</th>
<th>Ultimate pull-out resistance, Tmax at σv=5 kPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit</td>
<td>per single strip (kN/ε/strip)</td>
<td>per unit width (kN/ε/m)</td>
</tr>
<tr>
<td></td>
<td>per unit width for buried length of 0.5 m (kN/m)</td>
<td></td>
</tr>
<tr>
<td>PB</td>
<td>3.5-5.7</td>
<td>41-66</td>
</tr>
<tr>
<td>PE</td>
<td>0.31-0.36</td>
<td>105-121</td>
</tr>
</tbody>
</table>
| * Strips in the air ruptured under this tensile force.

The influential factor to affect the tilting behavior of GRS RWs observed in the present model tests would be the pull-out resistance mobilized at small displacement levels. As mentioned above, it was larger with the PB model than with the PE model, though the ultimate resistance was vice versa (Fig. 30). The slight difference between the two models in terms of the cumulative tilting angles as can be seen in Fig. 29a may have been affected by such pull-out properties at small displacement levels.

On the other hand, since no rupture or pull-out failure of reinforcements was observed, these properties would not have affected the tilting behavior in the present model tests.
Fig. 30 Pull-out tests on model reinforcements (Nakajima et al., 2007a).

Regarding the base sliding of GRS RWs shown in Fig. 29b, no significant difference was observed between the two models. This is possibly because the resistance against base sliding under the model configurations employed for the present model tests is not largely affected by the reinforcement properties but predominantly affected by the backfill and subsoil conditions around their interfaces. For small displacement levels, in addition, one needs to recall the effects of shear deformation of subsoil layer as discussed previously (Fig. 24), which is also independent from the reinforcement properties.

**Backfill and Subsoil Conditions**

Not only the conditions of reinforcement arrangement and facing rigidity as discussed above, but also the conditions of backfill and subsoil affect the seismic performance of GRS RWs.

For example, backfill soils that are not well-compacted may not mobilize sufficient pull-out resistance of reinforcements, no matter how the reinforcements themselves are stiff and strong enough. In addition, poorly-compacted backfill soils may suffer excessive settlement during their service period prior to earthquakes, resulting into local failure of reinforcements at their connection with the facing. Once pull-out or local failure of reinforcements takes place, it would trigger overall instability of GRS RWs during earthquakes as well as under working load conditions.

It should be noted that, as compared to the backfilling work without reinforcement that is employed for conventional type RWs, the backfill soil with reinforcements can be compacted in a more effective manner, since the existence of reinforcement would confine the lateral deformation of the backfill soil during compaction work. Therefore, by taking such advantage, due attentions shall be paid in constructing GRS RWs to compact the backfill soil sufficiently.

The above confinement of the lateral deformation of the backfill soil would in turn mobilize initial tensile forces in the reinforcements. Such effective mobilization of tensile forces would contribute to reduce the displacement of the facing as well.

Note also that, by placing reinforcements at a specified vertical spacing (equal to 30 cm in case of GRS RWs with full-height rigid facing shown in Fig. 6), one may ensure the lifting height of the backfilling work to be equal to or smaller than this vertical spacing.
When considering the effects of shear deformation of subsoil layer as discussed previously (Fig. 24), one may understand easily that the subsoil conditions also affect the seismic performance of GRS RWs. The effects of subsoil conditions are further discussed herein, referring to the review by Koseki et al. (2007) on relevant model test results, while focusing as well on the measures to improve the seismic performance.

**Model tests on RWs constructed on sloped ground** As an extension of the model tests presented previously (Fig. 19), the other series of 1-g model shaking tests was conducted by Kato et al. (2002) on RWs resting on sloped ground as typically shown in Fig. 31. For some models, in order to increase their stability, large diameter nail models having a diameter of 4 cm were added to the backfill and subsoil and connected to the wall. Refer to Nakajima et al. (2007b) for the detailed analysis of tensile loads induced in the large diameter nail models.

![Figure 31] RW models with large diameter nails on sloped ground (Kato et al. 2002).

Figure 32 shows the relationship between the seismic coefficient and the cumulative horizontal displacements near the wall top. For comparison, the aforementioned test results on RW models of the same type constructed on level ground (Fig. 19) are plotted together. The seismic stability of the respective wall model on slope was much lower than the corresponding one on level ground.

It should be noted that, as shown in Fig. 33, in case of GRS RW constructed on slope, a full failure plane in the unreinforced backfill and the sloped subsoil was formed. After the formation of such full failure plane, as shown in Fig. 34, the mobilization of the tensile forces in the reinforcement was reduced, followed by rapid accumulation of the wall displacement (Fig. 32).

![Figure 32] Comparison of residual wall top displacements (Kato et al. 2002).

![Figure 33] Full failure plane in unreinforced backfill and sloped subsoil of GRS RW model (Kato et al. 2002).
On the other hand, as shown in Fig. 32, the leaning-type and GRS RWs with the large diameter nails exhibited substantially higher seismic stability than those without nails. They yielded very limited amount of residual wall displacements even at seismic coefficients exceeding 1.0.

**FURTHER APPLICATIONS OF GEOSYNTHETIC-REINFORCEMENT**

In order to answer the question 3 raised in INTRODUCTION, the following three sub-topics are reviewed in this chapter:

How geosynthetics are used for
- combination with other reinforcement methods,
- application to bridge abutments and piers, and
- application to ballasted railway tracks?

**Combination with Other Reinforcement Methods**

In the model tests shown in the previous section, GRS RWs on sloped ground exhibited substantially high seismic stability when they were further reinforced with large diameter nails (Fig. 31).

In addition, as shown in Fig. 35, the residual tilting angle of the facing of GRS RW model could be effectively reduced by installing a sheet pile at the foot of the facing and connecting it to the facing (Nakajima et al. 2006).

Further, not only soil reinforcement methods but also soil improvement methods have been combined with geosynthetic-reinforcement. For example, Izawa et al. (2009) reported successful development and practical application of a new method that combines geosynthetic-reinforcement with a fibre-mixed soil-cement wall. The benefits of a similar approach to increase the stability by a combination of geosynthetic-reinforcement and cement-treatment will be also described in the next section on bridge abutments.

The above combination of geosynthetic-reinforcement with other reinforcement or improvement methods has been adopted in Japanese practice as well. Herein, some relevant case histories on its application to reconstruction works of earth structures damaged by earthquakes are reviewed briefly.
Case histories in 2004 Niigataken-chuetsu earthquake

Figures 36 and 37 show the collapse of a railway embankment and its reconstruction involving a combination of GRS RW and earth anchors. The benefits of a similar approach to stabilize the base of these structures using soil nails have been confirmed in the model tests (Fig. 30).

Fig. 36 Failure of railway embankment at Tenno tunnel caused by 2004 Niigataken-chuetsu earthquake.

Fig. 37 Reconstruction of railway embankment at Tenno tunnel (Kitamoto et al. 2006).

Figure 38a shows the collapse of another railway embankment. It was reconstructed by re-using the collapsed fill material (Morishima et al., 2005). As illustrated in Fig. 38b, the fill material was improved by adding a cement-origin stabilizer at a mixing ratio of 150 kg/m³ for the upper fill or 105 kg/m³ for the lower fill. It was further reinforced with geogrid sheets that were placed at a vertical spacing of 1.5 m as secondary reinforcement. In order to ensure the drainage, a gravel mat was placed at the bottom of the embankment.

Fig. 38 Failure of railway embankment at Tsukanoyama caused by 2004 Niigataken-chuetsu earthquake and its reconstruction (Morishima et al. 2005).

Case histories in 2007 Noto-hanto earthquake

As shown in Fig. 13b, the collapsed highway embankments were reconstructed using GRS RWs, where the collapsed fill material was reused after lime-treatment for the construction of the upper fill (Ishikawa Pref., 2007).

Application to Bridge Abutments and Piers

In past major earthquakes, bridge abutments suffered from several types of damage, including settlement of their backfill soil (Fig. 39), extensive residual displacements of the wall body, and structural failure of the wall body (Fig. 40).
In order to improve the stability of retaining walls supporting bridge girders, Watanabe et al. (2002) and Aoki et al. (2003) conducted a series of 1-g model shaking tests on different types of bridge abutments. As shown in Fig. 41, the wall displacement could be reduced significantly by using cement-treated backfill that was reinforced with geogrids (Fig. 42) as compared to the ordinary type wall model using unreinforced and untreated backfill sand. The backfill settlement could be also reduced extensively.

On the other hand, Aoki et al. (2003) revealed as well that using cement-treated backfill without reinforcements is not enough to improve the seismic stability of bridge abutments up to sufficiently high levels.
As reported by Aoki et al. (2005), a similar type bridge abutment using cement-treated backfill gravel for the GRS RW with a full-height rigid facing (Fig. 43) has been implemented in practice to construct an abutment for new bullet train in Kyushu Island, Japan. By employing this system, the construction cost could be saved by 20 % as compared to the ordinary method.

Tatsuoka et al. (2009) and Aizawa et al. (2007) conducted another series of 1-g model shaking tests on bridge abutments. The models included integral types with or without reinforcements in the backfill (Fig. 44a), where the bridge girder was firmly connected to both of the walls. They were subjected to several sequential horizontal excitations with 20 cycles of sinusoidal waves at a frequency of 5 Hz. As shown in Fig. 44b, the seismic stability could be improved significantly by combining the integral bridge and the GRS RW systems.

It should be noted that, with the above GRS integral bridges, in addition to the rupture strength and pull-out resistance of reinforcements, the connection strength between the facing and the reinforcement was found to be one of the key issues to maintain the high seismic stability (Hirakawa et al., 2007).

Note also that, for bridge piers to support girders, pre-loaded and pre-stressed gravel backfill for GRS RW with a full-height rigid facing has been also implemented in practice for a railway in Kyushu Island, Japan (Fig. 45, Uchimura et al. 2003). Its high seismic stability has been confirmed through model shaking tests (Shinoda et al. 2003), and its applicability to bridge abutments has been verified as well (Aoki et al. 2003).
Application to Ballasted Railway Tracks

In order to reduce deformation of ballasted railway tracks during large earthquakes, a method to reinforce their shoulders using stacked geosynthetic bags that are filled with ballast material, as schematically shown in Fig. 46a, was developed by Kachi et al. (2010). The stacked bags were further reinforced with iron bars by driving them through the bags and embedding them to the base layer.

This method can be regarded as an extension of conventional soil bag methods, while it employs a mesh-type bag to mobilize better interlocking at the interface between adjacent bags. The opening of the mesh is about 25 mm, which is much larger than those of conventional soil bags. As schematically shown in Fig. 47, the tensile force mobilized in the bag would in turn enhance the bearing capacity of the bag by adding an apparent cohesion. Refer to Matsuoka and Liu (2003) for more detailed discussions on the advantages of soil bags.

It should be noted that, during large earthquakes, the direction of the major principal stress that is mobilized in the ballasts will rotate from the vertical direction. Therefore, referring to the pioneering work by Matsushima et al. (2008), the method was improved by stacking the bags in an inclined manner as shown in Fig. 46b. In order to resist against the overturning moment more effectively, some of the reinforcing iron bars were also inclined. In addition, in order to increase the overall stiffness of the stacked bags, adjacent bags were connected to each other by using a U-shaped iron bar.
After confirming better performance of the improved version by horizontal monotonic loading tests, a full-scale 1-g model shaking test was conducted as shown in Fig. 48. The model was subjected to a severe horizontal excitation as shown in Fig. 49. The maximum response in terms of horizontal displacements during the excitation is shown in Fig. 50. The horizontal displacement at the tie position (see Fig. 46b) was as small as 6 mm, which implies that the improved structure can perform well even under severe earthquake loads in preventing high-speed trains from derailment. Thus, it has been adopted for actual reinforcing works of existing ballasted tracks for Tokaido bullet train (or Tokaido Shinkansen) of Central Japan Railway Company.

Kachi et al. (2010) reported as well that the newly developed method has good workability, as compared to an alternative method using a pre-cast concrete block with a mass of about 200 kg, and thus requires heavy equipments. With the new method, on the other hand, each of the geosynthetic bags can be handled easily without heavy equipments, since they are simply filled with about 25 kg of ballast material and can be easily compacted into a specified dimension of 400*400*100 mm with a plate compacter.

CONCLUSIONS
The contents of the present paper on the use of geosynthetic-reinforcement to improve seismic performance of earth structures can be summarized as follows.

1) As compared to unreinforced earth structures, geosynthetic-reinforced soil retaining walls (GRS RWs) performed well during past large earthquakes in Japan. Their ductile behavior under large earthquake loads was also confirmed by relevant model tests. Thus, reinforced earth structures have been adopted for new construction of important permanent structures as well as their damage rehabilitation works.

2) The seismic performance of GRS RWs is affected by rigidity of facing, pull-out resistance and arrangement of reinforcements, and their connection strength. Due attentions shall be also paid on the effects of shear deformation of subsoil and reinforced backfill layers and formation of full failure plane in these layers.

3) Combination with other reinforcement or improvement methods, such as nailing, anchoring, sheet-piling and cement-treatment, enhances further the seismic performance of GRS RWs. By employing such combination,
collapsed soil materials can be re-used effectively.

4) Due to the above advantages, the application of geosynthetic-reinforcement is extending to wider areas. For example, it has been successfully applied to bridge abutments and ballasted railway tracks in practice.

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REFERENCES


