New Horizons in Earth Reinforcement – Otani, Miyata & Mukunoki (eds)

A new type integral bridge comprising of geosynthetic-reinforced soil walls

F. Tatsuoka, D. Hirakawa, M. Nojiri & H. Aizawa
Tokyo University of Science, Japan

M. Tateyama & K. Watanabe
Railway Technical Research Institute, Japan

ABSTRACT: A new type bridge combining an integral bridge and a pair of geosynthetic-reinforced soil (GRS) retaining walls having full-height rigid (FHR) facings, called the GRS integral bridge, is proposed. The geosynthetic reinforcement layers are connected to the facings (i.e., RC parapets) that are integrated with a girder. The GRS integral bridge is much more cost-effective in construction and long-term maintenance while having a higher seismic stability than conventional-type bridges having a girder via movable and fixed supports on either gravity-type abutments or GRS retaining walls and also than the conventional integral bridges. The GRS integral bridge alleviates several problems with the other types, mostly resulting from that the backfill is not reinforced and girder-supports are used, while taking advantage of their superior features.

1 INTRODUCTION

Despite its wide use until today over the world, the conventional gravity-type bridge abutment (Fig. 1) has a number of drawbacks, due mostly to that the backfill is not reinforced and therefore the abutment is a cantilever structure while a girder is placed on movable and fixed supports, as listed below:

1) As the abutment is a cantilever structure, piles are usually necessary to resist against stresses concentrated near the toe of the abutment base.
2) The RC abutment is not allowed to displace once constructed. After constructed, however, it is subjected to earth pressure and effects of the settlement and lateral flow in the subsoil via its effects on the piles associated with the backfill construction. To prevent such displacements of the abutment, it may become necessary to increase the number and size of piles.
3) The construction and long-term maintenance of girder-supports are generally costly.
4) The seismic stability of the unreinforced backfill as well as the abutment supporting the girder via a fixed-support is relatively low, as observed in many previous major earthquakes. Watanabe et al. (2002) and Aizawa et al. (2006) confirmed this point by model shaking table tests.
5) A bump is formed behind the abutment by residual deformation of the backfill due to its self-weight and traffic and seismic loads.

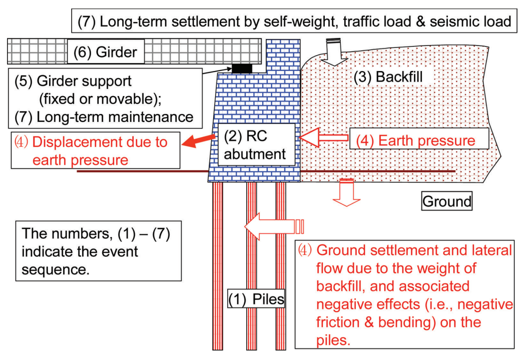

Figure 1. Conventional type bridge abutment (gravity type).

To develop bridge systems that are more cost-effective than the conventional bridge type while alleviating these problems described above, several solutions have been proposed as described in the next section.

2 PREVIOUS ATTEMPTS

2.1 *Improving the backfill*

One of the oldest attempts employed by the Japanese railways engineers is to construct a trapezoidal zone of well-compacted well-graded gravelly soil immediately behind the abutment (type *a*1 in Fig. 2). However,

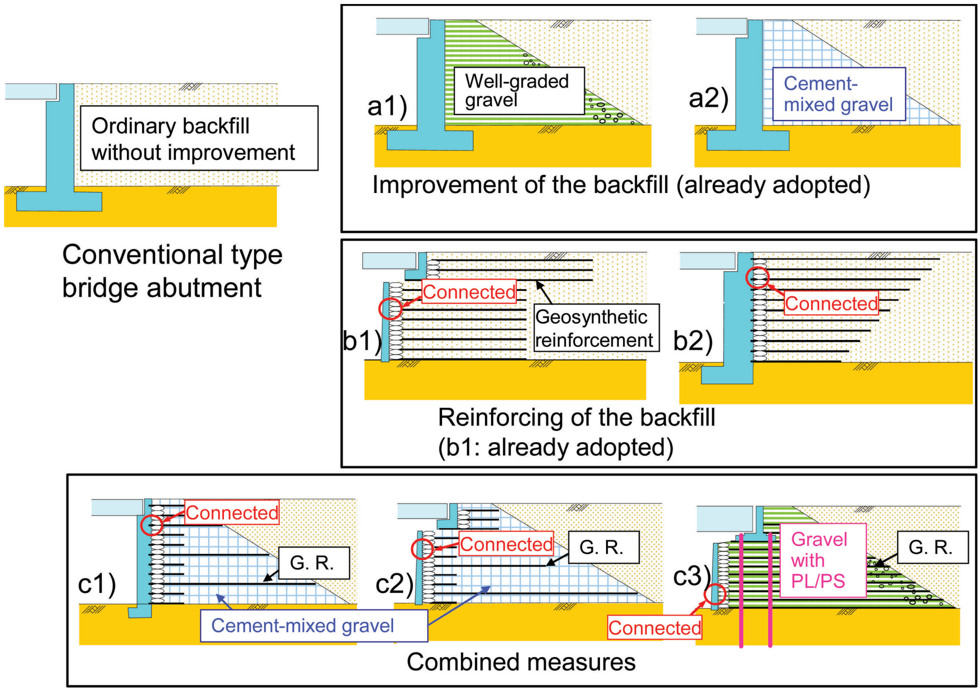

Figure 2. Conventional type versus proposed solutions of bridge (Tatsuoka, 2004; Tatsuoka et al., 2005).

the performance of this type of bridge during several previous earthquakes in Japan was not satisfactory (e.g., Tateyama et al., 2002). Watanabe et al. (2002) and Tatsuoka et al. (2005) confirmed the above by performing model shaking table tests.

They also showed that the seismic stability of another similar type constructing a trapezoidal zone of cement-mixed gravel (type *a*2, Fig. 2) is not sufficiently high either.

2.2 *Reinforcing the backfill*

Fig. 3 illustrates the staged construction of geosynthetic-reinforced soil (GRS) retaining wall (RW) with full-height rigid (FHR) facing. This is now one of the standardized RW construction technologies for railways in Japan while becoming popular also in other fields (such as highways). The main features of this technology are as follows:

1. The backfill is constructed with a help of gravel gabions placed at the shoulder of each soil layer.
2. Geosynthetic reinforcement layers are arranged with a vertical spacing of 30 cm. This small lift can facilitate a high compaction of the backfill.
3. After a geosynthetic RW is completed while sufficient compression of the backfill and settlement

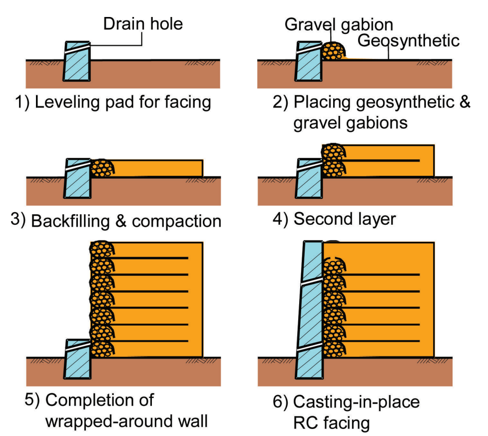

Figure 3. GRS RW with FHR facing (Tatsuoka et al., 1997).

of the supporting ground has taken place, a FHR facing is cast-in-place directly on the wrapped-around wall face ensuring a strong connection to the reinforced backfill so that:

a) negative interaction between the FHR facing and the compression of the backfill during

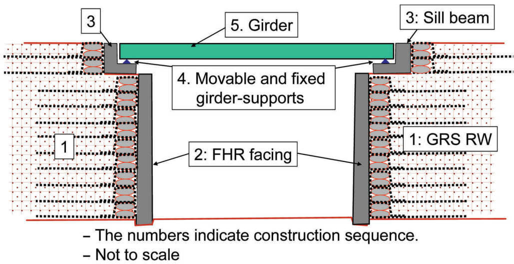

Figure 4. GRS RW bridge (type *b*1 in Fig. 2).

filling-up and compaction works can be avoided;

b) large compression of the supporting ground by the backfill construction can be accommodated ensuring the stability of wall;
c) the backfill immediately back of the wall face can be compacted dense with better mobilization of reinforcement tensile force; and
d) the alignment of completed wall face is easy.

Taking advantages of these features, a number of bridges comprising of a pair of GRS RWs with FHR facings that support a girder (type *b*1 in Fig. 2; Tatsuoka et al., 1997, 2005), which is herein called the GRS-RW bridge (Fig. 4), were constructed. Although this bridge type is structurally simpler and more cost-effective than the conventional type, it has the following limitations:

1. The girder cannot be very long due to low stiffness and potential large residual deformation of the backfill supporting the girder.
2. The construction and long-term maintenance of movable and fixed girder-supports is costly. This is the common problem with all of the bridge types presented in Fig. 2.
3. Despite that the dynamic stability of GRS RW with FHR facing is very high (e.g., Tatsuoka et al., 1998; Koseki et al., 2003), the dynamic stability of the sill beam on which a fixed girder-support is placed is not so (Aizawa et al., 2006; Hirakawa et al., 2007a). This is because the mass of the sill beam is much smaller than the inertia force of the girder while the anchorage capacity of the reinforcement layers connected to its back is small due to their shallow depths.

Type *b*2 (Fig. 2), placing a girder on the crest of the FHR facing, is dynamically more stable than type *b*1 (Watanabe et al., 2002; Tatsuoka et al., 2005). However, they also showed that the reinforced backfill behind the facing supporting the girder via a fixed-support would exhibit too large deformation when subjected to L2 design seismic load.

2.3 *Combining multiple-measures*

To substantially decrease long-term residual deformation of the backfill, it is very effective to vertically preload the reinforced backfill and then maintain some vertical prestress, which is typically about a half of the prestress, in the backfill during long-term service (i.e., the PL & PS technology). The above was validated by laboratory model tests (Shinoda et al., 2003a & b) and long-term performance of a prototype railway bridge pier (Uchimura et al., 2003). Moreover, Uchimura et al. (2003) and Tatsuoka et al. (2005) showed that the seismic stability of PL-PS reinforced bridge pier and abutment is very high. It is in particular the case if high prestress is maintained during dynamic loading and this can be ensured by using a ratchet mechanism as shown by model shaking table tests (Shinoda et al., 2003a & b). Type *c*3 in Fig. 2 consists of a PL-PS GRS RW with a ratchet system supporting a girder via a fixed-support. Its high seismic stability was validated by laboratory shaking table tests (i.e., Nakarai et al., 2002). Despite the above, any prototype bridge of this type has not been constructed, because possible long-term maintenance works of the ratchet system were not preferred by practicing engineers.

Types *c*1 and *c*2 were then proposed, which are combining types *b*2 and *b*1 with type *a*2. Type *c*1 was adopted by railway engineers and the first prototype was constructed for a new bullet train line in Kyushu (Tatsuoka, 2004; Tatsuoka et al., 2005). The conventional RC abutment (Fig. 1) supports the backfill with the earth pressure activated on its back. In comparison, with type *c*1 as well as type *b*2, the reinforced backfill zone laterally supports the RC parapet (i.e., facing) that is supporting a girder while without dynamic earth pressure activated on its back. Type *c*1 abutments are constructed by the staged procedure presented in Fig. 3.

3 CONVENTIONAL INTEGRAL BRIDGE

3.1 *State-of-the-art*

This type is very popular in the UK and the USA due mainly to high cost/performance by low construction and maintenance cost resulting from no use of girder-supports (Fig. 5). However, the backfill may exhibit large residual settlement by self-weight as well as traffic and seismic loads, while the seismic stability of both girder-parapet system and backfill is relatively low, as shown by Aizawa et al. (2006), Nojiri et al. (2006) and Hirakawa et al. (2007a) and also below. Moreover, as the girder is integrated with the parapets, seasonal thermal expansion and contraction of the girder results into cyclic lateral displacements at the top of the facings, which results in a gradual increase

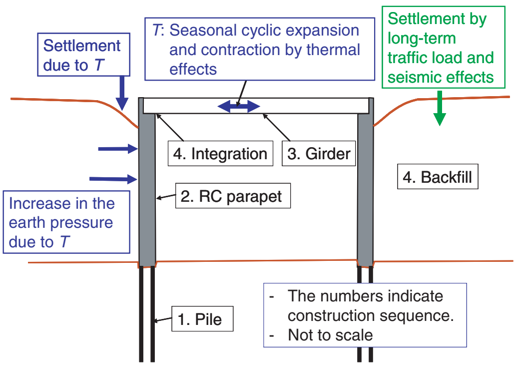

Figure 5. Integral bride and its inherent problems.

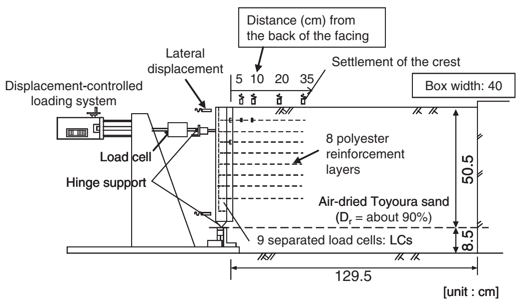

Figure 6. Static cyclic lateral loading test of the facing of RW model (Hirakawa et al., 2006, 2007b).

in the earth pressure and residual settlement in the backfill, as shown below.

3.2 *Effects of cyclic displacements of the facing*

Small-scaled model tests were performed in the laboratory (Fig. 6) to evaluate the detrimental effects of thermal cyclic displacements of the facing described above and also to examine whether this problem can be alleviated by reinforcing the backfill. The backfill was air-dried Toyoura sand produced by air-pluviation for the unreinforced backfill while by hand-tamping for the reinforced backfill. The reinforcement was a Polyester grid (strand diameter = 1 mm; spacing between the adjacent strands = 18 mm; covering ratio = 9.5%; and rupture tensile strength at an axial strain rate of 1.0%/min. = 19.6 kN/m). The FHR facing was cyclically displaced about the bottom hinge at a rotational displacement rate of 0.00053 degree/min.

Figure 7 summarizes the peak earth-pressure coefficients in the respective cycles, $K_{\text{peak}} = 2Q_{\text{peak}}/H^2\gamma$, where Q_{peak} is the peak total earth pressure per width in each cycle; H is the wall height (50.5 cm); and γ is the dry unit weight of the backfill (1.60 gf/cm^3), plotted against the ratio of the double amplitude of cyclic

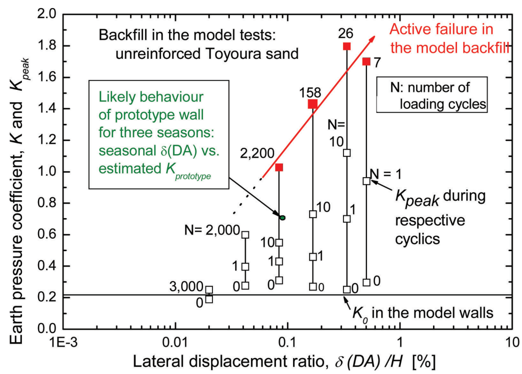

Figure 7. Peak earth pressure coefficients in the model tests and a field full-scale case (Hirakawa et al., 2007b).

displacement at the facing top to the facing height, $\delta(DA)/H$, at selected numbers of loading cycle, N. The facing top was allowed to move about 0.2 mm ($\delta/H = 0.04$ %) at the maximum toward the active direction associated with an increase in the earth pressure. The solid squares represent the cycles when the active failure plane appeared in the backfill. The earth pressure increases with an increase in $\delta(DA)/H$ and N. These test results are consistent with previous laboratory model tests (Ng et al., 1998; England et al., 2000) as well as the full-scale field behaviour for three seasons (i.e., $N = 3$). This earth pressure increase may result in structural damage to the facing and may push out the bottom of the facing. By reinforcing the backfill, this earth pressure increase does not reduce, but the facing is not structurally damaged and not pushed out at the bottom, as the FHR facing becomes a continuous beam supported by a number of reinforcement layers at a small spacing.

The other detrimental effect of cyclic displacement of the facing with unreinforced backfill is gradual but eventually large settlements in the backfill associated with the development of an active failure plane in the backfill (case NR in Fig. 8). The backfill settlement increases with an increase in the cyclic facing displacement, $\delta(DA)/H$. On the other hand, the backfill settlement becomes nearly null when the backfill is reinforced with reinforcement layers that are connected to the back of the facing (case R & C). Even slight heaving at the backfill crest takes place by dilatation of the backfill due to repeated passive movement of the facing. The benefits of reinforcing the backfill with reinforcement layers connected to the facing are as follow. Firstly, for the same thermal thrust from the girder, the displacements of the facing become smaller due to higher stiffness of the reinforced backfill. Secondly, for the same cyclic facing displacement, the residual settlement in the backfill decreases due to

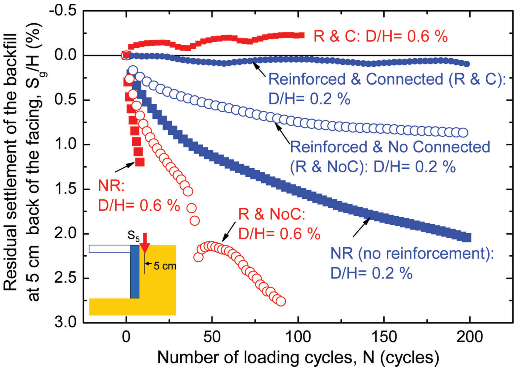

Figure 8. Residual settlement of backfill (when $\delta = 0$) by cyclic displacement of the facing and effects of reinforcing the backfill (modified from Hirakawa et al., 2006).

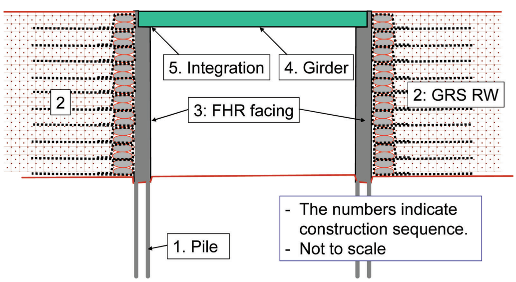

Figure 9. GRS integral bridge.

higher confining pressure in the backfill and membrane effects of reinforcement layers connected to the facing. It may also be seen from Fig. 8 that these positive effects of reinforcing the backfill become very small when the reinforcement layers are not connected to the facing (case R & NoC). This is because the deformation of the active zone cannot be effectively restrained by the reinforcement layers.

4 GRS INTEGRAL BRIDGE

4.1 *Features of IGS integral bridge*

A new bridge type (called the GRS integral bridge; Fig. 9), which is more cost-effective and more dynamically stable than the others described in this paper, is proposed. This type combines the GRS RW bridge (type b1, Fig. 2) and the integral bridge (Fig. 6), taking their advantages: i.e., stabilization of the backfill and non-soil structures by reinforcing the backfill (the GRS RW bridge) and simple and cost-effective non-soil structure without using girder-supports (the integral bridge), while alleviating their inherent problems. A GRS integral bridge may need a pile foundation to

Bridge type	Cost & period of construction	Maintenance cost	Seismic stability	Total
Conventional (gravity)	1 A, B	1 C, D	1 252 gal*	3
GRS RW	3	2 C	2 589 gal*	7
Integral	3	1 D, E	2 641 gal*	6
GRS Integral	3	3	3 1,048 gal*	9

(* Acceleration at failure in model shaking table test)

A = heavy abutment structure as a cantilever structure
B = piles are usually necessary
C = high cost for construction and long-term maintenance of girder-supports
D = bump due to settlement of backfill by self-weight, traffic load and seismic load
E = settlement of the backfill and structural damage to the facing by cyclic lateral displacements of facing due to seasonal thermal expansion and contraction of the girder

Figure 10. Features of four different bridge types.

support the girder, but a lighter one than the integral bridge may be sufficient, as needs for a pile foundation are usually much lower with GRS RWs. As seen from Fig. 8, the residual settlement of the backfill reinforced with reinforcement layers connected to the facing is very small. Moreover, a high seismic stability with small deformation and displacements can be expected because of integrated performance of the whole bridge system, as shown below.

Figure 10 compares the advantages and disadvantages in the three items listed in the top line of the four bridge types: i.e., conventional gravity type, GRS RW, integral and GRS integral. At the accelerations shown in the second column from the right, the respective bridge models collapsed in the shaking table tests, as described below. Letters *A* through *E* denote negative factors with the respective bridge types discussed in the above. Full points equal to three are assigned to each item, which are reduced one by one when these negative factors are relevant. So, the total full points are equal to nine, which are assigned only to the GRS integral bridge.

4.2 *Model tests*

Shaking table tests of the four bridge types listed in Fig. 10 were performed to validate a high-seismic stability of the GRS integral bridge (Aizawa et al., 2006; Hirakawa et al., 2007a). Fig. 11 shows the IGS integral bridge model. Assuming a length similitude ratio equal

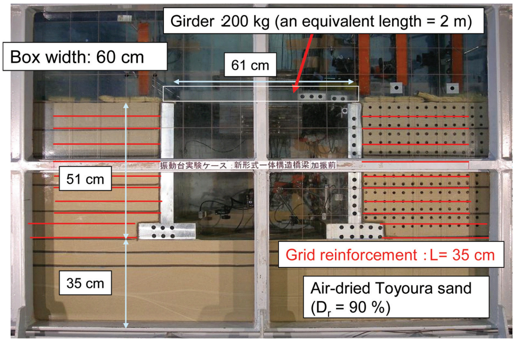

Figure 11. GRS integral bridge model (Aizawa et al., 2006).

to 1/10, the facings were 51 cm-high and the girder was 61 cm-long. By adding a mass of 200 kg at the center of the girder, the equivalent length became 2 m (i.e., 20 m in the assumed prototype). 20 sinusoidal waves with a frequency of 5 Hz was applied at the shaking table step by step while increasing the maximum acceleration α_{max} with an increment of 100 gal per step.

Figure 12a shows the backfill settlements at 5 cm back of the sill beam supporting the girder via a fixed support with the GRS RW type (Fig. 4) and back of the facing with the other three types. Fig. 12b shows the lateral displacements at the top and bottom of the facing. In Fig. 12b, d_t is the displacement of the sill beam with the GRS RW type. In Figs. 12a and b, with the gravity and GRS RW types, the displacements of the abutment or facing on the side supporting the girder via a fixed-support are presented. It can be readily seen that the GRS integral bridge, together with the backfill, is much more stable than the other types, while the conventional gravity type and its backfill is least stable. It may also be seen that the pushing out of the facing bottom is the major failure mode with the integral and GRS integral bridges.

5 CONCLUSIONS

A new type bridge structure, the GRS integral bridge, is proposed, which comprises of geosynthetic-reinforced backfill and an integral bridge. Its advantageous features, which are due mostly to no use of girder supports and reinforcing of the backfill, are: 1) a high cost-effectiveness in construction and long-term maintenance; 2) essentially zero settlement in the backfill and no structural damage to the facing by an increase in the earth pressure caused by thermal cyclic expansion and contraction of the girder; and 3) a very high seismic stability of both backfill and non-soil structural component (i.e., a pair of parapets and a girder) due to integrated dynamic performance of both components.

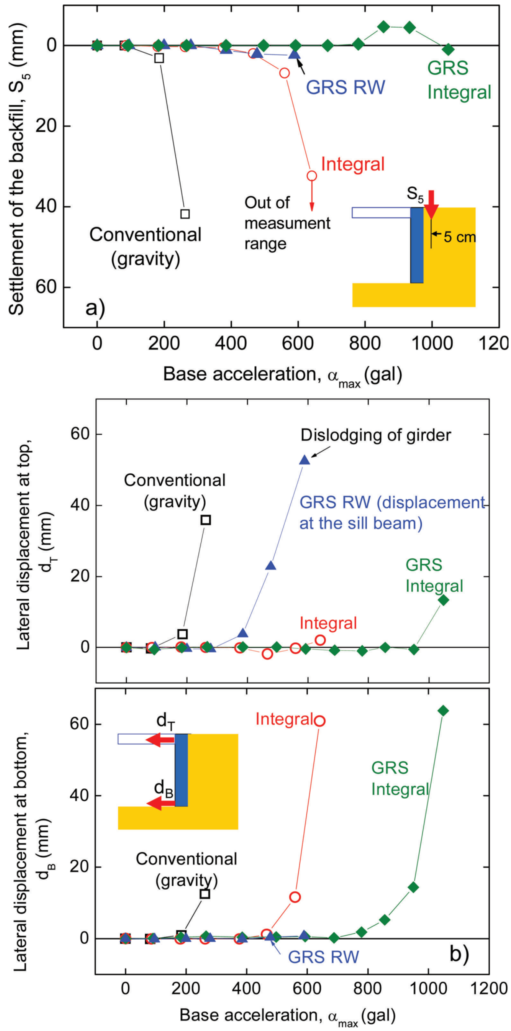

Figure 12. a) Backfill settlement; and b) outward lateral displacements of the facing in laboratory shaking table tests (Aizawa et al., 2006; Hirakawa et al., 2007a).

REFERENCES

Aizawa, H., Nojiri, M., Hirakawa, D., Nishikiori, H., Tatsuoka, F., Tateyama, M. and Watanabe, K. 2006, Comparison of cost-performance among conventional and new types of bridge system consisting of unreiforced or reinforced soil, *Geosynthetics Engineering Journal, IGS Japanese Chapter*, Vol. 21, pp. 175–182 (in Japanese).

England, G, L., Neil, C, M. and Bush, D, I. 2000, Integral Bridges, A fundamental approach to the time-temperature loading problem, *Thomas Telford.*

Hirakawa, D., Nojiri, M., Aizawa, H., Tatsuoka, F., Sumiyoshi, T. and Uchimura, T. 2006, Behaviour of geosynthetic-reinforced soil retaining wall subjected to forced cyclic horizontal displacement at wall face, *Proc.*

8th Int. Conf. on Geosynthetics, Yokohama, Vol. 3, pp. 1075–1078.

Hirakawa, D., Nojiri, M., Aizawa, H., Nishikiori, H., Tatsuoka, F., Tateyama, M. and Watanabe, K. 2007a, Effects of the tensile resistance of reinforcment embedded in the backfill on the seismic stability of GRS integral bridge, *Proc. of 5th International Symposium on Earth reinforcement (IS Kyushu* 2007).

Hirakawa, D., Nojiri, M., Aizawa, H., Tatsuoka, F., Sumiyoshi, T. and Uchimua, T. 2007b, Residual earth pressure on a retaining wall with sand backfill subjected to forced cyclic lateral displacements, *Soil Stress-Strain Behavior: Measurement, Modeling and Analysis, Geotech. Symposium in Roma* 2006 (Ling et al., eds.).

Koseki, J., Tatsuoka, F., Watanabe, K., Tateyama. M., Kojima, K. and Munaf, Y. 2003, Model tests of seismic stability of several types of retaining walls, *Reinforced soil engineering, Advances in Research and Practice*, Marcel Dekker, Inc. (Ling et al. eds.), pp. 317–358.

Nakarai, K., Uchimura, T., Tatsuoka, F., Shinoda, M., Watanabe, K. and Tateyama, M. 2002, Seismic stability of geosynthetic-reinforced soil bridge abutment, Proc. of 7th *Int. Conf. on Geosynthetics,* Nice, Vol. 1, pp. 249–252.

Ng, C., Springman, S. and Norrish, A. 1998, Soil-structure interaction of spread-base integral bridge abutments, *Soils and Foundations*, 38–1, pp. 145–162.

Nojiri, M., Aizawa, H., Hirakawa, D., Nishikiori, H., Tatsuoka, F., Tateyama, M. and Watanabe, K. 2006, Evaluation of dynamic failure modes of various bridge types by shaking table tests, *Geosynthetics Engineering Journal, IGS Japanese Chapter*, Vol. 21, pp. 159–166 (in Japanese).

Shinoda, M., Uchimura, T. and Tatsuoka, F. 2003a, Increasing the stiffness of mechanically reinforced backfill by preloading and prestressing, *Soils and Foundations,* 43–1, pp. 75–92

Shinoda, M., Uchimura, T and Tatsuoka, F. 2003b, Improving the dynamic performance of preloaded and prestressed mechanically reinforced backfill by using a ratchet connection, *Soils and Foundations,* 43–2, pp. 33–54.

Tateyama, M., Aoki, H., Yonezawa, T. 2002, Development of bridge abutment comprising of geosynthetic-reinforced cement-mixed backfill, Doboku Gijutsu, 57–2, pp. 54–61 (in Japanese).

Tatsuoka, F., Tateyama, M, Uchimura, T. and Koseki, J. 1997, Geosynthetic-Reinforced Soil Retaining Walls as Important Permanent Structures, 1996-1997 Mercer Lecture, *Geosynthetic International,* 4–2, pp. 81–136.

Tatsuoka, F., Koseki, J., Tateyama, M., Munaf, Y. and Horii, N. 1998, Seismic stability against high seismic loads of geosynthetic-reinforced soil retaining structures, Keynote Lecture, *Proc. 6th Int. Conf. on Geosynthetics,* Atlanta, Vol. 1, pp. 103–142.

Tatsuoka,F. 2004, Cement-mixed soil for Trans-Tokyo Bay Highway and railway bridge abutments, Geotechnical Engineering for Transportation Projects, *Proc. of Geo-Trans 04, GI, Los Angels, ASCE GSP No. 126 (Yegian & Kavazanjian eds.),* pp. 18–76.

Tatsuoka, F., Tateyama, M., Aoki, H. and Watanabe, K. 2005, Bridge abutment made of cement-mixed gravel backfill, *Ground Improvement, Case Histories, Elesevier Geo-Engineering Book Series, Vol. 3* (Indradratna & Chu eds.), pp. 829–873.

Uchimura, T., Tateyama, M., Koga, T. and Tatsuoka, F. 2003, Performance of a preloaded-prestressed geogrid-reinforced soil pier for a railway bridge, *Soils and Foundations*, 43–6, pp. 33–50.

Watanabe, K., Tateyama, M., Yonezawa, T., Aoki, H., Tatsuoka, F. and Koseki, J. 2002, Shaking table tests on a new type bridge abutment with geogrid-reinforced cement treated backfill, Proc. of 7*th Int. Conf. on Geosynthetics*, Nice, Vol.1, pp. 119–122.