KGS Fall National Conference 2007 / September 14~15, 2007 / Busan / Korea

Integral Bridge System with Geosynthetic-Reinforced Backfill

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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 FHR facings (i.e., RC parapets) that are integrated with a girder without using any girder-support. GRS integral bridges are basically much more cost-effective in construction and long-term maintenance while having a much higher seismic stability than conventional-type bridges having a girder via movable and fixed supports on a pair of cantilever abutments. GRS integral bridges are better than bridges using GRS retaining walls as abutments and also than conventional integral bridges with unreinforced backfill. To validate the above, a series of static cyclic lateral loading tests of the facing and a series of shaking table tests were performed on small-scaled models of different bridge types.

1. INTRODUCTION

The conventional type bridge abutment (Fig. 1) has been and is used widely over the world. Despite the above, it has the following drawbacks.

1) The abutment is a cantilever structure supporting unreinforced backfill on its back (Fig. 2). For this reason, piles are usually necessary to resist against stresses that are concentrated near the toe of the abutment base while the abutment becomes massive being heavily steel-reinforced due to high internal stresses.
2) The RC abutment is not allowed to noticeably displace once constructed. After constructed, however, it is subjected to earth pressure associated with the backfill construction together with associated detrimental effects by settlement and lateral flow in the subsoil via their effects on the piles. To limit displacements of the abutment in this event, the number and size of piles may be increased.
3) The construction and long-term maintenance of girder-supports are generally costly.
4) A bump may be formed behind the abutment by residual deformation of the backfill due to its self-weight and traffic loads during long term service. 5) 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 Tatsuoka et al. (2005) confirmed this point by model shaking table tests.

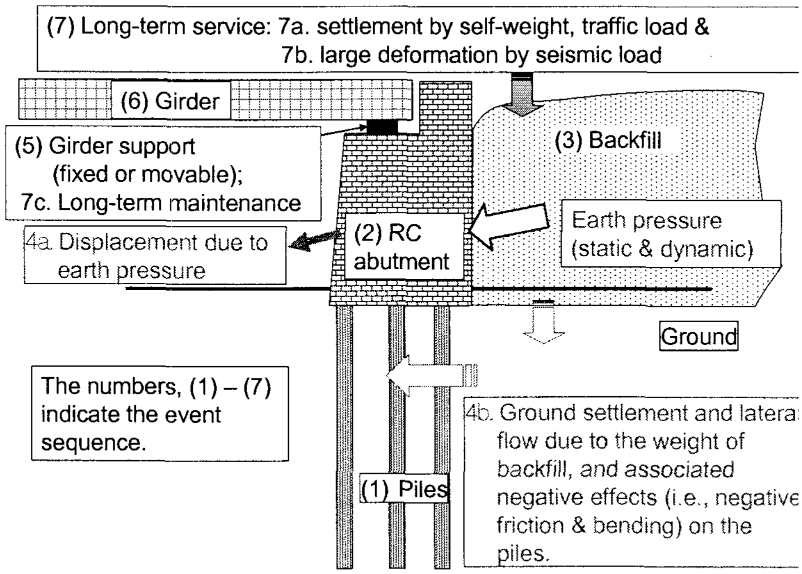
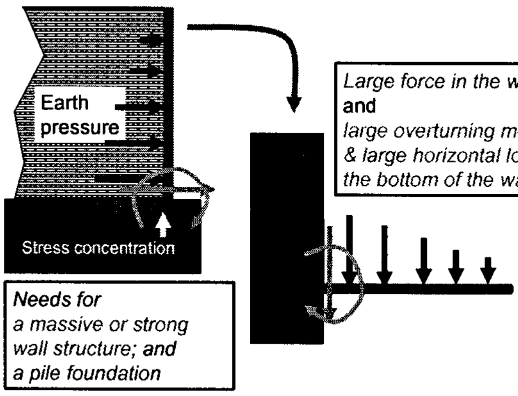


Figure 1. Construction sequence and technical problems with conventional type bridge abutments.

Two different bridge systems that are more cost-effective than the conventional bridge system have been proposed (Fig. 3). They have been developed to alleviate either several problems with the structural parts (i.e., a bridge girder and a pair of parapets) or those with the backfill. In this report, a new bridge type, called the GRS (geosynthetic-reinforced soil) integral bridge, is proposed. This new bridge system has been developed by combining these two solutions; a structure engineering solution and a geotechnical engineering solution. It is shown below that GRS integral bridges can alleviate several technical problems of the respective solutions while taking advantages of the respective superior characteristic features.



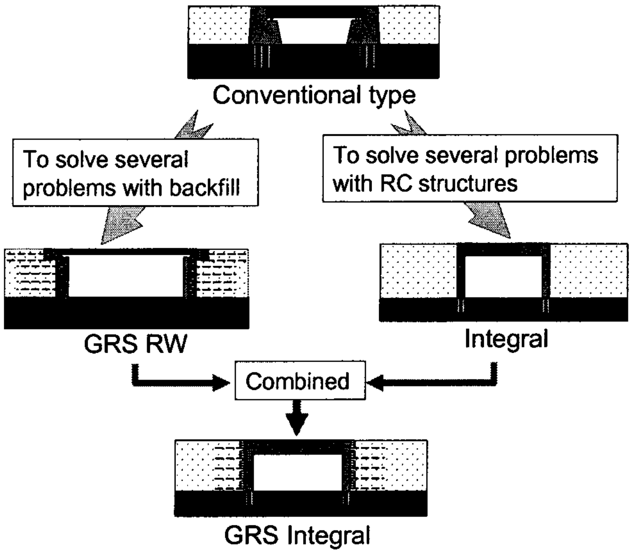
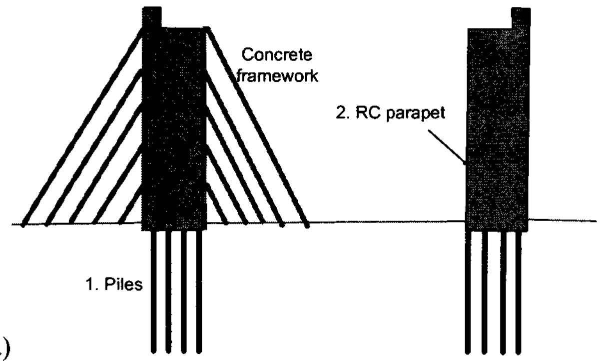


Figure 2 (left) Inherent problems with cantilever type retaining walls.
Figure 3 (right) Development of GRS integral bridge.

2. INTEGRAL BRIDGE

The integral bridge system is very popular in the UK and the USA due mainly to high cost/performance by low construction and maintenance cost of the structural part resulting from no use of girder-supports (Figs. 4a & b). However, the backfill may exhibit large residual settlement by self-weight and traffic load during long-term service as well as by seismic loads in the same way as the conventional type bridge. The seismic stability of the structural part (i.e., a girder and a pair of parapets) is higher than the conventional type (Fig. 1). However, the structural part and the backfill are not unified unlike the GRS integral bridge (Fig. 3) and either part does not help effectively the other part during seismic events. Therefore, its seismic stability cannot become very high as shown later in this paper. 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 parapets (Fig. 4c). This may cause two major detrimental effects; 1) an increase in the earth pressure on the back of the parapet; and 2) residual settlements in the backfill, as shown below.

Small-scaled model tests were performed in the laboratory (Fig. 5) to evaluate the effects of cyclic displacements at the top of a full-height rigid (FHR) facing (i.e., a parapet) and also to examine whether this problem can be alleviated by reinforcing the backfill (Hirakawa et al., 2006, 2007a). The backfill was air-dried Toyoura sand (D_r= 90 %) 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.



a)

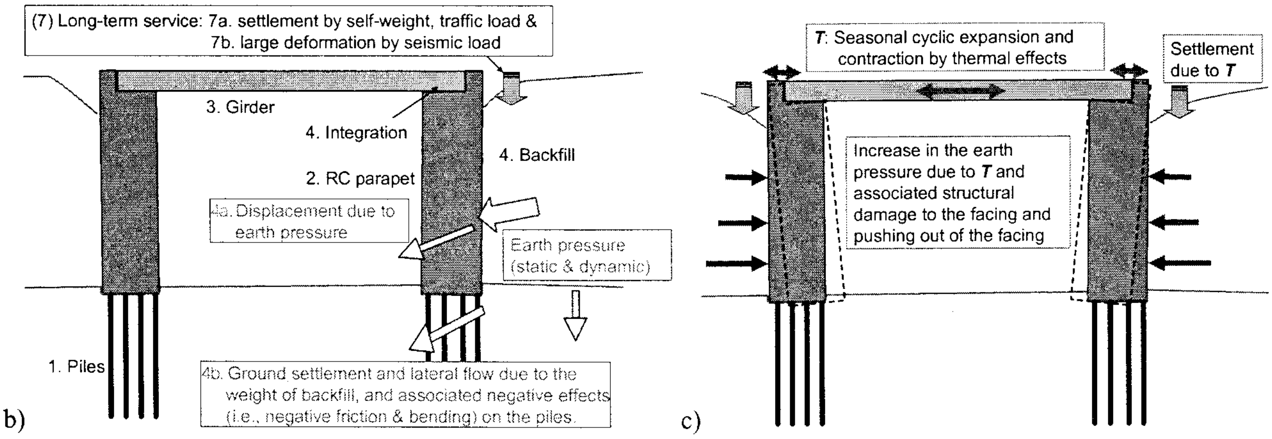


Figure 4 Integral bridge: a) & b) construction sequence and associated problems; and c) a new problem.

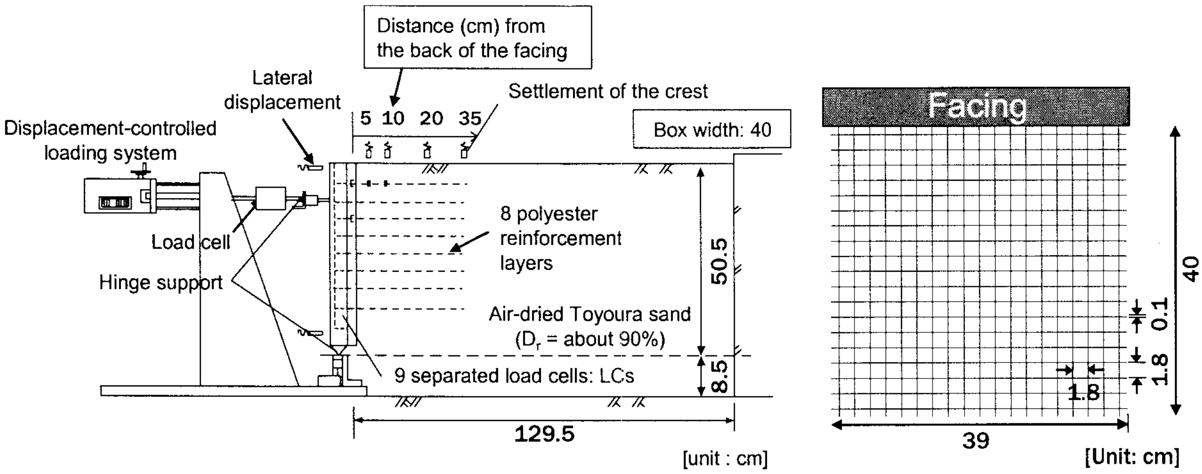


Figure 5 Model tests to evaluate the effects of cyclic horizontal displacement at the top of facing.

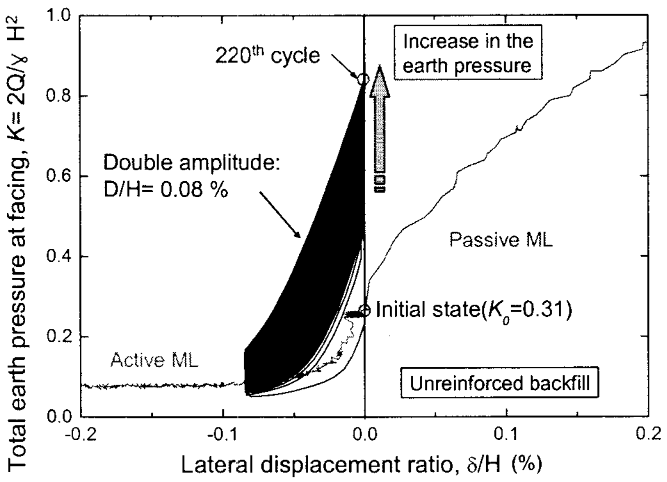


Figure 6 Increase in the earth pressure by cyclic displacements at the facing top (unreinforced backfill).

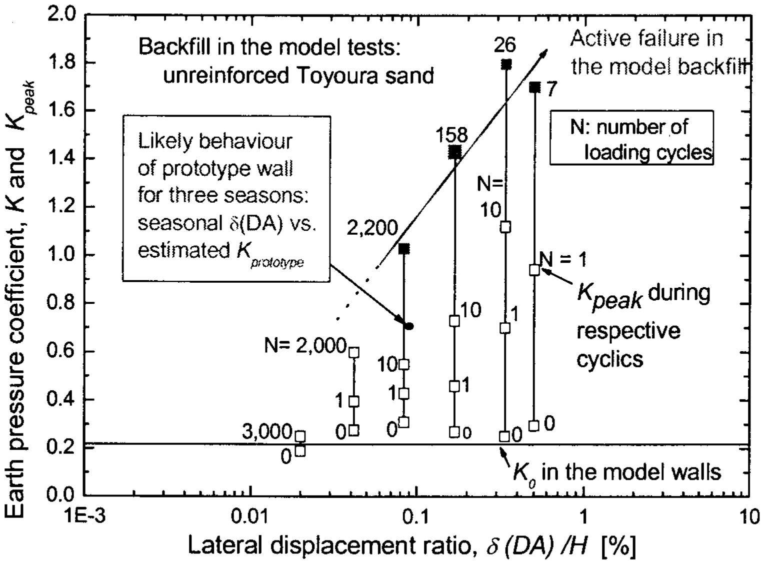


Figure 7 Peak earth pressure coefficients in the model tests and a field full-scale case.

Fig. 6 shows the relationships between the earth-pressure coefficient, $K= 2Q/H^2\gamma$, where Q is the total earth pressure per width; H is the wall height (50.5 cm); and γ is the dry unit weight of the backfill (1.60 gf/cm^3) and the ratio of the displacement at the facing top to the facing height, δ/H, during monotonic active and passive loading tests as well as during a typical cyclic loading test. Despite a very small facing displacement, the earth pressure at the most passive state in the respective cycle increases noticeably with cyclic loading. Fig. 7 summarizes the peak value of K in each cycle, K_{peak}, at selected numbers of loading cycle, N plotted against D/H. The solid squares represent the cycles when the active failure plane appeared in the backfill as shown in Fig. 8. The earth pressure increases with an increase in D/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; Hirakawa et al., 2006). This earth pressure increase may result in structural damage to the facing and may push out the bottom of the facing in field full-scale cases. The other major detrimental effect of cyclic displacement of the facing is gradual but eventually large settlements in the unreinforced backfill (case NR in Fig. 9) associated with the development of an active failure plane in the backfill (Fig. 8). The backfill settlement increases with an increase in D/H. Effects of reinforcing the backfill on the backfill settlement are discussed later in this paper.

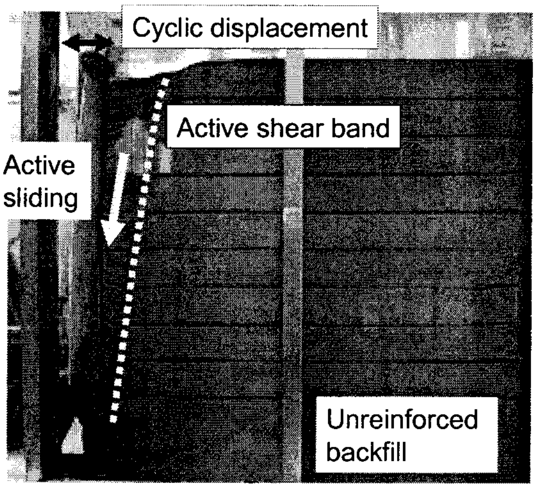


Figure 8 Active failure in unreinforced backfill subjected to cyclic lateral displacement o the facing (D/H= 0.4 %).

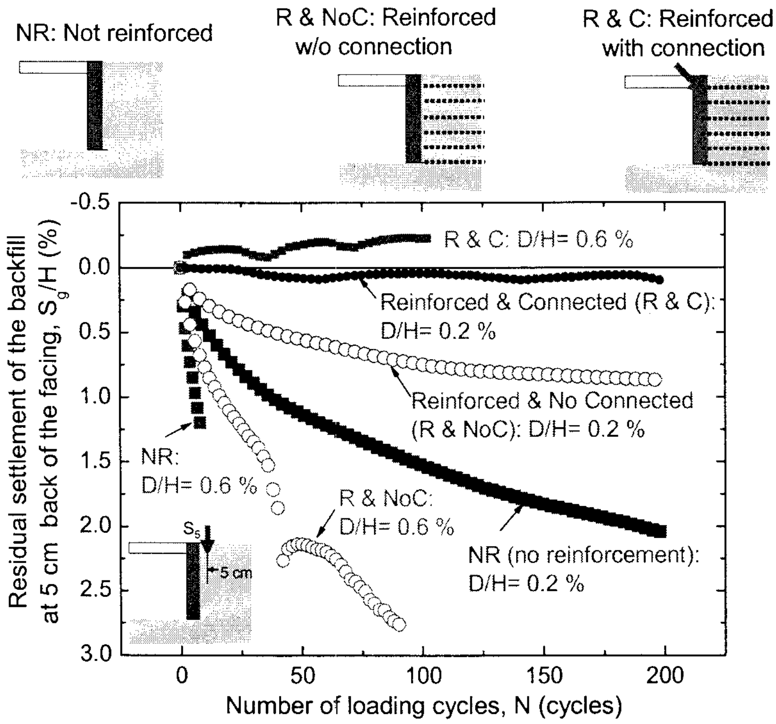


Figure 9 Residual settlements in the backfill (when □= 0) by cyclic displacement at the facing top and effects of reinforcing the backfill (modified from Hirakawa et al., 2007a).

These two effects of cyclic facing displacements can be explained by the following dual ratcheting mechanism in the backfill. Firstly, by a small active displacement of the facing in each cycle, small active sliding develops along the active shear band. When subjected to a small passive displacement of the facing, the active sliding mechanism explained above is not activated, because the passive failure mechanism in the backfill is much deeper with a much larger passive wedge zone. Therefore, despite a small amount in the respective cycle, the active sliding in each cycle is accumulated with cyclic loading and the accumulated active sliding eventually reaches the value when the active failure takes place in the backfill during monotonic active loading. Secondly, the passive sliding is also inactive when the facing moves in the active direction. Therefore, a small amount of passive sliding that takes place in each cycle is also accumulated with cyclic loading, which results in gradual mobilization of higher passive earth pressure with cyclic loading. As the passive displacement of the facing when the passive failure takes place in the backfill during monotonic passive loading is substantially larger, the passive failure is never reached during cyclic loading. Another mechanism that makes the earth pressure at the most passive state in the respective cycles increase with cyclic loading is an increase in the coefficient of horizontal subgrade reaction of the backfill (i.e., the slope of $K - \delta/H$ relation) with cyclic loading. This is due to an increase in the stiffness of the backfill associated with cyclic loading as observed in stress-strain tests on sand.

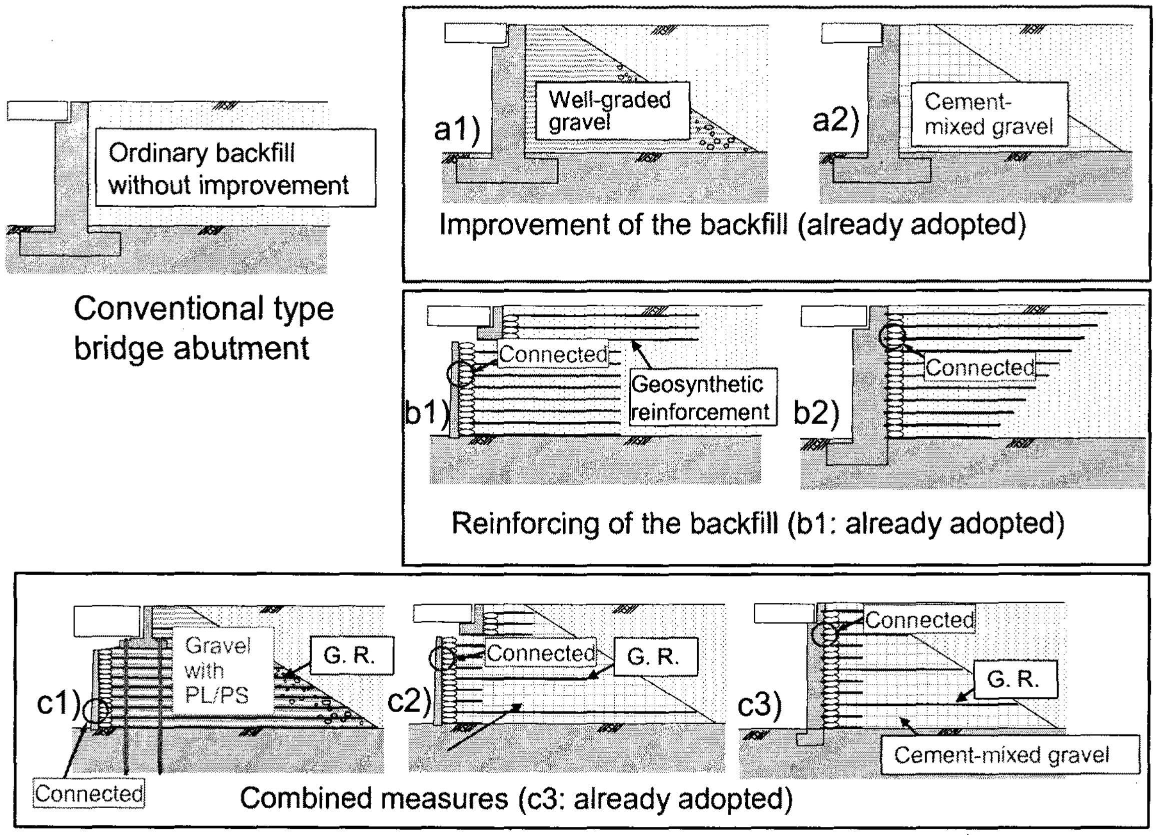


Figure 10 Different solutions to improve the performance of the backfill of bridge abutment.

3. IMPROVING THE PERFORMANCE OF BACKFILL

3.1 Several proposals

Fig. 10 shows different solutions that have been proposed to improve the performance of the backfill for a bridge system. The Japanese railways engineers constructed a trapezoidal zone of well-compacted well-graded gravelly soil immediately behind the abutment (type *a1* in Fig. 10). However, the performance of this type during several previous earthquakes in Japan was not satisfactory. 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 (type *a2*, Fig. 10), constructing a trapezoidal zone of cement-mixed gravel, is not sufficiently high.

3.2 Reinforcing the backfill

Fig. 11 illustrates the staged construction of geosynthetic-reinforced soil (GRS) retaining wall (RW) with full-height rigid (FHR) facing. This wall type is now one of the standardized RW construction technologies in Japan and has been constructed at more than 600 sites with a total wall length more than 95 km as of June 2007. The main features of this technology are as follows: 1) FHR facing functions as a rigid continuous beam supported at many levels with a small span, equal to 30 cm (Fig. 12), which makes the internal stresses in the facing and the stresses at the facing bottom much smaller than those of a cantilever wall (Fig. 2). Moreover, a high tensile force can be mobilized in the reinforcement largely resulting from high connection strength at the back of the FHR facing, which results in a higher confining pressure and therefore higher stiffness and shear strength in the active zone in the backfill. 2) The backfill is constructed with a help of gravel gabions placed at the shoulder of each soil layer. 3) Geosynthetic reinforcement layers are arranged with a vertical spacing of 30 cm. This small lift can facilitate a high compaction of the backfill. 4) After a full-height of geosynthetic RW is completed and then sufficient compression of the backfill and settlement of the supporting ground has taken place, a lightly steel-reinforced concrete facing is cast-in-place directly on the wrapped-around wall face ensuring a strong connection to the reinforced backfill. Therefore, negative interactions between the FHR facing and the compression of the backfill during filling-up and compaction works can be avoided, large compression of the supporting ground associated with the backfill construction can be accommodated ensuring the stability of wall, the backfill immediately back of the wall face can be compacted dense with better mobilization of reinforcement tensile force, and the alignment of completed wall face at will becomes easy.

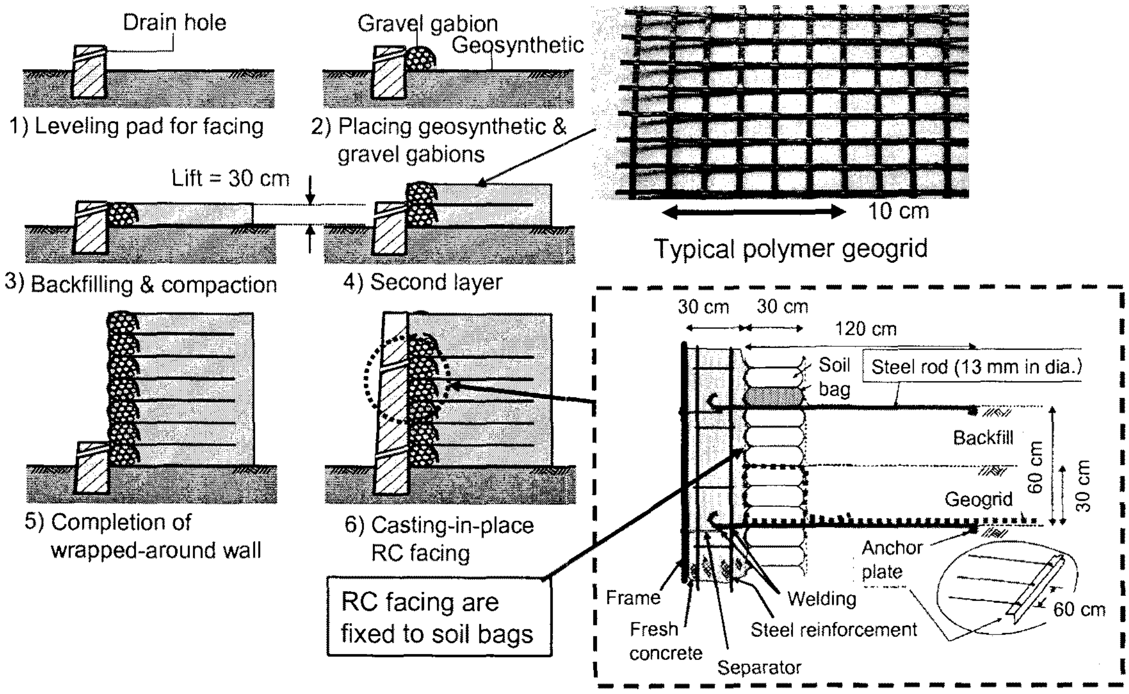


Figure 11 Staged construction of a GRW RW with a FHR facing.

Taking advantages of these features described above, a number of bridges comprising of a pair of GRS RWs with a FHR facing that support a girder (type *b1* in Fig. 10 and Fig. 13; Tatsuoka et al., 1997, 2005), were constructed. This bridge type is herein called the GRS-RW bridge. 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 girder-supports is costly. This is the common problem with all of the bridge types presented in Fig. 10. 3) Despite that the dynamic stability of GRS RW with a FHR facing is very high (e.g., Tatsuoka et al., 1998; Koseki et al., 2006), the dynamic stability of the sill beam having a fixed girder-support is not so (Aizawa et al., 2007; Hirakawa et al., 2007b). This is because the mass of the sill beam is very small when compared with the inertia of the girder while the anchorage capacity of the reinforcement layers connected to its back is small due to their shallow depths. Type *b2* (Fig. 10), placing a girder on the crest of the FHR facing, is dynamically more stable than type *b1* (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 some large deformation when subjected to high seismic loads. Furthermore, the problem 2 is still unsolved.

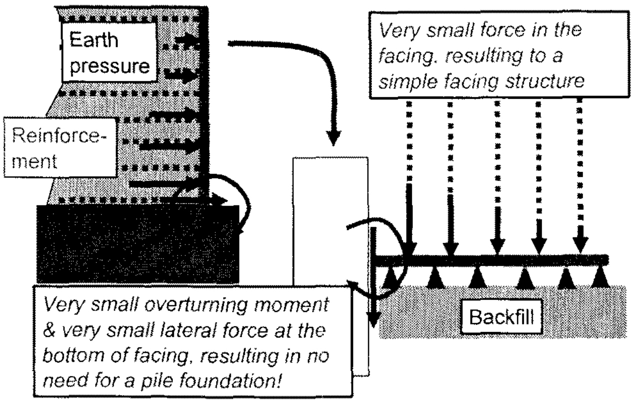


Figure 12 FHR facing as a continuous beam supported at many places with a small span.

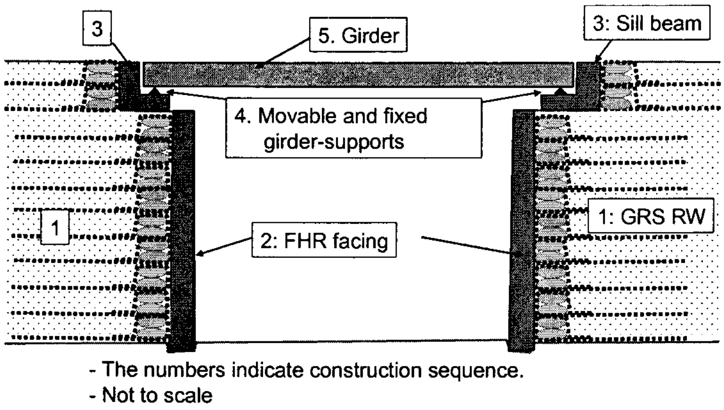


Figure 13 GRS-RW bridge.

3.3 Combining two measures

To substantially decrease long-term residual deformation of the backfill with type *b1*, it is very effective to vertically preload the reinforced backfill and then maintain some vertical prestress, typically about a half of the prestress, in the backfill during long-term service (i.e., the PL & PS technology; type *c1* in Fig. 10). The above was validated by laboratory model tests (Shinoda et al., 2003) and long-term performance of a prototype railway bridge pier (Uchimura et al., 2003a). Moreover, Uchimura et al. (2003b) 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 by using a ratchet mechanism to fix the ends of tie rods (Shinoda et al., 2003). Type *c1* in Fig. 10 consists of a PL-PS GRS RW with a ratchet system supporting a girder via a fixed-support. Despite that its high seismic stability was validated by laboratory shaking table tests, any prototype bridge of this type has not been constructed, because possible long-term maintenance works of the ratchet system are not preferred by practicing engineers.

Types *c2* and *c3* were then proposed, which combines types *b1* and *b2* with type *a2*. Type *c3* was adopted by railway engineers and the first prototype was constructed for a new bullet train line in Kyushu (Tatsuoka et al., 2005). Type *c3* abutments are constructed by the staged procedure presented in Fig. 11. The conventional RC abutment (Fig. 1) supports the unreinforced backfill which may activate large static and dynamic earth pressure on its back. In comparison, with type *c3* as well as type *b2*, the reinforced backfill laterally supports the RC parapet (i.e., facing) that is supporting a girder without activating dynamic earth pressure on the back of the parapet.

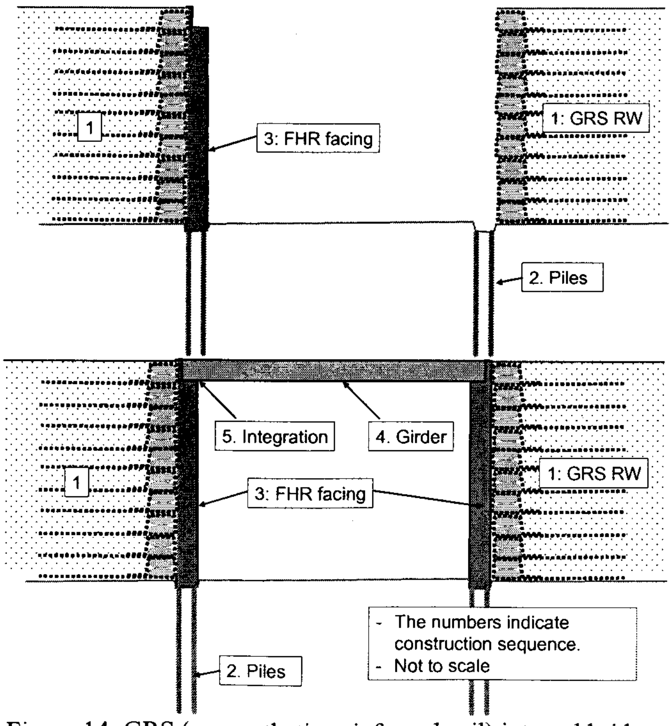


Figure 14 GRS (geosynthetic-reinforced soil) integral bridge.

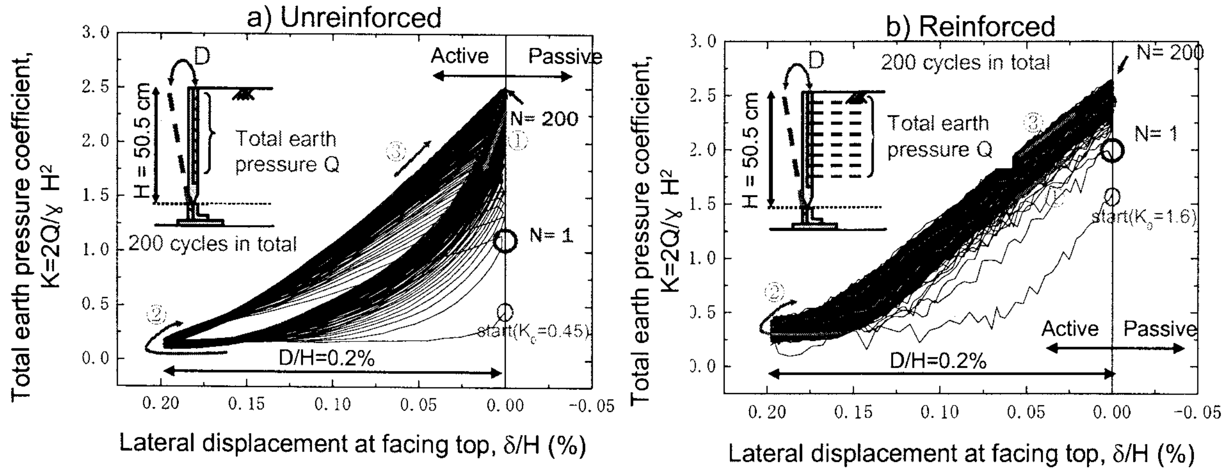


Figure 15 Increase in the earth pressure by cyclic displacements at the facing top (see Fig. 5 for the test method).

4. GRS INTEGRAL BRIDGE

4.1 Features of IGS integral bridge

A new bridge type (called the GRS integral bridge; Fig. 14), which is proposed herein, is more cost-effective and more dynamically stable than the others described in this paper. As illustrated in Fig. 3, this new type combines the integral bridge (Fig. 4) and the GRS RW bridge (Fig. 13) taking their advantages: i.e., stabilization of the backfill by geosynthetic-reinforcing (the GRS RW bridge) and simpler and more cost-effective RC structure without using girder-supports (the integral bridge). At the same time, their inherent drawbacks are alleviated. Firstly, a GRS integral bridge may also need a pile foundation to support the girder. However, a lighter one than the one for an integral bridge may be sufficient, because needs for a pile foundation are usually much lower as with GRS RWs. Secondly, detrimental effects of thermal cyclic contraction and expansion of the girder with integral bridges can be alleviated with the GRS integral bridge. As seen from Fig. 9, the backfill settlement when subjected to cyclic lateral displacements at the facing top becomes nearly null when the backfill is reinforced with reinforcement layers connected to the facing (case R&C). Even slight heaving in the backfill takes place. This trend of behaviour can be explained by the following factors. a) For the same thermal effects on the girder, the displacements of the facing become smaller due to increased stiffness of the reinforced backfill. b) For the same cyclic facing displacement, the residual settlement in the backfill decreases substantially due to higher confining pressure in the backfill as well as membrane effects of reinforcement layers connected to the facing. Note that, as seen from Fig. 9, 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. On the other hand, by reinforcing the backfill with reinforcement layers connected to the facing, the earth pressure increases largely with cyclic loading as with unreinforced backfill (Fig. 15). However, the facing is not structurally damaged and the facing bottom is not pushed out by this increased earth pressure, because the FHR facing functions as a continuous beam supported by a number of reinforcement layers at a small spacing (Fig. 12). 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.

Bridge type	Cost & period of construction	Maintenance cost	Seismic stability	Total
 Conventional	1 A, B	1 C, D	1 F, G 252 gal*	3
 Integral	2 B	1 D, E	2 F 641 gal*	5
 GRS RW	3	1 C, D	2 G 589 gal*	6
 GRS Integral	3	3	3 1,048 gal*	9

(* Acceleration at failure in model shaking table tests)

A= massive & strong parapets necessary because of cantilever structure.
B= piles usually necessary because the parapet is a cantilever structure and the backfill is constructed after construction of piles & parapets.
C= cost for construction and long-term maintenance of girder supports.
D= intolerable backfill settlement by self-weight and traffic loads during long-term service (intolerable settlement of the sill beam with GRS RW bridges).
E= thermal expansion and contraction of the girder, resulting in cyclic lateral displacements of parapets and associated increase in the earth pressure, which may damage the parapets and intolerable backfill settlement.
F= large settlements in the backfill and large dynamic earth pressure by seismic loads
G= Low seismic stability due to independent performance of abutments (unstable sill beam in seismic events with GRS RW bridges).

Figure 16 Comparison of the performances of four different bridge types.

Fig. 16 compares in an approximate way the advantages and disadvantages in the three items of the four bridge types: i.e., conventional, GRS RW, integral and GRS integral. The accelerations shown in the second column from the right are those at which the respective bridge models collapsed in the shaking table tests described below. The full point allocated to each item is equal to three, which is reduced one by one when negative factors *A* through *G* as listed are relevant to the respective bridge types. The total full points are equal to nine, which is assigned only to the GRS integral bridge.

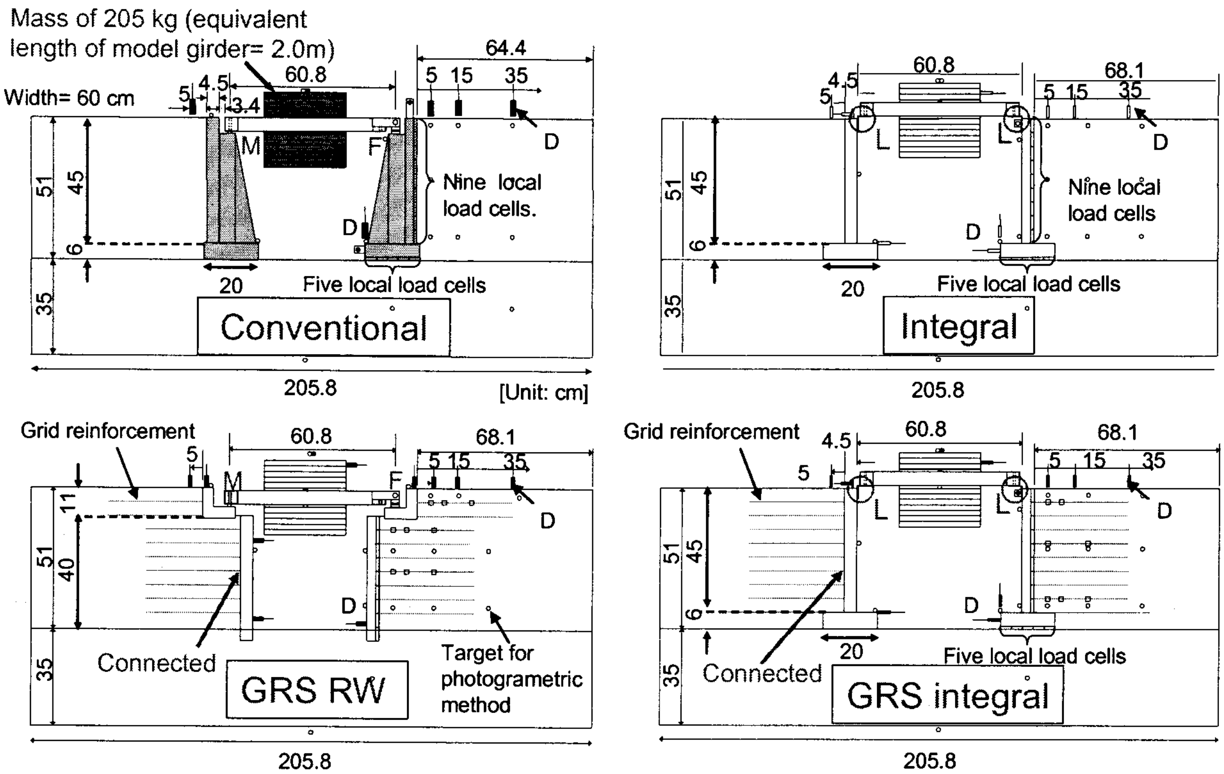
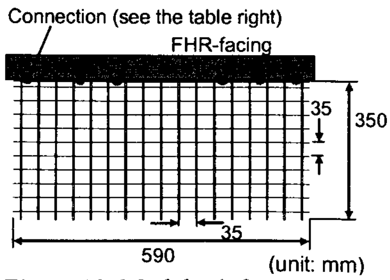


Figure 17 Four models for first series shaking table tests (the backfill is air-dried Toyoura sand with D_r= 90 %); D: displacement transducer; M: movable sliding support; F: fixed hinge support; and L: L-shaped metal fixture.

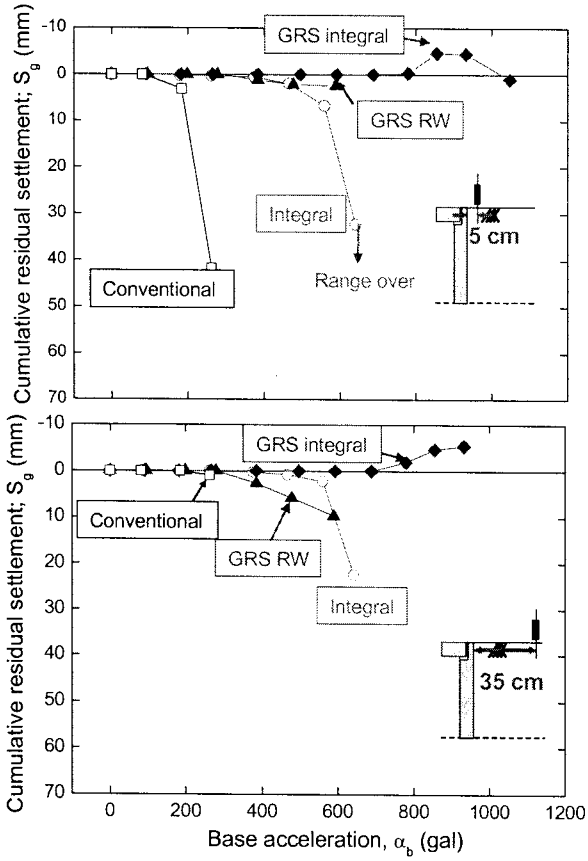
4.2 Model shaking table tests

Two series of shaking table tests were performed (Aizawa et al., 2007; Hirakawa et al., 2007b; Tatsuoka et al., 2007). The first series compared the seismic stabilities of the four models (Fig. 17) of the bridge types described in Fig. 16. The second series evaluated effects of the connection strength between the reinforcement and the facing of GRS integral bridge using four models (see the table inset in Fig. 18). The model in test 4 is the same as the GRS integral bridge model in the first series. Assuming a length similitude ratio equal to 1/10, the facing was 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). The reinforcement was a phosphor bronze grid consisting of 17 longitudinal strands with rupture strength of 359 N per strand (Fig. 18). The covering ratio of the grid was 10.1 %. The surface of the strands was made rough with a friction angle equal to 35 degrees at confining pressure equal to 50 kPa by gluing sand particles. Twenty sinusoidal waves at a frequency of 5 Hz was applied at the shaking table step by step increasing the maximum acceleration, α_b, with an increment of 100 gal.



Test name	No. of reinforcement layers	Connection conditions: Connection strength [N/layer]	Connection conditions: Connection method
Test 1	8	400	Melting, 4 points
Test 2	9	520	Bolt(M3), 4 points
Test 3	10	520	Bolt(M3), 4 points
Test 4	10	1,070	Bolt(M3), 6 points

Figure 18 Model reinforcement; and different reinforcement and connection conditions in the second series.



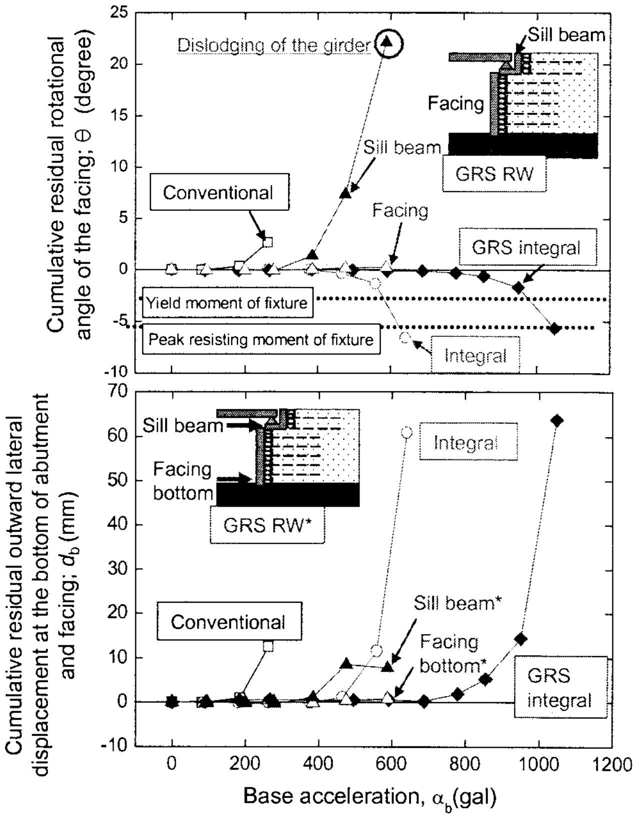
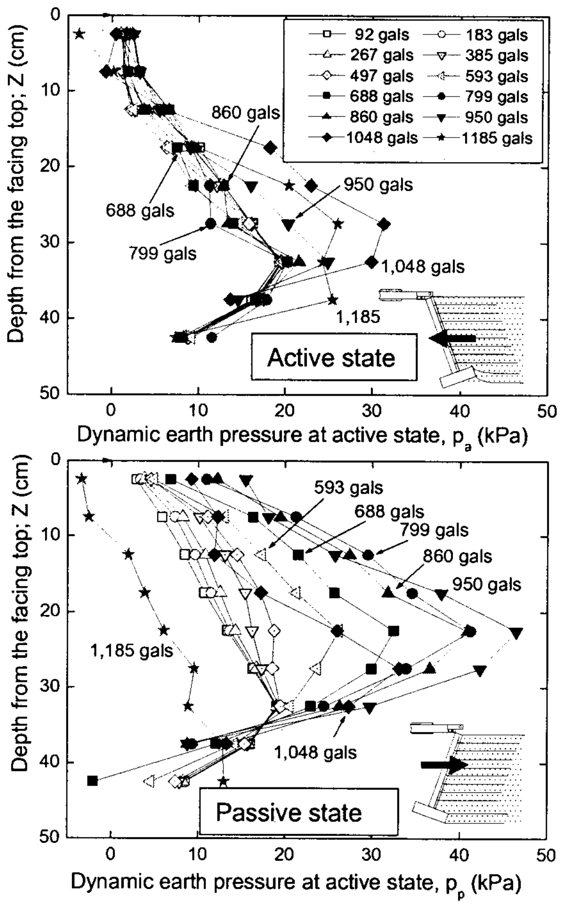


Figure 19 (left) Effects of bridge type on settlements in the backfill 5 cm and 35 cm back the facing, first series.
Figure 20 (right) Effects of bridge type on facing rotation and displacement at the facing bottom, first series.

Fig. 19 shows the backfill settlements 5 cm and 35 cm back the facing. In the top of Fig. 19, the settlement of the sill beam of the GRS RW bridge is presented. Fig. 20 shows the overturning angle and the lateral displacements at the bottom of the facing. In the top of Fig. 20, the rotational angle of the sill beam of the GRS RW bridge is also presented. In Figs. 19 and 20, with the conventional and GRS RW bridges, the displacements of the abutment or facing on the side supporting the girder via a fixed-support are presented. The following trends of behaviour may be seen. Firstly, the GRS integral bridge, together with reinforced backfill, is much more stable than the others, while the conventional type, with unreinforced backfill, is least stable. Secondly, the pushing out of the facing bottom associated with rotation of the facing is the major failure mode with the integral and GRS integral bridges. It may be seen from Fig. 20 that the strength of the fixture integrating the girder and the facings of the GRS integral bridge is insufficient to fully retain this displacement mode of the facing. It is considered that this situation is representative of full-scale behaviour.

Fig. 21 shows the earth pressure distributions on the facing at the 10th cycle at each stage of shaking with the GRS integral bridge from the first series. It may be seen that, in the upper part of facing, the largest earth pressure at the respective heights is activated under the passive condition at the facing top. On the other hand, in the lowest part of the facing, the largest earth pressure is activated under the active condition at the facing top. These trends are due to that the major critical displacement mode of the bridge system is the rotation of the facing. Fig. 22 shows the effects of the number of reinforcement layers and the connection strength on the dynamic behaviour of GRS integral bridge from the second series. It may be seen that the connection strength at the lower part of the facing controls the dynamic stability of the facing, therefore, of the GRS integral bridge. This point can be seen from Fig. 23: i.e., in test 4, the tensile force immediately behind the facing in the reinforcement layer near the facing bottom becomes very high, which is due to the rotational displacements of the facing. This result indicates that a high connection strength between the reinforcement and the facing is essential for a high seismic stability of GRS integral bridge.



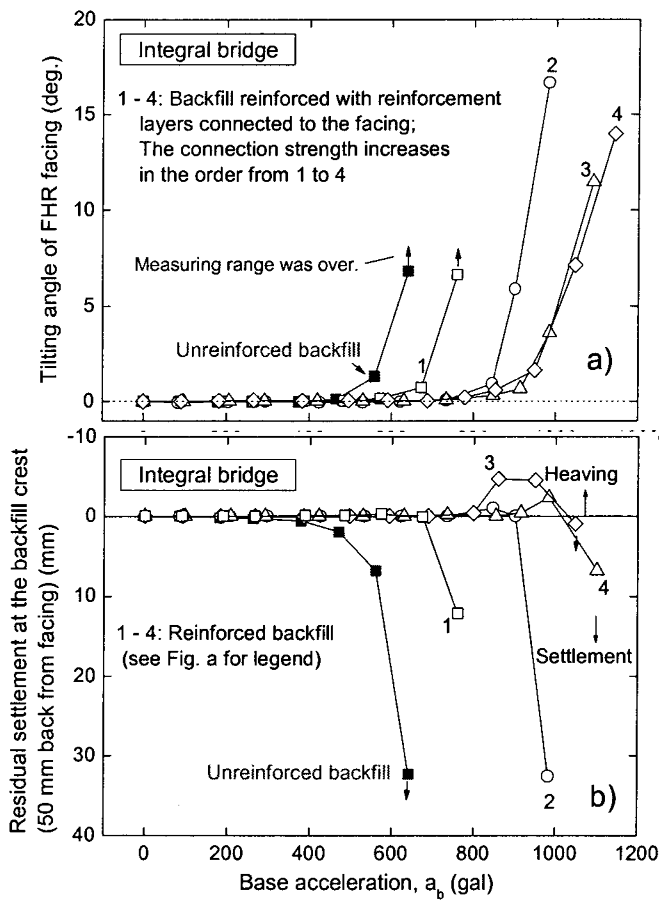


Figure 21 (left) Earth pressure distribution with depth on the facing at 10th cycle at each stage, GRS integral bridge.
Figure 22 (right) Effects of number of reinforcement layers and connection strength on; a) residual tilting angle of the facing; and b) residual settlement of the backfill 5 cm back the facing, second series (see Fig. 18 for test numbers 1 – 4).

Fig. 24 summarizes load and resistance components for the facing rotation with GRS integral bridges based on the test results presented above. The two major resisting components are the passive pressure in the upper part of the backfill and the tensile force of the reinforcement at the bottom part of the facing. The former can be increased by lightly cement- mixing relevant part of the backfill. The latter is the minimum value among the connection strength, the tensile rupture strength of the reinforcement and the pull-out strength of the reinforcement. Further study is necessary in this respect.

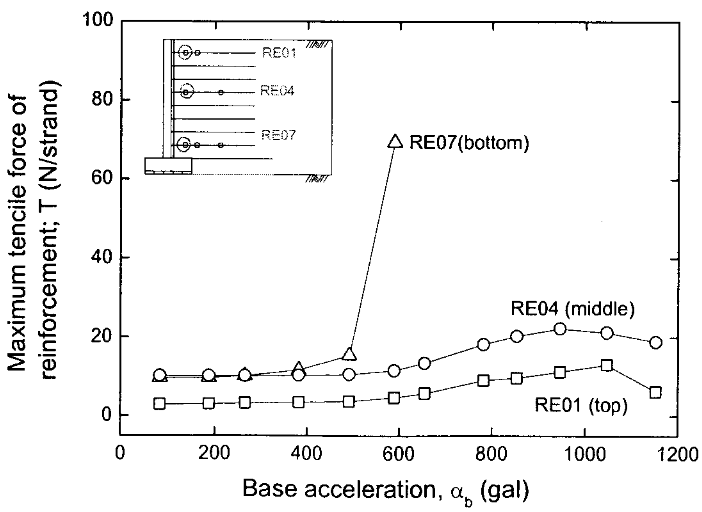


Figure 23 Relationship between reinforcement tensile force and base acceleration in test 4 (see Fig. 18).

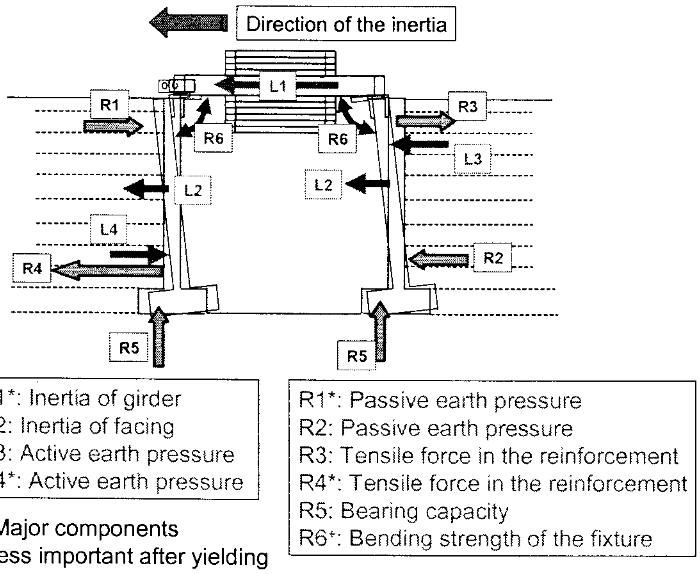


Figure 24 Load and resistance components for the stability in rotation of the facing of GRS integral bridge.

5 CONCLUSIONS

A new type bridge, called the GRS integral bridge, is proposed, which comprises of geosynthetic-reinforced backfill and an integral bridge. The GRS integral bridge can have a high cost-effectiveness in construction and long-term maintenance because of :1) no use of girder supports; 2) reinforcing of the backfill with geosynthetic reinforcement layers that are connected to the facing; and 3) construction of full-height rigid facing after the construction of piles and then backfill. Based on the model test results, it can be concluded that GRS integral bridges: 1) exhibit 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 2) have a very high seismic stability with both structural components (i.e., a pair of parapets and a girder) and backfill due to their integrated performance.

ACKNOWLEDGEMENTS

This report summarises the research work that was performed by Dr. Hirakawa,D., Department of Civil Engineering, Tokyo University of Science, Mr. Nojiri, M., Kawasaki Geological Engineering Co., Ltd, Tokyo, Japan (formerly Graduate Student, Tokyo University of Science), Mr. Aizawa, H. and Mr. Nishikiori, H., Department of Civil Engineering, Tokyo University of Science, and Dr. Tateyama, M. and Dr. Watanabe, K., Research Engineers, Railway Technical Research Institute, Tokyo, Japan. The author deeply appreciate their help in preparing this report.

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