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BENEFITS OF GEOSYNTHETIC-REINFORCING THE BACKFILL FOR INTEGRAL BRIDGES

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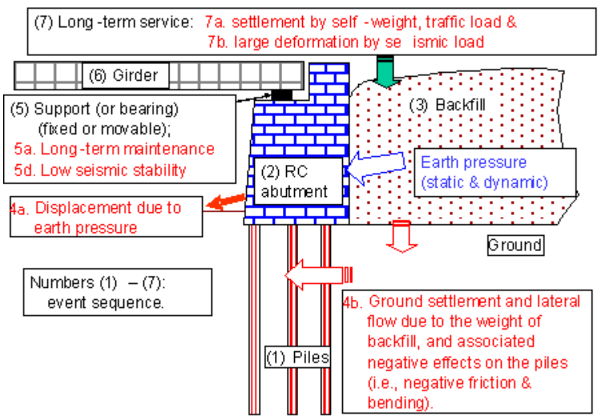
Abstract: To alleviate several problems with conventional type integral bridges, it is proposed to reinforce the backfill with geosynthetic reinforcement layers connected to the back of the full-height rigid facings (i.e., abutments). These problems include; 1) large residual settlements in the backfill, developing a bump immediately behind the abutments, and an increase in the residual earth pressure on the back of the abutments caused by seasonally thermal cyclic deformation of the girder as well as traffic loads on the backfill; and 2) large deformation of the backfill by seismic loads and its detrimental effects on the stability of the bridge. A newly proposed integral bridge is constructed by firstly constructing a geosynthetic-reinforced backfill, secondly full-height rigid (FHR) facings, which become vertical RC abutments of the bridge, and finally a continuous girder (i.e., deck) placed on the crest of the RC abutments and their integration. A series of static loading tests and shaking table tests were performed on scaled models of the conventional and new types of integral bridge as well as two conventional bridge types comprising RC gravity-type abutments and geosynthetic-reinforced soil retaining walls, both supporting a girder via supports (or bearings). The test results showed high static and dynamic performance, despite relatively low construction cost, of the newly proposed integral bridge, called the GRS integral bridge.

Keywords: Abutment, Geogrid reinforcement, Integral bridge, Model test, Retaining wall, Seismic behaviour

INTRODUCTION

It is well-known that the conventional type bridge abutments have a number of drawbacks, as listed below (Fig. 1):

1) As the abutment is a cantilever structure retaining unreinforced backfill, a pile foundation is usually necessary and the abutment may become very large to resist against activated earth pressures
2) As constructed abutments are generally settlement sensitive, a large number of long piles may become necessary to prevent movements due to earth pressure as well as settlement and displacement in the subsoil caused by the backfill weight.
3) The construction and long-term maintenance of girder-supports (or bearings) and connections between separated girders can be costly. The girder supports usually become a serious weak point when subjected to seismic loads.
4) A bump may be formed behind the abutment by settlement of the backfill due to its self-weight and traffic loads.
5) The seismic stability of the backfill and the abutment supporting the girder via a fixed-support is relatively low.



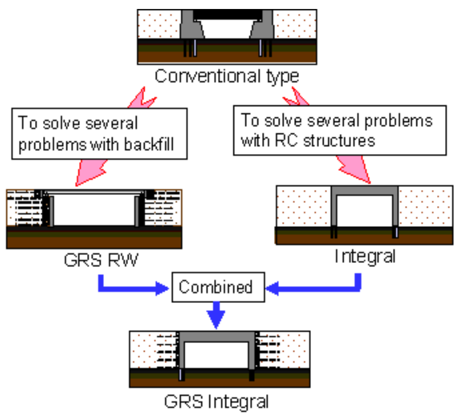


Figure 1 (left). Technical problems with conventional type bridges (in the case of gravity type abutment)
Figure 2 (right). History of the development of GRS integral bridge

To alleviate these problems (Fig. 1), two new more cost-effective bridge systems have been introduced (Fig. 2). The integral bridge (Fig. 3a) was introduced to mainly alleviate problems with RC structures. This bridge system is now widely used in the UK and the USA mainly due to high cost-effectiveness by low construction and maintenance cost resulting from eliminating the use of girder-supports (or bearings) and the use of a continuous girder (or deck). However, the backfill may exhibit excessive settlement by self-weight, traffic loads and seismic loads. Furthermore, as the RC structures and the backfill are not integrated, their seismic stability cannot be very high. Moreover, as a continuous girder is integrated to abutments, seasonal thermal expansion and contraction of the girder results in cyclic lateral displacements at the top of the abutments, which causes high earth pressure on the back of the abutments and large settlement in the backfill (as shown below). The other bridge type is the one using geosynthetic-reinforced soil

(GRS) retaining walls (RWs) with a full-height rigid (FHR) facing as abutments to mainly alleviate problems with the backfill (Fig. 3b), which is herein called the GRS-RW bridge. Although this bridge type is more cost-effective than the conventional type (Fig. 1), it has the following drawbacks: 1) the girder cannot be very long and heavy due to low stiffness of the backfill supporting the girder and a low seismic stability of the fixed support and the sill beam supporting the girder via the fixed support; and 2) construction and maintenance of the girder supports is costly.

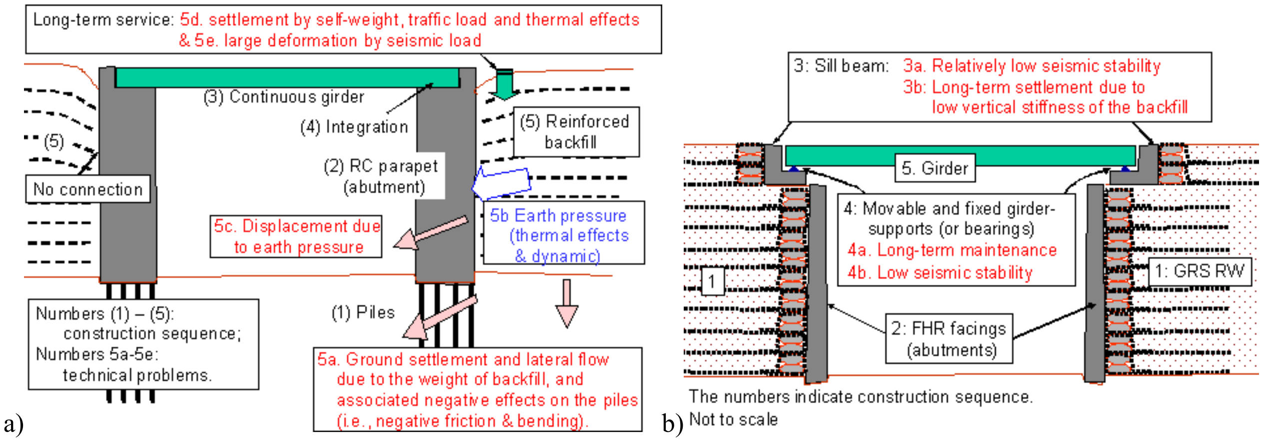


Figure 3. Construction sequence and old & new problems; a) integral bridge; and b) GRS-RW bridge

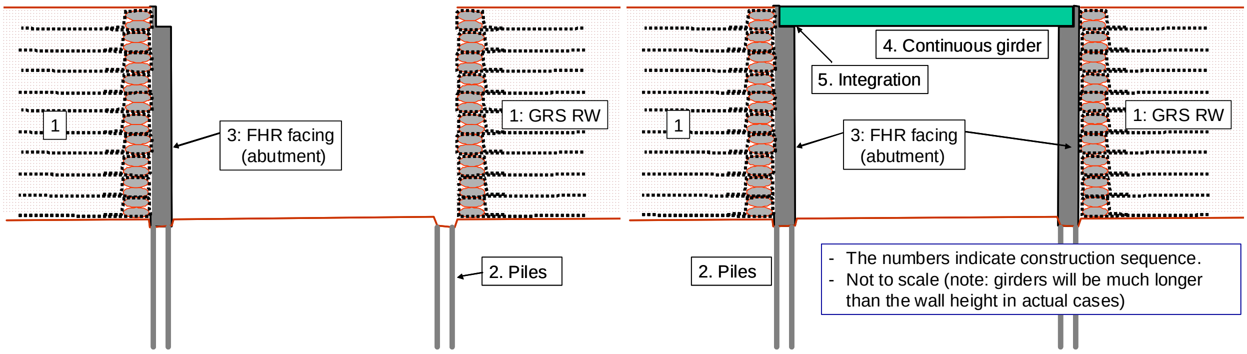


Figure 4. Construction sequence and structure of GRS integral bridge

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

(* Failure acceleration in model shaking table tests)

A= needs for massive abutments because of cantilever structure.
B= needs for piles because cantilever-structural type abutments and construction of backfill after piles & abutments.
C= high cost for construction and long-term maintenance of girder supports (i.e., bearings) and their low seismic stability.
D= long-term residual backfill settlement by self-weight and traffic loads.
D'= long-term residual settlement of sill beam.
E= thermal expansion and contraction of girder, resulting in cyclic lateral displacements of abutment tops and associated increase in the earth pressure and damage to abutments together with large backfill settlement.
F= large backfill settlement and large dynamic earth pressure
G= low seismic stability due to independent performance of two abutments
G'= low seismic stability of sill beam.

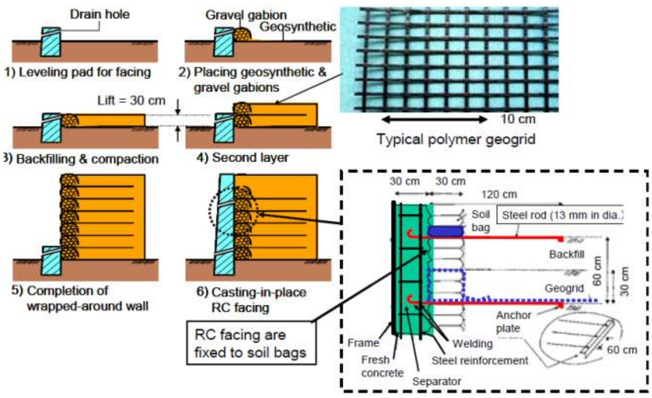
Figure 5. Comparison of different bridge types based on cost & performance

To alleviate these many problems with the conventional bridge type (Fig. 1) as well as these new problems with the integral bridge (Fig. 3a) and the GRS-RW bridge (Fig. 3b), the authors proposed a new type, called the GRS integral bridge (Fig. 4), which combines these two bridge types (Tatsuoka et al., 2007, 2008a & b; Aizawa et al., 2007; Hirakawa et al., 2007). Note that the length of the girder relative to the abutment height depicted in Fig. 4 is much shorter than that of actual GRS bridges. With conventional type bridges (Fig. 1) and GRS RW bridges (Fig. 3b), the length of the girder is restricted to avoid excessive load to be activated to the fixed girder-support. With conventional integral bridges (Fig. 3a), the girder length is limited to be shorter than 60 - 100 m to avoid excessive large cyclic lateral displacements at the top of the abutments by seasonal thermal deformation of the girder. The GRS integral bridge has no such restriction as above. Furthermore, the foundation becomes lighter than the one necessary for conventional type bridges and integral bridges, because of lower load activated to the foundation. Fig. 5 compares the features of these different bridge types. The rating shown in this figure is only an approximation. The score allocated to each item is three, which is reduced one by one when any of the negative factors *A* - *G* is relevant. The accelerations at which the respective bridge models collapsed in the shaking table tests (explained later) are listed in the second column from the right. A total score equal to nine is given only to the GRS integral bridge. This paper reports the results from a number of static and dynamic model tests performed to validate the high static and dynamic performance of the GRS integral bridge, as discussed above.

CHARACTERISTIC FEATURES OF GRS INTEGRAL BRIDGE

The main structural features of the GRS integral bridge (Fig. 4) are: firstly reinforcing of the backfill with geosynthetic reinforcement layers that are firmly connected to the back of the full-height rigid RC facings (i.e., abutments); and secondly the staged construction. A pair of geosynthetic-reinforced soil (GRS) retaining walls (RWs), is first constructed, which is followed by the installation of the abutment pile foundations, if necessary. Then, full-height rigid (FHR) facings, which become RC abutments of the bridge, are constructed by casting-in-place concrete on the wall face. Finally, a continuous girder is placed on the crest of the RC abutments and they are integrated. Such stage construction of GRS RWs as described above originates from the GRS RW technology (Figure 6), which has been standardized in Japan. The GRS RWs (Figure 6) have been constructed at more than 600 sites with a total wall length more than 90 km as of June 2007 in Japan. Its main features are as follows (Tatsuoka et al., 1997):

1) As FHR facing is a continuous beam supported by many geosynthetic reinforcement layers with a small vertical spacing (i.e., 30 cm), the internal stresses in the facing and the stresses concentrated at the facing base are much less than those with conventional RWs, which function as cantilever structures.
2) High tensile force in the reinforcement results from high connection strength at the back of FHR facing, which keeps high the confining pressure therefore providing high stiffness and shear strength in the active zone of the backfill.
3) The backfill construction to a full wall height with a help of gravel gabions placed at the shoulder of each soil layer at a small vertical spacing, 30 cm, between geosynthetic layers facilitates high compaction of the backfill.
4) After a full height of reinforced backfill has been completed and then sufficient deformation of the backfill and supporting ground has taken place, a lightly steel-reinforced concrete facing is cast-in-place directly on the wrapped-around wall face being strongly connected to the reinforced backfill. In this way, a) negative interactions between the FHR facing and the compression of the backfill during the backfill construction can be avoided; b) large compression of soft subsoil can be accommodated; c) the backfill near the wall face can be compacted densely better mobilizing reinforcement tensile force; and d) a good alignment of completed wall face can be easily achieved.



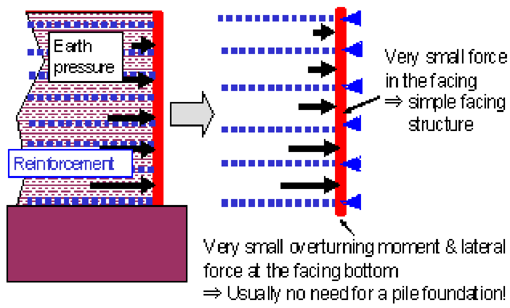


Figure 6 (left). Staged construction of GRW RW with a FHR facing (Tatsuoka et al., 1997)
Figure 7 (right). FHR facing as a continuous beam supported at many levels with a small span.

HORIZONTAL CYCLIC LOADING AT THE TOP OF THE FACIING

A series of static model tests were performed to evaluate the effects of cyclic lateral displacements at the top of the abutments on the performance of the backfill that is either reinforced (i.e., the GRS integral bridge, Fig. 4) or not (i.e., the conventional type integral bridge, Fig. 3a). Fig. 8 shows the model configurations. In test case *R&C*, reinforcement layers are connected to the facing to simulate the GRS integral bridge. In test case *R&No*, the reinforcement layers are not connected to the facing to evaluate the effects of connection between the reinforcement and the facing. The backfill was air-dried poorly graded sub-angular sand, Toyoura sand (D_r= 90 %). The unreinforced backfill was produced by air-blown sand, while the reinforced backfill by hand-tamping. The reinforcement was a Polyester geogrid with a strand width of 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 facing base was: either a) hinge-supported allowing only rotation about the hinge to simulate a FHR facing supported with a pile foundation; or b) placed in the subsoil with a depth of only 3.0 cm to simulate a FHR facing without any foundation. At a distance 11.5 cm down from the top, the FHR facing was cyclically displaced at a displacement rate of 0.004 mm/sec either between the neutral state (i.e., when the displacement at the facing top, d, is equal to zero) and an active state at a positive value of d with a fixed amplitude (D) (case *A*); or between d= D/2 and - D/2 (case *AP*). Cases *A* and *AP* simulate the behaviours of integral bridges completed in summer and fall, respectively. The trends of the backfill behaviour and the earth pressure in these two loading modes (i.e., cases *A* and *AP*) are similar (Tatsuoka et al., 2008b) and only the results in case *A* are reported in the paper.

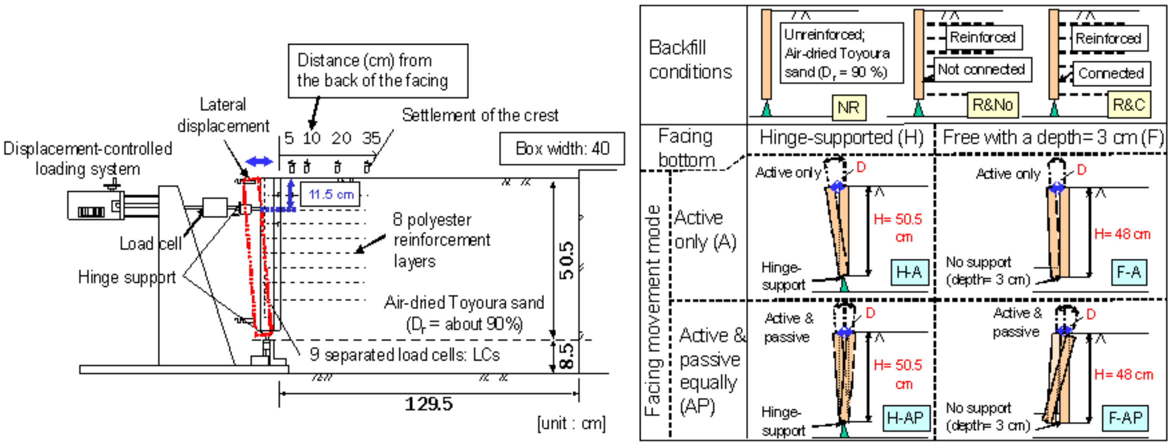


Figure 8. a) (left) Static model tests to evaluate negative effects of cyclic horizontal displacements at the facing top (this model is for case R&C and H-A); and b) (right) model test cases.

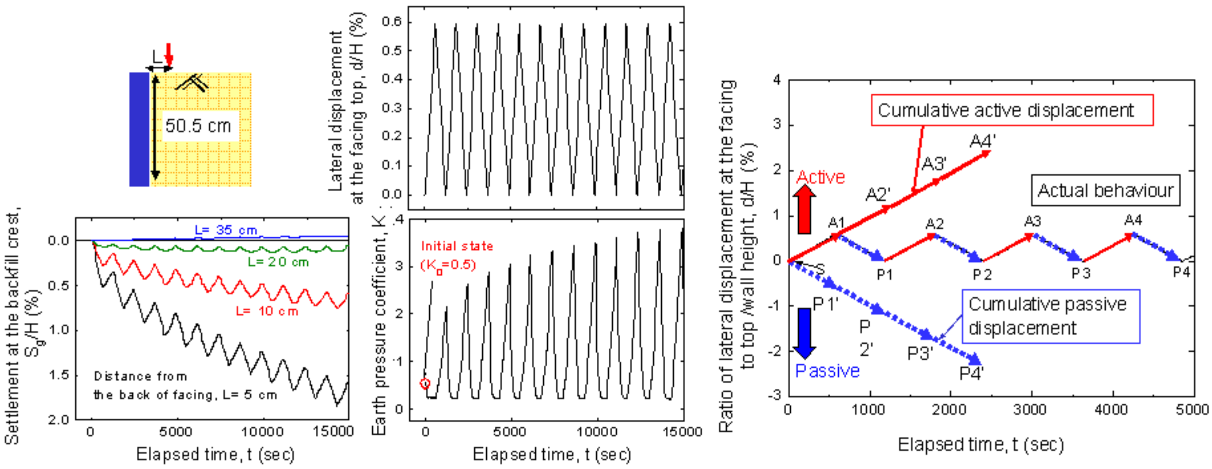


Figure 9. a) Typical time histories of horizontal movement at the facing top, earth pressure and backfill settlement, unreinforced Toyoura sand (case NR and H-A, Fig. 8b); and b) decomposition of actual displacement presented in Fig. 9a according to the concept of dual ratcheting mechanism

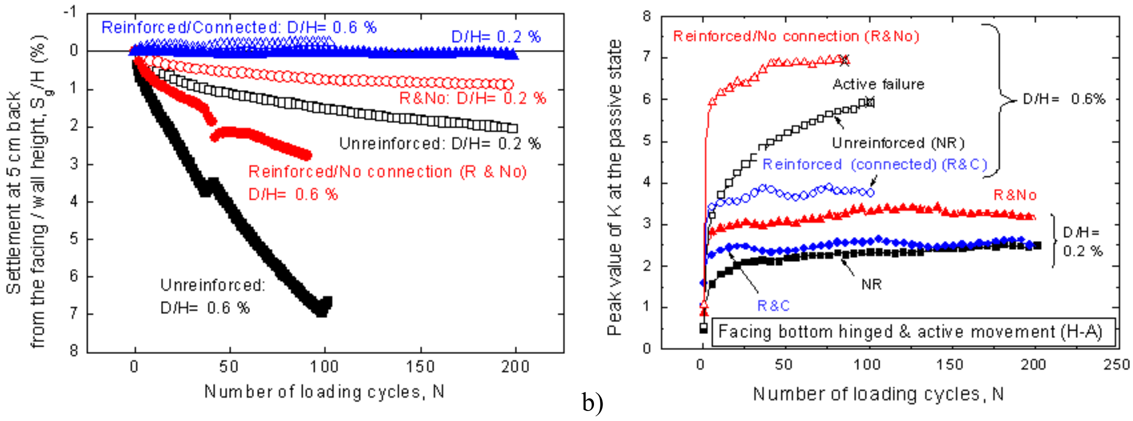
a)  b)

Figure 10. Increase with cyclic loading in; a) backfill settlement; and b) passive earth pressure when D/H= 0.2 % & 0.6 % and the footing bottom is hinge-supported (case H-A, Fig. 8b)

Fig. 9a shows the time histories of the lateral displacement at the facing top, d; the total earth pressure coefficient, $K= 2Q/H^2\gamma$, and the backfill settlement at different distances L from the back of the facing from a typical test on unreinforced backfill when D/H= 0.6 %. Here, Q is the total earth pressure per width of facing; H is the wall height (50.5 cm for the hinge-support and 48 cm for the free condition at the footing bottom); and γ is the dry unit weight of the backfill (1.60 gf/cm^3). It may be seen that the earth pressure increases significantly and the backfill experiences large settlements even by a small amplitude of cyclic lateral displacement of the top of the facing. This trend of behaviour can be explained by the dual ratchet mechanism in the backfill (England et al., 1995, Tatsuoka et al., 2008a, b). That is, by small active displacement of the facing in each cycle, small active sliding develops along the active shear band (e.g., processes S$\rightarrow$A1, P1$\rightarrow$A2 and so on in Fig. 9b). When subjected to small passive displacement of the facing (e.g., processes A1 → P1, A2 → P2 and so on), the active sliding is not activated but the passive deformation of the passive wedge zone, which is much larger than the active wedge zone, is activated with the active wedge deforming as part of the passive zone. Small active sliding in each cycle accumulates with cyclic loading toward the value at which the active failure takes place during monotonic active loading (i.e., the active ratchet mechanism). On the other hand, the passive sliding is inactive when the facing moves toward the active direction (e.g., processes S → A1, P1 → A2 and so on). Small passive strain in the passive zone accumulates with cyclic loading, gradually increasing the passive earth pressure (i.e., the passive ratchet mechanism). As the active displacement of the facing at the active failure during monotonic active loading is very small, the active failure takes place far before the passive failure takes place during cyclic loading. It is also to be noted that the dual ratchet mechanism becomes also active when relative lateral displacements take place between the facing and the backfill in seismic events. Therefore, firm connection of the reinforcement to the facing is essential to achieve high seismic stability of GRS integral bridge.

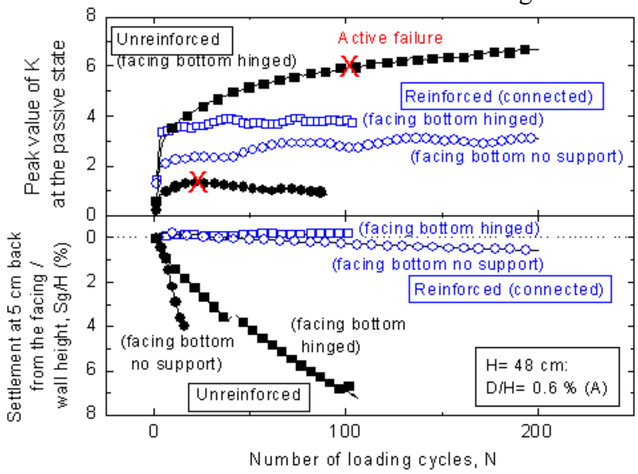


Figure 11. Effects of backfill-reinforcing and facing bottom conditions on passive earth pressure and backfill settlement, D/H= 0.6 and active displacement mode of facing

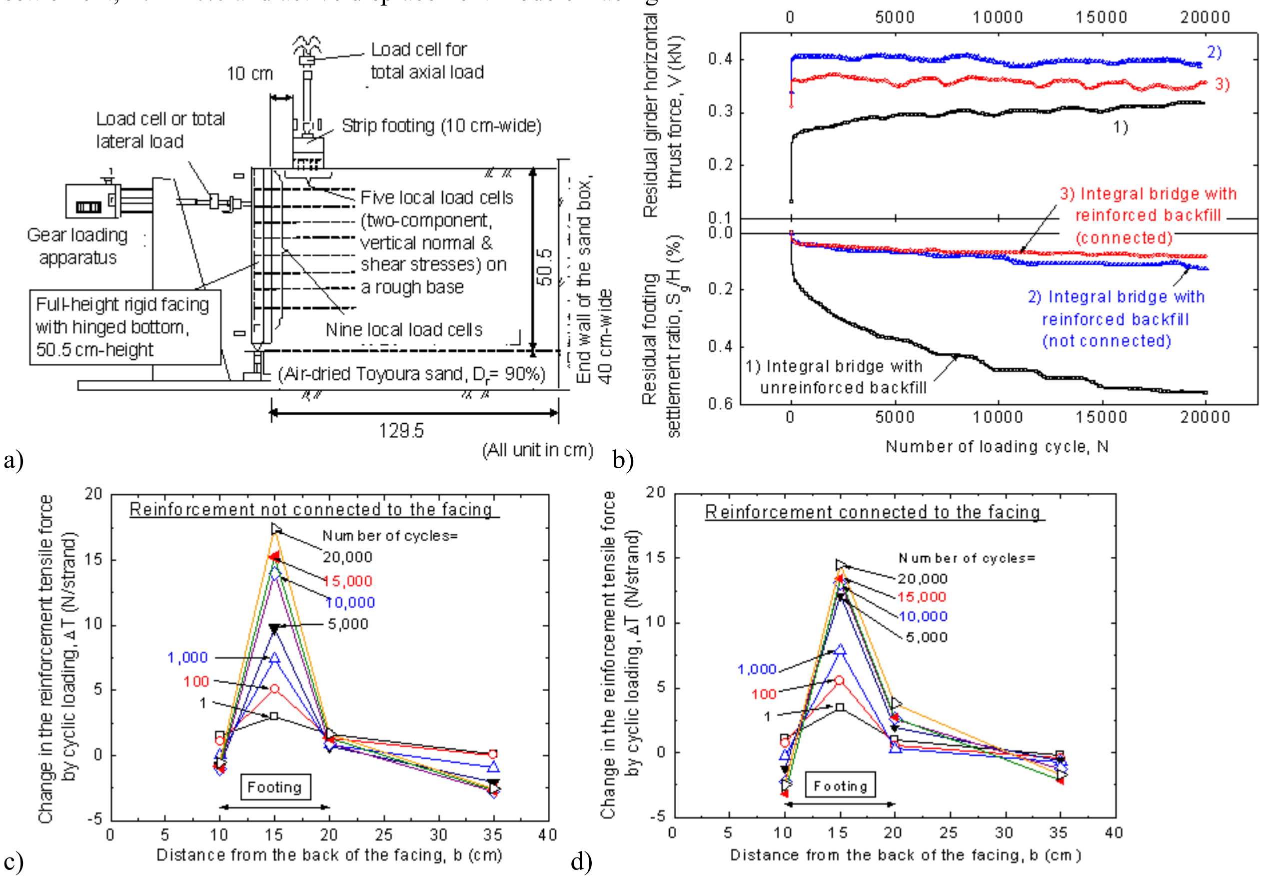


Figure 12. Effects of cyclic vertical loading of a footing on the backfill behaviour; a) model test configurations; b) earth pressure and backfill settlement; and c) tensile force in the reinforcement

Fig. 10a shows the relationship between the backfill settlement at *L*= 5 cm when *d*= 0 and the number of loading cycles. The backfill settlement increases with an increase in *D/H* in the respective test cases. Furthermore, the backfill settlement cannot be effectively restrained by reinforcing the backfill if the reinforcement is not connected to the facing (i.e., case *R&No*). On the other hand, the backfill settlement becomes nearly null when the reinforcement is connected to the facing (i.e., case *R&C*). This is because, when the reinforcement is connected to the facing, the confining pressure in the backfill becomes higher, which makes the backfill less deformable, and the membrane effect of the reinforcement prevents the formation of an active wedge. It may also be seen from Fig. 10a that, when the facing base is free, without support from a piled foundation, the active failure takes place easier in the unreinforced backfill associated with pushing out of the facing base, which increases the backfill settlement and decreases the earth pressure (Fig. 10b). It is also the case with the reinforced backfill if the reinforcement is not connected to the facing, although to a lesser extent. On the other hand, when the reinforcement is connected to the facing, even when the facing base is free, the active failure in the backfill and associated backfill settlement does not take place. These results clearly indicate the importance of connecting the reinforcement to the facing.

VERTICAL CYCLIC LOADING AT THE CREST OF THE BACKFILL

To evaluate the effect backfill deformation due to long-term traffic loads applied to the back of the abutment, on the backfill with geosynthetic layers connected to the facing, cyclic vertical load was applied to a 10 cm-wide strip footing with a rough base placed on the backfill of a bridge model having the facing that was fixed, as shown in Fig. 12a. The other test conditions are the same as the static tests described in Fig. 8a. The footing was allowed to rotate about a hinge at the central axis of the footing and 10 cm above the footing base. Cyclic average footing pressure in a range between 2 kPa and 17 kPa was applied at a period of 15 – 30 seconds. The test results are presented in Figs. 12b and c. The backfill settlement in the unreinforced backfill is particularly large. The settlement decreases by reinforcing the backfill whether the reinforcement is connected to the facing or not. Nearly no effect of the connection of the reinforcement to the facing is due to the fact that the major deformation in the backfill takes place immediately below the footing, as seen from a strong concentration of tensile strain to the part of the geosynthetic immediately below the footing (Fig. 12c). The earth pressure on the facing is initially relatively low in the unreinforced backfill (prepared by air-blowing), but it increases largely by cyclic loading. This factor can destabilize the abutment. The earth pressure is relatively high in the reinforced backfill (prepared by tamping), which does not increase significantly by cyclic loading. These test results indicate that the detrimental effects of traffic load on the performance of the integral bridge can be effectively prevented by reinforcing the backfill. It is very likely that positive effects of the connection of the reinforcement to the facing become significant when the vertical load is applied at locations closer to the facing than in these tests.

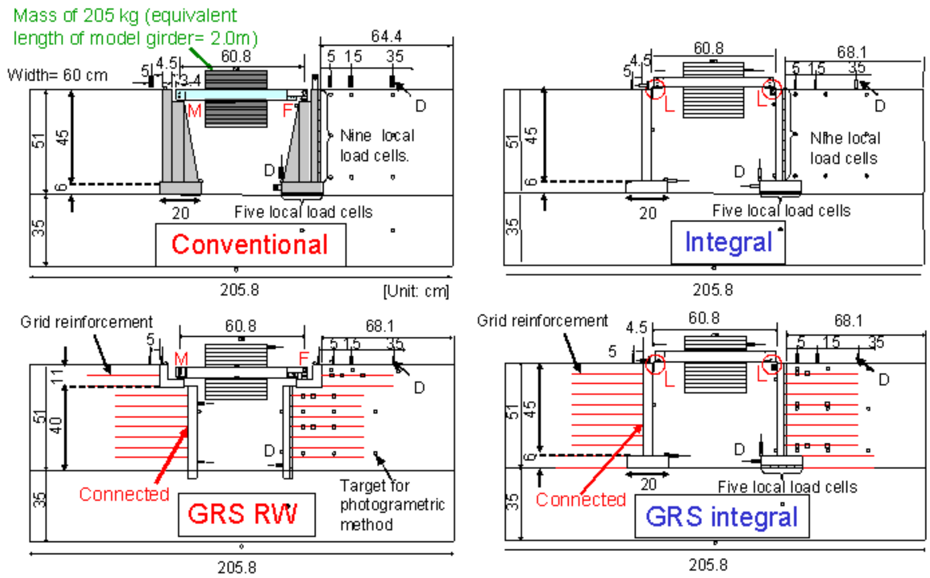
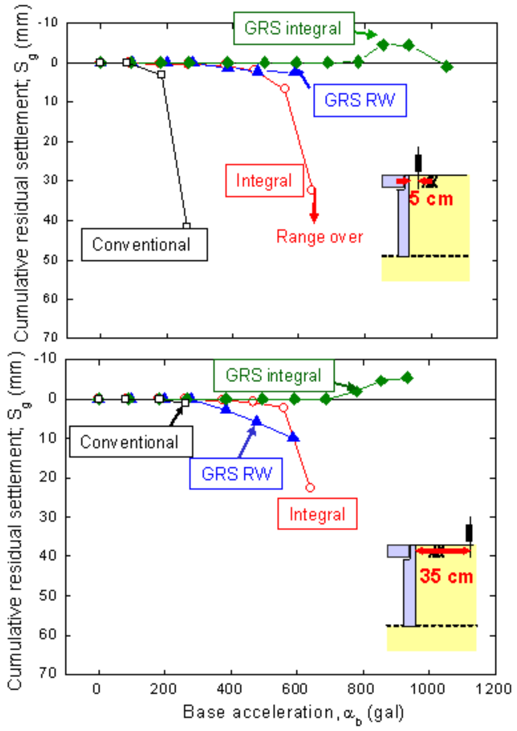


Figure 13. Four models for first series shaking table tests (the backfill is air-dried Toyoura sand with D_r= 90 %); **D**: displacement transducer; **M**: movable sliding support; **F**: fixed hinge support; and **L**: L-shaped metal fixture.

SHAKING TABLE TESTS

Three series of shaking table tests were performed. The first series compared the seismic stabilities of the four bridge types (Fig. 13). Assuming a ratio equal to 1/10, the facing was made 51 cm-high and the girder was made 61 cm-long. By adding a mass of 200 kg at the centre of the model 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 a high rupture strength, 359 N per strand, and a covering ratio of 10.1 %. The surface of the strands were roughened by gluing on sand particles with a friction angle equal to 35 degrees at confining pressure equal to 50 kPa. Twenty sinusoidal waves at a frequency of 5 Hz were applied at the shaking table step by step increasing the maximum acceleration, α_b, with an increment of 100 gal.



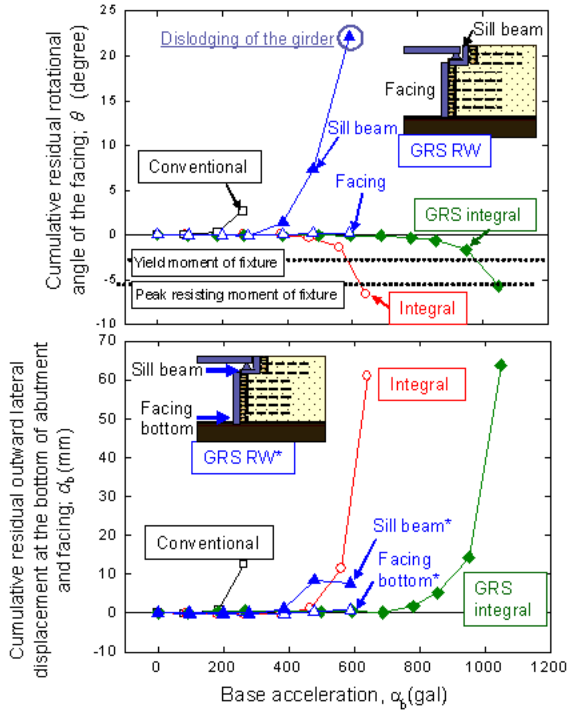


Figure 14 (left) Effects of bridge type on settlements in the backfill 5 cm and 35 cm back the facing, first series.
Figure 15 (right) Effects of bridge type on facing rotation and displacement at the facing bottom, first series.

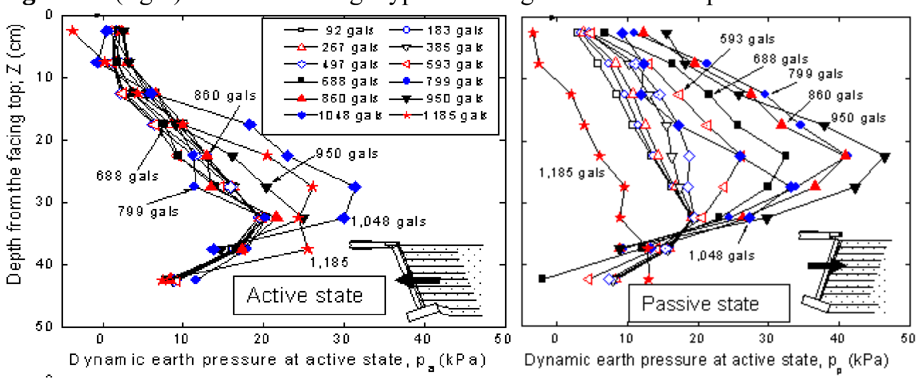


Figure 16. Earth pressure distribution with depth on the facing at 10th cycle at each stage, GRS integral bridge.

Figure 14 shows the backfill settlements at locations 5 cm and 35 cm back from the facing. In the top of Figure 14, the settlement of the sill beam of the GRS RW bridge is presented. Figure 15 shows the overturning angle and the lateral displacements at the bottom of the facing. In the top of Figure 15, the rotational angle of the sill beam of the GRS RW bridge is presented. In Figures 14 and 15, 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 may be observed. Firstly, the GRS integral bridge (with reinforced backfill) is much more stable than the others, while the conventional type bridge (with unreinforced backfill) is the least stable. Secondly, pushing out of the facing base associated with rotation of the facing is the major failure mode with the integral bridge and the GRS integral bridge. Integrating the girder and the facings of the GRS integral bridge is not adequate to sufficiently prevent this displacement mode of the facing (Figure 15). Figure 16 shows the earth pressure distributions on the facing at the 10th cycle at each stage of shaking with the GRS integral bridge. In the upper part of facing, the largest earth pressure at the respective heights is activated when the facing top is under the passive condition. On the other hand, in the lowest part of the facing, the largest earth pressure is activated when the facing top is under the active condition. These trends are due to that the major critical displacement mode of the bridge system is the rotation of the facing relative to the backfill. The two major resisting components for the facing rotation are the passive pressure in the upper part of the backfill and the tensile force of the reinforcement at the lower part of the facing. The former component can be increased by lightly cement-mixing relevant part of the backfill. The latter component is the minimum value among the connection strength, the tensile rupture strength of the reinforcement and the pull-out strength of the reinforcement. Therefore, high connection strength between the reinforcement and the facing is essential for a high seismic stability of GRS integral bridge. In fact, the results from the second series of dynamic model tests (Hirakawa et al., 2007; Tatsuoka et al., 2008b) showed that the seismic stability of the GRS integral bridge decreases with a decrease in the connection strength between the reinforcement and the facing.

Figure 17 shows the results from the third series of dynamic model tests, in which the effects of short piles supporting the facing on the seismic stability of the integral bridge and the GRS integral bridge were evaluated. The model piles had a length of 17 cm, equal to one third of the wall height (45 cm) plus the thickness of the footing base (6 cm). Each group consists of four rows of a pair, (eight in total), of Acrylic round bars with a diameter of 4 cm. The surface of the piles was roughened by gluing sandpaper No. 150 to the surface. The seismic stability of the integral bridge (with unreinforced backfill) increased significantly by using the piles preventing the active movement of the facing base. However, even using piles, significant settlement of the backfill by cyclic thermal horizontal displacements at the facing crest (Figure 10) and cyclic vertical loading on the backfill crest (Figure 12) cannot be prevented when the backfill is not reinforced. On the other hand, the effects of short piles on the seismic stability of the GRS integral bridge was marginal, due probably to that the piles were too short to prevent a global failure with failure planes passing behind the reinforced backfill zone and around the lower part of the piles. The effects may become significant by using longer piles. The test results also indicate that, if the supporting ground is not soft and residual settlement of the facing is not a serious problem, GRS integral bridges can have a sufficiently high seismic stability without a pile foundation.

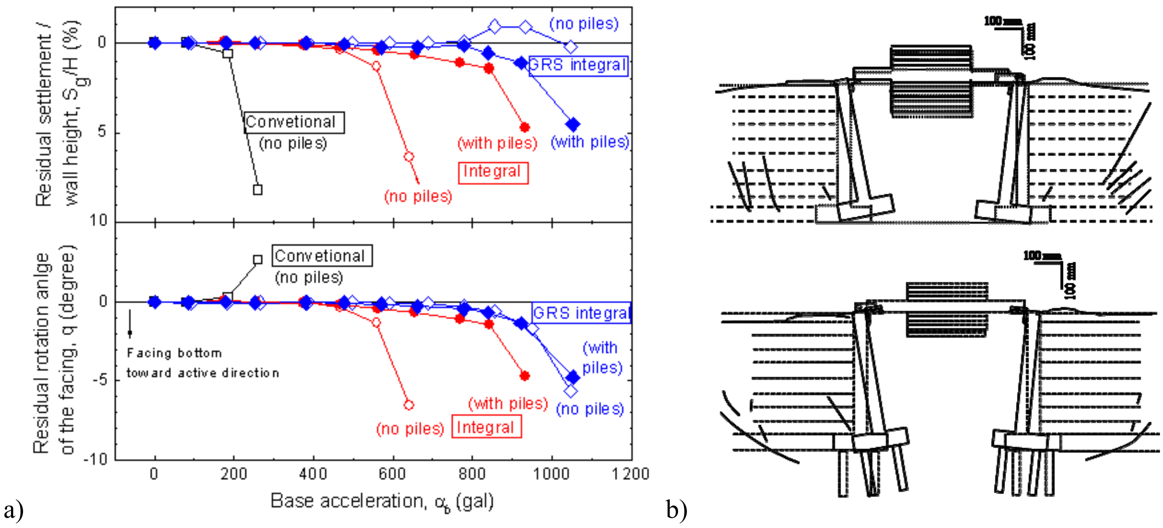


Figure 17. Effects of short piles on: a) backfill settlement and facing rotation of integral bridge and GRS integral bridge; and b) effects of short piles on the failure mode of GRS integral bridge.

CONCLUSIONS

A new bridge system, called the GRS integral bridge, is proposed, which combines the conventional integral bridge and a bridge using geosynthetic-reinforced soil retaining walls as the abutments. By reinforcing the backfill with geosynthetic reinforcement layers firmly connected to full-height rigid facings, integral bridges become very stable and highly cost-effective in construction as well as long-term maintenance. The staged construction of full-height rigid facings after the completion of reinforced backfill makes the construction of this new type bridge practical.

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