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RECENT APPLICATIONS OF GRS TECHNOLOGY TO MITIGATE NATURAL DISASTERS IN JAPAN

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ABSTRACT

The construction of permanent geosynthetic-reinforced soil (GRS) retaining walls (RWs) with a full-height rigid facing for railways, including high-speed train lines, started about twenty five years ago in Japan. The total length of this type of GRS RWs is now more than 125 km, replacing conventional gravity-type RWs or cantilever reinforced concrete RWs and steel-reinforced soil RWs. Many were also constructed to replace conventional type RWs and embankments that collapsed during recent earthquakes, heavy rains, floodings and storms. It is proposed to construct GRS coastal dikes with lightly steel-reinforced concrete facing connected to geosynthetic reinforcement layers as a tsunami barrier. By taking advantage of this technology, a number of bridge abutments with geosynthetic-reinforced backfill were constructed. The latest version, called the GRS integrated bridge, comprises a continuous girder integrated to a pair of RC facing, not using bearings, while the backfill is reinforced with geosynthetic reinforcement layers firmly connected to the back of the facings. The advantages of the GRS integral bridge are presented. It is proposed to apply this new bridge technology to the restoration of conventional type bridges that collapsed by earthquakes, floodings and tsunamis, as well as new construction of those that are highly resistant against these natural disasters.

Keywords: Earthquake, GRS retaining walls, GRS integrated bridge, Heavy rain, Tsunami

INTRODUCTION

Construction of geosynthetic-reinforced soil retaining walls (GRS RWs) and geosynthetic-reinforced steep-sloped embankments has become popular these two decades in Japan. A couple of unique technologies of GRS structure were developed, including several new type bridge abutments comprising geosynthetic-reinforced backfill. A number of such GRS structures as above were also constructed to replace conventional type retaining walls (RWs) and embankments that collapsed during recent earthquakes, heavy rains, floodings and storms. In this paper, it is proposed to construct GRS coastal dikes with a lightly-reinforced concrete facing connected to geosynthetic reinforcement layers as a tsunami barrier. A new bridge system has been proposed, called the GRS integral bridge, with which a continuous girder is integrated to a pair of RC facing, not using bearings, while the backfill is reinforced with geosynthetic reinforcement layers firmly connected to the back of the facings. It is also proposed to apply this new bridge system to the restoration of conventional type bridges that collapsed by earthquakes, floodings and tsunamis, as well as new construction of those are highly resistant against these natural disasters. These proposals have become relevant in particular after the 2011 East Japan Great Earthquake Disaster.

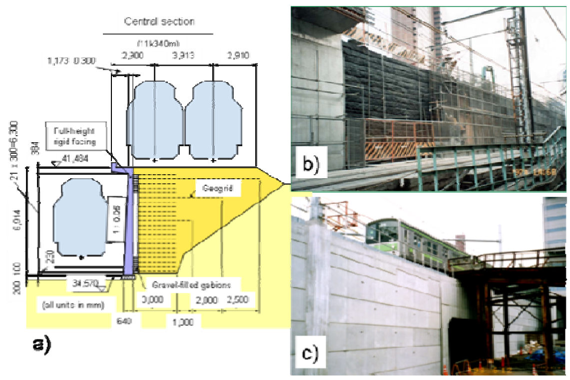


Fig 1. GRS RW having a FHR facing for Yamanote Line (one of the busiest urban railways in Japan) flying over Chuo Line (another busiest urban railway) constructed during 1995–2000 next to Shinjuku station, Tokyo: a) typical cross-section; b) wall under construction; and c) completed wall.

GRS RWS WITH STAGED-CONSTRUCTED FHR FACING: THE BASIC TECHNOLOGY

The GRS RW system having a stage-constructed full-height rigid (FHR) facing is now the standard RW construction technology for railways including bullet train lines in Japan, replacing conventional type RWs (Tatsuoka et al., 1997, 2007; Tatsuoka, 2008). Fig. 1 shows a typical wall. This new type GRS RW has been constructed at more than 850 sites in Japan, and the total wall length is more than 125 km as of March 2011 (Fig. 2).

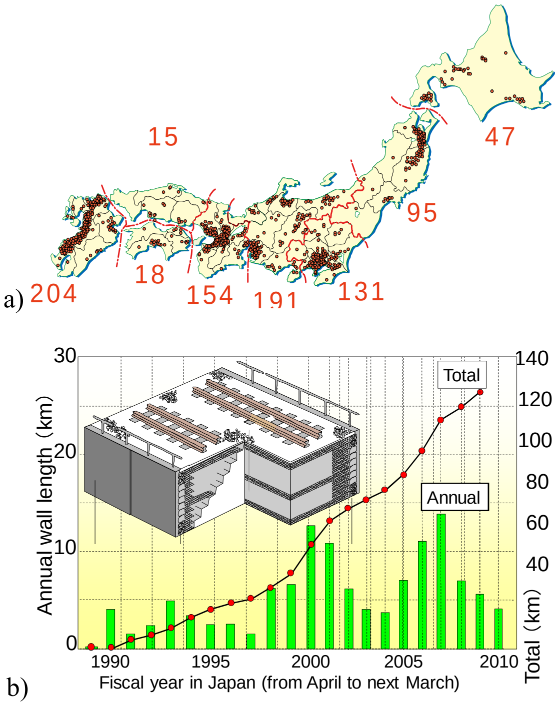


Fig. 2. a) Locations; and b) length of GRS RWs with staged-constructed FHR facing (as of March 2011).

This new type GRS RW has the following characteristic features (Fig. 3):

a) The use of a FHR facing that is constructed by cast-in-place fresh concrete on the wall face wrapped-around with geosynthetic reinforcement (i.e., the staged construction procedure explained below).

b) The use of a polymer geogrid for cohesionless soil to ensure good interlocking and a composite of non-woven and woven geotextiles for high-water content cohesive soils to facilitate both drainage and tensile reinforcing of the backfill. The latter makes possible the use of low-quality on-site soil as the backfill if necessary.

c) The use of relatively short reinforcement, made possible by using planar geosynthetic reinforcement, which has a relatively short anchorage length necessary to activate the tensile forces similar to the tensile rupture strength.

Staged-construction

The staged construction procedure consists of the following steps (Fig. 3a): 1) a small foundation element for the facing is constructed; 2) a full-height GRS wall with wrapped-around wall face is constructed by placing gravel-filled bags at the shoulder of each soil layer; and 3) after the major part of ultimate deformation of the backfill and the subsoil layer beneath the wall has taken place, a thin (i.e., 30 cm or more) and lightly steel-reinforced concrete facing is constructed by casting-in-place fresh concrete directly on the wall face, which makes the FHR facing firmly connected to the main body of the backfill. A high connection strength is essential for high static and dynamic wall stability.

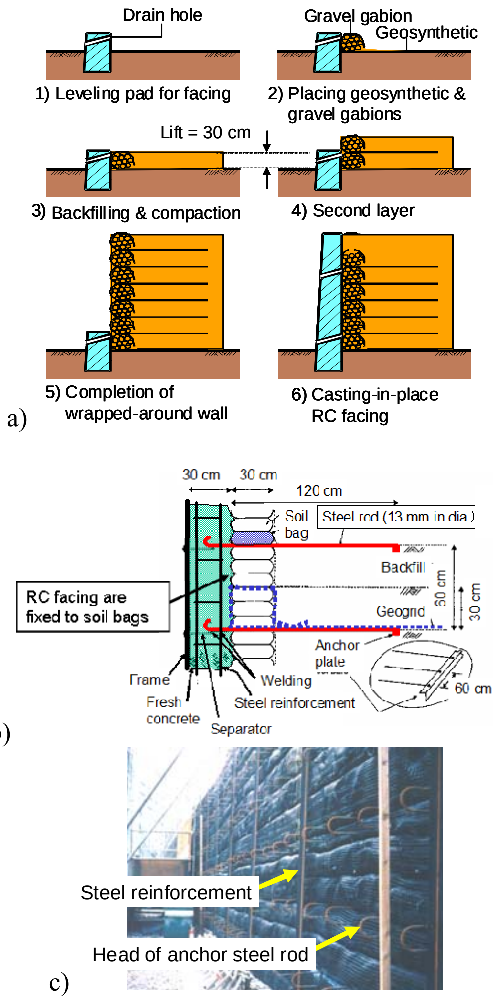


Fig 3. Staged construction of a GRS RW: a) construction steps; b) details of connection between the facing and the reinforced backfill; and c) typical wall face before casting-in-place fresh concrete for FHR facing (Tatsuoka et al., 1997, 2007).

Fig. 3b shows the details of the connection of the FHR facing to the main body of the reinforced backfill and Fig. 3c shows a typical wall face before casting-in-place fresh concrete for a FHR facing at the site presented in Fig. 1. A good connection between the RC facing and the main body of the geosynthetic-reinforced backfill can be ensured by the following two mechanisms. Firstly, the fresh concrete can be easily penetrated into the inside of gravel-filed gabions through openings of the geogrid. Secondly, extra water from fresh concrete is absorbed by gravel inside the gabions, which reduces negative effects of bleeding phenomenon of concrete. The gabions wrapped-around with geosynthetic reinforcement and filled with gravel that are placed at the shoulder of each soil layer function as a temporary facing structure during wall construction and resists against earth pressure generated by compaction, making backfill-compaction more easily, and further backfilling at higher levels of the wall. The gabions function also as a drainage layer after construction and as a buffer that protects the connection between the FHR facing and the reinforcement layers against relative displacements that may take place after construction. Moreover, with conventional cantilever RC RWs, the concrete form on both sides of the facing and its propping become necessary and the cost becomes increases at a high rate as the wall becomes higher. With this new GRS RW system, on the other hand, only an external concrete form without any external propping is necessary while not using an internal concrete form (Fig. 3b).

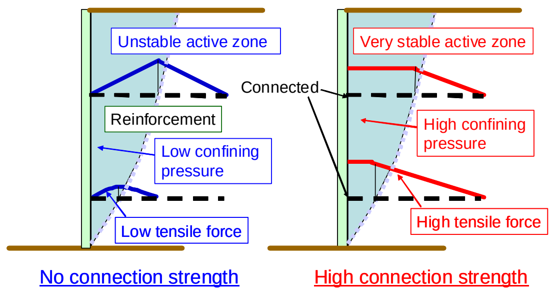


Fig. 4. Effects of firm connection between the reinforcement and the facing (Tatsuoka, 1992).

Importance of firm connection between geosynthetic reinforcement and facing

The importance of firmly connecting the reinforcement layers to the FHR facing is illustrated in Fig. 4. If the wall face is loosely wrapped-around with geosynthetic reinforcement without using gabions or a rigid facing or if the reinforcement layers are not connected to a rigid facing, only very small or nearly no tensile force is activated at the wall face in the reinforcement layers. In that case, no significant earth pressure is activated at the wall face, which means that no significant lateral confining pressure is activated inside the active zone of the backfill. This results in low stiffness and low strength of the active zone, therefore, intolerably large deformation and displacement of the active zone (Fig. 4a). On the other hand, with this new GRS RW system, as the gabions, which are wrapped around with geosynthetic reinforcement layers, function as a stable temporary facing structure, therefore, high earth pressure can be activated at the wall face, even before placing a FHR facing. As wrapping-around geosynthetic reinforcement at the wall face is buried in the concrete layer, eventually the reinforcement layers are firmly connected to the FHR facing. Hence, the earth pressure that has been activated to the temporary facing structure comprising gabions wrapped-around with a geogrid is transferred to a FHR facing consisting of a lightly steel-reinforced concrete layer and a pile of gabions.

The importance of such a firm connection as explained above for high wall stability is illustrated in Fig. 4b. That is, relatively large earth pressure, similar to the active earth pressure that develops in the unreinforced backfill retained by a conventional RW, can be activated on the back of the FHR facing. This high earth pressure at the facing results in high confining pressure in the backfill. Then, high stiffness and high strength of the backfill, which results in high performance of the wall, can be ensured.

The length of geosynthetic reinforcement layers that is required to maintain the stability of GRS RW having staged constructed FHR facing is relatively short when compared to metal strip reinforcement used in walls having a discrete panel facing. This is because: 1) the anchorage length of planar geosynthetic reinforcement to resist against the tensile load similar to the tensile rupture strength of reinforcement is much shorter; and 2) a FHR facing prevents the occurrence of local failure in the reinforced zone of the backfill and in the facing by not allowing failure planes to pass through the wall face at an intermediate height. Factor 2) becomes more important when the backfill is subjected to concentrated load on the top of the facing or immediately behind the wall face on the crest of the backfill.

GRS-RWs as non-cantilever strutures

A conventional type RW is a cantilever structure that resists against the active earth pressure from the unreinforced backfill by the resisting moment and lateral thrust force activated at its base (Fig. 5). Therefore, large internal moment and shear force is mobilized inside the facing structure while large overturning moment and lateral thrust force develops at the base of the facing. A large stress concentration may develop at and immediately behind the toe on the base of the facing, which makes necessary the use of a pile foundation in usual cases. These disadvantages become more serious at a high rate with an increase in the wall height.

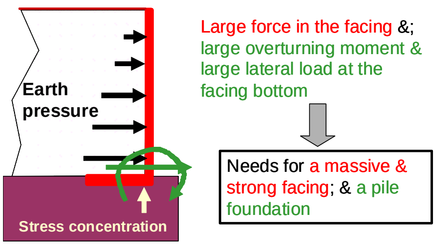


Fig. 5. Conventional type RW as a cantilever structure (Tatsuoka, 1992; Tatsuoka et al., 1997)..

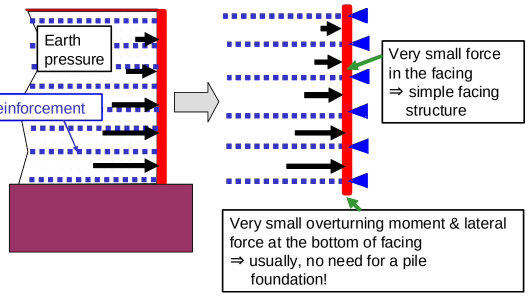


Fig. 6. GRS RW with a FHR facing as a continuous beam supported at many levels with a small span (Tatsuoka, 1992; Tatsuoka et al., 1997).

Relatively large earth pressure, similar to the one activated on the conventional type RW, may be activated on the back of the FHR facing of the new type GRS RW because of firm connection between the reinforcement layers and the FHR facing. Despite the above, as the FHR facing behaves as a continuous beam supported at many levels with a small span, typically 30 cm, only small forces are mobilised inside the facing structure (Fig. 6). Hence, the facing structure becomes much simpler and lighter than conventional cantilever RC RWs. Besides, the overturning moment and lateral thrust force activated at the facing base become small, which makes unnecessary the use of a pile foundation in usual cases. The case histories until today have validated that the GRS RW having stage-constructed FHR facing is much more cost-effective (i.e., much lower construction cost, much speedy construction using much lighter construction machines), therefore a much less total emission of CO_2 than the conventional type cantilever RC RW. Despite the above, the performance of the new type GRS RW is basically equivalent to, or even better than, conventional type cantilever RC RWs.

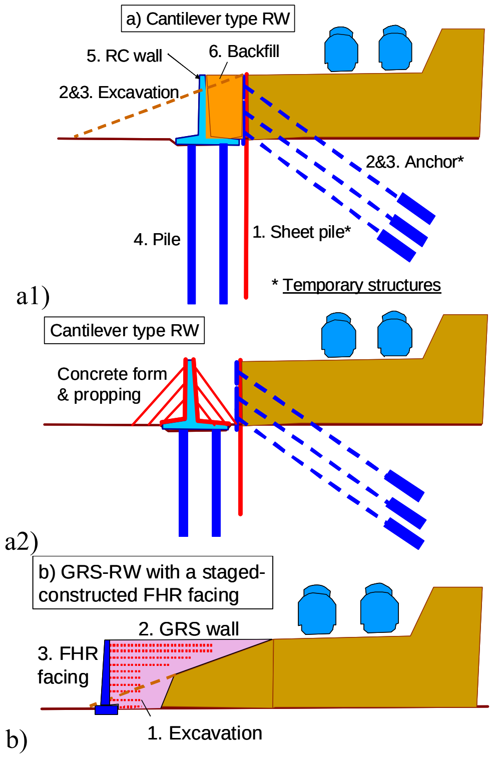


Fig. 7. Reconstruction of a gentle slope of embankment to a vertical wall: a1) & a2) the conventional method; and b) the new method (the numbers indicate construction sequence) (Tatsuoka et al., 1997).

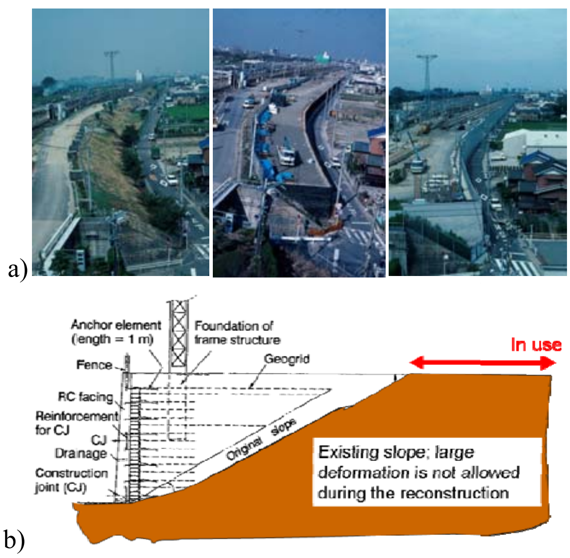
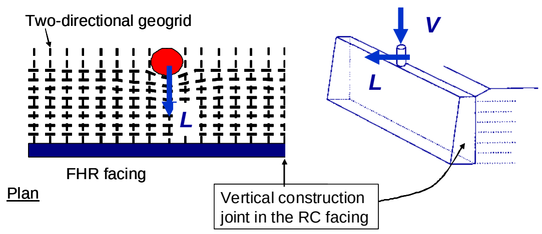


Fig. 8. a) Before, during, and after reconstruction of gentle embankment slope to a GRS RW having a FHR facing; and b) typical cross-section, a yard for bullet trains (Shinkan-Sen), Biwazima, Nagoya; average wall height= 5 m; total length= 930 m; & construction period= 1990 -1991 (Tatsuoka et al., 1997).

Reconstruction of slopes of existing embankments

One of the reasons for a popularity of this new type GRS RW is a high cost-effectiveness when reconstructing gentle slopes of existing embankment to vertical RWs, compared with the conventional method (Fig. 7a). In particular, with the conventional method, when the stiff bearing soil layer is deep, expensive temporary structures (i.e., ground anchor and sheet piles) become necessary. Besides, a concrete form with propping is necessary, which becomes more costly at a high rate with an increase in the wall height.

On the other hand, with the new GRS RW system (Fig. 7b), such temporary structures and concrete forms as mentioned above become un-necessary while the number of construction steps becomes much smaller, the occupied space becomes much smaller and the construction period becomes much shorter. Fig. 8 shows a typical case history showing the reconstruction illustrated in Fig. 7b.



a) b)

Fig. 9. 3-D resistance of FHR facing/geogrid system against lateral load acting to; a) a vertically long structure located inside the reinforced zone; and b) the top of the facing (Tatsuoka et al., 1997).

Moreover, with the new GRS RW system, taking advantage of a FHR facing supported by reinforcement layers for a full wall height, super-structures that may exert large lateral load, such as electric poles and high noise barrier walls, can be constructed either immediately behind the wall face without a deep pile foundation (Fig. 8d), or directly on the FHR facing. In this respect, three-dimensional effects of FHR facing (Fig. 9) make a GRS RW very strong against concentrated load applied to the top of the facing. That is, the FHR rigid facing of this new GRS RW system is continuous not only in the vertical direction but also in the lateral direction. The length of one unit of FHR facing, separated from horizontally adjacent units by vertical construction joints in the facing concrete, is typically 10 m. The whole FHR facing unit can effectively resist against concentrated vertical or lateral load applied to the facing top with a help of all the reinforcement layers that are connected to the facing unit. Moreover, a foundation for a super structure that is embedded inside the reinforced backfill zone may exert large lateral forces on the facing. The FHR facing unit can also effectively resist against such lateral load acting in the direction normal to the wall face as above (Fig. 9b). The features of FHR rigid facing described above become most advantageous when the FHR facing is used as a foundation for a super-structure. The most typical application is bridge abutments made of GRS RWs having staged constructed FHR facing, as described later in this paper.

Summary

No case of collapse and excessive deformation has been reported among many case histories of this new type GRS RW (Fig. 2). This may be attributed mainly to the following factors:

a) A good compaction of the backfill can be ensured because the vertical spacing of geogrid layers is rather small (i.e., 30 cm) and no rigid facing to which reinforcement layers are connected exists during backfill compaction.

b) All potential problems due to deformation and displacements of the backfill and supporting ground can be recognised and dealt with before the construction of a FHR facing.

c) Gabion bags stacked immediately behind the FHR facing ensure a sufficient stability of the wall during wall construction before constructing a FHR facing. The gabions also ensure good drainage. They also function as a buffer when relative displacements take place between the facing and the reinforced backfill during long-term service.

d) A planar geogrid, rather than metal strips (which are much easier to pull out from the backfill), is used.

f) FHR facing prevents the occurrence of overall failure of a wall even when local failure is going to take place in the backfill, the facing and the supporting ground.

g) These GRS RWs (Fig. 2) were designed against high seismic loads. Several GRS RWs with FHR facing performed very well in the severely affected areas of the 1995 Kobe Earthquake (Tatsuoka et al., 1998), as described in the next chapter. The design rupture strength of geogrid is usually determined by the a-seismic design, and, therefore, the design rupture strength is not reduced to account for creep rupture by long-term static loads. Yet, no case history in which the wall has exhibited noticeable creep deformation has been reported. Today, all GRS-RWs having FHR facing is designed using so-called level II design seismic load, equivalent to severe earthquakes motions experienced in Kobe city during the 1995 Kobe earthquake.

COLLAPSE BY NATURAL DISASTER AND RECONSTRUCTION

Collapse by earthquake motions

Numerous embankments and conventional type RWs collapsed by earthquakes, heavy rains, floodings and storm wave actions in the past in Japan (e.g., Fig. 10).

Fig. 10. Gravity type RW without a pile foundation at Ishiyagawa that collapsed during the 1995 Kobe Earthquake (Tatsuoka et al., 1997, 1998).

Fig. 11. GRS RW having a FHR facing at Tanata; a) immediately after construction; and b) one week after the 1995 Kobe Earthquake (Tatsuoka et al., 1997, 1998).

Previously, most of the collapsed soil structures were reconstructed to respective original conventional types despite that they are not cost-effective and their resistance against natural disasters is insufficient (as evident from the fact that they actually collapsed). From the beginning of the 1990's, reconstruction of railway embankments that collapsed by natural disasters to geosynthetic-reinforced steep embankments or GRS RWs having stage-constructed FHR facing or their combinations started based on the successful experiences described in the precedent chapter. High performance during the 1995 Kobe Earthquake of a GRS RW having stage-constructed FHR facing that had been constructed at Tanata validated its high-seismic stability (Fig. 11). Many gentle sloped embankments and conventional type RWs that collapsed by the 1995 Kobe Earthquake and subsequent earthquakes were reconstructed to GRS RWs having stage-constructed FHR facing (Tatsuoka et al., 1977, 1998).

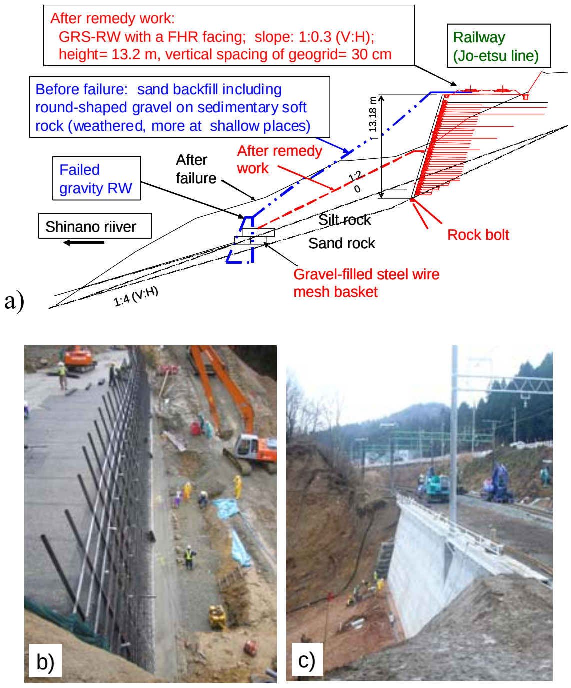


Fig. 12. a) Railway embankment that collapsed during the 2004 Niigata-ken Chuetsu Earthquake and its reconstruction to a GRW RW having FHR facing; b) the wall during reconstruction; and c) the completed wall, Jo-etsu line, East Japan Railway Co. (Morishima et al., 2005).

Fig. 12 shows reconstruction of one of the three railway embankments supported by gravity type RWs on a slope that totally collapsed during the 2004 Niigata-ken Chuetsu Earthquake. GRS RWs having a FHR facing were constructed at these three sites because of much lower construction cost, much higher stability (in particular for soil structures on a steep slope), much faster construction and much smaller earthwork than the reconstruction to the original gently sloped embankments. During this earthquake, road embankments collapsed at numerous places in mountain areas and many of them were reconstructed to GRS RWs or embankments having geosynthetic-reinforced steep slopes.

The March 25th 2007 Noto-Hanto Earthquake caused severe damage to embankments of Noto Toll Road, which was opened in 1978. The north part of this road runs through a mountainous area for a length of 27 km. The damage concentrated into this part, where eleven high embankments filling valleys extensively collapsed (Koseki et al., 2008). The collapsed embankments were basically reconstructed to geosynthetic-reinforced slopes and RWs ensuring good drainage of ground and surface water.

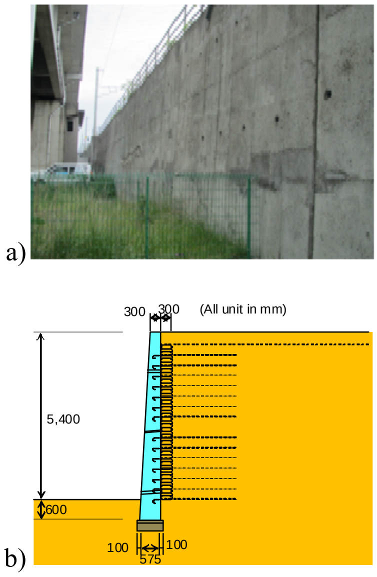


Fig. 13. Geosynthetic-reinforced soil retaining wall have a staged constructed full-height rigid facing for Tohoku line (railway), next to Natori River bridge, Sendai City; a) a view in May 2011 (Okamoto, M.); and c) typical cross-section (by the courtesy of East Japan Railway Co.).

The 2011 Great East Japan Earthquake is the most disastrous earthquake in Japan after the World War II. The damage was a combination of those from earthquake motions and the accompanying great tsunami. A great number of embankments and RWs that had not been designed and constructed following the current seismic design standard collapsed. In comparison, a number of GRS RWs having staged constructed FHR facing described in the preceding chapter that had been constructed in the severely affected areas of this earthquake performed very well (Fig. 13). Several embankments that collapsed were reconstructed to this type of GRS-RWs. Fig. 14 shows one of the three embankments of Iiyama line that collapsed during the Nagano-Niigata Border Earthquake induced one day after the Great East Japan Earthquake and reconstructed to GRS RWs with staged constructed FHR facing. More about these case histories will be reported in the near feature.

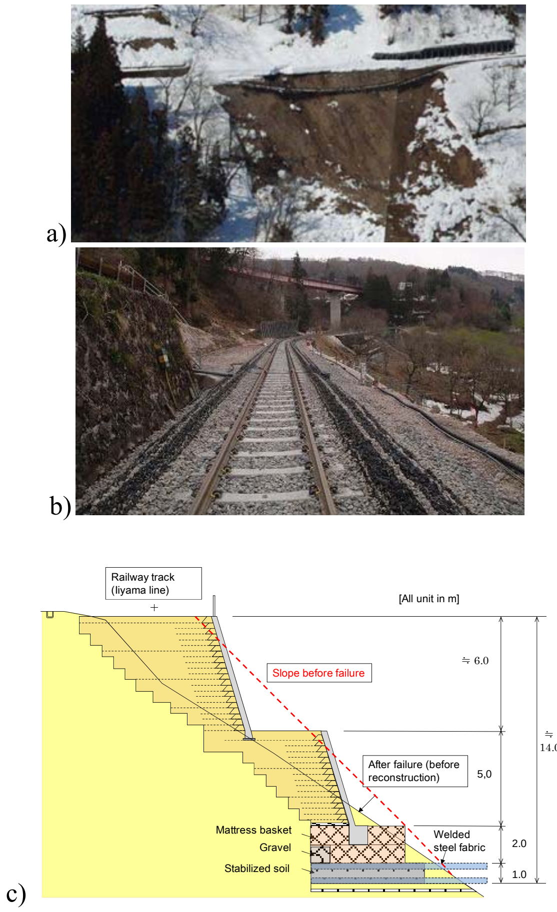


Fig. 14. a) Embankment between Yokokura and Morinomiya stations, Iiyama Line, that collapsed during the Nagano-Niigata Border Earthquake; b) embankment reconstructed to a GRS-RW having FHR facing; and c) typical cross-section before and after reconstruction (by the courtesy of the East Japan Railway Co.)

Collapse by flooding and storm

The GRS-RW technology described in the preceding chapter has also been used to replace soil structures that collapsed by flooding and storm. Fig. 15 shows a typical case. Railway embankments that had been constructed in several narrow valleys collapsed by over-flow of flood water by a heavy rain. These embankments were reconstructed by using the GRS technology. A large diameter drainage pipe was arranged crossing the respective embankments. The interaction between the drainage pipe and the FHR facing after the embankment had been completed was very small, because the FHR facing was constructed after major deformation of the embankment and supporting ground had taken place. This restoration method was employed also in many other similar cases (Tatsuoka et al., 1997).

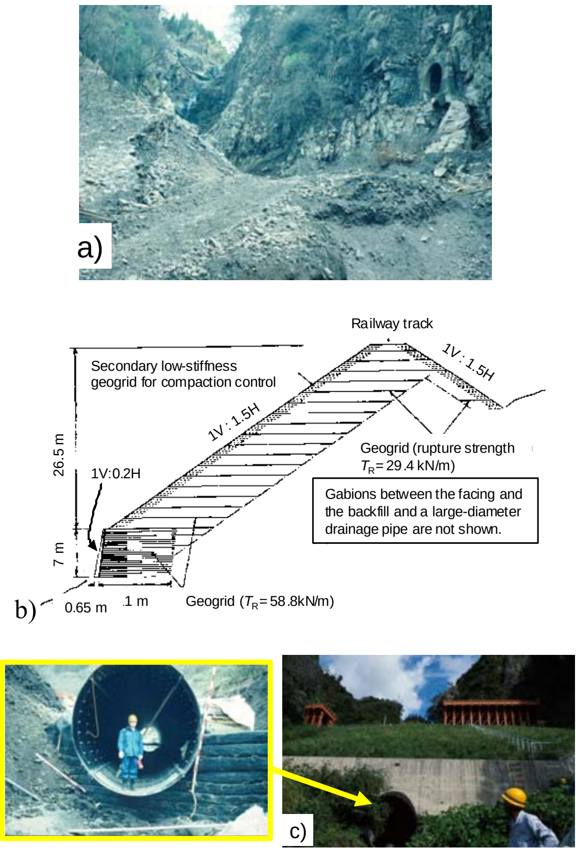


Fig. 15. a) Railway embankment damaged by rainfall in 1989, Kyushu; b) reconstructed cross-section; and c) after reconstruction in 1991 (Tatsuoka et al., 1997; 2007).

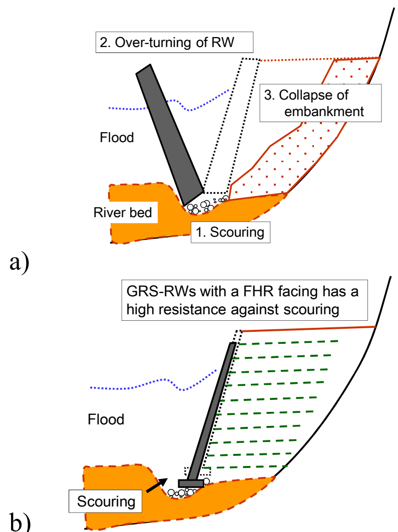


Fig. 16. Schematic diagrams showing: a) collapse of conventional type RW by scouring due to flooding (the numbers show the sequence of events); and b) high performance of GRS-RW with a FHR facing (Tatsuoka, 2008)

A great number of embankments for roads and railways retained by gravity-type or leaning type RWs along rivers and seashores collapsed by flooding, usually over-turning failure of the RWs caused by scouring in the supporting ground (Fig. 16a). This type of failure is easy to take place, because the conventional type RW is a cantilever structure of which the stability is strongly controlled by the bearing capacity at the bottom of the RW (Fig. 5). On the other hand, GRS-RWs with FHR facing is not such a cantilever structure as above (Fig. 6), therefore, much more stable and cost-effective (Fig. 16b). It is particularly important that the backfill can survive even if the facing displaces to some extent by scouring in the supporting ground.

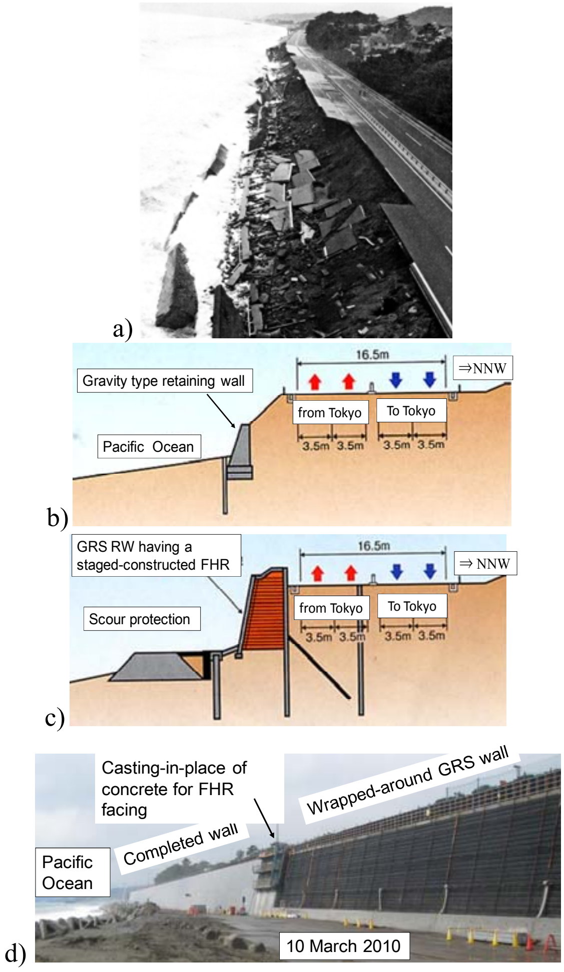
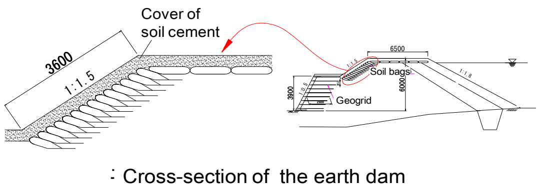


Fig. 17. Seawall for Seisho by-pass of National Road No. 1, Kanagawa Prefecture, southwest of Tokyo: a) collapse for a length of more than 1 km by Typhoon No. 9, 29th Aug. 2007; b) original structure; c) reconstructed seawall; and d) the wall during construction (a, b & c: by the courtesy of the Ministry of Land, Infrastructure, Transport and Tourism, Japanese Government) .

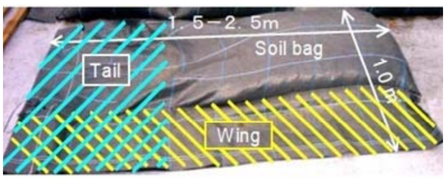
Very recently, a high embankment retained by a masonry gravity-type RW at the lower part on the left bank of Aganoriba, in Niigata Prefecture, for West Ban-Etsu line (a railway of JR East) was collapsed by flooding 30th July 2011 by the mechanism illustrated in Fig. 10a. It was reconstructed to a 9.4 m-high and 50 m-long GRS RW with a FHR facing. Fig. 17a shows this type of failure of a gravity-type RW for a length of more than 1.0 km along a seashore facing the Pacific Ocean, west-south of Tokyo. The failure was triggered by scouring in the supporting ground by strong ocean wave actions during a typhoon September 2007. The wall was restored by constructing a GRS-RW having staged-constructed FHR facing (Figs. 17c & d). The FHR facing has a strong resistance against storm ocean waves, while the wall is stable against scouring in the supporting ground if it takes place. On the other hand, reinforced soil RWs with a discrete panel facing is not relevant in such cases, as the lost of stability of a single panel by, for example, rupture failure at the connections between the reinforcement and the facing panel, or erosion of the backfill from joints between adjacent panels, may easily result into a failure of the whole wall.



a)

b)

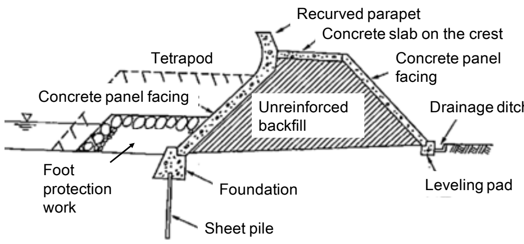
c)



d)

Fig. 18. Restoration of the earth-fill dam of Hirata Ike, Ishikawa Prefecture, that collapsed by the 2007 Noto-Hanto Earthquake (Mohri et al., 2009).

Every year, a number of old small scale irrigation earth-fill dams collapsed by earthquakes and floodings in Japan (Mohri et al., 2009). Fig. 18 shows the restoration of such a dam as above that collapsed by the 2007 Noto-Hanto Earthquake. The spillway section was reconstructed by constructing a GRS RW with FHR facing at the lower part of the downstream slope. Besides, the fill at the higher part was reinforced by using soil bags having a tail and a wing to be better integrated to the adjacent soil bags and the backfill. The soil bags were stacked inclined to have a high resistance against over-flow of flood water and seismic loads (Matsushima et al., 2008).



a)

b)

c)

d)

e) 

Fig. 19. Embankment-type coastal dikes; a) typical cross-section (Technical research group of coastal protection facility, 2004); and the concrete slab at the crest and the top concrete panel facing on the downstream slope lifted up and washed away followed by erosion of the backfill, caused by over-flow of tsunami; b) Koshikiri, Sanriku-cho, Ohunato City; c) Tsugaruishi, Miyako-minami (note: the full-section of dike was lost at sections close to these sites); and d) & e) west of Benten-zaki, Aketo, Tanohata village, Shimo-hei gun, Iwate Prefecture.

RESTORATION FROM DAMAGE BY THE 2011 GREAT EAST JAPAN EARTHQUAKE

To counter the danger of tsunami, multiple tsunami defense systems and relocating residential areas to higher ground have been proposed. In this earthquake, a great number of embankment type coastal dikes were seriously damaged and did not function as expected by the tsunami that significantly exceeded the anticipated height. Most of them collapsed due to erosion of the backfill and foundation failure caused by scouring associated with the overflowing water. Coastal dikes of embankment type are typically faced with concrete panels on three sides: the two sloping surfaces and the crest (Fig. 18a). However, in many cases, the overflowing tsunami caused strong upward suction as water rushed down the landward face, leading to lifting-up and then peeling of the crest concrete slab and the concrete facing on the top portion of the landward side slope, both not tied to the embankment. As soon as this happened, erosion of the embankment began and pealing of the other panel on the landward side slope followed. The dikes damaged as above were further damaged by backwash of the tsunami, leading eventually to complete loss of the cross-section. Figs. 18b and c show two examples of the early stage of this pattern of collapse. Fig. 19 shows a typical case in which the full-cross-section for some length was lost by the over-flow of tsunami.

To prevent such overflow of tsunami as the one during this earthquake, embankments that are high enough (i.e., 15 m or more depending on the site) should be constructed. If conventional type coastal dikes of gently-sloped embankment type are constructed, the base width of the dikes and the quantity of earthworks would be extremely large. Where road and railway embankments are expected to function as secondary tsunami barrier and evacuation location, a specific height will be required. Again, if conventional type embankments with gentle slopes are constructed, they will be wide and involve large quantities of earthworks. Another proposal is to relocate residential areas to higher ground. However, during this earthquake, a great number of embankments and retaining walls in

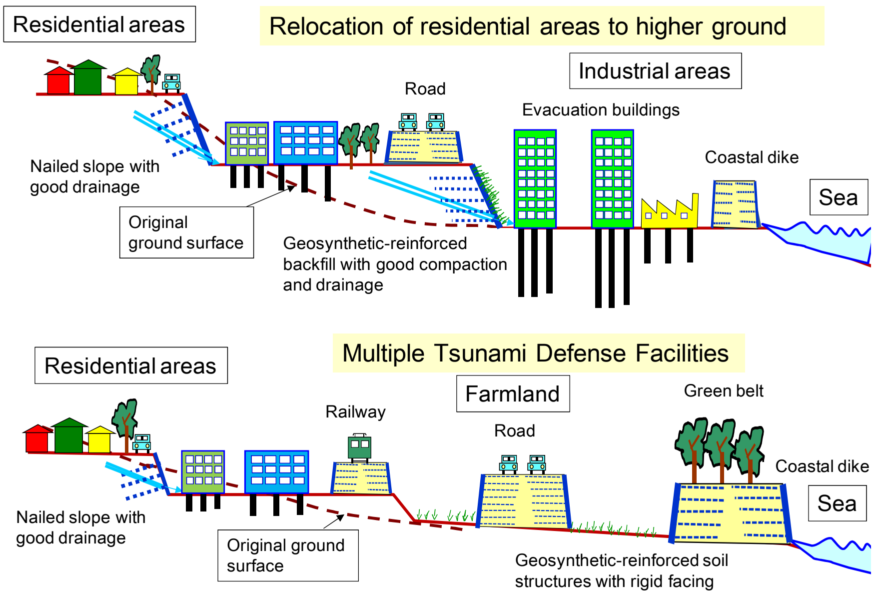


Fig. 20. Multiple tsunami defense facilities and relocating residential areas to higher ground using soil-reinforcement technology (the Japanese Geotechnical Society, 2011).

residential areas that had been constructed using old technologies collapsed in and around Sendai City. To make the projects of multiple tsunami defense facilities and relocating residential areas to higher ground successful, it is proposed to use the technology of GRS-RW with a FHR facing explained in the preceding sections and nailing, in addition to the execution of appropriate compaction control of backfill and provision of suitable drainage. Figs. 20a and b schematically show this proposal.

GRS INTEGRAL BRIDGE

Problems with conventional type bridges

A conventional type bridge comprises a single simple-supported girder supported by a pair of abutments via fixed (or hinged) and moveable bearings, or multiple simple-supported girders supported by a pair of abutments and a single or multiple pier(s) via bearings. The conventional type abutment may be a gravity structure (unreinforced concrete or masonry) or a RC structure. Such conventional type bridges as above have the following many drawbacks (Fig. 21a).

Firstly, as the abutment is a cantilever structure that retains unreinforced backfill (Fig. 5), the earth pressure activated to its back induces large internal forces as well as large thrust forces and overturning moment at the bottom of the abutment. Therefore, usually, the abutment becomes massive. In addition, a pile foundation becomes necessary unless the supporting ground is strong enough. These drawbacks become more serious at an increasing rate with an increase in the abutment height. Secondly, although only small movement is allowed with the abutments once constructed, the backfill are constructed after the abutments have been completed. Hence, when constructed on thick soft ground, many long piles may become necessary to prevent movements of the abutments caused by the earth pressure on the back of the abutment as well as settlement and lateral flow in the subsoil caused by the backfill weight. Moreover, large negative friction may develop along the piles. In many cases in which the soft ground is thick, the piles become much longer than the abutment height. Thirdly, the construction and long-term maintenance of the bearings and the connections between simple-supported girders are generally costly. Moreover, the bearings and girder connections are the weakest portions when subjected to seismic loads. Fourthly, a bump may be formed behind the abutment by long-term settlement of the backfill due to its self weight, traffic loads and seismic loads. Lastly, the seismic stability of the backfill is relatively low and the backfill may deform largely by seismic loads. The seismic stability of the abutment supporting the girder via a fixed bearing is also relatively low: i.e., the girder may dislodge at a moveable shoe.

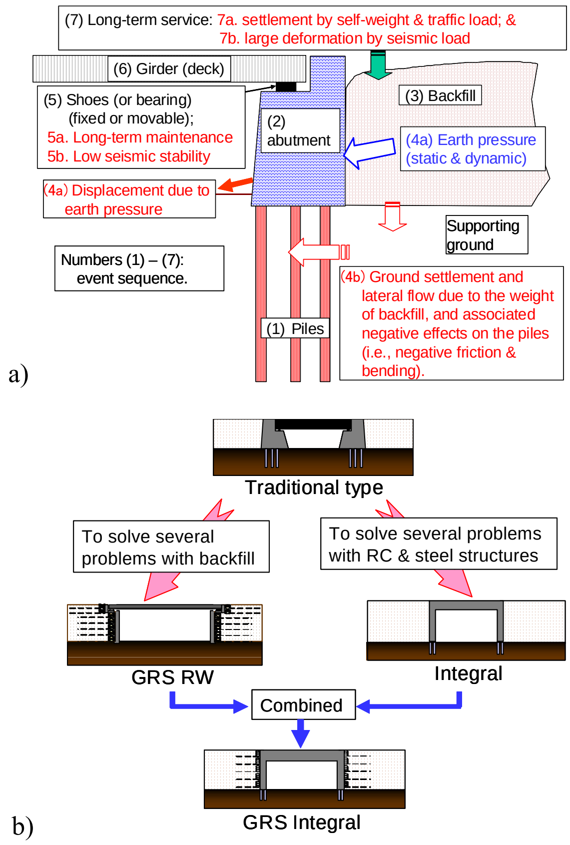


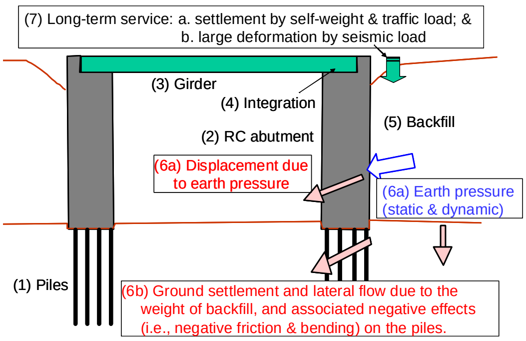
Fig. 21. a) Several major problems with conventional type bridge system; ande b) development of new bridge types (Tatsuoka et al., 2009).

Integral bridge and GRS-RW bridge

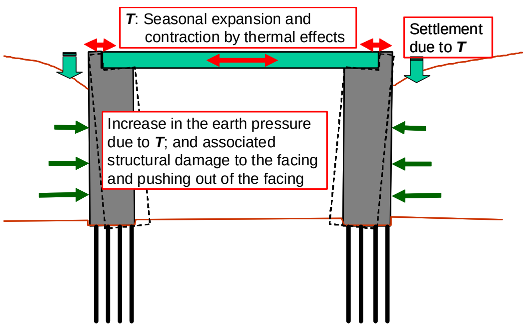
To alleviate these problems with the conventional type bridge described above, three new bridge types have been proposed. Fig. 21b shows the flow of their development.

Firstly, the integral bridge has been developed to alleviate problems with the structural part, usually of reinforced concrete, of the conventional type bridges. This type is now widely used for roads in the UK, the USA and Canada. The main reason for the above is low cost for construction and maintenance resulting from no use of bearings and the use of a continuous girder. Furthermore, the seismic stability of integral bridge is higher than the conventional type (as shown later). However, as the backfill is not reinforced, thus not integrated to the structural part, the backfill and the structural part do not help each other. Therefore, this bridge type cannot alleviate some old problems with the conventional type bridges (Fig. 22a), hence, their long-term performance and seismic stability is not very high.

Moreover, as the girder is integrated to the abutments, seasonal thermal expansion and contraction of the girder results into cyclic lateral displacements at the top of the abutments (Fig. 22b). This loading history results in: 1) development of high earth pressure on the back of the abutment (i.e., the facing); and 2) large settlements in the backfill (England et al., 2000). Tatsuoka et al. (2009, 2010) showed that these trends of behavior are due to the dual ratchet mechanism in the backfill. The effects of daily thermal effects are negligible.



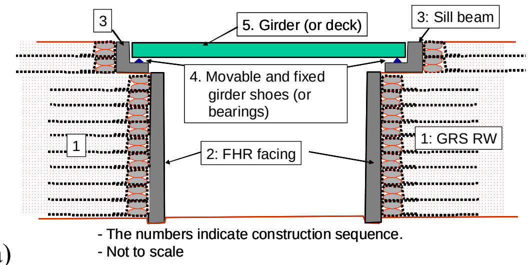
a)



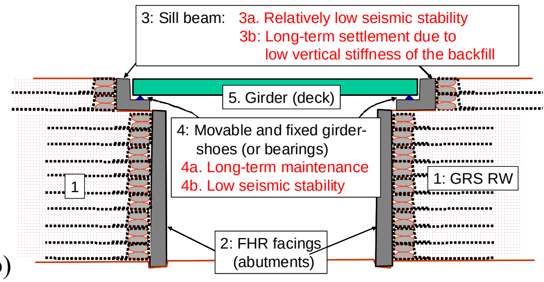
b)

Fig. 22. Integral bridge: a) construction sequence and associated problems; and b) a new problem.

On the other hand, taking advantage of the stage-construction procedure (Fig. 3), a number of bridges comprising a pair of GRS RWs with FHR facing that supports a simple-supported girder via bearings, called the GRS-RW bridge, were constructed (Fig. 23; Tatsuoka et al., 1997). Although this bridge type is more cost-effective than the conventional type (Fig. 20), it has the following drawbacks: 1) The length of the girder is limited due to low stiffness of the backfill supporting the sill beam. 2) The construction and long-term maintenance of bearings is costly. Moreover, the bearings are weak against seismic loads. 3) Although the seismic stability of GRS RWs with FHR facing is very high (e.g., Tatsuoka et al., 1998; Koseki et al., 2006), it is not the case with the sill beam supporting the girder via a fixed bearing, because the mass of the sill beam is much smaller than the girder, while the anchorage capacity of the reinforcement layers connected to the back of the sill beam is relatively small due to their shallow depths.



a)



b)

Fig. 23. GRS-RW bridge: a) construction sequence; and b) unsolved old problems.

It was then proposed to place a girder on the top of the FHR facings via bearings (Watanabe et al., 2002; Tatsuoka et al., 2005). This proposal was adopted by Japanese railway engineers and the first prototype was constructed for a new bullet train line in Kyushu (Fig. 24). The construction started November 2002 and continued for about two months. The abutment was constructed by the staged construction procedure (i.e., the geosynthetic-reinforced backfill is first constructed, followed by the construction of the facing by casting-in-place fresh concrete on the wall face wrapped-around with geogrid reinforcement, Fig. 24c). In this case for a high-speed train, a trapezoidal-shaped zone of backfill back of the facing was constructed using a well-compacted cement-mixed well-graded gravelly soil so that the long-term residual deformation is kept very small and to ensure a high seismic stability (Tatsuoka et al., 2005, 2009). Finally, a girder was placed on the top of a thin RC facing via a fixed bearing.The conventional type RC abutment (Fig. 20) laterally supports the unreinforced backfill, which exerts static and dynamic earth pressures on the back of the abutment. In contrast, with this new type abutment, the reinforced backfill laterally supports a thin RC facing that supports the girder, therefore, the backfill does not exerts large dynamic earth pressure on the facing. After the completion of this project, a number of similar bridge abutments (nearly 60) were designed or have been constructed. Despite the above, it is true that this type of abutment is not free from several problems due to the use of bearings, as illustrated in Fig. 23b.

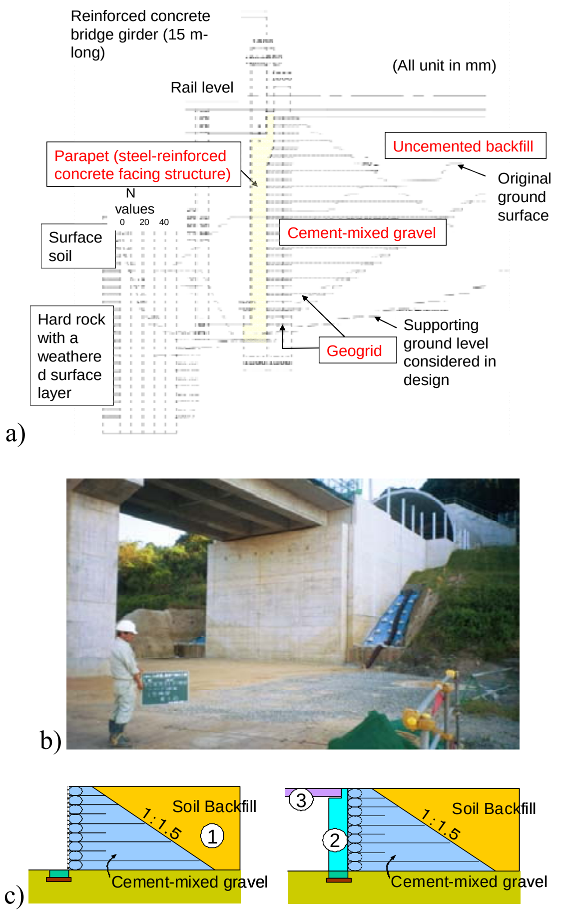


Fig. 24. A bridge abutment at Takada, Kyushu, for a new bullet train line (Tatsuoka et al., 2005); a) structural details; b) completed bridge; and c) staged construction .

Development of GRS integral bridge system

To alleviate the many problems with the conventional type bridge (Fig. 20) as well as the new problems with the integral bridge (Fig. 22b) and those with the GRS-RW bridge (Fig. 23b), Tatsuoka et al. (2008a & b, 2009) proposed another new type bridge system, called the GRS integral bridge (Fig. 25). This is a combination of the integral bridge and the GRS-RW bridge, taking advantage of their superior features while alleviating their drawbacks.

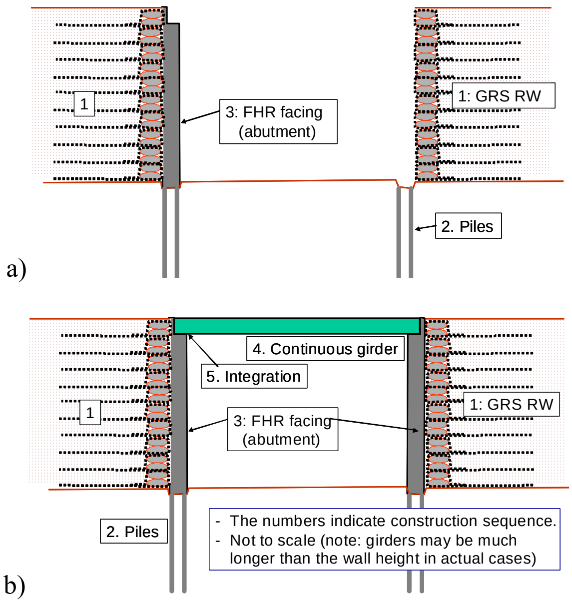


Fig. 25. GRS integral bridge (Tatsuoka et al., 2009).

The GRS integral bridge has the following characteristic features:

1) The backfill is reinforced with geosynthetic reinforcement layers that are firmly connected to the back of the FHR facings (i.e., the abutments). If it is necessary to ensure very small deformation during long-term service and very high performance during severe earthquakes, part of the backfill immediately back of the facing is improved by cement-mixing.
2) The abutments are constructed by the following staged construction procedure (Fig. 25):
   a) A pair of GRS walls with the wall face wrapped-around with geogrid reinforcement is firstly constructed.
   b) If the supporting ground is soft and weak, a zone in the supporting ground below the facing may be improved by, for example, cement-mixing. Otherwise, a pile foundation to support the FHR facings may be constructed. If the deformation of the supporting ground by the construction of the backfill is not significant, the pile foundation may be constructed before the construction of the GRS walls for better constructability.
   c) FHR facings are constructed by casting-in-place fresh concrete on the wall face wrapped-around with geogrid reinforcement.
   d) A continuous girder is placed on the crest of the facings and integrated to the facings.

This staged construction procedure is a modification of the one described in Fig. 3, therefore, it has the same advantage. Firstly, the connection between the reinforcement and the facing is not damaged by differential settlement between the facing and the backfill during wall construction. Then, construction of abutments on relatively compressible subsoil becomes possible without using heavy piles. Secondly, by compacting well the backfill allowing sufficient outward movements at the wall face, sufficient tensile forces can be mobilized in the reinforcement during the construction of geosynthetic-reinforced backfill (before constructing FHR facings).

With conventional type bridges (Fig. 20) and GRS-RW bridges (Fig. 23), the length of a single simple-supported girder is restricted to avoid excessive lateral seismic loads to be activated to the abutment on which a fixed bearing supports the girder. With integral bridges (Fig. 22), the girder length is limited to avoid excessive large cyclic lateral displacements at the top of the abutments by seasonal thermal expansion and contraction of the girder. The girder length is restricted also to limit the lateral seismic load activated at the top of the abutments. On the other hand, with the GRS integral bridge, such restrictions as above are much looser, therefore, the actual length of the girder relative to the abutment height could be much longer than the one depicted in Fig. 25. The girder length limit for GRS integral bridges would be larger than the value for conventional type integral bridges, which is presently specified to be 50 - 60 m in the USA to restrict the maximum thermal deformation of the girder to four inches (about 10 cm). More research will be necessary in this respect.

For last six years, many series of model tests (cyclic or static loading tests and shaking table tests) were performed to validate the advantageous features of the GRS integral bridge system over other types of bridge system (Tatsuoka et al., 2009). Fig. 26 shows small models for 1g shaking table tests of the four bridge types illustrated in Fig. 21a. The considered scaling factor for length between these models and corresponding prototype bridges is 10.

The supporting ground and backfill were produced by pluviating air-dried Toyoura sand (e_{max}= 0.970; e_{min}= 0.602; G_s= 2.65; U_c= 1.64; and D_{50}= 0.179 mm) to obtain a relative density D_r of about 90 %. Polymer geogrid reinforcement used for actual full-scale structures was simulated by a regular grid comprising longitudinal members (made of thin and narrow phosphor-bronze strips, 0.2 mm-thick and 3 mm-wide, having a rupture strength 359 N per strip) welded at nodes to transversal members (made of mild steel bar, 0.5 mm in diameter). The surface of the strips was made rough by gluing particles of Toyoura sand. The models were densely instrumented at many locations relevant to monitor displacements, earth pressures and accelerations (and tensile forces in the model reinforcement layers when used). The locations of the accelerometers are indicated in Fig. 26c.

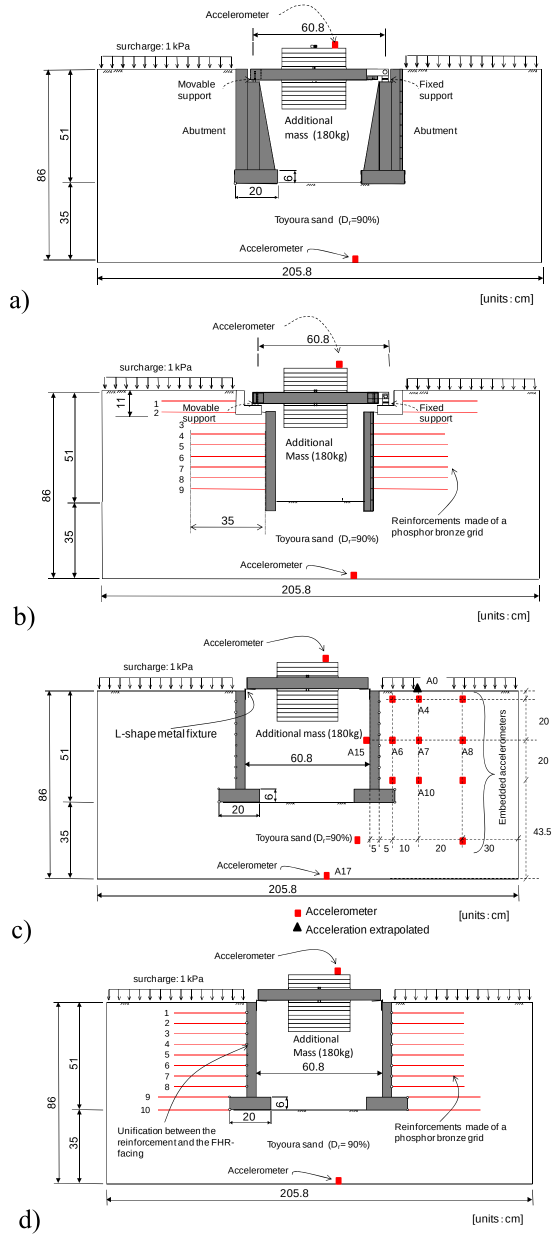


Fig. 26. Small bridge models for 1g shaking table tests: a) conventional type bridge; b) GRS-RW bridge; c) integral bridge; and d) GRS integral model (Tatsuoka et al., 2009; Munoz et al., 2012).

The bridge models were subjected to sinusoidal base acceleration motion at a frequency f_i= 5 Hz having twenty cycles at each stage. The acceleration amplitude was increased stage by stage with an increment of 100 gal from 100 gal until failure took place. f_i =5 Hz was selected to be lower than the initial natural frequencies (f_0) of the models by considering the fact that the typical predominant frequencies of strong earthquake motions in the full scale (1 – 3 Hz) are lower than the natural frequencies under undamaged conditions of the full-scale bridges, including GRS integral bridges, examined in this study.

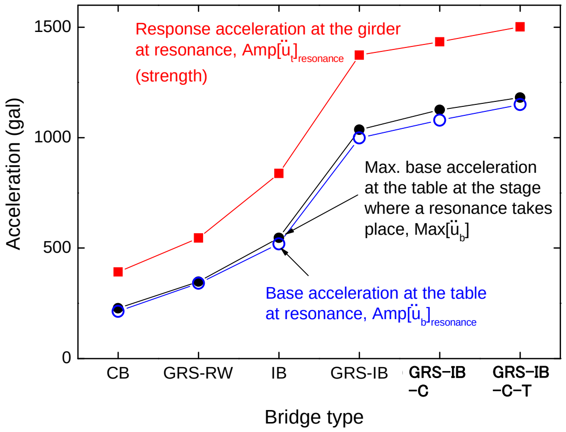


Fig. 27. Dynamic strengths of the bridge models; CB (conventional type bridge); GRS-RW bridge (GRS-RW bridge); IB (integral bridge); and GRS-IB (GRS integral bridge) (Munoz et al., 2012).

Fig. 27 compares the base accelerations at resonance, Amp[ü$_b$]$_{resonance}$,, and the response acceleration at the girder, Amp[ü$_t$]$_{resonance}$,, when failure started taking place, with all the bridge models illustrated in Fig. 26. For reference, the maximum base accelerations, Max[ü$_b$], at the stage during which resonance took place are also plotted. It may be seen that the dynamic strength defined as the response acceleration when failure starts taking place increases consistently from model CB to model GRS-RW by reinforcing the backfill (while using bearings), from model CB to model IB by integrating the girder to the facings (with unreinforced backfill), and from model IB to model GRS-IB by reinforcing the backfill with geogrid layers connected to the facing. It was examined whether the seismic stability of GRS integral bridge increases by arranging a lightly cement-mixed sand zone immediately behind the facings (i.e., models GRS-IB-C and GRS-IB-C-T). It may be seen that the dynamic strength noticeably increases by cement-mixing part of the backfill (in particular in a trapezoidal shape, GRS-IB-C-T). Among the increases in the dynamic strength by the different factors, the one by reinforcing the backfill of IB model is largest.

The most critical failure mode with the integral bridge and the GRS integral bridge is the rotation of the RC FHR facing relative to the backfill with displacements at the base in the active direction about its top, as typically seen from Fig. 28a (Tatsuoka et al., 2009). The major and secondary load and resistance components with the GRS integral bridge are summarized in Fig. 28b. 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 lower part of the facing. The model GRS-IB-C-T was tested to examine the effectiveness of the increase in the resistance of the backfill against passive pressure.

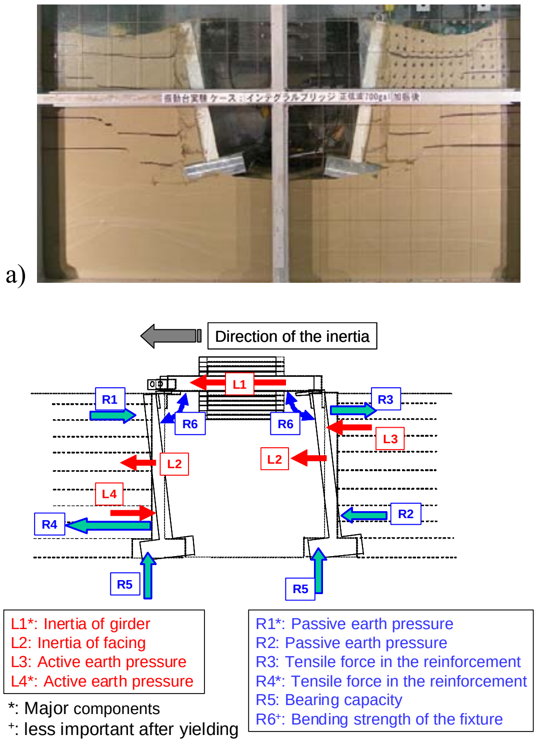
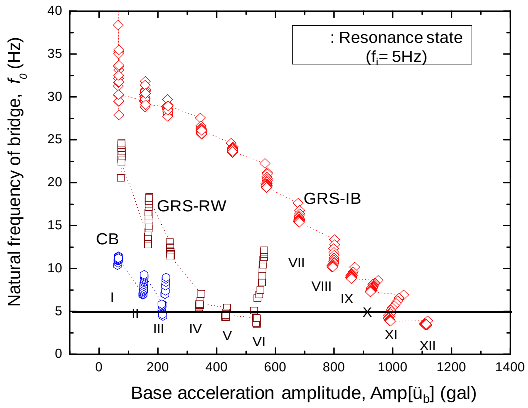
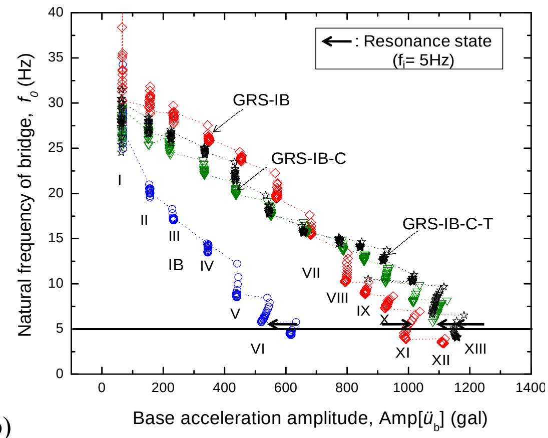


Fig. 28. a) Failure mode after collapse at stage VII with the integral bridge model; and b) load and resistance components for facing rotation, GRS integral bridge (Tatsuoka et al., 2009).

Serious dynamic failure and full collapse of a structural system may take place when the system resonate, or dynamically largely responds, to an input motion after the natural frequency (f_0) has decreased to the predominant frequency (f_p) of the input motion, or a value close to f_p, from an initial value higher than f_p by structurally deteriorating during dynamic excitation. The deterioration is by non-linear behaviour of the structural components, their connections, the backfill and the supporting sub-soil. It was also the case with the model tests in the present study. That is, as shown in Fig. 29, the natural frequency (f_0) of the respective models decreases with an increase in the number of loading cycles and an increase in the base acceleration due to the non-linear behavior of the bridge system. The respective transient values of f_0 at each moment were obtained from the acceleration amplification ratio and phase difference between the accelerations measured at the shaking table and the model girder (at the locations indicated in Fig. 26) by dealing with the respective models as a single-degree-of-freedom (SDOF) system.



a)



b)

Fig. 29. Relationships between transient natural frequency f_0 and Amp[ü$_b$] of bridge models: a) CB, GRS-RW and GRS-IB; and b) IB, GRS-IB, GRS-IB-C and GRS-IB-C-T (Munoz et al., 2012).

The following trends may be seen from Fig. 29:

1) The initial values of f_0 at the start of shaking increases markedly from about 10 Hz (CB) to about 25 Hz (IB & GRS-RW), then to about 35 Hz (GRS-IB). This increase results in a marked decrease in the initial ratio of f_i (input frequency) to f_0, thereby a decrease in the initial amplification ratio. The effects of cement-mixing of part of the backfill on the initial value of f_0 are negligible.

2) The decreasing speed of the f_0 value with an increase in the base acceleration amplitude Amp[u$_b$] in the course of decrease in f_0 toward a value at resonance (equal to f_i= 5 Hz) is similar among the different models. Then, due to a large difference in the initial values of f_0 value ($[f_0]_{initial}$) among the different models, the decreasing rate, $\{df_0/[f_0]_{initial}/d\{Amp[u_b]\}$, is larger in the order of CB (largest), GRS-RW and IB and GRS integral bridge (smallest). Consequently, the resonant state, at which failure starts, is reached earlier at a smaller input acceleration in the order of CB (earliest), GRS-RW and IB and GRS integral bridge (latest). Cement-mixing of part of the backfill has some positive effects on this trend.

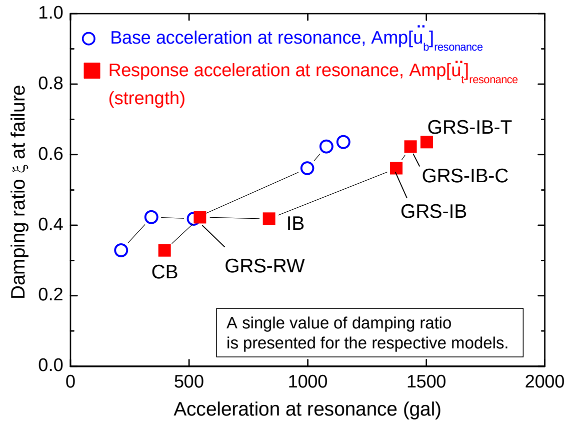


Fig. 30. Relationship between the damping ratio ξ and base acceleration at resonance (Munoz et al., 2012)

The damping ratio (ξ) of a structural system represents the capability of: 1) dissipating the dynamic energy of the system (mainly of the girder and facings in the present case) toward the outside of the system; and 2) consuming the dynamic energy inside the system (mainly in the backfill in the present case). The response acceleration at resonance for a given input motion decreases with an increase in the damping ratio. The values of ξ at resonance (i.e., the value when failure started in the present case) of the bridge models were evaluated as SDOF systems and have been plotted is Fig. 30. It may be seen that the ξ value is larger in the order of GRS IB (largest), GRS-RW and IB, and CB (smallest). The ξ value increases slightly by cement-mixing the backfill of the GRS integral bridge.

In summary, the dynamic stability of a given bridge system increase with an increase in the following three factors:

1) Dynamic strength defined as the response acceleration (or response acceleration) when failure starts (Fig. 27).

2) The initial value of natural frequency (f_0), which increases with an increase in the initial stiffness of a given bridge system (Fig. 29).
3) The dynamic ductility (i.e., a higher dynamic ductility means that f_0 decreases toward the predominant frequency of input motion at a lower rate, $\{df_0/[f_0]_{initial}/d\{Amp[u_b]\}$, Fig. 29).
4) The capacity of energy dissipation (Fig. 30).

The test results described above show that all of the four factors increase by integrating the girder to the facings and the backfill to the facings. That is, the GRS integral bridge is much more dynamically stable than the other bridge types, in particular the conventional type bridge.

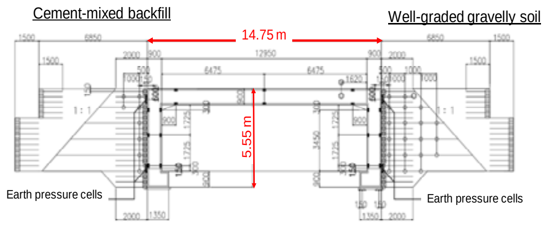
Bridge type	Cost & period of construction	Maintenance cost	Seismic stability	Total
Conventional	1 *A, B*	1 *C, D*	1 *F, G* 252 gal*	3
Integral	2 *B*	1 *D, E*	2 *F* 641 gal*	5
GRS RW	3	1 *C, D'*	2 *G'* 589 gal*	6
GRS Integral	3	3	3 1,048 gal*	9

(* Shaking table acceleration when the model fully collapsed)

A= needs for massive abutments because of cantilever structure.
B= needs for piles because of cantilever-structural type abutments and construction of backfill after piles & abutments.
C= high cost for construction and long-term maintenance of girder shoes (i.e., bearings) and low seismic stability.
D= long-term backfill settlement by self-weight and traffic loads.
D'= long-term settlement of sill beam.
E= cyclic lateral displacements of abutment top caused by thermal expansion and contraction of girder, resulting in high earth pressure and 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 the sill beam.

Fig. 31. Ratings of four different bridge types (a higher point means better performance & higher performance/cost ratio; Tatsuoka et al., 2009).

Fig. 31 compares the characteristic features of the four different bridge types illustrated in Fig. 21a. The ratings presented in this figure are only an approximation showing a general trend. The full point allocated to each item is three, which is reduced one by one when any of the listed negative factors *A* - *G* is relevant. The horizontal accelerations at which the respective bridge models fully collapsed in the shaking table tests are listed in this table. A total full point equal to nine is given only to the GRS integral bridge. It may be understood that the GRS integral bridge is superior in many aspects over the other bridge types. One of the other factors that are not considered in this evaluation is tsunami force, which is discussed in the next section.



a)

b)

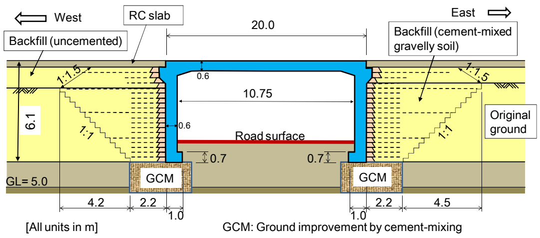
c)

Fig. 32. A full-scale model of GRS integral bridge at Railway Technical Research Institute, Japan: a) dimensions; b) one of the abutments under construction; and c) completed model.

A full-scale model of GRS integral bridge was constructed at Railway Technical Research Institute in 2008 - 2009 (Fig. 32). The abutments were constructed in a pair of full-scale models of GRS-RW with a FHR facing constructed in 1998. A high constructability of the GRS integral bridge system was confirmed. The long-term behaviour is now being observed.

In 2011, the first prototype GRS integral bridge was constructed for a new bullet train line at the south end of Hokkaido (Fig. 33). It was estimated that the construction cost for this GRS integral bridge is about an half of the one for a box girder type bridge (the most conventional solution in this case) and less than a half of the one for a GRS RW bridge of the type described in Fig. 20. The bridge is heavily instrumented to observe the behaviour during construction and after opening to service. Although the span is not long, it is considered that this historical case is the first step for a number of GRS integral bridges to be constructed in the coming years. The details of the construction and behaviour during and after construction of the full-scale model and the prototype bridge will be reported in the near future.

a)



b)

Fig. 33. The first full-scale GRS integral bridge, for the new bullet train line, at Kikonai, the south end of Hokkaido: a) overall dimensions (by the courtesy of Japan Railway Construction, Transport and Technology Agency); and b) a picture of the east side abutment, 14th October 2011.

Bridges to withstand severe earthquakes and great tsunamis

A great number of bridges lost their girders by strong tsunami forces caused by the 2011 East Japan Great Earthquake (Figs. 34 & 35). With these girders that were washed away by the great tsunami, measures had been taken to prevent dislodging from the abutments by seismic forces, but not designed to resist against the tsunami forces. On the other hand, many short-span single-girders for local roads, located near the modern bridges that lost their girders by tsunami force, survived the tsunami (Fig. 35). This is likely because these short-span bridges are plugged into the RC RWs on both sides of a small river without using bearings: i.e., these bridges are actually integral bridges. This fact indicates that GRS integral bridges may have a high resistance against tsunami forces.

Fig. 34. A view from the upstream of Tsuyagawa Bridge, between Motoyoshi and Rikuzenkoizumi stations, Kesen-numa line, East Japan Raiway (taken by the author)

Fig. 35. A view from the seaside of Namiitagawa bridge, between Namitagawa and Kirikiri stations, Otsuchicho, Yamada Line, East Japan Railway.

Fig. 36. A view from the seaside of Yonedagawa bridge, between Rikuchu-noda and Rikuchu-tamagawa stations, Noda village, Iwate Prefecture, North-Rias Line, Sanriku Railway

Moreover, as seen from Figs. 35 and 36, the unreinforced backfill of many bridges was washed away by over-flow of tsunami, like many embankment-type coastal dikes that were seriously damaged (Figs. 17 and 18). The backfill of GRS integral bridges can a high resistance against the overflow of tsunami forces by protecting the side faces of the backfill with FHR facings connected to the reinforcement layers. It is proposed to reconstruct these bridges that collapsed by the great tsunami to GRS integral bridges. The GRS integral bridge is more cost-effective than the conventional type bridge even when not designed against severe seismic loads and tsunami forces. However, with necessary provisions for protecting the foundations from scouring in the supporting ground by tsunami, the GRS integral bridge is particularly relevant for newly constructed bridges that should have a high resistance against tsunami forces, as well as seismic load.

CONCLUSIONS

Geosynthetic-reinforced soil retaining walls (GRS RWs) having stage-constructed full-height rigid (FHR) facing have been constructed as important permanent RWs for a total length of more than 125 km in Japan since 1999 until today (October 2011). Although these GRS RWs were constructed mainly for railways, many others were also constructed for highways and other types of infrastructure. Its current popular use is due to not only a low cost/performance ratio, but also high long-term performance and seismic stability that are equivalent to, or even better than, other modern RWs. This success can be attributed to the following factores:

1) A proper type of reinforcement is used (i.e., geogrids for cohesionless soil and nonwoven/woven geotextile composites for high-water content cohesive soil);
2) FHR facing is constructed by such a staged construction procedure that fresh concrete is cast-in-place on the wrapped-around wall face of full-height geosynthetic-reinforced backfill. By this procedure, reinforcement layers are firmly connected to the facing.
3) The rigidity of the facing is taken into account in design.

A number of embankments and conventional type RWs that collapsed during recent severe earthquakes, heavy rains and associated floodings and storms in Japan were reconstructed to embankments with geosynthetic-reinforced steep slopes or GRS RWs with stage-constructed FHR facing or their combinations. It was validated that this technology is highly cost-effective in restoring collapsed old soil structures.

A new bridge system, called the GRS integral bridge, has been developed, which comprises an integral bridge and geosynthetic-reinforced backfill. GRS integral bridges exhibit essentially zero settlement in the backfill and no structural damage to the facing when subjected to lateral cyclic displacements at the top of the facings caused by seasonal thermal expansion and contraction of the girder, while their seismic stability is very high. These features and high cost-effectiveness of the GSR integral bridge are due to that: 1) bearings are not used to support the girder; 2) the girder is continuous without any connections; 3) the backfill is reinforced with geogrid layers firmly connected to the facing; and 4) FHR facings are stage-constructed after the construction of full-height geosynthetic-reinforced soil walls and then pile foundations (if necessary).

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