CHAPTER 23

Design, Construction, and Performance of GRS Structures for Railways in Japan

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23.1 INTRODUCTION

Several types of geosynthetic-reinforced soil (GRS) structures have become standard soil structures (retaining walls, box culverts, bridge abutments, and bridge systems) for railways, including high-speed train (HST) lines in Japan (called Shinkansen). GRS structures are now adopted in place of conventional-type structures.

The first HST line (or Shinkansen) is the Tokaido Shinkansen between Tokyo and Osaka, which opened in 1964 immediately before the 1964 Tokyo Olympic Games. The south part between Shin-Aomori and Shin-Hakodate of the Hokkaido Shinkansen was the most recently constructed (Fig. 23.1(a)) and will open in 2016.

Three parts are now under construction. The total constructed length (not including two narrow-gauge lines) has reached 2765 km, which comprises of the following three generations: (1) Tokaido; (2) Sanyo (the west part between Osaka and Okayama, which opened in 1972, the east part between Okayama and Hakata, which opened in 1975, the south part of Tohoku between Tokyo and Morioka, which opened in 1982, and Joetsu, which also opened in 1982; and (3) everything constructed after 1982.

With the first generation (Tokaido), for grade separation, the embankment was constructed for more than a half of the total length (Fig. 23.1(b)) following the design standard at that time. From the start of train operation, continuous extensive track maintenance works were necessary to alleviate problems due to settlements of the embankment, in particular bumps immediately after bridge abutments and box culverts. Following serious damage to railway embankments on other lines from heavy rains and severe earthquakes that took place after the opening of Tokaido, the embankment at

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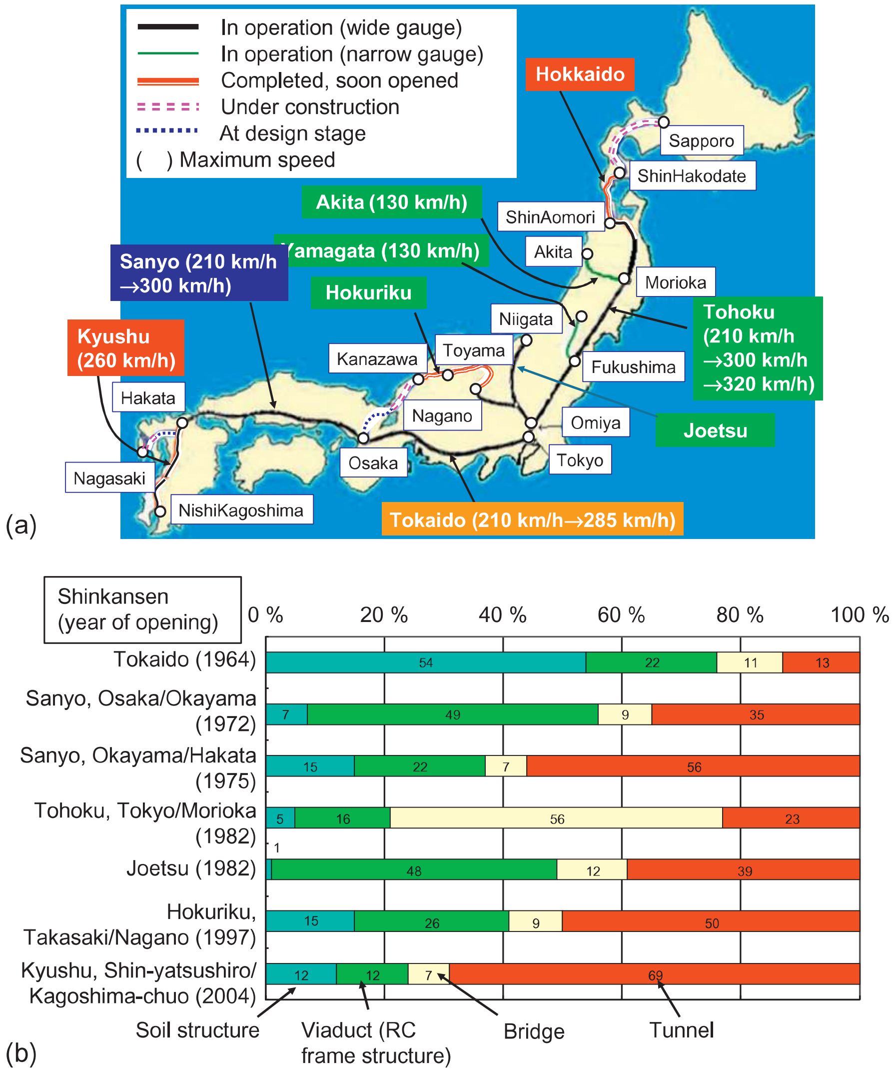


Figure 23.1 (a) Current network of HST in Japan (Shinkansen) and (b) length ratios of various structure types for the Shinkansen.

many places of Tokaido was reinforced to ensure a sufficient stability during heavy rains and severe earthquakes. Costly reinforcing works have been executed.

Although a gentle-sloped embankment occupies a wider base area than a reinforced concrete (RC) frame-structure such as a viaduct, the construction cost per length of embankment is much lower than a viaduct. Experience from the HST line construction in Japan showed that, roughly on average, the construction cost (not including the land cost) is 1/7of that of a viaduct and 1/15 of that of a bridge, and, even when the ground improvement becomes necessary as it does with an embankment, the cost is still 1/2 of that of a viaduct and 1/5 of that of a bridge. Moreover, by constructing embankments at nearby places using soil excavated from tunnels and ground, the construction cost and impact on the environment can be reduced significantly. However, based on the lessons of the Tokaido Shinkansen embankment, for the second generation (i.e., Sanyo, the south part of Tohoku and Joetsu), the amount of embankment and associated retaining wall decreased drastically (Fig. 23.1(b)).

During the 1980s, a GRS retaining wall (RW) with stage-constructed full-height rigid (FHR) facing (Figs. 23.2 and 23.3) was developed (Tatsuoka et al., 1997a,b). The first railway GRS RW of this type was constructed in 1989. During the 1995 Great Kobe earthquake, several GRS RWs of this type performed very well (Fig. 23.4), while a number of embankments and conventional-type RWs, as well as other types of reinforced soil RWs, were seriously damaged or collapsed (Fig. 23.5) (Tatsuoka et al., 1998). Many viaducts were also seriously damaged or collapsed at places along the Sanyo Shinkansen. These experiences showed that the stability of GRS RWs of this type, in particular against high seismic load such as during that earthquake, is much higher than embankments and conventional-type RWs.

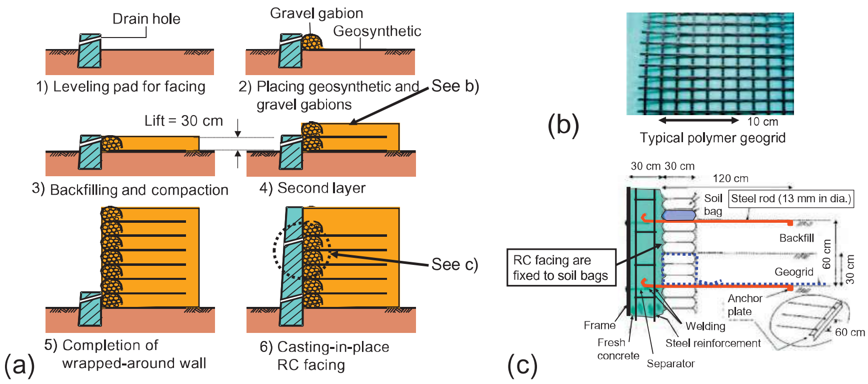
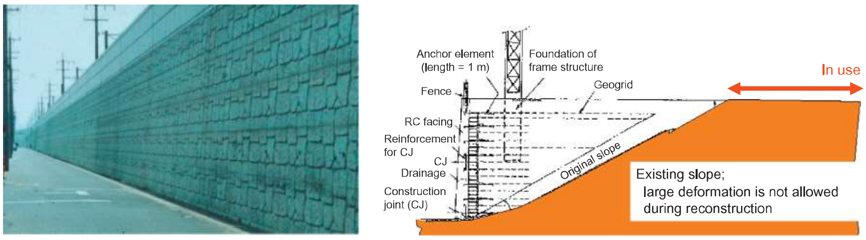


Figure 23.2 GRS RW with FHR facing: (a) staged-construction procedure, (b) a typical geogrid, and (c) facing construction. *(Source: From Tatsuoka et al. (1997a)).*



(a) (b)

Figure 23.3 Reconstruction of slopes of an existing embankment to a vertical wall for a high-speed train yard, 1990–1991, at Biwajima, Nagoya—average height = 5 m and total length = 930 m: (a) a view in 1991 and (b) a typical cross section. *(Source: From Tatsuoka et al. (1997a))*.

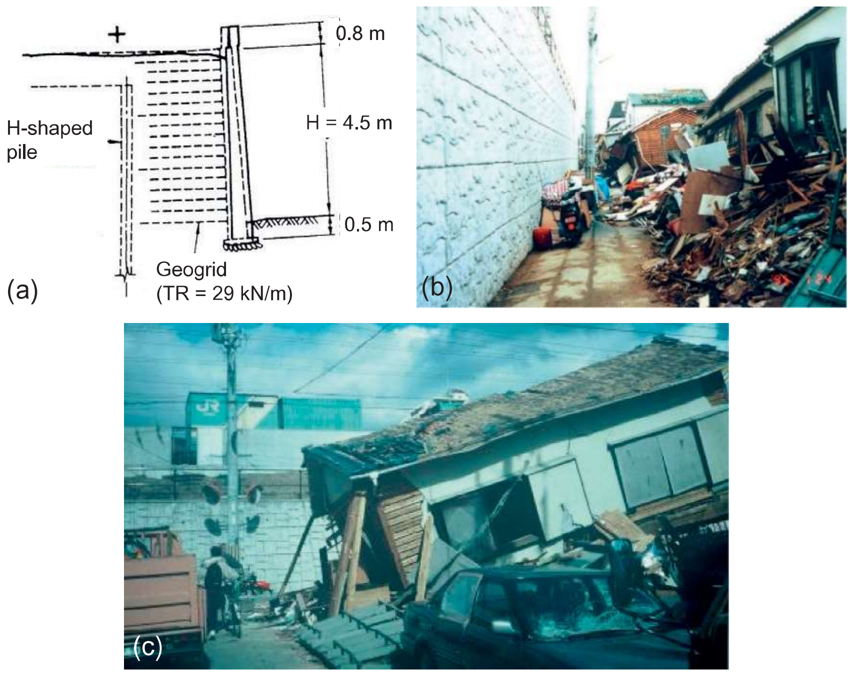


Figure 23.4 A GRS RW having FHR facing at Tanata, Kobe: (a) typical cross section and (b) and (c) views of the wall one week after the earthquake. *(Source: From Tatsuoka et al. (1977a,b, 1998))*.

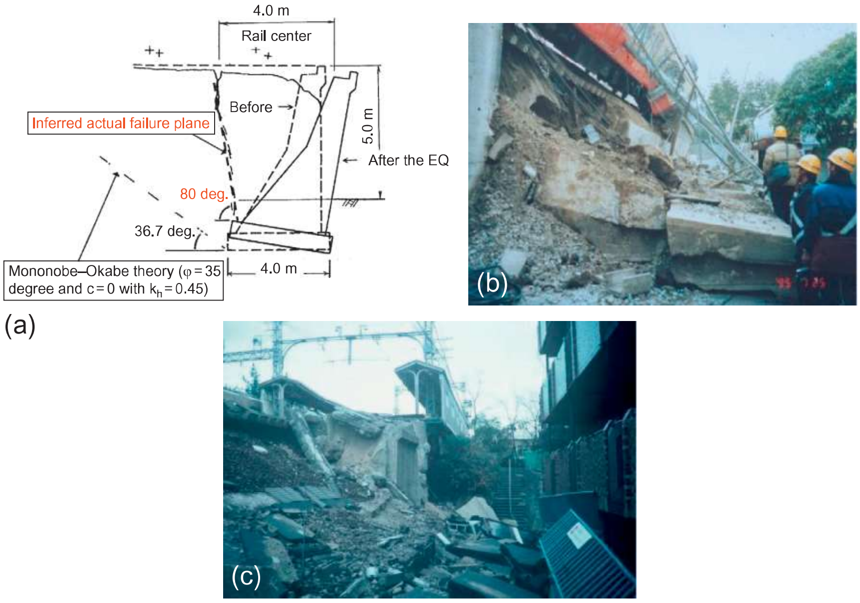


Figure 23.5 Typical damage to gravity-type unreinforced concrete RWs (without a pile foundation) at Hanshin Railway's Ishiyagawa Station during the 1995 Great Kobe earthquake: (a) sketch of typical section and (b) and (c) typical damaged sections on the opposite sides of the embankment. *(Source: From Tatsuoka et al. (1977a, b, 1998)).*

The design and construction policy of soil structures for Japanese railways has been drastically revised in the 20 years since the 1995 Great Kobe earthquake, as follows:

- The standard type of RW has fully changed from the conventional cantilever RW to the GRS RW having staged-constructed FHR facing with a strong connection between the facing and the reinforcement layers (Fig. 23.2; Tatsuoka et al., 1997a; Tatsuoka, 2001, 2008a). GRS RWs of this type have been constructed for a total length of about 160 km (as of June 2014), mainly for railways, including HST lines (Fig. 23.6; Tatsuoka et al., 2012a,b, 2014a,b). Figure 23.3 shows a typical case.
- It has also become the standard practice to reconstruct conventional-type embankments and RWs that collapsed by earthquakes, heavy rains, and floods to GRS RWs (Tatsuoka et al., 2007, 2012b, 2014b).
- A couple of new bridge systems using the GRS technology were developed in place of the conventional-type bridges. With GRS bridge abutments, a girder is placed via bearings on the top of the facing of

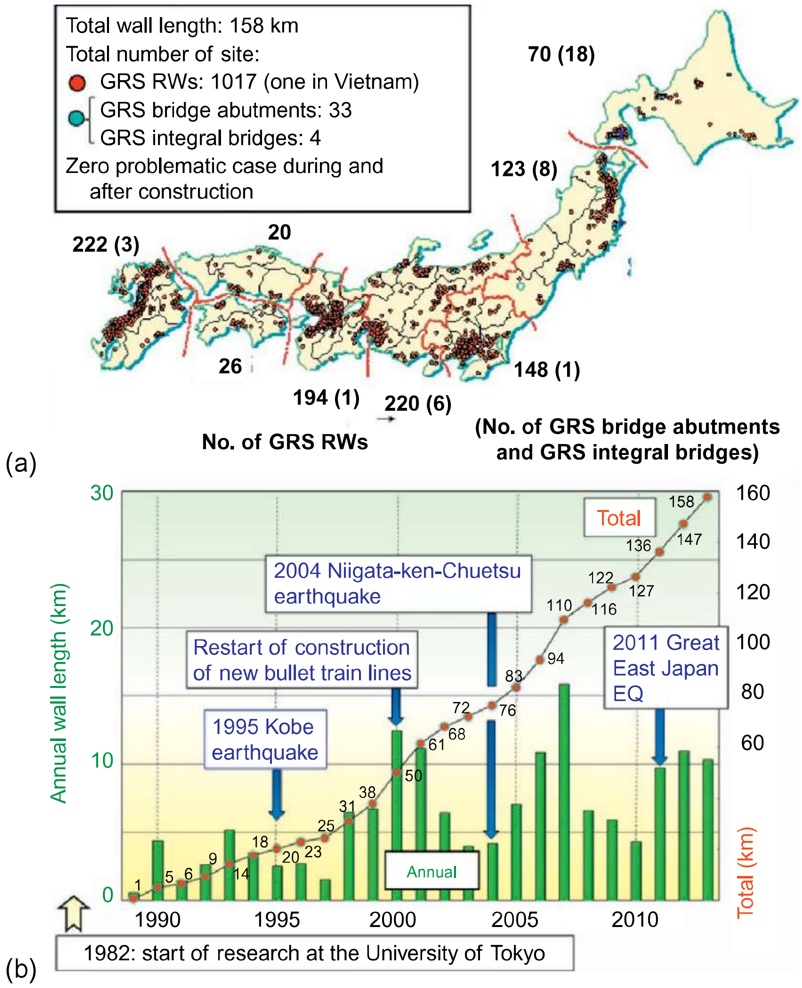


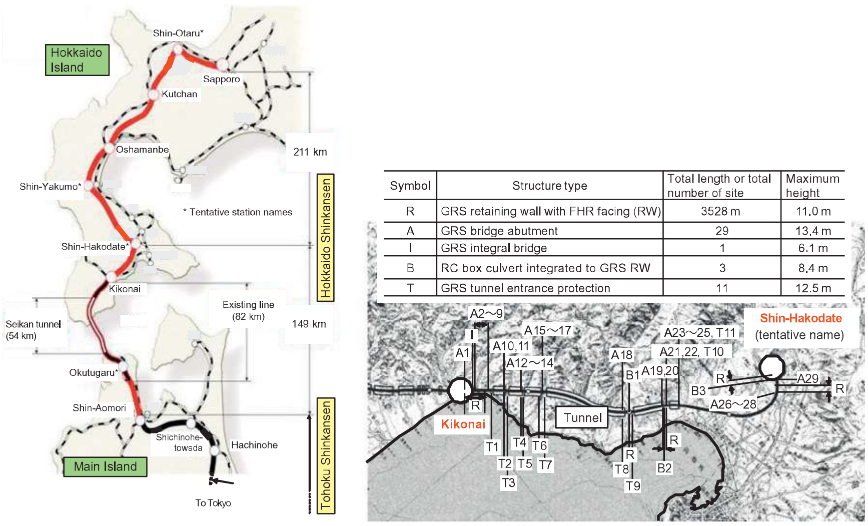
Figure 23.6 (a) Locations of GRS RWs with a staged-constructed FHR facing as of June 2014 and (b) annual and cumulative wall lengths.

the GRS RW (Aoki et al., 2005; Tatsuoka et al., 2005). About 50 GRS abutments of this type have been constructed. The latest bridge type is the GRS integral bridge (Tatsuoka et al., 2008a,b, 2009), which comprises a continuous girder of which both ends are structurally integrated without using bearings into the facings of a pair of GRS RWs (Fig. 23.2). The first GRS integral bridge was completed in 2012 for a high-speed train line and three others were completed in 2014.

- These GRS structures were and will be extensively used for the construction of high-speed train lines (Tatsuoka et al., 2012a,b, 2014a), which are among the most critical and important infrastructures in Japan.
- Soil structures are now designed to withstand very high seismic loads (called Level 2 design seismic load) as experienced during the 1995 Great Kobe earthquake, in a similar way as RC and metal structures (Tatsuoka et al., 1998, 2010; Koseki et al., 2006, 2008; Koseki, 2012).

So far, there have been no problems with any of the GRS structures, as indicated in Fig. 23.6(a). The statistics are shown in Fig. 23.6(b). Having experienced a couple of major earthquakes such as the 1995 Great Kobe and the 2011 Great East Japan earthquakes as well as heavy rains and floods, it has been proven that the GRS technology is very cost-effective, largely for having very high resistance against these severe types of natural disaster.

Most recently, various types of GRS structure were densely constructed for a new high-speed train line, the Hokkaido Shinkansen (Fig. 23.7(a)); Yonezawa et al., 2013, 2014). Construction began in 2005 and ended in 2014. At many sites along the 37.6 km route between Kikonai and Shin-Hakodate stations (Fig. 23.7(b)), the following various types of GRS structure were constructed:



Symbol	Structure type	Total length or total number of site	Maximum height
R	GRS retaining wall with FHR facing (RW)	3528 m	11.0 m
A	GRS bridge abutment	29	13.4 m
I	GRS integral bridge	1	6.1 m
B	RC box culvert integrated to GRS RW	3	8.4 m
T	GRS tunnel entrance protection	11	12.5 m

Figure 23.7 (a) Location of the Hokkaido Shinkansen (high-speed train) and (b) locations of GRS structures. *(Source: From Yonezawa et al. (2013, 2014)).*

- GRS RWs having FHR facing (at sites denoted by R in Fig. 23.7(b)) for a total length of 3.5 km with the largest wall height of 11 m, while no conventional-type cantilever RW was constructed.
- 29 GRS bridge abutments (denoted by A in Fig 23.7(b)), while no conventional-type bridge abutment was constructed. The tallest one is 13.4-m high.
- A GRS integral bridge (denoted by I in Fig 23.7(b)) at Kikonai, which is the first prototype of this new bridge type.
- 3 GRS box culverts to accommodate local roads underpassing the railway (denoted by B in Fig 23.7(b)). Each RC box structure is integrated to GRS RWs at both sides. The tallest one is 8.4-m high.
- 11 GRS tunnel entrance protections (denoted by T in Fig 23.7(b)). A GRS arch structure stabilizes the slope immediately above the tunnel entrance to protect trains against falling rocks and sliding soil masses. The tallest one is 12.5-m high.

Figure 23.8(a) shows a series of GRS structures at the Mantaro section of the Hokkaido Shinkansen (east of Kikonai indicated in Fig. 23.7(a)):

(R) GRS RWs with FHR facing (Fig. 23.9)
(A) GRS bridge abutments (Fig. 23.8(b))
(B) a GRS box culvert
(T) a GRS tunnel entrance protection

These GRS structures were chosen because of their very high cost-effectiveness (i.e., compared with conventional types); they need a lower construction and maintenance cost with a higher functionality including a higher seismic stability. In particular with GRS bridge abutments, GRS

Figure 23.8 (a) A view of a variety of GRS structures at the Mantaro section of the south part of the Hokkaido Shinkansen and (b) GRS bridge abutment (13.4-m high) during and after construction near tunnel.

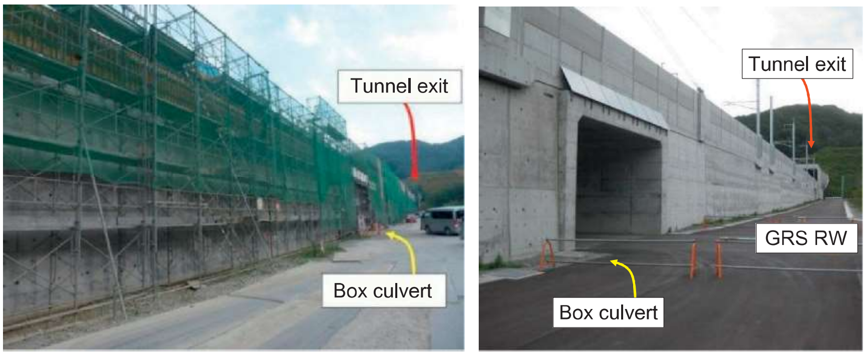


Figure 23.9 Views during and after Stage 6 in Fig. 23.2(a) of GRS RWs at both sides of a box culvert, site B2 in Fig. 23.7(b), Hokkaido high-speed line. *(Source: From Yonezawa et al. (2013, 2014))*.

integral bridges, and GRS box culverts, the settlement in the backfill immediately behind the facing (i.e., the bump) by long-term train loads and seismic loads becomes negligible, unlike conventional-type structures.

In this chapter, updating the content of Tatsuoka et al. (2014a), the lessons from experiences with these GRS structures gained during the last 25 years and the essence of the new seismic design method are summarized.

23.2 GEOSYNTHETIC-REINFORCED SOIL RETAINING WALLS WITH FULL-HEIGHT RIGID FACING

23.2.1 Staged construction

GRS RWs with FHR facing (see Fig. 23.2) are constructed as follows. After the major part of the residual deformation of the subsoil and the backfill due to the construction of geosynthetic-reinforced backfill has taken place, as shown in Fig. 23.2(a), FHR facing is constructed by casting-in-place concrete in the space between the outer concrete frame, which is temporarily supported by steel bars anchored in the backfill, and the wall face of the GRS wall wrapped around with geogrid reinforcement (Tatsuoka et al., 1997a). The facing and the reinforcement layers are firmly connected to each other, because fresh concrete can easily enter the gravel-filled gravel bags through the aperture of the geogrid wrapping around gravel bags that are part of the main reinforcement layer.

Figure 23.2(b) shows a typical type of geogrid. As the geogrid is directly in contact with fresh concrete exhibiting strong alkaline properties, a geogrid made of polyvinyl alcohol (PVA), which is known to have high resistance against high alkali environment, is normally used. Extra water from fresh concrete is absorbed by the gravel bags, which reduces the negative bleeding phenomenon of concrete. By this staged construction procedure, the connection between the reinforcement and the FHR facing is not damaged by differential settlement between them that may take place if the FHR facing is constructed prior to the construction of geosynthetic-reinforced backfill. In addition, before the construction of FHR facing, the backfill immediately behind the wall face can be well compacted. Then, the construction on a thick soft deposit becomes possible.

Before the construction of FHR facing, the gravel bags piled at the wall face function as a temporary but stable facing resisting against earth pressure generated by backfill compaction and the weight of overlying backfill. With help of these gravel bags, backfill compaction becomes efficient. For completed GRS RWs, the gravel bags function as a drainage and as a buffer protecting the connection between the FHR facing and the reinforcement against potential relative vertical and horizontal displacements.

Moreover, to construct a conventional-type cantilever RC RW, concrete forms supported by a propping system are necessary on both sides of the facing and they become more costly at an increasing rate with an increase in the wall height. With this type of GRS RW, only an external concrete form, temporarily supported with steel rods anchored in the backfill, is necessary without using any external propping and an internal concrete form supported by another propping system (see Fig. 23.2(c)).

23.2.2 Roles of full-height rigid facing

If the wall face is loosely wrapped with geogrid reinforcement without using a pile of gravel bags (or their equivalent), or if the reinforcement layers are not connected to a rigid facing, no or only very low lateral earth pressure is activated at the wall face (Fig. 23.10(a)). Then, the stiffness and strength of the active zone becomes low, which may lead to intolerably large deformation, or even collapse in extreme cases, of the active zone.

On the other hand, with the GRS RW system, before the construction of FHR facing, the gravel bags function as a temporary stable facing, therefore, high earth pressure can be activated at the wall face (Fig. 23.10(b)). This high earth pressure is transferred to the FHR facing after its construction, which results in high confining pressure at the wall face, thus high stiffness and strength of the active zone, and high performance of the wall. This mechanism is particularly important to ensure high seismic stability.

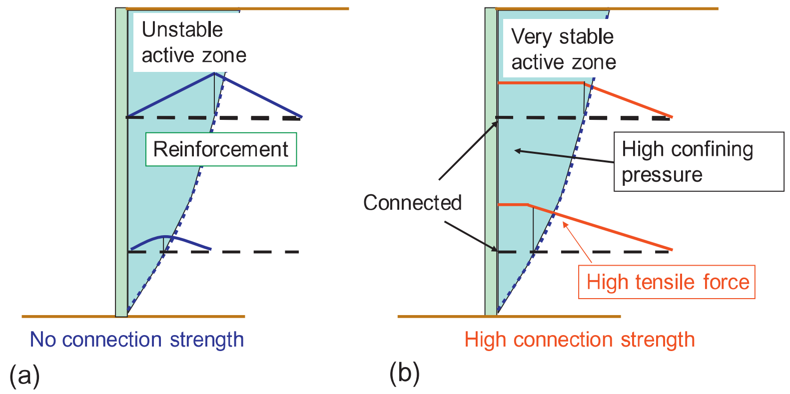


Figure 23.10 Importance of a firm connection between the reinforcement and the rigid facing for wall stability. *(Source: From Tatsuoka (1992))*.

A conventional-type RW is a cantilever structure that resists the active earth pressure from the unreinforced backfill. Therefore, large internal moment and shear forces are mobilized in the facing while large overturning moment and lateral thrust force develops at the base of the facing. Thus, a pile foundation usually becomes necessary, particularly when constructed on thick soft subsoil. These disadvantages become more serious at an increasing rate with an increase in the wall height. In contrast, as the FHR facing of this GRS RW system is a continuous beam supported by many reinforcement layers with a small span (i.e., 30 cm), only small forces are mobilized in the FHR facing even by high large earth pressure.

Therefore, the FHR facing becomes much simpler and lighter than conventional cantilever RC RWs. As only small overturning moment and lateral thrust force is activated at the bottom facing, a pile foundation is not used in normal cases. If constructed on relatively soft ground, usually shallow ground improvement by cement mixing is performed to ensure sufficient bearing capacity. These features make the GRS RW with FHR facing much more cost-effective (i.e., much lower construction and maintenance cost and much speedier construction using much lighter construction machines despite higher stability) than cantilever type RC RWs.

These features of the FHR facing become more important when concentrated external load is activated at the top of the facing or the crest of the backfill immediately behind the facing. The load is distributed to large parts of the FHR facing then to many reinforcement layers, thereby resisted by a large mass of the wall. FHR facing is often used as the foundation for electric poles (typically one pole per 50 m) and noise barrier walls. GRS bridge abutment and GRS integral bridges were developed by taking advantage of this mechanism. In those cases, a negligible bump develops immediately behind the facing at the bridge abutment, which is among the very important advantages. In comparison, reinforced soil RWs having discrete panel facing lack structural integrity, as previously noted, exhibiting much lower resistance against concentrated load. Local failure of the facing (e.g., loss of a single panel) may result in the collapse of the whole wall.

23.2.3 A brief history of geosynthetic-reinforced soil retaining walls with full-height rigid facing

Until June 2014, GRS RWs with FHR facing had been constructed for a total length of about 160 km at more than 1000 sites, mainly for railways and many for high-speed train lines (see Fig. 23.6). It is important that any problematic case during construction as well as during long-term service has been reported. In urban areas, near-vertical retaining walls have significant advantages over conventional gentle-sloped embankments as railway structures because of: (1) more stable behavior with smaller residual displacements; (2) much smaller base areas, which significantly reduce the cost for land acquisition; (3) no need for barrier walls, protection work, vegetation, and long-term maintenance of the embankment slope; and (4) a much smaller volume of ground improvement of soft sublayer is required.

For these reasons, a great number of conventional-type RWs (unreinforced concrete gravity type or RC cantilever type) have been constructed in urban areas. However, in rural areas, conventional gentle-sloped embankments are usually constructed due to the high construction cost of conventional-type RWs, in particular when long piles are necessary. It is much more cost-effective to construct GRS RW with FHR facing than to construct gentle-sloped embankments not only in urban areas but also in rural areas, especially in the Hokkaido Shinkansen project (see Fig. 23.2(b)).

RC slabs for ballast-less tracks are basically free from long-term maintenance works, while conventional ballasted tracks need continuous maintenance, which can be very costly. RC slabs for ballastless tracks are not allowed to be constructed on conventional embankments having gentle slopes or those supported by conventional-type retaining walls, as very small tolerable residual settlement of RC slabs for ballast-less tracks cannot be ensured. Instead, RC slabs for ballast-less tracks have been constructed on the backfill supported by the GRS RWs with FHR facing. Until today, no problematic case with track maintenance has been reported with GRS RWs with FHR facing.

23.2.4 Seismic design

A number of conventional-type RWs collapsed during the 1995 Great Kobe earthquake. Figure 23.5 shows typical collapsed gravity-type RWs. They were constructed about 85 years ago based on the pseudostatic seismic design at that time using a horizontal seismic coefficient of 0.2. The walls failed in the overturning mode by seismic loads that were much higher than the design value. In contrast, the GRS RW with FHR facing (Fig. 23.2) exhibited a very high seismic stability during the 1995 Great Kobe earthquake; Fig. 23.4 shows how it is typically seen.

This GRS RW was constructed in 1992, so it was designed before the 1995 Great Kobe earthquake based on the pseudostatic limit equilibrium stability analysis (Horii et al., 1998) requiring a minimum safety factor in terms of horizontal earth pressure of 1.5 against a horizontal seismic coefficient k_h of 0.2. This safety factor comprises a safety factor of 1.25 for the global structural equilibrium times a safety factor for the tensile rupture failure of geogrid of 1.25 (i.e., $1.25 \times 1.25 = 1.5$).

This good seismic performance of GRS RWs, despite the fact that the actual seismic load is much higher, is due likely to a sufficient amount of redundancy that was implicitly included in the design of this wall, as discussed in details by Tatsuoka et al. (2014b). A high seismic stability of the GRS RWs of this type was reconfirmed by many similar cases during the 2011 Great East Japan earthquake (Fig. 23.11; Tatsuoka et al., 2012a,b).

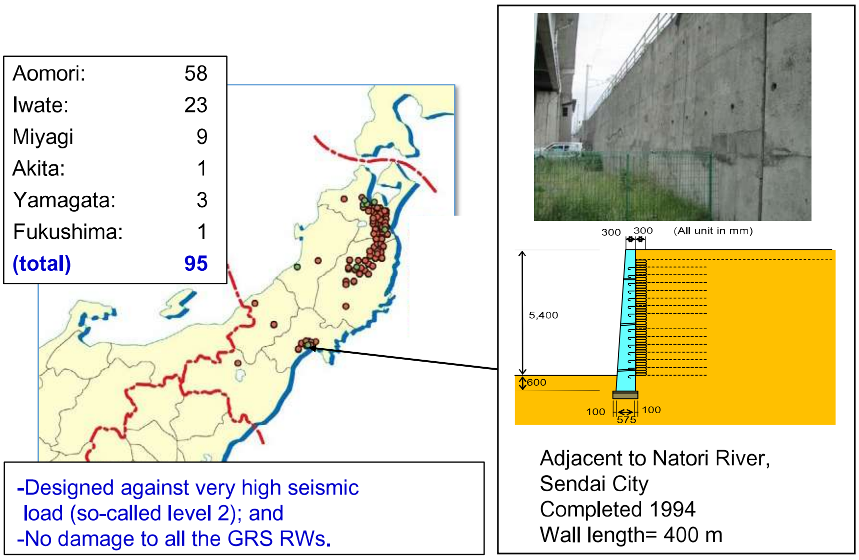


Figure 23.11 High performance of GRS RWs with FHR facing for railways, including HSTs constructed before the 2011 Great East Japan earthquake.

Based on these experiences, a number of conventional-type RWs and embankments that collapsed during the 1995 Great Kobe earthquake, the 2011 Great East Japan earthquake, and others, as well as those that collapsed by heavy rains, floods, and ocean wave action during typhoons, were reconstructed to this type GRS RWs (Tatsuoka et al., 2014a,b). Some recent case histories are described later in this chapter.

The seismic design code of railway soil structures, including GRS structures, was substantially revised based on lessons learned from the performance of soil structures during the 1995 Great Kobe earthquake (Koseki et al., 1997, 2006, 2007, 2009; Tatsuoka et al., 2010; Koseki, 2012). Since then, the code has been consistently revised referring to new lessons from subsequent earthquakes. The latest version of the Design Standard for Railway Soil-Retaining Structures was published in 2012 (Railway Technical Research Institute, 2012). The recent seismic design of Japanese railway soil structures, including GRS RWs and GRS integral bridges, are characterized by the following features among others:

- Introduction of very high design seismic load (Level 2).
- The use of peak and residual shear strengths with well-compacted backfill (while ignoring apparent cohesion) (Tatsuoka, 2011).
- Design based on the limit equilibrium stability analyses.
- Evaluation of seismic performance based on residual deformation obtained by modified Mononobe–Okabe (Koseki et al., 1007) and modified Newmark method (Horii et al., 1998; Tatsuoka et al., 2010, 2014b).
- No creep reduction factor for the design tensile rupture strength of geosynthetic reinforcement against seismic loads.
- Recommendations of the use of GRS structures when relevant and possible.
- Tatsuoka et al. (2010, 2014b) explain in detail these characteristic and unique features of the new seismic design code.

23.2.5 High cost-effectiveness

A cost comparison was made between a typical pair of conventional-type embankments retained by cantilever RC RWs and one retained by GRS RWs with FHR facing for the same backfill properties and ground conditions (i.e., a 20-m thick relatively soft ground) following the Japanese railway design codes, while based on the current Japanese market prices (Fig. 23.12). The pseudostatic seismic stability analysis (Horii et al., 1998; Tatsuoka et al., 2010) was performed using a horizontal seismic coefficient at a ground surface k_h equal to 0.2 (so-called Level 1; Tatsuoka et al., 2010).

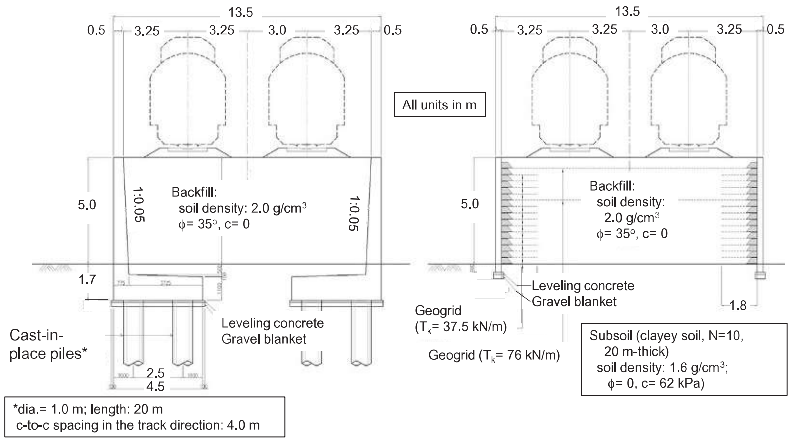


Figure 23.12 A typical pair of embankments retained by conventional-type RWs and GRS RWs for a cost comparison.

This coefficient is similar to those used in many other countries. No amplification of acceleration in the embankment was assumed.

Under this ground condition, GRS RWs can be constructed without using piles, while the conventional RWs should be supported by piles. This example demonstrates a very high cost-effectiveness of GRS RW (refer to Fig. 23.2) such that the construction cost ratio is 0.32; the maintenance cost ratio for 20 years is 0.5; and the total cost ratio is 0.33. Even when the ground is relatively firm and no piles are used with the conventional-type RWs, the construction cost ratio is 0.81; the maintenance cost ratio is 0.5; and the total cost ratio is 0.77. If using such high seismic load as the one during the 1995 Great Kobe earthquake, the ratio becomes even lower.

23.3 GEOSYNTHETIC-REINFORCED SOIL STRUCTURES FOR BRIDGES

23.3.1 Geosynthetic-reinforced soil bridge abutment

A conventional-type bridge comprises a single simple-supported girder supported by a pair of abutments via fixed (or hinged) and movable shoes (or bearings), or multiple simple-supported girders supported by a pair of abutments and a single or multiple pier(s) via shoes. The abutment, which may be a gravity structure (unreinforced concrete or masonry) or an RC structure, has a number of drawbacks, as follows (Fig. 23.13(a)):

- As the abutment is a cantilever structure that retains unreinforced backfill, earth pressure activated on its back induces large internal force as well

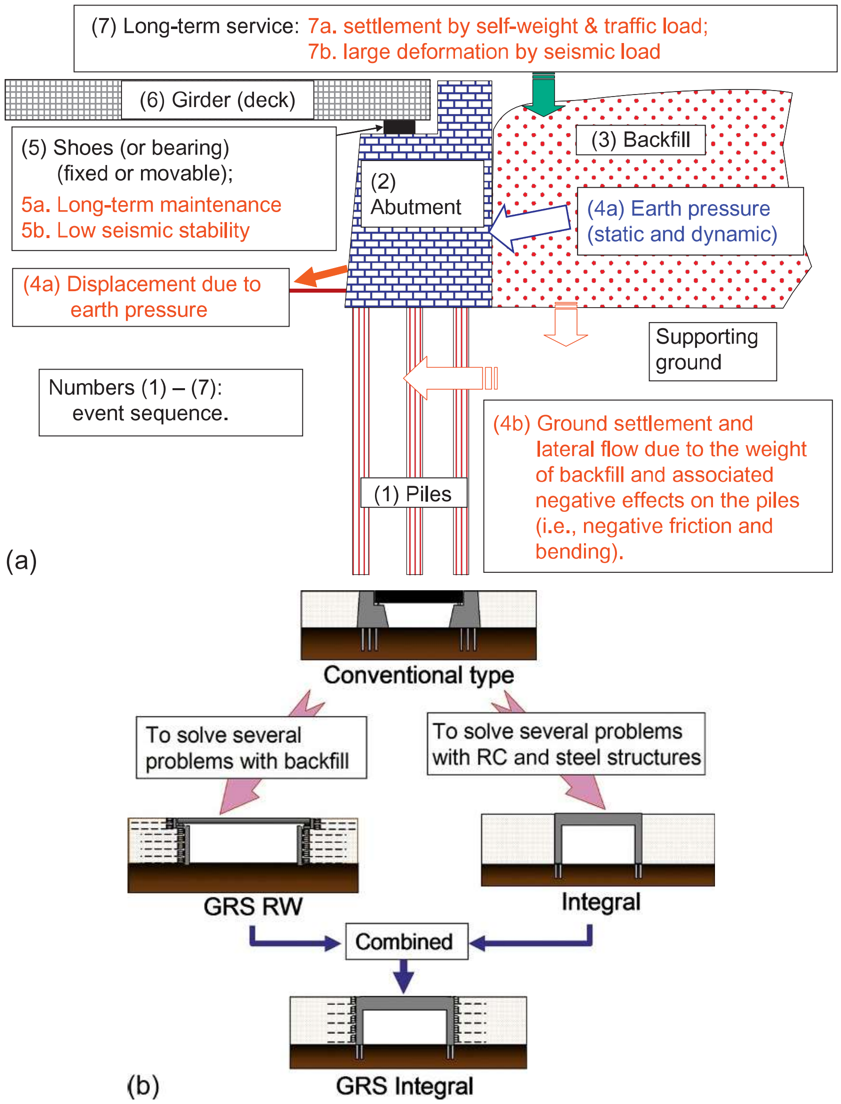


Figure 23.13 (a) A number of technical problems with conventional-type bridge abutment and (b) development of new-type bridges alleviating technical problems of conventional-type bridges. *(Source: From Tatsuoka et al. (2009))*.

as large thrust force and overturning moment at the bottom of the abutment. Therefore, the abutment may become massive, while a pile foundation is necessary unless the supporting ground is strong enough. This drawback becomes more serious at an increasing rate with an increase in the wall height.

- Despite that only small movement is allowed once constructed, abutments are constructed prior to the construction of the backfill. Therefore, when constructed on relatively soft ground, a large number of piles may become necessary to prevent movements due to earth pressure as well as settlement and lateral flow in the subsoil caused by the backfill weight. Large negative friction may be activated on the piles. The piles may become much longer than the wall height when the soft ground is thick.
- The construction and long-term maintenance of girder shoes and connections between separated simple-supported girders are generally costly. The girder shoes are weak part of the whole bridge system when subjected to seismic loads.
- A bump may be formed behind the abutment by long-term settlement of the backfill due to its self-weight and traffic loads.
- The seismic stability of the backfill and the abutment supporting the girder via a fixed shoe is relatively low. A large bump may be formed behind the abutment if the backfill deforms largely by seismic loads.

To alleviate these problems, three new bridge systems have been proposed and introduced (Fig. 23.13(b)). The integral bridge (Fig. 23.14(a)

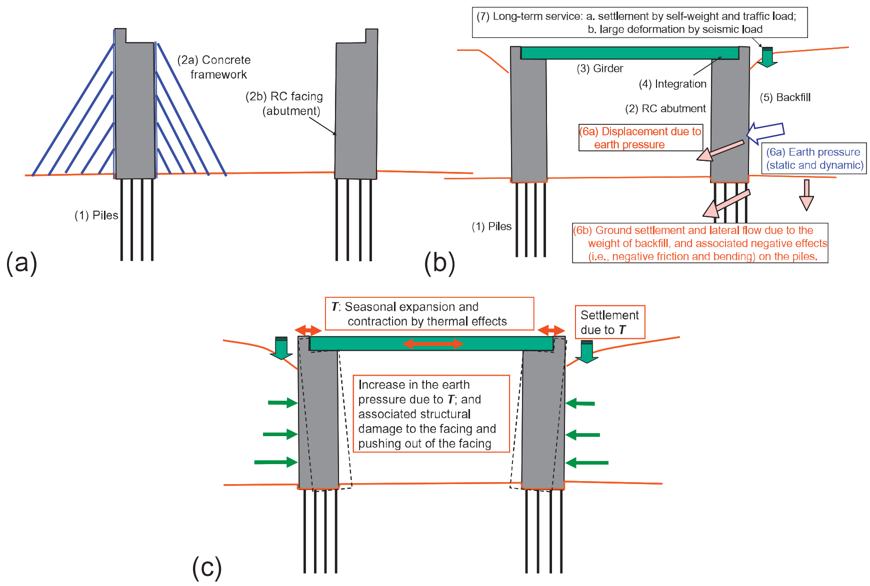


Figure 23.14 Integral bridge: (a) and (b) construction sequence and associated problems and (c) a new problem by seasonal thermal expansion and contraction of the girder.

and (b)) has been proposed to mainly alleviate problems with the structural part of reinforced concrete (RC) and/or steel of the conventional-type bridge. This new bridge system is now widely used in the United Kingdom and North America (in particular, the United States and Canada), mainly due to high cost-effectiveness by low construction and maintenance cost resulting from no use of girder shoes (or bearings) and the use of a continuous girder (or deck). Furthermore, the seismic stability of the structural part (i.e., a girder and a pair of abutments) is higher than the conventional type (Fig. 23.13(a)). However, this new bridge type cannot alleviate some of the problems with conventional-type bridges (Fig. 23.14(b)), while a new problem by seasonal thermal expansion and contraction of the girder may take place (Fig. 23.14(c)), as discussed later in this chapter.

As previously mentioned, the development of large bumps immediately behind a bridge abutment by depression of the unreinforced backfill and displacements of the wing RWs and the abutment during a long period of service and by severe earthquakes, is one of the most serious problems with conventional-type bridge abutments (Fig. 23.13(a)) and integral bridges (Fig. 23.14). To alleviate this problem, an approach block comprising compacted well-graded gravelly soil was introduced in the 1967 Design Standard for Railway Soil Structures. However, it was revealed that this measure is not effective. Subsequently, the authors and their colleagues developed a new type bridge abutment (Fig. 23.15) (Aoki et al., 2005; Tatsuoka et al., 2005).

One end of a bridge girder is placed on the top of the FHR facing of a GRS RW via a fixed (i.e., hinged) bearing, while the other end is placed on the top of a pier via a movable (i.e., roller) bearing; or both ends are placed on the top of the FHR facings of a pair of GRS RWs via a set of bearings (hinged and roller). To ensure high performance of bridges, in particular when

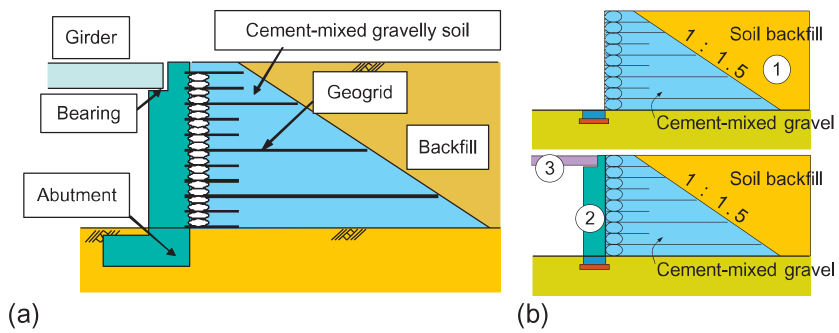


Figure 23.15 (a) GRS bridge abutment and (b) construction procedure.

constructed for high speed trains, the backfill immediately behind the facing is well-compacted, lightly cement-mixed, well-graded gravelly soil that is reinforced with geogrid layers connected to the facing. The mixing proportion, field compaction control, and strength and deformation characteristics of cement-mixed soil currently used in the present practice are described in detail in Tatsuoka et al. (2005) and chapter 12 of this book. Yet, the gravel bags immediately behind the facing are filled with uncemented gravelly soil so as to function as a drainage layer and a buffer that can absorb potential relative lateral displacements between the facing and the cement-mixed backfill caused by annual thermal deformation of the girder and seismic loads.

The first advantage of the GRS bridge abutment is a much higher seismic stability with a minimum bump even against severe seismic loads. This new type bridge abutment is much more cost-effective than the conventional-type bridge abutment because the RC facing is much more slender and usually a pile foundation is not used. Without including a cost reduction with the foundation structure and long-term maintenance, the construction cost decreases typically by about 20% when compared with the conventional-type bridge abutment.

The first GRS bridge abutment of this type was constructed from 2002–2003 at Takada for the Kyushu Shinkansen (Aoki et al., 2005; Tatsuoka et al., 2005). By performing full-scale vertical and lateral loading tests of the FHR facing, it was confirmed that the connection strength between the FHR facing and the geogrid-reinforced backfill is sufficiently high. For the Hokkaido Shinkansen, in total 29 GRS bridge abutments of this type were constructed while no conventional-type bridge abutment was constructed. The tallest GRS bridge abutment is 13.4-m high (Fig. 23.16). Until today, in total about 50 GRS abutments of this type have been constructed for railways.

23.3.2 Geosynthetic-reinforced soil integral bridge

The use of bearings (movable or fixed or both) to support a girder is the one remaining serious problem with the GRS bridge abutment (Fig. 23.15). To alleviate this problem, the GRS integral bridge, illustrated in Fig. 23.17, was developed based on a series of model shaking-table tests (Tatsuoka et al., 2008a,b, 2009, 2012a,b; Munoz et al., 2012) and the construction of a full-scale model (Figs. 23.18(a) and (b); Suga et al., 2011) and loading tests performed three years after its construction (Fig. 23.18(c); Koda Koda et al., 2013). The stability of a full-scale model of a GRS integral bridge was confirmed by applying two-directional cyclic lateral loads simulating design

Figure 23.16 GRS abutment at Mantaro for the Hokkaido Shinkansen (A21 in Fig. 23.7 (b)—views under construction: (a) from the front side, (b) from the backside, and (c) completed. *(Source: From Yonezawa et al. (2013, 2014))*.

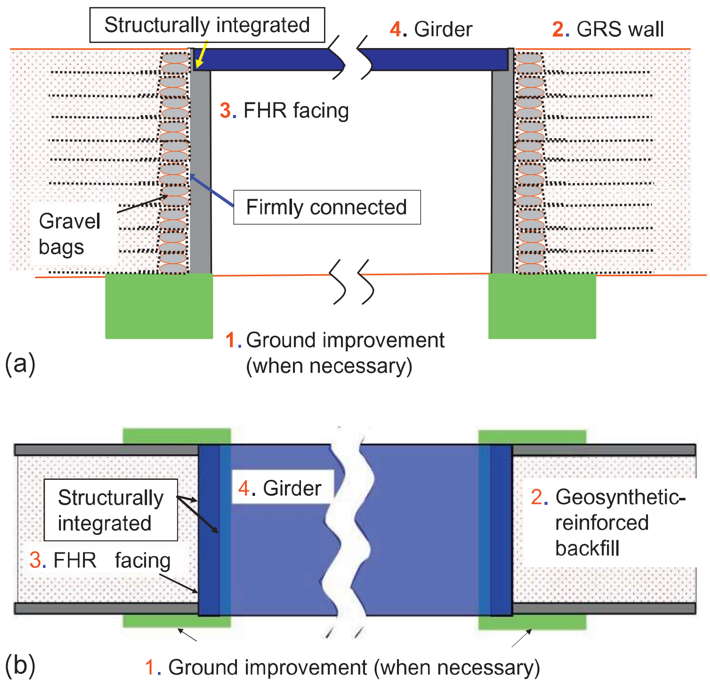


Figure 23.17 Construction sequence of GRS integral bridge: (a) elevation and (b) plan.

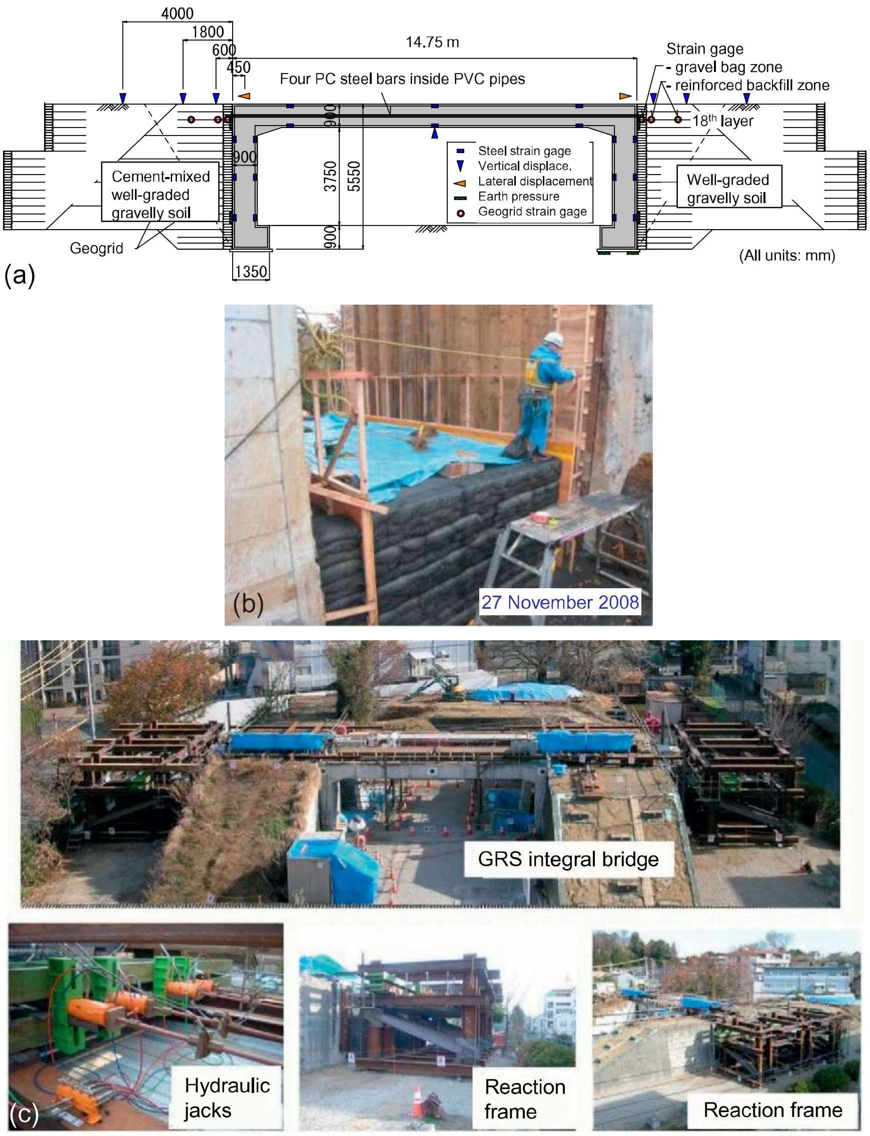


Figure 23.18 A full-scale model of GRS integral bridge constructed at Railway Technical Research Institute: (a) overall structure, (b) left-side abutment under construction, and (c) full-scale loading test performed in January 2012.

thermal deformation of the girder and Level 2 design seismic load to the girder of the model. The current seismic design method of a GRS integral bridge is described in Yazaki et al. (2013).

The GRS integral bridge (Fig. 23.17) exhibits negligible settlement in the backfill immediately behind the facing and negligible structural damage to the facing by cyclic lateral displacements of the facing caused by seasonal thermal expansion and contraction of the girder (Tatsuoka et al., 2009). The only, but significant, difference of the GRS integral bridge (Fig. 23.17) from the GRS bridge abutment (see Fig. 23.15) is that, with the GRS integral bridge, both ends of a continuous girder are integrated to the top of the FHR facing of a pair of GRS RWs without using bearings.

The first advantage of GRS integral bridges over bridges comprising GRS bridge abutments is that the construction and maintenance of a bearing becomes unnecessary. Second, the RC girder becomes much more slender due to a significant reduction (by a factor of ~0.5) of the maximum moment resulting from flexural resistance at the connection between the girder and the facing. Third, as demonstrated by various model tests and numerical analysis, the seismic stability increases significantly due to an increased structural integrity and a reduced weight of the girder. Fourth, due to higher structural integrity and a smaller cross section of the girder, the resistance against tsunami loads increases significantly.

The first GRS integral bridge was constructed as the overroad bridge at Kikonai for the Hokkaido Shinkansen (Fig. 23.19). As this is the first full-scale GRS integral bridge and as this is for high speed trains, its high stability was confirmed by monitoring the behavior continuously from the start of construction until sometime after the start of service (scheduled to be April 2016) (Kuriyama et al., 2012; Tatsuoka et al., 2015). The ambient temperature and strains in the steel reinforcement in the RC structures, strains in the geogrid, the displacements of the RC structures, and the backfill and earth pressures at representative places are being observed. It is evident that the structure is not overstressed at all. Results of detailed analysis will be reported by the authors in the near future. The other three GRS integral bridges that were subsequently constructed are described in Section 23.5.2.

23.4 GEOSYNTHETIC-REINFORCED SOIL BOX CULVERT

At three sites (B1, B2, and B3 shown earlier in Fig. 23.7(b)), where the Hokkaido Shinkansen crosses local roads, RC box culverts (i.e., underpass structures) integrated to the geogrid-reinforced backfill on both sides (called GRS box culverts) were constructed. Figure 23.20(a) shows the structure

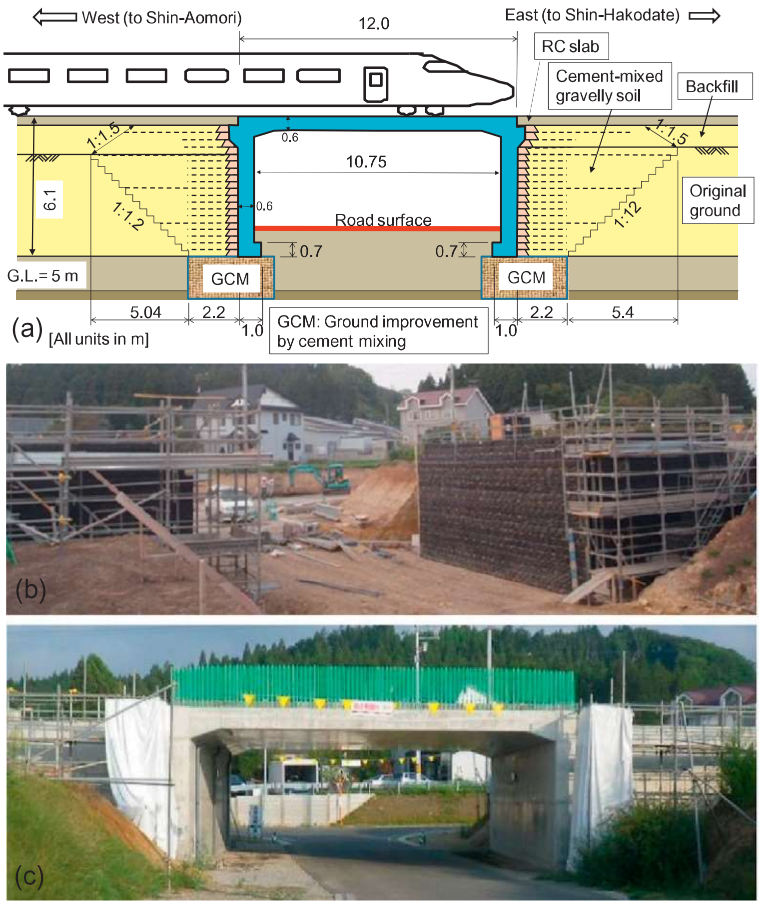
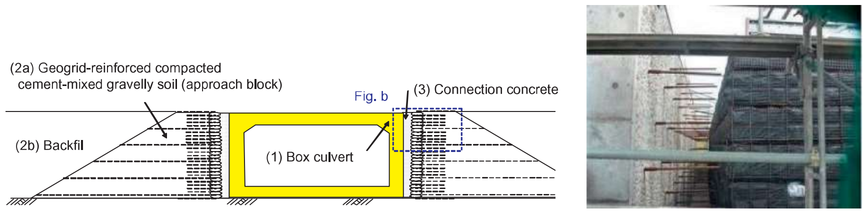


Figure 23.19 GRS integral bridge at Kikonai, Hokkaido Shinkansen (at site **I** in Fig. 23.7(b): (a) details, (b) during construction (October 14, 2011), and (c) completed (July 31, 2012).



(a) (b)

Figure 23.20 GRS box culvert for the Hokkaido Shinkansen: (a) general structure—numbers denote the construction sequence (site B2 in Fig. 23.7(b)); and (b) space between the RC box structure and the approach block before step 3 (site B1 in Fig. 23.7(b)). *(Source: From Yonezawa et al. (2013, 2014)).*

of those constructed at sites B2 and B3. At each of these sites, an RC box structure was constructed first as it was requested for the reopening of a local road as soon as possible. Then, GRS RWs comprising well-compacted lightly cement-mixed well-graded gravelly soil reinforced with geogrid layers were constructed at both sides, leaving a narrow space, as shown in Fig. 23.20(b).

Finally, concrete was cast-in-place into this space to integrate the RC box culvert to the GRS RWs. For a high integrity of the whole structure, horizontal anchor steel rods connected to the steel reinforcement framework of the RC box structure had been protruded into the space. When constructed on a thick soft soil deposit, it is more relevant to first construct approach fills on both sides, followed by the construction of an RC box structure after the ground settlement due to the weight of the approach fills has taken place sufficiently so that the RC box structure becomes free from negative effects of ground settlement.

A GRS box culvert has nearly the same superior features as a GRS integral bridge over a conventional-type box culvert (in contact with unreinforced backfill on both sides). Yet, a GRS box culvert in the completed form is different from a GRS integral bridge in that it has the bottom RC slab. Therefore, the contact pressure at the bottom face of the bottom RC slab of a GRS box culvert is much lower than the one at the facing bottom of a GRS integral bridge. Therefore, the stability of a GRS box culvert is higher than a GRS integral bridge under otherwise identical conditions. On the other hand, for a longer span for which the bottom RC slab cannot be constructed, a GRS integral bridge becomes relevant.

23.5 FLOOD AND TSUNAMI

23.5.1 Flood

A great number of embankments for roads and railways retained by conventional-type cantilever RWs along rivers and seashores have collapsed because of floods and storm wave actions, usually triggered by overturning failure of the RWs caused by scouring in the supporting ground (Tatsuoka et al., 2007, 2014a,b). After the collapse of the RW, the backfill is quickly and largely eroded, resulting in closure of the railway or road. This type of collapse takes place easily, as the stability of a cantilever RW fully hinges on the bearing capacity at the bottom of the RW and the stability of the backfill fully hinges on the stability of the RW (Fig. 23.21(a)). However, a GRS RW with an FHR facing is much more stable against the scouring in the

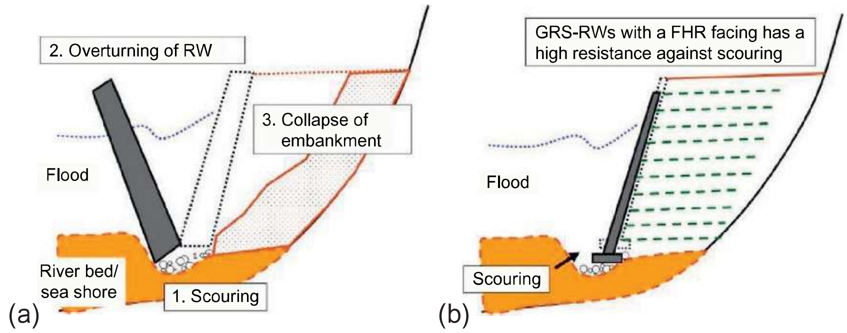


Figure 23.21 (a) Collapse of cantilever-type RW by scouring and (b) high performance of GRS RW with FHR facing.

supporting ground (Fig. 23.21(b)). It is particularly important that the facing does not overturn easily and the backfill can survive unless the supporting ground is extremely scoured.

Floods occurred in many rivers during the Niigata-Fukushima heavy rainfall at the end of July 2011. In Tōkamachi city, the maximum rainfall intensity was 120 mm/h and 294 mm/day. A high embankment retained by a masonry gravity-type RW at the lower part on the left bank of Agano river in the Niigata Prefecture, for the Ban'etsu West Line of East Japan Railway (JR East), collapsed by the mechanism illustrated in Fig. 23.21(a). The wall was reconstructed to about 9.4-m high and 50-m long GRS RW with an FHR facing. Because of this heavy rainfall, soil structures at more than 150 sites of the Iiyama Line of JR East were seriously damaged. Among them, a masonry wing RW of the approach fill of the Iruma River Bridge collapsed by the same mechanism (Figs. 23.22(a) and (b)). The railway was required to reopen only 10 days after the collapse. It would have taken longer than that if the original masonry RW was reconstructed. However, it was feasible and less costly with a GRS RW (Fig. 23.22(b)). Figure 23.22(c) shows the bridge during reconstruction. The railway was reopened with slowed-down running of trains before the construction of the FHR facing. Figure 23.22(d) shows the completed wall.

23.5.2 Tsunami

After the 2011 Great East Japan earthquake, a massive tsunami brought destruction along the Pacific coastline of East Japan. Coastal dikes at many places fully collapsed by the following collapse mechanism caused by the deep overtopping tsunami current (Fig. 23.23(a)):

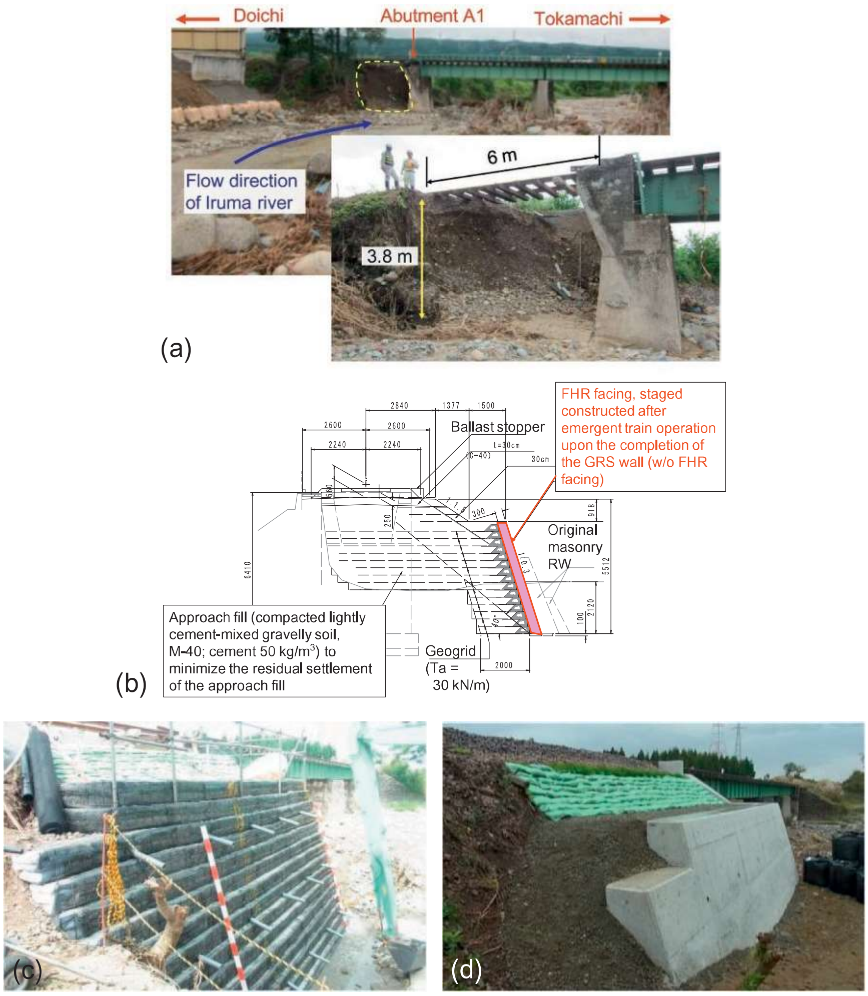


Figure 23.22 (a) Collapse of a masonry RW for the approach fill of a bridge by scouring of the supporting ground, followed by erosion of the backfill by flood (July 2011); and (b), (c), and (d) restoration to a GRS RW with FHR facing Iiyama Line, JR East. *(Source: From Tatsuoka et al. (2012)).*

1. The ground in front of the toe of the downstream slope was scoured. The concrete panels at the crest and around the downstream corner at the crest were lifted up by the tsunami current.
2. The stability of the concrete panels on the crest and the downstream slope, which were not fixed to the backfill, was lost and washed away.
3. The erosion of the backfill started, and eventually the backfill was fully washed away and the full section was lost (Fig. 23.23(b)).

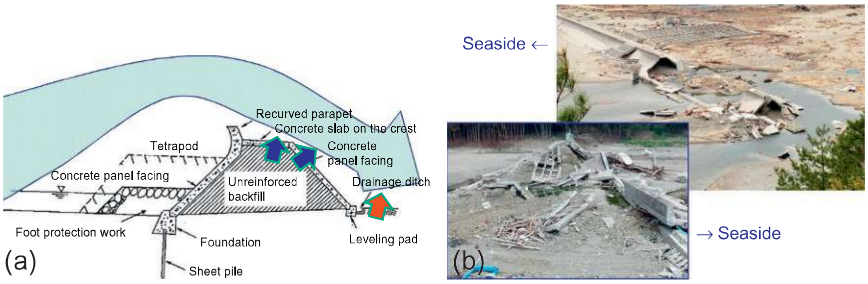


Figure 23.23 (a) Failure mechanism of coastal dikes by overtopping tsunami current and (b) typical fully collapsed coastal dike at Aketo, Tanohara, Iwate Prefecture.

As a result, the dikes could not work at all as a barrier against subsequent tsunamis. Small scale model tests (Yamaguchi et al., 2013) indicated that coastal dikes that comprise the geogrid-reinforced backfill covered with continuous lightly steel-reinforced concrete facings firmly connected to the reinforcement, such as those illustrated in Fig. 23.24, have much stronger resistance against deeply overtopping tsunami current.

The girders and/or approach fills behind the abutments of a great number of road and railway bridges (more than 340) were washed away by the tsunami (Kosa, 2012), as seen in Figs. 23.25, 23.26(b), and 23.27(b). It was confirmed that a girder supported by bearings has a very low resistance against uplift and lateral forces of tsunami current, while the unreinforced

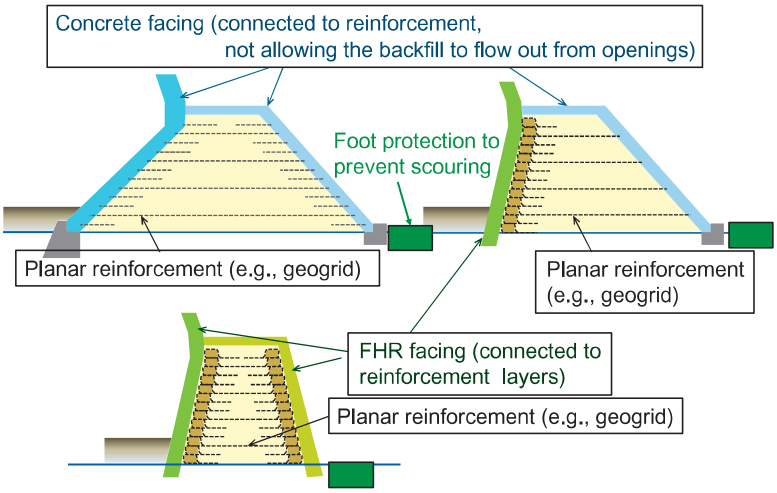


Figure 23.24 GRS coastal dikes as a tsunami barrier designed to survive deep overtopping tsunami current.

Figure 23.25 (a) Tsuyano-kawa bridge at JR East Kesen-numa Line that lost multiple simple-supported girders as a result of tsunami forces; and (b) a view of the back of the right bank abutment of Yonedagawa bridge at Noda, Iwate Prefecture, North Rias Line, Sanriku Railway.

backfill is easily eroded by overtopping tsunami current. In many cases, the connectors and anchors that had been arranged to prevent dislodging of the girders from the abutments and piers by seismic loads could not prevent the flow away of the girders by tsunami forces. These cases showed that the girder bearings and unreinforced backfill are two major weak points of the conventional-type bridges not only for seismic loads but also for tsunami loads. The results of small scale model tests (Kawabe et al., 2013, 2015) support this feature.

Tatsuoka and Tateyama (2012a) proposed the construction of GRS integral bridges (see Fig. 23.17) and geosynthetic-reinforced (GR) embankments/dikes (Fig. 23.24) to restore the conventional-type bridges and embankments of railways and roads that collapsed by the great tsunami of the 2011 Great East Japan earthquake. Small model tests (Kawabe et al., 2013, 2015) indicated that, due to a high structural integrity, GRS integral bridges have a much higher resistance against tsunami currents than conventional-type bridges.

The Sanriku Railway, opened in 1984, runs along the coastline where the tsunami damage was very serious. In particular, three bridges located between tunnels in narrow valleys facing the Pacific Ocean at three sites just south of the site shown in Fig. 23.23(b), totally collapsed. Figures 23.26, 23.27,

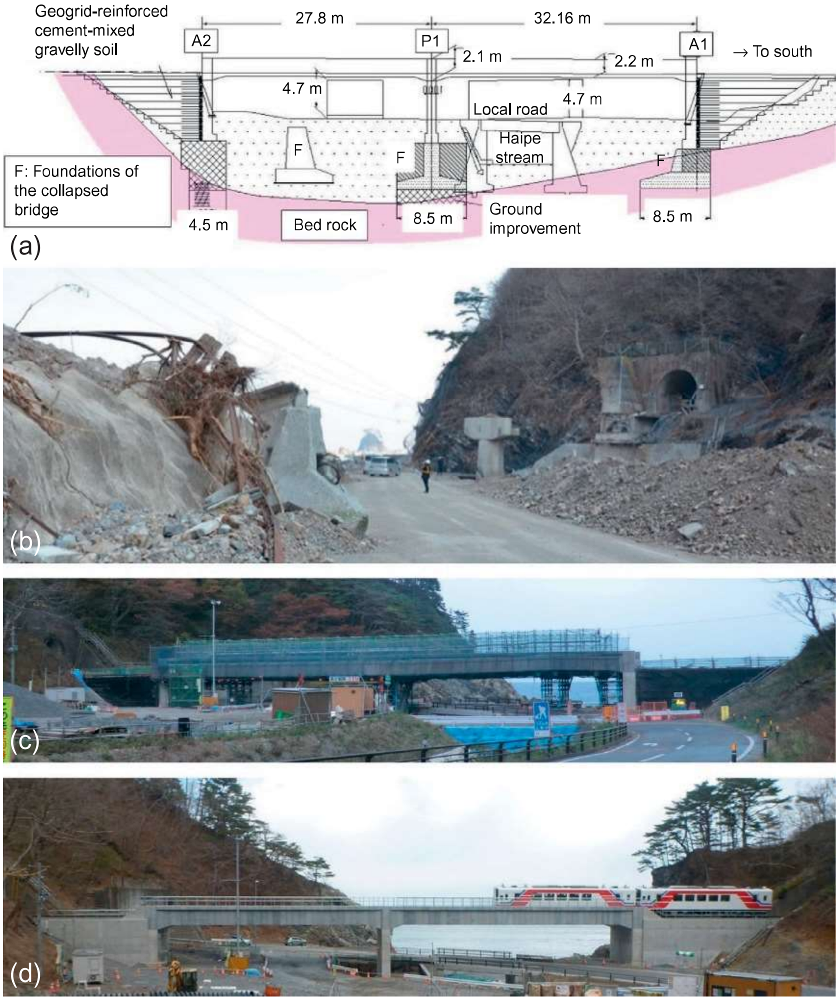


Figure 23.26 (a) Plan of GRS integral bridge seen from the inland side; (b) immediately after collapse (March 30, 2011); (c) under construction (November 3, 2013); and (d) completed (April 6, 2014) at Haipe, Sanriku Railway.

and 23.28 show these three sites. Tsunami loads were particularly high with these bridges, because: (1) the track level is lowest (12.3–14.5 m) at these three sites along this railway, (2) the sites are closest to the coastline, and (3) there was no coastal dike between the sites and the coastline. Based on the successful case histories described in the preceding sections and considerations that GRS integral bridges should have a high resistance against tsunamis, it was decided to construct GRS integral bridges to restore these three bridges. Figures 23.17 and 23.26 show two of the three GRS integral bridges. The total span length

Figure 23.27 (a) Plan of GRS integral bridge seen from the inland side; (b) immediately after collapse (March 30, 2011); (c) under construction (November 3, 2013); and (d) completed (April 6, 2014) at Koikorobe, Sanriku Railway.

of the GRS integral bridge at Haipe is 60 m, which is much longer than the one at Kikonai (see Fig. 23.19). The railway was reopened on April 6, 2014, about three years after the earthquake.

Figure 23.28(a) shows Shimanokoshi Station of Sanriku Railway before the earthquake. The level of the railway track at the site was about 14 m from

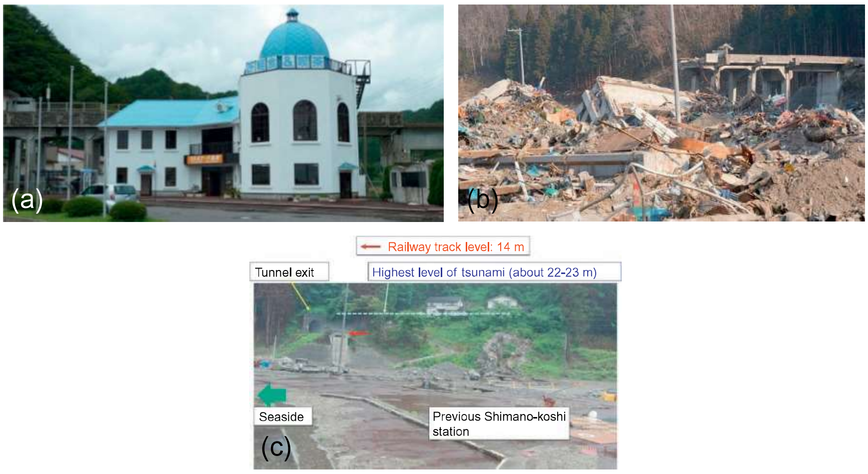


Figure 23.28 (a) A view before the earthquake; (b) seen from inland immediately after earthquake (March 30, 2011); and (c) a view on July 14, 2013, at Shimanokoshi Station, Sanriku Railway.

the sea level. This track level was determined based on the previous tsunami disasters in 1896 and 1933. However, the tsunami height this time was much higher (22–23 m at this site) and the tunnel was inundated (Fig. 23.28(c). The RC framework structure (i.e., the viaduct) was seriously damaged and the station was totally washed away (Fig. 23.28(b). On the request of the residents at the site, a GR embankment was constructed as a tsunami barrier following the proposal shown earlier in Fig. 23.24 in place of the previous RC framework structure (Fig. 23.29(a)). Figure 23.29(b) shows the

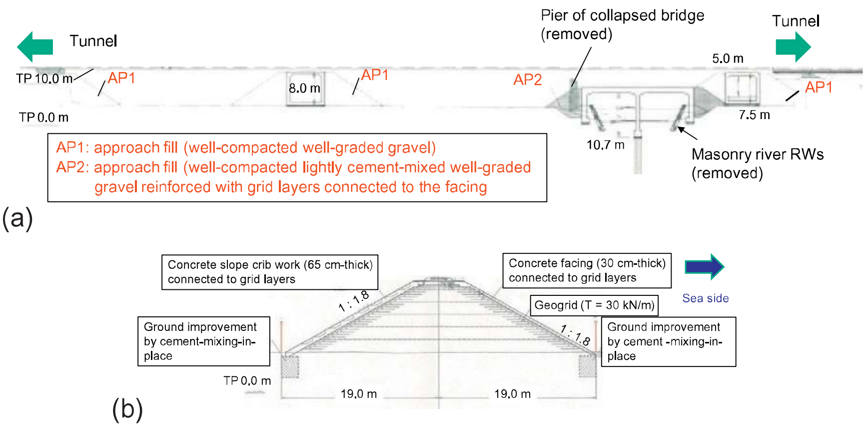


Figure 23.29 (a) Overall plan of GRS structures; (b) representative cross section of GR embankment;

(Continued)

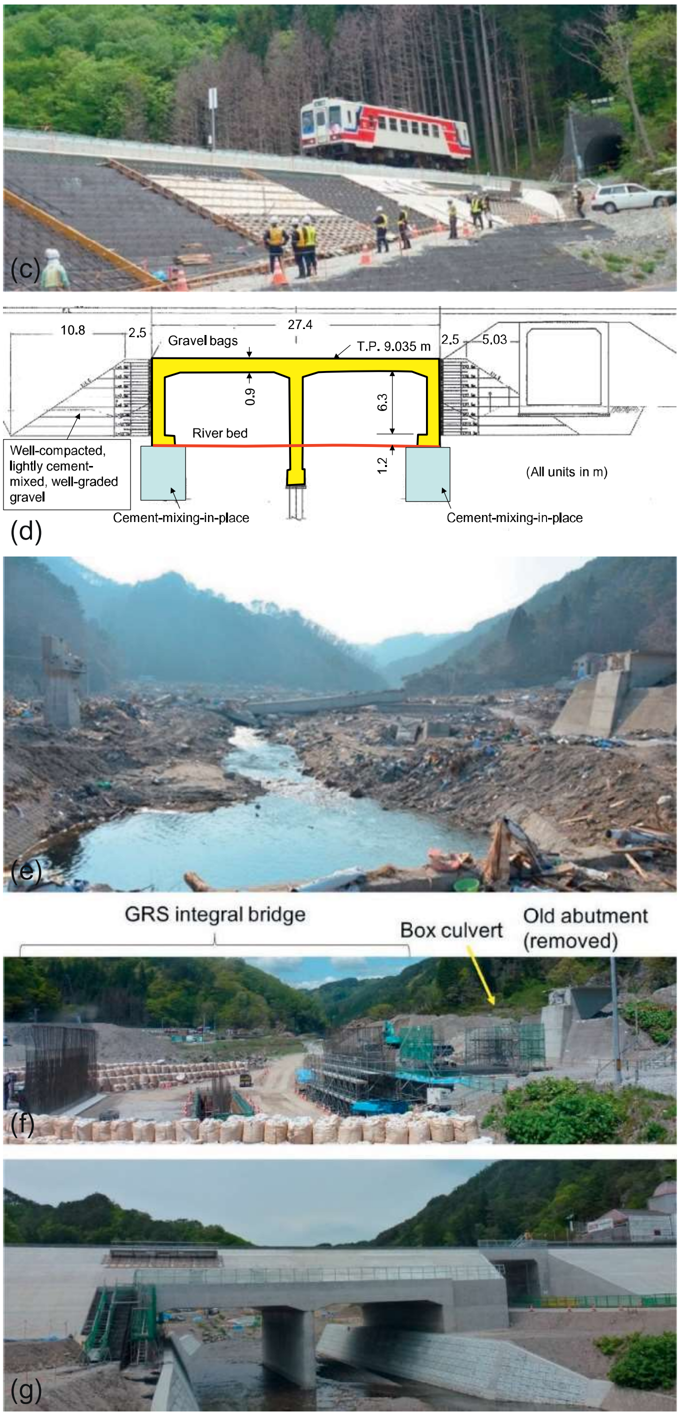


Figure 23.29 cont'd (c) embankment seen from the inland side (May 20, 2014); (d) GRS integral bridge; (e) immediately after earthquake (March 30, 2011) seen from the sea side; (f) GRS integral bridge and RC box culvert during construction (June 19, 2013); and (g) completed (May 20, 2014) at Shimanokoshi Station, Sanriku Railway.

representative cross section of the GR embankment and Fig. 23.29(c) shows a view of the completed GR embankment.

Both slopes of the embankment are covered with lightly steel-reinforced concrete facing firmly connected to the geogrid layers reinforcing the backfill. The restoration work at the site includes the construction of another GRS integral bridge (Fig. 23.29(d)). The bridge is covered with a backfill layer to reduce as much as possible the size of the opening. Figure 23.29 (e) shows a view at the site from the seaside immediately after the earthquake. Figs. 23.29(f) and (g) show the GRS integral bridge and RC box culvert during construction and after completion.

Based on the experiences described in this chapter, adopting GRS structures as described in this section for railway and road structures that are required to be designed against severe earthquakes and strong tsunami currents can be recommended. Even at locations where such a design is not necessary, GRS integral bridges are actually much more cost-effective and consequently several bridges are now at the stage of design and construction.

23.6 CONCLUSION

A number of geosynthetic-reinforced soil retaining walls (GRS RWs) having a staged-constructed full-height rigid (FHR) facing have been constructed as important permanent RWs in Japan. It is now the standard RW technology for railways, including high-speed train lines. Other types of GRS structure, including GRS bridge abutments, GRS integral bridges, and GRS coastal dikes, have been developed based on this GRS RW technology. These GRS structures are seismic-designed against very high design seismic loads (called Level 2) as experienced during the 1995 Great Kobe earthquake and the 2011 Great East Japan earthquake. The GRS structures described in this chapter have been designed and constructed to have high redundancy so that they perform well under extreme conditions, and this has been the case, as demonstrated by a number of case histories. With these GRS structures, the cost of this redundancy outweighs the cost of failure/collapse and increased maintenance.

The following conclusions can be derived from the case histories described in the chapter:

1. The current popularity of GRS structures for railways is due to a high cost-effectiveness (i.e., low construction/maintenance cost, high construction speed and high stability), in particular high performance during severe earthquakes.

2. The GRS integral bridge, comprising a continuous girder of which both ends are structurally integrated to the top of the facing of a pair of GRS RWs, has high resistance against seasonal thermal expansion and contraction of the girder, severe seismic loads and tsunami loads, and is highly cost-effective. As demonstrated by several case histories, it can be expected that this new bridge type is adopted in many other cases.
3. A number of conventional-type soil structures (i.e., embankments and RWs and bridge abutments) that collapsed by earthquakes, heavy rains, floods, and storm wave actions were reconstructed to GRS RWs with FHR facing, GRS bridge abutments and GRS integral bridges. This standardized practice is due also to a high cost-effectiveness of these types of GRS structure.
4. A great number of coastal dikes were fully eroded by the tsunami during the 2011 Great East Japan earthquake and a great number of bridges running along the seashore lost their girders and/or approach fills. GRS coastal dikes covered with continuous facing connected to geogrid layers reinforcing the backfill can perform much better than the conventional type, surviving both high seismic loads and subsequent deep overtopping tsunami current. Geosynthetic-reinforced embankments that function also as coastal dikes and GRS integral bridges were constructed to restore a railway that was seriously damaged by the tsunami.

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