Geosynthetic-reinforced soil technology in railway applications – from walls to bridges

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ABSTRACT: The history of the development and construction of various types of geosynthetic-reinforced soil (GRS) structures mainly for railways for the last thirty five years in Japan is described. The total wall length of these GRS structures, without any problematic case, became more than 170 km in 2018. In the 1980's, GRS retaining wall (RW) with full-height rigid (FHR) facing was developed. The FHR facing is staged-constructed firmly connected to the temporary wall face after the backfill and subsoil has deformed sufficiently. In the early 1990's, based on this GRS RW technology, GRS Bridge Abutment, supporting one end of a simple girder on the FHR facing via a fixed pin bearing, was developed. In the early 2000's, GRS Integral Bridge was developed, in which both ends of a continuous girder are structurally integrated to the top end of the FHR facings of a pair of GRS RWs. A number of these GRS bridge structures were constructed while many others are at the stage of design. It is explained that the use of FHR facing, the staged construction and the structural integration are the three major breakthroughs for the development of these GRS structures.

Keywords: FHR facing, geosynthetic, GRS Bridge Abutment, GRS Integral Bridge, GRS Retaining Wall, high speed railway, staged construction

1 INTRODUCTION

This paper reports briefly the history for the last thirty five years of the development and construction of various types of geosynthetic-reinforced soil (GRS) structures mainly for railways (including high speed railways, HSRs) and also for roads and others in Japan. The development of these GRS structures started from retaining walls (RWs) and has been extended to bridge structures. During the 1980's, we constructed consecutively five test embankment retained by GRS RWs (Tatsuoka & Yamauchi, 1996, Tatsuoka et al., 1990, 2000, 2008). The first one was a clay embankment and the wall face was wrapped-around with a non-woven geotextile used as reinforcement. It exhibited very large deformation. Subsequently, the other four, having gradually improved facing structure, were constructed. A series of model tests were also performed in the laboratory to evaluate the roles of facing rigidity and effects of the length of reinforcement (Tatsuoka et al., 1989; Tatsuoka, 1992). Finally, GRS Retaining Wall (RW) with staged-constructed full-height rigid (FHR) facing was proposed at the end of the 1980's (Fig. 1a, Tatsuoka et al., 1997b). It was confirmed that FHR facing is essential for a high wall stability with small deformation against mechanical, hydraulic and fire attacks to the wall face and against long-term traffic loads, severe seismic loads, heavy rains, floods and so on. Moreover, it was confirmed that the length of the geogrid reinforcement layers at low elevations can be made relatively short maintaining a sufficiently high wall stability by using FHR facing and extending several reinforcement layers at high elevations to the plane of repose when unreinforced. This reinforcement arrangement reduces the amount of excavation when constructing GRS RWs on existing slopes and makes unnecessary the use of anchors and sheet piles. As shown in Fig. 1a, after the deformation of backfill and subsoil has taken place sufficiently, the FHR facing is constructed by casting-in-place concrete directly on the temporary wall face comprising gravel-filled bags wrapped-around with reinforcement (i.e., geogrid). In this way, the FHR facing and the reinforcement layers are firmly connected while the connection is not damaged by relative settlement between them that would take place if the facing were constructed before or simultaneously with wall construction. Until today (April 2018), GRS structures with FHR facing have been constructed for a total length more than 170 km, all successfully without any problematic case (Fig. 2).

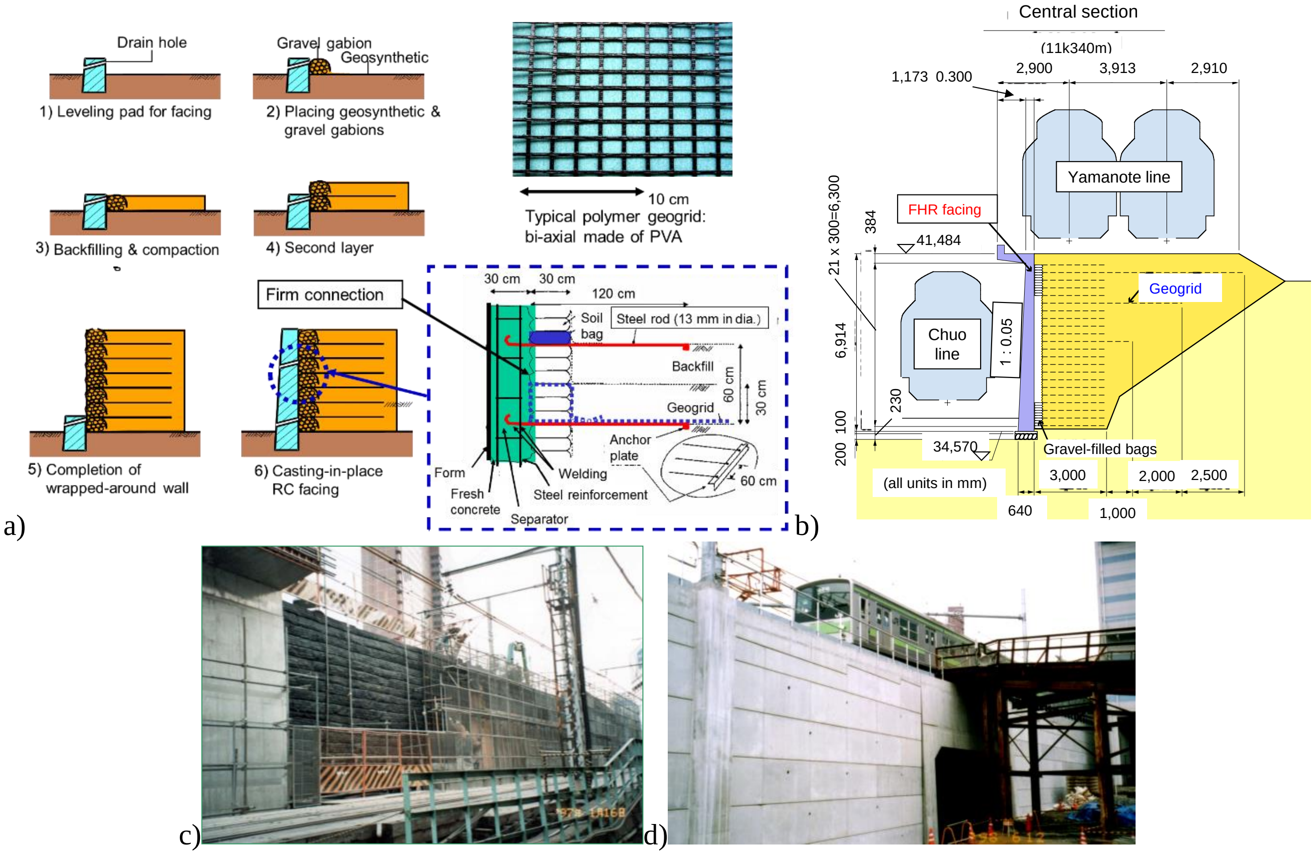


Figure 1. a) Staged-construction of GRS RW with FHR facing; and a typical wall near Shinjuku Station, Tokyo, constructed during 1995 – 2000: b) cross-section; and c) & d) views during construction and completed.

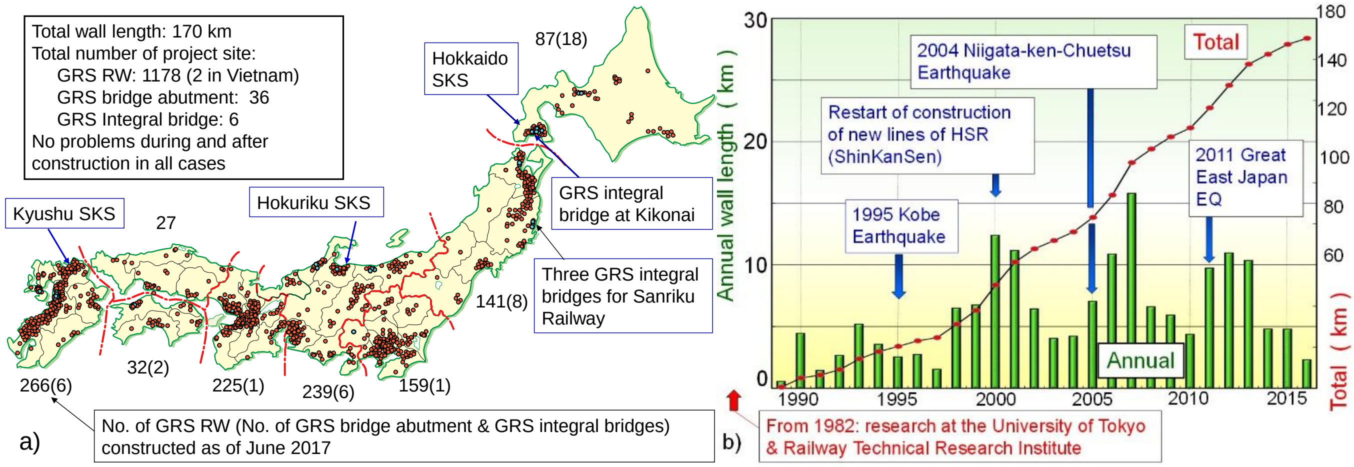


Figure 2. a) Locations; and b) statistics of GRS structures with FHR facing constructed as of June, 2017.

Polymeric reinforcement (i.e., geosynthetic reinforcement) is more extensible exhibiting a stronger trend of creep deformation than metallic reinforcement (often defined as inextensible reinforcement) under otherwise the same conditions. However, it was confirmed by a comprehensive series of experimental and theoretical study that the backfill soil, which has essentially no strength and stiffness in tension, can be effectively tensile-reinforced with polymeric reinforcement while post-construction creep deformation of GRS structure can be effectively retrained by proper structural design, adequate construction, in particular good compaction, and proper drain (Tatsuoka, 2004).

FHR facing can support other structures on its top, such as noise barrier walls, crash barrier walls, electric power supply facilities etc. Taking advantage of this characteristic feature, Tatsuoka et al. (2005) proposed GRS Bridge Abutment, in which the FHR facing of a GRS RW supports one end of a simple girder via a fixed pin bearing, after having attempted several other bridge abutment types, including a preloaded/prestressed GRS abutment and pier (Tatsuoka et al., 1997c; Shinoda et al., 2003a, b; Uchimura et al., 2003, 2005). More than 60 GRS Bridge Abutments have been constructed while many others are at the design stage. Extending this technology, Tatsuoka et al. (2009) proposed GRS Integral Bridge (Fig. 3), in which a continuous girder is constructed with both ends structurally integrated to the top of the FHR facings of a pair of GRS RWs. Its design and construction codes have been published (Yazaki et al., 2013; Koda et al., 2018).

For this very long project, the author worked with a great number of students and researchers of University of Tokyo, Tokyo University of Science and Railway Technical Research Institute, Japan, and engineers of Japan Railway Construction, Transport and Technology Agency, a number of railway companies, consulting and construction companies and others. This report summarizes lessons learned from many projects in which the author and these many people were, and are being, involved.

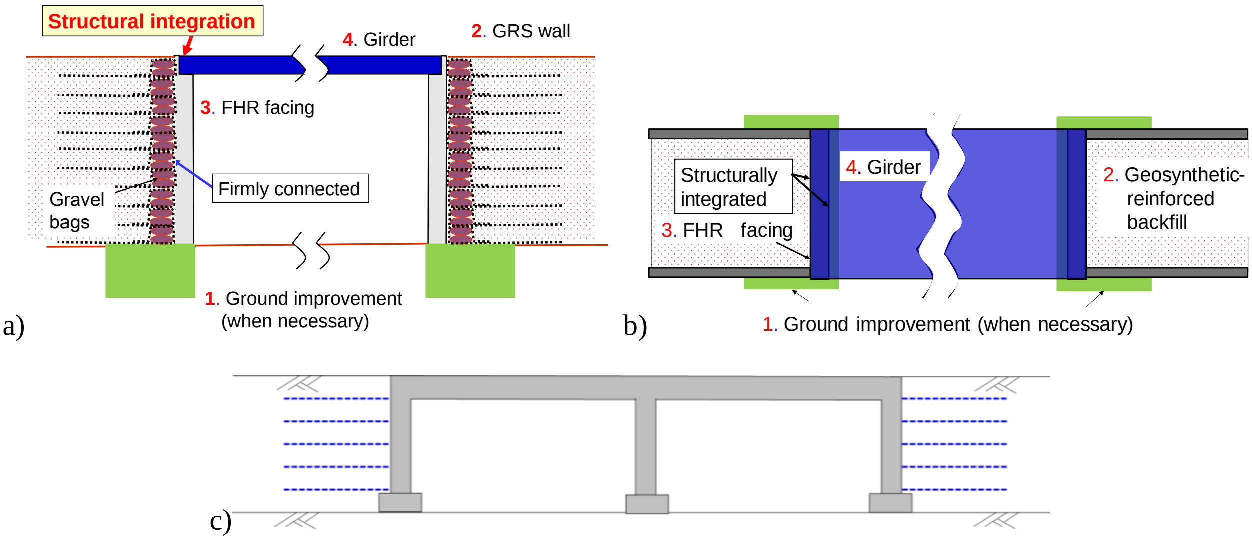


Figure 3. GRS Integral Bridge: a) elevation; b) plan; and c) a long-span girder vertically supported by a pier at its center (Tatsuoka et al., 2009, 2014, 2015, 2016).

2 GRS RETAINING WALLS

2.1 *From wrapped-around facing to FHR facing*

At the beginning stage of our research (i.e., in the early 1980's), we were not aware of the paramount importance of the stiffness (or rigidity) of the facing for wall stability. We learned it from the performance of five full-scale test embankments consecutively constructed in the 1980's. The wall facing was gradually improved: from wrapped-around, via soil bags, shotcrete and discrete panels, finally to FHR (Fig. 1a). The first test embankment (Fig. 4) was constructed in June 1982 at Chiba Experiment Station, Institute of Industrial Science, University of Tokyo to examine whether stable near vertical walls can be constructed even by reinforcing clayey soil with very flexible reinforcement: i.e., a non-woven geotextile (spun-bonded 100 % polypropylene) that is usually used as a drain material. The backfill was on-site volcanic ash clay, called Kanto loam, with water content w_i= 100 – 129 %; S_r= 85 – 90 %; and ρ_d= 0.55 – 0.69 g/cm^3. Although intact natural deposits of this soil are rather stable due to a small true cohesion, this soil becomes very soft once remolded during earthwork, becoming a so-called difficult soil to construct soil structures (embankments, retaining walls etc.). The wall face was wrapped-around with a non-woven geotextile (without using soil bags) and nearly flat when constructed. The wall on the left side shown in Fig. 4b, in which the initial vertical spacing between adjacent non-woven geotextile sheets was equal to 80 cm, deformed largely already during construction and much more by heavy rainfalls after wall completion. By the mechanism illustrated in Fig. 4e, the compression of soil layers at low elevations was trigged by the loss of matrix suction due to wetting during rainy days. This resulted in the loss of contact between the backfill and the wrapping-around geotextile, which did not allow the confining pressure to develop and accelerated soil compression. The other wall on the right side shown in Fig. 4b, having an initial vertical spacing between non-woven geotextile sheets equal to 40 cm, also deformed noticeably (Fig.

4c), although it was less than the left side wall. We learned that large deformation is one of the actual serious problems with wrapped-around GRS RWs.

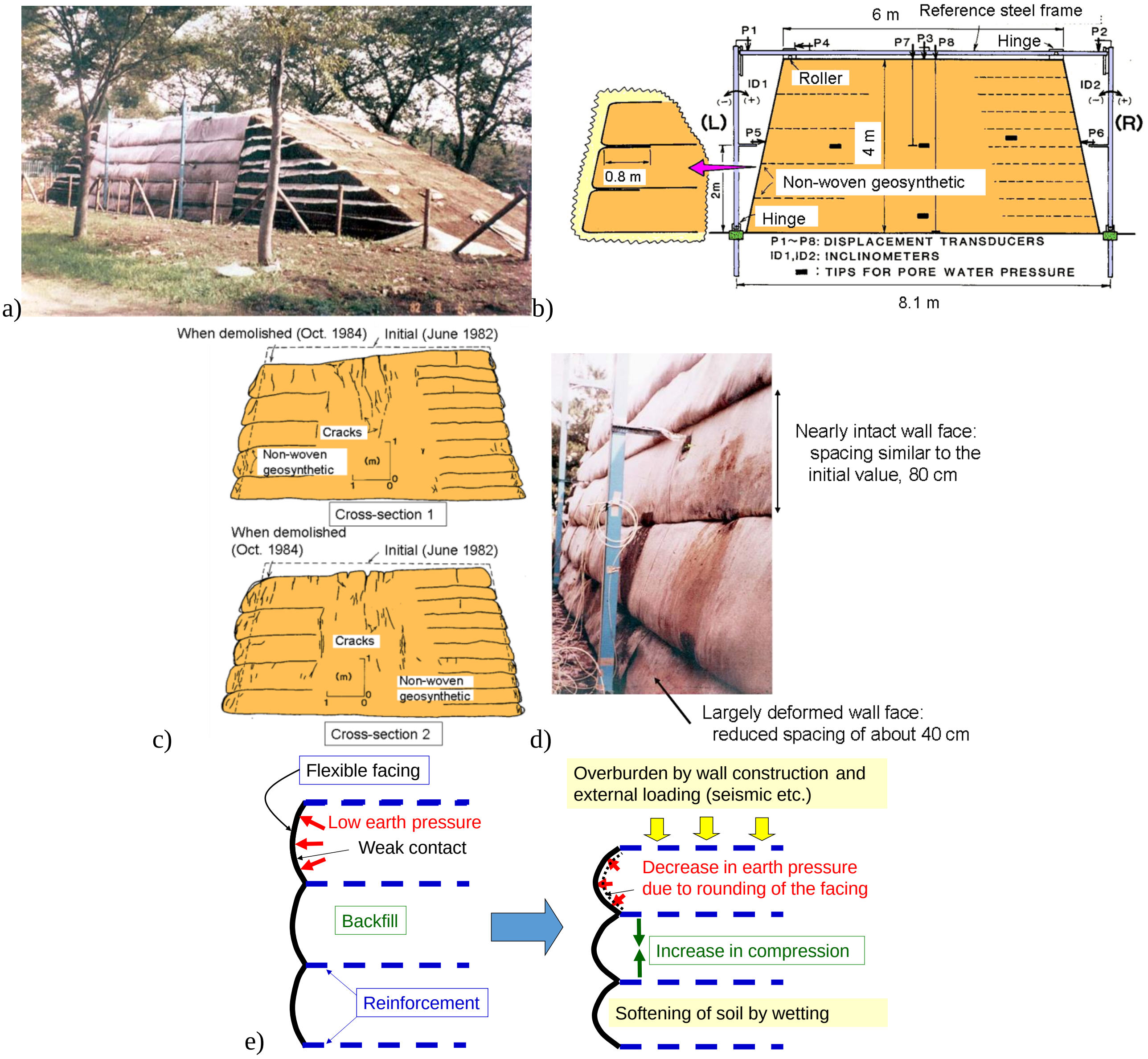


Figure 4. Chiba No. 1 embankment of clayey soil (Tatsuoka & Yamauchi, 1986; Tatsuoka et al., 1990, 2000, 2008): a) immediately after completion; b) initial cross-section; c) cross-section exposed when demolished (about two years later after the construction); d) wall face after large deformation at the bottom of the wall by several heavy rainfalls; and e) mechanism of the development of large deformation at flexible wall face.

Chiba No. 2 embankment (Fig. 5a) was constructed in March 1984 using the same types of clayey soil and reinforcement of non-woven geotextile as No. 1 to evaluate the effects of soil bags placed at the shoulder of each soil layer wrapped-around with the reinforcement on the construction and stability of GRS RW. The soil bags were made of the same non-woven geotextile and filled with the same clayey soil as those used to construct the main body of the embankment. The post-construction wall deformation for a period of one and a half years was much smaller than No.1. To study the failure mechanism, the walls were brought to failure by supplying in total 70 m^3 of water from the crest for a period of eight days in October 1985. Although the two walls largely deformed, in particular the right side wall having very short reinforcement, they did not collapse (Fig. 5b). It was found that the use of such soil bags as those shown in Fig. 5a is very effective for both good soil compaction and high wall stability.

Chiba No. 3 embankment (Fig. 6a) was constructed in 1986 also using the same clayey soil and non-woven geotextile as Nos. 1 and 2 to directly compare the behaviours of the GRS RWs having different facing structures: wrapped-around; discrete concrete panels (w/o soil bags); and wrapped-around soil bags covered with a 8 cm-thick shotcrete layer. Their behaviours were very different showing the paramount effects of facing structure on the wall stability (Fig. 6b). The wall with wrapped-around facing (w/o soil bags) largely deformed during the subsequent several years after wall completion. The deformation of the wall with facing of relatively small pre-cast concrete panels (50 cm x 50 cm x 5 cm-thick with a weight of 34 kgf) was much smaller. Yet, the deformation was noticeably, showing that this type of facing is not rigid enough. Besides, it was very difficult to assemble these panels on site and compact backfill soil behind the facing ensuring a good wall face alignment. The deformation of the wall with facing of clay-filled soil bags wrapped-around with reinforcement of non-woven geotextile that was covered with a shotcrete layer was also much smaller than the deformation of the rear side wall. Yet, the deformation was noticeable to be used as a permanent RW allowing a limited amount of deformation while the shotcrete facing was not aesthetically acceptable unless when constructed in a remote area. On the other hand, the performance of these two walls at the central section indicates that it is feasible to construct vertical walls acceptable as ordinary permanent important RWs even by reinforcing nearly saturated clayey soil with so called extensible reinforcement (such as non-woven geotextile) if the facing is more rigid and aesthetically acceptable.

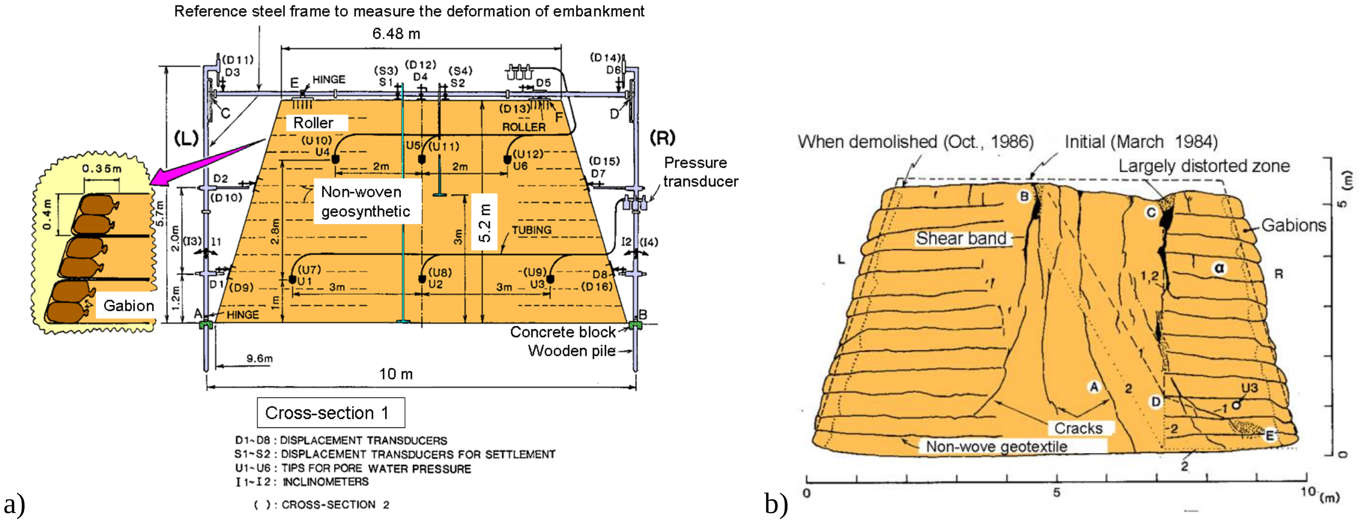


Figure 5. Cross-sections of Chiba No. 2 embankment: a) when completed; and b) one year after an artificial heavy rainfall test (Tatsuoka & Yamauchi, 1986; Tatsuoka et al., 1990, 2000, 2008).

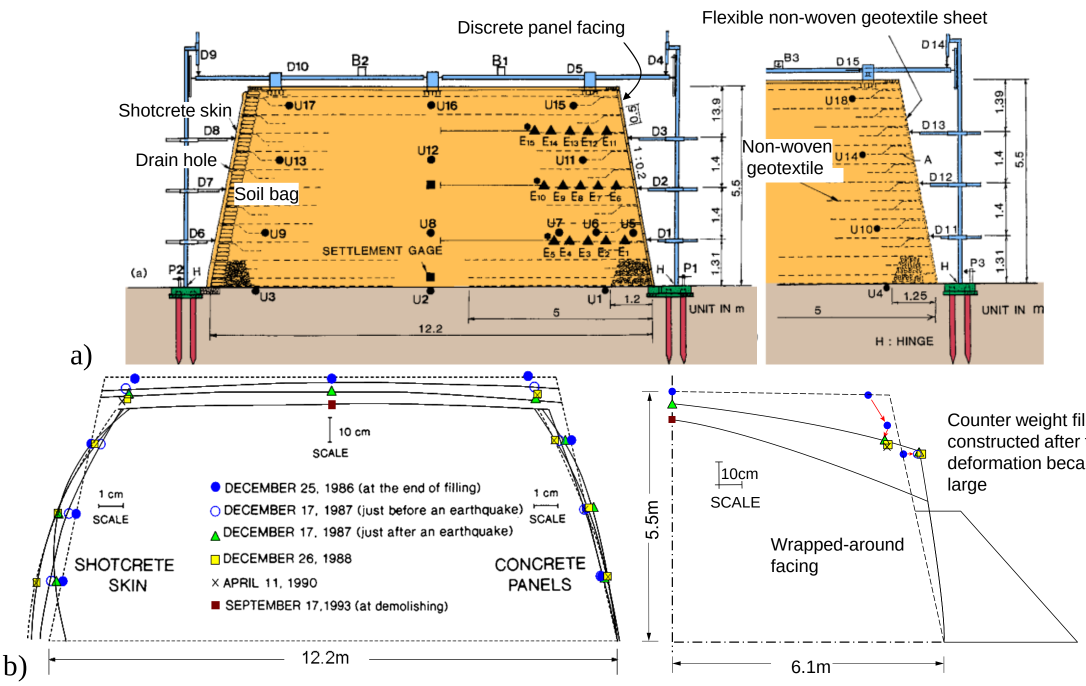


Figure 6. Chiba No. 3 embankment: a) cross-sections at the center and at the rear side; and b) deformation of the cross-sections at the center and at the rear side (Tatsuoka et al., 1990, 2000, 2008).

These experiences with the three test embankments, described above, led us to the GRS RW with staged-constructed FHR facing depicted in Fig. 1a. JR No. 1 test embankment was constructed using Inagi sand during a period of 1987 – 1988 (Fig. 7a) to examine whether this type of GRS RW can support important railways including high speed railways (HSRs). The reinforcement was a polymeric geogrid having a tensile strength equal to 2.8 tonf/m. The following two types of facing were employed to confirm the importance of facing rigidity. That is, five wall segments had staged-constructed FHR facing, as shown in Fig. 1a, but without steel-reinforcement unlike the FHR facing currently used for all the GRS RWs. Only segment *h* had discrete panel facing with each panel being fixed to bags filled with gravelly soil. Subsequently, JR No. 2 test embankment of clay backfill (Fig. 7b) was constructed in the beginning of 1988 to examine the effects of backfill soil type on the stability of GRS RW with FHR facing. The clayey soil is volcanic ash clay (called Kanto loam) similar to the one used to construct the three Chiba test embankments, having a high initial water content w_i equal to 120 - 130 %, a high degree of saturation S_r= 90 %; and a low dry density ρ_d= 0.55 - 0.60 g/cm^3. Three test sections of the clayey soil embankment were reinforced differently: section a-a with a non-woven geotextile (as Chiba Nos. 1, 2 and 3 test embankments); section b-b with a polymeric geogrid sandwiched between two thin gravelly soil drainage layers; and section c-c with a geo-composite consisting of a woven geotextile sheet for tensile-reinforcing sandwiched by two layers of non-woven geotextile layers for drainage. Despite different reinforcement materials and backfill soil types, all the segments having FHR facing of the two embankments performed very well for a period of about two years after wall construction, whereas segment *h* (having discrete panel facing) of No. 1 embankment exhibited noticeable deformation (Fig. 7a). This case showed again the paramount importance of facing rigidity for the stability of GRS RW.

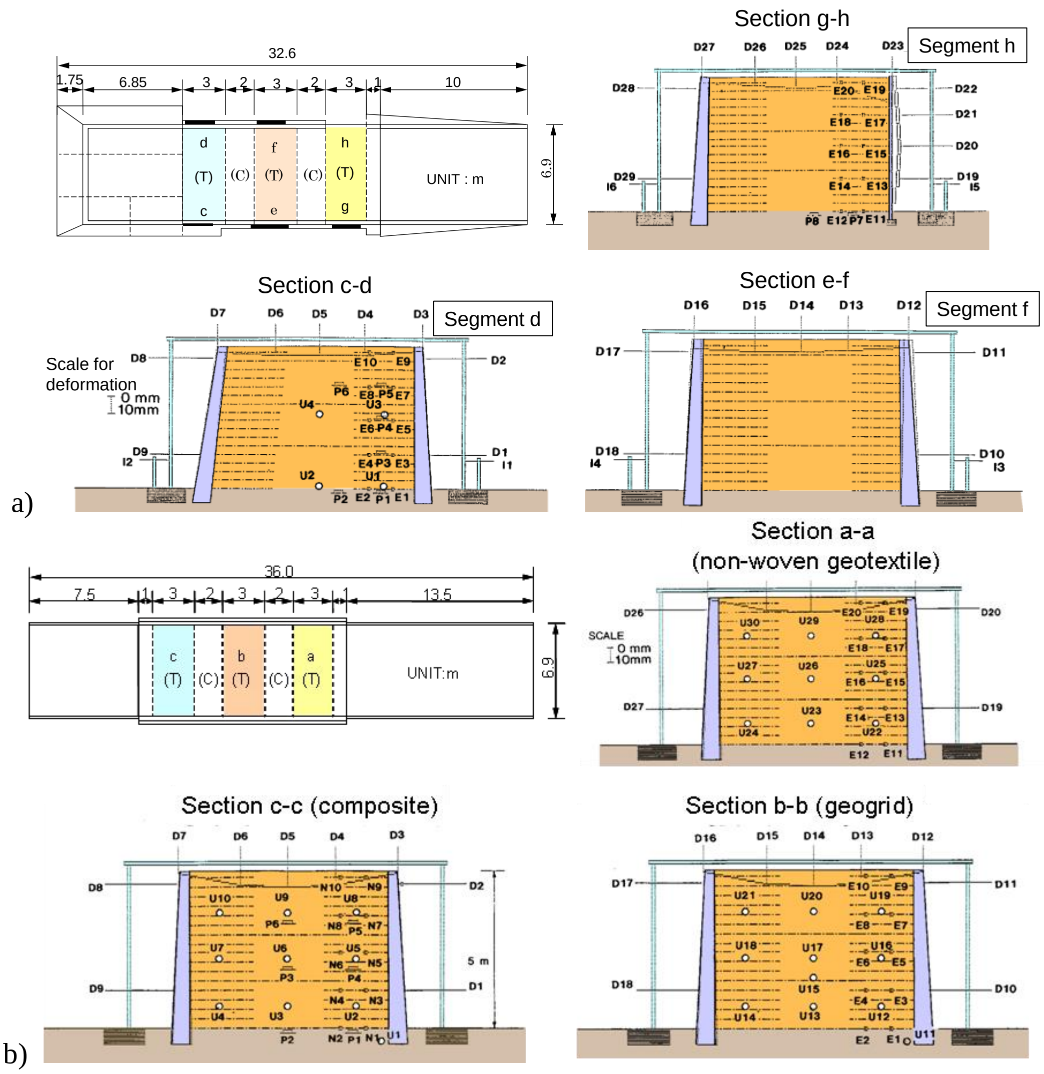


Figure 7. Plan and cross-sections two years after wall construction: a) JR No. 1 test embankment with sand backfill; and b) JR No. 2 test embankment with clayey soil backfill (Tatsuoka et al., 1990, 2000, 2008).

Segments *d*, *f* and *h* were vertically loaded to examine the ultimate wall stability (Figs. 8a & b). As seen from Fig. 8c, segment *h* exhibited the largest deformation. Fig. 8d shows the cross-section of segment *h* exposed after the loading test. A set of shear band (i.e., failure plane) developed from the footing heel, which extended first vertically and then towards the bottom of the wall face, inducing buckling of the facing at an intermediate height (Fig. 8b). A two-wedge failure mechanism developed with the failure plane entering the reinforced zone at low elevations. These trends of behavior indicate that this type of facing is not rigid enough to be used for permanent important wall structures allowing a limited amount of deformation. With the same relatively short geogrid length L= 2 m compared with a wall height H= 5 m, segment *d* (with FHR facing) exhibited much smaller wall deformation than segment *h*. With the same FHR facing, segment *f* with even shorter reinforcement (L= 1.5 m) exhibited less stable behaviour than segment *d* (L= 2.0 m). Importantly, segment *f* is rather stable when subjected to ordinary design load at the crest (i.e., q= 30 kPa) in spite of very short reinforcement (i.e., L= 1.5 m). The full-height concrete facing for segments *d* and *f* had no steel reinforcement (unlike the current practice). They failed at the upper construction joint (denoted by letter CJ in Fig. 8b), controlling the yield strength of segments *f* and *d* (Fig. 8c). It was considered that, if the facing had been made stronger by using slight steel-reinforcement, segments *d* and *f* would have been more stable. The current design code of GRS RW specifies that the FHR facing is lightly steel-reinforced. Besides, a bi-axial geogrid of PVA is usually used because of its high resistance against high PH environment by concrete, high adhesiveness with concrete and good anchorage in the facing concrete and the backfill. The vertical spacing between the geogrid layers is 30 cm to ensure good backfill compaction in a lift of 15 cm and strong integration of the FHR facing to the reinforced backfill.

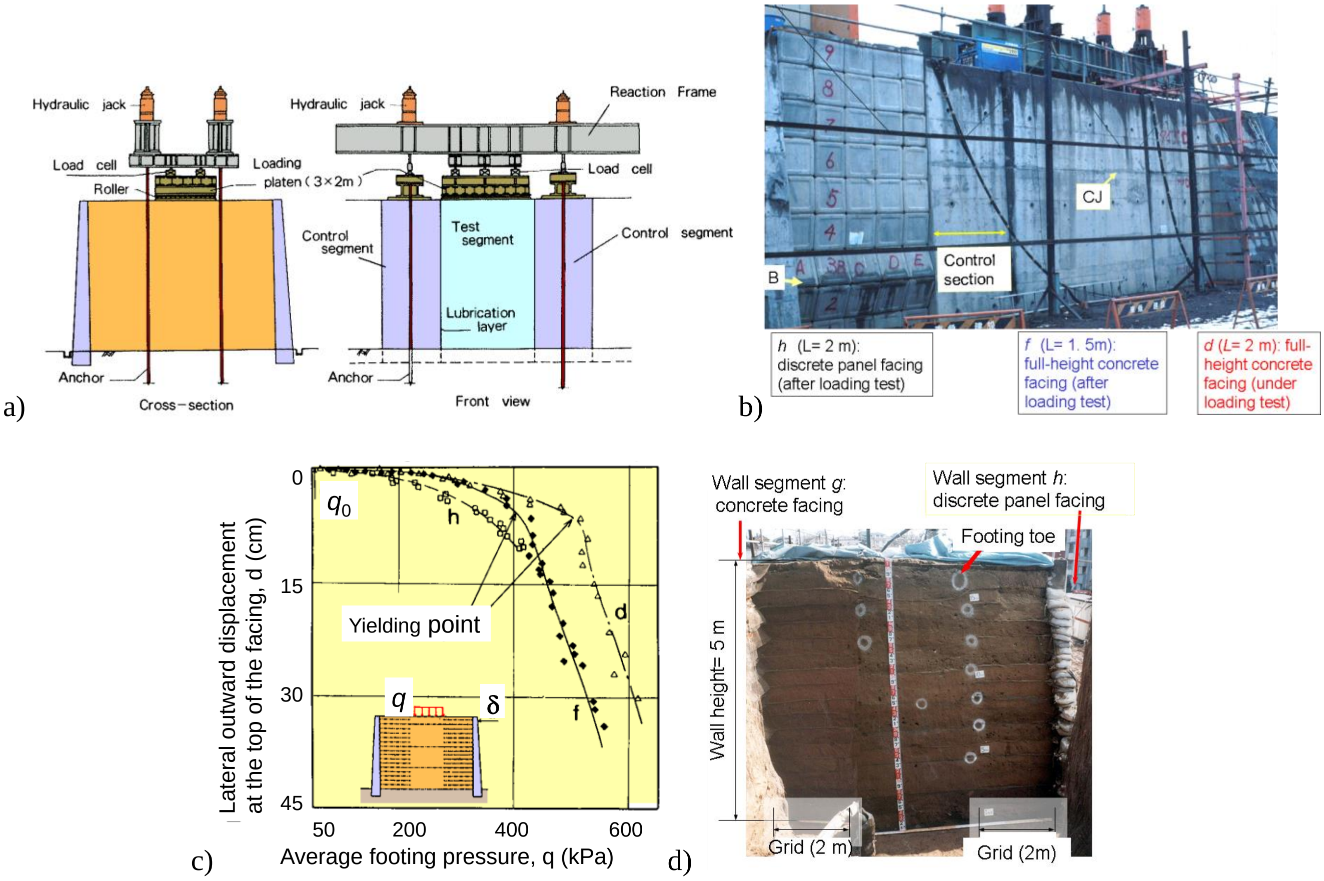


Figure 8. Vertical loading test of JR No. 1 sand embankment (Tatsuoka et al., 1990, 2000, 2008): a) loading method; b) view of wall segments *d*, *f* and *h* after loading test; c) lateral displacement at the facing top vs. footing load relations of wall segments *d*, *f* and *h* (q_0= average pressure due to the weight of loading apparatus); and d) shear bands observed in segment *h* (having discrete panel facing).

2.2 *Reinforcement and facing, stiff versus flexible*

There are four combinations of facing (stiff vs. flexible) and reinforcement (stiff vs. flexible) (Table 1). It seems that, previously, many considered that inextensible reinforcement should be used, not extensible reinforcement, to better retrain the wall deformation, while stiff (or rigid) facing is unnecessary but flexible (or deformable) facing is sufficient. Polymeric reinforcement, such as geogrid, was defined as extensible reinforcement exhibiting large creep deformation. The isochronous theory was used to model the load-strain-time behavior of polymeric reinforcement. According to this theory, the ultimate rupture strength at the end of life time decreases by creep deformation at intermediate stages. These notions gave a wrong impression about polymeric reinforcement to civil engineers (Tatsuoka et al., 2004).

A combination of metallic strips as inextensible reinforcement and flexible facing of metallic skin was employed with Terre Armée RWs at the initial stage (Fig. 9a). Later, the metallic skin facing was replaced by concrete panel facing (Fig. 9b). It was explained that this replacement is for easier construction and better aesthetics. The author considered however that this was due also to that the metallic skin facing cannot effectively retrain the wall deformation by the mechanism illustrated in Fig. 4e. Also with GRS RWs, flexible wrapped-around facing (Fig. 9c) was often employed to construct not only temporary walls but also permanent walls. The performance of many wrapped-around GRS RWs, typically those explained in the above, showed that this type of facing is not suitable for permanent important RWs because of a low durability of wall face and relatively large wall deformation. In this respect, there was a mixing-up of different issues, the stiffness of facing and the stiffness of reinforcement, among many engineers and researchers. Many considered that large wall deformation, if it takes place, is due to the use of flexible (i.e., extensible) reinforcement (i.e., polymeric reinforcement). It is true that the wall deformation during construction may become somehow larger when using polymeric reinforcement than when using metallic strip reinforcement under otherwise the same conditions. However, a limited amount of wall deformation during construction is usually not a practical issue. In most cases, excessive post-construction wall deformation is due to the use of flexible facing, enhanced by poor backfill compaction and poor drain, in many cases. The performance of many GRS RWs with modular block facing (Fig. 9d) and FHR facing (Fig. 9e) indicated that properly designed GRS RWs having relatively stiff facing exhibit excellent post-construction performance despite the use of relatively extensible reinforcement.

Moreover, the results of a comprehensive series of research on the visco-elasto-plastic load-deformation properties of polymeric geosynthetic reinforcement showed that the post-construction time-dependent deformation of GRS structure is a combination visco-plastic deformations of backfill and geosynthetic reinforcement (e.g., Tatsuoka et al., 2004; Kongkitkul et al., 2010). They showed a case history of a high GRS walls in which the tensile forces working in the geosynthetic reinforcement decreased with time without showing a possibility of ultimate creep failure. As a more fundamental point, with ordinary construction materials including polymeric reinforcement, creep is not a degrading phenomenon and the ultimate strength does not decrease by creep deformation at intermediate loading stages unless the mechanical properties deteriorate by bio-chemical effects with time (e.g., Hirakawa et al., 2003; Tatsuoka et al., 2004; Kongkitkul et al., 2004).

Table 1. Classification of reinforced soil RW according to reinforcement type and facing structure type.

Facing / Reinforcement	Flexible, not developing high earth pressure on the back of facing	Stiff (i.e., rigid), developing high earth pressure on the back of facing
Stiff (or inextensible): e.g., metallic strip	Metallic skin（Fig. a）	e.g., Discrete concrete panel （Fig. b）
Flexible (or extensible): e.g., polymeric & planar (grid or sheet)	Wrapped-around (Fig. c）	e.g., Modular block (Fig. d); Discrete concrete panel; or FHR (Fig. e)

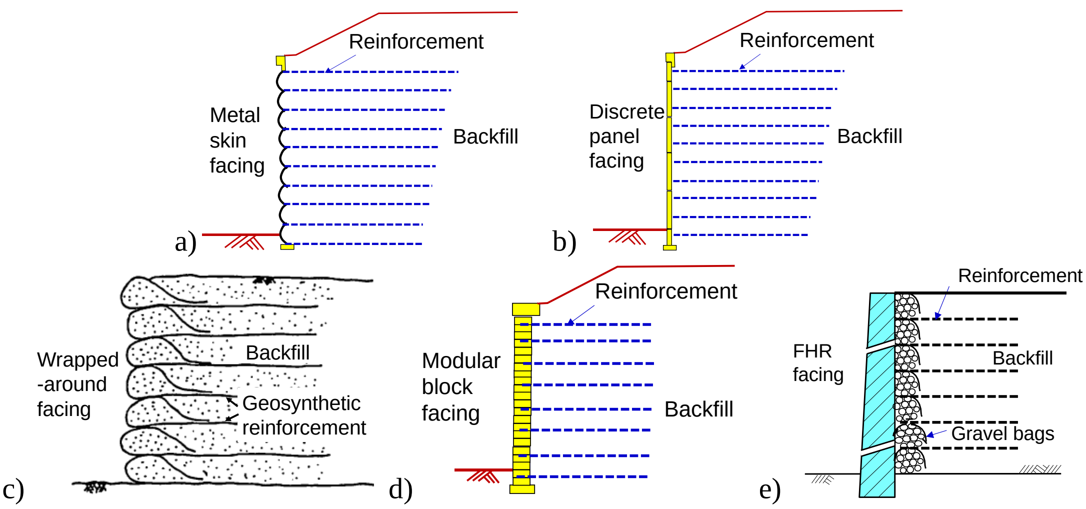


Figure 9. Different combinations of facing and reinforcement (refer to Table 1).

2.3 *Advantages of FHR facing*

The use of FHR facing is effective for not only a high durability of wall face but also a high wall stability in the following ways:

Non-cantilever structure: Conventional type RW is basically a cantilever structure that resists the earth pressure activated by the unreinforced backfill (Fig. 10a). Therefore, large internal moment and shear forces are activated in the facing, while large overturning moment and lateral thrust forces develop at the base of the facing, which usually makes necessary the use of a pile foundation. As the lateral thrust force at the facing base is basically proportional to the square of wall height (H) and the overturning moment to H^3, the conventional type RW become less cost-effective at an increasing rate with an increase in H. With GRS RWs with FHR facing (Fig. 1), on the other hand, a thin lightly steel-reinforced FHR facing without a pile foundation is usually sufficient. This is because the FHR facing is a continuous beam supported at many geogrid reinforcement layers with a small span, typically 30 cm (Fig. 10b). Therefore, only small forces are activated in the FHR facing, which results in a much simpler facing structure, while insignificant overturning moment and lateral thrust forces develop at the facing base, which makes unnecessary the use of a pile foundation in usual cases.

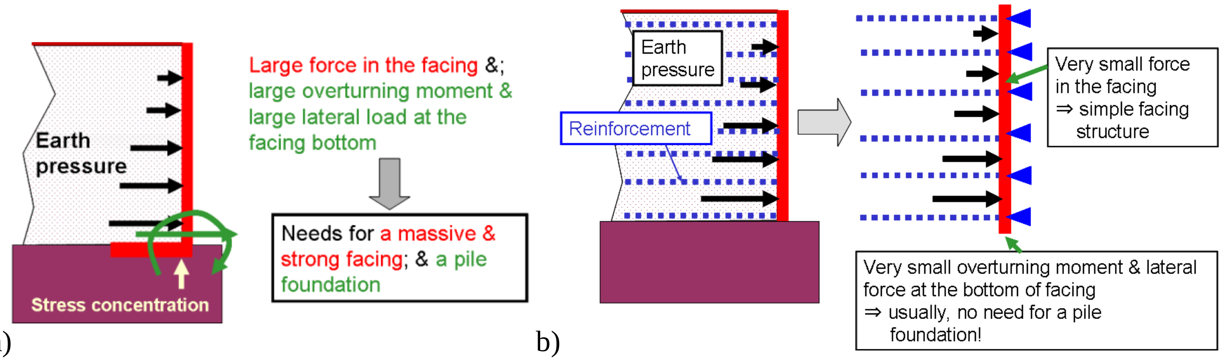


Figure 10. a) Disadvantages of conventional type cantilever RW; and b) advantages of GRS RW with FHR facing as a continuous beam supported at many points (Tatsuoka, 1993; Tatsuoka et al., 1997b).

High earth pressure: If the face is flexible or if the reinforcement layers are not connected to stiff facing, only insignificant or no earth pressure is activated at the wall face and only insignificant or no connection forces are activated between the facing and the reinforcement (Fig. 11a). A significant reduction in the earth pressure at the wall face from the one with conventional type RWs was often considered as one of the primary advantages of reinforced soil RW. However, this notion is theoretically wrong and quite misleading in practice. That is, insignificant or no earth pressure at the wall face results in insignificant or no lateral confining pressure in the active zone, which results in very low strength and stiffness of the backfill, in particular in the zone close to the wall face, which results in unacceptable large wall deformation and a low wall stability.

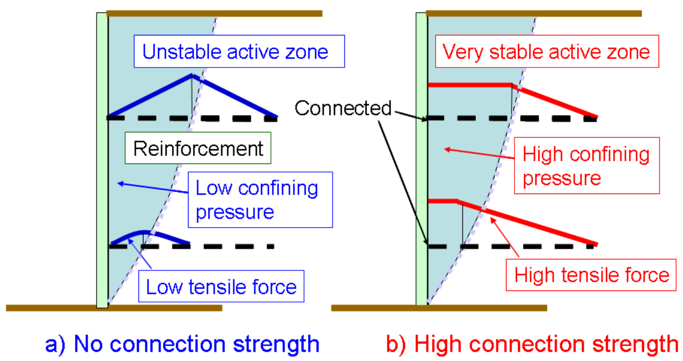


Figure 11. Different wall behaviours: a) no connection force between the reinforcement and the facing; and b) high connection force (Tatsuoka, 1992).

On the other hand, Schlosser (1990) reported the distributions of tensile forces along metallic strip reinforcement during backfilling in a typical Terre Armée wall (Fig. 12a) and in a full-scale model having relatively short reinforcement (Fig. 12b) both having standard discrete concrete panel facing. In both cases, the ratio of the reinforcement tensile force at the back face of facing (i.e., the connection force) T_w to its maximum value T_{max} is generally large, not smaller than 50 %, and is larger at lower elevations. FEM analysis (Fig. 12c) shows that the ratio T_w/T_{max} increases with the stiffness (or rigidity) of facing. Tatsuoka (1992) reports many other similar cases of full-scale walls and model walls showing a similar trend. Schlosser (1990) stated that "for standard reinforced concrete panel facing, and for depths greater than 0.6H, the ratio T_w/T_{max} can approach one". These results clearly indicate that discrete concrete panel facing can actually confine very well the active zone with the earth pressure being nearly equal to the active earth pressure in the unreinforced backfill.

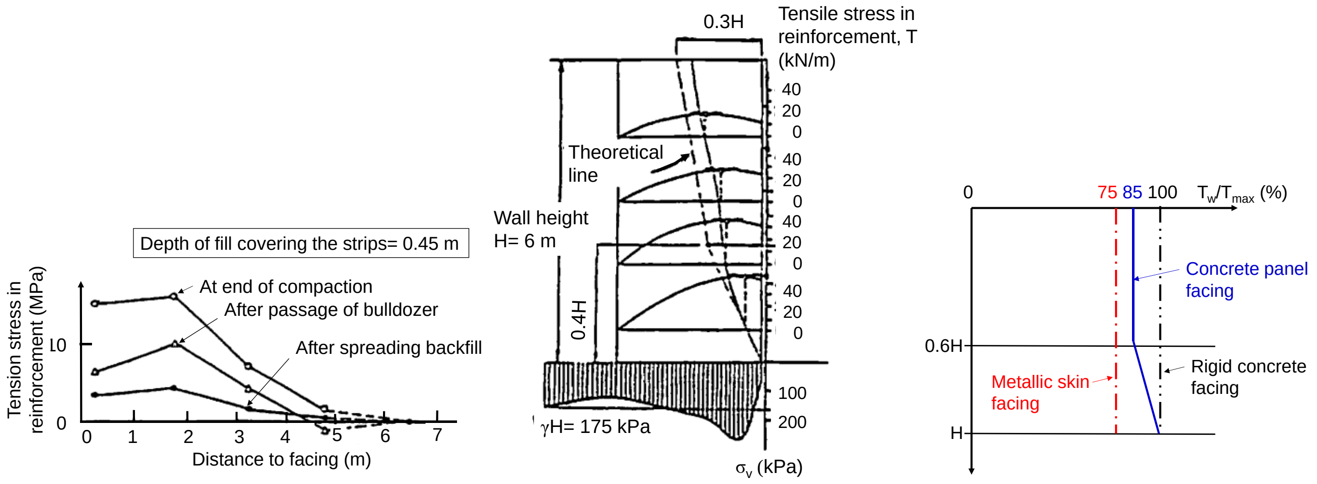


Figure 12. Distribution of tensile forces along reinforcement for Terre Armée RWs: a) measured during backfilling; b) measured in a test wall (H= 6 m): c) the effect of facing rigidity by FEM analysis (Schlosser, 1990).

With GRS RW with FHR facing (Fig. 1), the gravel-filled bags wrapped-around with geosynthetic reinforcement function as a temporary facing structure having some stiffness (or rigidity) that can develop high earth pressure at the wall face during wall construction. As they are subsequently integrated to the FHR facing, the reinforcement layers are firmly connected to the FHR facing and the earth pressure that has been activated to the gravel bags is transferred to the FHR facing. As illustrated in Fig. 11b, high facing/reinforcement connection force results in high earth pressure that is similar to, or larger than, the active earth pressure that would develop in the unreinforced backfill retained by a conventional cantilever RW. Therefore, high confining pressure develops in the active zone, which results in high stiffness and strength of the backfill, therefore much better performance than walls having flexible facing (Fig. 11a). That is, a substantial reduction of earth pressure is <u>not</u> the target of the reinforced soil RW technology.

Short reinforcement: Fig. 13a shows the active failure mechanism of unreinforced backfill retained by a conventional cantilever RW. With reinforced soil RWs, the active and transitional zones in the unreinforced backfill are stabilized with reinforcement together with facing. When the facing is not FHR (such as wrapped-around, modular block and discrete panel), to restrain the shear deformation, overturning and lateral translation of the reinforced zone by making small enough the unreinforced transitional zone, usually all the reinforcement layers are truncated to an equal length that is large enough (typically $L \geq 0.7H$) (Fig. 13b). When all the reinforcement layers are short (Fig. 13c), the unreinforced transitional zone becomes large while the reinforced zone becomes narrow. In this case, when the facing is not FHR (i.e., deformable), the shear deformation, overturning and lateral translation of the reinforced zone become large and the facing may buckle at low elevations in particular when loaded on the crest (as shown in Fig. 8b) or by seismic loading. When the facing is FHR, the shear deformation, overturning and lateral translation of the reinforced zone become smaller and the facing does not buckle (Fig. 13d). However, when subjected to severe seismic loads, the shear deformation, overturning and lateral translation of the reinforced zone may become large. Tatsuoka et al. (2014b) reported an actual case of this wall deformation mode experienced during the 1995 Kobe Earthquake and its numerical analysis. Koseki et al. (2008) reported shaking table model tests simulating this case. As shown in Fig. 13e, by extending several reinforcement layers to the plane at repose when the backfill is unreinforced, the unreinforced transitional zone disappears. Then, also taking advantage of a high pull-out capacity of planar reinforcement (i.e. geogrid), several reinforcement layers at low elevations can be made relatively short while maintaining a sufficiently high wall stability (Fig. 13e). The current design code specifies that the allowable minimum length of the basic reinforcement is: 1) 35 % of wall height; or 2) 1.5 m; or 3) the length required for a sufficiently high stability against shear deformation, over-turning and lateral translation, whichever is the larger value.

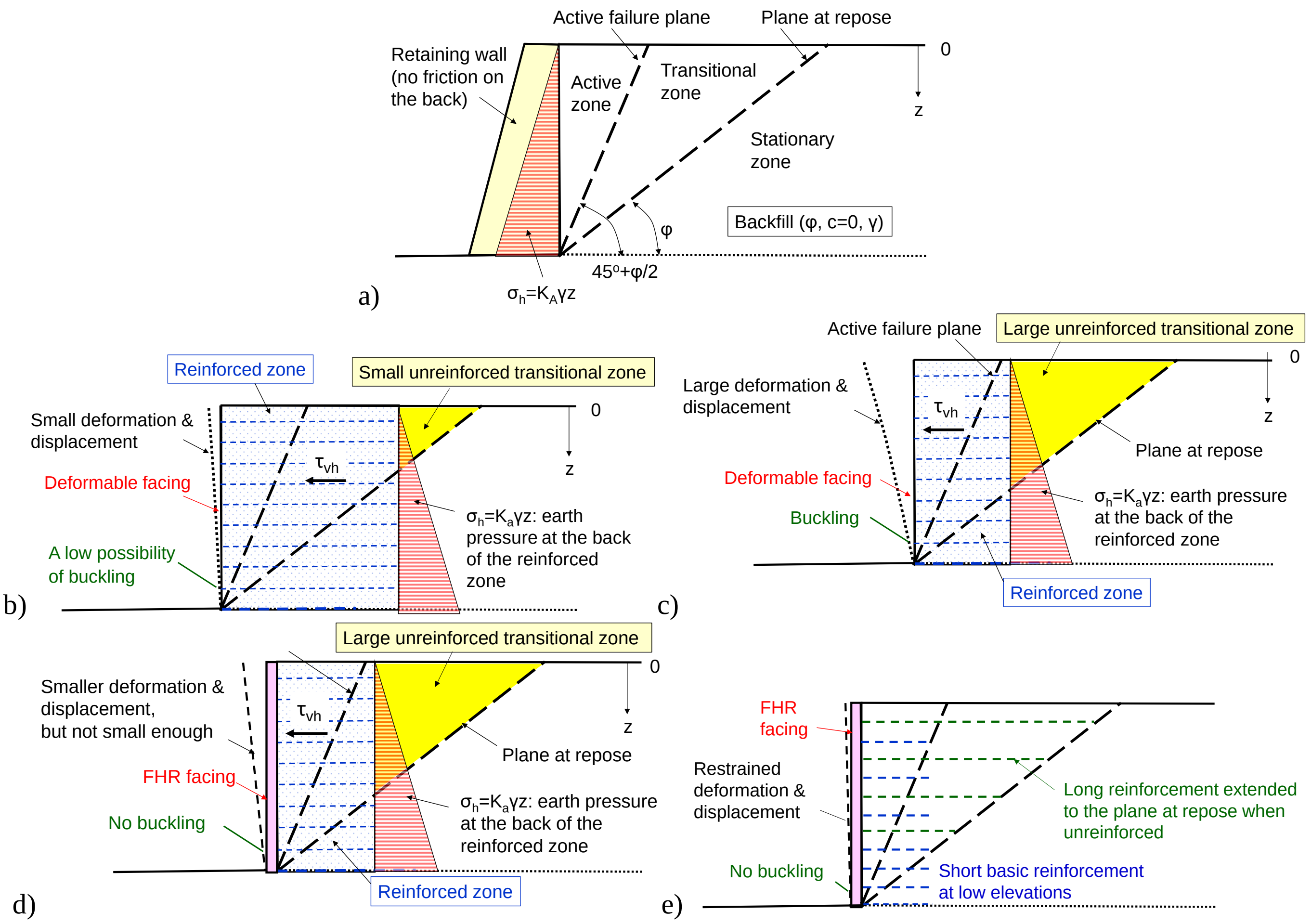


Figure 13. Active failure mechanism: a) conventional cantilever RW; and reinforced soil RWs with: b) flexible facing & long reinforcement; c) flexible facing & short reinforcement; d) FHR facing & short reinforcement; and d)FHR facing & several long reinforcement layers and short reinforcement layers at low elevations.

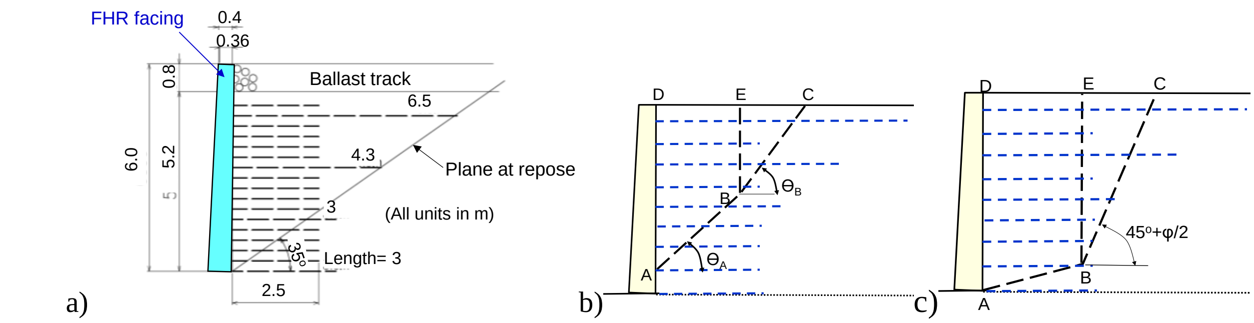


Figure 14. a) Typical GRS RW with FHR designed following the current design code; and two-wedge stability analysis: b) general; and c) most likely critical failure planes when the facing is FHR.

Fig. 14a shows a typical GRS RW with FHR facing designed following the current code. The global stability of GRS RW with FHR facing is analyzed by the two-wedge limit equilibrium-based stability analysis (Fig. 14b). The critical failure mechanism that provides the minimum ratio of the total tensile force of reinforcement available to the tensile force of reinforcement required to maintain the global force equilibrium is sought by changing the locations of point A along the back of facing AD and point B at any location inside the backfill and changing the angles θ_A and θ_B. When the facing is FHR, point A is located at the facing base (Fig. 14c), which substantially increases the wall stability, in particular against large loads applied near the wall face on the crest, compared with the case when the facing is not FHR (as discussed below). The stability analysis for global lateral translation, over-turning and shear deformation under severe seismic loading conditions taking into account strain-softening of the backfill from the peak shear strength toward residual shear strength is explained in Tatsuoka et al. (1998, 2010a, 2014b) and Koseki et al. (2008, 2009, 2012).

Fig. 15a shows a conventional cantilever RC RW constructed on a gentle slope of an existing embankment used typically for railway or road in a congested urban area to create new space on the embankment crest. An anchored sheet pile is often used to keep small the deformation of the embankment caused by relatively large excavation to construct the RW. Besides, a pile foundation is usually used for the RW. These arrangements make this construction very costly. Fig. 15b shows the construction of a reinforced soil RW having relatively long reinforcement, typically L ≥ 0.7H for GRS RWs with modular block facing. L/H is even larger when the reinforcement is metallic strips, because the pull-out capacity is usually lower than the tensile rupture capacity and the anchorage length required to maintain global wall stability is much longer than the one of planar polymeric reinforcement (e.g., geogrid). In this case, although a pile foundation may is unnecessary, an anchored sheet pile may be necessary due to large slope excavation required to accommodate long reinforcement. On the other hand, Fig. 15c shows the construction of a GRS RW with FHR facing having relatively short geogrid layers at low elevations, such as the one shown in Fig. 14a. In this case, slope excavation becomes small, therefore an anchored sheet pile becomes unnecessary in usual cases, which results in a large cost reduction.

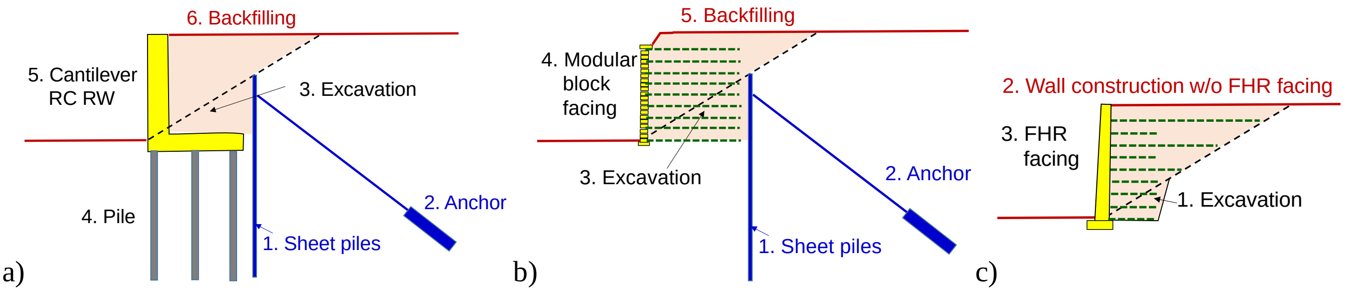
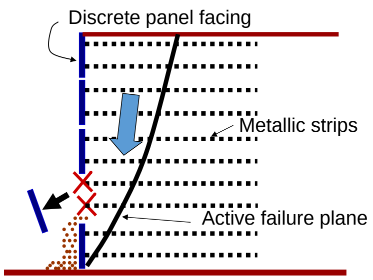


Figure 15. Construction of RW on slope: a) conventional cantilever RC RW; b) reinforced soil RW with relatively long reinforcement; and c) GRS RW having FHR facing with several long geogrid layers at high elevations and short geogrid layers at low elevations (Tatsuoka et al., 1997b). Note: Numbers refer to construction stage.

High structural integrity: With discrete panel facing, local failure, if it takes place, may develop to the overall wall failure. Lee et al. (1994) reported the failure of a set of reinforced soil RWs constructed on a natural slope in which metallic strips at low elevations were relatively short to avoid large slope excavation. Some of them were pull-out which resulted in the collapse of the whole wall (Fig. 16). A similar failure took place in Japan. Although the walls did not fully collapsed, standard discrete panel facings of several Terre Armée walls deformed largely with panels separated from each other while losing a wall face alignment during the 1995 Kobe earthquake (Tatsuoka et al., 1997a), the 2004 Niigata-ken Chuetsu Earthquake (Kitamura et al., 2005), the 1999 Kocaeli Earthquake in Turkey (Kempton et al., 2008) and the 2011 Great East Japan Earthquake (Kuwano et al., 2014). This type of failure does not take place with GRS RWs having FHR facing due to a high structural integrity of facing.



Failure mechanism

1. Local lateral outward movement of the facing due to local pull-out of the reinforcement
2. Downward displacement of the active zone
3. Rupture of the connection due to relative vertical displacement between the facing and the reinforcement
4. Local dislocation of a single panel (or multiple panels)
5. Failure of the entire facing system due to the lose of equilibrium in the facing associated with spilling out of the backfill.

Figure 16. A typical failure mode of steel strip-reinforced soil RW having a discrete panel facing.

When the facing is deformable (e.g., wrapped-around or discrete panel or modular block) is subjected to load on the top of the facing and/or the crest of the backfill immediately back of the facing, such local failure passing the wall face at an intermediate elevation as shown in Fig. 17a may take place (Tatsuoka et al., 1989). On the other hand, when the facing is FHR connected to reinforcement layers, the failure plane starts from the facing base (Fig. 17b) and all the reinforcement layers can resist the applied load, which significantly increases the wall stability. With GRS RW with FHR facing (Fig. 1), one unit of FHR facing is basically 20 m-long separated by vertical construction joins. The whole of each unit can effectively resist concentrated loads applied to the facing unit. Fully taking advantage of these features of FHR facing and the staged wall construction (Fig. 1a), GRS Bridge Abutment and GRS Integral Bridge with FHR facings directly supporting a continuous girder were developed, as explained later

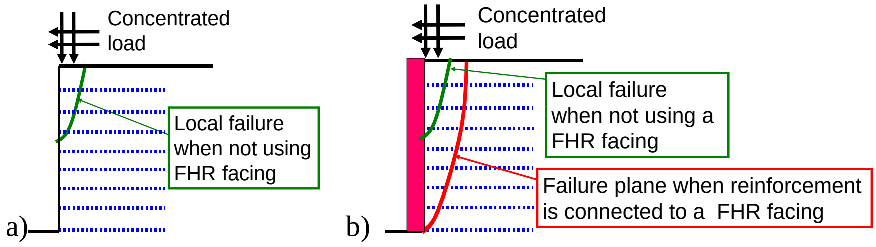


Figure 17. Failure of reinforced soil RW when subjected to concentrated load near the wall face at the crest of wall: a) deformable facing; and b) FHR facing connected to reinforcement layers.

In congested urban areas, elevated structures for railways and roads are required to be compact occupying only narrow space. Then, like RC viaducts, the RC facing of the conventional type RW supports other facilities, such as noise barrier, crash barrier, electric power supply and others (Fig. 18a). GRS RW with FHR facing (Fig. 1) maintains these features in a more cost-effectively way (Fig. 18b). On the other hand, when the facing is not FHR, such as discrete panel or modular block facing, some width from the wall face on the crest is not reliable enough to arrange a railway track or a road, while a foundation structure other than the facing becomes necessary to support other facilities (Fig. 18c). Besides, recent HSRs in Japan, Shinkansens, use continuous RC slab tracks in place of ballasted tracks for a lower life cycle cost resulting from a large reduction in the maintenance cost despite relatively high construction cost. Continuous RC slab tracks are not constructed on embankment and the backfill retained by a conventional type RW, as the settlement may exceed a very small value allowed with continuous RC slabs. On the other hand, the continuous RC slab track for HSRs are constructed on the GRS RWs with FHR facing now as the standard practice. This practice has become feasible by the use of selected backfill soil, its good compaction (enhanced by the staged construction, Fig. 1a) and a high stability of this type of GRS RW with FHR facing. No problematic case has been reported.

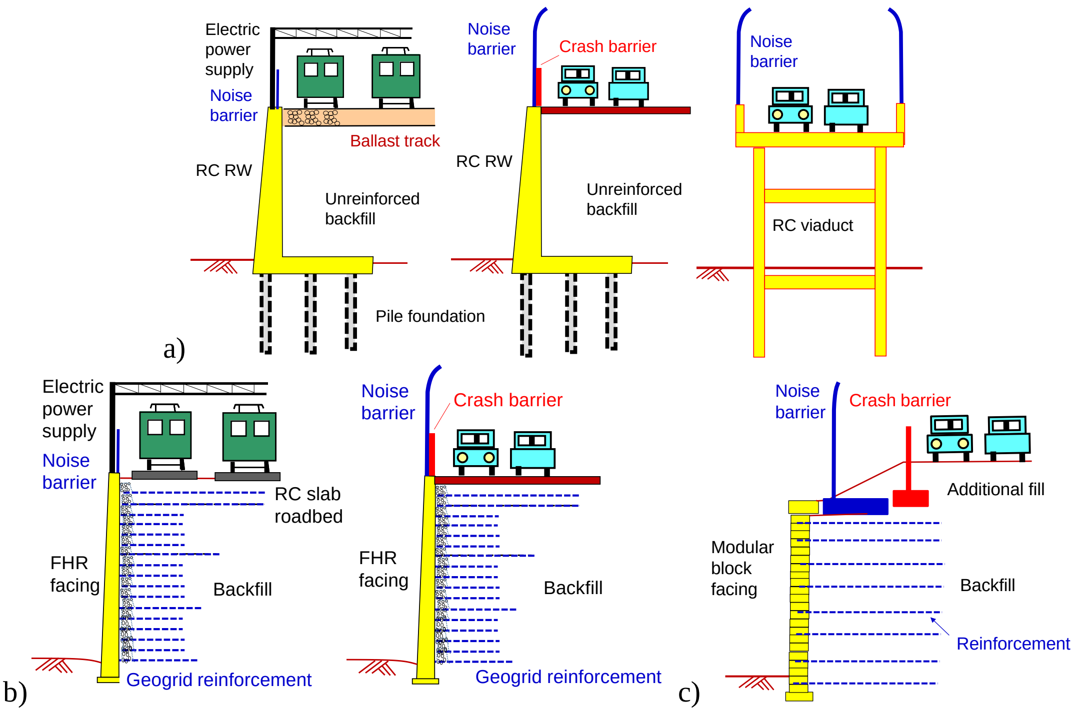


Figure 18. Typical elevated structures: a) conventional (piled cantilever RC RW & RC viaduct); b) GRS RW with FHR facing; and c) reinforced soil RW with modular block facing.

The construction cost for GRS-RW with FHR facing (Fig. 1) is basically lower than conventional cantilever RC RW, in particular when a pile foundation is used with conventional type RW. Besides, when based on the life cycle cost evaluated by taking into account these features described above as well as a high stability against severe earthquakes, heavy rains and floods, GRS RW with FHR facing is better than not only conventional type cantilever RC RW but also other types of reinforced soil RWs not having staged constructed FHR facing. For these reasons, GRS RW with FHR facing has been adopted nearly fully in place of conventional type RWs at many places as summarized in Fig. 2.

3 BRIDGES

3.1 *Conventional type simple girder bridge*

Both ends of the girder of the conventional simple girder bridge are typically supported by a pair of abutments via a pair of bearings (i.e., a fixed pin and a movable roller) while the abutments comprise unreinforced approach fills retained by piled RC RWs. Due to these structural features, the following several serious problems are often encountered (Fig. 19a). 1) As the abutment is a cantilever structure supported at its base, with an increase in the abutment height and with a decrease in the bearing capacity of the supporting ground, the abutment becomes larger and more massive while a more costly pile foundation becomes necessary to keep sufficiently small the displacement caused by earth pressure and ground movements associated with the construction of approach fills and by external disturbances (including seismic loads and scouring). 2) Installation of the girder bearings, together with additional arrangements to prevent the dislodging of the girder by seismic loads, and their long-term maintenance to prevent their corrosion and other detrimental effects are both rather costly. 3) The seismic stability of the girder at a movable roller bearing and the unreinforced backfill is rather low. 4) A large bump may develop immediately back of the abutment gradually by self-weight and long-term traffic loads and suddenly by seismic loads, enhanced by displacements of the abutment and deformation of the supporting ground.

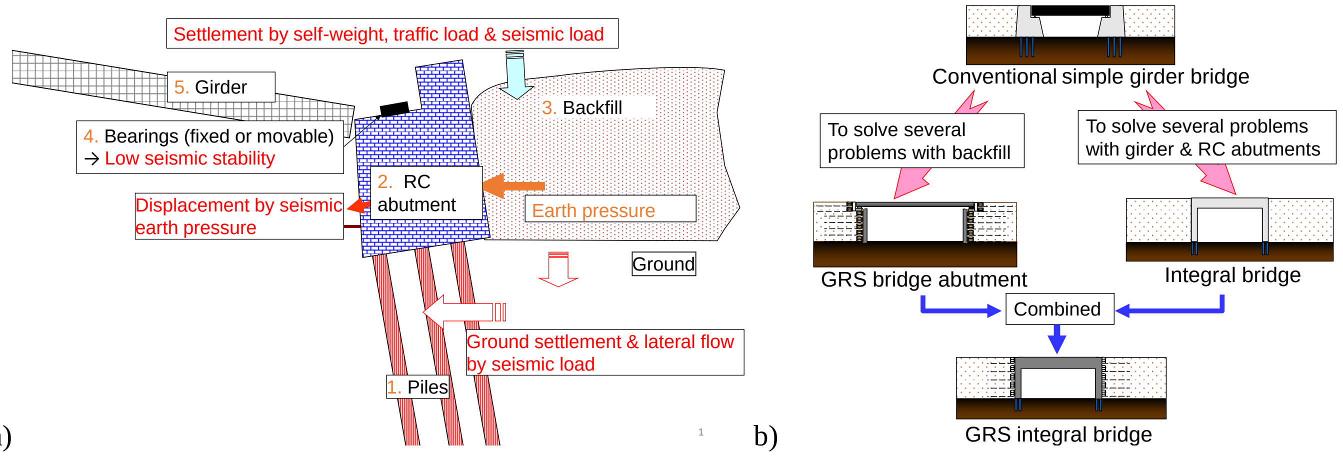


Figure 19. a) Typical serious problems with conventional type simple girder bridge; and b) development towards GRS Integral Bridge (Tatsuoka et al., 2009, 2016).

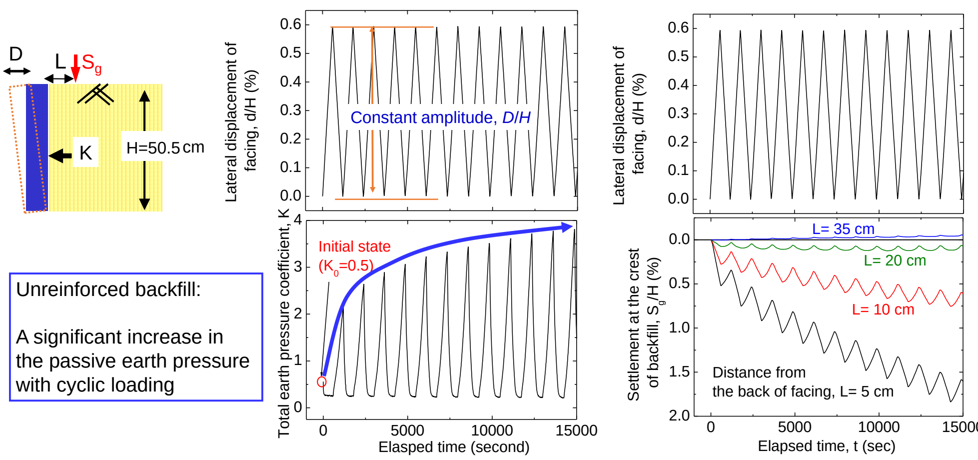


Figure 20. Settlement in dense air-dried Toyoura sand (D_r= 90 %) and an increase in the passive earth pressure on the back of the facing by cyclic lateral displacements at the top of the FHR facing in a small model test (Tatsuoka et al., 2009, 2010b).

3.2 *Integral bridge*

To alleviate these problems with the conventional simple girder bridges (Fig. 19a), Integral Bridge was developed in the UK and the North America (Fig. 19b). Both ends of a continuous girder is structurally integrated to the top of a pair of RC abutments, followed by the construction of approach fills. A great number of Integral Bridge have been constructed, typically as bridges overpassing a highway, in the UK and the North America. However, several problems due to that the approach fill is unreinforced soil remain unsolved, while the following new problem arises. Fig. 20 shows results from laboratory model tests in which the top of FHR facing was subjected to small cyclic lateral displacements simulating those by annual thermal deformation of the girder of Integral Bridge. An active wedge developed in the backfill of air-dried Toyoura sand, which resulted in the settlement in the backfill developing a bump immediately back of the FHR facing. At the same time, the passive earth pressure at the back of the facing increased significantly. Integral Bridge is required to be designed against this large earth pressure. These phenomena are due to the dual ratcheting mechanism in the active and passive failures of the backfill (Tatsuoka et al., 2009, 2010b, 2012).

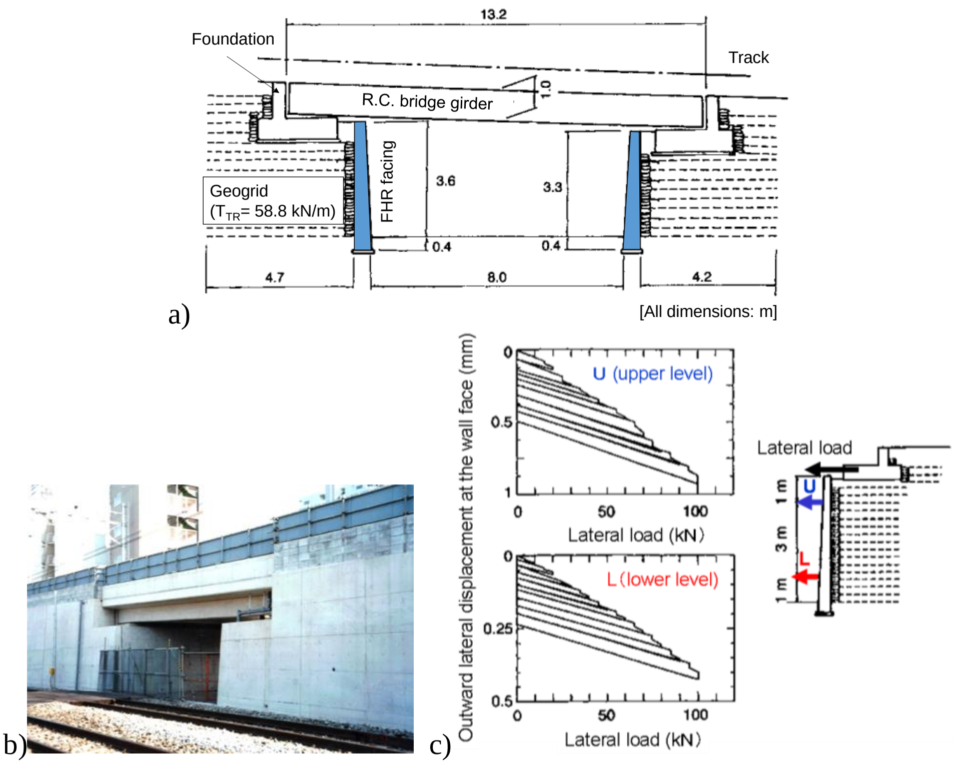
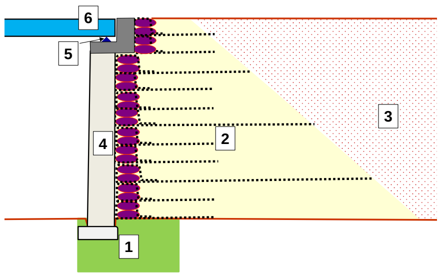


Figure 21. GRS Bridge Abutments (first generation) near Sakuradai station, Seibu Ikebukuro line, Tokyo; a) elevation of the bridge; b) completed bridge; and c) lateral loading test of a GRS Bridge Abutment (other than those shown in Figs. a & b), performed 24 June 1993.

3.3 *GRS Bridge Abutment*

GRS Bridge Abutment was developed to alleviate the problems due to that the approach fill is not reinforced (Fig. 19b, Tatsuoka et al., 1997b, 2005). Fig. 21 shows the one of the GRS Bridge Abutments of the first generation, constructed for a busy urban railway in Tokyo in 1993. Both ends of a simple girder were placed on a pair of bearings (a movable roller and a fixed pin) arranged on a pair of foundation placed immediately back of the FHR facing on the crest of the reinforced backfill. These GRS Bridge Abutments were constructed directly on a deposit of intact volcanic ash clay (Kanto loam) without using a pile foundation. The bridge was designed in such that the FHR facing effectively resist the lateral seismic loads from the girder. This capacity was confirmed by applying a lateral outward load up to 98 kN to the foundation for the girder of another GRS bridge abutment at the site (Fig. 21c). The maximum lateral movement at the top of the facing was only 0.9 mm. Besides, the displacement at level L was approximately one half of that at level U. These trends of behaviour shows that the FHR facing laterally supported by many reinforcement layers connected to the back of the FHR facing very effectively resisted the lateral load applied to the crest of the wall. A number of GRS RWs with modular block facing were constructed as abutments to support a simple girder in the USA and their good performance is reported (e.g., Zornberg et al., 2001, Abu-Hejleh et al., 2002, Lee & Wu, 2004). The girder is placed on a foundation arranged on the crest of reinforced backfill, in a similar way as the bridge shown in Fig. 21a. GRS RW having FHR (first generation) is more stable due to the use of FHR facing. However, when the girder becomes long, it is difficult to keep very small the settlement of the girder foundations and to ensure a high seismic stability of the foundation supporting the girder via. a fixed pin bearing and the girder at a movable roller bearing.

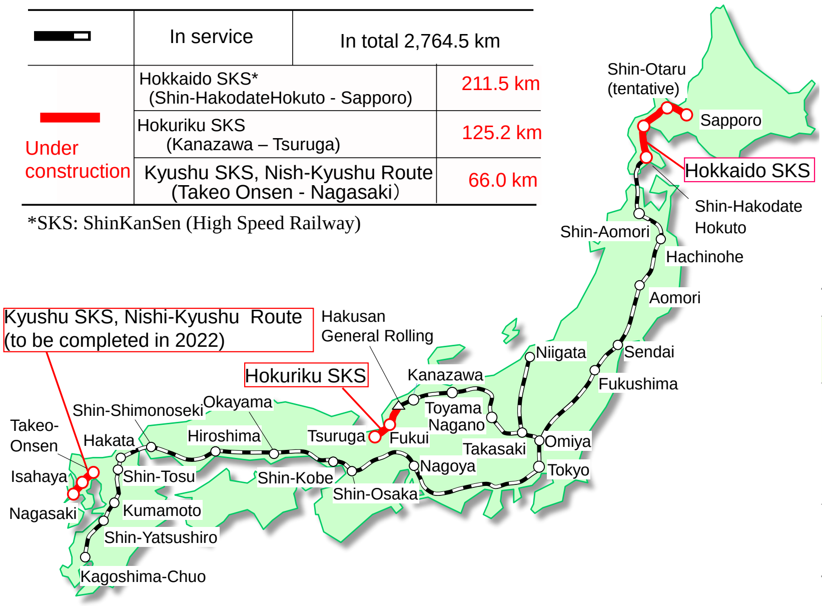


Construction sequence:
1. Ground improvement (when necessary)
2. Approach block of compacted lightly-cement-mixed gravelly soil reinforced with geogrid layers connected to the facing
3. Unbound fill
4. FHR facing
5. Fixed bearing
6. Simple girder

a)

b)

Figure 22. GRS Bridge Abutment (second generation): a) structure and construction procedure; and b) views during construction on a slope at Sugamuta viaduct, Nishi-Kyushu Route, Kyushu Shin-kan-sen (High Speed Railway) (Soga et al., 2018).



		Saga Pref.	Nagasaki Pref.	Total
①	GRS RW with FHR facing	1,009 m	3,979 m	4,988 m
②	GRS tunnel entrance protection	15	40	55*
③	GRS bridge abutment	34	46	80
④	GRS integral bridge	2	5	7

* 89 % of the total tunnel entrance protections (62)

Figure 23 (left). Network of High Speed Railway in Japan (Shin-kan-sen) (Soga et al., 2018).
Table 2 (right). GRS structures for Nishi-Kyushu Route, Kyushu Shin-kan-sen (Soga et al., 2018).

Then, GRS Bridge Abutment (the second generation) was developed (Fig. 22a). To alleviate the drawbacks due to the use of bearing foundations with the first generation described above, one end of a simple girder is placed on a fixed pin bearing arranged on the top of the FHR facing of a GRS RW while the other end of the girder is placed on a movable roller bearing arranged on the top of the adjacent RC pier. To ensure a high seismic stability and for essentially zero bump immediately back of the FHR facing, the approach block (zone 2 in Fig. 22a) is well-compacted lightly cement-mixed well-graded gravelly soil reinforced with geogrid layers connected to the back of the FHR facing. For a continuous increase from zero in the thickness of unbound fill behind (zone 3) and for a higher stability when subjected to large lateral seismic inertial loads of the girder at the crest, the shape of zone 2 is trapezoidal with the base wider than the crest. Fig. 22b shows the standard construction procedure: i.e., 1) natural slope is bench-cut; 2) & 3) an approach block is constructed by compacting lightly cement-mixed gravelly soil in a lift of 15 cm to dry density at least 95 % of the maximum dry density at a water content equal to the optimum by Modified Proctor (Tatsuoka et al., 2017); 4) geogrid layers are arranged in the approach block; and 5) & 6) a GRS abutment is completed by constructing FHR facings on the front face and the two lateral side faces of the approach block. The first one was constructed in 2002 at Takada along Kagoshima Route of Kysuhu Shin-kan-sen (HSR) (Tatsuoka et al., 2005, 2014a, 2016, Tatsuoka & Watanabe, 2015). Fig. 23 shows the current Shin-kan-sen network. Since the construction of the first one, in total thirty six have been constructed until today. For Nishi-Kyushu Route of Kyushu Shin-kan-sen now under construction, in total eighty GRS Bridge Abutments were adopted nearly fully in place of conventional type bridge abutments (Table 2), many at tunnel entrances. The tallest one is 13.5 m-high.

3.4 *GRS Integral Bridge*

Structural integration of the girder to the FHR facing: GRS Bridge Abutment is not free from several problems due to the use of bearing when its pair support both ends of a simple girder using a pair of bearing (a movable roller and a fixed pin). GRS Integral Bridge (Fig. 3) was then developed to alleviate these problems while maintaining the advantages of the use of the FHR facing of GRS RW as an abutment. That is, GRS Integral Bridge is a combination of Integral Bridge and GRS Bridge Abutment alleviating their drawbacks while maintaining their advantageous features (Fig. 19b). As shown in Fig. 3, GRS Integral Bridge comprises a continuous girder with both ends structurally integrated to the top of a pair of FHR facings (without using bearings) that are firmly connected to geogrid layers reinforcing the backfill. GRS Integral Bridge is constructed in stages as follows: First, a pair of GRS RWs are constructed as shown in Fig. 1a. When the span is long, say longer than 30 m, a central pier may also be constructed to support vertically the center of the girder (Fig. 3c). After sufficient deformation of the backfill and subsoil has taken place, FHR facings as abutments are constructed as shown in Fig. 1a. Finally, a continuous girder is constructed with both ends structurally integrated to the top of a pair of FHR facings. In this way, the connections between the FHR facing and the geogrid layers and between the girder and the FHR facings become free from damage due to the deformation of the backfill and subsoil associated with the construction of GRS RW. Eventually, GRS Integral Bridge alleviates all of the serious drawbacks with conventional simple girder bridges illustrated in Fig. 19a.

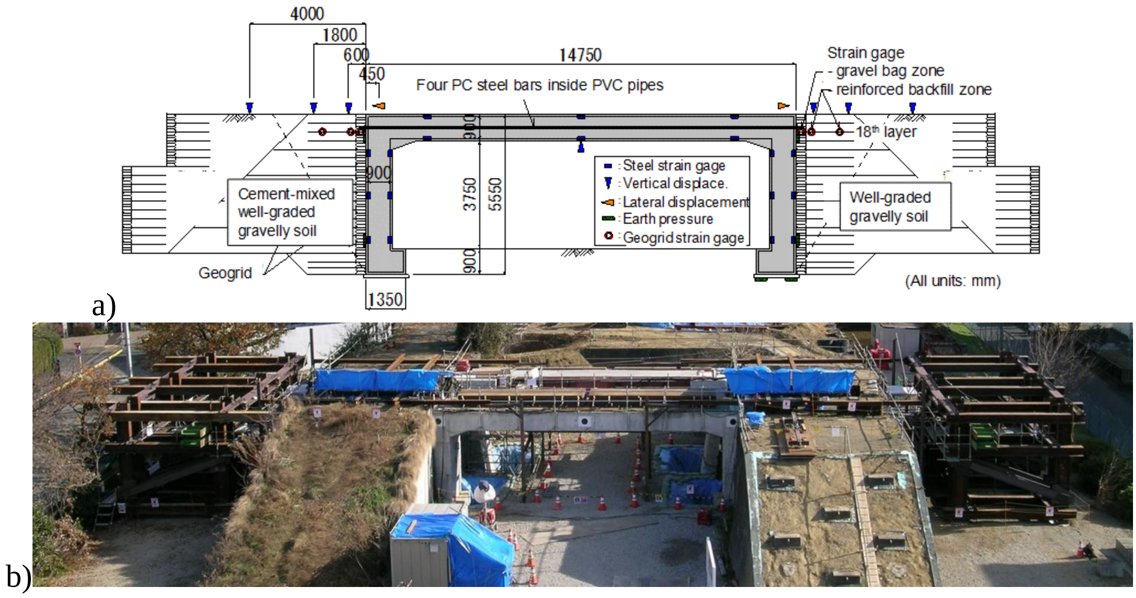


Figure 24. a) Full-scale model of GRS Integral Bridge at Railway Technical Research Institute; and b) view during lateral cyclic loading tests (Koda et al., 2013; Tatsuoka et al., 2016).

Research: A series of static and dynamic loading tests were performed on small models in the laboratory and on full-scale models in the field to establish the design and construction procedures of GRS Integral Bridge. A full-scale model comprising a 14.75 m-long and 3 m-wide girder integrated to a pair of 5.55 m-high abutments was constructed at Railway Technical Research Institute in 2009 (Fig. 24). The results of these studies showed that GRS Integral Bridge is highly cost-effective while highly stable against long-term traffic load as well as against severe earthquakes, heavy rains and floods. Several key findings from the research are presented below. Fig. 25 shows the settlement immediately back of the facing in the backfill of air-dried Toyoura sand caused by cyclic lateral displacements at the top of FHR facing in model tests in the laboratory. The FHR facing was hinged at the bottom; H was the wall height= 50.5 cm; and D was the double amplitude displacement. Three model walls were tested: NR (the backfill was unreinforced); R&NoC (the backfill was reinforced with geogrid layers not connected to the back of the FHR facing); and R&C (the backfill was reinforced with geogrid layers connected to the back of the FHR facing simulating the abutment of GRS Integral Bridge). Only R&C model exhibited nearly zero settlement in the backfill by preventing the development of an active wedge in the backfill.

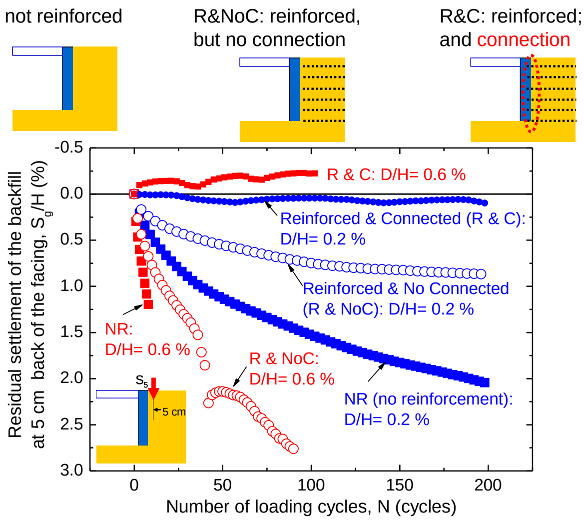


Figure 25. Settlement at 5 cm back of the FHR facing in the backfill of dense air-dried Toyoura sand (D_r= 90 %) by cyclic lateral displacement at the top of FHR facing with a height H= 50.5 cm hinged at the bottom in three model walls (see Fig. 20 for the details of the model tests) (Tatsuoka et al., 2009, 2010b).

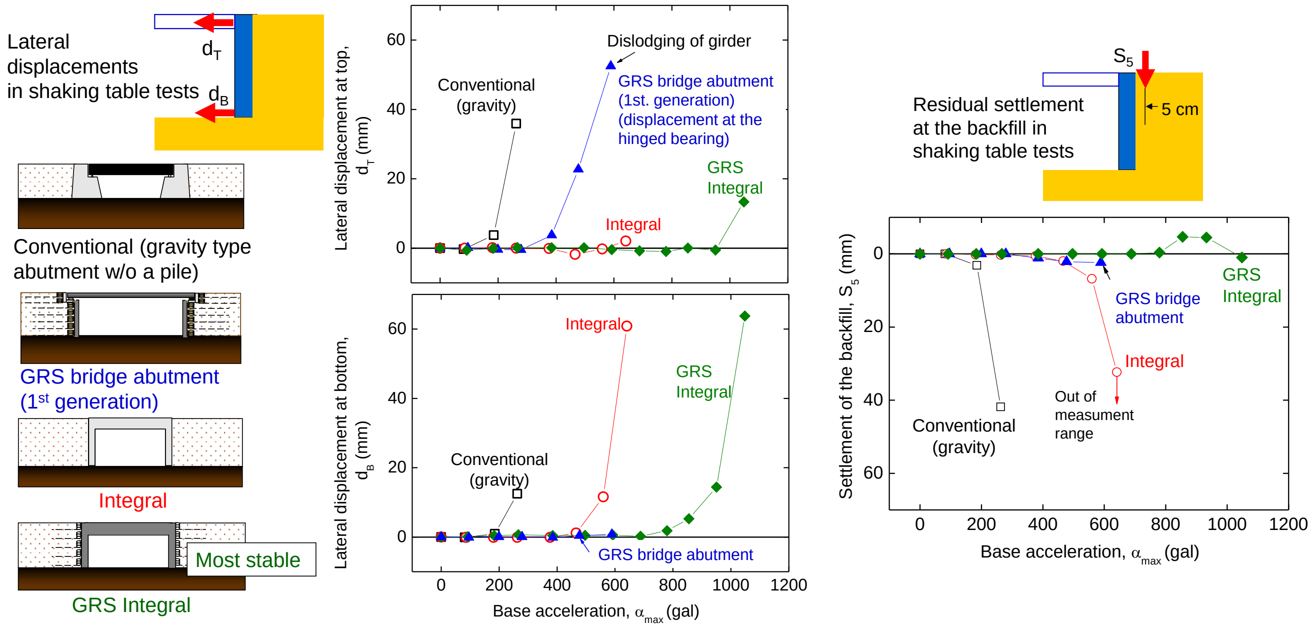


Figure 26. Residual lateral displacements at the top and bottom of the FHR facing and residual settlement at 5 cm back of the facing in shaking table tests of four bridge models (α_{max}: the amplitude of input acceleration at a frequency of 5 Hz) (Tatsuoka et al., 2009).

Fig. 26 shows the residual lateral displacements at the top and bottom of the FHR facing and the residual backfill settlement at 5 cm back of the facing in shaking table tests of four bridge models, where α_{max} is the amplitude of input acceleration at the shaking table. 20 sinusoidal waves at an input frequency f_i= 5 Hz were applied at each stage increasing stepwise α_{max}. Already at a low α_{max} (= 200 gal), the model of conventional simple girder bridge with gravity type abutments not supported by a pile started exhibiting large lateral displacements at both top and bottom of the abutment, which resulted in large settlement in the backfill. The bridge model comprising a pair of GRS Bridge Abutment (1st generation) supporting the two ends of a simple girder via. a pair of fixed pin and movable roller barings on a pair of foundation placed on the crest of the reinforced backfill and Integral Bridge were both more stable. However, they were not as stable as GRS Integral Bridge model. The GRS Integral Bridge model lost its stability due to the rupture at the facing/geogrid connection. Tatsuoka et al. (2018) shows that GRS Integral Bridge becomes more stable by increasing this connection strength. The relevant structure of this connection, which becomes more important with an increase in the bridge span, was studied by performing a full-scale loading tests (Koda et al., 2018). It may be seen from Fig. 26 that, with the GRS Integral Bridge model, the settlement in the backfill was kept very small even after the lateral displacement at the bottom of the facing started largely increasing. This is one of the most advantageous features of GRS Integral Bridge.

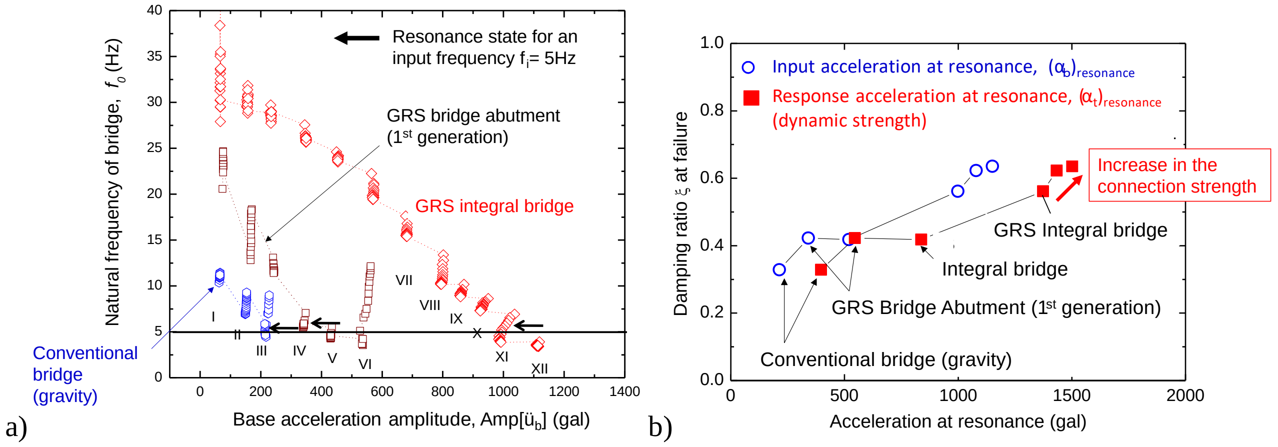


Figure 27. a) Decrease in the natural frequency f_0 with an increase in the input acceleration at a frequency equal to 5 Hz; and b) damping ratio plotted against input acceleration and response acceleration at resonance of different bridge models in the shaking table tests described in Fig. 26 (Munoz et al., 2012).

A very high seismic stability of GRS Integral Bridge compared with other bridge types is due to the following four specific mechanisms resulting from its high structural integrity. Firstly, the initial value of the natural frequency f_0 of GRS Integral Bridge is basically much higher than the predominant frequencies of typical severe seismic motions, which was simulated by sinusoidal waves with an input frequency f_i= 5 Hz in this model test (Fig. 27a). This feature results in a low initial response acceleration. In this model test, the initial f_0 value of the conventional simple girder bridge is much lower, about one third of the value of GRS Integral Bridge, which results in a much higher initial response acceleration. Secondly, GRS Integral Bridge exhibits a low decreasing rate of f_0 with an increase in the input acceleration, α_{max}, which results in a slow approach to the resonance state, where $f_0 = f_i$ (Fig. 27a). This trend is due to smaller structural damage by dynamic loading. The α_{max} value when GRS Integral Bridge reaches the resonance state for f_i= 5 Hz exceeds 1,000 gal, which is substantially higher than the value when the conventional type bridge reaches the resonant state, equal to about 200 gal. Thirdly, GRS Integral Bridge exhibits a large damping ratio at failure that takes place immediately after having reached the resonant state (Fig. 27b), which results in a smaller response acceleration ratio at failure. This trend is due to a high dissipation rate of the dynamic energy of the girder and facings toward the backfill and subsoil. Lastly, GRS Integral Bridge has a large strength against response acceleration. In particular, the stability of the continuous girder structurally integrated to the FHR facings is substantially higher than a simple girder of conventional type bridge.

Construction: Table 3 lists the GRS Integral Bridges that have been constructed, or will be constructed, for a new HSR under construction, Nishi-Nagasaki Route, Kyushu Shin-kan-sen. Fig. 28 shows the structure and construction of the one of them. In the same way as GRS Bridge Abutment, the approach block comprises well compacted lightly cement-mixed well-graded gravelly soil that is reinforced with geogrid layers firmly connected to the FHR facing to ensure a very high seismic stability of the approach block and for essentially zero bump immediately behind the FHR facing. With ordinary roads, on the other hand, this cement-mixing may not be necessary if constructed using well compacted unbound soil of high quality. The two approach blocks for the full-scale model (Fig. 24) were either geogrid-reinforced well-compacted cement-mixed well-graded gravelly soil or geogrid-reinforced well-compacted unbound well-graded gravelly soil. It was confirmed by long-term observation of their performance and full-scale loading tests simulating annual thermal displacements and strong seismic loads (Fig. 24b) that an appropriately designed and constructed approach block comprising geogrid-reinforced unbound well-graded gravelly soil also performs satisfactorily. So, when the allowable settlement is not as strict as the one supporting a continuous RC slab for a HSR, the approach block for GRS Integral Bridge could be constructed using geogrid-reinforced well-compacted unbound soil.

Table 3. GRS Integral Bridges for Nishi-Kyushu Line, Kyushu Shin-kan-sen (Soga et al., 2018).

Location	Bridge name	Span	Girder structure
Between Takeo-Onsen & Ureshino-Onsen stations	Momoki No.1 Overbridge	12.00 m	RC slab
	Tsubakihara Overbridge	10.00 m	
Between Ureshino-Onsen & Shin-Omura stations.	Onibashi No.1 Overbridge (for a line to Omura General Rolling)	10.10 m	
Between Isahaya & Nagasaki stations	Genshu Overbridge*	30.00 m	Four PPC T-shaped main girders
	Genshu Bridge	20.00 m	RC slab
	Kaizu Bridge	15.00 m	
	Funaishi No.4 Overbridge	15.00 m	

* see Fig. 28

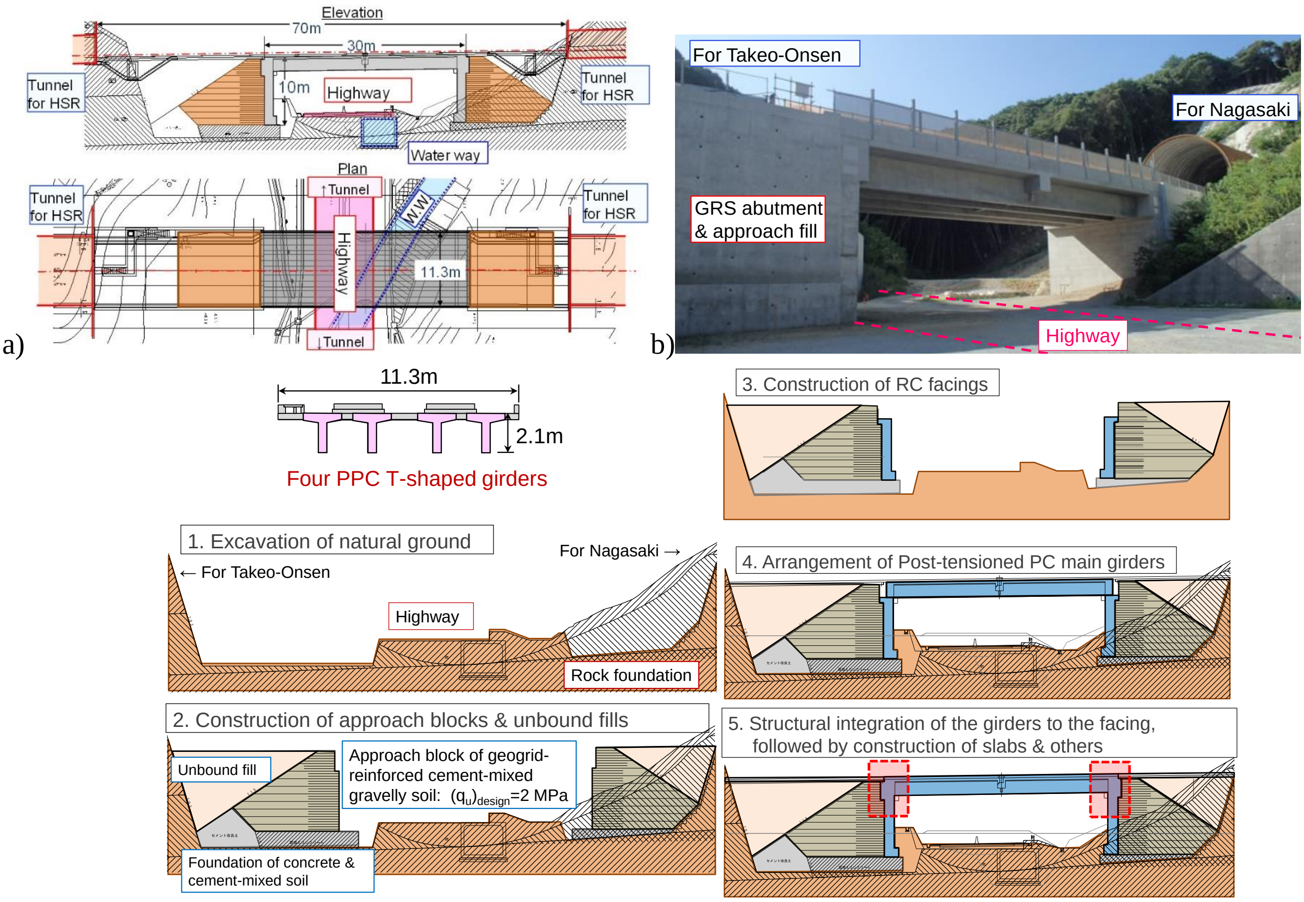


Figure 28. GRS Integral Bridge at Genshu, Nishi-Nagasaki Route, Kyushu Shin-kan-sen: a) structure; b) completed bridge; c) cross-section and construction procedure (Soga et al., 2018).

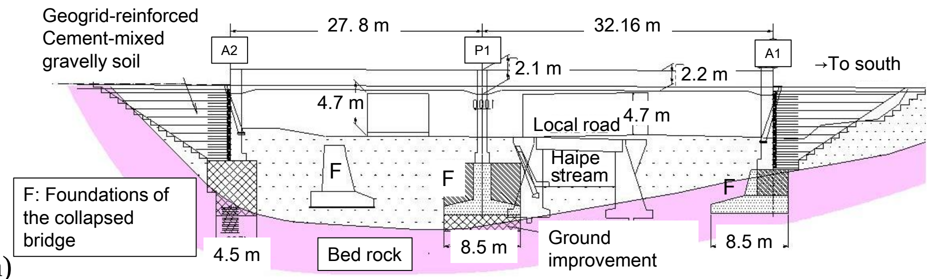
The first GRS integral bridge was constructed in 2013 as an over-road bridge at Kikonai for Hokkaido Shin-kan-sen (see Table 4). Yonezawa et al. (2014) reported that the construction cost of this bridge was estimated to be about one half of that of the equivalent conventional type simple girder bridge. More than 340 bridges located in coastal areas, mostly simple girder bridges, collapsed by the great tsunami of the 2011 Great East Japan Earthquake. Kawabe et al. (2015) reported the results of model tests performed to evaluate the stability of GRS Integral Bridge against a deep overtopping tsunami flow compared with the stability of the conventional simple girder bridge. It was confirmed that, due to a high structural integrity of the three major components, the girder, the FHR facings and the approach blocks, GRS Integral Bridge exhibits a substantially higher stability than simple girder bridges. Three conventional simple girder railway bridges that were fully washed away by a deep overtopping tsunami flow (4.4 - 8.2 m-deep) of the 2011 Great East Japan Earthquake were reconstructed to GRS Integral Bridges (Table 4, Figs. 29 & 30).

These bridges were designed to be stable against the overtopping tsunami flow by which the previous bridges were washed away. It was confirmed that it is not feasible to cost-effectively construct other types of bridges that can withstand such a deep over-topping tsunami flow.

Table 4. Major GRS Integral Bridges other than those listed in Table 3.

Railway	Bridge name	Span	Girder structure	Note
Hokkaido SKS, between Shin-Aomori & Shin-HakodateHokuto stations	Tyugakkousen Overbridge (at Kikonai)	12.00m	RC slab	First prototype
Sanriku Railway (local & ordinary) between Shimanokoshi & Tanohata stations	Matsumaegawa Bridge	27.40m 13.7m+13.7m	RC slab	Continuous girder with two spans
	Koikorobesawa Bridge	39.86m 19.93m+19.93m	RC slab	
	Haipesawa Bridge	60.00m 32.16m+27.84m	SRC* through girder	

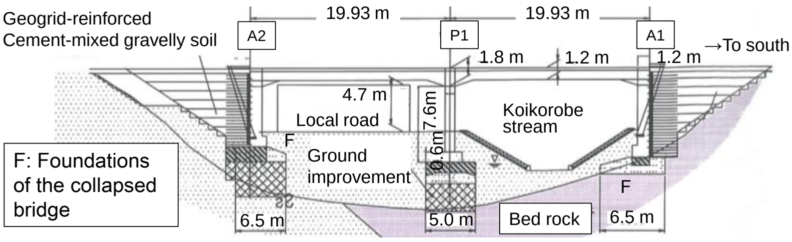
* Steel-framed steel-reinforced concrete



a)

b)

Figure 29. GRS Integral Bridge at Haipe, Sanriku Railway (the continuous girder is vertically supported by a central pier): a) structure; and b) April 2014 (Tatsuoka et al., 2015, 2016; Tatsuoka & Watanabe, 2015).



a)

b)

Figure 30. GRS Integral Bridge at Koikorobe, Sanriku Railway (the continuous girder is vertically supported by a central pier): a) structure; and b) April 2014 (Tatsuoka et al., 2015, 2016; Tatsuoka & Watanabe, 2015).

4 SUMMARY

The history for about 35 years of the development and construction of geosynthetic-reinforced soil (GRS) structures that started with retaining walls (RWs) and has developed to GRS Integral Bridge is briefly described in this paper. The following three are the breakthroughs that changed and reversed our design and construction concept of GRS structures:

1) Full-Height Rigid (FHR) facing, which changed low earth pressure to high earth pressure in wall design and changed the facing from a secondary component to a primary component.
2) Staged construction reversing the construction sequence from the backfill last to the backfill first.
3) Structural integration of the girder to the FHR facings, which changed the bridge structure from a statically determinate but unstable one to a statically in-determinate but stable one.

FHR facing connected to the reinforcement layers ensures not only a high durability of wall face but also a high wall stability by: developing high earth pressure on the back of the facing, which results in high confining pressure thereby high stiffness and strength of the backfill; and making monolithic the wall behaviour by preventing the development of local failure towards the global failure. Besides, unlike conventional type cantilever RWs, the FHR facing behaves as a continuous beam laterally supported by many geogrid layers. Therefore, the shear forces and moments activated in the FHR facing and the overturning moment and lateral thrust forces activated at the facing base do not become as large as those in the cantilever facing, resulting in a much lighter facing structure and making unnecessary the use of a pile foundation under ordinary conditions.

In the staged construction, a GRW RW is first constructed with help of gravel bags (or its equivalent) placed at the shoulder of each soil layer and, after the deformation of the backfill and subsoil has taken place sufficiently, FHR facing is constructed by casting-in-place concrete on the temporary wall face. In this way, the connections between the FHR facing and the reinforcement (and between the girder and the FHR facings with GRS Integral Bridge) are not damaged by their relative displacements.

GRS Integral Bridge was developed taking advantage of these features of staged constructed FHR facing. As GRS Integral Bridge is an integrated statically indeterminate structure, its design is more complicated than a conventional simple girder bridge, which is basically a statically determinate structure. However, this is not a serious issue in today's computerized design. Rather, benefits by structural integration of the three components, the girder, the FHR facings and the approach fills, are more significant in increasing the stability and decreasing the residual deformation resulting in a much higher life cycle-cost effectiveness. As a result, GRS Integral Bridge has been accepted by many engineers (in particular railway engineers). The design and construction codes and manuals have been published.

GRS Box Culvert integrated to GRS RWs on both sides and GRS Tunnel Exit Protection were also developed. It is believed that the development of these GRS structures is one of recent significant technological advances of the Geosynthetic Engineering discipline exploring new applications of geosynthetics.

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