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Geosynthetic-Reinforced Soil Structures for Railways: Twenty Five Year Experiences in Japan

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ABSTRACT: Geosynthetic-reinforced soil retaining walls (GRS-RWs) have been constructed for a total length more than 135 km mainly for railways, including high-speed train lines. A full-height rigid (FHR) facing is constructed, firmly connected to the reinforcement layers, after a full-height wrapped-around GRS wall has been constructed and the major residual deformation of the backfill and supporting ground has taken place. A number of this type GRS RWs performed very well during the 1995 Kobe Earthquake and the 2011 Great East Japan Earthquake. The seismic design code has been revised to be prepared for such level seismic loads as experienced during the 1995 Kobe EQ. A number of conventional type RWs and embankments that collapsed during these and other earthquakes, heavy rains, floods and storm wave actions were reconstructed to this type GRS RWs. A couple of new bridge types comprising GRS structures have been developed. The latest version is GRS integral bridge, which comprises a continuous girder integrated to the top of the facings of a pair of GRS RWs without using bearings. The first prototype was constructed for a high-speed train in 2012 and three others were also constructed to restore bridges that fully collapsed by great tsunami during the 2011 Great East Japan EQ.

1. INTRODUCTION

The design and construction policy of soil structures for Japanese railways has been significantly improved over the last 25 years, in particular in the following five terms. Firstly, the standard type of retaining wall (RW) has fully changed from the conventional cantilever RW to the geosynthetic-reinforced soil (GRS) RW having staged constructed full-height rigid (FHR) facing with a strong connection between the facing and the reinforcement layers (Figure 1: Tatsuoka et al., 1997a).

Secondly, 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 of this type. Thirdly, a couple of new bridge systems using the GRS technology have been developed and are now constructed in place of the conventional type bridges. With GRS bridge abutments, a girder is placed via bearings on the top of the facing of GRS RW (Aoki et al., 2005; Tatsuoka et al., 2005). About 50 GRS abutments of this type have been constructed. The latest bridge type, called the GRS integral bridge, comprises a continuous girder that is structurally integrated to the facings of a pair of GRS RWs (Tatsuoka et al., 2008a & b, 2009). Fourthly, these GRS soil structures were extensively used for the construction of high-speed train lines, which is among the most critical and important infrastructures in Japan (Tatsuoka et al., 2012a & b). The first GRS integral bridge was constructed 2012 for a high-speed train line and three others were also constructed in 2013. Fifthly, soil structures are now designed against very high seismic loads (called Level 2 design seismic load) as experienced during the 1995 Kobe Earthquake, in the similar way as RC and metal structures (Tatsuoka et al., 1998, 2010; Koseki et al., 2006, 2008; Koseki, 2012). So far, all these GRS structures have performed very well. Having experienced the 1995 Kobe and the 2011 Great East Japan Earthquakes and others and many events of heavy rains and floods, these GRS technologies have been validated as are very cost-effective, in particular by having very high resistance against these severe natural disasters.

Most recently, various types of GRS structures were densely constructed for a new high-speed train line, called Hokkaido Shinkansen (Figure 2a; Yonezawa et al., 2013).

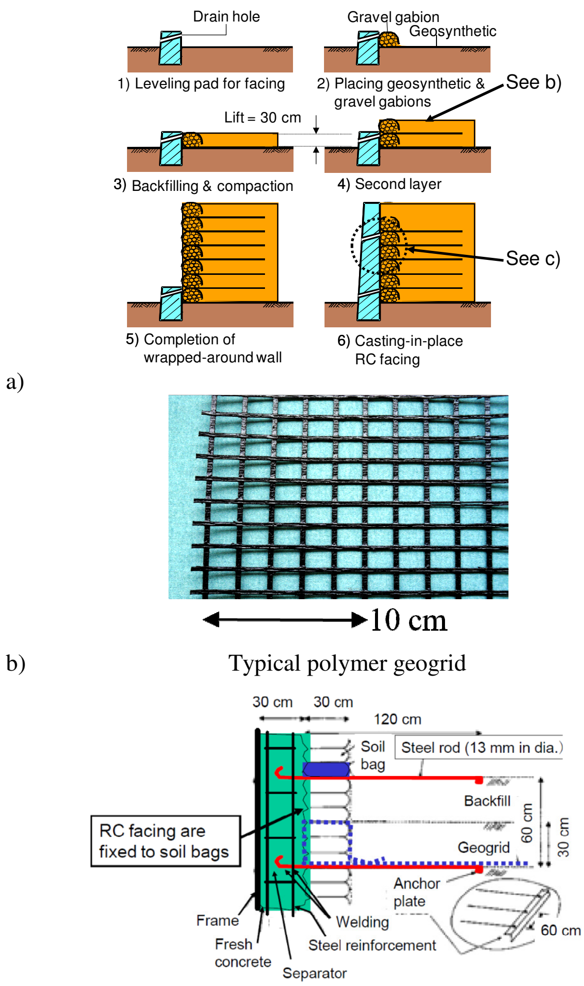
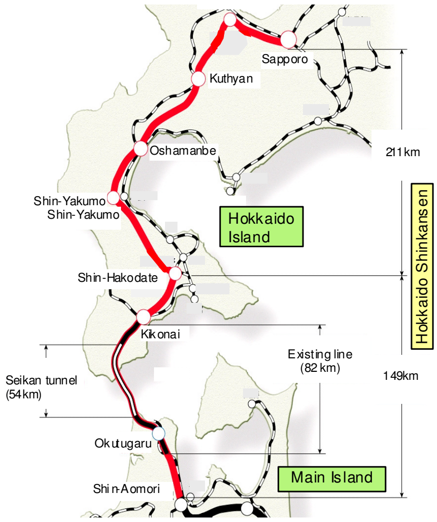


Figure 1 GRW RW with FHR facing:
a) staged construction; b) a typical geogrid; and
c) facing construction (Tatsuoka et al., 1997a)

The construction started in 2005 and will end by the end of 2014. At many sites within a length of 37.6 km between Kikonai and Shin-Hakodate Stations (Figure 2b), the following GRS structures were constructed: 1) GRS RWs having FHR facing (at sites denoted by R in Figure 2b) for a total length of 3.5 km with the largest wall height of 11 m: no conventional type cantilever RW was constructed. Figure 3 shows a typical case. 2) In total 29 GRS bridge abutments (denoted by A): no conventional type bridge abutment was constructed. The tallest one is 13.4 m-high. 3) A GRS integral bridge (denoted by I) at Kikonai. 4) Three GRS Box Culverts to accommodate local roads under-passing the railway (denoted by B): a RC box structure is integrated to GRS RWs at both sides. The tallest one is 8.4 m-high. 5) Eleven GRS Tunnel Entrance Protections (denoted by T): 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.



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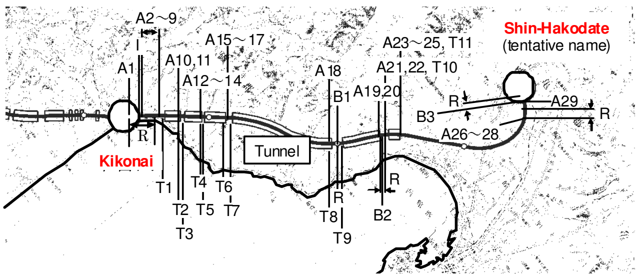


b)

Figure 2 a) Location of Hokkaido Shinkansen (high-speed train); and b) locations of GRS structures (Yonezawa et al., 2013)

Figure 3 A view at stage 6 in Figure 1a of GRS RWs at both sides of a box culvert, site B2 in Figure 2b (Yonezawa et al., 2013)

These GRS structures were selected because of their very high cost-effectiveness; i.e., lower construction and maintenance cost with a higher functionality including a higher seismic-stability. In particular with GRS bridge abutments, GRS integral bridges and GRS box culverts, the settlement in the backfill immediately behind the facing by long-term train loads and seismic loads becomes negligible, unlike the conventional type structures.

In this paper, the lessons from experiences with these GRS structures gained during the last 25 years and the essence of the new seismic design method are summarised.

2. GRS RW with FHR FACING

2.1 Staged construction

As shown in Figure 1a, after the deformation of the subsoil and the backfill upon the construction of geosynthetic-reinforced backfill has taken place sufficiently in such that the residual deformation would not damage the completed wall, full-height rigid (FHR) facing is constructed by casting-in-place concrete in the space between the outer concrete frame 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 reinforcement wrapping-around gravel bags. Figure 1b 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 usually used. Besides, 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 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, as it is before the construction of FHR facing, the backfill immediately in back of the wall face can be well compacted.

Before the construction of FHR facing, the gravel bags function as a temporary but stable facing that resist 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 provide drainage and act 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 and its propping are necessary on both sides of the facing and they become more increasingly costly with an increase in the wall height. With this type of GRS RW, on the other hand, only an external concrete form, anchored with steel rods in the backfill, is necessary without using any external propping (Figure 2c). Figure 3 shows a typical case during the construction of FHR facing.

2.2 Roles of full-height rigid facing

Tatsuoka et al. (1989, 1992, 200), Horii et al. (1994) and Murata et al. (1994) reported results from a series of static loading and shaking table tests on small-scale and full-scale models of GRS RWs and numerical analysis to evaluate the effects of facing rigidity on the stability of GRS RWs. They showed the following. If the wall face is loosely wrapped-around with geogrid reinforcement without using 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 (Figure 4a). 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 this GRS RW system, before the construction of FHR facing, the gravel bags function as a temporary facing; therefore, high earth pressure can be activated at the wall face (Figure 4b). This high earth pressure is transferred to the FHR facing upon its construction, which results in high confining pressure at the wall face, thus high stiffness and strength of the active zone, then, high performance of the wall. This mechanism is particularly important to ensure high seismic stability.

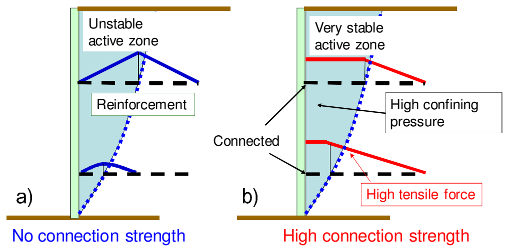


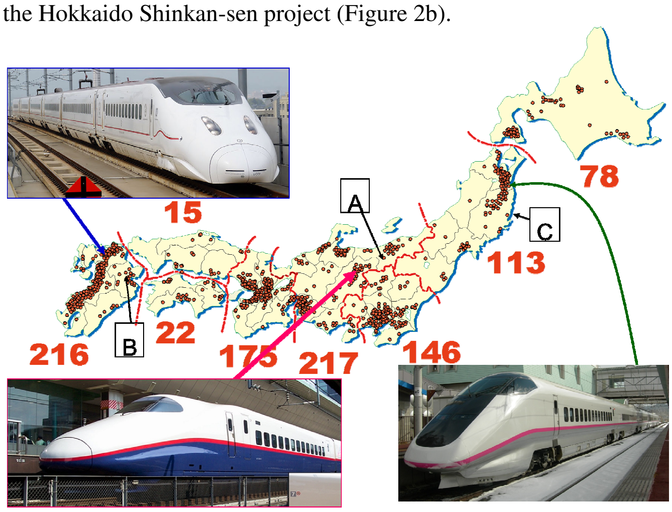
Figure 4 Effects of firm connection between the reinforcement and the rigid facing (Tatsuoka, 1992)

A conventional type RW is a cantilever structure resisting the active earth pressure from the unreinforced backfill. Therefore, large internal moment and shear force is 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, in particular when constructed on thick soft subsoil. These disadvantages become more increasing serious with an increase in the wall height. 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 mobilised in the FHR facing even by high large earth pressure. Hence, the FHR facing becomes much simpler and lighter than conventional cantilever RC RWs. Besides, as only small overturning moment and lateral thrust force is activated at the facing bottom, a pile foundation is not used in usual cases. If constructed on relatively soft ground, shallow ground improvement by cement-mixing is usually 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 speedy 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 to the top of the facing or the crest of the backfill immediately behind the facing. The load is distributed to large part of 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.

As described in Section 4, GRS bridge abutments and GRS integral bridges were developed by taking advantage of these features of FHR facing described above. In particular, with these GRS bridge structures, a negligible bump develops immediately behind the FHR facing constructed as the bridge abutment. On the other hand, reinforced soil RWs having discrete panel facing lack such a structural integrality as above, exhibiting much lower resistance against concentrated load. Besides, local failure of the facing (such as loss of a single panel) may result in the collapse of the whole wall.

2.3 A brief history of GRS RW with FHR facing

Until today (June 2013), GRS RWs with FHR facing have been constructed for a total length more than 150 km at 982 sites, mainly for railways and many for high-speed train lines (Figure 5: Tatsuoka et al., 2012a, b). In urban areas, near vertical retaining walls (RWs) have significant advantages over conventional gentle-sloped embankments as railway structures because of: a) much more stable behaviour; b) much smaller base areas, which significantly reduces the cost for land acquisition; c) no needs for barrier walls, protection work, vegetation and long-term maintenance of the embankment slope; and d) a much smaller volume of ground improvement of soft sub-layer if required. For these reasons, a great number of conventional type RWs (unreinforced concrete gravity type or RC cantilever type) had been constructed in urban areas. On the other hand, in country sides, conventional gentle-sloped embankments are usually constructed due to a high construction cost of conventional type RWs, in particular when long piles are necessary. However, now, it is much more cost-effective to construct GRS RW with FHR facing not only in urban areas but also at country sides, typically in the Hokkaido Shinkan-sen project (Figure 2b).



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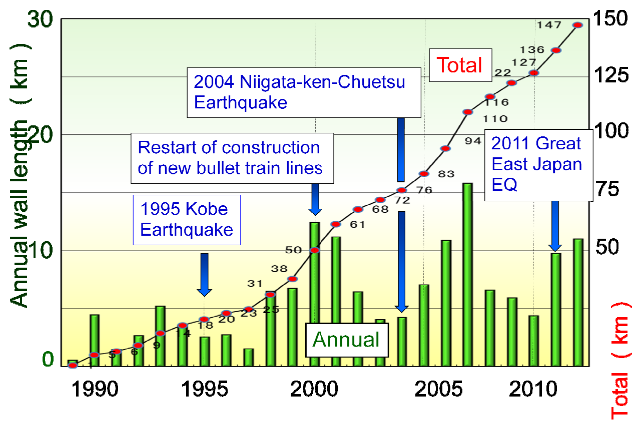


b)

Figure 5 a) Locations of GRS RWs with a staged-constructed FHR facing as of June 2013; and b) annual and cumulative lengths

3. SEISMIC DESIGN

The very high seismic stability of the GRS RW of this type was validated by its high performance during the 1995 Kobe Earthquake, as typically seen from Figure 6. This feature was re-confirmed by many similar cases during the 2011 Great East Japan Earthquake (Tatsuoka et al., 2012a, b). Based on these experiences, a number of conventional type RWs and embankments that collapsed by these and other earthquakes, as well as those that collapsed due to heavy rains, floods and coastal wave actions during typhoon, were reconstructed to this type GRS RWs. Some recent case histories are described in Section 5.

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 Kobe Earthquake (Koseki et al., 2006, 2007, 2009; Koseki, 2012;

Tatsuoka et al., 2010). Since then, the code has been consistently revised referring to new lessons from subsequent earthquakes. The latest version of *Design Standard for Railway Soil-Retaining Structures* (edited by Railway Technical Research Institute) was published in 2012. The new seismic design code has several characteristic and unique features including the followings.

Figure 6 A GRS RW having FHR facing at Tanata, Kobe city, one week after the 1995 Kobe Earthquake (Tatsuoka et al., 1997a, 1998)

Firstly, depending on the importance of concerned structures, three ranks of required seismic performance are specified in the same way as other civil engineering structures (Table 1); e.g., soil structures supporting RC slabs for ballast-less tracks of high speed train lines are required rank I, those supporting ballasted tracks for important railways are required rank II, and other non-critical soil structures are required rank III. Level 1 design seismic load is used in the pseudo-static seismic stability analysis, which is assigned to be a horizontal seismic coefficient at the ground surface k_h equal to 0.2. This design seismic load is equivalent to the conventional one that had been used before the recent revision of the code (i.e., before the 1995 Kobe Earthquake). It is assumed that the acceleration is not amplified inside soil structures. Level 2 design seismic load was newly introduced, which is equivalent to severe seismic loads experienced during the 1995 Kobe Earthquake. This is assigned in terms of standard time histories of horizontal acceleration at the ground surface and is used to evaluate the residual deformation of soil structure by the modified *Newmark* sliding block analysis. Depending on the natural period T_g of the ground at a given site, different wave forms and amplitudes are assigned. The assigned peak accelerations a_{max} are very high, in a range from 500 to 920 gals (cm/sec^2).

Table1 Three performance ranks for two design seismic load levels

Design EQ loads / Required performance	**Level 1:** Conventional design EQ load[1)]	**Level 2:** Severe seismic loads as experienced during the 1995 Kobe EQ[2)]
Very important soil structures: e.g., high speed trains (**rank I**)	Limited deformation: expected functions can be maintained without repair works	Allowed to exhibit deformation as far as their functions can be restored by quick repair works
Important soil structures: e.g., urban trains (**rank II**)		Should not exhibit devastating deformation. The functions can be restored by repair works.
Other non-critical soil structures (**rank III**)	Should not collapse	Not specified

1) anticipated to take at a given site place several times during the design life time.
2) the largest seismic load anticipated at a given site during the design life time.

Secondly, it is among the most important lessons learned from failure and collapse of a great number of embankments and conventional type RWs by heavy rains, floods and severe earthquakes that good backfill compaction and good drainage are essential to prevent such failure and collapse. Based on this lesion, to facilitate good compaction of the backfill, with GRS RW having FHR facing, the spacing between vertically adjacent geosynthetic layers is specified to be 30 cm, while the standard compacted lift of soil layer is 15 cm. Besides, it is allowed to use the φ_{peak} values listed in Table 2 in the design against Level 2 seismic load only when good compaction is ensured. For example, for very important soil structures that are required to exhibit performance rank I against Level 2 seismic load, both of the following criteria should be satisfied to use these φ_{peak} values; 1) all measured values of D_c (Standard Proctor) $\geq$ 92 %; and the average $\geq$ 95 %; and 2) all measured values of the coefficient of vertical sub-grade reaction (K_{30}) obtained by plate loading tests using a 30 cm-diameter plate $\geq$ 70 MN/m^2; and the average $\geq$ 110 MN/m^2. The standard design shear strengths listed in Table 2 were determined conservatively based on results of a comprehensive series of drained triaxial compression tests on many backfill samples representative of the railway soil structures in Japan. Note that even higher peak strengths of the backfill can be used if they are confirmed by relevant investigations including laboratory stress-strain tests.

With the GRS RW having FHR facing, gravel bags are placed at the shoulder of each soil layer to help better backfill compaction. Based on the lesson that good drainage is another key for high performance of soil structures, the gravel bags are also expected to function as a vertical drainage during service. The water percolating into the gravel bags from the backfill is drained to the outside of the wall through small pipes arranged for every 2 to 4 m^2 in the facing. It is considered that, with good drainage, positive water pressure may not develop even during heavy rains. At the same time, with all soil types, the apparent cohesion, which is basically due to matrix suction, is ignored (i.e., c= 0) in the wall design under not only static but also seismic loading conditions. This is because the apparent cohesion may disappear in an uncontrolled manner with an increase in the moisture content, typically by heavy rainfall; therefore, it is not reliable. By the same concept, the saturated unit weight of soil is used in all cases.

Thirdly, the limit equilibrium stability analysis (i.e., static analysis and pseudo-static analysis as the first approximation of rigorous dynamic analysis) is the basis for the design. On the other hand, the earth pressure in the unreinforced backfill of full-scale RW and tensile geosynthetic forces in full-scale GRS RWs that are measured under ordinary conditions are usually substantially lower than respective design values that are determined for critical and unusual conditions (i.e., heavy rains and severe earthquakes). This is because the measured values are significantly affected by matrix suction, which may disappear with an increase in the moisture content. Besides, the earth pressure and reinforcement forces measured under ordinary conditions do not include the effects of severe seismic loads. Furthermore, even under saturated conditions, the actually operated drained shear strength of well-compacted backfill may be significantly higher than the conservatively determined design values; usually low default design values similar to the $\varphi_{residual}$ values listed in Table 2 are used. For these reasons, these measured values are not referred to in the wall design for railways, as well as for roads.

Table 2 Standard design values of density and shear strength for wall design

Soil type	φ_{res}	φ_{peak}
1. Well- graded gravelly soil	40°	55°
2. Well-graded sandy soil	35°	50°
3. Poorly graded sand (FC<30%)	30°	45°
4. Soils with fines (FC>30%)	30°	40°

Apparent cohesion due to suction is ignored (i.e., c = 0)

These φ_{peak} values can be used only when well-compacted:
1) all measured values of D_c (standard Proctor) ≥ 92 %, & the average ≥ 95 %; and
2) plate loading test criteria are satisfied
Otherwise, φ_{res} should be used.

Fourthly, the seismic performance of a given soil structure against Level 1 design seismic load is evaluated based on the factor of safety obtained by pseudo-static limit equilibrium stability analysis. On the other hand, the performance against Level 2 design seismic load of unreinforced embankment is evaluated based on residual displacement obtained by the modified *Newmark* sliding block theory. The basis for this analysis is also limit equilibrium stability analysis. With well-compacted backfill, it is conservatively assumed that, after having reached the peak value φ_{peak}, the angle of internal friction φ suddenly drops to the residual angle $\varphi_{residual}$. With actual granular soils, the thickness of shear band is essentially proportional to the particle size, while the shear strain increment in the shear band that is necessary to reach the residual state is rather independent of the particle size (i.e., of the order of 100 %) (Yoshida et al., 1995, 1997; Okuyama et al., 2003; Tatsuoka et al., 1998; Tatsuoka, 2001). Therefore, the strength fully drops from the peak value to the residual value only after a shear deformation increment that is essentially proportional to the particle size takes place in the shear band.

The residual deformation of RWs, including GRS RWs, is obtained by the modified *Newmark* theory based on the earth pressure analysis by the modified *Mononobe-Okabe* seismic earth pressure theory. The original *Mononobe-Okabe* theory evaluates the seismic earth pressure in the framework of Coulomb's theory using a single linear failure plane in the case of unreinforced backfill. As it is assumed that the peak friction angle φ is kept constant everywhere and every time, the failure plane moves for every change in the input seismic load. For example, when the input seismic load continuously increases, the failure plane continuously becomes deeper (i.e., in Figure 7a, the angle α continuously decreases). In actuality, however, with well-compacted backfill, the φ value drops from φ_{peak} toward $\varphi_{residual}$ only inside a shear band (i.e., a failure plane), while φ remains equal to φ_{peak} in the other unfailing zones. Therefore, when the input seismic load becomes higher than a certain level at which the first failure plane has started developing, this first failure plane develops further without forming another deeper failure plane until the input seismic load becomes large enough. Therefore, during a given time history of seismic load, multiple failure planes may stepwise develop in the backfill. Based on this consideration, Koseki et al. (1997) modified the original *M-O* theory.

For a simple RW configuration with unreinforced backfill (Figure 7a), Figures 7b and c compare the horizontal earth pressure coefficient K_A and the size of the failure zone plotted against the horizontal seismic coefficient k_h obtained by the original and modified *M-O* theories. For the modified *M-O* theory, it is conservatively assumed that φ suddenly drops from φ_{peak} to $\varphi_{residual}$ (i.e., the particle size is assumed to be zero). The following trends may be seen from Figures 7b and c. Firstly, the K_A value by the original theory using $\varphi_{residual}$ becomes extremely high when k_h becomes higher than a certain value. By this feature, the conventional seismic design of RWs for Level 2 seismic load becomes unrealistic when based on the original theory using $\varphi_{residual}$. On the other hand, the K_A value by the modified theory increases stepwise with a continuous increase in k_h, while this K_A value is always smaller than the value by the original theory using $\varphi_{residual}$. Secondly, with a continuous increase in k_h, the failure zone by the modified theory becomes larger stepwise and is consistently smaller than both of those by the original theory using φ_{peak} and $\varphi_{residual}$. This trend is consistent with the model shaking table tests (Koseki et al., 2007, 2009) and field observations (Tatsuoka et al., 1997, 1998).

The seismic stability analysis of GRS RWs with FHR facing is based on the pseudo-static limit equilibrium stability analysis by the two-wedge (TW) method using both of φ_{peak} and $\varphi_{residual}$ (Tatsuoka et al., 1998). This modified TW method is a direct extension of the modified *M-O* theory. A possible increase in the tensile resistance of reinforcement associated with residual deformation of the wall is ignored as a conservative simplification. For a typical GRS RW wall configuration depicted in Figure 8a, Figure 8b compares the overall safety factors for failure by sliding and overturning obtained by the TW method using φ_{peak} and $\varphi_{residual}$ with those by the TW method using a single φ value, either φ_{peak} or $\varphi_{residual}$. The response amplification inside the RW is ignored. In this particular analysis, it is assumed that the first failure takes place in the backfill when k_h= 0.28. The critical failure planes obtained by the modified TW method under this condition are depicted in Figure 8a. The safety factor by the modified TW method (using φ_{peak} and $\varphi_{residual}$) is always in between the values by the TW method using $\varphi_{residual}$ (i.e., the conventional design) and the TW method using φ_{peak}.

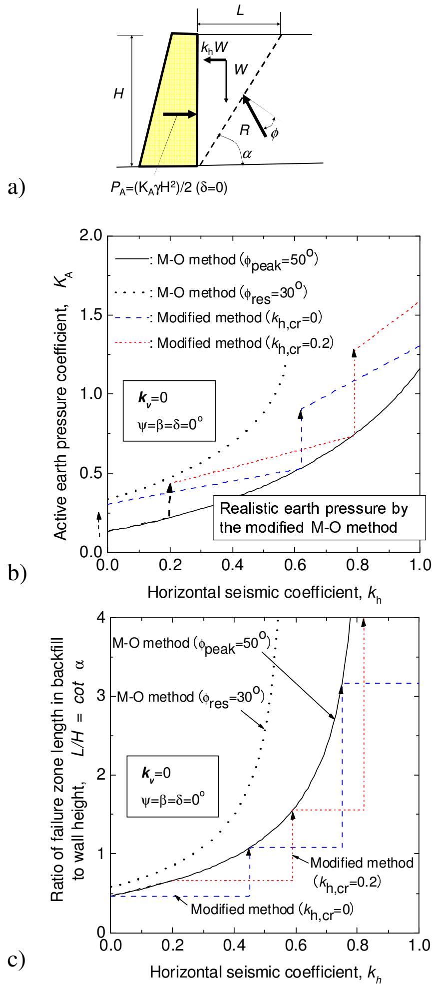


Figure 7 a) Considered simple wall configurations; and comparisons between the original and modified *Mononobe-Okabe* theories of: b) horizontal seismic earth pressure coefficient, K_A; and c) the size of failure zone, L (K_A and L are defined in Figure a) (Koseki et al., 1997)

Based on such results of analysis as shown above, 1) horizontal sliding displacement; 2) overturning displacement; and 3) shear deformation of the reinforced backfill are evaluated by the modified *Newmark* method. The allowable residual deformation of a given soil structure is specified by the owner of the concerned soil structure based on the criteria shown in Table 1. For example, for performance rank III, the ballasted track may allow a maximum residual settlement of 50 cm.

Fifthly, in the same way as other ordinary design procedures for GRS structures, the design rupture strength for long-term static loading conditions $(T_d)_{static}$ of geosynthetic reinforcement is obtained by applying a set of reduction factors to "tensile rupture strength obtained by fast loading test of new product T_{ult}" (Figure 9). These reduction factors account for: 1) installation damage; 2) long-term degradation; 3) the possibility of creep rupture; and 4) overall safety factor. With respect to the creep reduction factor, it is specified in the related *Japanese Railway Design Code* that the T_{ult} value (before applying the global safety factor, $(F_s)_{static}$) is equal to the maximum value at which the creep failure does not take place at the end of 50 years. It is postulated that the above condition is satisfied if the strain rate after 500 hours becomes smaller than 3.5 x 10^{-5}/h in all three creep loading tests on a given geosynthetic reinforcement type.

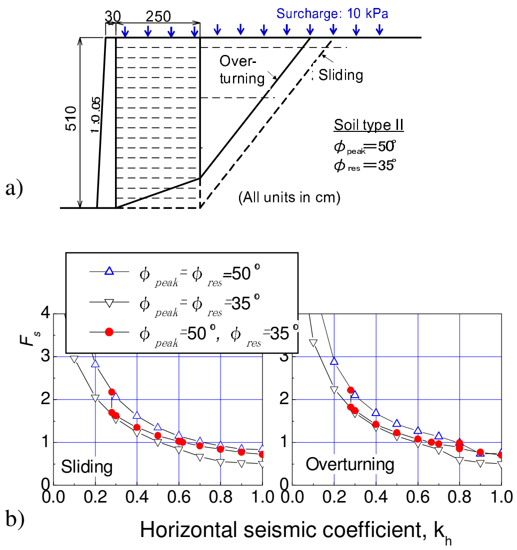


Figure 8 a) GRS RW with FHR facing and critical failure planes by modified TW method; and b) results of stability analysis: the tensile rupture strength of reinforcement T_d= 30 kN/m; and the friction angle at the reinforcement/backfill interface (φ_B) and the back and bottom of facing (δ_w)= $\varphi_{residual}$ of the backfill (Horii et al., 1998)

In the *Japanese Railway Design Codes*, the design seismic rupture strength $(T_d)_{seismic}$ is obtained without taking into account the creep reduction factor that is determined to avoid creep rupture under static loading conditions for the following three reasons:

1) The design rupture strength $(T_d)_{static}$ (before applying $(F_s)_{static}$) required for a given GRS RW is determined by limit equilibrium stability analysis using several conservative assumptions (i.e., using conservative design values of ϕ while ignoring apparent cohesion and toe resistance). The creep reduction factor is determined by assuming that the tensile load is kept to this design static strength during the lifetime of the structure. As explained earlier, the actual tensile load (L_a in Figure 9) activated under ordinary non-critical conditions, which occupies most of the design lifetime, is considerably lower than this value of $(T_d)_{static}$.
2) As illustrated in Figure 9, the creep process is conservatively assumed to start after the geosynthetic reinforcement has fully deteriorated by the end of the lifetime, although, in actuality, the creep process starts contemporarily with material degradation (Kongkitkul et al., 2007b).

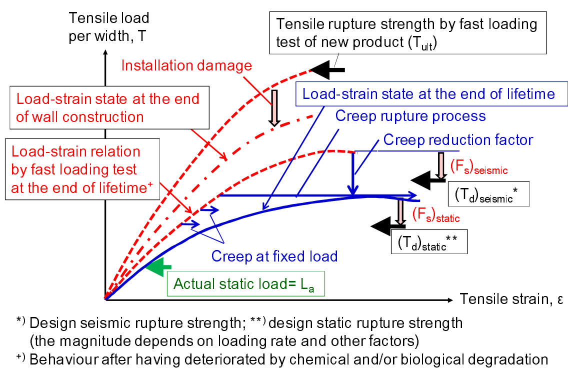


Figure 9 Procedure to obtain the design rupture strengths $(T_d)_{static}$ and $(T_d)_{seismic}$ of geosynthetic reinforcement under long-term static and seismic loading conditions, compared with actual long-term static load L_a

3) Figure 10 shows typical tensile loading test results. In one of the three tests, sustained loading (SL) was applied for 30 days during otherwise monotonic loading (ML) at a constant strain rate. Upon the restart of ML at a constant strain rate after SL, the load-strain relation soon rejoins the one from the continuous ML loading tests. The rupture strength in this test is a rather unique function of the strain rate at rupture and essentially the same as those obtained by two continuous ML tests not including SL at an intermediate stage. This result indicates that, unless the material property degrades with time by chemical and/or biological effects, the original strength for a given strain rate of a given geosynthetic reinforcement is maintained until late in its service life. When subjected to seismic loads after some long service period under constant load conditions, the original strength at a fast strain rate can be fully activated (Greenwood et al. 2001; Tatsuoka et al., 2004, 2006; Tatsuoka, 2008; and Kongkitkul et al., 2007a, b).

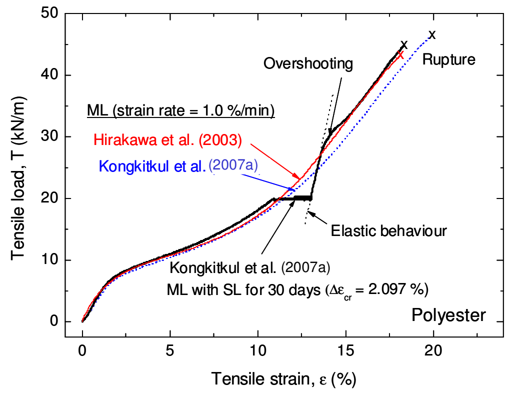


Figure 10 Comparison of tensile load - strain relations from three monotonic loading tests with and without creep loading for 30 days at an intermediate load level, a PET geogrid (Kongkitkul et al., 2007a).

Lastly, as a whole, it is highly recommended to employ GRS structures in place of conventional type embankments, RWs and bridge abutments with unreinforced backfill when relevant and feasible. In fact, it is extremely difficult to cost-effectively design conventional type soil structures against Level 2 seismic load. On the other hand, when the backfill is well-compacted and its effect on the design shear strength of backfill is taken into account (as described above), GRS structures become a cost-effective solution.

4. GRS STRUCTURES FOR BRIDGE

4.1 GRS bridge abutments

Large bumps immediately behind the bridge abutments that may develop by depression of the unreinforced backfill due to long-term train loads and displacements of the wing RWs and the abutment during severe earthquakes is one of the most serious problems with conventional type bridge abutments. 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, the full-scale field behavior showed that this measure is not satisfactory and this behaviour was confirmed by laboratory model shaking table tests (Aoki et al., 2005; Tatsuoka et al., 2005). Then, the authors and their colleagues developed a new type bridge abutment (Figure 11). The bridge girder is placed either 1) via a fixed (or hinged) bearing on the top of the FHR facing of a GRS RW and via a movable (or roller) bearing on the top of a pier; or 2) via a set of bearing (hinged and roller) on the top of the FHR facings of a pair of GRS RWs. To ensure high performance of bridges, in particular 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 are described in details in Tatsuoka et al. (2005). The gravel bags immediately behind the facing are filled with un-cemented gravelly soil so as to function as 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.

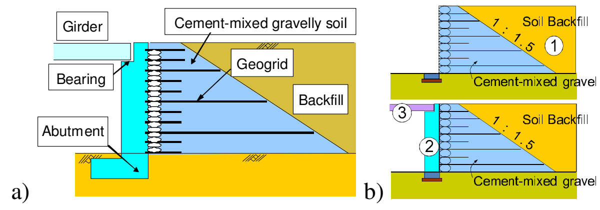


Figure 11 a) GRS bridge abutment; and b) construction procedure

The first advantage of GRS bridge abutment is a much higher seismic stability with a minimum bump even if subjected to very severe seismic loads. Yet, it is much less costly resulting from much more slender RC facing and usually also by no use of a pile foundation. 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 during a period of 2002 - 2003 at Takada for Kyushu Shinkansen. By performing full-scale vertical and lateral loading tests of the facing, it was confirmed that the connection strength against separation between the FHR facing and the geogrid-reinforced backfill is sufficiently high (i.e., much higher than the inertia of the girder when subjected to Level 2 seismic load that the abutment is to support). For 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 (Figure 12). Until today, in total about 50 GRS abutments of this type have been constructed for railways.

a)

b)

c)


Figure 12 GRS abutment at Mantaro for Hokkaido Shinkansen (A21 in Fig. 2b): views under construction; a) from the front side: and b) from the backside; and c) completed (Yonezawa et al., 2013)

4.2 GRS integral bridge

The use of bearings (movable or fixed or both) to support the girder is the most serious remaining problem with GRS bridge abutment. To alleviate this problem, GRS integral bridge, illustrated in Figure 13, was developed based on a series of model shaking table tests (Tatsuoka et al., 2008, 2009, 2012; Munoz et al., 2012) and the construction and loading tests of a full-scale model (Suga et al., 2012). The only but significant difference of GRS integral bridge (Figure 13) from GRS bridge abutment (Figure 11) is that, with a GRS integral bridge, a continuous girder is 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 bearing becomes unnecessary. Secondly, the RC girder becomes more slender due to a significant reduction of moment resulting from flexural resistance at the connection between the girder and the facing. Thirdly, as demonstrated by various model tests and numerical analysis, the seismic stability increases significantly due to an increased structural integrality and a reduced weight of the girder. Fourthly, due to higher structural integrality and a smaller cross-section of the girder, the resistance against tsunami loads increases significantly. In addition, GRS integral bridges exhibit essentially zero settlement in the backfill and no structural damage to the facing by lateral cyclic displacements of the facing caused by seasonal thermal expansion and contraction of the girder (Tatsuoka et al., 2009).

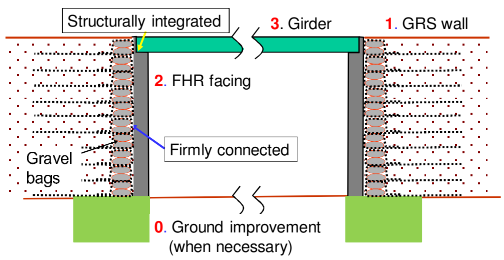
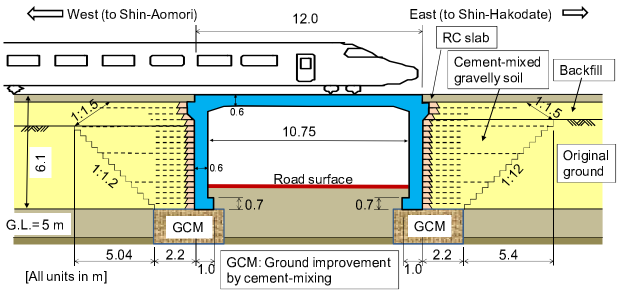


Figure 13 Construction sequence of GRS integral bridge

The first GRS integral bridge was constructed as the over-road bridge at Kikonai (Figure 14). 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 its behavior continuously from the start of construction until sometime after the start of service (scheduled to be April 2014) (Kuriyama et al., 2012). 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 was confirmed that the structure is not over-stressed at all. Results of detailed analysis will be reported by the authors in the near future.

4.3 GRS box culverts

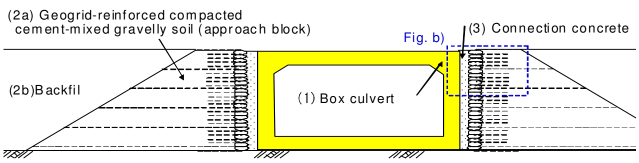
At three sites (B1, B2 and B3 in Figure 2b), where Hokkaido Shinkansen crosses local roads, RC box culverts (i.e., underpass structures) integrated to the geogrid-reinforced backfill on both sides (called GRS box culverts) were constructed. Figure 15a shows the structure of those constructed at sites B2 and B3. A RC box structure was first constructed to reopen the local road as soon as possible and then GRS RWs comprising of 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 Figure 15b. Finally, concrete was cast-in-place into this space to integrate the RC box culvert to the GRS RWs. For a high integrality 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 a RC box structure after the ground settlement due to the weight the approach fills has taken place sufficiently so that the RC box structure becomes free from negative effects of ground settlement.



a)

b)

Figure 14 GRS integral bridge at Kikonai, Hokkaido Shinkansen (at site I in Figure 2b)



a)

b)

Figure 15 GRS culvert box culvert for Hokkaido Shinkansen: a) general structure; the numbers dente the construction sequence (site B2 in Figure 2b); and b) a space between the RC box structure and the approach block before step (3) (site B1) (Yonezawa et al., 2013)

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

5. FLOOD AND TSUNAMI

5.1 Several latest case histories of flood

A great number of embankments for roads and railways retained by conventional type cantilever RWs along rivers and seashores collapsed by floods and storm wave actions, usually triggered by over-turning failure of the RWs caused by scouring in the supporting ground (Figure 16a: Tatsuoka et al., 2007). Upon the collapse of RW, the backfill is quickly and largely eroded, resulting in the closing of railway or road. This type of collapse easily takes place, as the stability of a cantilever RW fully hinges on the bearing capacity at the bottom of the RW. On the other hand, GRS RWs with a FHR facing is much more stable against the scouring in the supporting ground (Figure 16b). It is particularly important that the facing does not overturn easily and the backfill can survive unless the supporting ground is extremely scoured. Tatsuoka et al. (2012a, b) reported large-scale overturning collapse of gravity type RW for a road (Seisho bypass) for a length of about 1.5 km along a seashore facing the Pacific Ocean. The collapse of the RW was triggered by scouring in the supporting ground, as illustrated in Figure 16a, by strong ocean waves during a typhoon No. 9, 29th Aug. 2007. The wall was reconstructed to a GRS RW with FHR facing.

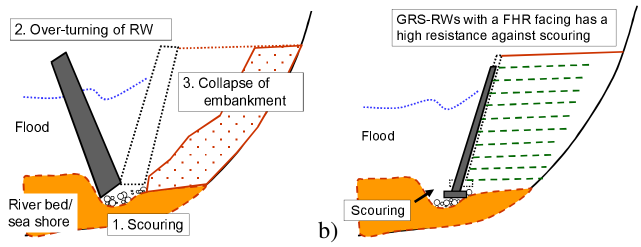
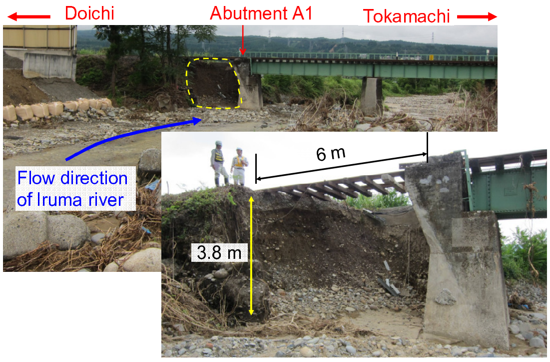


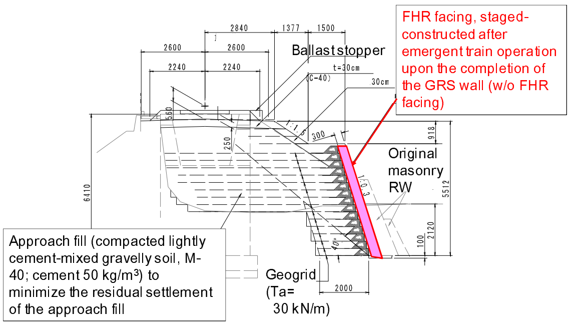
Figure 16 a) Collapse of cantilever RW by scouring in the supporting ground; and b) much better performance of GRS RW with FHR facing.

Flooding took place in many rivers by the Niigata-Fukusima heavy rainfall at the end of July 2011. In Tokamachi city the maximum rainfall intensity was 120 mm/hour 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, Niigata Prefecture, for West Ban-Etsu Line of East Japan Railway collapsed by the same mechanism. The wall was reconstructed to an about 9.4 m-high and 50 m-long GRS RW with a FHR facing. By this heavy rainfall, soil structures at more than 150 sites of Iiyama Line were seriously damaged. Among them, a masonry wing RW of the approach fill of Iruma River Bridge (site A in Figure 5a) collapsed by the same mechanism (Figures 17a & b). It was required to re-open the railway in ten days after the failure. It takes much more days if the original masonry RW is reconstructed. On the other hand, it was feasible with a GRS RW (Figure 17b). Figure 17c is a view during construction. The railway was re-opened with slowed-down operation of trains before the construction of a FHR facing. Figure 17d shows the completed wall.

At site B indicated in Figure 5a in the Mt. Aso area in Kyushu Island, a series of railway embankments located in narrow valleys between tunnels for Hohi Line fully collapsed during heavy rainfall on 2 July 1990 (Figure 18). Flood water was trapped in back of the upstream slope of each embankment due to the clogging of a drain pipe crossing the embankment. The embankments collapsed by over-topping flood water. Debris flows took place, as seen from Figure 18a, and attacked residential houses at the lower reach of the embankments. The entire sections of the six embankments were reconstructed to geosynthetic-reinforced embankments, as typically shown in Figure 19, to reduce the amount of earthwork while keeping a high stability of embankment. To arrange a 3 m-diameter drain corrugate pipe, a nearly vertical GRS RW with a FHR facing was constructed at the downstream toe of each embankment.



a)



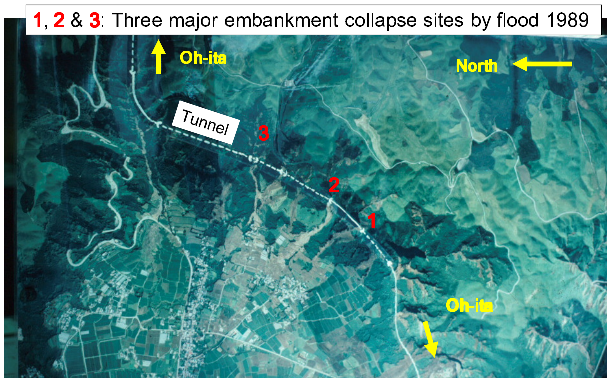
b)

c)

d)

Figure 17 a) Collapse of a masonry RW of 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) & d) restoration to a GRS RW with FHR facing, Iiyama Line, JR East (Takisawa et al., 2012)

On the 12th through 14th July 2012, 22 years after the event described above, another, more severe rainfall attacked these sites (Figure 20). The total precipitation during a period from early morning 12 July till evening 14 July reached 816.5 mm with a peak of 500 mm for 5 hours and 106 mm/hour, which was much more intense than the 1990 heavy rainfall with a total precipitation of 650 mm and a peak of 67 mm/hour. A number of embankments, including those that did not collapse by the 1990 heavy rainfall, were seriously damaged or totally collapsed by scouring, erosion by over-topping flood flow and the development of positive pore water pressure after a loss of suction by seepage flow of rain water. The total number of the damage sites of the railways of JR Kyushu was 201, among which 133 sites were along Ho-hi Line. The total damage cost exceeded five billion yen.



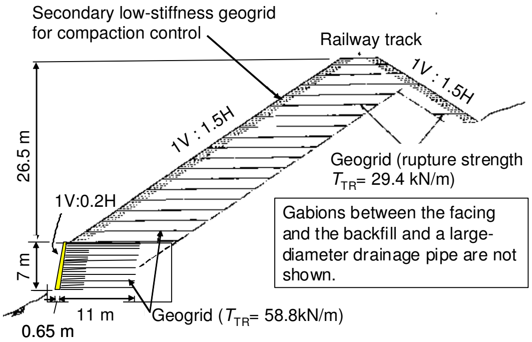
a)

b)

Figure 18 a) Locations of three major embankment failures by heavy rainfall in 1990 (site C in Fig. 5a), Hohi Line, JR Kyushu; and b) a view from the downstream at site 2 (Tatsuoka et al., 1997a)

The three major geosynthetic-reinforced (GR) embankments that were reconstructed in 1991 were attached by over-topping flood due to clogging of the 3 m-diameter corrugate drain pipes caused by mudflow from the upper reach. However, these GR embankments were only partially eroded, despite that they were not designed against such over-topping flood. In Figure 21a, at site 2, the left-hand section of the embankment located between two tunnel exits is unreinforced backfill that survived the 1990 flood and remained unchanged during the restoration work. This section was severely eroded by the overtopping flood by the 2012 rainfall (Figure 21b). In Figure 21c, the eroded section of the unreinforced embankment had been excavated to some extent for restoration works.

On the other hand, the right-hand section of the embankment seen in Figure 21a is located over the deepest place of the valley. This part was fully eroded by the 1990 flood and reconstructed to a GRS structure (Figure 19). This section performed very well during the 2012 heavy rainfall. It may be seen from Figure 21c that only some surface portion of the downstream slope of this section was eroded. The exposed cross-section of the GR embankment section is shown in Figure 21d. Although relatively deep gullies were formed in the unprotected downstream slope of the GR embankment section, these gullies did not further develop due likely to the resistance of geogrid layers against erosion. As seen from Figure 20, this time, debris flows did not attack the houses at the lower reach of the embankments, due to barriers constructed in 1991 and a limited scale of failure of the GR embankments. The reconstruction of the damaged embankments to GRS structures was completed by the end of August 2013.



a)

b)

c)

Figure 19 a) Cross-section; b) a view during reconstruction in 1991; and c) a view in 1994 of the reconstructed GR embankment. Site 2 in Figs. 18a & 20 of Ho-hi Line, JR Kyushu

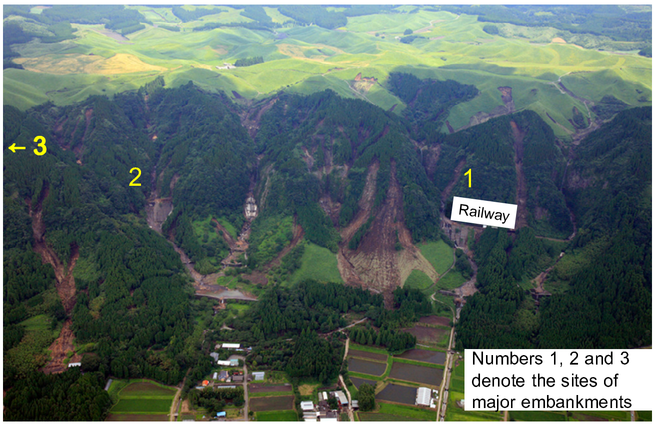


Figure 20 Aerial photograph of Hohi Line immediately after the 2012 heavy rainfall. The picture was provided by PASCO Corporation (http://www.pasco.co.jp/disaster_info/120713/)

a)

b)

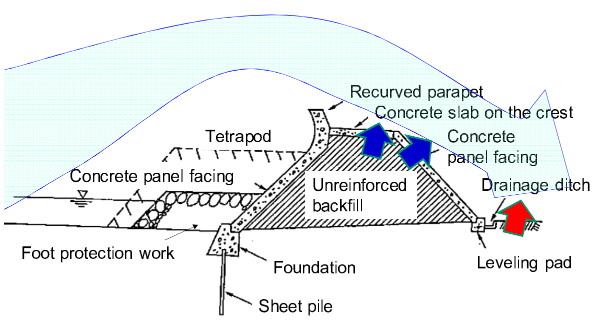
c)

d)

Figure 21 a) Aerial photograph; and b) a close view from upstream (a & b: immediately after the 2012 heavy rainfall, by the courtesy of JR Kyushu); and c) a view from the downstream; and d) exposed cross-section of geosynthetic-reinforced section (c & d: 26 November 2012). Site 2 in Figures 18a & 20 of Ho-hi Line, JR Kyushu

5.2 Collapse of coastal dykes and bridges by tsunami and their restoration

By the 2011 Great East Japan Earthquake, massive tsunami brought destruction along the Pacific coastline of east Japan. Coastal dykes at many places fully collapsed by the following collapse mechanism by deep overtopping tsunami current (Figure 22a): 1) The ground in front of the toe of the downstream slope was scoured. At the same time, the concrete panels at the crest and around the downstream corner at the crest were lifted up by the tsunami current of which the velocity suddenly increased when running down the downstream slope of the dyke. 2) Then, 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. Eventually the backfill was fully washed away and the full-section was lost. On the other hand, small scale model tests (Yamaguchi et al., 2013) indicated that coastal dykes that comprise the geogrid-reinforced backfill covered with continuous lightly steel-reinforced concrete facings firmly connected to the reinforcement, such as those illustrated in Figure 23, have much stronger resistance against deep over-topping tsunami current.



a)

b)

Figure 22 a) Failure mechanism of coastal dyke by overtopping tsunami current; b) typical fully collapsed coastal dyke, Aketo, Tanohara, Iwate Prefecture (site C in Figure 5a).

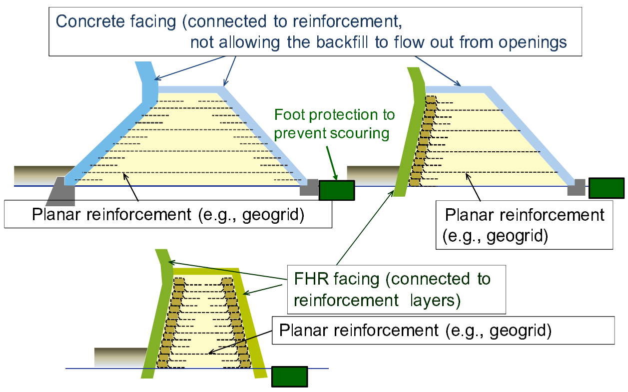


Figure 23 GRS coastal dykes as a tsunami barrier designed to survive deep over-topping tsunami current

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 great tsunami (Kosa, 2012), as typically seen from Figure 24. It was confirmed that a girder supported by bearings has a very low resistance against uplift and lateral forces of tsunami while the unreinforced backfill is very weak against erosion by overtopping tsunami current. 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 bearings and unreinforced backfill are two weak points of the conventional type bridges not only for seismic loads but also for tsunami loads. Tatsuoka and Tateyama (2012) proposed to construct GRS integral bridges (Figure 14) and GRS embankments/dykes (Figure 23) to restore the conventional type bridges of railways and roads that collapsed by the great tsunami of the 2011 Great East Japan Earthquake. A small model test (Kawabe et al., 2013) indicated that, due to a high structural integrity, GRS integral bridge has a much higher resistance against tsunami current than conventional type bridges.

a)

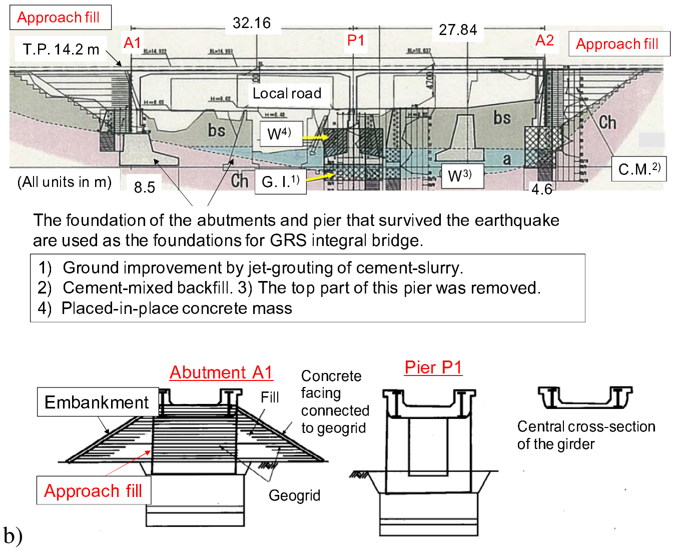
b)

Figure 24 a) Tsuyano-kawa bridge, JR Kesen-numa Line, that lost multiple simple-supported girders by tsunami forces; and b) a view of the back of the right bank abutment of Yonedagawa bridge, Noda, Iwate Prefecture, North-Rias Line, Sanriku Railway

Sanriku railway, which was opened 1984, is running along the coastline where the tsunami damage was very serious. In particular, the bridges located between tunnels in consecutive three narrow valleys facing the Pacific Ocean located just south of the site shown in Figure 22b totally collapsed. Figure 25a shows one of these three sites four months after the earthquake. Tsunami loads were particularly large with these bridges, because: 1) the track level is lowest (12.3 – 14.5 m) along this railway; b) the sites are closest to the coastal line (see Figure 25a); and c) there was no coastal dyke between these bridges and the coastal line. Based on the successful case histories described in this paper and considerations that GRS integral bridges should have a high resistance against tsunami current, it was decided to construct GRS integral bridges to restore these bridges. Figure 25b shows one of the three GRS integral bridges. The total span length of this GRS integral bridge is 60 m, which is much longer than the one at Kikonai.

Figure 26a shows Shima-no-koshi Station of Sanriku railway and its adjacent area before the earthquake. The level of the railway track at the site was 14 m above the sea level. This level was determined based on the previous tsunami disasters in 1896 and 1933. However, this level was not sufficient for this great tsunami. The RC framework structure and the station totally collapsed (Figure 26b) and the tunnel was inundated (Figure 26c).

a)



b)

Figure 25 a) Fully collapsed conventional type bridge (13th July 2012); and b) plan of GRS integral bridge seen from the seaside (under construction) at Haipe (site C in Figure 5a), Sanriku Railway

On the request of the residents at the site, a railway GR embankment was designed as a tsunami barrier following the proposal shown in Figure 23 and was constructed in place of the previous RC framework structure (Figure 27a). Figure 27b shows the representative cross-section of the embankment. The reconstruction includes another GRS integral bridge (Figure 27c). The bridge is underlain by a backfill layer on its top to reduce as much as possible the size of the opening.

Based on these case studies, the adoption of such GRS soil structures as described in this section is recommended for transportation structures that need to be designed against severe earthquakes and strong tsunami currents.

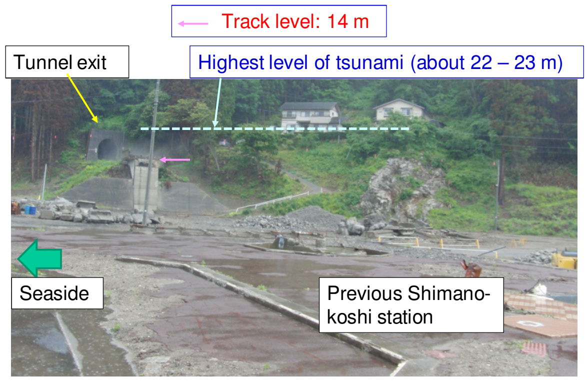
6. IMPORTANCE OF RELEVANT REDUNDANCY

Two of the important lessons that can be learned from the case histories described above and others are that: 1) some relevant redundancy should be intentionally introduced at the design stage to prevent collapse by un-anticipated extreme loads in the future; and 2) the redundancy that the GRS structure inherently has may explore its new applications.

Among the three case histories that are most typical showing the importance of relevant redundancy, the first one is the GRS RW with FHR facing at Tanata (Figure 6). The wall survived Level 2 seismic load during the 1995 Kobe Earthquake, despite that the wall had been designed against much lower seismic load (Level 1). This GRS RW was constructed in 1992; i.e., the wall was designed about five years before the 1995 Kobe earthquake based on the pseudo-static limit equilibrium stability analysis (Horii et al., 1998) requiring a minimum safety factor in terms of horizontal earth pressure equal to 1.5 against a horizontal seismic coefficient k_h equal to 0.2. This safety factor comprises a safety factor equal to 1.25 for the global horizontal equilibrium times a safety factor for tensile rupture failure of geogrid equal to 1.25 (i.e., 1.25 times 1.25 equal to 1.5). It is very likely that the following redundancy prevented the collapse of the wall (Tatsuoka et al., 1998).

a)

b)

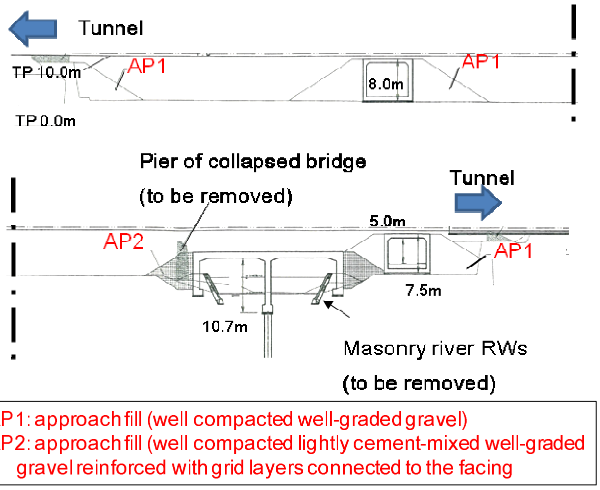


c)

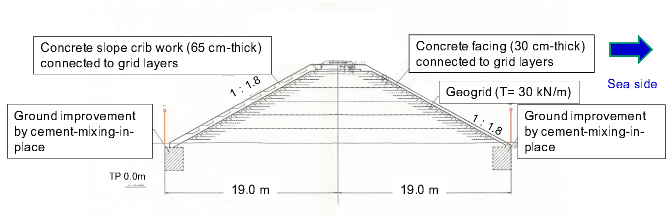
Figure 26 a) A view before the earthquake; b) a view immediately after the earthquake; and c) a view 14 July 2013, Shimano-koshi Station (site C in Fig. 5a), Sanriku Railway

1) The design friction angle ϕ for the backfill (well-graded sandy soil) was a default value (i.e., 35 degrees). As seen from Figure 28, this ϕ value corresponds to a degree of compaction D_c (standard Proctor) equal to about 90 %, which was the allowable lower limit in the field compaction control of the backfill for the wall. The average value of the actual D_c values of the backfill of the wall should have been much higher; therefore, the actual ϕ_{peak} value should be much higher than 35 degrees. Tatsuoka et al. (1998) inferred ϕ= 42 degrees as a much realistic peak value in this case.
2) The apparent cohesion c due to the matrix suction was ignored, despite that its effect on the seismic stability of the wall should have been significant, as it had been no major rainfalls for a long period by the time of the earthquake and the backfill is a well-graded sandy soil with a fines content of about 9 %.
3) The toe resistance was ignored, although it is very likely that this factor was not negligible (see Figure 6).

The second case history is the geosynthetic-reinforced railway embankments that survived over-topping flood by the 2012 heavy rainfall (Figure 21). At the stage of design after the disaster by the 1990 heavy rainfall, overtopping flood in the future was not anticipated, assuming that a 3 m-diameter drain pipe is sufficient. The collapse was prevented due likely to redundancy resulting from geosynthetic-reinforcing of the backfill that was adopted to reduce the amount earthwork by a steep slope while keeping sufficiently high stability. The GR embankments exhibited unexpectedly high resistance against erosion by over-topping flood due to its inherent high integrity.



a)



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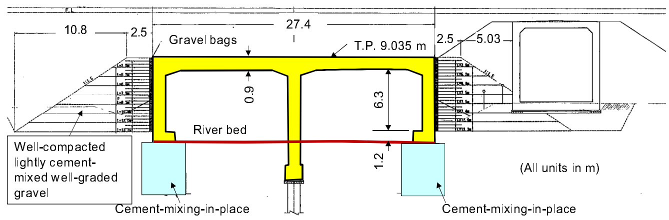

c)

Figure 27 a) Whole reconstructed GR structures; b) cross-section of GR embankment; and c) GRS integral bridge (a & c: seen from the seaside), Shimano-koshi Station (site C in Figure 5a), Sanriku Railway

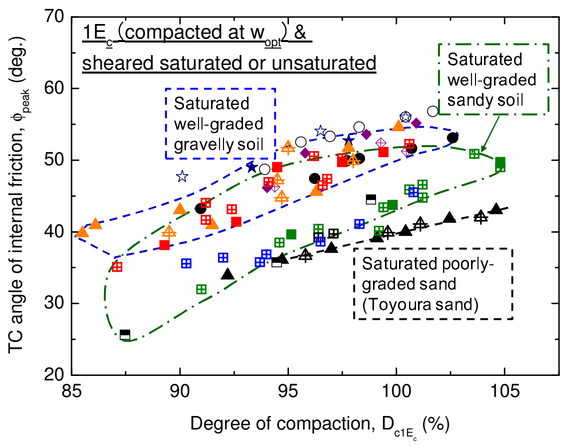

Figure 28 Angle of internal friction (c= 0) of saturated sandy and gravelly soils compacted at respective optimum water contests (mostly by the modified Proctor) from a series of drained triaxial compression tests at confining pressure of 50 kPa (Tatsuoka, 2011).

The third case history is the GRS integral bridges that were constructed to restore three railway bridges that fully collapsed by tsunami during the 2011 Great East Japan Earthquake (Figures 25 - 27). The GRS integral bridge had been developed aiming at a lower cost for construction/long-term maintenance and a higher seismic stability, not aiming at a high resistance against tsunami loads. This adoption was due to a high stability against not only seismic loads but also tsunami loads resulting from integration of girder, facing and reinforced backfill.

The relevant redundancy addressed above is the safety margin that is not covered by the global safety factor that is always used in design. These case histories indicate that the introduction of relevant redundancy is essential to reduce the risk of failure/collapse of soil structures by extreme events that may take place in the future, the whole of which cannot be predicted at the stage of design. The authors believe that the above becomes possible only by such good structure, good design and good construction as described below:

1) Good structure by the following factors:
   a) High structural strength: i.e., large load is necessary to start failure.
   b) High structural ductility: i.e., large energy is necessary to reach full collapse after the start of failure.
   c) High structural integrality: i.e., local failure does not easily result into the collapse of whole structure.

These factors can be realized cost-effectively by means of GRS structures described in this paper.

2) Good design at least by the following means:
   a) Relevant seismic design is done for soil structures in seismic zones. Relevant seismic design also improves long-term performance under static conditions (i.e., small residual deformations). Some soil structures that have not been seismic-designed may survive seismic loads lower than a certain limit. This should be due to redundancy that those soil structures have under ordinary static conditions. However, such case histories observed under limited conditions as above cannot warrant no-seismic design of all soil structures for seismic loads lower than a certain limit. In fact, a number of reinforced soil walls were seriously damaged or fully collapsed during previous earthquakes, due likely to no or no serious seismic design and associated low level of seismic stability (e.g., Tatsuoka et al., 1997b; 1998; Kosek et al., 2006, 2008; Koseki, 2012; Kuwano et al., 2012). No seismic design policy will result into a global reduction of redundancy, thus, a global level down of the stability of soil structure and, therefore, will increase the number of failure/collapse.
   b) With GRS RWs, relevant facing structure and firm facing/reinforcement connection, in addition to relevant geosynthetic reinforcement arrangement, is essential.
   c) The whole of the redundancy created by the adoption of good structure and by the execution of good construction should not be fully taken into account in the stability analysis in design, but part of the created redundancy should be preserved by using conservative shear strength, ignoring the apparent cohesion and toe resistance and others. The use of ϕ_{peak} in addition to $\phi_{residual}$ is to give reward for good compaction while it reduces the redundancy. However, at the same time, the redundancy may increase as this reward encourages good compaction by using design values of ϕ_{peak} that are determined conservatively.
   d) Evaluation of positive effects of structural ductility on the stability, for example based on residual deformation of soil structure, also reduces the redundancy. However, this is only partial evaluation of structural ductility while the whole of positive effects of structural integrity are not evaluated in the current design. Therefore, the evaluation of structural ductility in design encourages the adoption of

soil structures having larger structural ductility and integrality, therefore, having more redundancy.

3) Good construction by the following means:
   a) Use of good backfill, as much as possible.
   b) Good compaction, encouraged by the use of φ_{peak}.
   c) Good drain, by which it can be expected that no positive pore water pressure develops even during heavy rains with walls constructed at water collecting places. Good compaction with good drain may result in significant suction even in such cases as above. This factor is also related to the issues of good structure and good design.

In summary, high redundancy can be produced only by a combination of good structure, good design and good construction. Highly redundant soil structures perform well under extreme conditions. Very importantly with the GRS structures described in this paper, not only their construction cost is usually much lower than respective corresponding conventional type soil structures (i.e., RWs and bridge abutments), but also the cost of this high redundancy can outweigh the cost of failure/collapse and increased maintenance. A great number of case histories has validated the above.

7. CONCLUSIONS

A great amount of geosynthetic-reinforced soil retaining walls (GRS RWs) having a stage-constructed full-height rigid (FHR) facing have been constructed as important permanent RWs in Japan. It is now the standard RW technology for railways. Other types of GRS structure, including GRS integral bridges and GRS coastal dykes, were developed based on this technology. The following conclusions can be derived from the case histories described above:

1. The current popular use of GRS RWs with FHR facing for railway soil structures 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 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, while it 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. The recent seismic design of Japanese railway soil structures, including GRS RWs and GRS integral bridges, are characterized by: 1) introduction of very high design seismic load (Level 2); 2) the use of peak and residual shear strengths with well-compacted backfill (while ignoring apparent cohesion); 3) design based on the limit equilibrium stability analyses; 4) evaluation of seismic performance based on residual deformation obtained by modified *Mononobe-Okabe* and *Newmark* methods; 5) no creep reduction factor for the tensile rupture strength of geosynthetic reinforcement against seismic loads; and 6) recommendations of the use of GRS structures when relevant and possible.
4. A number of conventional type soil structures (i.e., embankments and RWs) that collapsed by earthquakes, heavy rains, floods and storm wave actions were reconstructed to GRS RWs with FHR facing. This standardized practice is due also to a high cost-effectiveness of this type of RW.
5. By the great tsunami during the 2011 Great East Japan Earthquake, a great number of coastal dykes were fully eroded; and a great number of bridges running along the seashore lost their girders and/or approach fills. GRS coastal dykes 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 over-topping tsunami current. GRS coastal dykes and GRS integral bridges were constructed to restore a railway that was seriously damaged by the great tsunami.
6. The GRS soil structures described in this paper can be and have been designed and constructed to have high redundancy so that they can perform well under extreme conditions that are not fully taken into account at the design stage. With these GRS soil structures, not only their construction cost is usually much lower than respective corresponding conventional type soil structures (i.e., RWs and bridge abutments), but also the cost of this high redundancy can outweigh the cost of failure/collapse and increased maintenance.

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