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General overview of experimental studies on seismic stability of Geosynthetic Reinforced Soil Structures and recent research activities

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ABSTRACT

Geosynthetic-Reinforced Soil Retaining Wall (GRS RW) with full-height rigid facing has been constructed for a total length more than 160 km at more than 1,100 sites mainly for railways in Japan. A very high cost-effectiveness with low life-cycle costs and a high stability against heavy rains and severe earthquakes have been validated for the last 30 years. In this paper, the history of the application of GRS structures is first briefly introduced and the experimental studies on seismic stability of GRS structures which were mainly performed in these 15 years are overviewed. Based on these test results and field observation, the design procedure of these GRS structures together with conventional RW and bridge abutment were established and published as “Design Standards for Railway Structures and Commentary (Earth Retaining Structure)” which follows the concept of performance-based design. Finally, the recent research activities applying geosynthetics for the railway structure will be introduced.

1. INTRODUCTION

Geosynthetic-reinforced soil (GRS) retaining wall (RW) staged-constructing full-height rigid (FHR) facing (Fig. 1, Fig.2) was developed about 30 years ago (Tatsuoka et al. 1997). Based on this technology, GRS bridge abutment, placing a girder via a hinged bearing on the top of a FHR facing of a GRS RW, or via hinged and roller bearings on the top of FHR facings of a pair of GRS RWs, was developed in 1990s (Fig. 3: Aoki et al. 2005; Tatsuoka et al. 2005). In 2000s, GRS integral bridge was developed (Tatsuoka et al. 2009, 2015), which integrates, without using bearings, both ends of a continuous girder to the top of the FHR facings of a pair of GRS RW. GRS integral bridge is now becoming one of the standard bridge types for railways in Japan (RTRI, 2012). These types of GRS structure have been constructed for a total wall length of more than 160 km (Fig. 4), mainly for railways including high-speed train lines (Shinkansen in Japanese). Many of them were constructed in place of gentle-sloped embankments, cantilever RC RWs, conventional type bridge abutments, RC viaducts and conventional type bridges.

The primary reason for this popular use of GRS structures is high cost-effectiveness associated with a relatively short construction period and high performance. Among the GRS structures that have been constructed so far (Fig. 4), any problem has not taken place during construction and long-term service and also by prolonged/heavy rainfalls, floods and severe earthquakes for the last about 30 years. Yet, the life cycle cost for the construction and maintenance for a full life span is much lower than conventional type gentle-sloped embankments, conventional cantilever RWs and bridge abutments.

The following is the important factors for the high cost-effectiveness of GRS structures.

1) More environment-friendly than RC viaducts
2) Little displacement or settlement against cyclic traffic load which enable to support RC slab track whose maintenance cost is much lower than the conventional type embankments supporting ballast tracks.
3) GRS bridge abutments and GRS integral bridges exhibit negligible bumps by long-term train loads and seismic loads immediately behind the facing.
4) High seismic stability, as well as the long-term static stability, of this type of GRS RW is very high.

In this paper, we focused on the mechanism of high seismic stability of GRS structure which was proved by the shaking table model tests, In addition, the recent study on the development of GRS which can resist severe earthquake and prolonged overflows caused by Tsunami will be introduced.

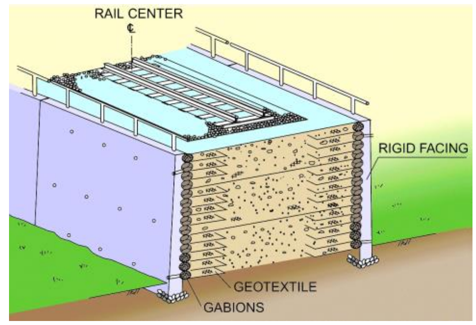


Figure 1. GRS retaining structure

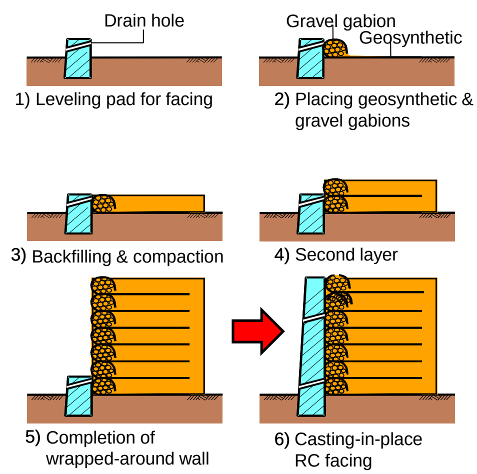


Figure 2. Staged construction of GRS RW with FHR facing

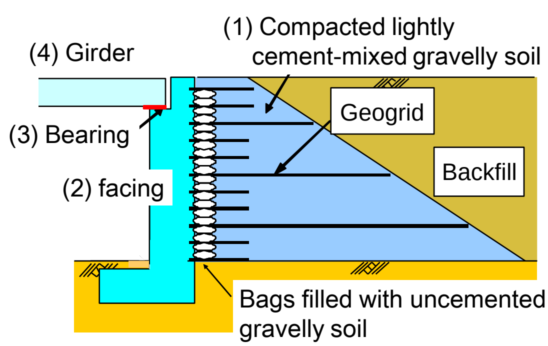


Figure 3. GRS bridge abutment (the numbers denote the construction steps).

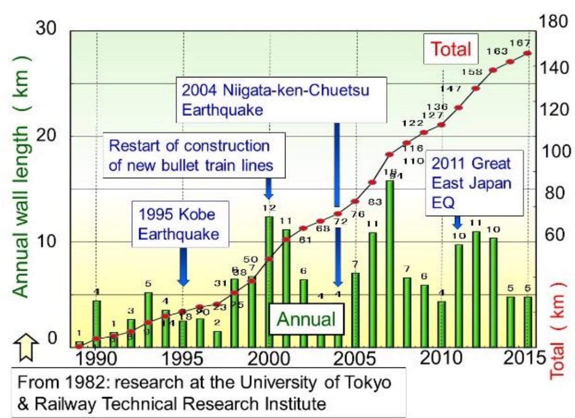


Figure 4. Annual and total length of GRS structures (as of June 2016)

2. SEISMIC STABILITY OF GRS STRUCTURES

2.1 Retaining walls

In order to establish practical design procedures to evaluate seismic stability of retaining walls(RWs) against high seismic loads, a series of shaking table tests were conducted on RW models consisting of six different types (Height: 50-53cm). Seismic loads were applied by shaking the sand container horizontally with an irregular acceleration which was recorded as N-S component during the 1995 Hyogoken- Nanbu earthquake. Its amplitude and time scale were adjusted so that the base acceleration has a prescribed maximum amplitude with a predominant frequency of 5 Hz. Watanabe et al. (2003) summarized the details of the model and the similitude adopted in these tests.

Fig.5 shows the residual displacement of the wall and the residual deformation of the backfill, which were observed at the end of final shaking step. For all RWs, the major failure pattern of the walls was overturning, which was associated with bearing capacity failure for the cantilever, leaning, and gravity type RWs.

For the GRS RWs, no failure plane was observed at the bottom of the front wedge in the reinforced zone. The front wedge did not behave as rigid, but it suffered simple shear deformation along horizontal planes. This is because the resistance against the formation of failure plane penetrating though the reinforcement was larger than that against the simple shear deformation of the rein-forced zone. This simple shear deformation of the reinforced backfill should be considered to evaluate the residual displacement of the reinforced-soil RWs (see Fig.14 in the next chapter).

Fig.6 shows relationships between the seismic coefficient (k_h), and the horizontal displacement (d_{top}) at the end of each shaking step. The seismic coefficient (k_h) was defined as k_h=a_{max}/g, where a_{max} is the maximum base acceleration at the active state for each shaking step, and g is the gravitational acceleration.

In the early steps of irregular shaking tests (up to k_h value of about 0.5), the d_{top} value accumulated in a similar manner among different types of RWs. On the other hand, when the k_h value exceeded about 0.5, the rate of increase in the d_{top} value was larger for the three conventional type RWs than that for the GRS RWs. Further, though the total length of reinforcement of type2 was 80% as much as that of type 3, the seismic stability of them was on the same level. Such different extents of ductility in each type of RW agree with the damage observed after Hyogoken-Nanbu earthquake (1995). This is caused by the different resistance mechanism against the external forces acting on the wall such as inertia force and seismic earth pressure.

The conventional type RWs resist against the overturning by the reaction force from subsoil. On the other hand, the GRS RWs resist against the overturning moment by the tensile force in the reinforcements in each layer. Fig.7 shows the relationship between the reaction force from subsoil and the horizontal displacement of the wall d_{top} for gravity type RW. In the early shaking steps, the normal stress measured at the toe of the base footing increased rapidly (①～⑥ in Fig. 7). After attaining the peak state, the d_{top} value suddenly increased due to loss of bearing capacity near the toe of the base footing(⑦～⑨ in Fig. 7). This behavior caused large decrease in the resisting moment against overturning, which led to the low ductility of conventional type RWs. Fig.8 shows the relationship between the tensile force and the horizontal displacement of the wall d_{top}. For all types of reinforced-soil RWs, the tensile force increased with the d_{top} value, not showing such a sudden drop as observed in the reactions from subsoil for gravity type RW (Fig.7). This behavior explain the ductile behavior of GRS RWs.

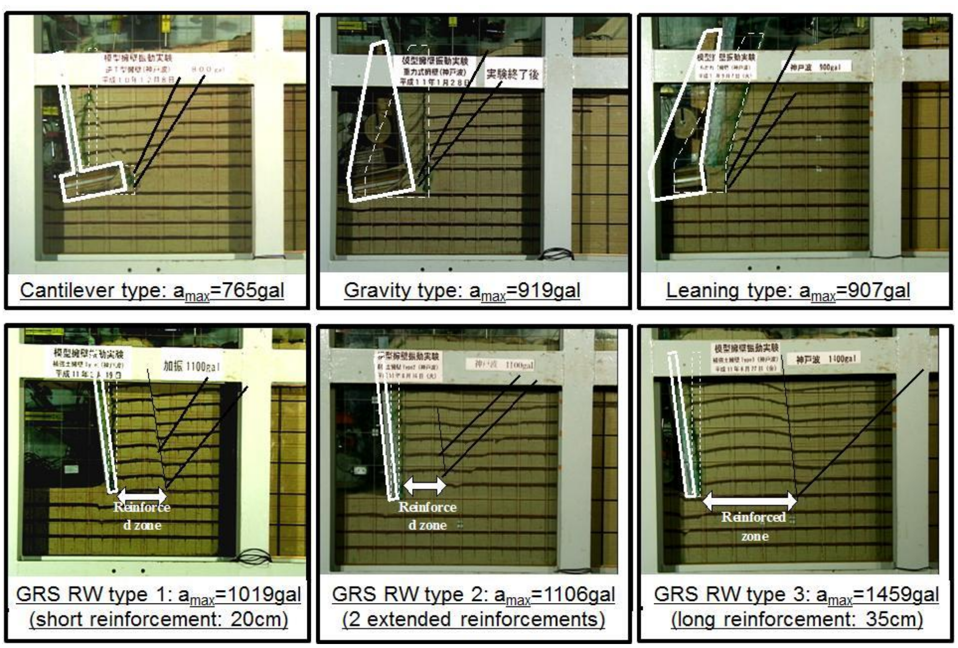


Figure 5. Residual displacement of the wall

Fig.9 shows the locations of failure plane and the reinforcements for GRS-RW type 2. The arrows indicate the end of longer reinforcement at the moment when the failure planes were formed. The two failure planes were formed almost simultaneously. The upper one developed from the back of the

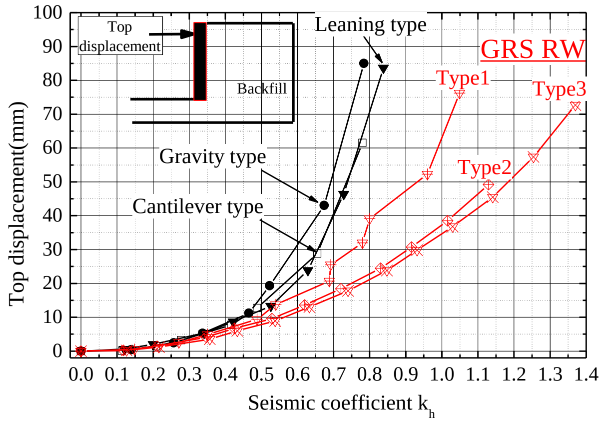


Figure 6. Accumulation of residual horizontal displacement near the top of the wall

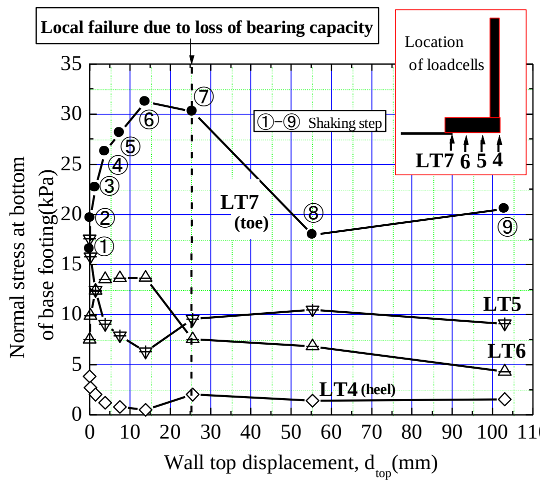


Figure 7. Measured reactions from subsoil for gravity type retaining wall

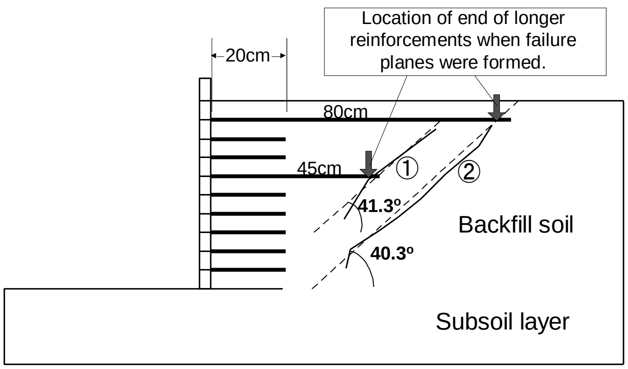


Figure 9. Locations of failure planes and longer reinforcement layers for GRS-RW (type 2)

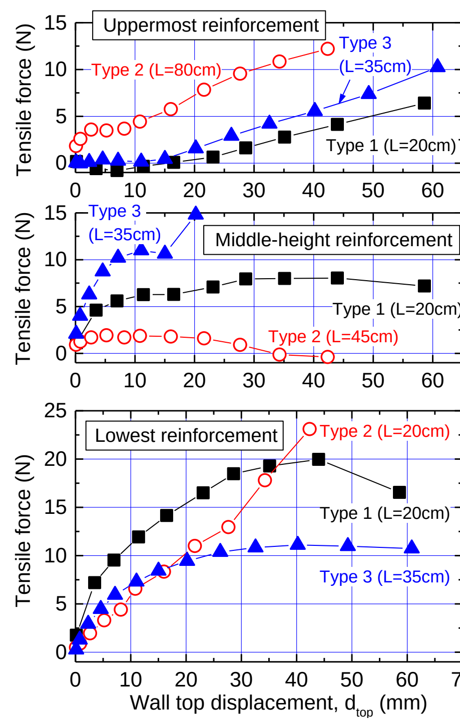


Figure 8. Tensile forces in reinforcement layers measured at a distance of 2.5cm from facing of GRS RWs

reinforced zone towards just beside the end of the extended reinforcement (45cm), stopping somewhere below the longest reinforcement. On the other hand, the lower failure plane was formed just beside the end of the longest reinforcement (80cm) and reached the surface of the backfill. This demonstrates that the reinforcement resisted against the formation of the failure plane, and the location of the failure plane was strongly governed by the existence of the extended reinforcement. Accordingly large tensile force was mobilized in the extended reinforcements as shown in Fig 8, which lead to the high ductility of GRS RW type2.

2.2 Bridge abutments

A great number of conventional type railway bridge abutments were seriously damaged with a large relative settlement between the bridge abutment and the backfill during the 1995 Hyogoken-Nambu Earthquake. Such relative settlement as above could endanger safe train operation, even when it is small, say several centimeters. In view of the above, a long-term research project started 1997 jointly at Railway Technical Research Institute and University of Tokyo aiming at developing new aseismic types of bridge abutment. The following new structural types have been proposed and studied as feasible ones (Fig.3):

- the backfill consists of a zone of geogrid-reinforced cement-mixed gravel immediately behind a full-height rigid facing structure supporting a bridge girder
- the ends of reinforcement layers are connected to the back of the facing that is constructed after the full-height back-fill is completed

In order to evaluate the seismic stability of GRS Bridge Abutment, a series of shaking table model tests were performed (Aoki et al. 2005). The abutment models investigated are all listed in Table 1 (conventional type; models 1, 2 & 3, GRS type; models 4 & 5). The facing structure was made of aluminum to have a height of 620 mm with a footing base having a width of 390mm (models 1 – 3), 290 mm (model 4) or 200 mm (model 5). The facing structure supported a model bridge girder with a mass of 200 kg through a hinged support (so the lateral seismic load acting to the girder was transmitted to the facing structure). Models 4 and 5 simulate the GRS types having a trapezoidal-shaped approach block made of cement-mixed gravel that are reinforced with geogrid reinforcement layers connected to the back face of the facing directly supporting a bridge girder.

Table 1. The Type of bridge abutment models

	Approach Block	Reinforcment	Input motion	Width of footing
Model 1 (Conventional)	sand (Dr=75%)	No	Sinusoidal & Kobe wave	390mm
Model 2 (Conventional)	dry gravel (ρ_d=1.9g/cm^3)	No	Sinusoidal Wave	390mm
Model 3 (Conventional)	cement-mixed soil	No	Sinusoidal Wave	390mm
Model 4 (GRS type)	cement-mixed soil	Yes	Kobe wave & Sinusoidal	290mm
Model 5 (GRS type)	cement-mixed soil	Yes	Kobe wave & Sinusoidal	200mm

Fig 10 shows the deformed models after the respective test which was observed through the transparent side wall of the shaking table. The following trends of behavior can be seen:

1. The deformation of model 1 became very large, showing ultimate failure with a well-developed single failure plane fully extending in the backfill, when a_{max} was 450 gals.
2. Model 2 exhibited brittle failure when a_{max} was 450 gals, where the deformation of the approach block of gravel became very large, in particular at the upper part.
3. The deformation of model 3, in particular the settlement at the crest of the approach block, was noticeably smaller than that of model 2. Despite the above, when a_{max} became 500 gals, the facing started separating from the approach block as a result of a high dynamic response, because of no connection between them.

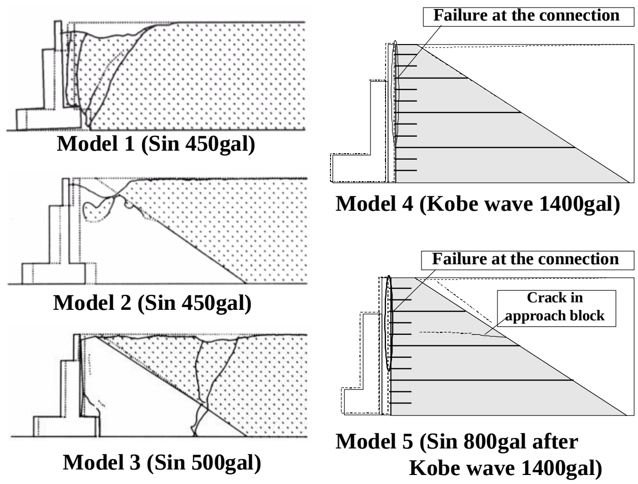


Figure 10. The deformation of the models after shaking

These results shown above indicate that the seismic stability of these conventional types of abutment could be insufficient when subjected to high seismic load, while the seismic stability of abutment can be increased by the following three measures:

1) Constructing an approach block using a stiffer and stronger material such as cement-mixed soil can substantially reduce the settlement of backfill immediately behind the facing structure supporting a bridge girder.
2) A high integrity of the approach block can be ensured by arranging horizontal reinforcement layers preventing the development of cracks in the zones where the tensile stress may exceed the tensile strength of cement-mixed soil, despite that an increase in the shear strength by using reinforcement layers of cement-mixed soil before the appearance of cracks cannot be expected.
3) The ends of reinforcement layers should be connected to the back of the facing structure directly supporting a bridge girder to restrain a relative settlement between them and to ensure a high integrity of the whole abutment structure.

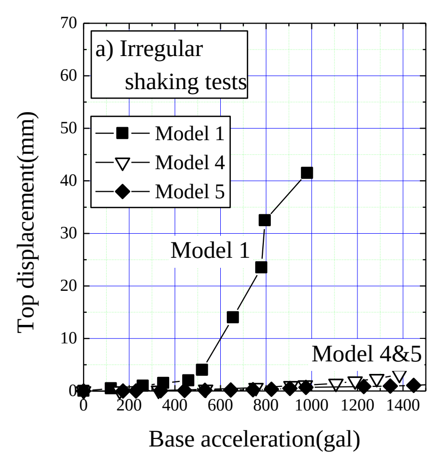


Figure 11. The residual displacement after each shaking step

Based on the above, eleven layers of horizontal reinforcements were placed inside the approach block of cement-mixed soil of models 4 and 5 with the ends connected by soldering to the back face of the facing structure.

Fig. 11 shows the relationships between the maximum and residual displacement at the top of the facing and the a_{max} value for models 4 and 5, subjected to irregular input motions, together with the relationship between the residual displacement of the facing and the a_{max} value for model 1. This results indicates that GRS types (Models 4 and 5) were much more dynamically stable than model 1 (i.e., the most conventional type abutment).

Fig. 12 shows the time histories of dynamic components of dynamic earth pressure, the tensile force in reinforcement and the dynamic reaction force acting on the base of the footing together with the displacement at the top of the facing and the input acceleration at one shaking stage using irregular waves with a_{max}= 539 gals on model 4. The following trends of behavior may be seen from this figure:

1) Under dynamically active condition where the dynamic component of the displacement of the facing was directing outwards, the resisting components (i.e., the reaction force near the toe of footing and the reinforcement tensile force) increased. At this moment (as denoted A in Fig. 12), the contact force near the heel of the footing decreased, indicating overturning displacements of the facing structure.
2) Importantly, under this active condition, the dynamic component of earth pressure decreased, showing that the facing structure was less stable than the backfill including the approach block. On the other hand, the maximum earth pressure in each cycle of dynamic loading was attained under dynamically passive condition (Point B in Fig. 12).

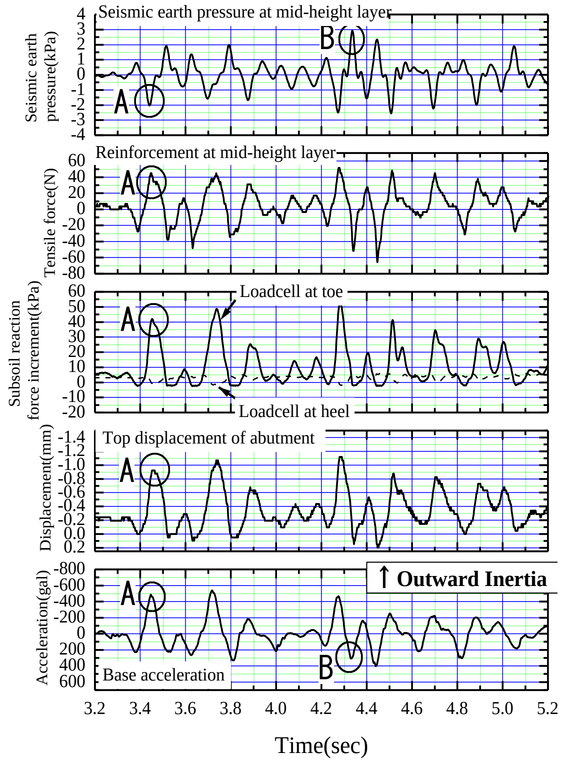


Figure 12. Typical time history of external forces for GRS bridge abutment Model 4 (Irregular shaking, a_{max}=539gal)

These trends of earth pressure are opposite to those assumed in the design of conventional retaining walls, in which the dynamic active earth pressure is considered to destabilize the retaining structure under dynamically active condition. Another important implication of this fact is that a high connection strength between the facing and the reinforcement is essential for a high seismic stability of this type of bridge abutment.

3. DESIGN STANDARD FOR JAPANESE RAILWAY STRUCTURES

Several types of earth retaining structures including conventional structures as well as GRS structure are widely used for the railway in Japan (Fig.13).

Conventional type retaining walls and bridge abutments are categorized as “earth pressure-resisting structures”. On the other hand, GRS structures such as GRS RW and GRS bridge abutment was categorized as one of “earth structure”. This is because the reinforced-earth construction method (including GRS method), originally applied to high-grade earth structures which allow little deformation against severe earthquakes. For such background and process of new technology development, the design method for earth retaining structures used in railways therefore appeared in different design standards for these reasons.

For the general design procedure, first the choice of a conventional retaining structure or GRS structures is

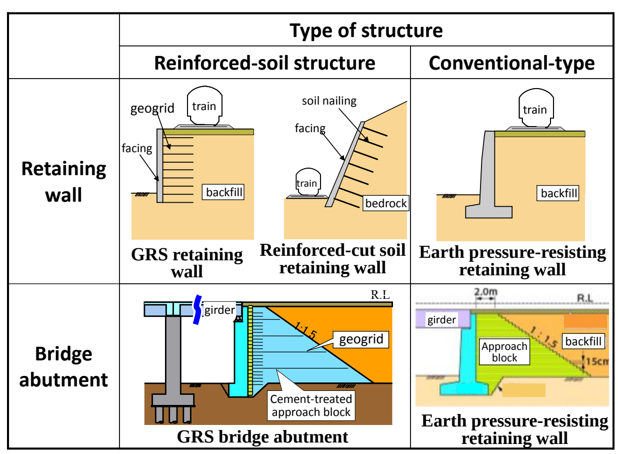


Figure 13 Classification of earth retaining structures covered under the new soil retaining structure standard (R.T.R.I, 2012)

made considering the importance of the intended structure, its required performance and the construction site conditions. However, it was difficult to compare such performances between conventional type and GRS type, since design procedure of them are described in different standard.

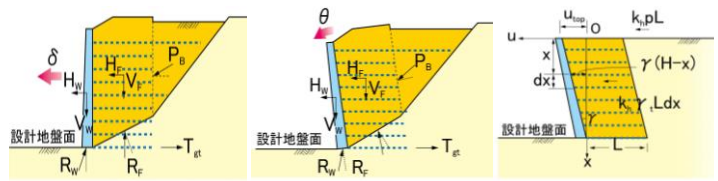
Consequently, suggestions have been made indicating that it is necessary to cover these structures in the same design standard, which offers a means to verify their performance with an equivalent index.

Considering above suggestion and objective, the Railway Technical Research Institute (R.T.R.I), under the guidance of Ministry of Land, Infrastructure and Transportation, revised the earth retaining structure standard on 2012, which now corresponds to a performance based design method. Structures covered under this new standard are shown in Fig.13. The revised standard now offers a harmonized method covering both conventional retaining structures and reinforced-soil structures.

By following the revised design standard, it become possible for the designer to select the conventional type or GRS type by comparing their performances with equivalent index, such as stability against traffic load, residual displacement caused by sever earthquake (such as Level 2 earthquake in Japanese design code) and so on (Table 2, Figure 14).

Table 2 Definition of deformation level and respective settlement for slab track and ballast track (RTRI, 2012)

Deformation level		Settlement		
		Slab track	Ballast track	
			Embankment/ Backfill of retaining wall	Transition zone (backfill of bridge abutment, box culvert)
1	Almost no deformation (functionality can be used without repair.)	No settlement / No residual displacement		
2	Some deformation (functionality can be restored in a short time)	Less than 5cm	Less than 20cm	Relative settlement less than 10cm
3	Large residual deformation (functionality can be restored by partial rebuilding)	More than 5cm, less than 15cm	More than 20cm, less than 50cm	Relative settlement more than 10cm , less than 20cm
4	Extremely large residual deformation (functionality cannot be restored without a total rebuild)	More than 15cm	More than 50cm	Relative settlement more than 20cm



(a)Sliding deformation mode (b) Overturning deformation mode (c) Shear deformation mode

Figure 14. Deformation modes considered in the seismic design of GRS RW (RTRI, 2012)

4. GRS EMBANKMENT RESISTANT TO SEVERE EARTHQUAKES AND PROLONGED OVERFLOWS CAUSED BY TSUNAMI

4.1 Background

Railway embankments sustained extensive damages from the tsunami triggered by the 2011 off the Pacific coast of Tohoku earthquake and the operations of railway lines were suspended for an extended period of time.

Several studies have been conducted to enhance the earthquake resistance of railway embankments, however, there have been few studies related to enhancing the tsunami resistance of railway embankments, and there is a need for an optimum restoration method for railway embankments vulnerable to tsunamis.

Railway embankments in coastal regions are generally constructed on the inner side of coastal levees. When a large tsunami flows over a coastal levee, the railway and road embankment structures are often expected to become the secondary barriers (multiple protection) for reducing damages at inland area.

The Railway Technical Research Institute conducted an analysis based on onsite surveys as well as wave model experiments and concluded that damages were primarily caused in the sequence described below (Fig. 15). Based on this analysis, detailed examination on railway embankment structures that can withstand a tsunami was conducted.

1. The main structure and the protective surface (embankment body) of the railway embankment sustains damages from an earthquake prior to the onset of a tsunami.
2. The tsunami occurs while the structure is in this damaged condition and the embankment is

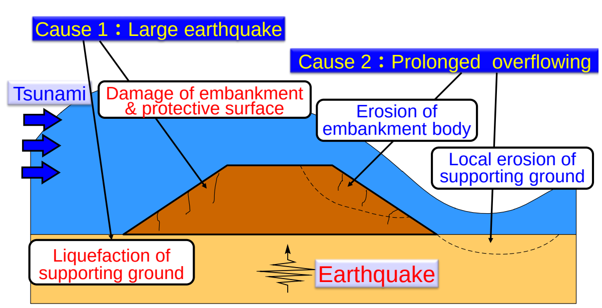


Figure 15. Main causes of damage of railway embankment

eroded from the prolonged tsunami overflows as well as the uplift pressures acting on the protective surface.
3. The water overflowing the embankment erodes the supporting ground around the lower section of the embankment (toe of the slope) on the inland side, which further destabilizes the embankment.

There have been several studies on the impact of wave pressure from a tsunami using large wavemakers. However, there have only been a small number of studies that focus on the effects of prolonged tsunami overflows and large-scale earthquakes that occur prior to the onset of a tsunami.

Furthermore, no method for evaluating the resistance characteristics of embankments considering the prolonged overflows has been established because there are many unknown factors regarding the phenomenon of the soil when eroded by the flow of water, like a tsunami overflow that occurs for an extended period of time.

Railway Technical Research Institute started the new study evaluating the effect of large-scale earthquakes that occur prior to the onset of a tsunami, the phenomenon of prolonged tsunami overflows. An experimental device capable of simultaneously reproducing both such phenomena was developed (Fig. 16). This device was used to systematically conduct experiments on models (one-tenth scale) of conventional embankments as well as GRS structures.

Fig. 17 shows the setup of the tsunami overflow experiment conducted on a model of a conventional embankment. A newly-established conventional type railway embankment on a conventional railway line was assumed. This was a model of a railway embankment with an earthquake-resistant design, using better banking materials, sufficient compaction control, and thickness control that comply with the Railway Design Standard for Earth Structures.

In the shaking table tests which simulated a large-scale earthquake (prior to reproducing the tsunami overflow), the residual displacement was about 0.2 to 0.3 mm in both horizontal and vertical directions, confirming that the embankment had sufficient earthquake resistance. When the tsunami overflow experiment was conducted on this railway embankment model (after the shaking table tests), the embankment eroded from the inland side, as shown in Fig. 18, and the erosion of the supporting ground spread to the banking embankment and about half of the banking embankment was eroded in only two minutes from the start of the overflow (equivalent to 6 minutes in actual scale).

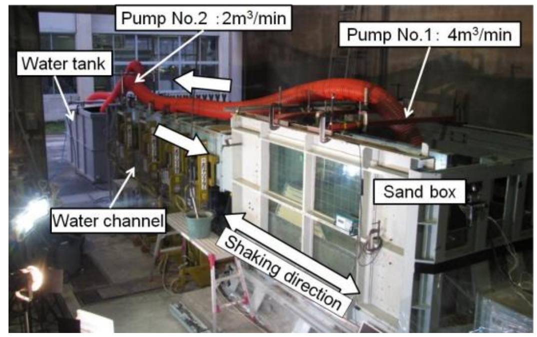


Figure 16. New apparatus for simulating large earthquake and prolonged overflowing

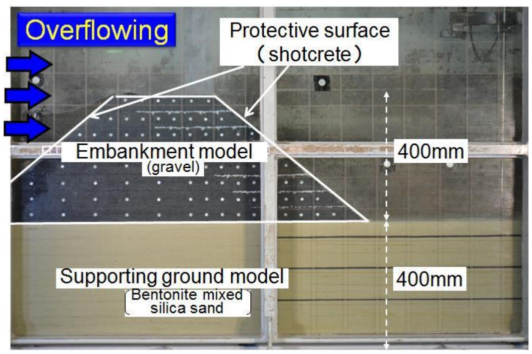


Figure 17. Model of conventional embankment (before test)

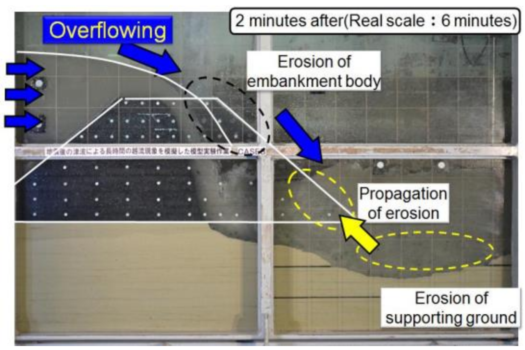


Figure 18. Overflowing model test on conventional railway embankment (2 minutes from the start of overflowing)

4.2 Stability of GRS embankments against prolonged overflow

An experiment focusing on the tsunami resistance of GRS embankments was performed considering the results obtained from the experiment conducted on the conventional embankment model. First, a GRS embankment model of 100 mm in height was prepared inside a small channel and a model experiment was systematically conducted by changing the installation methods of the reinforcement materials. The following conclusions were made by the preliminary experiments: (1) the arrangement of geotextiles inside the embankment drastically improved the erosion resistance of the embankment; (2) the tsunami resistance of the embankment was drastically improved when geotextiles were arranged on all layers (installed entirely from the left to the right edge of the embankment) and turned over on the sandbags at the edges of the embankment; and, (3) the levee body can be potentially unstable if the supporting ground is eroded, even if the embankment is stable.

The reason for (1) was the increased erosion resistance of the embankment against the dragging force caused by the high-speed overflowing. The erosion resistance was not mobilized only by the high tensile strength of geotextile but also by the lattice or grid patterns of geotextile which reduces the effect of tractive force to the soil beneath the geotextile.

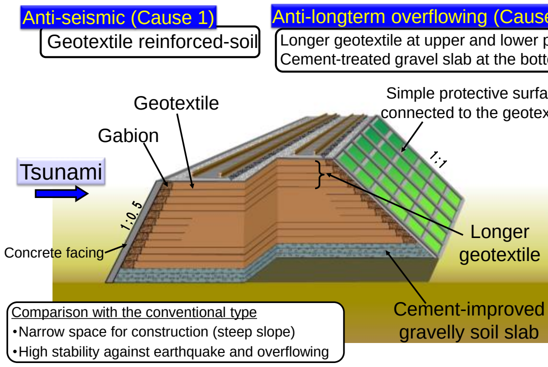


Figure 19. Suggestion of GRS embankment

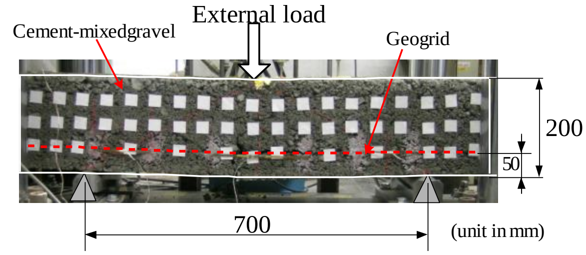


Figure 20. Bending load test of cement-mixed gravelly soil slab with geogrid

Figure 21. Application of cement-mixed gravelly soil slab to the construction of railway embankment on the soft ground

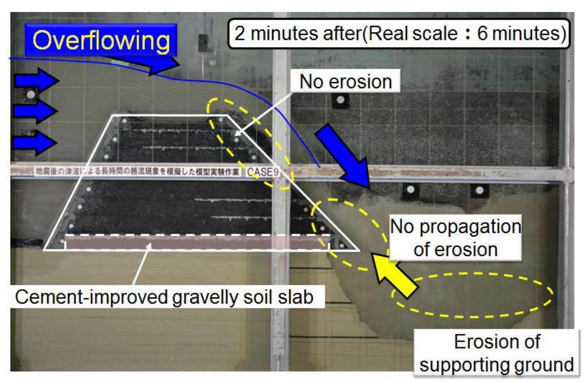


Figure 22. Overflowing model test on GRS embankment (2 minutes from the start of overflowing)

This confirmed that the GRS structure widely used after the 1995 Hyogoken-Nambu Earthquake for its earthquake-resistant properties is also more tsunami resistant than conventional embankments.

Based on the experimental results, a new GRS embankment (Fig.19, hereinafter "proposed structure") was proposed that has sufficient stability against large earthquake and prolonged tsunami overflows.

The required performance is primarily met by using a GRS embankment to sustain large-scale earthquakes. The proposed structure is an improvement on the conventional GRS embankment. Long strips of geotextile are arranged in the upper and lower layers of the embankment where is readily eroded by prolonged tsunami overflows and are subsequently wrapped onto the sandbags located at the edges of the embankment. Furthermore, the outflow of the protective surface due to uplift pressures (negative pressure) during prolonged tsunami overflows can be prevented by integrating the protective surface with the geotextile.

Additionally, the bottom layer of the embankment is constructed by the "cement-mixed gravelly soil slab" which is made of cement-mixed gravelly soil and geogrid. The cement-mixed gravelly soil is same material used for the backfill immediately behind the GRS bridge abutment (approach block, Fig.3), mixing small amount of cement (usually about 50kg/m^3) with gravelly soil. This is a composite material of cement-mixed gravelly soil (compressive member) and geogrid (tension member). Watanabe et al. (2011) shows this high bending deformation characteristic of this material by bending load tests (Fig.21) and these composite material have applied to actual railway project in Japan, where the embankment (maximum height; 3.5m, length: 110m) was constructed on the cohesive soft ground. Since this composite material can be constructed much easier and faster than reinforced concrete, large cost reduction could be realized by this proposed method. The ratio of ground improvement was decreased almost by half.

Applying this cement-treated gravelly soil slabs to the bottom layer of the embankment, as shown in Figure 19, can prevent deformation and damage to the embankment even when the structure becomes unstable because of the erosion of the supporting ground at the edge (toe) of the embankment during prolonged tsunami overflows.

Although conventional type embankment structures was eroded by tsunami overflows even after exhibiting sufficient earthquake resistance (Fig.18), the proposed embankment structure has been demonstrated to be less likely to erode under tsunami overflows occurring over longer durations as shown in Fig.22, and the embankment body is also less likely to become unstable when the supporting ground is eroded.

While this study focused on tsunami overflows that occur after major earthquakes, the resisting performance of earth structures against prolonged overflows is required not only when a tsunami occurs, but also when a severe rainstorm occurs. For example, during the severe rainstorm that occurred in the Kyushu region in 2012, the railway embankments constructed in the mountainous areas along the Hohi Line sustained extensive damages from erosions caused by overflow. The geotextile reinforced embankments that were installed in the vicinity of the embankments sustained only minimal damages. In such cases, emphasis has always been placed on water-drainage measures and protection of slopes for designing embankments against rainfalls. However, there have been incidents involving damages from severe rainstorms due to erosions from overflows and outflows of railway embankments constructed in water catchments of mountainous areas and river basins in recent years. The GRS structure proposed in this paper is useful in the construction of embankments in such locations as well.

6. CONCLUSIONS

In this paper, the history of the application of GRS structures to the Japanese Railway is first briefly introduced and the experimental studies on seismic stability of GRS structures are overviewed. These experiments revealed that firm connection of geogrid to the full-height rigid facing is the essential points for the stability and ductile behavior of GRS structures against severe earthquake.

Based on these test results and field observation, the design procedure of these GRS structures together with conventional RW and bridge abutment were established and published as “Design Standards for Railway Structures and Commentary (Earth Retaining Structure)” on 2012 which follows the concept of performance-based design. By following the revised design standard, it become possible for the designer to select the conventional type or GRS type by comparing their performances with equivalent index, such as stability against traffic load, residual displacement caused by sever earthquake.

Finally, the recent research activities which developed the new GRS embankment having sufficient earthquake resistance and resilience against prolonged overflowing caused by tsunamis was introduced. While this study focused on tsunami overflows that occur following major earthquakes, the proposed GRS embankment can be applied to the water catchments areas or river basins where the embankment of dike are potentially subjected to prolonged overflowing caused by heavy rainfall.

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