Recent research and practice of GRS integral bridges for railways in Japan

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ABSTRACT

Geosynthetic-reinforced soil (GRS) integral bridge was developed to overcome several inherent serious problems with conventional type bridges typically comprising a simple-supported girder (or girders), RC abutments and approaches of unreinforced backfill: i.e. high construction/maintenance cost while bumps immediately behind the abutments; a low stability of the bearings and backfill against seismic and tsunami loads; massive abutment structures; needs for piles etc. A GRS integral bridge is constructed by constructing firstly a pair of GRS walls and an intermediate pier (or piers) if necessary; secondly lightly steel-reinforced full-height-rigid (FHR) facings by casting-in-place concrete on the wall face wrapped-around with the geogrid reinforcement; and finally a continuous girder with both ends integrated to the top of the FHR facings. The background of the development of GRS integral bridge is explained. The first four case histories, completed in 2012 and 2014, are reported.

Keywords: earthquake, full-height rigid facing, geosynthetic-reinforced soil, integral bridge, maintenance, tsunami

1. INTRODUCTION

Geosynthetic-reinforced soil (GRS) retaining wall (RW) with staged-constructed full-height rigid (FHR) facing (Fig. 1) was developed in the mid-1980s (Tatsuoka et al. 1997). In 1990s, extending this GRS RW technology, GRS bridge abutment, placing a girder via a hinged bearing on the top of the FHR facing of a GRS RW, was developed (Aoki et al. 2005; Tatsuoka et al. 2005). In 2000s, GRS integral bridge, integrating without using bearings both ends of a continuous girder to the top of the FHR facings of a pair of GRS RW, was developed (Tatsuoka et al. 2009). GRS integral bridge is now one of the standard bridge types for railways. These types of GRS structure have been constructed for a total wall length of about 160 km as of June 2014 (Fig. 2b), mainly for railways including high-speed train lines (i.e. Shinkansen in Japanese). Many of them were constructed in place of gentle-sloped embankment, cantilever RC RWs, conventional type bridge abutments, RC viaducts and conventional type bridges, typically for Hokkaido Shinkansen (see Fig. 2a for its location; Yonezawa et al. 2014).

The reason for the above is high cost-effectiveness with high performance. With the GRS structures constructed at more than 1,050 sites (Fig. 2a), any problem has not taken place during construction and long-term service and also by heavy rainfall and severe earthquakes despite a wide variety of topological, geotechnical, structural and loading conditions.

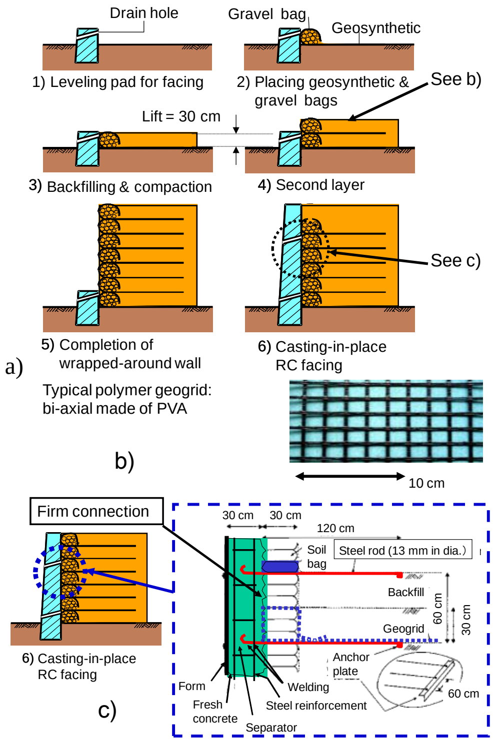

Fig. 1. GRW RW with FHR facing: a) staged construction; b) a typical geogrid; and c) details of facing construction at stage 6.

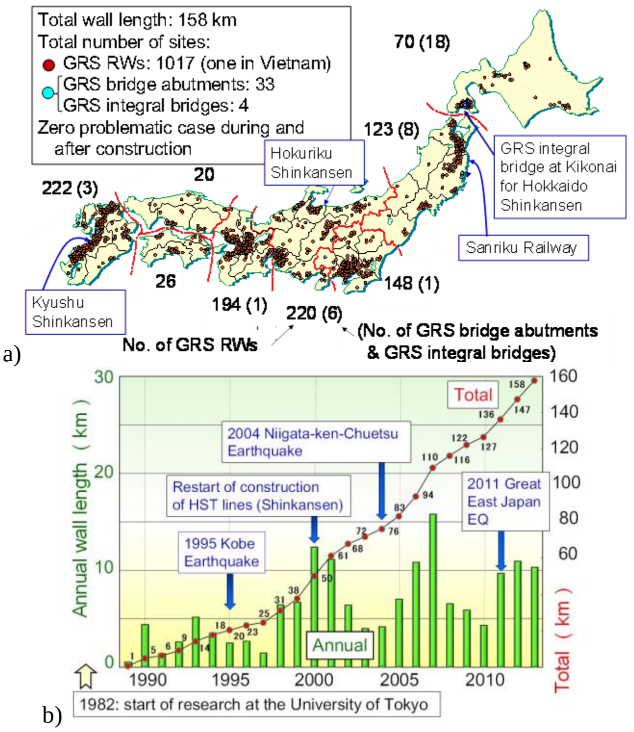

Fig. 2. a) Locations; and b) history of GRS RWs with a staged-constructed FHR facing, including GRS abutments and GRS integral bridges (as of June 2014; Tatsuoka et al. 2014).

During the 1995 Great Kobe and the 2011 Great East Japan Earthquakes, gentle-sloped embankments, conventional type RWs, other types of reinforced soil RW, RC viaducts and conventional type bridge abutments were seriously damaged or fully collapsed at many places, whereas a number of GRS RWs of this type performed very well (Tatsuoka et al. 1997, 1998, 2014). These experiences showed that the seismic stability, as well as the static stability, of this type of GRS RW is very high. High performance against floods, heavy rains, tsunamis is also confirmed.

Despite the above, the construction and maintenance cost (i.e. the life cycle cost) of these GRS structures is much lower than conventional type soil structures, in particular those supported with piles. In this respect, ballast-less RC slab track was introduced in 1970s to reduce the maintenance work of the tracks of Shinkansen. However, their use was initially limited to tracks on RC viaducts and bridges. For Hokuriku Shinkansen (opened 1997) and subsequent ones, RC slab tracks were constructed also on embankment retained by GRS RWs. It has been confirmed that the long-term residual settlement of RC slab tracks on GRS structures are negligible. Thus, the life cycle cost of GRS structures supporting RC slab tracks is much lower than not only RC viaducts but also unreinforced embankment retained by conventional type RWs supporting ballast tracks. Besides, GRS bridge abutments and GRS integral bridges exhibit negligible bumps by long-term train loads and seismic loads, because the geogrid layers reinforcing the approach fill are firmly connected to the back of the FHR facing.

2 FEATURES OF GRS RW WITH FHR FACING

The characteristic features of the GRS RW system (Fig. 1) summarized below are the basis for the development of GRS integral bridge.

2.1 Structural features

Reinforced soil RWs should be stable for "global failure along a global failure plane" and "local failure of backfill immediately behind the wall face" among other local failure modes. The minimum lateral confining pressure required for this type of local stability is the active earth pressure in unreinforced backfill. If the wall face is too flexible or if the connection strength between rigid facing and reinforcement is too low, the available earth pressure at the wall face becomes lower than the one required for this type of local stability. Then, the tensile forces in the reinforcement become very low at low levels of the wall, where the width of the active zone bound by the global failure plane starting from the facing bottom is very small (Fig. 3a). This results in low confining pressure thus low stiffness/strength in the active zone, leading to large wall deformation. With this type of GRS RW having FHR facing with high connection strength (Fig. 1), on the other hand, the available earth pressure at the back of the facing is high enough, thus the tensile forces in the reinforcement can become high enough (Fig. 3b). This results in high confining pressure thus high stiffness/strength in the active zone, preventing the local failure in the backfill and leading to a high global stability and small wall deformation.

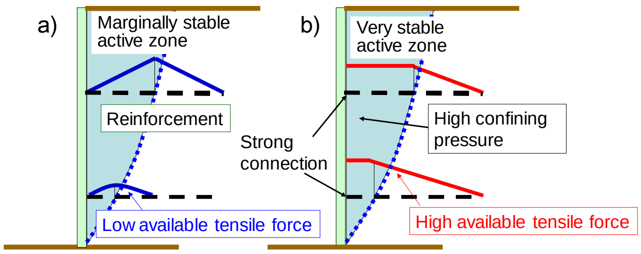

Fig. 3. Available reinforcement tensile forces when the connection strength is: a) zero; and b) high (Tatsuoka 1992).

A conventional type RW is a cantilever structure that resists earth pressure. Therefore, large internal force is mobilized in the facing while large overturning moment and lateral thrust force develops at the facing base. Then, a massive facing structure, usually supported by piles, becomes necessary. These disadvantages become more serious at an increasing rate with wall height. On the other hand, the FHR facing of this GRS RW (Fig. 1) is a continuous beam supported by many geogrid layers with a small vertical spacing (i.e. 30 cm). Therefore, only small force is mobilised in the facing even when the earth pressure is very high. Hence, the structure of the facing becomes much simpler using a much less amount of steel reinforcement than cantilever RC RWs. Besides, as only small overturning moment and lateral thrust force is required at the facing base to maintain the global wall stability, piles are not used.

When concentrated load is applied to the top of the facing or near the wall face on the crest of the backfill, high integrity of the active zone becomes particularly important for high local and global stabilities of the wall. This requirement can be satisfied by using a FHR facing connected to the reinforcement layers. In this case, concentrated load is transmitted to the FHR facing then to all reinforcement layers, thereby resisted by the whole wall. Therefore, FHR facing is often used as the foundation for electric poles and noise barrier walls. GRS bridge abutment and GRS integral bridge fully take advantage of these features of FHR facing.

2.2 Staged construction of FHR facing

After potential deformation of subsoil and backfill by the weight of the backfill has taken place sufficiently, at stage 6 in Fig. 1a, FHR facing is constructed by casting-in-place fresh concrete in the space between the geogrid-wrapped-around wall face and the concrete form temporally supported with steel rods anchored in the backfill (Fig. 1c). In this way, the facing/geogrid connection is not damaged by differential settlement between the facing and the backfill that may take place if the FHR facing is constructed prior to, or simultaneously with, the construction of the backfill. Besides, with conventional type RC RWs, concrete forms and their propping are necessary on both sides of the facing and they become more costly at an increasing rate with wall height. With this type of GRS RW, only the outside concrete form is necessary while not occupying the space in front of the wall.

Fresh concrete enters the inside of the gravel-filled bags through the aperture of the geogrid reinforcement wrapping-round the gravel bags and the geogrid of the gravel bags (Fig. 1c). Then, the facing is eventually firmly connected to the reinforcement layers. As the front end of the geogrid reinforcement is buried in the facing, the geogrid should have very high resistance against high alkali environment and high adhesiveness with concrete. So, bi-axial geogrid made of polyvinyl alcohol (PVA) (Fig. 1b) is usually used.

With help of gravel bags placed at the shoulder of each soil layer, the backfill immediately behind the wall face can be compacted efficiently. Before the construction of FHR facing, the gravel bags function as a temporary but stable facing unit resisting earth pressure generated by backfill compaction and the weight of overlying backfill. With completed GRS RWs, the gravel bags function as a drain and a buffer protecting the facing/geogrid connection against potential relative vertical and/or horizontal displacements that may take place between them during a full life period.

3 GRS BRIDGE ABUTMENT

With conventional type bridges, intolerable bumps often develop immediately behind abutments gradually by depression of unreinforced backfill during long-term service and suddenly by seismic loads. The bump increases if the abutment and/or the wing RWs is displaced. The other problems include needs for a massive RC abutment structure and piles. To alleviate these problems, a new type bridge abutment, called GRS bridge abutment, was developed (Fig. 4) (Aoki et al. 2005; Tatsuoka et al. 2005): i.e. the girder is placed via a hinged bearing on the top of the FHR facing of a GRS RW. For railways, to ensure essentially no bump and a high stability, an approach block is usually constructed immediately behind the facing by well compacting lightly cement-mixed well-graded gravelly soil that is reinforced with geogrid layers connected to the facing. GRS abutment exhibits much higher long-term and seismic stabilities than the conventional type, while it is much less costly due to more slender RC facing and usually no use of piles. The first GRS abutment was completed in 2003 for Kyushu Shinkansen (see Fig. 2a for the location). For Hokkaido Shinkansen, in total 29 GRS abutments were constructed fully in place of conventional type abutments. Until today, more than 50 GRS abutments were constructed for railways.

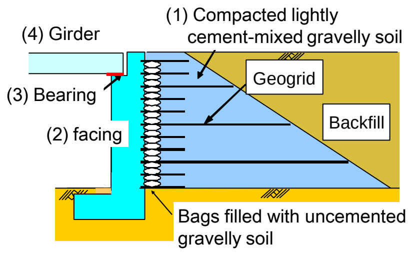

Fig. 4. GRS bridge abutment (the numbers denote the construction sequence).

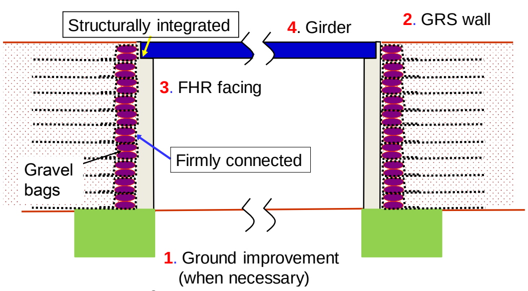

Fig. 5. Structure of GRS integral bridge (the numbers denote the construction sequence).

4 GRS INTEGRAL BRIDGE

GRS abutment still has a serious problem of high life cycle cost for the bearings and a low seismic stability of the girder at the bearings. To alleviate these problems, GRS integral bridge was developed (Fig. 5) (Tatsuoka et al. 2009; 2014); i.e. both ends of a continuous girder is structurally integrated to the top of the FHR facing of a pair of GRS RWs. GRS integral bridge is more cost-effective exhibiting higher performance than GRS abutment. Firstly, the construction and maintenance of the bearings becomes unnecessary. Secondly, the girder becomes shorter than the conventional simple-supported girder. Besides, the girder becomes more slender due to a reduction of the maximum bending moment at the center of the girder by a factor of about 0.5, resulting from flexural resistance at the girder/facing connections. Thirdly, the stability against seismic loads and tsunami increases significantly due to increased structural integrity and reduced mass and thickness of the girder.

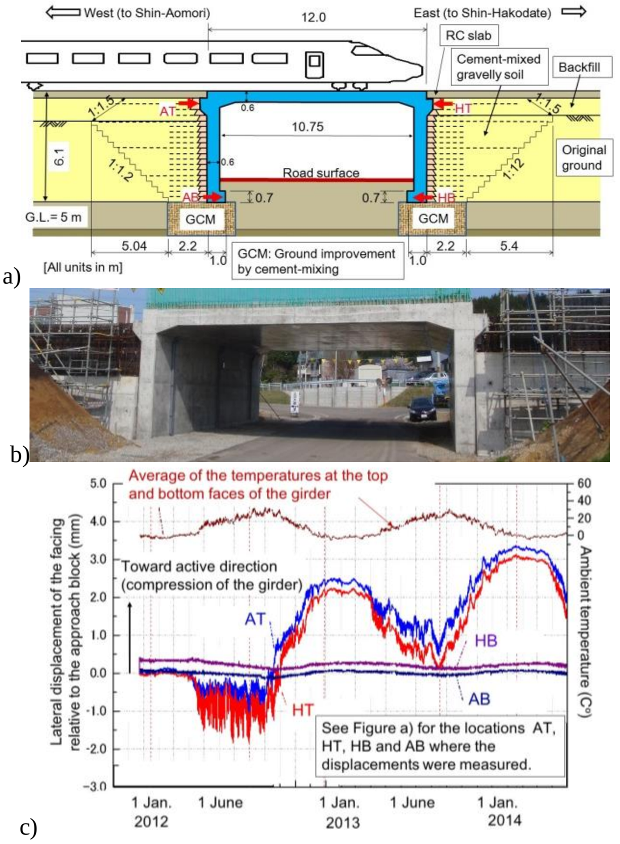

Fig. 6. GRS integral bridge at Kikonai: a) structure; b) nearly completed; and c) time histories of ambient temperature and horizontal displacements at the facing (Sasaki et al. 2014).

The first GRS integral bridge was constructed as an over-road bridge for Hokkaido Shinkansen (Fig. 6: see Fig. 2a for the location). The construction cost of this bridge is about a half of that of the equivalent conventional type bridge. Small- and full-scale model tests (Tatsuoka et al. 2009; Koda et al. 2013) and numerical analysis (Yazaki et al. 2013) showed a high stability against thermal deformation of the girder and severe seismic loads. To confirm the above, ambient temperature, strains in the geogrid and steel reinforcement, displacement and earth pressure have been observed at selected places (Yonezawa et al. 2014). Fig. 6c shows the time histories of ambient temperature and lateral displacements relative to the approach block at the top and bottom of both facings. The amplitude of the lateral displacement at the top of each facing is about 3 mm, about 0.05 % of the wall height, 6 m. Thus, the amplitude of the annual thermal girder length change is about 6 mm, about 0.05 % of the girder length, 12 m. By the thermal expansion of the girder in summer, the top of the facing is pushed towards the approach block and the geogrid tension decreases. By the thermal contraction of the girder in winter, the top of the facing is pull from the approach block and the geogrid tension increases. These responses are negligible at the bottom of the facing. These and other measurements showed that the bridge is not over-stressed at all. This behaviour can be attributed to that the bridge is structurally highly integrated.

The maximum active displacement relative to the approach block at the top of the facing in the second winter is slightly larger than the one in the first winter (Fig. 6c). This is not due to an increase in the maximum contraction of the girder but due to slight rotation of the approach block in the passive direction about the facing bottom caused by the delayed compression of the subsoil beneath the approach block by the weight of a thin top backfill. Although this backfilling should have been done before the construction of the facing and girder, it was performed at the final construction stage due to construction restraint. Although the displacement is very small and its effect is negligible, this is an incident from which we should learn a lesson.

During the 2011 Great East Japan Earthquake, the girders and/or the approach fill of more than 340 bridges for roads and railways near coasts were washed away by a great tsunami. Sanriku Railway, opened 1984, is running along the coastline (see Fig. 2a for the location). Although this railway was constructed at a relatively high elevation based on the previous tsunami disasters in 1896 and 1933, the tsunami this time was much higher than had been anticipated (i.e., the run-up height was 22 – 23 m at Shimanokoshi). The tunnels were inundated and the damage was very serious at many sites. Three simple-supported girder bridges in three narrow valleys between tunnels were washed away (see Figs. 7a, 7b, 8a, 9a & 9d). At these three sites, the track level was lowest (12.3 – 14.5 m) and the bridges were located closest to the coastal line along this railway, while there was no coastal dyke. These collapsed bridges were restored by constructing three GRS integral bridges as tsunami-resistant bridges. The railway was re-opened 6th April 2014, about three years after the earthquake.

The total length of the continuous girder of the GRS integral bridge at Haipe is 60 m (Fig. 7c), much longer than the one at Kikonai (Fig. 6). The central pier is designed to support only the vertical load. Fig. 7e shows the time histories of temperature and geogrid strains at four points immediately behind the facing at a level near the crest of the wall (see the figure inset in

Fig. 7f) of the north abutment for a period during construction in the year of 2013.

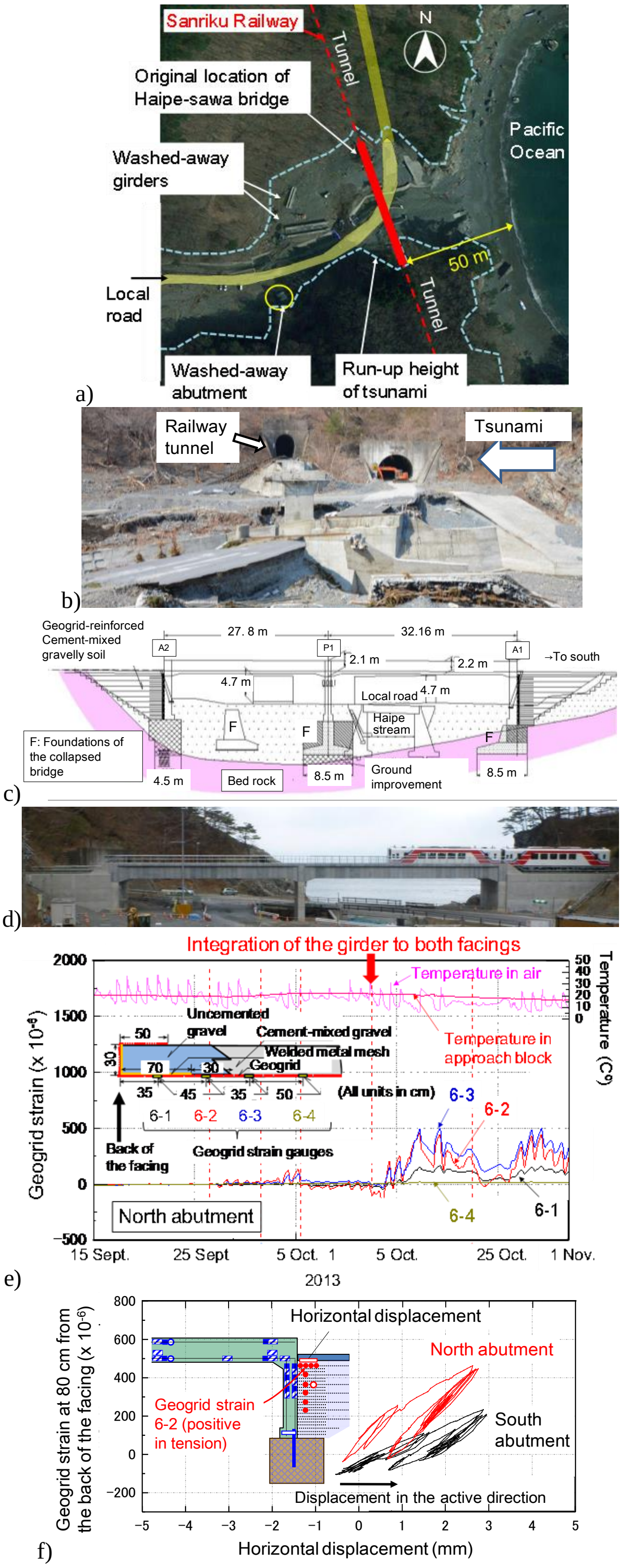

Fig. 7. Haipe-sawa bridge, Sanriku Railway: a) aerial photo immediately after collapse and b) seen from south (30 March 2011); c) GRS integral bridge seen from the inland: d) completed (6th April 2014); e) typical time histories of temperature and geogrid strains; and f) typical horizontal displacement vs. geogrid strain relations (Figs. e & f: Yamazaki et al. 2014).

In Fig. 7e, points 6-1 and 6-2 are located in the uncemented gravel while points 6-3 and 6-4 are in the cemented gravel of the approach block. The geogrid strains at points 6-1 and 6-4 are relatively small due to restraint by a welded metal mesh (used in place of gravel bag) and cement-mixed gravel, respectively. Upon the integration of the girder to the facings, the geogrid strains at point 6-2 and 6-3 started sensitively responding to ambient temperature changes, similarly as the GRS integral bridge at Kikonai. Fig. 7f shows the relationships between the horizontal displacement of the facing relative to the approach block and the geogrid strains at point 6-2 in both abutments. The amplitude of the length change of the girder during this observation was about 6 mm, while the amplitude during the first full year period was about 10 mm, 0.017 % of the girder length, 60 m. Prepared for relatively large yearly length changes with this long girder, the width of the uncemented gravel layer immediately behind the facing was made 100 cm, compared to 30 - 40 cm with the GRS integral bridge at Kikonai and ordinary GRS RWs (Fig. 1). The maximum geogrid tensile strain is about 0.05 % in Fig. 7e. The maximum strain during the first full year period was larger, about 0.14 %, which was still substantially lower than the design allowable value, 3 %. These measurements show that the bridge has been highly stable.

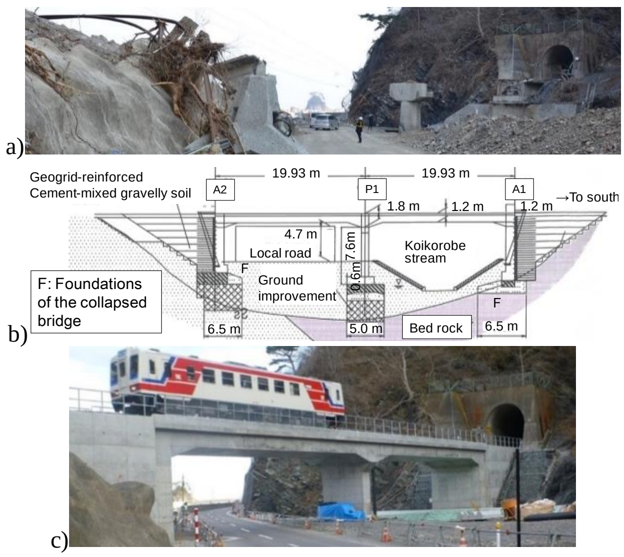

Fig. 8. Koikorobe-sawa bridge, Sanriku Railway: a) 30 March 2011; b) GRS integral bridge seen from the inland; and c) completed (6th April 2014).

Fig. 8 shows the damaged previous two-span simple-supported girder bridge at Koikorobe and another GRS integral bridge constructed at the site.

The RC viaduct at Shimanokoshi fully collapsed by tsunami (Fig. 9a). On the request of the residents at the site, geosynthetic-reinforced (GR) embankment was constructed as a tsunami barrier in place of RC viaduct (Figs. 9b & 9c). Both slopes of the GR embankment are covered with lightly steel-reinforced concrete facing firmly connected to the geogrid layers reinforcing the backfill. At this site, another GRS integral bridge was constructed (Figs. 9e & f) to restore the bridge that fully collapsed by tsunami (Fig. 9d). The GRS integral bridge is underlain by a backfill layer to reduce as much as possible the size of the opening.

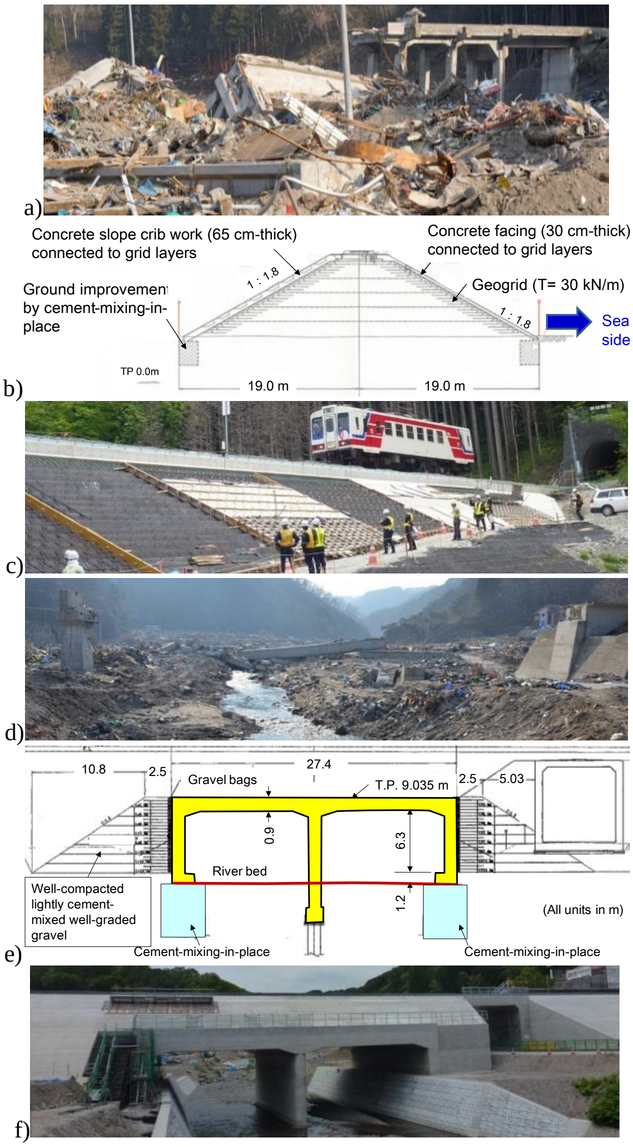

Fig. 9. Shimanokoshi, Sanriku Railway: a) RC viaduct collapsed by tsunami seen from the inland (30th March 2011); b) cross-section of GR embankment; c) GR embankment seen from the inland (20 May 2014); d) immediately after the earthquake seen from the seaside (30th March 2011); e) GRS integral bridge seen from the seaside; and f) completed (20th May 2014).

5 CONCLUSIONS

GRS integral bridge was developed by extending the technology of GRS RW with FHR facing. Compared with the conventional type bridge, GRS integral bridge is much more cost-effective and its performance is much higher with negligible bumps behind the facing and a high stability during long-term service and against severe earthquakes and tsunamis. These features can be attributed to the staged construction of FHR facing firmly connected to the geogrid reinforcement layers and structural integration of a continuous girder, facings and approach fills.

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