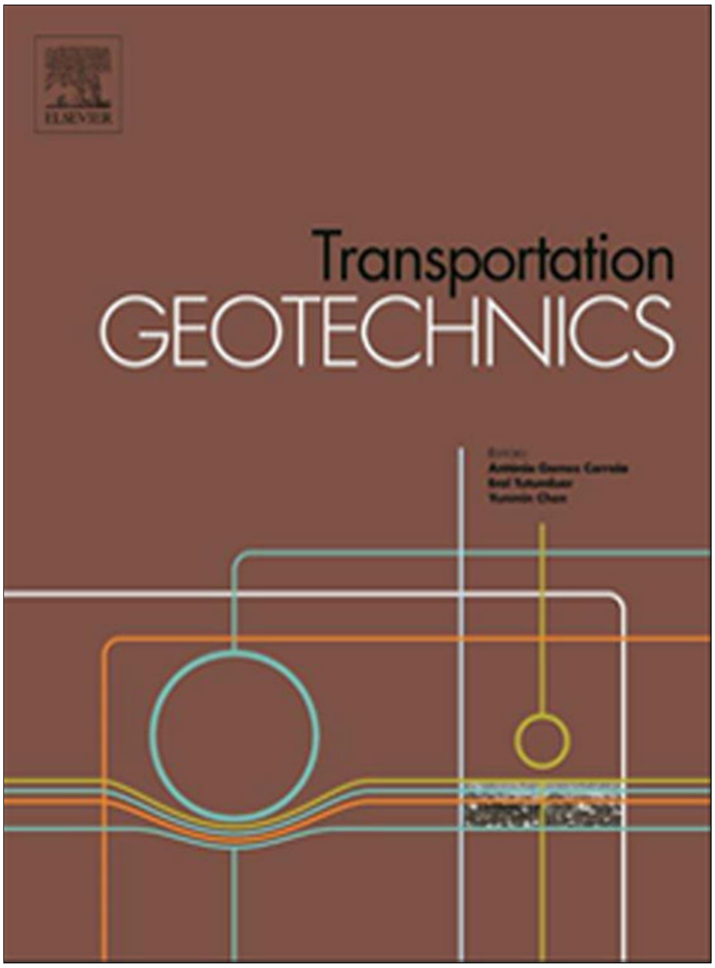
ELSEVIER
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GEOTECHNICS

Transportation Geotechnics 1 (2014) 3–20

Contents lists available at ScienceDirect

Transportation Geotechnics

journal homepage: www.elsevier.com/locate/trgeo

Design and construction of geosynthetic-reinforced soil structures for Hokkaido high-speed train line

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ARTICLE INFO

Article history:
Available online 8 January 2014

Keywords:
Box culvert
Bridge
Geosynthetic-reinforced soil
High-speed train
Retaining wall
Seismic design

ABSTRACT

The southern part between Shin-Aomori Station (at the north end of Main Island) and Shin-Hakodate Station (at the south end of Hokkaido Island) of a new high-speed train line called Hokkaido Shinkansen is nearly completed as of the end of 2013 and will be opened in 2015. In a range of 37.3 km at the south end of Hokkaido Island, a number of various types of geosynthetic-reinforced soil (GRS) structure were constructed: i.e., (1) GRS retaining walls (RWs) having full-height rigid facing for a length of about 3.5 km, having fully replaced the conventional type RWs; (2) in total 29 GRS bridge abutments, having fully replaced the conventional type bridge abutments; (3) a GRS integral bridge, the world-first one at Kikonai; (4) three GRS box culvert structures integrated to adjacent GRS RWs; and (5) nine GRS protection structures at the tunnel entrance. These GRS structures are those that have been constructed most densely ever for railways, which is definitely so for high speed trains. In this paper, the design and construction of these GRS structures is described while several lessons learned from this project are summarized.

Introduction

A new line of High Speed Train (HST), called Hokkaido Shinkansen, is an extension from the north end of one of the existing HST lines from Tokyo (Tohoku Shinkansen) (Fig. 1). The construction of the southern part between Shin-Aomori Station (at the north end of Main Island) and Shin-Hakodate Station (at the south end of Hokkaido Island) for a length of 149 km of Hokkaido Shinkansen was started in 2005 and will be completed in 2014. The construction of the northern part for a length of 211 km extending from Shih-Hakodate Station to the north until Sapporo City, the capital of Hokkaido Island, was started in 2012.

As shown in Fig. 2, at many sites in a range for a length of 37.6 km between Kikonai and Shin-Hakodate Stations at the south end of Hokkaido Island, the following various types of geosynthetic-reinforced soil (GRS) structure were constructed:

(1) GRS retaining walls (RWs) having staged-constructed full-height rigid (FHR) facing (at sites denoted by R in Fig. 2) for a total wall length of 3.5 km: in contrast, no conventional type cantilever RW and no gentle-sloped embankment was constructed.
(2) In total 29 GRS Bridge Abutments (denoted by A): the two ends of a bridge girder are placed on the top of the FHR facings of a pair of GRS RWs via a fixed and movable bearing or one end is placed on the top of the FHR facing of a single GRS RW via a fixed bearing with the other end of the girder being placed on the top of a pier via a movable bearing. The tallest abutment of this type is 13.4 m-high (as shown later in Fig. 11). In contrast, no conventional type bridge abutment was constructed.

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http://dx.doi.org/10.1016/j.trgeo.2013.12.001

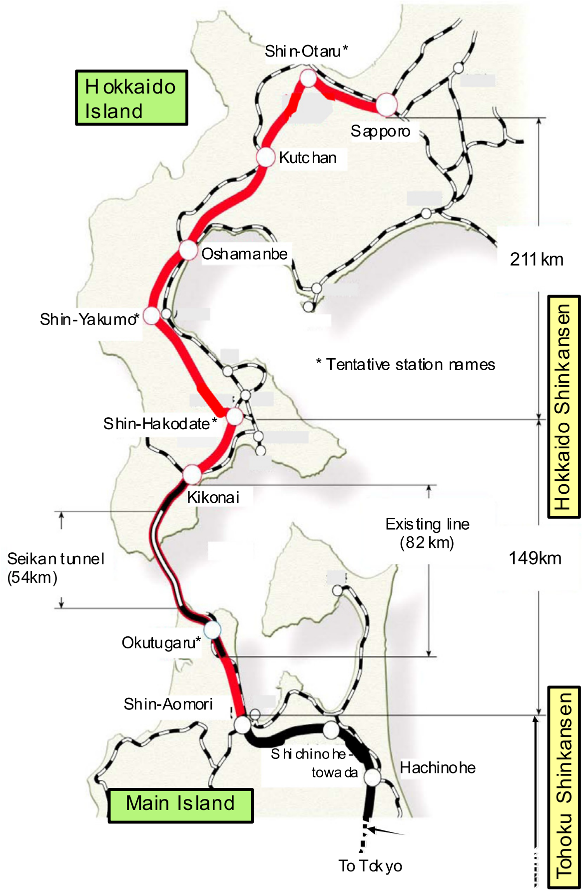

Fig. 1. Outline of Hokkaido Shinkansen.

(3) A GRS Integral Bridge at Kikonai as the world-first one (denoted by I): both ends of the girder are integrated without using any bearing to the top of the FHR facing of a pair of GRS RWs.

(4) Three GRS Box Culverts to accommodate local roads under-passing the railway (denoted by B): each of the RC box structures is integrated to adjacent GRS RWs at both sides.

(5) Eleven GRS Tunnel Entrance Protections (denoted by T): a GRS arch structure was constructed at the entrance (or exit) of several tunnels to stabilize the slope immediately above the tunnel entrance and to protect trains against falling rocks and sliding soil masses.

Importantly, these types of GRS structures were constructed fully in place of respective conventional type structures and most densely ever for railways, definitely so for HST lines. They were selected because of their very high cost-effectiveness: i.e., a lower construction and maintenance cost while a higher functionality including smaller residual deformation and a higher seismic-stability against severe earthquakes. In particular with GRS Bridge Abutments, GRS Integral Bridge and GRS Box Culverts, bumping by settlement in the backfill immediately behind the facing due to long-term train loads and seismic loads can be expected to be negligible, unlike the conventional type structures. In the following, the design and construction of these GRS structures is described, while several lessons learned from this project are summarized. Part of the content of this paper has been reported by Yonezawa et al. (2013).

GRS RW with FHR facing

Staged construction

As shown in Fig. 3a, after the deformation of the subsoil and the backfill by the construction of geogrid-reinforced backfill has taken place sufficiently, full-height rigid (FHR) facing is constructed by casting-in-place concrete

Symbol	Structure type	Total length or total number of site	Maximum height
R	GRS retaining wall with FHR facing (RW)	3,528 m	11.0 m
A	GRS bridge abutment	29	13.4 m
I	GRS integral bridge	1	6.1 m
B	RC box culvert integrated to GRS RW	3	8.4 m
T	GRS tunnel entrance protection	11	12.5 m

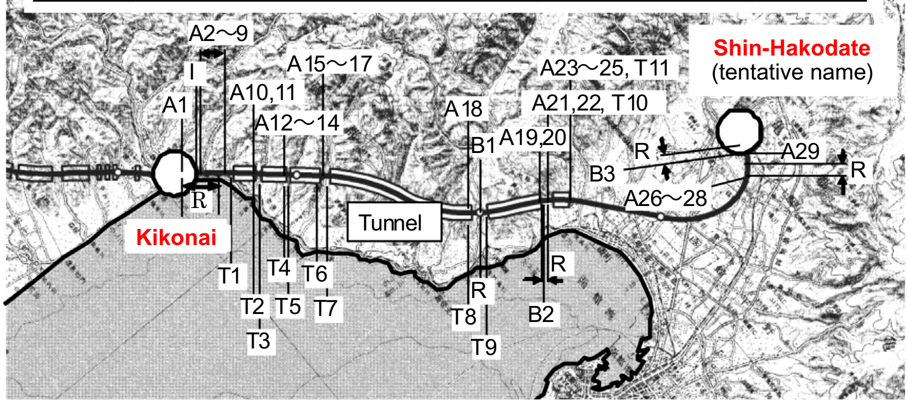

Fig. 2. GRS structures constructed between Kikonai and Shin-Hakodate Stations of Hokkaido Shinkansen (see Fig. 1 for the location of this zone).

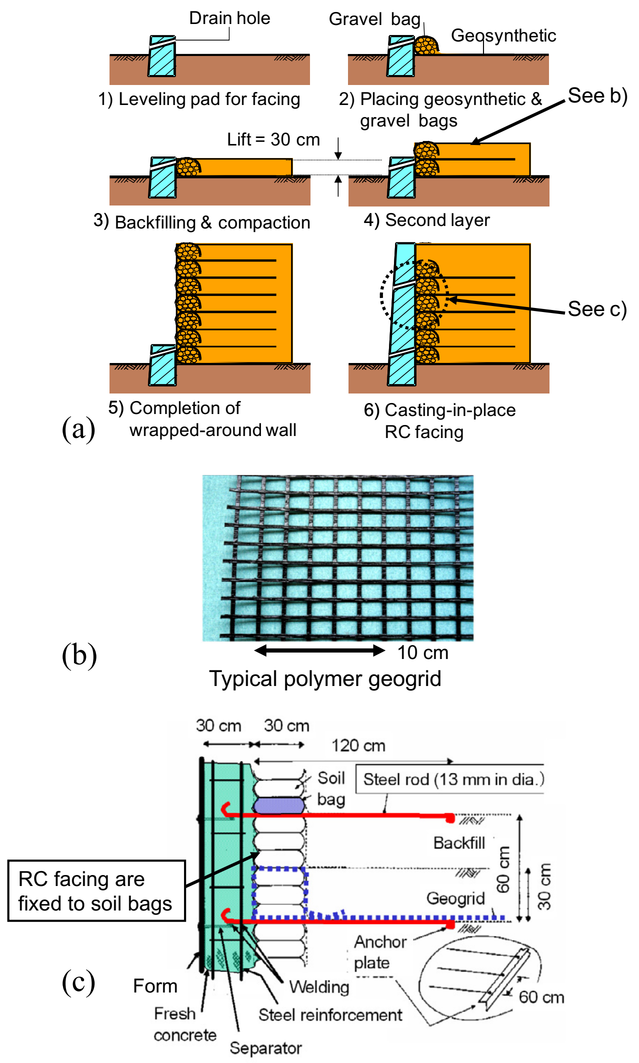

Fig. 3. GRS RW with FHR facing: (a) staged construction; (b) a typical geogrid; and (c) details of facing construction (Tatsuoka et al., 1987).

in the space between the outer concrete form, supported with steel rods anchored in the geogrid-reinforced backfill, and the wall face wrapped-around with geogrid reinforcement (Tatsuoka et al., 1987). The steel rods are temporarily used only to anchor the steel reinforcement inside the facing and the outer concrete frame during facing construction and are not expected to function as a connector between the facing and the reinforced backfill after the GRS RW is completed. Fig. 3b 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. The facing and the reinforcement layers are firmly connected, because fresh concrete can easily enter the inside of the gravel-filled bags through the aperture of such a geogrid as shown in Fig. 3b wrapping-around the gravel bags. Besides, extra water from fresh concrete is absorbed by the gravel bags, which reduces the negative effects due to the bleeding phenomenon of concrete. In this way, 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 geogrid-reinforced backfill. Figs. 4a and b shows typical GRS RWs between stages 5 and 6 (Fig. 3a) constructed for Hokkaido High Speed Train line. Figs. 4c and d show the completed GRS RWs.

Before the construction of FHR facing, the gravel bags function as a temporary but stable facing resisting against earth pressure generated by backfill compaction and the weight of overlying backfill. In particular, with help of gravel bags placed at the shoulder of each soil layer, the backfill immediately back of the wall face can be compacted efficiently. For completed GRS RWs, the gravel bags function as a drain and as a buffer protecting the connection between the FHR facing and the reinforcement against relative vertical and horizontal displacements that may take place during a long-term period of service. Moreover, for the construction of a conventional cantilever RC RW, concrete forms and its propping are necessary on both sides of the facing. This arrangement becomes more costly at an increasing rate with an increase in the wall height. With this type of GRS RW, on the other hand, only an external concrete form anchored to the GR backfill is necessary without using an internal concrete form (Fig. 3c).

Advantages of FHR facing

The stability of GRS RW is controlled by both of global stability against failure along global failure planes and local stability, in particular, for compression collapse of backfill immediately behind the wall face at low levels of the wall (Tatsuoka, 1992). With respect to the latter, the minimum lateral confining pressure σ_h required to keep the backfill stable is the local active earth pressure when unreinforced, which is equal to $\sigma_h = \sigma_v K_A$, where $K_A = (1 - \sin\phi)/(1 + \sin\phi)$, in the case of backfill having no cohesion behind smooth vertical wall back face without back slope under static conditions. The active earth pressure with actual wall configurations (e.g., unreinforced or reinforced backfill with back slope and inclined non-smooth back wall face under static or seismic loading condition) can be evaluated by the two-wedge limit-equilibrium stability analysis (Horii et al., 1994; Tatsuoka et al., 1998). 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 rigid facing, only earth pressure substantially lower than the active earth pressure $\sigma_v K_A$ can be activated at the wall face, Then, as illustrated in Fig. 5a, no or only very low tensile forces can be activated at the connection between the facing and the reinforcement. Then, at low levels of the wall, as the active zone is very narrow, only very small tensile forces can be activated in the reinforcement. That is, near the wall face, the available tensile force in the reinforcement $T_{available}$ is much below the tensile force required to maintain local stability $(T_{required})_{local}$. This results in low confining pressure, therefore low stiffness and strength of the active zone, which may lead to intolerably large deformation of the active zone. Then, when there is something to support the soil at the front from just flowing outwards, the active zone can be marginally stable. However, if the local failure starts from near the wall face and

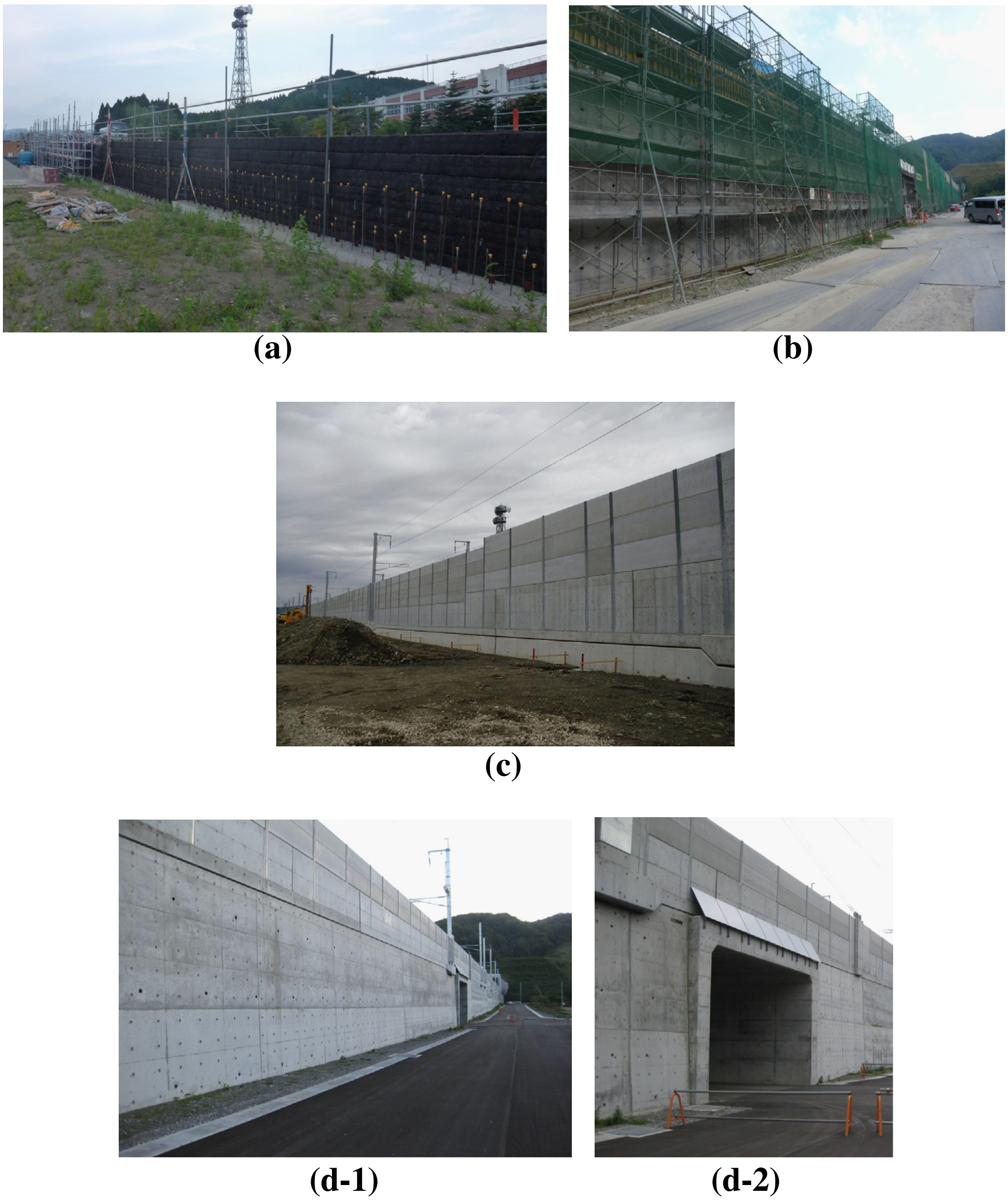

Fig. 4. GRS RW with FHR facing before stage 6; (a) at the western side of the GRS Integral Bridge at Kikonai; and (b) at both sides of box culvert B2; and (c and d) completed GRS RWs.

it progressively develops towards inner locations, the global failure plane is pushed inwards. This situation becomes more likely to take place at low levels of the wall. In this case, the active zone becomes wider, which increases the maximum tensile force in the reinforcement at the location crossing the failure plane to maintain the global stability. In extreme cases, this mode of failure may result in the overall collapse of the whole RW, as have been observed with full-scale test embankments (Tatsuoka et al., 1987; Tatsuoka, 1992).

With this GRS RW system, on the other hand, before the construction of the FHR facing (i.e., at stage 5 in Fig. 3a), the gravel bags function as a temporary facing structure on which high earth pressure can be activated. Relatively high earth pressure that has been activated to the gravel bags are transferred to the FHR facing upon its construction. Then, as illustrated in Fig. 5b, high connection forces with high tensile forces in the reinforcement in the active zone are realized. Such a pattern of reinforcement tensile force distribution as illustrated in Fig. 5b was observed in many full-scale RWs having rigid facing connected to reinforcement (Tatsuoka, 1992). High tensile forces in the reinforcement in the active zone result in high confining pressure in the active zone, thus high stiffness and strength of the active zone, then only a limited amount of deformation of the active zone takes place. Near the wall face, the reinforcement tensile force required to maintain local stability of the backfill ($(T_{required})_{local}$) is equal to the active earth pressure times the vertical spacing of reinforcement layers S_v. In this case, as the active zone is stable, shear stresses due to relative displacements mobilised at the interface between the backfill and the reinforcement become low.

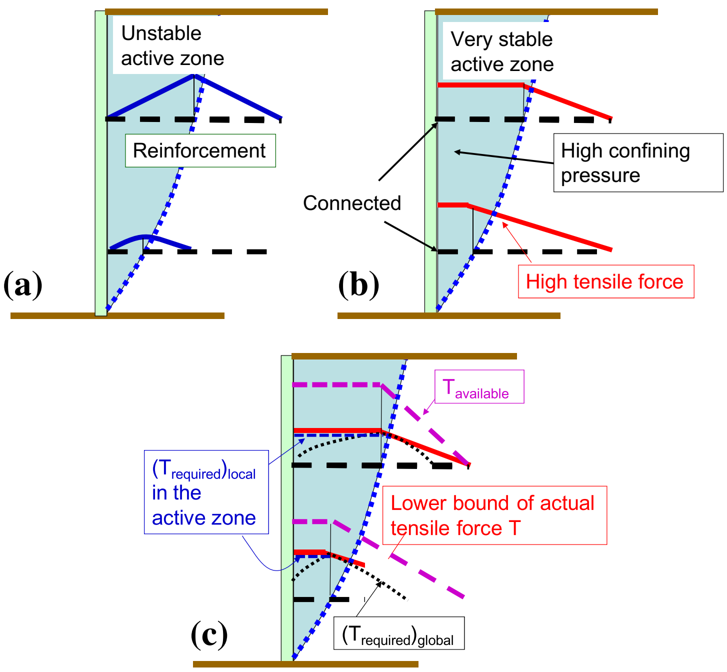

Fig. 5. Schematic figures showing effects of firm connection between the reinforcement and the rigid facing on the distribution on tensile forces (T) in the reinforcement T (Tatsuoka, 1992): (a) when the connection force is zero; and (b and c) when connection force is high.

Then, it can be assumed that the lateral earth pressure is constant in the active zone. Besides, the maximum value of $(T_{\text{required}})_{\text{global}}$ along respective reinforcement layers (T_{max}) develops at the point at which the critical failure plane is crossing. The value of T_{max} is equal to the active earth pressure at that point times S_v. Then, it is relevant to assume that $(T_{\text{required}})_{\text{local}}$ at the wall face, which is equal to the active earth pressure times S_v, is equal to the value of T_{max}. In that case, the tensile force in the reinforcement becomes essentially constant in the active zone as illustrated in Fig. 5b.

The following additional notions are also important:

(1) When the facing is rigid and when the connection strength is high enough, the available tensile force $T_{\text{available}}$ is equal to the smaller value of the tensile rupture strength of reinforcement and the pull-out strength in the stationary zone (in the back of the failure plane). The value of $(T_{\text{required}})_{\text{local}}$ is usually much lower than this value of $T_{\text{available}}$ (Fig. 5c). That is, the local failure of the backfill near the wall face can be easily prevented by using rigid facing connected to the reinforcement layers.

(2) Near the wall face, the actual T value may become higher than $(T_{\text{required}})_{\text{local}}$, for example, immediately after very good compaction. With completed GRS walls, this situation becomes more likely at higher levels of the wall, while, at lower levels of the wall, due to vertical compression taking place by subsequent wall construction after compaction of the soil layer, the actual T value may become similar to $(T_{\text{required}})_{\text{local}}$.

(3) In Fig. 5c, $(T_{\text{required}})_{\text{global}}$ denotes the reinforcement tensile force that required <u>only</u> to maintain global stability of the wall. The value of $(T_{\text{required}})_{\text{global}}$ is largest at the location crossing the critical failure plane and becomes smaller at locations more distant from the critical failure plane. Then, near the wall face in the active zone, the value of $(T_{\text{required}})_{\text{global}}$ may be smaller than $(T_{\text{required}})_{\text{local}}$, therefore may be smaller than the actual tensile force T. If the actual tensile force T can reach only the value $(T_{\text{required}})_{\text{global}}$, the deformation of the backfill in the active zone becomes too large due to local failure, as in the case illustrated in Fig. 5a.

(4) Under the seismic conditions, both $(T_{\text{required}})_{\text{local}}$ and $(T_{\text{required}})_{\text{global}}$ increase. In particular, the value of $(T_{\text{required}})_{\text{global}}$ near the wall face increases due to the outward inertia of facing. Therefore, high connection strength is particularly important to ensure a high seismic stability of the wall.

(5) When concentrated loads, either vertical or lateral in the outward direction or both, are applied to the crest immediately back of the wall face, both $(T_{\text{required}})_{\text{local}}$ and $(T_{\text{required}})_{\text{global}}$ increase near the wall face. As the location of critical failure plane becomes closer to the wall face, at some levels of the wall, $(T_{\text{required}})_{\text{global}}$ may become similar to $(T_{\text{required}})_{\text{local}}$. By using rigid facing connected to the reinforcement layers, the value of $T_{\text{available}}$ easily can become higher than the tensile forces required for local and global stabilities. Therefore, GRS RWs using FHR facing with high connection strength becomes stable by preventing the global failure along the critical failure plane that crosses the wall face at an intermediate height (Tatsuoka, 1992).

A conventional type RW is a cantilever structure that resists against the active earth pressure from the unrein-

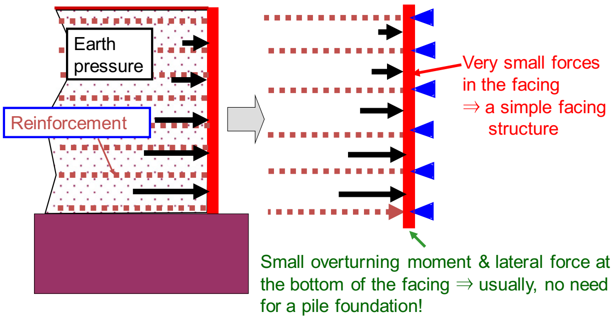

Fig. 6. Mechanical features of FHR facing (Tatsuoka et al., 1987).

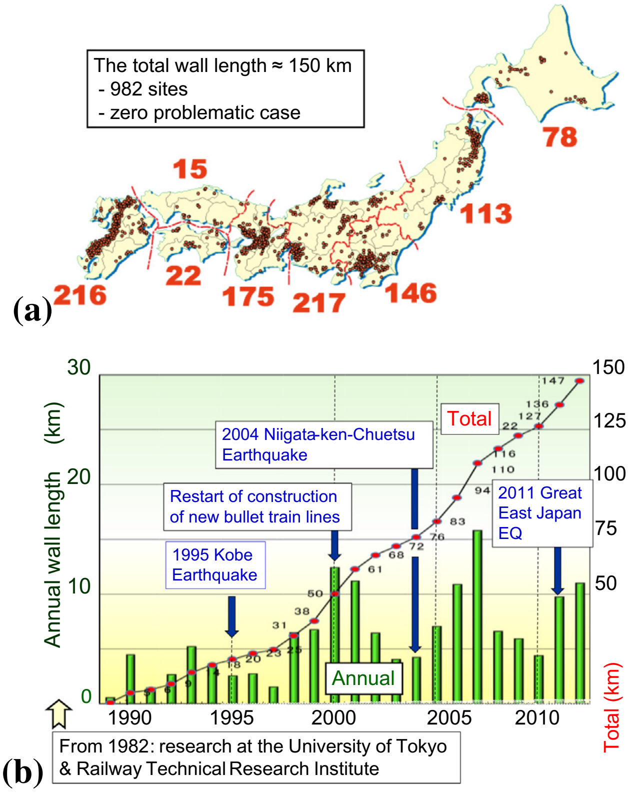

Fig. 7. (a) Locations; and (b) history of GRS RWs with a stage-constructed FHR facing as of June 2013.

forced backfill. Therefore, large internal moment and shear force is mobilized inside the facing while large overturning moment and lateral thrust force develops at the base of the facing. Thus, a pile foundation is often used, which is particularly so when constructed on thick soft subsoil. These disadvantages become more serious at an increasing rate with an increase in the wall height. On the other hand, the FHR facing of this GRSRW system (Fig. 3) is a continuous beam supported by many reinforcement layers with a small span (i.e., 30 cm) (Fig. 6). Therefore, only small forces are mobilised inside the FHR facing even when subjected to high earth pressure. Hence, the structure of FHR facing becomes much simpler and lighter than conventional cantilever RWs. Besides, as only small overturning moment and lateral thrust force is required at the facing base to maintain the global stability of the wall, a pile foundation is not used in usual cases. 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 RWs. The use of FHR facing becomes more advantageous when concentrated external load is activated to the top of the facing or the crest of the backfill immediately behind the facing. Concentrated load is transmitted to the whole FHR facing then to the all reinforcement layers, thereby resisted by the whole of the wall. GRS Bridge Abutment and GRS Integral Bridge were developed by taking advantage of this feature. In comparison, reinforced soil RWs having discrete panel facing lack such a structural integrality as above, thus they have lower resistance against concentrated load. Besides, local failure of the facing (such as loss of stability of a single panel), if it takes place, may result in the collapse of the whole wall.

A brief history of GRS RW with staged-constructed FHR facing

The major advantages of near vertical retaining walls (RWs) over conventional gentle-sloped embankments for railway structures, in particular for HST, are: (a) much smaller base areas required, which significantly reduces the cost for land acquisition; (b) no needs for barrier walls; (c) no needs for protection work, as well as vegetation and its long-term maintenance, of the slope; and (d) a much

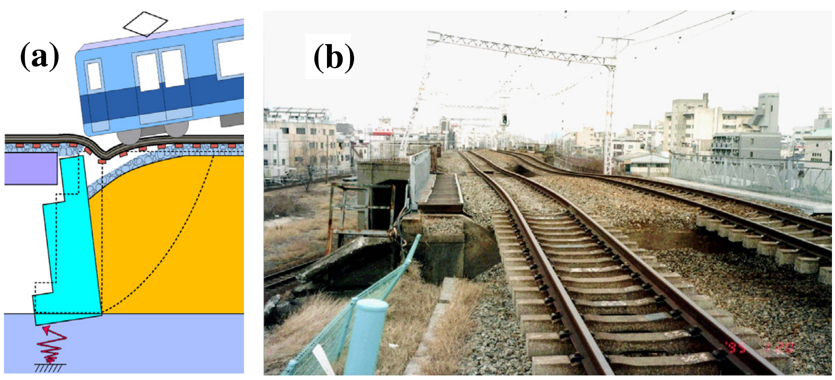

Fig. 8. (a) Illustration of a large bump behind the conventional type abutment; and (b) a typical case near Shin-Nagata Station of JR Kobe Line during the 1995 Kobe earthquake.

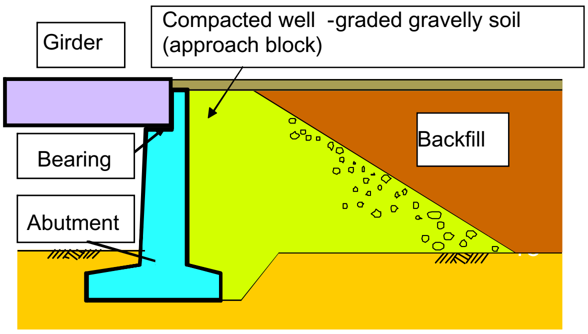

Fig. 9. Improved conventional type bridge abutment having an approach block.

smaller volume of ground improvement of soft sub-layer when required. For these reasons, for railways, conventional type RWs (typically unreinforced concrete gravity type and RC cantilever type) were often constructed in place of gentle-sloped embankments. However, due to their high construction cost, it has been used only in urban areas, where land restriction is very strong. Besides, their construction is very costly when dense long piles are necessary.

Until June 2013, GRS RWs with FHR facing have been constructed for a total length of nearly 150 km, mainly for railways (including Shinkansen lines), in place of conventional type RWs (Fig. 7). This is due to a much higher cost-effectiveness resulting from: (1) no need for a pile foundation (i.e., with GRS RWs with FHR facing, usually shallow ground improvement by cement-mixing to ensure the bearing capacity is sufficient even when constructed on soft ground); (2) a much higher stability, in particular against severe seismic loads; (3) the use of FHR facing as the foundation for electric poles (typically one pole per 50 m) and noise barrier walls; and (4) a negligible bump immediately behind the bridge abutment (as explained in the next section).

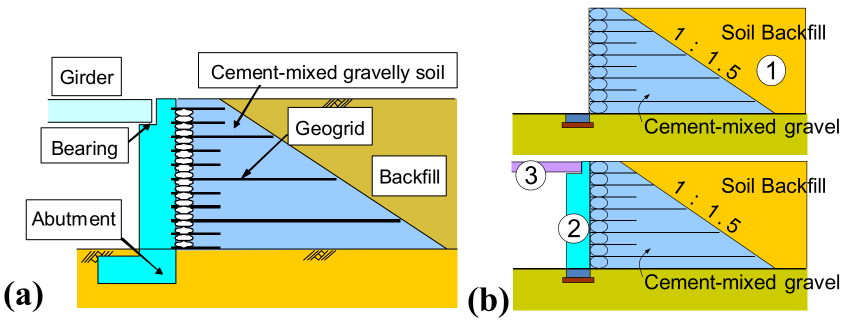

Fig. 10. GRS bridge abutment; (a) structure; and (b) construction procedure.

(a) During construction

(b) During construction (from the backside)

(c) Completed

(d) Completed

Fig. 11. GRS Bridge Abutment at Mantaro (A21 in Fig. 2).

A GRS RW of this type for HST was constructed for the first time between Takasaki and Karuizawa Stations of Hokuriku Shinkansen in 1991 (Tatsuoka et al., 1987). In the meantime, a very high seismic stability of this type of GRS RWs was validated by its high performance during the 1995 Kobe Earthquake (Tatsuoka et al., 1998) and the 2011 Great East Japan Earthquake (Tatsuoka et al., 2012). The other advantages over gentle-sloped embankments and conventional type RWs, described above, were also demonstrated by a number of successful cases not only in urban areas but also at country sides. The RWs of Hokkaido Shinkansen were all constructed by this technology, as typically shown in Fig. 4a through d. It may be seen from Fig. 4c and d, the FHR facing is used also as the foundation for noise barrier walls. The seismic design code for this type of GRS RW has been developed (Koseki et al., 2007a,b; Tatsuoka et al., 2010).

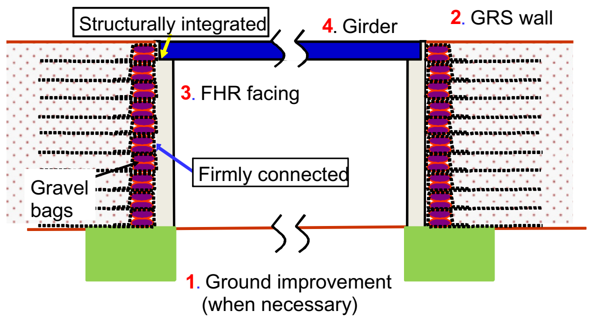

Fig. 12. Structure of GRS integral bridge (the numbers show the construction sequence).

GRS Bridge Abutment

Problems with conventional type bridges

The development of bumps immediately behind the bridge abutment by depression of the unreinforced backfill, which may be escalated by displacements of the side RWs and the abutment, during a long-term service period and by seismic loads (Fig. 8) is one of the most serious problems with conventional type bridge abutments. To alleviate this problem, an approach fill of compacted well-graded gravelly soil, called an approach block (Fig. 9), was introduced in the 1967 design standard for railway structures. However, it was revealed that this measure is not effective enough. Typically, during the 1993 Hokkaido Nansei Earthquake, at Tate-Arigawa Bridge, despite that the maximum horizontal peak acceleration was only 200 cm/s^2, the backfill immediately behind a 8.5 m-high abutment exhibited about 50 cm settlement, which exceeds the allowable limit, in particular with high speed trains. Then, a new bridge abutment was developed based on results from a series of model tests in the laboratory (Figs. 10a and b) (Aoki et al., 2005; Tatsuoka et al., 2004; Tatsuoka et al., 2005). With this new bridge type, the two ends of a girder are placed via a pair of bearings (movable and hinged) on the top of the FHR facings of a pair of GRS RWs that have been constructed by the procedure described in Fig. 3, or one end of a girder is placed via a

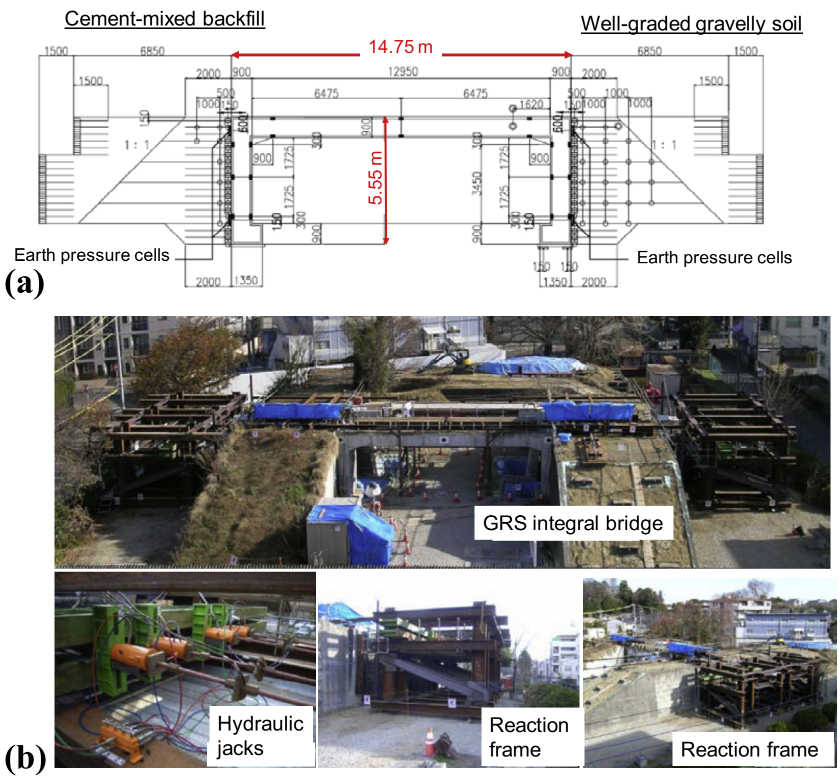

Fig. 13. (a) A full-scale model of GRS integral bridge, completed Feb. 2009 at Railway Technical Research Institute, Japan (Suga et al., 2012); and (b) cyclic lateral loading tests applying simulating thermal deformation of the girder and level 2 design seismic loads (Jan, 2012) Koda et al., 2013.

bearing (usually hinged) on the top of the FHR facing of a GRS RW while the other end is placed on the top of a pier. To ensure long-term high performance of the GRS bridge abutments for HST lines, the backfill immediately behind the facing is lightly cement-mixed geogrid-reinforced well-graded gravelly soil, while the gravel bags immediately behind the facing are filled with un-cemented gravel. The mixing proportion, field compaction control and the strength and deformation characteristics of cement-mixed soil are described in details elsewhere (Tatsuoka et al., 2004, 2005).

Another type of GRS bridge abutment has been developed by other researchers (Zornberg et al., 2001; Helwany et al., 2003). Unlike the system described in Fig. 10, with their bridge system, a girder is placed via a bearing on the crest of geogrid-reinforced backfill immediately behind a discrete facing consisting of modular blocks.

The first advantage of GRS Bridge Abutment illustrated in Fig. 10 is a much higher seismic stability with a minimum bump even against very severe seismic loads. Yet, it is much less costly resulting from much more slender RC facing and usually no use of a pile foundation. Not including

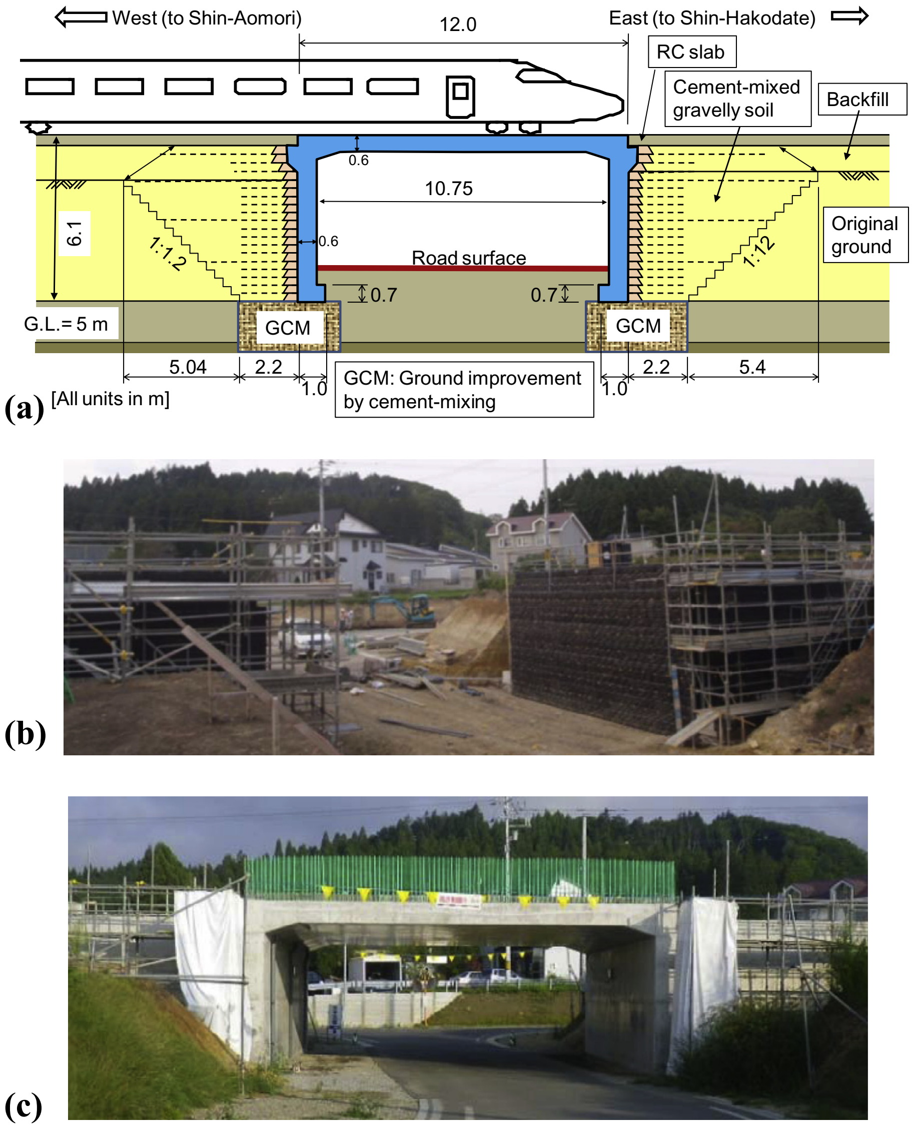

Fig. 14. GRS Integral Bridge at Kikonai (at site I in Fig. 2): (a) structural details; (b) during construction (14 Oct. 2011); and (c) completed (31 July 2012).

Table 1
Comparison of different plans for the over-road bridge at Kikonai.

	Original plan: box culver	Alternative plan No. 1: PC through girder bridge	Alternative plan No. 2: GRS integral bridge
Figure number	Fig. 15	Fig. 16	Figs. 14 and 17
Construction cost ratio	1.0	1.25	0.5
Maintenance of bearings	None	Yes	None
Total rating	Reasonable	Less acceptable than the original plan	Much more acceptable than the original plan

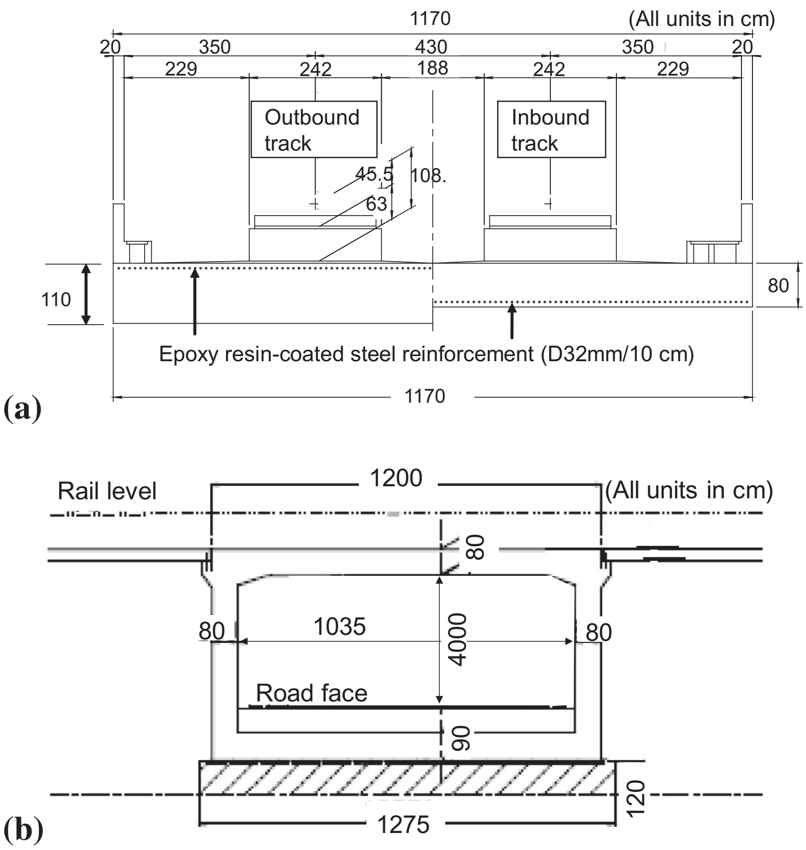

Fig. 15. Box culver (the original plan): (a) transversal cross-sections of the girder at the top of the facing (left) and the center (right); and (b) longitudinal cross-section of the bridge.

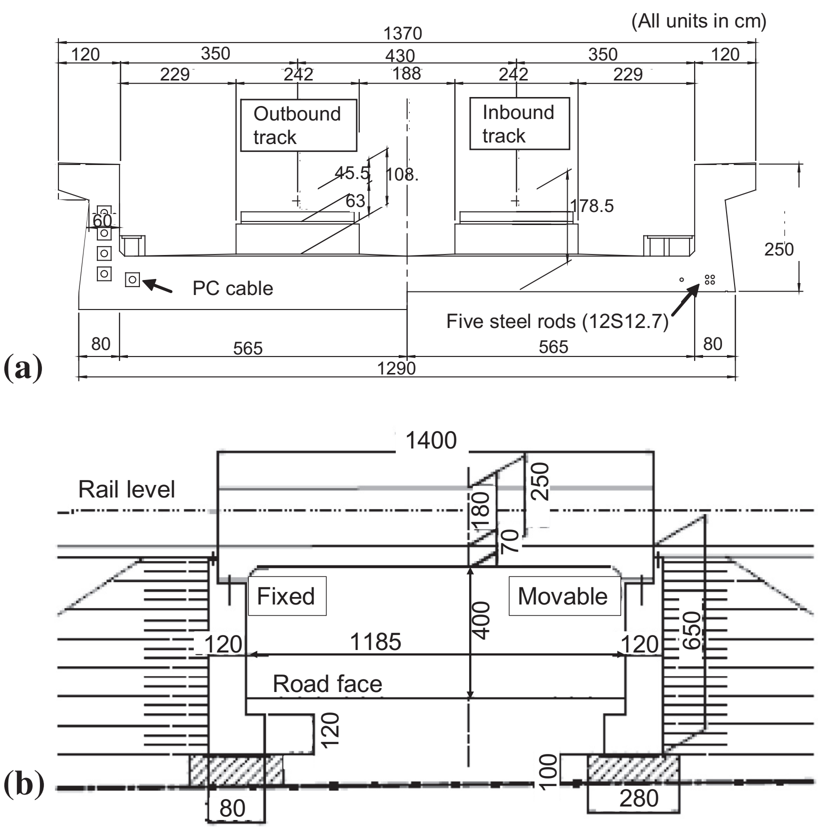

Fig. 16. PC through bridge (alternative plan 1): (a) transversal cross-sections of the girder at the top of the facing (left) and the center (right); and (b) longitudinal cross-section of the bridge.

a cost reduction with the foundation structure and long-term maintenance, the construction cost decreases by about 20% when compared with the conventional type. The first one was constructed during a period of 2002–2003 at Takada for a new HST line in Kyushu Island, called Kyushu Shinkansen, with which full-scale vertical and lateral loading tests of the facing were performed to confirm its high stability (Aoki et al., 2005; Tatsuoka et al., 2005). It was confirmed that the tensile rupture strength at the back of the FHR facing is sufficiently high to resist against the lateral inertial force of the girder and the facing by high seismic loads by which a number of RC structures collapsed during the 1995 Great Kobe Earthquake (i.e., Level 2 seismic design load). For Hokkaido Shinkansen, in total 29 bridge abutments were constructed by this technology while no conventional type bridge abutment was constructed. The tallest GRS bridge abutment is 13.4 m high (Fig. 11a–d). Until today, about 50 abutments of this type have been constructed for railways in Japan.

GRS Integral Bridge

A brief history of development

Most of the remaining serious problems with GRS Bridge Abutment (Fig 10) are due to the use of a girder bearing (movable or fixed). To alleviate those problems, GRS Integral Bridge (Fig. 12) was developed based on results from a comprehensive series of model tests in the laboratory (Tatsuoka et al., 2008, 2007; Munoz et al., 2012), the construction of a full-scale model and loading tests on the full-scale model (Fig. 13) (Suga et al., 2012; Koda et al., 2013). The only but significant difference from GRS Bridge Abutment (Fig. 10) of GRS Integral Bridge (Fig. 12) is that the girder is structurally integrated to the top of the FHR facing of a pair of GRS RWs without using bearings.

The advantages of GRS Integral Bridge are as follows:

(1) The construction and maintenance of the bearings becomes unnecessary.
(2) The RC girder becomes more slender due to a significant reduction of the moment at the center of the girder (becoming about a half) resulting from flexural resistance at the connections between the girder and the facing at both ends of the girder.
(3) The seismic stability increases significantly due to a significantly increased structural integrality and a reduction of the weight of the girder and the facings.
(4) Due to higher structural integrality and a smaller cross-section of the girder, the resistance against tsunami current increases significantly.

The first GRS integral bridge was constructed as the over-road bridge at Kikonai (Fig. 14). Taking of the advantages listed above, three other GRS integral bridges were adopted to restore the bridges that fully collapsed

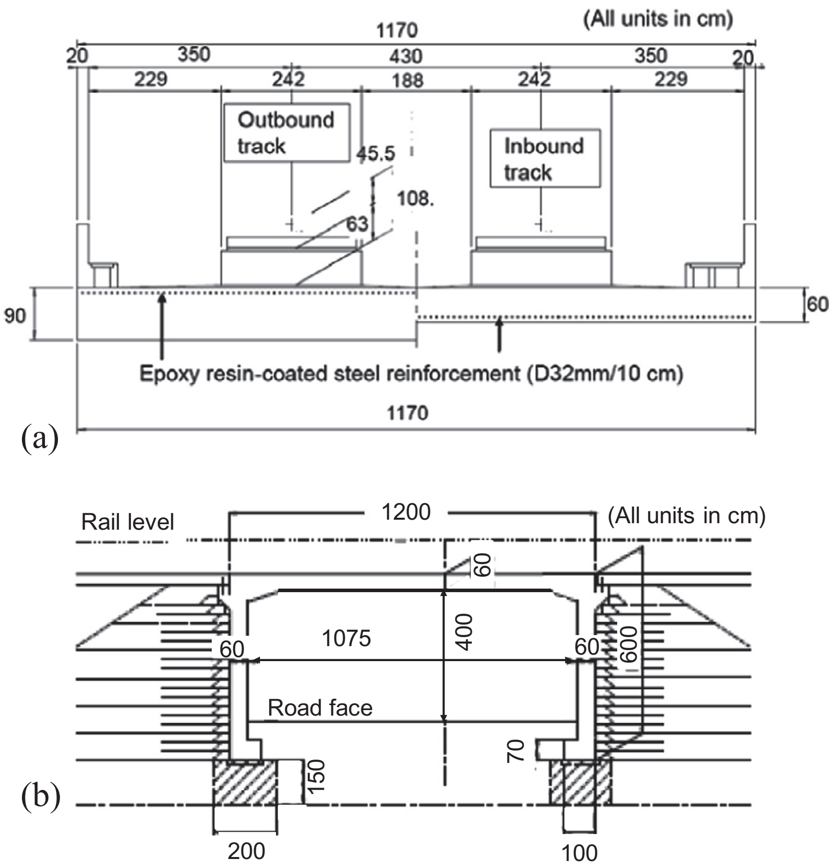

Fig. 17. GRS integral bridge (alternative plan 2): (a) transversal cross-sections of the girder at the top of the facing (left) and the center (right); and (b) longitudinal cross-section of the bridge.

by tsunami during the 2011 Great East Japan Earthquake (Tatsuoka et al., 2012) and they were completed by the end of 2013.

Advantages of GRS Integral Bridge

GRS Integral Bridge was selected for the bridge at Kikonai based on the following comparative study (Table 1). The original plan was a RC box culvert (i.e., an underpass structure) (Fig. 15), which is basically relevant for this short span. However, due to a rather thick top slab, the road face should be made lower by excavation to have an inner height required for traffic (i.e., 4.0 m). The ground should be excavated largely also for a rather thick bottom slab. The first alterative plan was a PC through-girder bridge (Fig. 16). However, the vertical walls (i.e., facings) become rather thick and their foundations become relatively large and the span of the girder, supported by a pair of bearings, becomes relatively long (14 m). For these reasons, this structural type is more costly than the original plan (Fig. 15). Moreover, as the ratio of the span to the girder length is rather small compared with many ordinary ones, the pre-axial stress at the center of the central concrete slab that is to be introduced becomes rather small and some sophisticated numerical analysis by the FEM becomes necessary to estimate this pre-stress with confidence. The second alternative was GRS Integral Bridge (Fig. 17), which was finally adopted for the following reasons. Firstly, by the bending moment activated at the connections between the girder and the facings, the maximum bending moment in the girder, which is developed at the center of the girder, becomes nearly a half of the value in the case of simple-supported girder. This makes the girder much thinner. Secondly, due to no use of bearings, the girder span becomes 12 m, shorter than the PC through bridge (Fig. 16). Thirdly, with a lighter girder and by being supported with many geogrid layers and the backfill, the facings become thinner and their foundations become smaller than the PC through bridge (Fig. 16). Fourthly, due to a thinner girder and no bottom slab, the depth of ground to be excavated to achieve the required inner height (i.e., 4.0 m) becomes much shallower than the box culvert (Fig. 15). Lastly, the volume of ground improvement becomes much smaller than the box culvert (Fig. 15). For these reasons, the construction cost becomes lowest with the GRS integral bridge (Fig. 17), despite the higher performance described above.

This GRS integral bridge is the first full-scale one while it is for a HST line. For these reasons, to confirm its high stability against thermal deformation and seismic load indicated by various model tests (Tatsuoka et al., 2008, 2007; Munoz et al., 2012) and numerical analysis (Yazaki

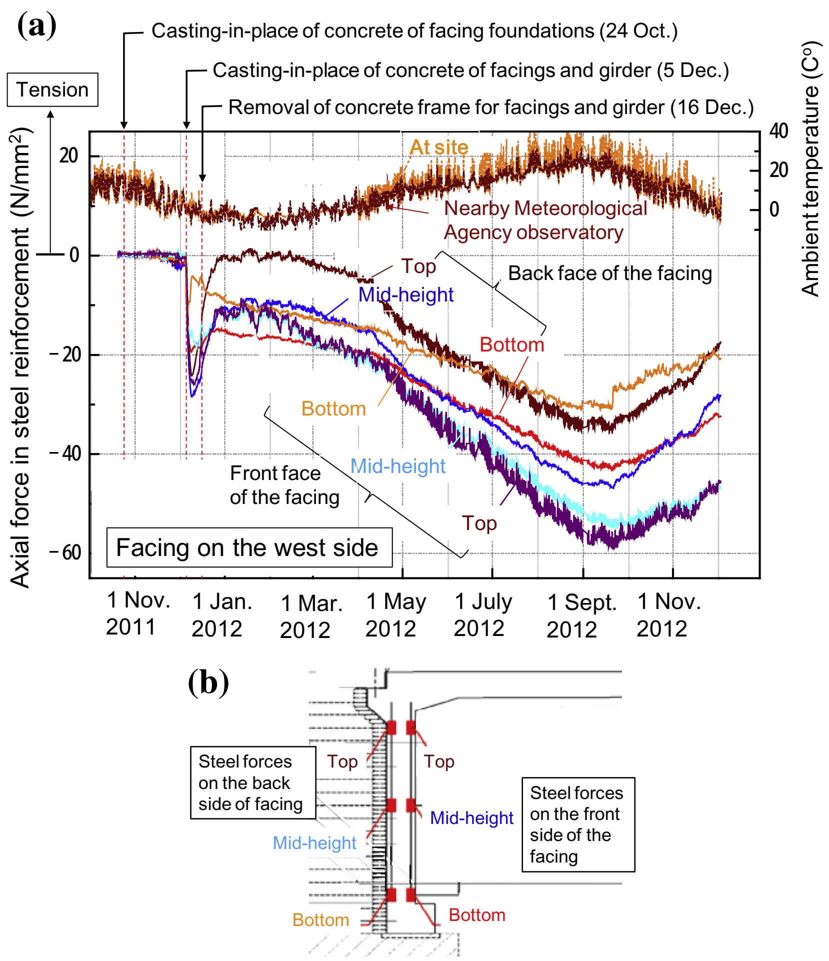

Fig. 18. (a) Time histories of ambient temperature and axial forces in the steel reinforcement in the west side facing; and (b) locations of measurement.

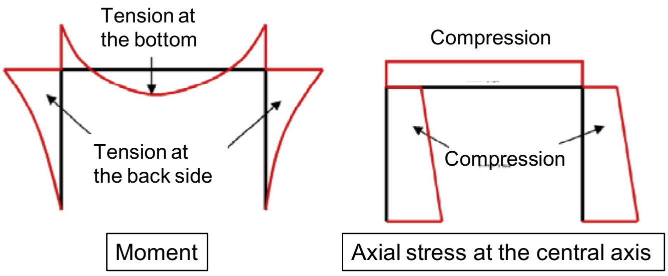

Fig. 19. Schematic diagram showing the stress distribution patterns in the steel reinforcements when subjected to the self-weight of RC members.

et al., 2013), the behavior of this bridge was monitored continuously from the start of construction (Kuriyama et al., 2012). The monitoring will be continued until sometime after the start of service. 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 selected places are being observed. Typically, Fig. 18 shows the time histories of ambient temperature and axial force in the steel reinforcement (corrected for effects of temperature changes) in the west side facing for more than one year from the start of its construction. It may be seen that the compressive stress in the steel reinforcement on both sides of the facing increases by: (a) the weight of concrete generated by the casting-in-place of concrete of the facing and the removal of concrete form and its support for the construction of the girder; (b) continuing shrinkage by drying of concrete; (c) axial extension of concrete that is smaller than that of steel reinforcement when the ambient temperature raises; and (d) the restraint to the thermal expansion of RC members by the reinforced-soil approach fill when the ambient temperature raises. Fig. 19 shows the general distribution patterns of the stresses in the steel reinforcement by "factor a". That is, the tensile stress in the steel reinforcement in the back of the facing increases by moment caused by the weight of the girder (upon the removal of the concrete form and its support). It seems that the effect of earth pressure on the stress in the steel reinforcement is negligible (although this effect may become large during severe earthquakes). These results show that the structures are not over-stressed at all. Detail analysis will be made to quantify the effects of these factors and will be reported by the authors in the near future.

Box culverts integrated to GRS RWs

At three sites (B1, B2 and B3 in Fig. 2), 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 for the first time. Fig. 20a shows the structure

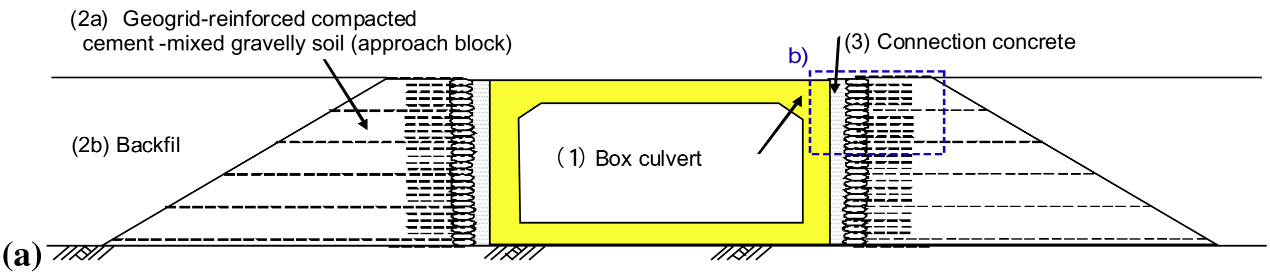

Fig. 20. Grs culvert box (site B2): (a) general structure; the numbers show the construction sequence; and (b) a space between the RC box structure and the approach block before step (3) (at site B1).

of those constructed at sites B2 and B3. At these sites, a RC box structure was firstly constructed as it was requested to re-open local roads as soon as possible. Subsequently, GRS approach blocks comprising of the backfill of well-compacted geogrid-reinforced lightly cement-mixed well-graded gravelly soil are constructed at both sides leaving a narrow space as shown in Fig. 20b. Note that this construction sequence can be reversed depending on respective site conditions. As discussed below, it is in particular the case when constructed on a thick soft deposit and the compression of the soft deposit upon the construction of the GRS approach blocks is too large. Finally, concrete is cast-in-place into this space to integrate the box culvert to the GRS approach blocks. For a high structural 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. Fig. 4d-2 shows the completed GRS Box Culvert at site B2.

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 culverts (in contact with unreinforced backfill, or reinforced backfill without connecting the reinforcement to the back face of the side vertical walls of the box culvert, on both sides) and the conventional type bridges. That is, the cost-effectiveness is higher with a higher seismic stability and essentially no bump in the backfill immediately behind the box culvert. Besides, compared with GRS Integral Bridge, the contact pressure at the bottom face of

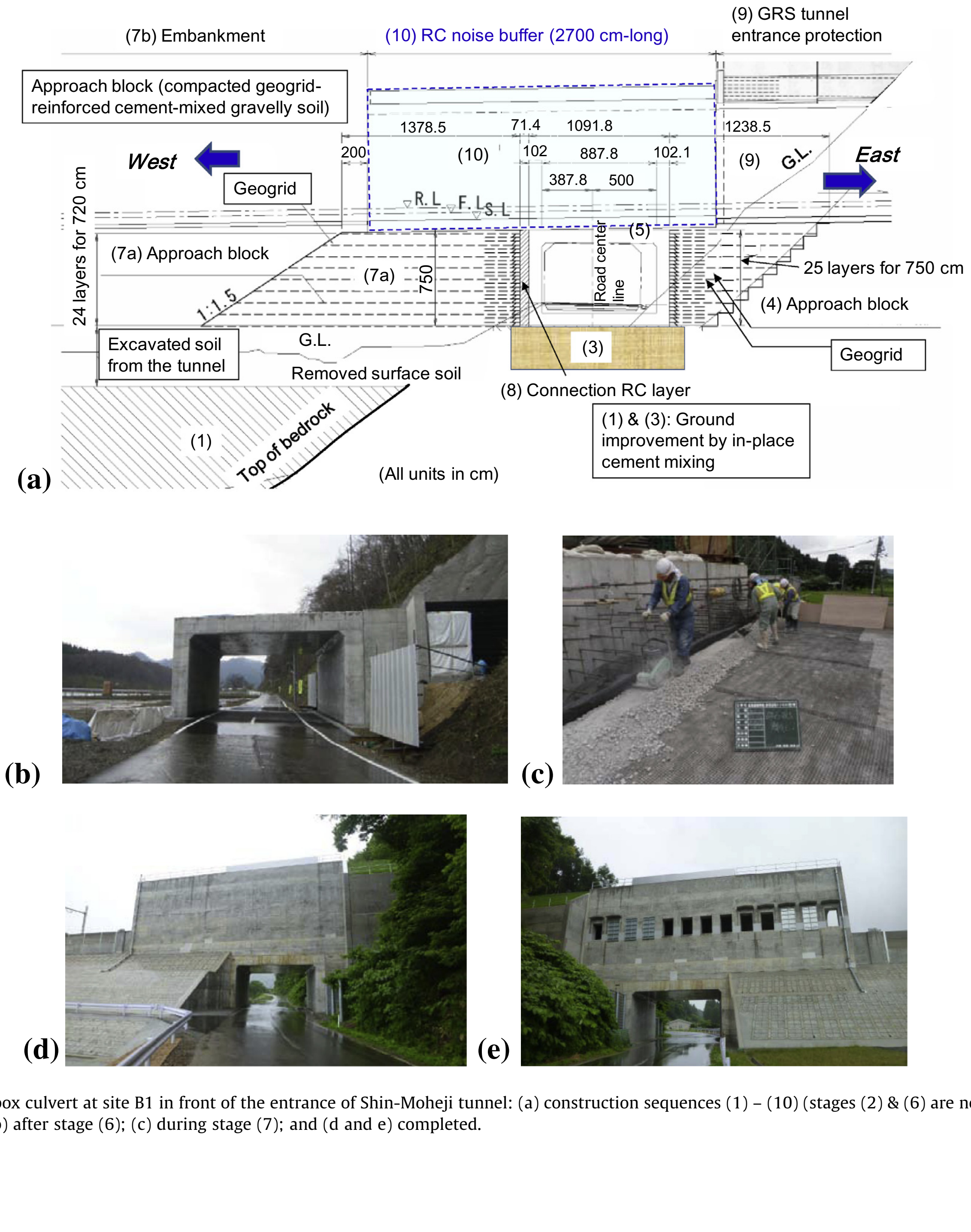

Fig. 21. RC box culvert at site B1 in front of the entrance of Shin-Moheji tunnel: (a) construction sequences (1) – (10) (stages (2) & (6) are not indicated); and views (b) after stage (6); (c) during stage (7); and (d and e) completed.

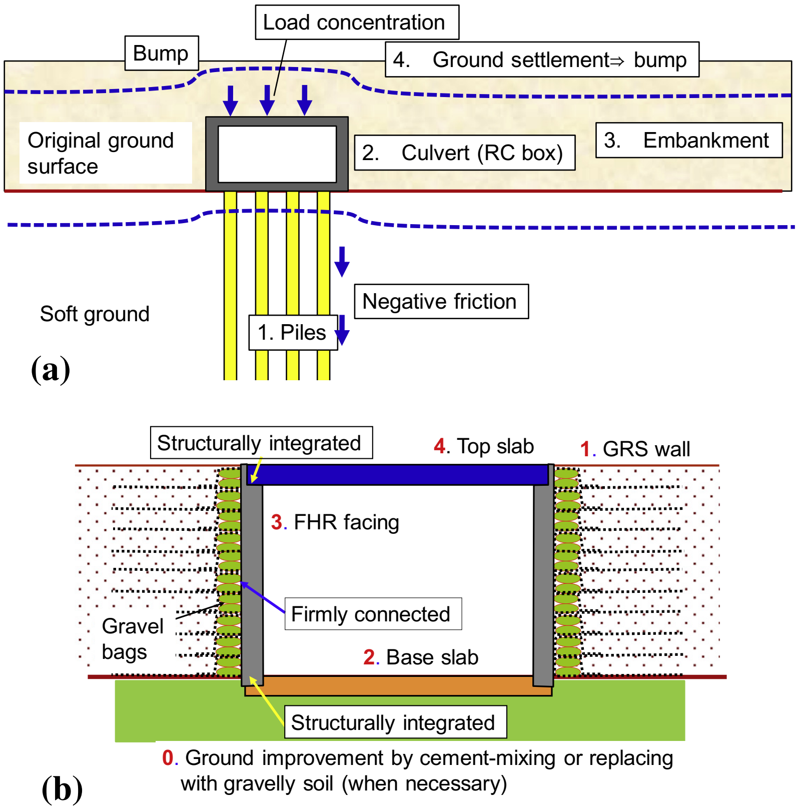

Fig. 22. (a) Typical problems with a box culvert structure crossing motorway/railway embankment constructed by the conventional method on a deep soft deposit (the numbers indicate the construction sequence); and (b) most recommendable construction method of a box culvert taking advantage of the "GRS RW with FHR facing technology" (Fig. 3): the numbers show the construction sequence.

the bottom RC slab is much lower, therefore the stability of GRS Box Culvert is higher. On the other hand, for a longer span for which the bottom RC slab cannot be constructed, GRS Integral Bridge becomes more relevant.

Fig. 21a shows a GRS Box Culvert constructed in front of the entrance of Shin-Moheji tunnel (at site B1). This GRS Box Culvert was designed and constructed under the following specific conditions. That is, to decrease the level of the noise generated by high speed trains going out from the tunnel, a relatively heavy RC buffer structure was to be constructed over this box culvert (Figs. 21d and e for the completed buffer). As the bedrock is inclined and overlain by a non-uniform soft talus deposit, uneven settlement may take place damaging the RC buffer structure (in particular during severe earthquakes) and this should be minimized. Besides, the local road should be kept in service during the construction. To alleviate these problems, the following construction sequence was adopted:

(1) As shown in Fig. 21a, a soft soil deposit below the left side of the box structure is improved by in-place cement-mixing and the surface soft layer is replaced with better backfill.
(2) The local road is moved to the detour constructed on the left side of the box structure.
(3) A shallow thin loose talus deposit below the box structure is improved by in-place cement-mixing.
(4) A GRS approach block at the tunnel entrance is constructed on the right side of the box structure.
(5) A RC box structure is constructed with the right side vertical thin RC wall (i.e., facing) firmly connected to the right side GRS approach block constructed by the procedure described in Fig. 3a–c, as shown in Fig. 21b.
(6) The road is relocated to its original place (inside the box structure) (Fig. 21b).
(7) The other GRS approach block and uncemented backfill behind the left-side facing of the box structure is constructed leaving a space between the approach block and the facing (Fig. 21c).
(8) The RC box structure is integrated to the GRS approach block on the left side by filling the space with concrete.
(9) A GRS tunnel entrance protection (described below) is constructed.
(10) Finally, a RC noise buffer is constructed over the GRS Box Culvert (Figs. 21d and e). It may be seen from Fig. 21e that the RC noise buffer has a series of window at the lateral wall to decrease the level of the noise generated by high speed trains going out from the tunnel.

The experiences from the construction of these GRS Box Culverts indicated that this type of box culvert structure can be widely used at many other places for railway and motorway embankments passing over other railways or motorways, in particular at places where the conventional construction method develops several serious

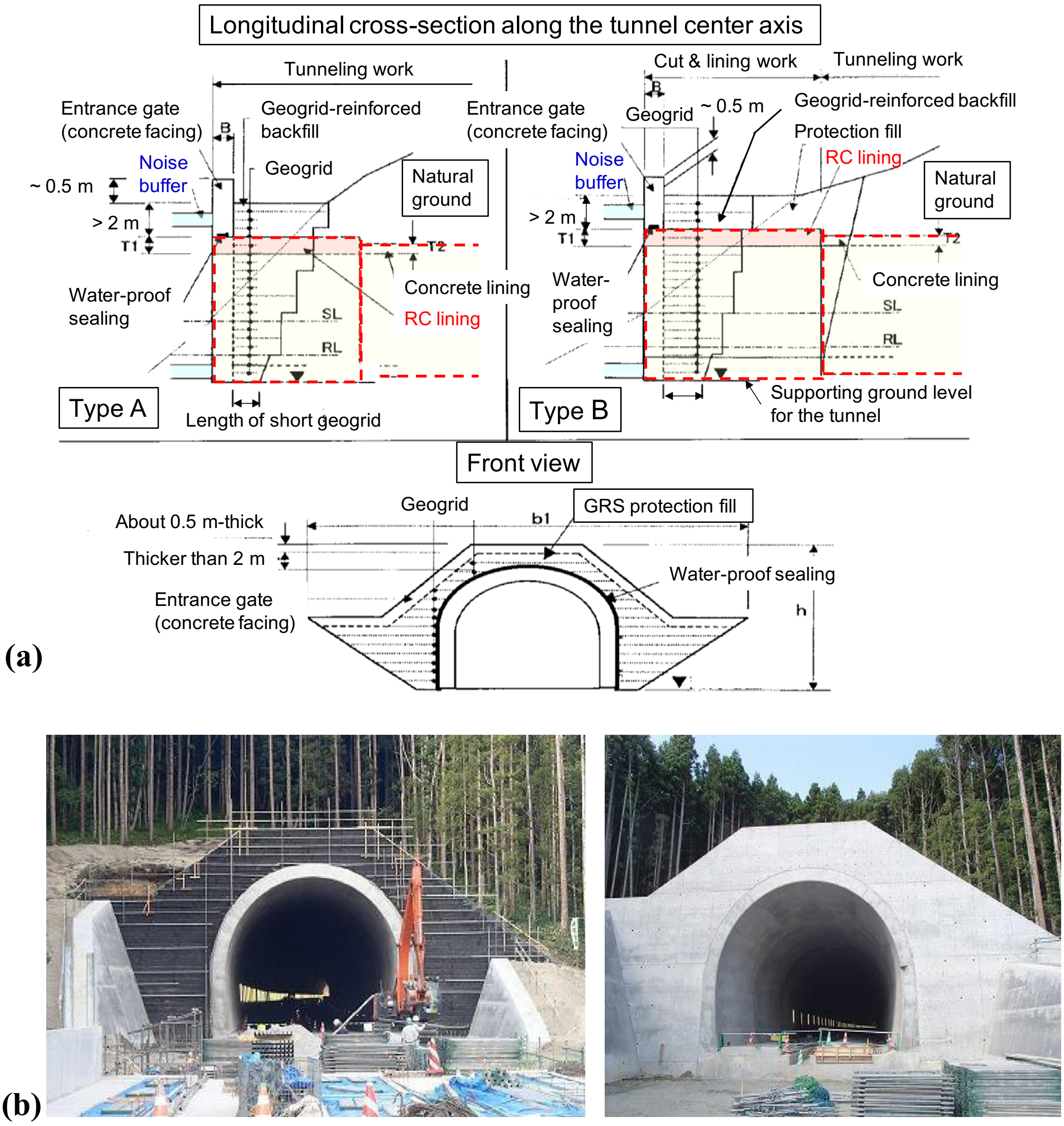

Fig. 23. GRS Tunnel Entrance Protection: (a) two types: and (b) before and after the construction of the front facing at site T10 (at the entrance of Mantaro tunnel).

problems as illustrated in Fig. 22a. In such a case, it is more relevant to firstly construct the GRS approach fills on both sides, followed by the construction of the RC box structure after the settlement of ground by the construction of the approach fills has taken place sufficiently, as illustrated in Fig. 22b. This construction method is most recommendable irrespective of the subsoil condition if suitable to given site conditions.

GRS Tunnel Entrance Protection

To stabilize the natural slope immediate above the entrance of a tunnel, in particular against severe seismic loads and heavy rains, and to protect trains against falling rocks and sliding soil masses, two types of GRS Tunnel Entrance Protection (Fig. 23) were constructed at eleven sites (T1–T11 in Fig. 2). Type A was adopted where the natural slope above the tunnel entrance is relatively steep. After constructing a GRS embankment, the tunnel was excavated in the embankment in the same way as ordinary tunneling works in the natural ground. Type B was adopted where the natural slope above the tunnel entrance is relatively gentle. The natural ground was cut from the top and the tunnel lining was constructed in open-air, followed by the construction of GR backfill covering the tunnel lining. The embankment around the tunnel becomes much more stable by geogrid-reinforcing than otherwise. In some cases, the GR backfill was lightly cement-mixed so that the stiffness became similar to the natural rock ground. The structure was completed by casting-in-place of concrete for the front facing (Fig. 23b).

Discussions

A number of successful case histories of different types of geosynthetic-reinforced soil retaining walls (GRS RWs) have been reported in the literature [e.g., 20–25]. In particular, GRS RWs having a stage-constructed full-height rigid (FHR) facing (Tatsuoka et al., 1987, 1998), which is described in this paper, have been constructed as important permanent RWs for a total length of about 150 km in Japan and many of them are for HST lines. This RW technology has been extended to more advanced GRS structures (i.e., GRS Bridge Abutments, a GRS Integral Bridge, RC box culverts integrated to GRS approach fills and GRS Tunnel Entrance Protections) and all these types of GRS structure were extensively constructed for Hokkaido HST line, as described in this paper. These GRS technologies are now the standard soil structure technologies for railways. Their current popular use is due basically to high cost-effectiveness, in particular high performance during severe earthquakes.

This characteristic feature has been ensured by relevant design, including seismic design, and relevant construction. From the structural point of view, this feature can be attributed largely to a high structural integrality resulting from the use of FHR facing that is connected to geosynthetic reinforcement layers with all the GRS structures described in this paper while integrated to the bridge girder with GRS Integral Bridge. A high structural integrality reduces the possibility of local failure while preventing the development of local failure at certain location, if it takes place, quickly toward the global failure.

With respect to the construction procedure, the construction of FHR facing after the major potential deformation of the backfill and supporting ground has taken place is essential to avoid several serious interaction problems between the backfill and the FHR facing. Besides, any problematic excessive deformation of the backfill or the supporting ground or both, if it takes place, can be detected and can be alleviated before the construction of FHR facing (i.e., also before the construction of the girder with GRS Bridge Abutment and GRS Integral Bridge).

Concluding remarks

The following types of geosynthetic-reinforced soil (GRS) structures were constructed at a number of sites at the south end of Hokkaido Island for a new high-speed train line (Shin-kansen): (1) GRS Retaining Walls (RWs) with staged constructed full-height rigid facing, in place of conventional type cantilever RC RW and gentle-sloped embankments; (2) GRS Bridge Abutments, in place of conventional RC abutments with unreinforced backfill approaches; (3) a GRS Integral Bridge, in place of conventional bridge supporting the girder via bearings; (4) box culverts integrated to GRS approach fills, in place of the conventional type box culverts; and (5) GRS Tunnel Entrance Protections. These GRS structures were selected because they can satisfy very high performance requirements, including negligible bumps immediately back of bridge abutments and box culverts and a high stability for severe earthquakes, and a high cost-effectiveness, definitely higher than the conventional type soil structures. These types of GRS structure are now the standard soil structures for high-speed train lines, as well as ordinary train lines, constructed fully in place of respective conventional type structures.

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