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PERFORMANCE OF SOIL RETAINING WALLS FOR RAILWAY EMBANKMENTS

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ABSTRACT

During the Hyogoken-Nambu earthquake of January 17, 1995, many conventional masonry and unreinforced concrete gravity-type retaining walls completely collapsed, whereas many modern cantilever-type reinforced concrete retaining walls were also seriously damaged. Geogrid-reinforced soil retaining walls having a full-height concrete facing with a total length of about 2 km performed very well. It should be noticed that a wall located in one of the most severely shaken areas deformed only slightly. The performance of geogrid-reinforced soil retaining wall, as documented, will foster future confirmation and development of aseismic design procedures.

Key words: case history, earthquake damage, earthquake resistant, (geogrid), (the 1995 Hyogoken-Nambu earthquake), railroad, retaining wall (IGC: E8/H2)

INTRODUCTION

At approximately 5:46 a.m. on the 17th of January, 1995, a devastating earthquake measuring 7.2 on the Richter scale hit the southern part of Hyogo Prefecture (part of Hanshin district), including Kobe City and neighbouring urban areas (Fig. 1). Figure 2(a) shows the areas where the Japanese seismic intensity scale was seven or higher estimated from the collapse ratio of wooden houses equal to 30% or greater. To the best of the authors' knowledge, the damage to retaining walls (RWs) and embankments was not triggered by soil liquefaction in the foundation soil or the backfill, but was induced rather by high seismic forces applied to the wall structure and the backfill/embankment soil.

Figure 2(b) shows the geographical and geological classification of the areas corresponding to Fig. 2(a). Figure 3 shows a typical geological cross-section in the N-S direction (Line 12 in Fig. 2). A relatively thin upper Holocene soil layer (denoted by As) is underlain by terrace deposits of the later Pleistocene Epoch (Ts), which are underlain by middle Pleistocene deposits of the Osaka Group (Os-g). The thickness of these Holocene and Pleistocene deposits decreases to the north.

As seen in Figs. 2(a) and 2(b), the foundation subsoil in the areas where the collapse rate of wooden houses was highest is mainly 1) Holocene fan deposits (the hatched zones in As) and Later Pleistocene terrace deposits (Ts) located between the Middle Pleistocene terrace deposits at higher elevations (Os-g) and Holocene sandy and gravelly alluvial fans at lower elevations (in the area north-east of Kobe Station), and 2) Holocene clayey deposits at the lowest elevations (part of the white zones in As), located south-east of the fan and terrace deposits (in the area south-west of Kobe Station). It should be noted that the highest damage rate was not observed in the southern areas along the present seashore, where the Holocene clay deposit is relatively thick. Along a particular line (Line 12, Fig. 2), the maximum damage rate was observed immediately south of the JR Tokaido Line (currently named Kobe Line), where the surface soil consists of Holocene fan deposits (mainly gravel and sand) (Fig. 4).

In the severely affected areas, an extensive length of railway embankment had been constructed a long time ago, which is presently used by the JR Tokaido Line (Kobe Line) of West Japan Railway (JR) Company (one of the most important railways in Japan), Kobe Line of Hankyu Railway Company and Main Line of Hanshin Railway Company (*see* Fig. 2). About 60 years ago, in the central zone of Kobe City, a large length of these embankments was replaced by elevated reinforced concrete (RC) frame structures, many of which were very seriously damaged by this earthquake. The railway embankments existing at the time of this earthquake included a number of older retaining walls (RWs) located in high seismic intensity areas (Fig. 2(a)), and most of them were seriously damaged. Only a very limited length of road embankment, however, existed in the affected area, while only some RWs were damaged.

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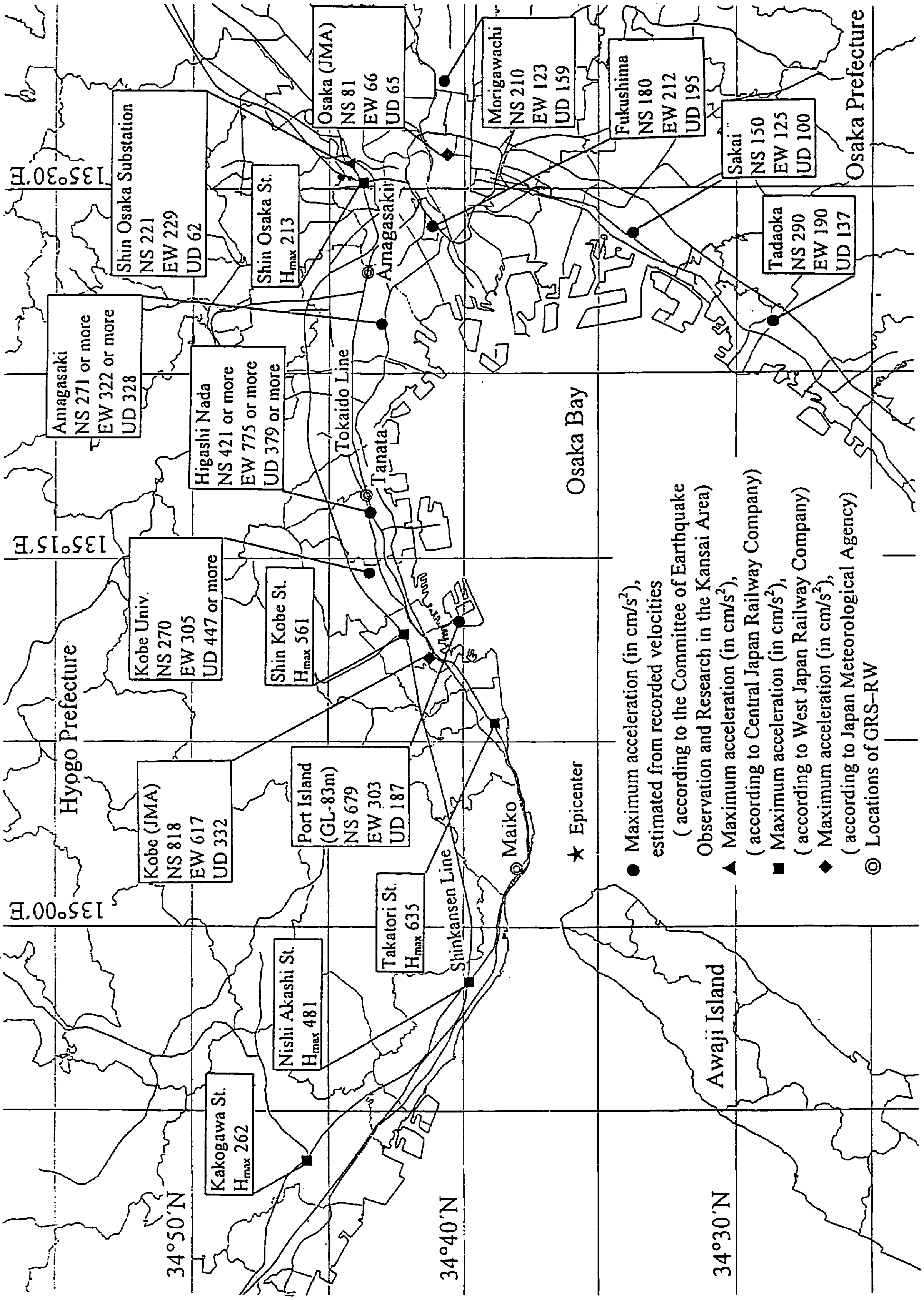
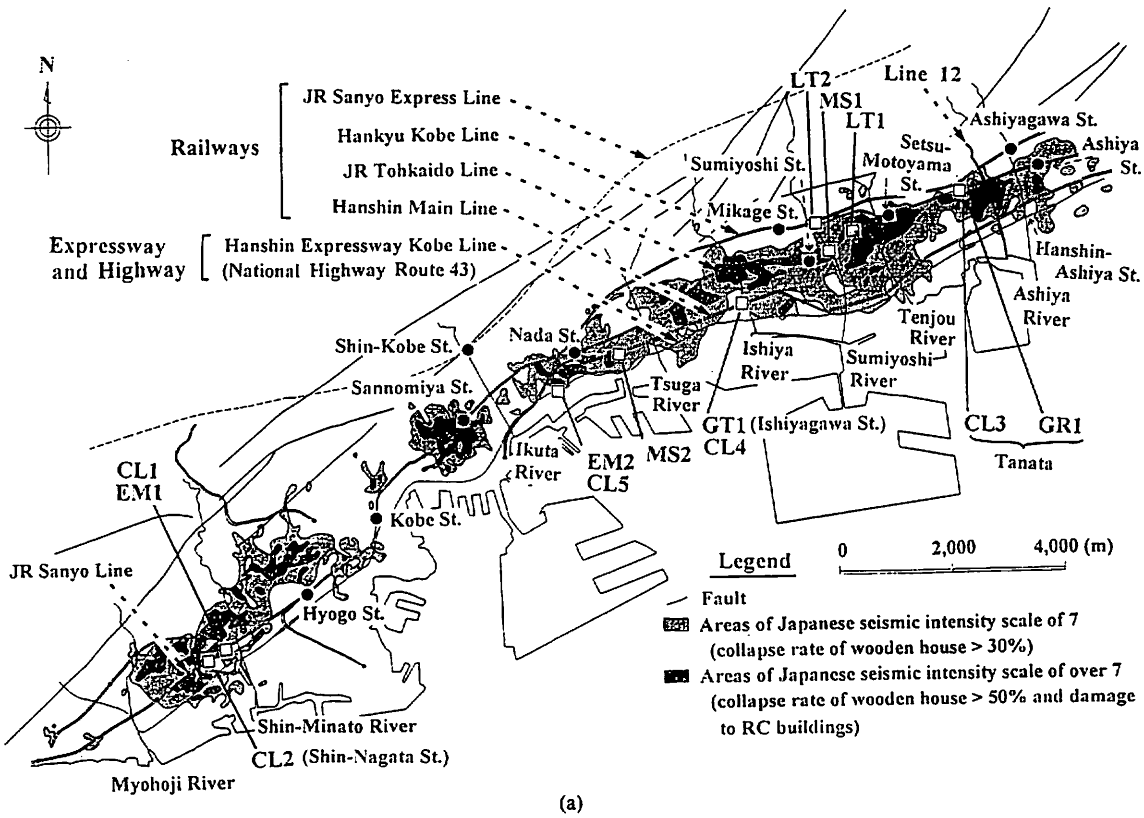
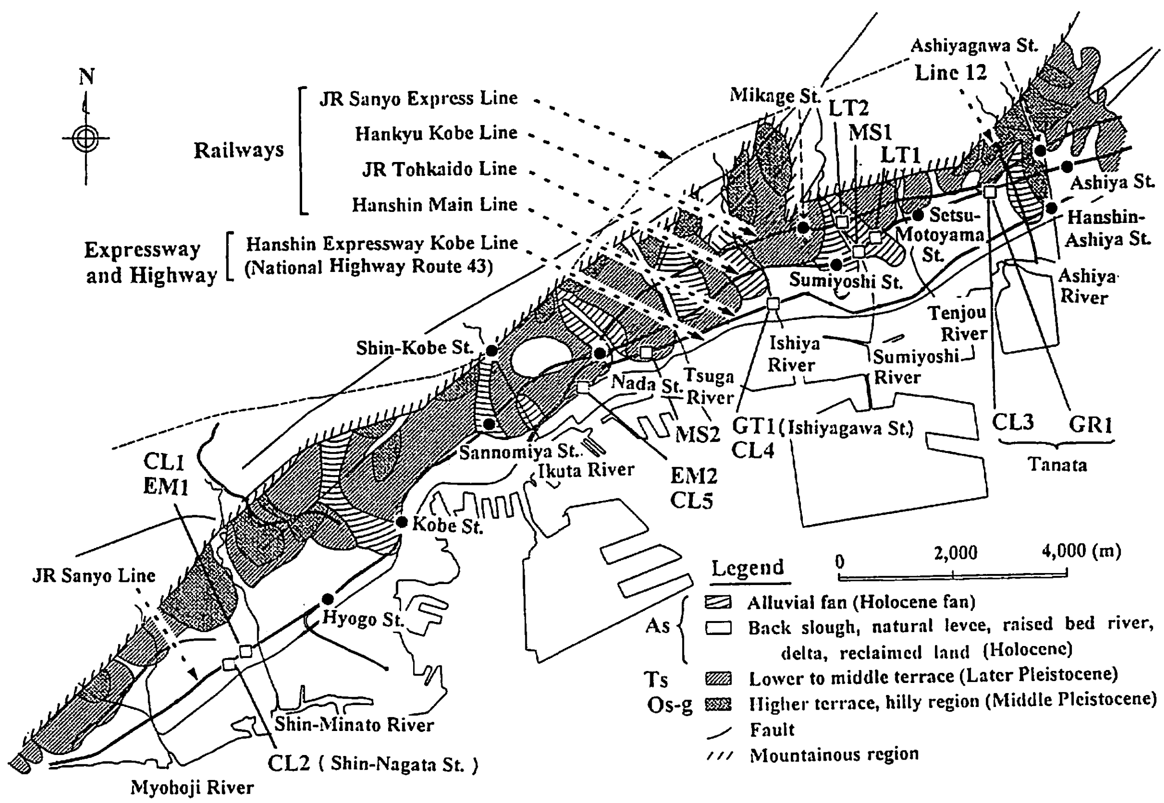


Fig. 1. Epicenter and recorded maximum acceleration values



(a)



(b)

Fig. 2. (a) Areas of Japanese seismic intensity scale of seven or higher; (b) Ground conditions and locations of retaining walls (RWs) reported in this paper (Chuo Kaihatsu Corp., 1995)

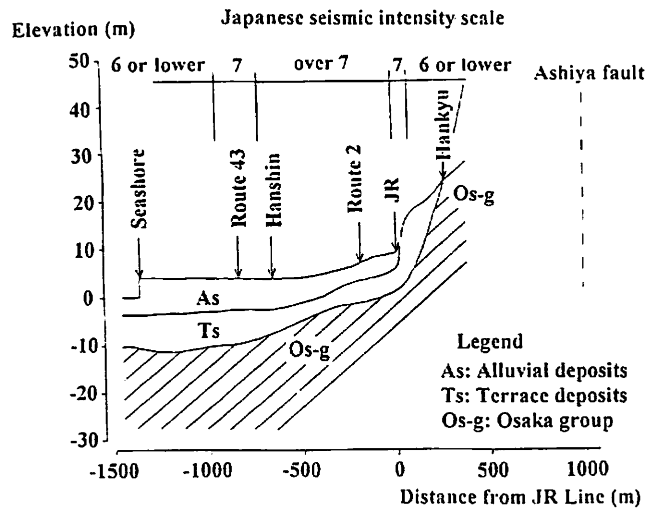


Fig. 3. Geological cross-section in the north-south direction, Line 12 shown in Fig. 2 (Chuo Kaihatsu Corp., 1995)

These RWs for railway embankments can be categorized into the following five groups:

1) Masonry RWs,
2) Leaning-type (supported type) unreinforced concrete RWs,
3) Gravity-type unreinforced concrete RWs,
4) Cantilever-type or inverted T-shaped type steel-reinforced concrete (RC) RWs,
5) Geogrid-reinforced soil retaining walls (GRS-RWs).

The first three types of conventional RW were most seriously damaged. The damage to the cantilever-type or inverted T-shaped type RC RWs was generally less serious than that to the conventional RWs, but many of them were seriously damaged and had to be demolished. The damage to the GRS-RWs was either negligible or minimal.

This paper will describe some typical case histories of the performance of the RWs for railway embankments together with some of the embankment slopes, and some lessons learned from them.

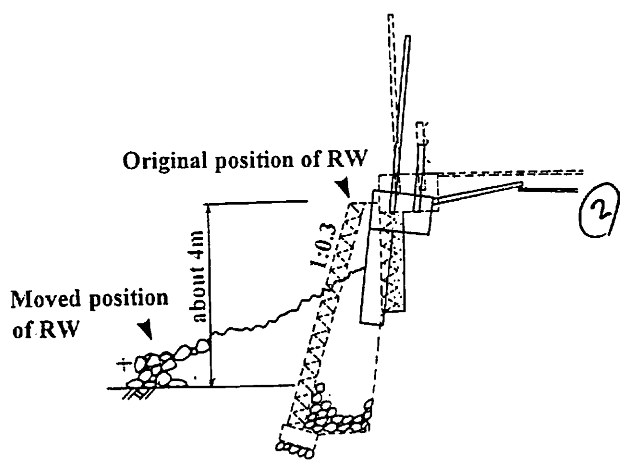


Fig. 5. Typical damaged masonry RW; embankment on the north side along JR Tokaido Line between Setsu-Motoyama and Sumiyoshi Stations (Site MS1 in Fig. 2)

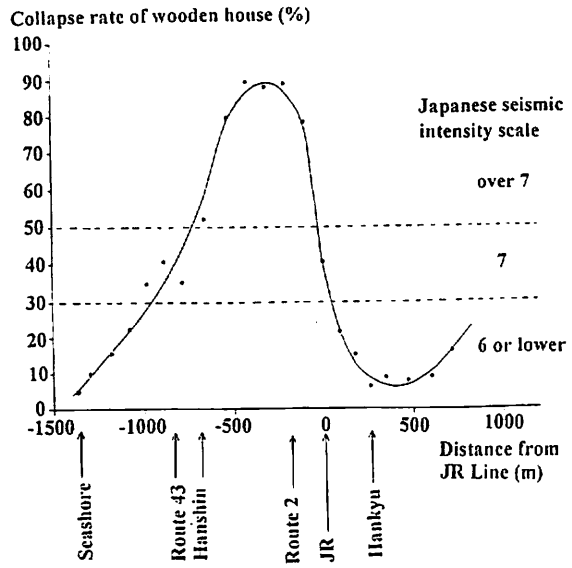


Fig. 4. Collapse rate of wooden houses along Line 12 shown in Fig. 2 (Chuo Kaihatsu Corp., 1995)

DAMAGE TO CONVENTIONAL TYPE RETAINING WALLS

Masonry RWs

This is the oldest type of RW. Most of them were constructed more than about 70 years ago without seismic design. They were, on an average, most seriously damaged among all the types of RWs. Most of the RWs of this type, located in the areas where the JMA scale was equal to 7th or higher, were more-or-less damaged. Figure 5 shows a typical one, constructed 64 years ago (at the time of the earthquake). A pile of stones totally collapsed into stone pieces.

Figure 6 shows another case, where the damage was worse in the sections approaching the bridge abutments. This damage may have been caused by the impact force between the RWs and the abutment due to their different response. These RWs were removed, and GRS-RWs were subsequently constructed to replace them.

Leaning-type Unreinforced ~~Concrete RW~~

~~Most~~ of these were constructed more than about 60 years ago without seismic design. Figure 7 shows a typical one, constructed 58 years ago to support the embankment for the Tokaido Line. Extending over a long length

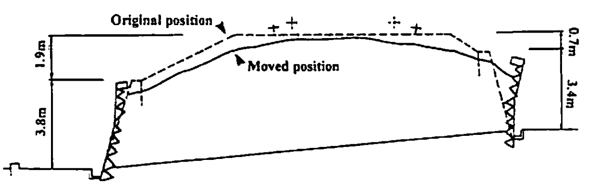


Fig. 6. Typical damage to masonry RW; embankment for Main Line of Hanshin Railway Co. adjacent to Nishi-Nada Station (Site MS2 in Fig. 2)

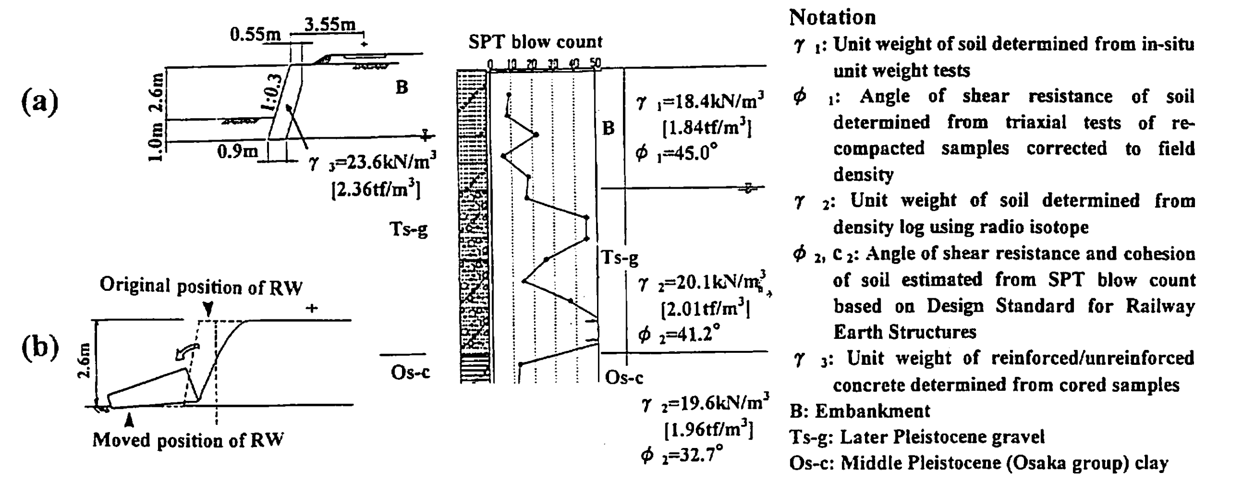


Fig. 7. Typical damaged leaning-type unreinforced concrete RW; embankment along JR Tokaido Line between Setsu-Motoyama and Sumiyoshi Stations (Site LT1 in Fig. 2); (a) cross section and (b) sketch

(about 600 m), the RW was either broken at the level of the ground surface and the upper part overturned completely to the ground surface or overturned about the bottom, resulting in the back face upside down. This damage was perhaps triggered by both a large horizontal seismic force acting on the RW structure itself and a large seismic earth pressure imposed on to its back face from the backfill. It seems, however, that the former should be the major factor for complete overturning, since the backfill soil did not follow the movement of the RW structure. After removing the broken RW structures, GRS-RWs have been constructed except for some locations where reinforcing space was limited.

Figure 8 shows another RW of this type with a height of about 8 m, constructed 57 years ago on both sides of a railway embankment. These walls tilted largely outwards exhibiting a horizontal crack near the bottom for a length of about 500 m. Its complete over-turning was prevented perhaps by the resistance provided by a series of steel-frame structures for electricity supply that were constructed on the top of the RWs. They have been totally demolished including removal of the backfill soil between them, and U-shaped RWs filled with cement-treated soil have been constructed in their place.

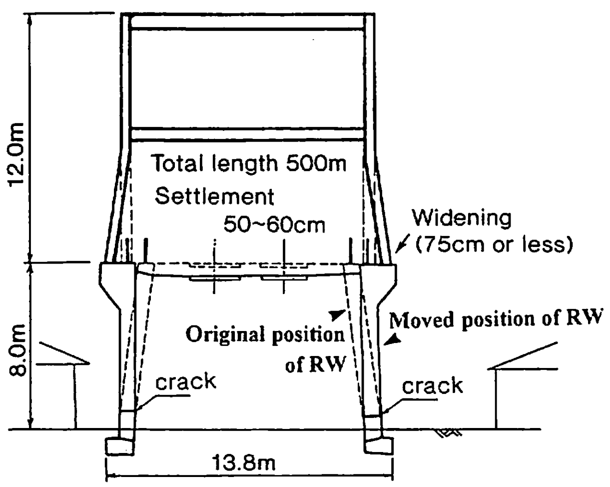


Fig. 8. Typical damage to leaning-type unreinforced concrete RWs; embankment along Kobe Line of Hankyu Railway Co. between Okamoto and Mikage Stations (Site LT2 in Fig. 2)

Gravity-type Unreinforced Concrete RW

Most of these walls were constructed more than about 60 years ago. Figure 9 shows one case of the most serious damage, in which a length of RW tilted largely (the total length was about 460 m), while a 200 m section was totally over-turned. These RWs were constructed 66 years ago based on the standard design used at that time, where pseudo-static stability analysis was adopted using a horizontal seismic coefficient of 0.2. It seems, however, that the strength of the wall structure itself and the bearing capacity of the supporting subsoil below the toe of the wall were not sufficient to prevent, respectively, the failure of the wall structure and the large tilting of the wall. It seems that their complete over-turning was

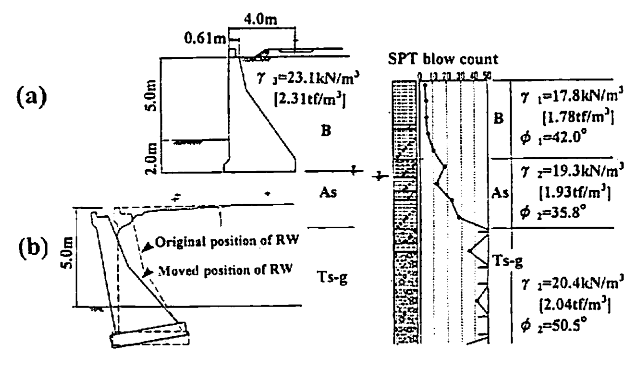


Fig. 9. Typical damage to gravity-type unreinforced concrete RWs; embankment for Main Line of Hanshin Railway Co. adjacent to Ishiyagawa Station (Site GT1 in Fig. 2); (a) cross section (*see* Fig. 7 for notation of γ and ϕ) and (b) sketch

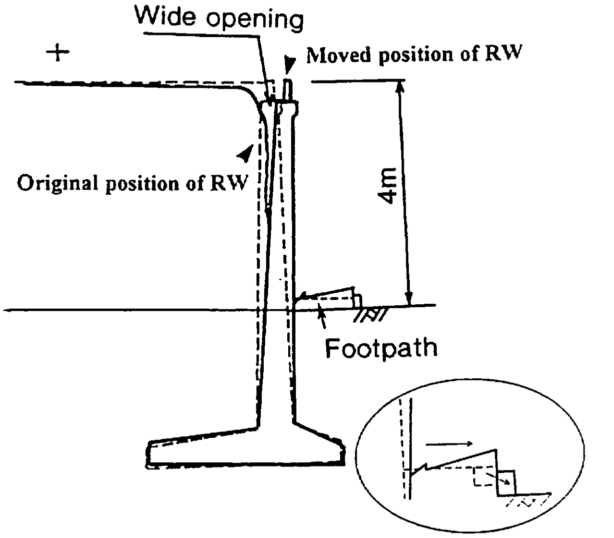


Fig. 10. Typical damage to cantilever-type RC RWs for embankment along JR Sanyo Line between Hyogo and Shin-Nagata Stations (Site CL1 in Fig. 2)

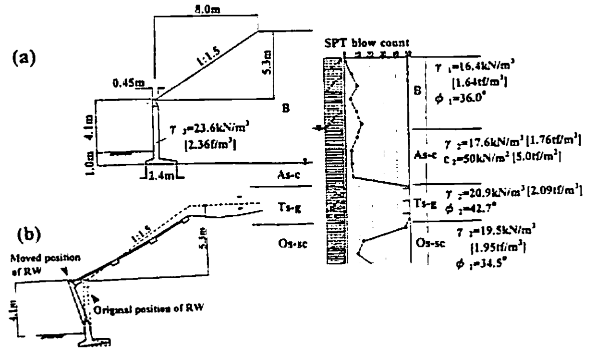


Fig. 11. Rupture of the facing of cantilever-type RC RW along embankment of JR Shin-Nagata Station (Site CL2 in Fig. 2); (a) cross section (*see* Fig. 7 for notation of γ and ϕ) and (b) sketch

caused not only by a large seismic earth pressure but also by a large horizontal seismic force acting on them. These RWs have been demolished, and elevated bridge structures have been constructed. At many other sites, RWs of this type more-or-less tilted outwards, which resulted in a large settlement at the crest of the railway embankment.

The three types of RWs described above were designed so that the gravity resistance of the wall structure was large enough to resist the lateral earth pressure exerted from the backfill. The damaged cases shown above and others, however, indicated that these types of RWs had very low seismic stability. It seems that even without seismic earth pressure applied to the back face, some of the masonry and leaning-type RWs would have tilted or even completely over-turned due to seismic force exerted on the wall structure itself. Masonry or leaning-type RWs are, therefore, not suggested for important civil engineering structures as specified in the Design Standard for Railway Retaining Wall Structures (Japan National Railway 1986). A gravity-type unreinforced concrete RW having a very wide bottom supported by firm subsoil would have been stable even during this level of earthquake, but that type of construction was not considered practical at the time.

Cantilever-type or Inverted T-shaped Type RC RWs

This is a modern type of RW. They were aseismically designed. Figure 10 shows a typical damaged cantilever-type RC RW, constructed about 30 years ago, without a pile foundation. The RW tilted largely outward inducing a large settlement at the top of the railway embankment. The footpath in front of the RW was pushed out laterally by this wall movement. It has been repaired by anchoring the facing structure to the backfill.

Shin-Nagata Station of the JR Sanyo Line was constructed about 30 years ago on a sloped embankment (both sides), supported by this type of RW for a total length of about 800 m (Fig. 11). Most of these RWs tilted and slid outward at the bottom, but the most serious damage which occurred was cracking in the facing. It seems that this type of failure can be explained only by an extra-ordinary large seismic earth pressure acting from the slope above the RW. Due to the failure of these RWs, Shin-Nagata station and the railway tracks at the site were seriously damaged. These seriously damaged cantilever-type RC RWs have been removed, and reconstruct-

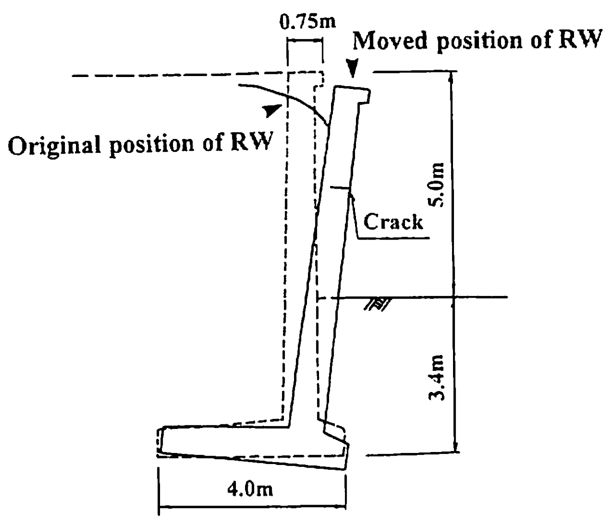


Fig. 12. Damage to cantilever-type RC RW; embankment for Main Line of Hanshin Railway Co. adjacent to Ishiyagawa Station (Site CL4 in Fig. 2)

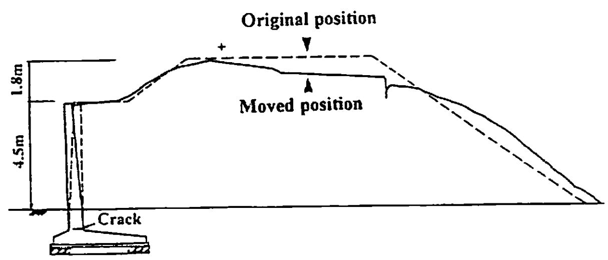


Fig. 13. Damage to cantilever-type RC RW; embankment of freight branch along JR Tokaido Line between Higashi-Nada and Kobekou (Site CL5 in Fig. 2)

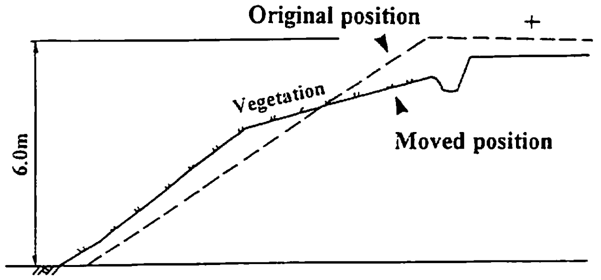


Fig. 14. Typical damaged embankment; embankment of freight branch along JR Tokaido Line between Higashi-Nada and Kobe-kou (Site EM1 in Fig. 2)

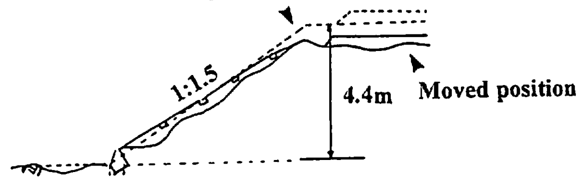


Fig. 15. Typical damage to embankment with facings; embankment along JR Sanyo Line between Hyogo and Shin-Nagata Stations (Site EM2 in Fig. 2)

ed using anchored sheet piles, in front of which either cantilever-type RC RWs or GRS-RWs have been constructed.

Figure 12 indicates the collapse of a cantilever-type RC RW which tilted largely similarly to the adjoining gravity-type RWs as mentioned before (Fig. 9). Figure 13 shows another case, which has been repaired by a combination of cut-off sheet piles and tie-rods driven in the embankment. After removing the backfill soil between the damaged RWs and the sheet piles, geogrid-reinforced embankment was placed. Finally the space between the damaged RWs and the geogrid-reinforced embankment was backfilled with concrete.

The seismic behaviour of the RC RW which was constructed most recently (in 1992) to support the embankment of JR Tokaido Line (Kobe Line) at Tanata site (site CL3 in Fig. 2), next to a GRS-RW (site GR1), will be discussed later in this paper.

DAMAGE TO EMBANKMENTS

Many of the railway embankments located in the affected areas suffered from large settlement and lateral movement (Fig. 14), accompanied by longitudinal tensile cracks on the top near the slope and along the slope surface. Figure 15 shows a typical damaged embankment with a facing consisting of a cast-in-place concrete lattice and precast concrete plates, which are used to protect the slope against heavy rainfall. The damage indicates that the facings were not effective in preventing the deformation of the embankment. Both the facing structures and the soil loosened on the slope surface were removed, and slopes which were reinforced with geotextile sheets were constructed. Similarly, many embankments with a facing of precast concrete plates slipped down along the slope surface. This damage was induced by the high seismic force applied to the facing itself.

GEOGRID-REINFORCED SOIL RETAINING WALLS

GRS-RWs had been constructed at four locations for 1990–1994 in the affected area (Fig. 1). They were aseismic-designed. This type of RW was introduced, for the first time, in the Design Standard for Railway Earth Structures approved in 1992 by the Ministry of Transport. The chronological order of construction was as follows:

a) Amagasaki, No. 1 (Fig. 16)

This wall has an average height of about 5 m for a total

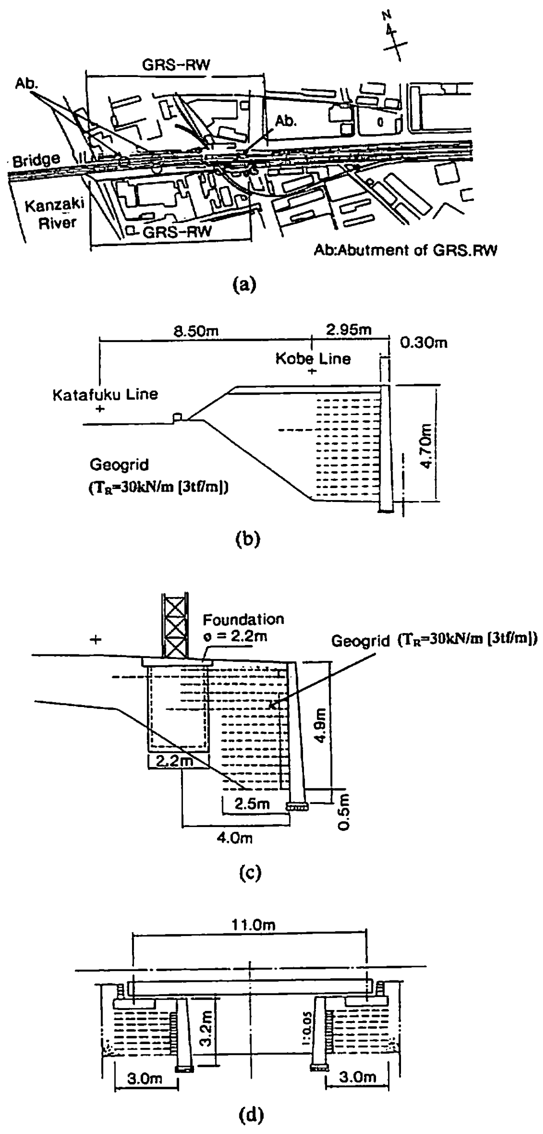
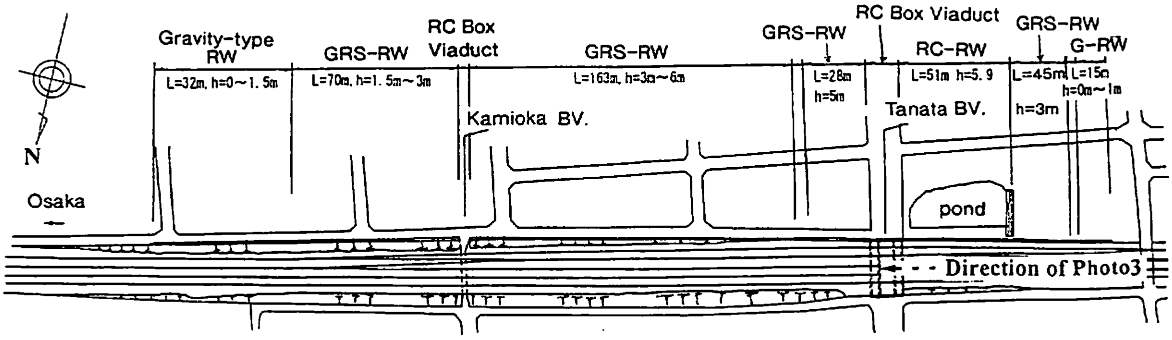


Fig. 16. (a) plan and (b), (c), (d) typical cross-sections of GRS-RWs for JR Tokaido Line, between Tsukamoto and Amagasaki Stations (*see* Fig. 1)

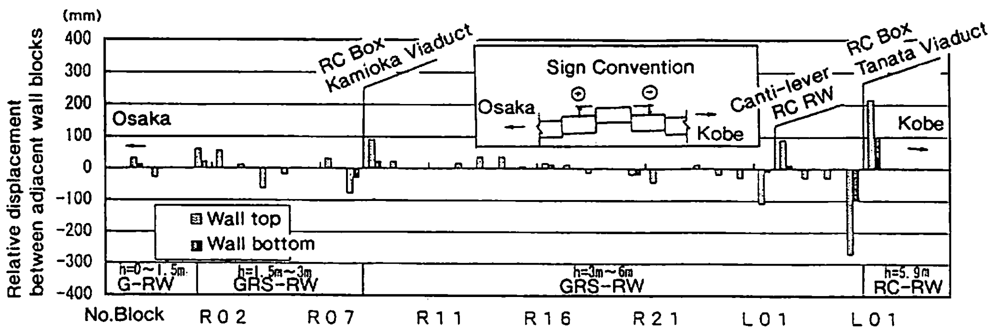
length of about 1 km. It was completed in April 1992 to support two new tracks that were added to the existing four tracks, on both sides of an existing railway embankment of JR Tokaido Line (Kanazawa et al., 1994). At some sections, foundations for a steel frame structure for electricity supply were constructed inside the reinforced zone (Fig. 16(c)). Four pairs of bridge abutments of GRS-RW were also constructed to support a bridge girder directly (Fig. 16(d)).

b) Tanata (Fig. 17)

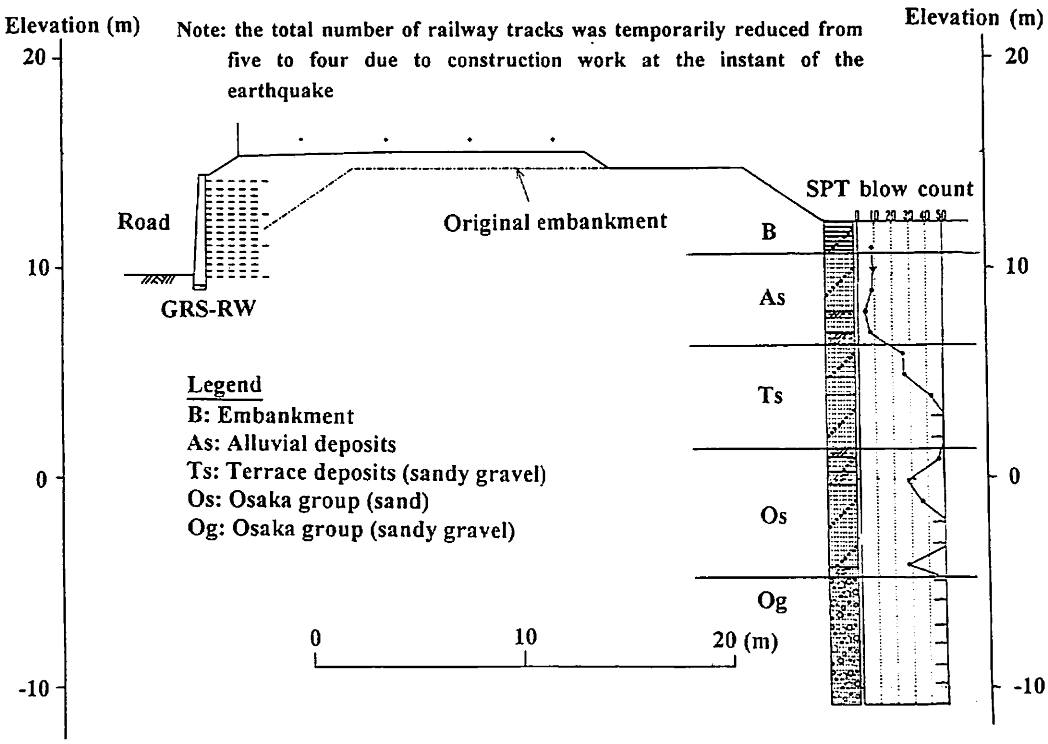
This was completed in February 1992 at Mori-Minami-



(a)



(b)



(c)

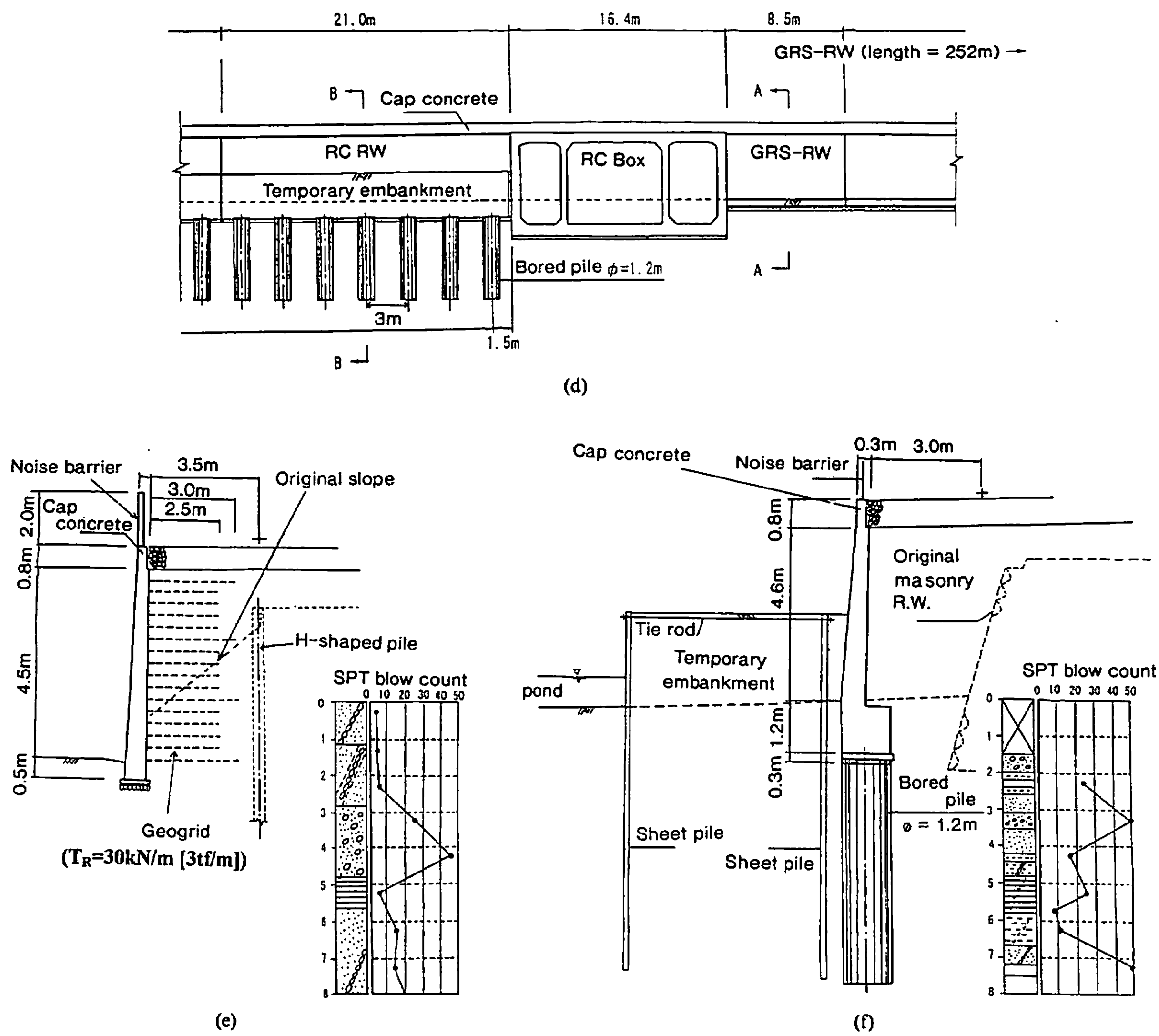


Fig. 17. (a) Plan of the site, (b) Relative displacement between adjacent facing sections of GRS-RW and cantilever-type RC RW; (c) Cross-section of embankment, (d) south view of GRS-RW and cantilever-type RC RW; (e) Typical cross-section of GRS-RW, and (f) typical cross-section of RC RW at Tanata (*see* Fig. 1, and sites CL3 and GR1 in Fig. 2(b))

cho 1-chome in Higashi-Nada-ku, Kobe City (the local name is Tanata) on the south slope of the existing embankment of JR Tokaido Line to increase the number of railway tracks from four to five (Fig. 17(c)) (at the time of the earthquake, the number of tracks was still four). The total length of the wall is 305 m, and the largest height is 6 m (Fig. 17(e)). In sections where the wall became higher than 1.5 m, a series of H-shaped steel piles with some temporary anchors were provided to retain the embankment before the portion of the slope was excavated for the construction of the GRS-RWs.

c) Maiko (Fig. 18)

The site is in Tarumi-ku, Kobe City, and the wall was completed in May 1993 in order to expand the top of the road next to one of the approach roads to Akashi Kaikyo (Strait) Bridge which is under construction. They are the only GRS-RWs for road.

d) Amagasaki, No. 2 (Fig. 19)

The site is west of the Amagasaki No. 1 GRS-RW, adjacent to Amagasaki station, completed in March 1994 for the JR Fukuchiyama Line to support a new approach fill to the bridge over a length of about 400 m with a height of 3 to 8 m.

These GRS-RWs were constructed as follows (Fig. 20; Tatsuoka et al., 1992): 1) A leveling pad was constructed. 2)–5) A wrapped-around wall was constructed to its full height by compacting each soil layer with the use of gabions filled with gravel placed on the shoulder of each soil layer. 6) A lightly steel-reinforced concrete facing was cast-in-place directly on the wrapped-around wall face so that the facing is firmly connected to the main body of the wall. A full-height rigid facing; 1) increases the stability of wall, 2) decreases the deformation of wall, in particular at the wall face and the backfill zone adjacent to the facing, 3) increases the durability of the wall

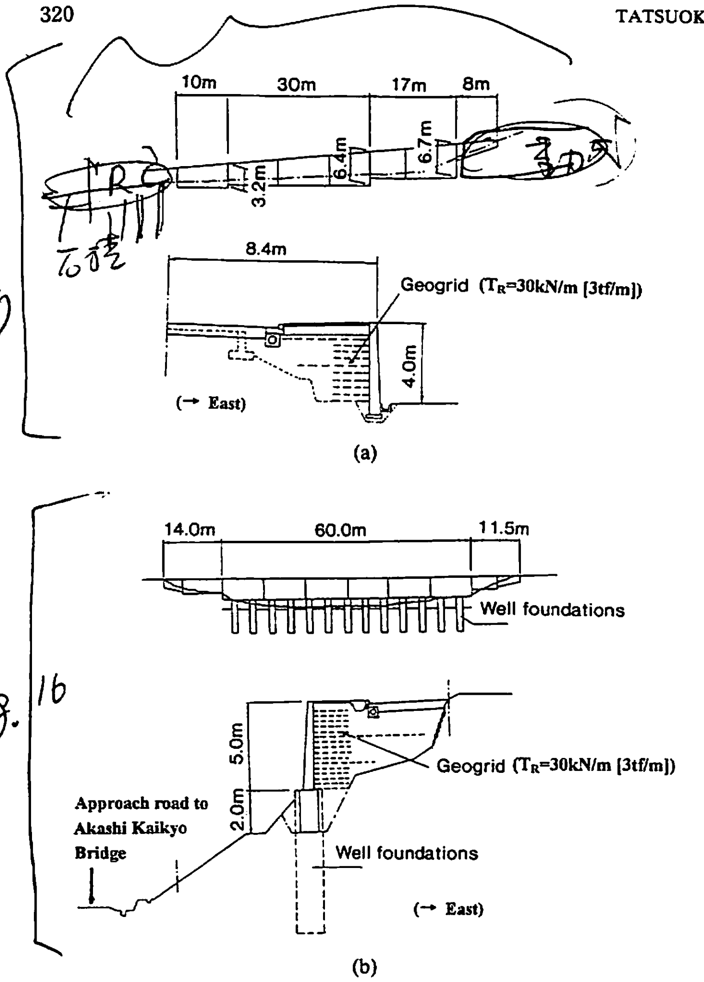


Fig. 18. (a) and (b) elevations and cross-sections of GRS-RWs at Maiko (*see* Fig. 1)

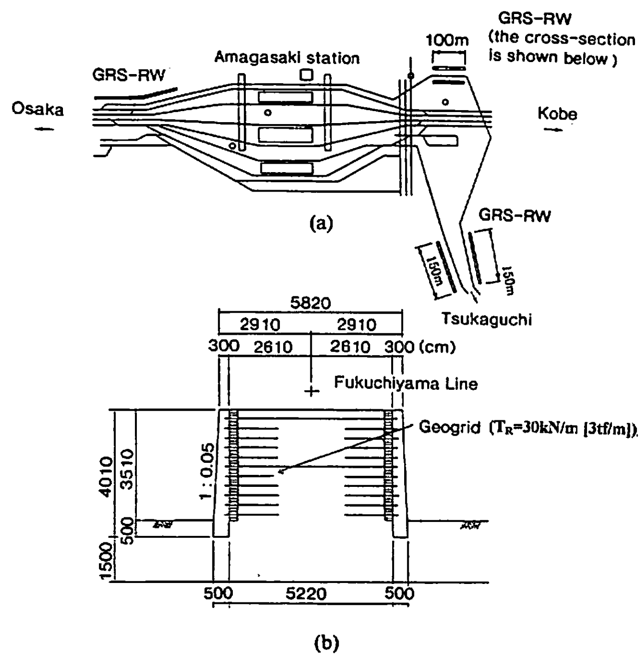


Fig. 19. (a) plan and (b) typical cross-section of GRS-RW adjacent to Amagasaki station of JR Fukuchiyama Line (*see* Fig. 1)

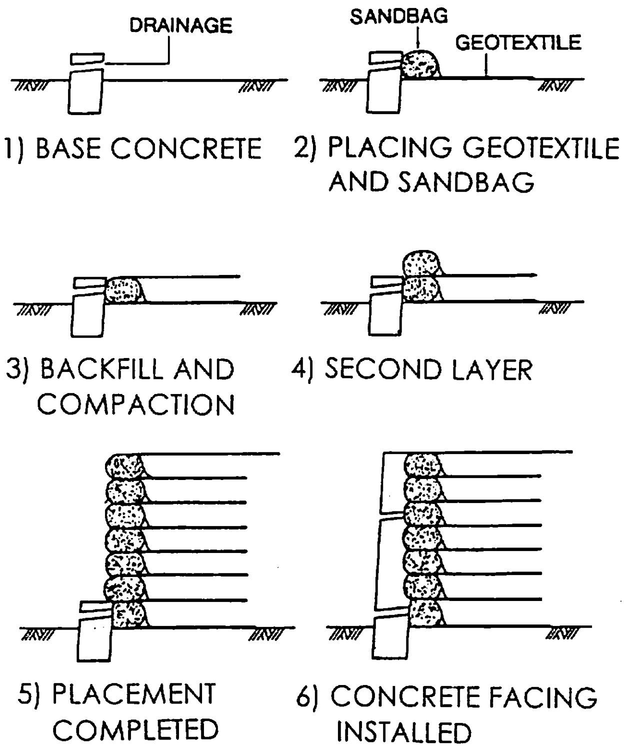


Fig. 20. Standard staged construction procedures for a GRS-RW having a full-height rigid facing

face, and 4) improves the aesthetics when compared with wrapped-around walls (Tatsuoka, 1993, Tatsuoka et al., 1994).

The reinforcement used for these GRS-RWs is a grid made of fibers of polyvinyl alcohol (the trademark is Vinylon) coated with soft PVC for protection, with a nearly rectangular cross-section of 2 mm times 1 mm and an opening of 20 mm. The nominal tensile rupture strength is 30 kN/m [3 tonf/m]. The back-fill soil is basically cohesionless soil including a small amount of fines.

GRS-RWs a), c) and d) were located in the areas where the Japanese seismic intensity scale was five or six. In these areas, some of the wooden houses, railway and highway embankments and conventional types of RW were seriously damaged. The degree of damage was, however, not as severe as that in the areas where the seismic intensity scale was equal to seven or higher (Figs. 1 and 2(b)). We cannot therefore conclude only from their good performance that this type of GRS-RW has a very high seismic stability.

BEHAVIOUR OF TANATA GRS-RW

The surface layer in the subsoil for foundation consists of relatively stiff terrace soils (Figs. 17(d) and (e)), located next to older gravelly fans in the east and north. This subsoil condition is, however, considered much better than that of a thicker Holocene clay deposit in the southern areas (Fig. 3).

This wall deformed and moved only slightly during the earthquake. Figure 17(b) shows the relative horizontal displacements between two adjacent facing sections at their top and bottom. The largest outward displacement occurred at the highest part of the wall which was in contact with a RC box culvert structure crossing the railway embankment (26 cm and 10 cm at the top of the wall and at the ground surface level respectively) (Photo. 1). The wall moved outward at the bottom an average of about 5 cm relative to the supporting foundation subsoil, and pushed the soil in front of the wall laterally. Associated with this movement, the railway track located above the reinforced zone of the backfill settled a maximum of about 15 cm. This value was not particularly large when compared with that observed at the other three tracks located on the unreinforced zone of the embankment (Fig. 17(c)). It seems that the settlement was mainly due to dynamic compaction of the main embankment and ballast.

Despite the deformation and movement of the wall as described above, the performance of the GRS-RW wall was considered quite satisfactory based on the following factors;

a) Extra-ordinarily High Seismic Intensity at Tanata Site

The peak ground acceleration at the Motoyama First Primary school, which is about 1 km west of Tanata, was extremely high (Fig. 1), which can be inferred also from a very high collapse rate of the Japanese wooden houses at the site (*see* Fig. 21). The wooden houses which collapsed

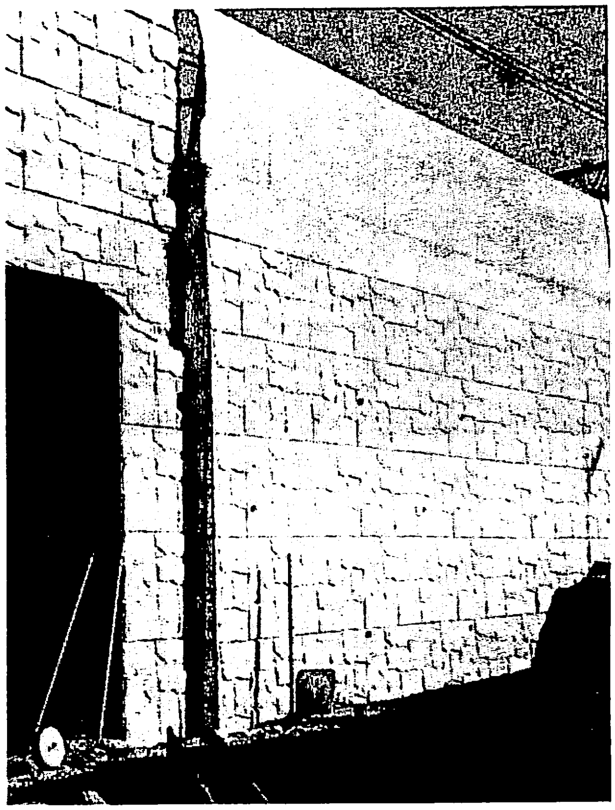


Photo. 1. Displacement of the Tanata GRS-RW relative to a RC box culvert

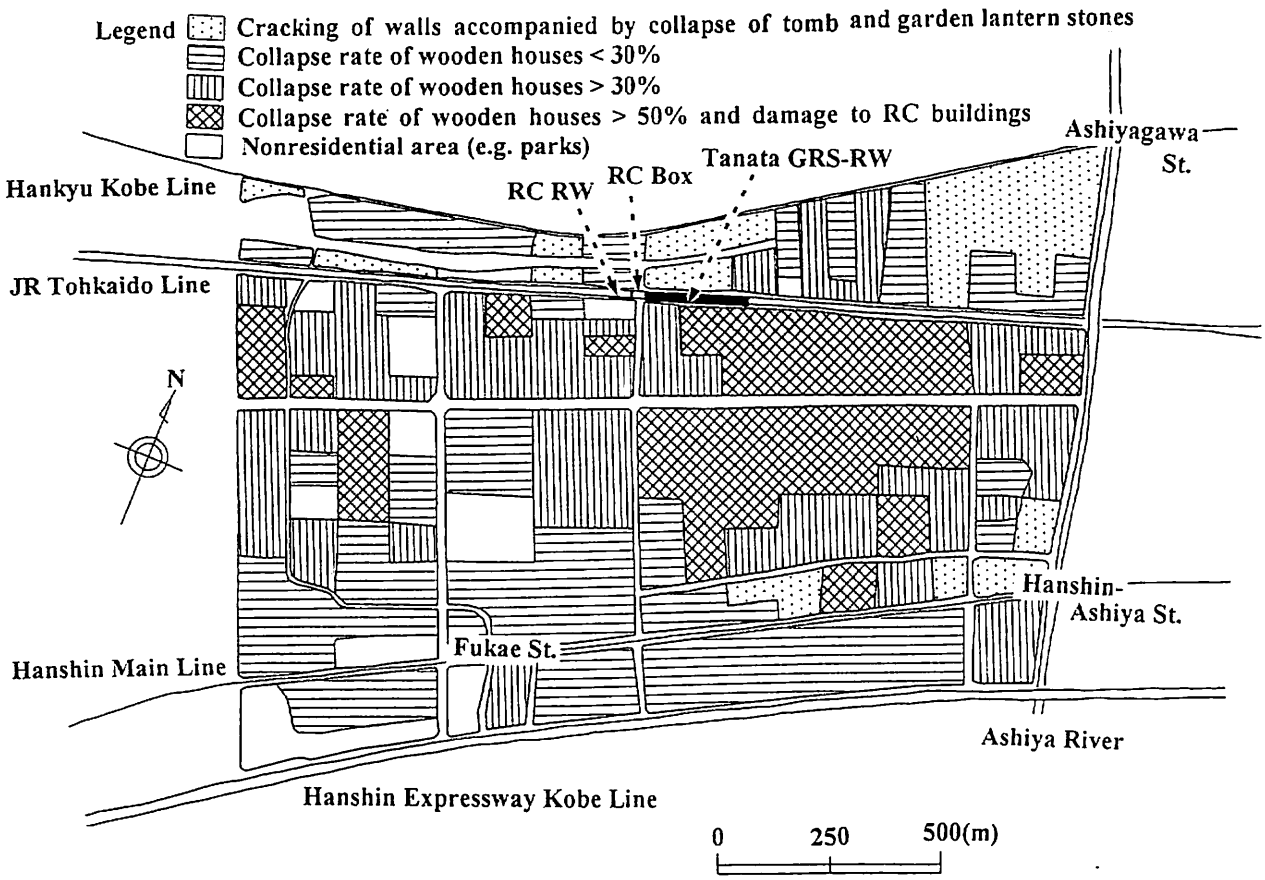


Fig. 21. Details of the damage to wooden houses in the area in front of the Tanata GRS-RW

Photo. 2. View in front of the GRS-RW at Tanata

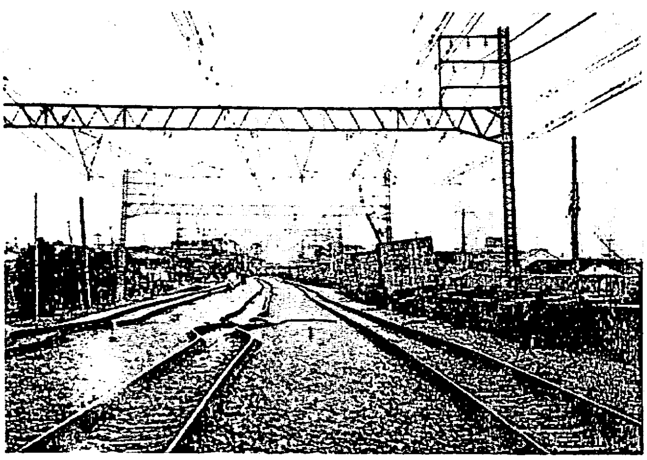


Photo. 3. View along the crest of the embankment at Tanata site

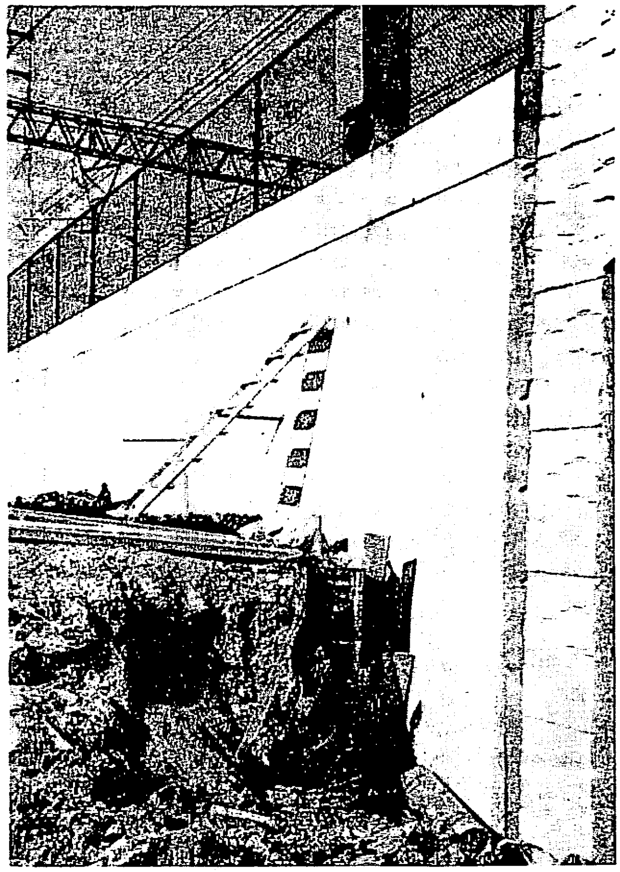


Photo. 4. Displacement of the RC RW at Tanata relative to a RC box culvert

totally are not necessarily old, and many were constructed less than about ten years ago. Photograph 2 shows a view in front of the GRS-RW. Photograph 3 shows the top of the embankment taken from the direction indicated in Fig. 17(a). Considerable distortion of the continuous welded rail as seen in this picture also indicates an extraordinary severe shaking at the site. In the area surrounding the Tanata site where the seismic intensity was estimated to be the same, or even less severe, the damage to many RC buildings and columns of highway and railway elevated RC frame structures was serious. In particular, the damage to many gravity-type RWs and cantilever-type RC RWs for railway structures was considered quite serious. It is definite that this GRS-RW experienced the highest seismic load among the other modern GRS-RWs.

b) Comparable Performance of an Adjacent RC RW (Fig. 17(e))

On the side opposite the GRS-RW of the RC box structure, a RC RW with the largest height of about 6 m had been constructed concurrently with the GRS-RW, supported by a very good foundation of a row of bored piles. Although the subsurface conditions for the RC RW (Fig. 17(f)) were similar to those for the GRS-RW (Fig. 17(e)), it was decided to construct a pile foundation for the RC RW considering the relatively high water table. The GRS-RW was not supported by such a pile foundation. Consequently, the construction cost per wall length of the RC RW was nearly double or triple that for the GRS-RW. In addition, a temporary cofferdam still existed at the time of the earthquake in front of the RC RW, which may have somewhat contributed to the stability of the RC-RW during the earthquake.

Despite the differences described above, the RC RW displaced similarly to the GRS-RW; i.e., at the interface with the side of the RC box structure, the outward lateral displacement was 21.5 cm at the top and 10 cm at the ground surface level (Fig. 17(b) and Photo. 4).

c) Shortest Reinforcement

The length of geogrid reinforcement for this type of GRS-RW is in general shorter than that for the metal strip-reinforced soil RWs or other type of GRS-RWs. This results from the greater pull-out resistance of the geogrid and the contribution of a full-height rigid facing to the wall stability. The Design Standard for Railway Earth Structures (Railway Technical Research Institute, 1992) specifies the minimum allowable length of grid reinforcement for the GRS-RW system as the larger value of either 35% of the wall height or 1.5 m. For most of the

GRS-RWs constructed so far, for conservatism, several top reinforcement layers have been made longer than the others at lower levels (Figs. 16(a), 17(a) and 17(b)). For the Tanata GRS-RW, unfortunately, the length of all the reinforcement layers were installed to nearly the same length (Fig. 17(e)) due to the construction restraint of the wall being constructed while allowing trains to run in the area where the top several reinforcement layers should be extended. This arrangement may have reduced the seismic stability of the wall; the titling of the wall would have been smaller if the several top grid layers had been longer.

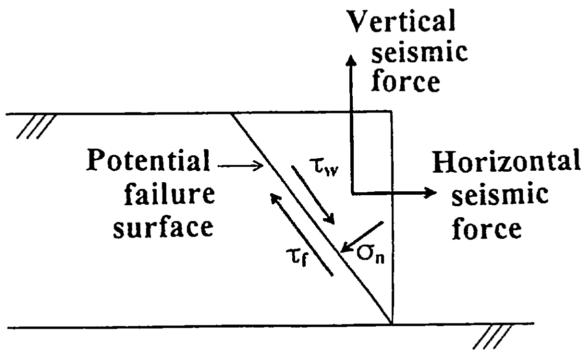


Fig. 22. Stresses in backfill responding to seismic force

DISCUSSION

a) Origin of the High Seismic Stability of GRS-RW

The principal mechanism which makes the GRS-RWs much more stable during an earthquake than the conventional gravity-type RWs is that the reinforced zone together with a rigid facing can behave as a monolith having a width/height ratio much larger than that of the gravity type RWs. The horizontal outward seismic force increases the disturbing shear stress τ_w working along the potential failure plane (Fig. 22). In an unreinforced backfill, the normal stress σ_n on the potential failure surface decreases at the same time, which leads to a reduction in the soil shear strength $\tau_f = \sigma_n \cdot \tan\phi$. The upward seismic force reduces the value of τ_w, but at the same time it reduces the value of σ_n. Therefore, without horizontal seismic force, the net effect on the ratio τ_f/τ_w depends on many factors. With a given horizontal seismic force, the upward seismic force reduces the ratio of τ_f/τ_w. In a reinforced backfill, the increase in τ_w due to horizontal seismic force is resisted directly and the reduction in σ_n due to the horizontal seismic force is restrained by the tensile force mobilized in the reinforcement. The reduction in σ_n due to the vertical seismic force may also be restrained to some extent by a full-height rigid facing. Accordingly, a reinforced soil mass could perform like a monolith, which is much more ductile than a RC structure.

b) Aseismic Design

Figure 23(a) shows the design method for RC RWs such as the one at the Tanata site. Seismic earth pressure calculated by the Mononobe-Okabe method using a horizontal seismic coefficient $k_h = 0.2$ is resisted by the lateral and rotational resistance of the pile foundation, which results mainly from passive earth pressure in the subsurface soil in front of the pile foundation. It can be seen that some lateral displacement of the facing is inevitable for a high seismic load, since the mobilization of high passive earth pressure requires a relatively large soil deformation.

The seismic stability of GRS-RWs is evaluated by the two-wedge method (Figure 23(b); Horii et al., 1994). Horizontal seismic force is applied to the facing and backfill soil using $k_h = 0.2$. The seismic force is resisted mainly by the tensile force in the reinforcement and partly by the reaction force at the bottom of the facing. Sliding at the base of the facing and the reinforced zone of the back fill and the overturning of the facing and the active zone F as a monolith are also examined. To prevent sliding of the base, several bottom reinforcement layers should be long enough. This precaution should be applied equally to cantilever-type RC RWs without a pile foundation.

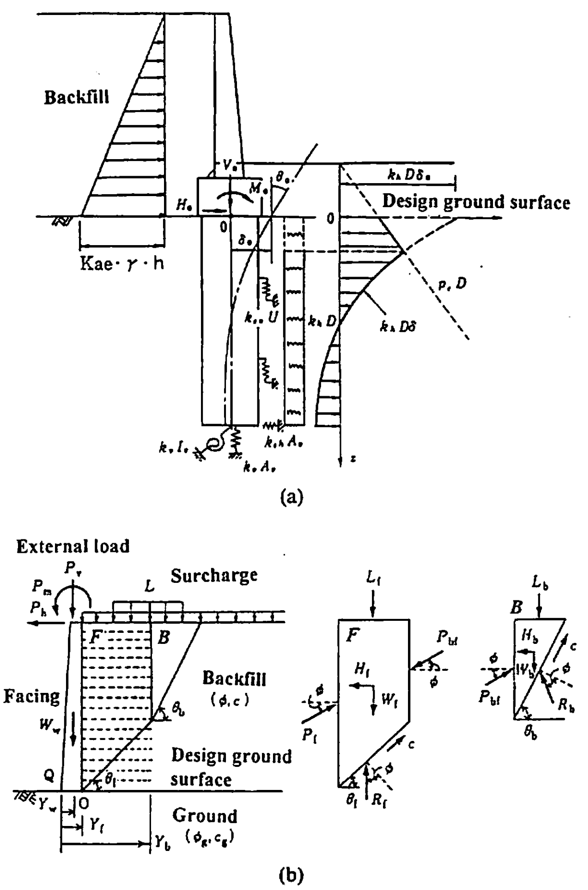


Fig. 23. Current seismic design methods for (a) RC RWs and (b) GRS-RWs, based on the design code for railway structures

The tilting of the GRS-RW resulted from the shear deformation of the backfill. It seems that the facing and reinforcement layers did not contribute directly to decreasing this deformation since they are oriented in the direction of zero-extension in the shear deformation. However, they should have contributed indirectly by maintaining the confining pressure in the backfill as discussed above, which resulted in the maintenance of the original shear modulus of the soil. Accordingly, shorter reinforcement, in general, leads to larger shear deformation of the backfill, resulting in larger titling of the wall face. To prevent this, the use of several longer top reinforcement layers is effective.

It is not clear why the Tanata GRS-RW survived despite the use of $k_h=0.2$ in the design, which is certainly much lower than the maximum horizontal acceleration divided by the gravitational acceleration at the site. It could have resulted from the high ductility of the GRS-RW and some added conservatism, which may include a) an under-estimated backfill soil shear strength, and b) no consideration of passive earth pressure in front of the facing in the seismic design. No consideration of the effects of H-shaped steel piles remaining behind the reinforced zone would be another factor, but its effect would be small. The stability of the wall was controlled mainly by the horizontal seismic force. However, the effect of vertical acceleration cannot be ignored although it was not considered using the present design code. For example, vertical upward acceleration decreases the safety factor against both sliding at the wall base and overturning of the facing and the reinforced backfill as a monolith.

c) Full-height Rigid Facing

A very thin crack with a width of about 2 mm appeared at the mid-height of the highest wall section, which may indicate some bulging deformation mode of the facing. It is certain that a full-height rigid facing is better than a facing of discrete panels in reducing this mode of deformation. Under the same seismic conditions, however, the stresses in the facing of GRS-RW should be much smaller than those in the facing of a cantilever-type RC RW, since the facing of a GRS-RW is supported at many levels by geogrid layers, which results in a very short continuous beam span (Tatsuoka, 1993).

d) No Further Deformation of the Tanata GRS-RW Expected

The Tanata GRS-RW is a nearly self-standing structure. Therefore, when the reinforcement was essentially undamaged, it could be expected that the wall would not exhibit further deformation under ordinary static load conditions. This assumption is supported by the following test: A 5 m high full-scale proto-type GRS-RW model wall was constructed at Kunitachi by Railway Technical Research Institute. The wall was loaded to failure at the crest using a footing having a base width of 2 m × 3 m with a set back of 2 m from the wall face (Tatsuoka et al., 1992). With the largest average footing pressure as high as 600 kN/m^2 [6.0 kgf/cm^2], the average footing settlement was 70 cm, which resulted in a lateral outward movement of 40 cm at the top of the facing. The facing, which was not steel-reinforced, ruptured at the mid-height construction joint. After the load test, the wall was left without any repair for about three years, but it did not exhibit any further noticeable deformation.

e) Interface with the RC Box Culvert

The behaviour of both RC RW and GRS-RW at Tanata site indicated that the interface with the RC box culvert was a structural weak point. An increase in the shear resistance at the interface may be effective in reducing relative lateral displacement and settlement between the backfill and the RC box structure.

CONCLUSIONS

Different types of soil retaining walls (RWs) located in severely shaken areas during the 1995 Hyogoken-Nambu earthquake performed differently. In general, older RWs were damaged more seriously, while gravity-type RWs showed a very low stability against strong seismic shaking. In addition, many cantilever-type reinforced concrete RWs behaved poorly.

A geogrid-reinforced soil retaining wall constructed in 1992 at Tanata performed very well despite the fact that the site was in one of the most severely shaken areas. Other GRS-RWs also performed very well. Based on these experiences, many damaged conventional gravity-type RWs were reconstructed as GRS-RWs.

The performance of geogrid-reinforced soil retaining wall will foster future confirmation and development of aseismic design procedures.

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