Performance of Soil Retaining Walls during the Great Hanshin-Awaji Earthquake

by

Tatsuoka,F.[1], Koseki,J.[2], Tateyama,M.[3], and Horii, K.[4]

ABSTRACT

During the Great Hanshin-Awaji Earthquake of January 17, 1995, many conventional masonry and unreinforced concrete gravity-type retaining walls totally collapsed, while many modern cantilever reinforced concrete retaining walls also were seriously damaged. Geogrid-reinforced soil retaining walls having a full-height concrete facing with a total length about 2km performed very well, and it is of particular importance that a wall located in one of the most severely shaken areas deformed only slightly.

INTRODUCTION

Around 5:46 a.m. on the 17th of January, 1995, a devastating earthquake measuring 7.2 in the Richter scale hit the southern part of Hyogo Prefecture (part of Hanshin district), including Kobe City and the neighbouring urban areas (**Fig. 1**). **Fig. 2a** shows the areas where the Japanese seismic intensity scale was seventh or higher estimated from a collapse ratio of wooden houses equal to 30 % or more. In addition to the vast damage to wooden houses, many important civil engineering structures, including steel-reinforced concrete (RC) frame structures (in particular their columns) supporting highways and railways, RC buildings, subway structures and port and harbour structures, were seriously damaged.

Extensive soil liquefaction took place in uncompacted reclaimed lands. As far as the authors know, however, the damage to several types of retaining walls (RWs) was not triggered by soil liquefaction in the supporting ground or the backfill, but it was induced by high seismic forces applied to the wall structure and the backfill.

Fig. 2b shows geographical and geological classification of the area corresponding to Fig. 2a, and **Fig. 3** shows a typical geological cross-section in the N-S direction, the location of which is indicated in Fig. 2. A relatively thin top Holocene soil layer (denoted by As) is underlain by terrrace deposits of the later Pleistocene Epoch (Ts), which is underlain by middle Pleicetoce deposits of Osaka Group (Os-g). The thickness of these Holocene and later Pleistocene deposits decreases to the north.

As seen from Figs. 2a and 2b, the foundation ground of the areas where the collapse rate of wooden houses was highest is mainly 1) later Pleistocene terrace deposits (Ts) located between Holocene sandy and gravelly alluvial fans at relatively higher elevations, and 2) Holocene clayey deposits at the lowest elevations, located south of the fan and terrace deposits. Note, however, 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 Tohkaido Line, where the surface soil is alluvial fan deposits consisting of mainly gravel and sand (**Fig. 4**).

[1]Professor, Institute of Industrial Science, University of Tokyo, Tokyo, Japan,
[2]Associate Professor, ditto,
[3]Railway Technical Research Institute, Tokyo, Japan,
[4]Chuo Kaihatsu Corporation, Tokyo, Japan

In the affected area, an extensive length of embankment had been constructed long time ago for JR Tohkaido Line of West Japan Railway (JR) Company, which is 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 them was replaced by elevated RC frame structures, which were seriously damaged by this earthquake. The railway embankments existing at the moment of this earthquake had a number of RWs of old type in these high seismic intensity areas (Fig. 2a), and most of them were seriously damaged. On the other hand, only a very limited length of road embankment existed in the affected area, while only some RWs were damaged.

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

1) Masonry RWs,
2) Leaning-type (supported type) unreinforced concrete RWs, and
3) Gravity-type unreinforced concrete RWs.

These three conventional types of RW were most seriously damaged. RWs of types 1 and 2 are not allowed to be constructed according to the current design code for railway structures (Design Standard for Railway Structures).

4) Cantilever-type or inverted T-shaped type steel-reinforced concrete (RC) RWs, to which the damage was on average less serious than that to the above, but many were seriously damaged to be demolished.
5) Geogrid-reinforced soil retaining walls (GRS-RWs), to which the damage was practically none.

This paper will attempt to describe some typical case records of the performance of the RWs for railway embankments to obtain some lessons from them, but it will not cover all the case histories. It will be shown that in general older RWs were damaged more seriously.

This paper also describes good performance of geogrid-reinforced soil retaining walls (GRS-RWs) for railway embankments. Among them, one constructed in 1992 at Tanata site (Fig. 1) was damaged only slightly, although it was located in one of the most severely shaken area. At several sites, other types of RWs which were seriously damaged have been removed, and in place of them, GRS-RWs will be constructed (as of the end of March, 1995).

DAMAGE TO CONVENTIONAL TYPES OF RETAINING WALLS

Masonry RWs: This is the oldest type. Most of them were constructed more than about 70 years ago. No seismic design was performed. They were on 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. **Fig. 5** shows a typical case, constructed 64 years ago. A stack of stones totally collapsed into stone pieces.

Leaning-type unreinforced concrete RW: Most of them were constructed more than about 60 years ago. No seismic design was performed. **Fig. 6** shows a typical one, constructed 58 years ago to support the embankment for JR Tohkaido Line. Continuously for a large length, the RW was broken at the bottom and the upper part overturned completely to the ground showing the back face upside, which was perhaps triggered by both large horizontal seismic force worked to the RW itself and large seismic earth pressure exerted from the backfill. It seems, however, that the former should be the major factor for complete overturning, since the backfill soil did not move outwards following the movement of the RW.

Fig. 7 shows another RW of this type with a height of about 8 m, constructed 57 years ago on the both sides of a railway embankment. They largely tilted outwards with 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 constructed on the crest of the RW. They were totally demolished while removing the backfill soil in between.

Gravity-type unreinforced concrete RW: Most of them were constructed more than about 60 years ago. **Fig. 8** shows the case of most serious damage, in which a length of RW tilted largely (Fig. 8b), while some section totally over-turned for a length of 200 m (Fig. 8c). These RWs were constructed 66 years ago based on the standard design, where pseudo-static stability analysis was adopted using a horizontal seismic coefficient of 0.2. It seems that their complete over-turning was caused by large horizontal seismic force worked to them. These RWs have been demolished. At many other sites, the RWs of this type more-or-less tilted outwards considerably, which resulted in a large settlement at the crest of the railway embankment.

These three types of RWs described above are designed so that the gravity resistance of the RW be large enough to resist against the lateral disturbing earth pressure exerted from the backfill. The damaged cases shown above and others indicate that these types of RWs had a very low seismic stability against the seismic forces actually expected. It seems that even without seismic earth pressure applied to the back face, some of these RWs would have tilted or even completely over-turned. Construction of masonry or leaning-type RWs for important civil engineering structures is, therefore, not suggested (as specified in the current design standard for railway structures). A gravity-type unreinforced concrete RW having a very wide bottom would have been stable during this level of earthquake, but it is not practical.

Cantilever and inverted T-shaped RC RWs: This is a rather modern type. They were aseismic designed (as described later). **Fig. 9** shows a typical damaged cantilever RC RW, constructed about 30 years ago, without using a pile foundation. The RW largely tilted outward inducing a large settlement at the crest of railway embankment. The footpath in front of the RW was pushed out laterally by this wall movement.

Shin-Nagata station of JR Sanyo Line was constructed about 30 years ago atop an embankment with the both sides being supported by a slope and this type of RW for a total length of about 800 m (**Fig. 10a**). Most of these RWs tilted and slid out at the bottom, but the most serious damage is cracking in the facing (**Fig. 10b**). It seems that this type of failure can be explained only by extra-ordinary large seismic earth pressure which resulted 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 (**Fig. 10c**). The previous cantilever RC RWs which were seriously damaged have been removed, and GRS-RWs are planned to be constructed in place of them, while relatively lightly damaged ones were repaired by anchoring from one side to the other side.

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

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. One of them at Maiko is for road embankment, while the others are directly supporting railway tracks of Japan Railway (JR) company. This type of RW was introduced, for the first time, into Design Standard for Railway Structures approved in 1992 by

the Ministry of Transport. In the order of construction time, they are:
a) Amagasaki, No. 1 (**Fig. 11**): This has an average height of about 5 m for a total length of about 1 km, completed in April 1992 to support two new tracks, added to the existing four tracks, on both slopes of an existing railway embankment of JR Tohkaido Line (Kanazawa et al. ,1994). At some sections, foundations for a steel frame structure for electricity supply were constructed inside the reinforced zone (Fig. 11c). Four pairs of bridge abutments of GRS-RW, which are supporting directly a bridge girder, were also constructed (Fig. 11d).
b) Tanata (**Fig. 12**): This was completed in February 1992 at Mori-Minami-cho 1-chome in Higashi-Nada-ku, Kobe City (Tanata in the local naming) on the south slope of the existing embankment of JR Tohkaido Line to increase the number of railway tracks from four to five (Fig. 12c). The wall is 305 m in total length with the largest height of 6 m (Fig. 12e). At sections 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 some part of the slope was excavated.
c) Maiko (**Fig. 13**): The site is in Tarumi-ku, Kobe City, completed in May 1993 for expanding the crest of one of the approach roads to Akashi Kaikyo (Strait) Bridge under construction.
d) Amagasaki, No. 2 (**Fig. 14**): The site is west of the Amagasaki No. 1 GRS-RW, adjacent to Amagasaki station, completed in March 1994 to support a new approach fill to the bridge for a length of about 400 m with a height of 3 - 8 m for JR Fukuchiyama Line.

These GRS-RWs were constructed as follows (**Fig. 15**; Tatsuoka et al., 1992): 1) A leveling pad is constructed. 2) A wrapped-around wall is constructed to its full height by compacting each soil layer with a help of gabions filled with gravel that are placed on the shoulder of each soil layer. 3) A lightly steel-reinforced concrete facing is cast-in-placed directly on a 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 face, and 4) improves the aesthetics when compared with wrapped-around walls (Tatsuoka, 1993, Tatsuoka et al., 1994).

The reinforcement used for those 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 aperture of 20 mm. The nominal tensile rupture strength is 3 tonf/m. The back-fill soil is basically cohesionless soil including some amount of fines.

GRS-RWs a), c) and d) were located in the areas where the Japanese seismic intensity scale was 5th or 6th. In these areas, some of 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 of the seismic intensity scale equal to 7th or higher (Figs. 1 and 2b). Therefore, we cannot 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 soil layer consists of relatively stiff terrace soils (Figs. 12d and e), located next to older gravelly fans in the east and north. Yet, this ground condition is much better than that of a thicker Holocene clay deposit in the southern areas (Fig. 3).

This wall deformed and moved slightly during the earthquake. Fig. 12b shows the relative horizontal displacements between two adjacent facing sections at their top and bottom. The largest outward displacement occurred at the tallest part, in contact with a RC box culvert structure crossing the railway embankment, which was 26 cm and 10 cm at the top of the wall and at the ground surface level (**Plate 1**). The wall moved outward at the bottom by about 5 cm on average relative to the supporting ground, while pushing laterally the soil in front of the wall. Associated with the above, the railway track located above the reinforced zone of the backfill settled down about 15 cm at largest. This value was not particularly large when compared with that of the other three tracks located on the unreinforced zone of the embankment (Fig. 12c). It seems that the settlement due to the dynamic compaction of the embankment body and ballast was also very large.

Despite the deformation and movement of the wall described above, we consider that the performance of the GRS-RW wall is highly satisfactory when considering the following factors;

a) Extra-ordinarily high seismic intensity at Tanaka site: The peak ground acceleration at 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 Japanese wooden houses at the site (see **Fig. 16**). The totally collapsed wooden houses are not necessarily old, but many were constructed less than about ten years ago. **Plate 2** shows a scene in front of the GRS-RW. **Plate 3** shows the crest of the embankment taken from the direction indicated in Fig. 12a. Considerable distortion of the continuous welded rail as seen in this picture also indicates extra-ordinarily severe shaking at the site. In the area surrounding Tanaka site where the seismic intensity was estimated similar, 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 RC RWs for railway structures was uncomparably more serious. It is certain that this GRS-RW experienced the highest seismic load among other modern GRS-RWs.

b) Comparable performance of an adjacent RC RW (Fig. 12e): On the side opposite to the GRS-RW of the RC box structure, a RC RW with a 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 ground condition for the RC RW (Fig. 12f) is similar to that for the GRS-RW (Fig. 12e), it was decided to construct this pile foundation for the RC RW considering a relatively high water table. On the other hand, the GRS-RW is not supported by such a pile foundation. Consequently, the construction cost per wall length of the RC RW became nearly double as high as that for the GRS-RW. Besides, a temporary cofferdam still existed in front of the RC RW, which may 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. 12b and **Plate 4**).

c) Shortest reinforcement: The length of geogrid reinforcement for this type of GRS-RW is in general shorter than that of metal strip-reinforced soil RWs. This results from much better pull-out resistance of grid and the contribution of a full-height rigid facing to the wall stability. Design Standard for Railway Earth Structures (1992) specifies the minimum allowable length of grid reinforcement for the GRS-RW system as the larger of 35 % of the wall height and 1.5 m. For most of the GRS-RWs constructed so far, for conservatism, several top reinforcement layers were made longer than the others at lower levels (Figs. 11a, 12a and 12b). For the Tanata GRS-RW, unfortunately the length of all the reinforcement layers were truncated to nearly a same length (Fig. 12e), due to such a construction restraint as that the wall should be constructed while trains were running on the area to which 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.

DISCUSSIONS

a) Origin of the high seismic stability of GRS-RW: The principal mechanism which makes the GRS-RWs much more stable during 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. Namely, horizontal outward seismic force increases the shear stress τ_w working along the potential failure plane (**Fig. 17**). 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 $\tau_f = \sigma_n \cdot tan\ \phi$. Upward seismic force reduces the value of τ_w, but at the same time it reduces the value of σ_n. Therefore, the net effect depends on many factors. On the other hand, in a reinforced backfill, the reduction in σ_n due to horizontal seismic force is restrained by tensile force mobilized in the reinforcement. The reduction in σ_n due to vertical seismic force may also be restrained to some extent by a full-height rigid facing.

b) Aseismic design: **Fig. 18a** shows the design method for RC RWs such as the one at 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 the passive earth pressure in the ground in front of the pile foundation. It may be understood that some lateral displacement of the facing is inevitable for high seismic load, since the mobilization of high passive earth pressure needs relatively large soil deformation.

On the other hand, the seismic stability of GRS-RWs is evaluated by the two wedge method (**Fig. 18b**; 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 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 is also examined. To prevent base sliding, the several bottom reinforcement layers should be long enough. This caution is applied equally to cantilever RC RWs without a pile foundation.

The tilting of the GRS-RW resulted from the shear deformation of the backfill soil. It seems that the facing and reinforcement layers did not contribute directly to decrease 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 soil. Accordingly, shorter reinforcement, in general, leads to larger shear deformation of the backfill soil, resulting in larger titling of the facing. To prevent this, the use of longer reinforcement at all the levels is effective, but the use of long several top reinforcement layers would be sufficient.

We should understand why the Tanata GRS-RW survived despite the use of k_h = 0.2, which is certainly much lower than the maximum horizontal acceleration at the site. It would have resulted from hidden 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 in back of the reinforced zone would be another factor, but its effect would be small. The stability of the wall is controlled mainly by the horizontal seismic force. Yet the effect of vertical acceleration cannot be ignored although it is not considered in the present design code. For example, vertical upward acceleration decreases the safety factor against the sliding at the wall base.

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 budging 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. On the other hand, under the same seismic conditions, the stresses in the facing of GRW-RW should be much smaller than those in the facing of a cantilever RC RW, since the facing of a GRS-RW is supported at many levels by geogrid layers, which results into a very short span for a continuous beam (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 is essentially undamaged, it can be expected that the wall will not exhibit further deformation under ordinary static load conditions. This infer is supported by the following fact. A 5 m high full-scale proto-type GRS-RW model wall was constructed at Kunitachi by Railway Technical Research Institute. The wall was brought to failure by loading at the crest using a footing having a base of 2 m x 3 m wide with a set back of 2 m from the wall face (Tatsuoka et al., 1992). At the largest average footing pressure applied of as large as 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 loading test, the wall have been left without any repair for about three years, but it did not exhibit any further deformation.

e) Interface with the RC box culvert: The behaviour of both RC RW and GRS-RW at Tanata site showed that the interface with the RC box culvert was a structural week point. The 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 Great Hanshin-Awaji earthquake performed in different ways. In general, older RWs were damaged more seriously, while gravity-type ones showed a very low stability against strong seismic shaking. Even many cantilever reinforced concrete RWs behaved poorly.

A geogrid-reinforced soil retaining wall constructed in 1992 at Tanata performed very well despite that the site was in one of the most severely shaken areas. Other GRS-RWs also performed very well.

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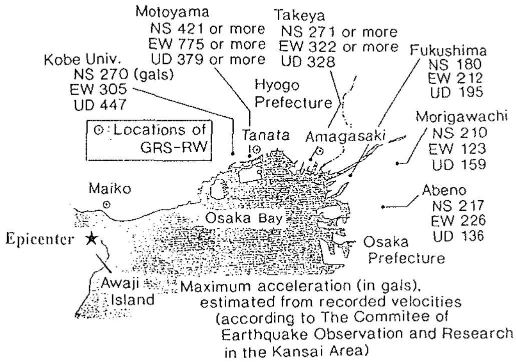

Fig. 1 Epicenter and recorded maximum acceleration values

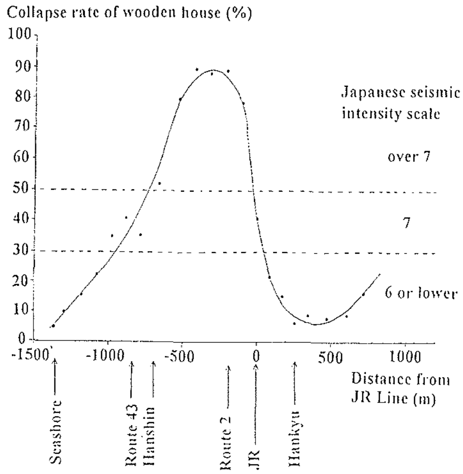

Fig. 4 Collapse rate of wooden houses along Line 12 shown in Fig. 2

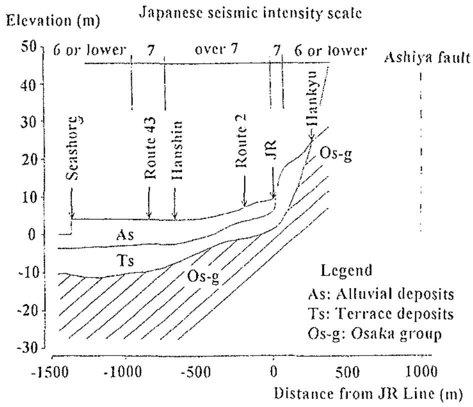

Fig. 3 Geological cross-section in the NS direction, Line 12 shown in Fig. 2

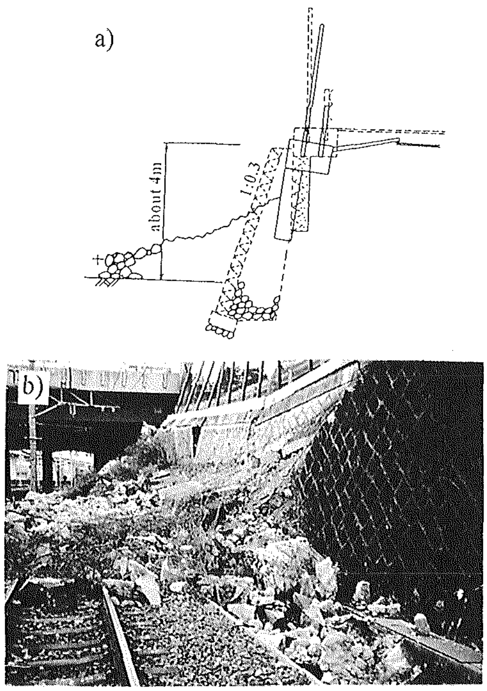

Fig. 5 Typical damaged masonry RW; embankment along JR Tohkaido Line between Setsu-Motoyama and Sumiyoshi Stations (Site MS1 in Fig. 2); a) sketch and b) picture

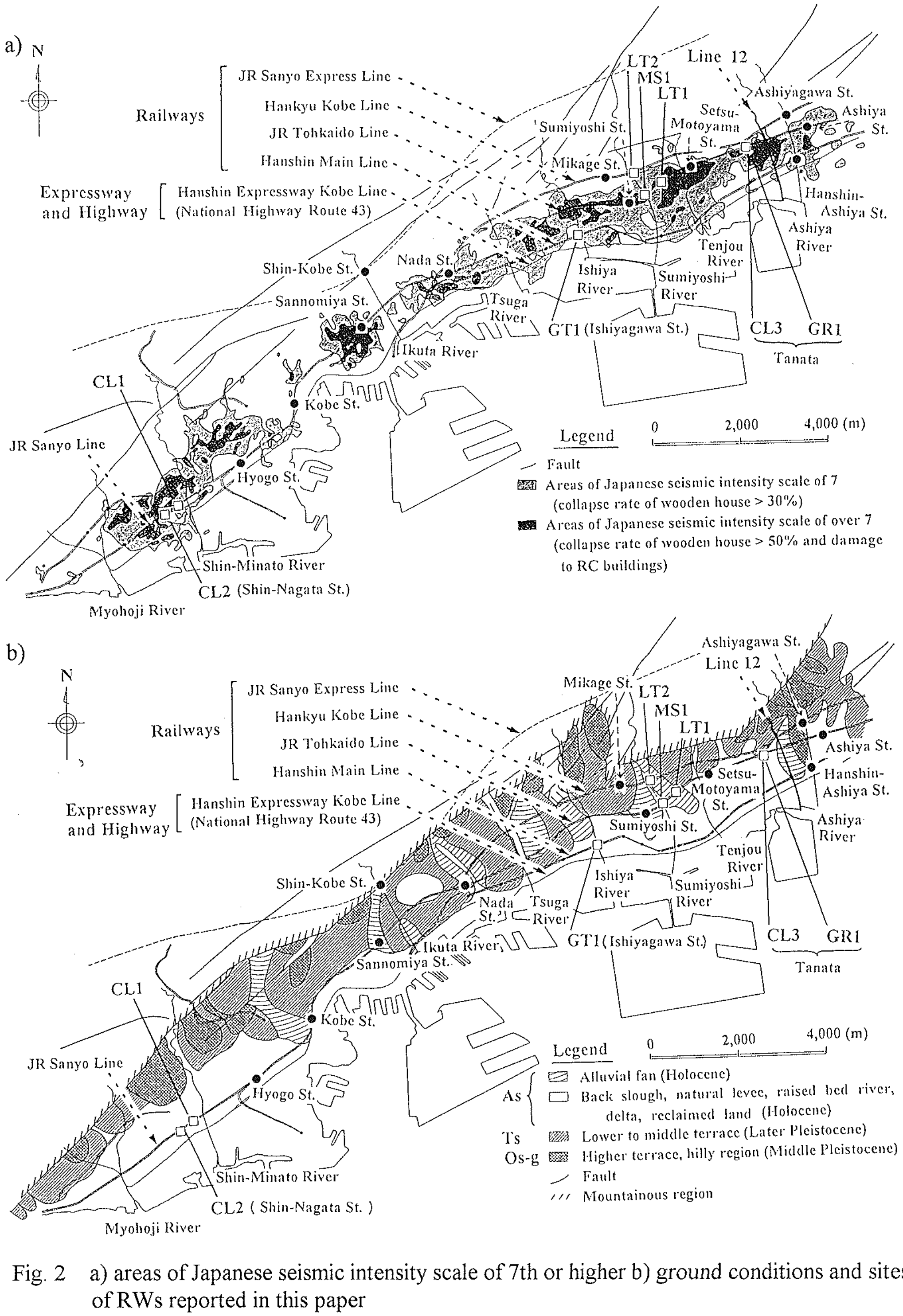

Fig. 2 a) areas of Japanese seismic intensity scale of 7th or higher b) ground conditions and sites of RWs reported in this paper

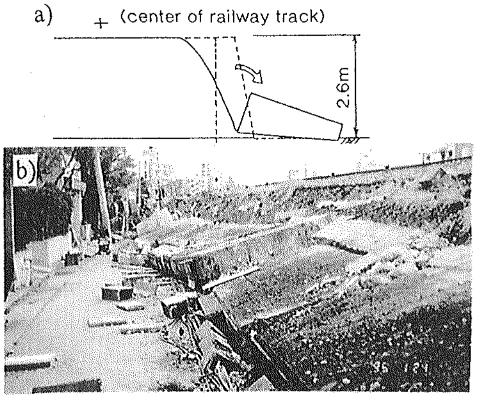

Fig. 6 Typical damaged leaning-type unreinforced concrete RW; embankment along JR Tohkaido Line between Setsu-Motoyama and Sumiyoshi Stations (Site LT1 in Fig. 2); a) sketch and b) picture

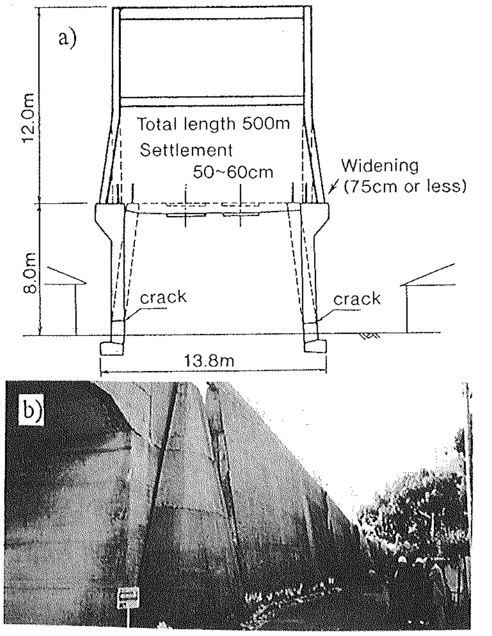

Fig. 7 Typical damage to leaning-type unreinforced concrete RWs; embankment along Kobe Line of Hankyu Railway Co. between Okamoto and Mikage Stations (Site LT2 in Fig. 2); a) sketch and b) picture

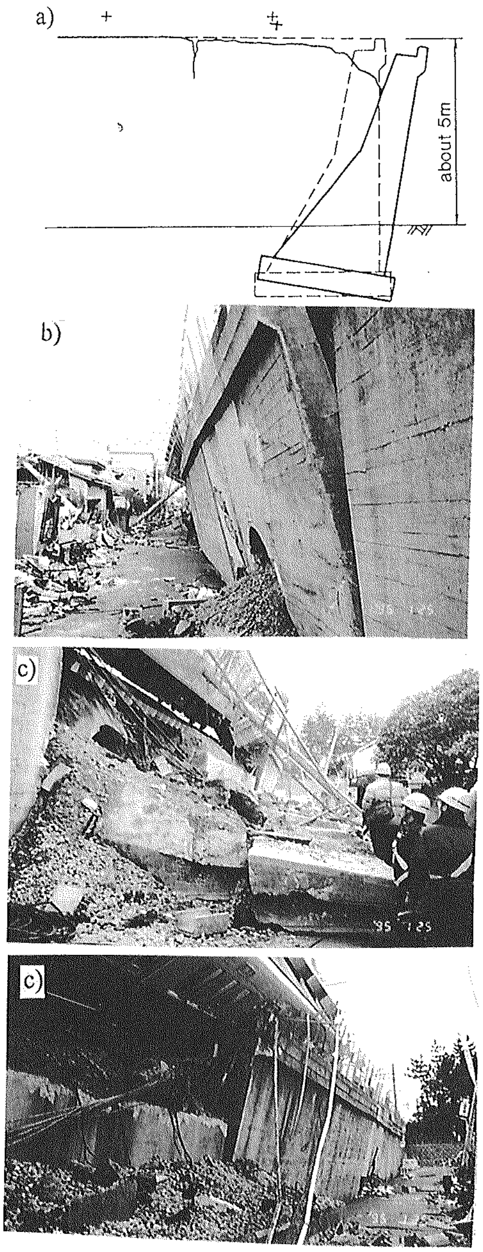

Fig. 8 Typical damage to gravity-type unreinforced concrete RWs; embankment for Main Line of Hanshin Railway Co. adjacent to Ishiyagawa Station (Site GT1 in Fig. 2); a) sketch, b) picture and c) pictures

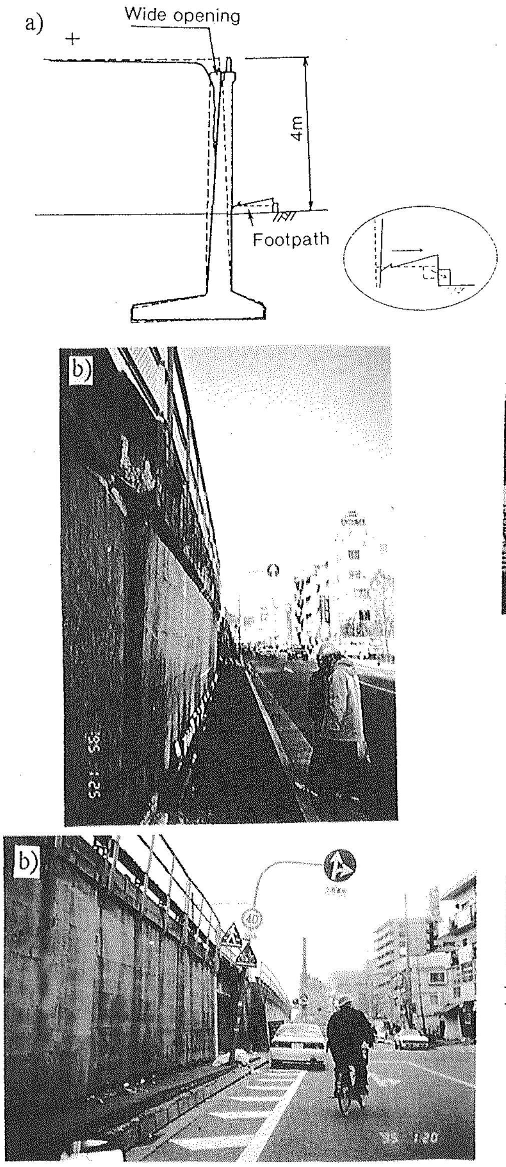

Fig. 9 Typical damage to cantilever RC RWs for embankment of JR Sanyo Line between Hyogo and Shin-Nagata Stations (Site CL1 in Fig. 2); a) sketch and b) pictures

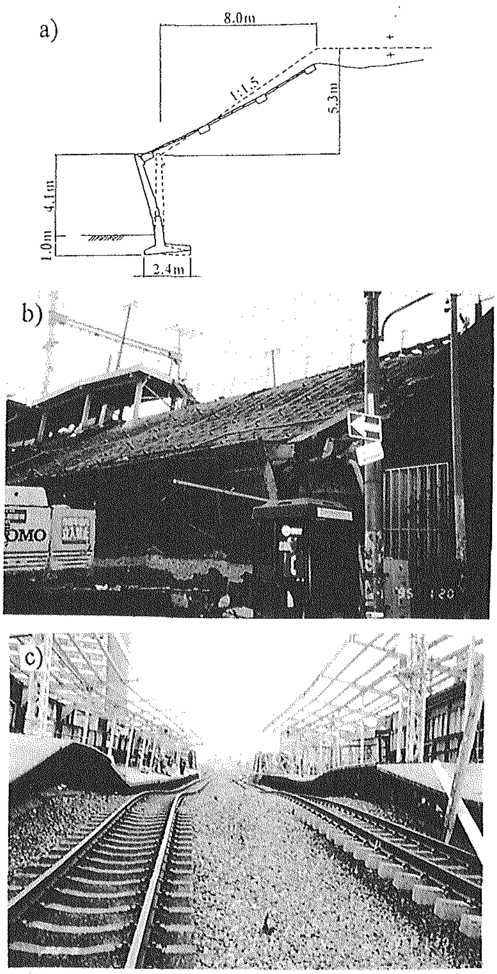

Fig. 10 Rupture of the facing of cantilever RC RW; embankment of JR Shin-Nagata Station (Site CL2 in Fig. 2); a) sketch and b) picture of damaged RC RW, and c) view of the previous Shin-Nagata station

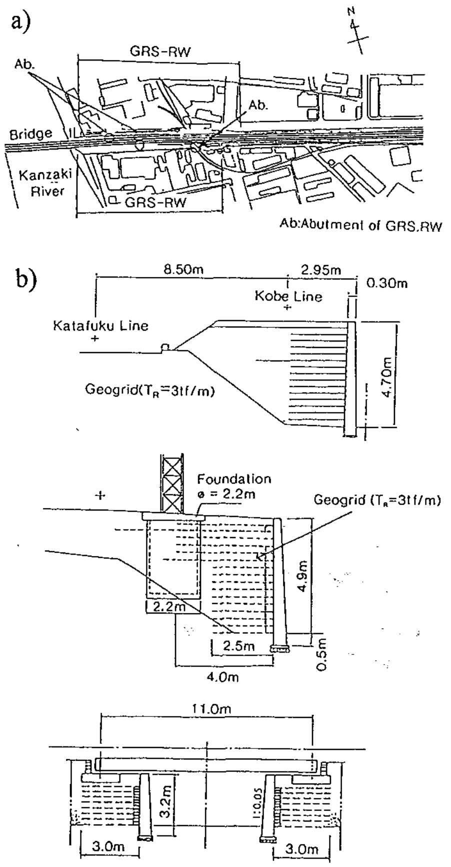

Fig. 11 a) plan and b) typical cross-section of GRS-RWs for JR Tokaido Line, between Tsukamoto and Amagasaki Stations (see Fig. 1)

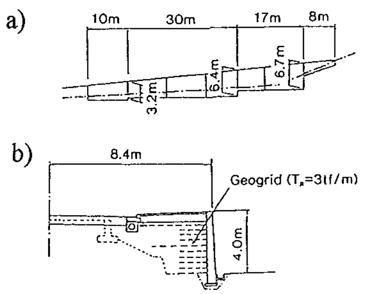

Fig. 13 a) and b) elevations and cross-sections of GRS-Rws at Maiko (see Fig. 1)

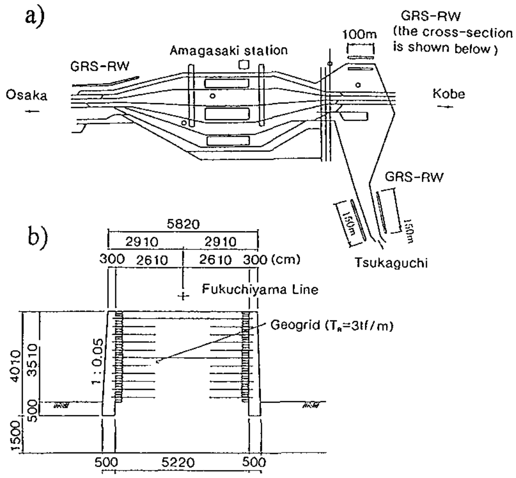

Fig. 14 a) plan and b) typical cross-section of GRS-RW adjacent to Amagasaki station of JR Fukuchiyama Line (see Fig. 1)

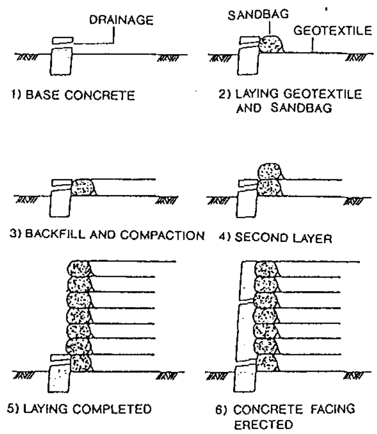

Fig. 15 Standard staged construction procedures for a GRS-RW having a full-height rigid facing

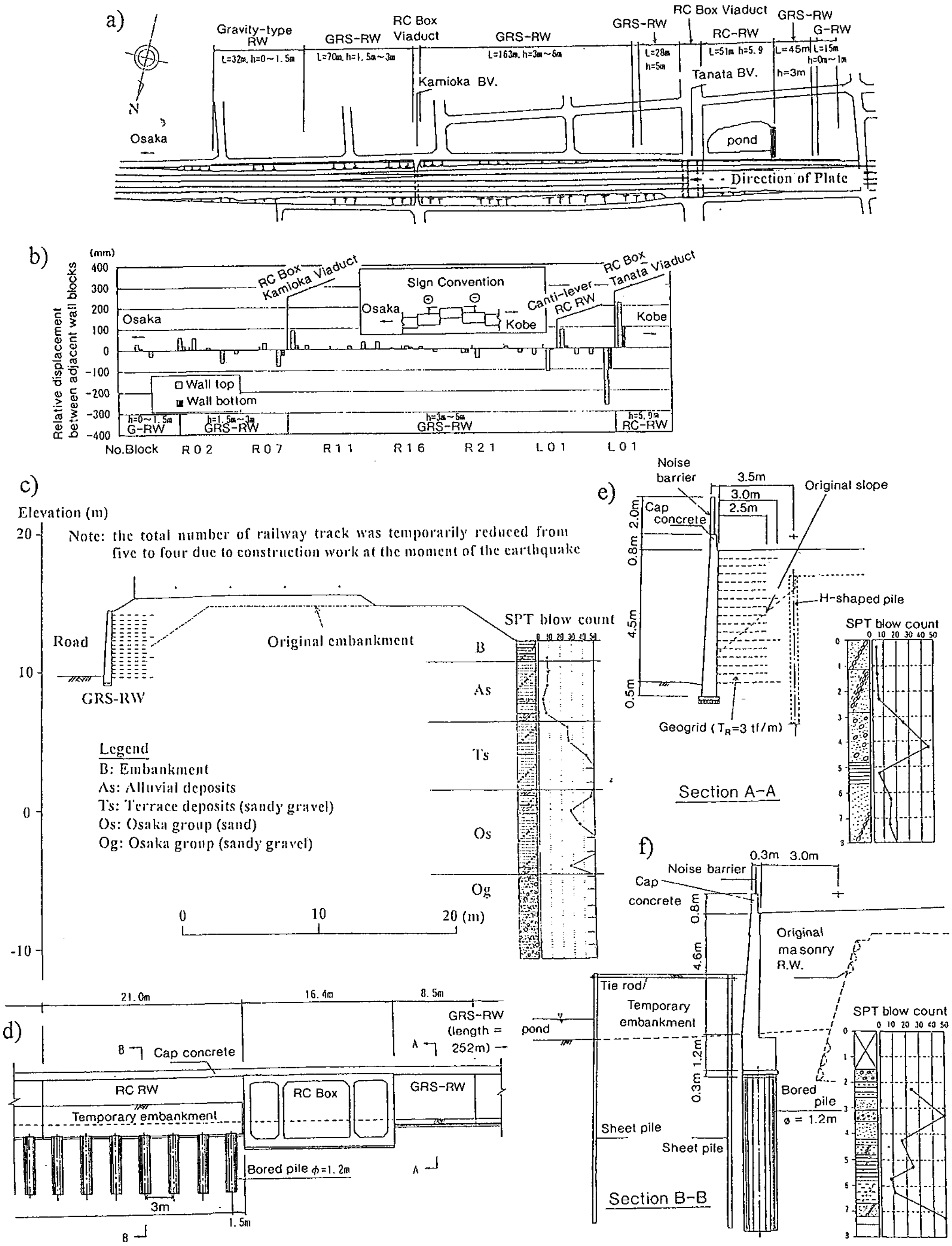

Fig. 12 a) plan of the site, b) relative displacement between adjacent facing sections of GRS-RW and cantilever RC RW, c) cross-section of embankment, d) front view from south of GRS-RW and cantilever RC RW, e) typical cross-section of GRS-RW, and f) typical cross-section of RC RW at Tanata (see Fig. 1, and sites CL3 and GR1 in Fig.2)

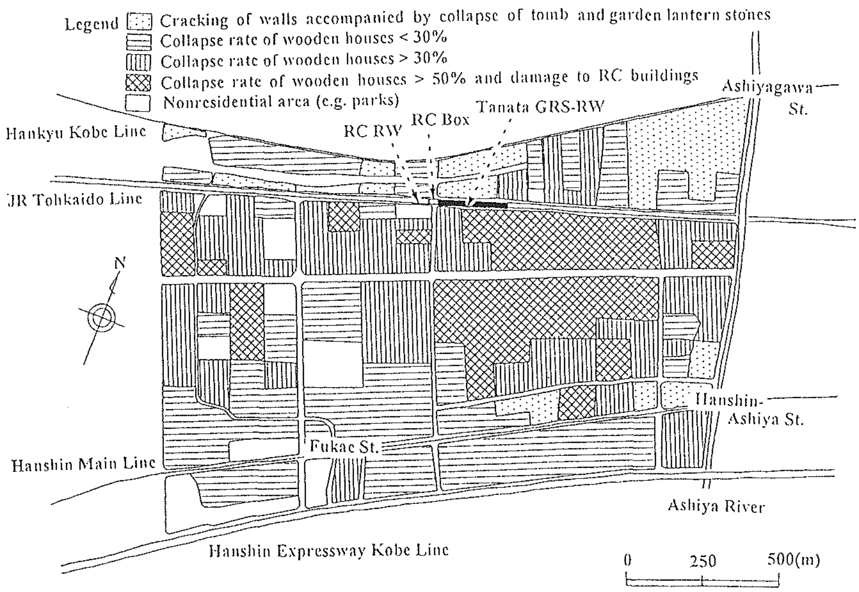

Fig. 16 Details of the damage to wooden houses in the area in front of Tanata GRS-RW

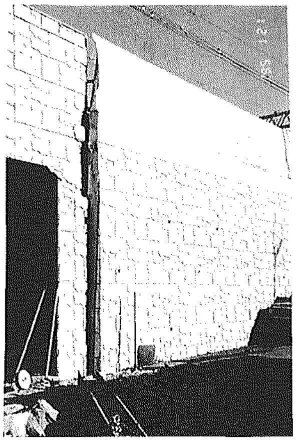

Plate 1 Displacement of the Tanata GRS-RW relative to a RC box culvert

Plate 2 Scene in front of the GRS-RW at Tanata

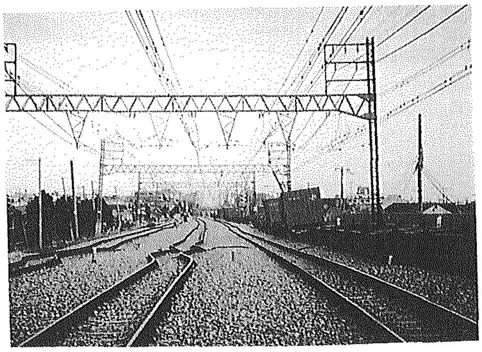

Plate 3 Scene of the crest of the embankment at Tanata site

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

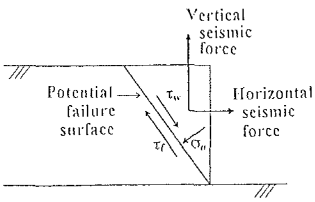

Fig. 17 Stresses in the backfill responding to seismic force

a)

b)

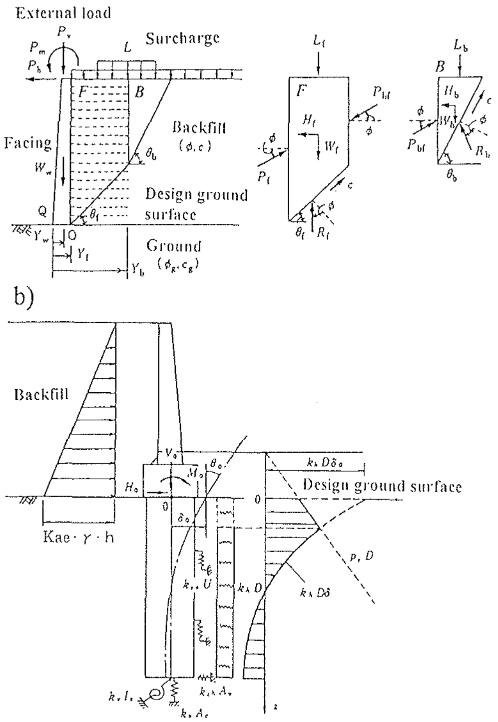

Fig. 18 Current seismic design methods for a) RC RWs and b) GRS-RWs, according to the design code for railway structures