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Geosynthetic-reinforced soil retaining walls as important permanent structures

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ABSTRACT: For the last decade, geosynthetic-reinforced soil retaining walls with a full-height rigid facing cast-in-place by staged construction procedures were constructed in Japan to a total length exceeding 20 km as important permanent structures mainly for railways. These include retaining walls for embankments, bridge abutments, a wall backfilled with a nearly saturated clay constructed on a thick very soft clay deposit, a wall which survived a very severe earthquake, and those constructed to support bullet train tracks. A new method to make reinforced soil very stiff by preloading and prestressing is also described.

1 INTRODUCTION

A construction system for permanent geosynthetic-reinforced soil retaining walls (**the GRS-RW system**) is now widely used in Japan. It can be characterized by the following features:

a) the use of a full-height rigid facing cast-in-place by staged construction procedures (**Fig. 1**);

b) the use of a polymer grid reinforcement for cohesionless soils to enhance good contact with soil and of a composite of non-woven and woven geotextiles for nearly saturated cohesive soils to facilitate both drainage and tensile-reinforcing of the backfill;

c) the use of relatively short reinforcement; and

d) the use of low-quality on-site soil as the backfill, if necessary.

The staged construction method (Fig. 1) consists of the following steps:

1) a small foundation for the facing is constructed;

2) a geosynthetic-reinforced soil wall with wrapped-around wall face is constructed with help of gravel-filled bags placed at the shoulder of each soil layer; and

3) a thin lightly steel-reinforced concrete facing is cast-in-place directly on the wall face after the deformation of the backfill and the subsoil layer beneath the wall has taken place while a good connection between the facing and the main body of the wall is assured.

1.1 Typical latest case history

Geosynthetic-reinforced soil retaining walls with a full-height rigid facing (FHR) constructed by this particular con-

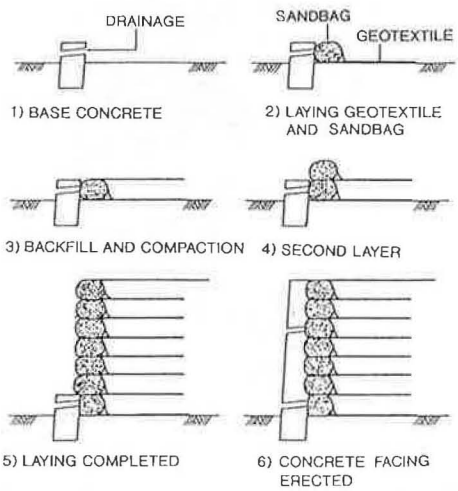

Fig. 1 Standard staged construction procedures for the GRS-RW system

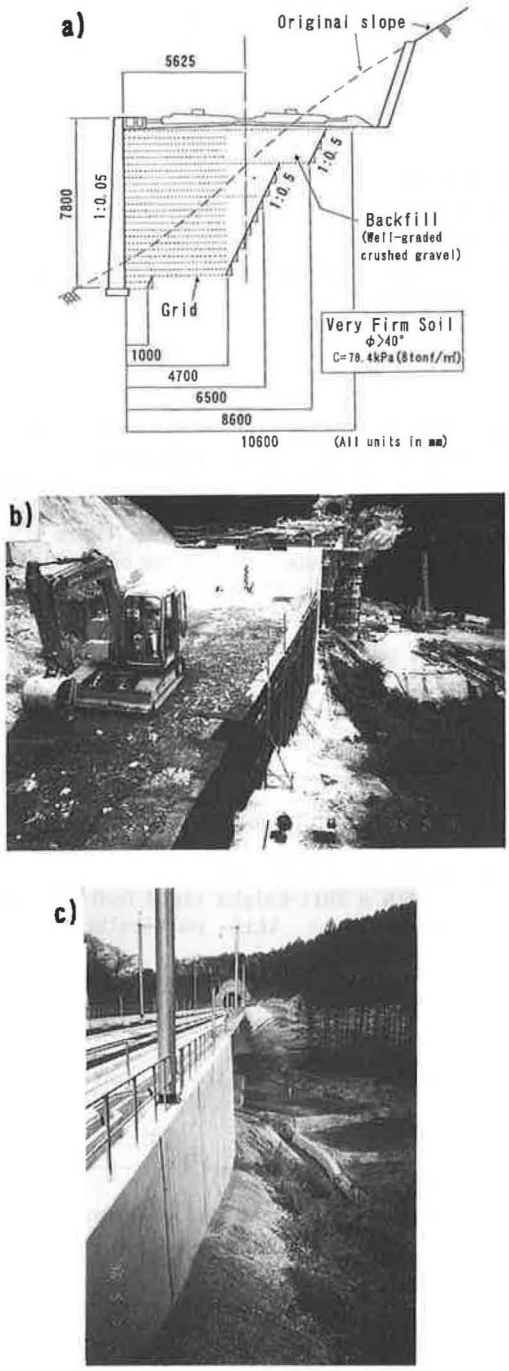

Fig. 2 a) Typical cross-section, b) view of the wall under construction and c) view of the completed wall; a GRS-RW completed in 1995 for Hokuriku Shinkansen (bullet train), west of Karuizawa (No. 50 in Fig. 3)

struction system will herein be called **the GRS-RW with a FHR facing. Fig. 2** shows one of the latest projects. The wall supports main tracks of a new bullet train line (Hokuriku Shinkansen), which is now under construction between the north of Tokyo and Nagano City, where the Winter Olympic Games will be held in 1998. The wall has a height of 4.6 to 8.6 m with a length of 260 m between a bridge abutment and a tunnel exit. The wall looks like a conventional RC cantilever wall (**Fig. 2c**). The backfill is a well-graded crushed gravel in this case, which is reinforced with a special type of polyester grid having a rupture strength T_R = 35.3 kN/m (3.6 tonf/m) and 68.6 kN/m (7.0 tonf/m) for relatively low and high walls, respectively. This wall is one of the first walls that support directly main tracks of the bullet trains, which is one of the most critical civil engineering structures in Japan. For this new line, GRS-RW's with a FHR facing were constructed at eleven sites for a total length of 3,330 m. These include the ones constructed at a yard in Nagano City, which are described later.

1.2 Brief history

The study of the GRS-RW system started in 1982. Since 1987, particularly since the approval in 1992 by the Ministry of Transport of Japan, a large number of permanent GRS-RW's with FHR facings, typically 5 m-high, to support important railway tracks have been constructed to a total length exceeding 20 km (**Fig. 3**). Two large-scale projects typical for railway applications are the one at Nagoya (No. 4 in Fig. 3; Tateyama et al., 1994) and the one at Amagasaki (No. 7 in Fig. 3; Kanazawa et al., 1994). Despite with the late start, GRS-RW's with FHR facings for highways have been constructed for a total length of 760m. The GRS-RW system has been validated by excellent post-construction performance of the walls (Tatsuoka et al., 1992, 1996, Murata et al., 1991, 1994 and Doi et al., 1994).

Before the GRS-RW system was introduced in the Japanese market, the Terre Armee technique had dominated the construction market of the permanent reinforced-soil retaining wall structures. Japan National Railways, which is the previous organization of the present Japan Railways (JR's),

is the first nation-wide organization which constructed extensively Terre Armee walls. On the other hand, many conventional type GRS-RW's with wrapped -around wall faces have been used only as temporary walls or for secondary applications, and they could not compete with Terre Armee walls. Therefore, the fact that the recent complete stop of the use of the Terre Armee technique in the construction of the railway structures in Japan has impacted the soil reinforcement market in Japan (Tatsuoka et al., 1994). In the current JR design standard for soil retaining structures, the design and construction methods of both the GRS-RW system and the Terre Armee wall system are specified in parallel to each other so that each designer can select either of them or another conventional technique. In many railway projects, however, the GRS-RW system has been adopted after competition with conventional RC retaining wall techniques and the Terre Armee technique. Among the case histories shown in Fig. 3, we will describe particularly the following recent ones; **Seibu bridge abutment** (No. 42 in Fig. 3), **Nagano wall** using a nearly saturated backfill soil on a very soft clay deposit (No. 38), **railway embankments** damaged by flooding and reconstructed by using the GRS-RW system in the southern Kyushu (No. 35), and **Tanata wall** (No. 5), which survived a very severe earthquake. The important lessons that we could learn from these case histories include those concerning the issues of;

1) cost performance;
2) deformability of wall;
3) stability of wall; and
4) durability and acceptable aesthetics of wall face.

These issues are discussed by comparing GRS-RW's with a FHR facing with conventional type retaining walls and conventional steel-reinforced soil retaining walls. The issue of durability of geosynthetics is beyond the scope of this report, lsince this subject is discussed in great detail elsewhere.

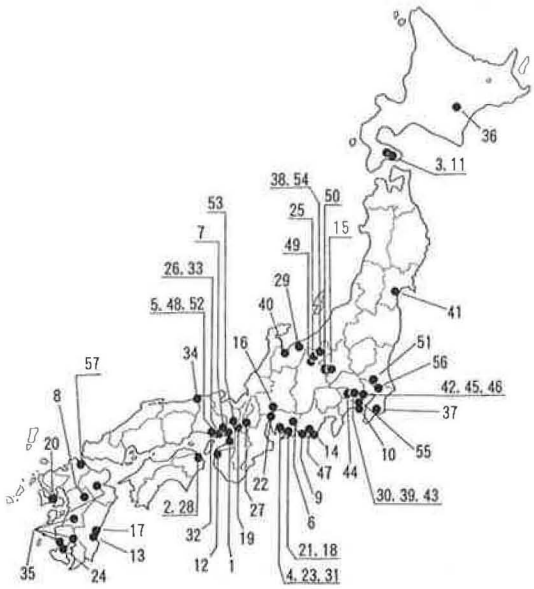

Fig. 3 Locations of the major projects of the GRS-RW system (numbered in the chronological order)

2 LOW COST/PERFORMANCE RATIO

The reinforced soil retaining wall system is cost-effective because of the use of a facing structure that is by far simpler than those of most conventional retaining systems; i.e., lower construction cost, higher construction speed, and use of lighter construction machines while the wall performance is equivalent to, or even better than, that of the conventional type retaining systems. In addition, when the wall is flexible, the pile foundation to support the facing is unnecessary, which makes the system more cost-effective.

2.1 Why can the facing be simpler ?

Against the design earthpressure, which is usually the active earthpressure in unreinforced backfill, a conventional type retaining wall is designed as a cantilever structure supported at its base (**Fig. 4a**). For this reason, large moment and shear force may be mobilized in the facing structure with large overturning moment and sliding force acting at the bottom of the wall structure. In the case of a reinforced soil retaining wall, the backfill is retained by tensile reinforcement (**Fig. 4b**). The conventional explanation is that, because of the above, only very small

earthpressure acts on the back face of the facing, and accordingly only a very light and flexible facing is sufficient to prevent spilling out of the backfill soil.

In reality, however, the earthpressure acting on the back face of the facing can never be nearly zero unless the backfill is of a soil having large cohesion or unless the arch action between two vertically adjacent reinforcement layers is large enough. If the earthpressure activated on the back face of the facing is nearly zero, only nearly zero tensile force can be activated at the connection between the reinforcement and the back face of the facing. This suggests a large reduction of the soil retaining capability of reinforcement (**Fig. 5a**) (Tatsuoka, 1993). Then, as the confining pressure on soil in the active zone decreases, the active zone becomes more deformable and less stable. That is, the maximum available tensile force T_{max} in each reinforcement member is obtained as;

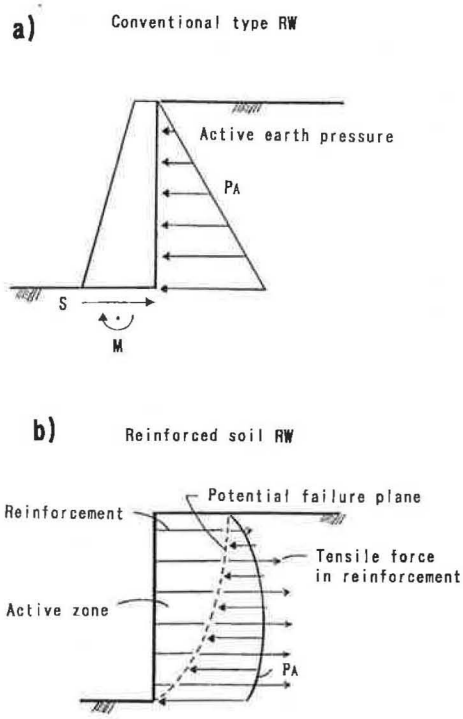

Fig. 4 Force equilibrium for; a) a conventional type retaining wall, and b) a reinforced soil retaining wall

$$T_{max}=\mathrm{Min}\{T_R,\ T_{anchor},\ T_{retain}+T_{W\,max}\} \quad (1)$$

where T_R is the tensile rupture strength of each reinforcement member, T_{anchor} is the available anchoring strength, which is approximately proportional to the anchorage length L_a, T_{retain} is the available retaining strength, which is approximately proportional to the retaining length L_r, and $T_{W\,max}$ is the available tensile force at the connection between the reinforcement and the back face of the facing, which increases with the increase in the available earthpressure on the back face of the facing. As $T_{W\,max}$ decreases, the distribution of reinforcement tensile force T becomes like the ones denoted by B1 and B2 (**Fig. 5b**). Then, at lower levels in the wall, the maximum available tensile

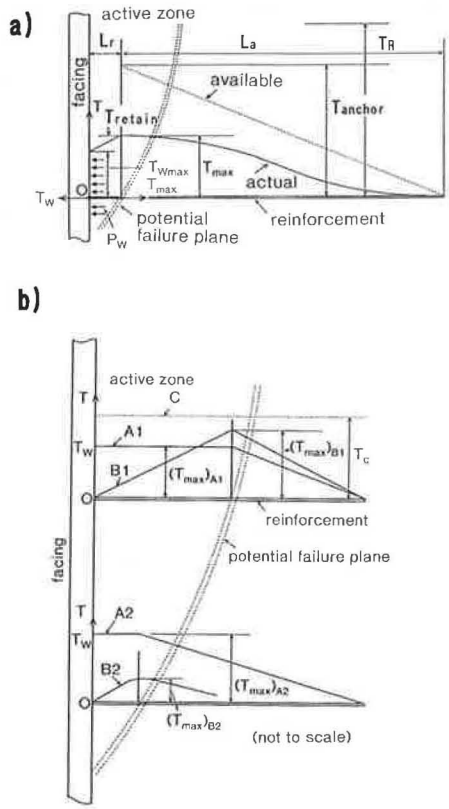

Fig. 5 a) Schematic diagram to explain the available maximum tensile force in reinforcement member, and b) two types of tensile force distribution in reinforcement (n.b., C means the distribution for a prestressed anchoring system) (Tatsuoka, 1993)

force T_{max} cannot become large enough, while a large $T_{w\,max}$ value results in a large T_{max} value (i.e., the pattern A2 in Fig. 5b). In Fig. 5b, it is assumed that for the patterns A1 and A2, the active zone is retained very well without straining and therefore bond stresses are not mobilized on the surface of reinforcement. In this case, the reinforcement tensile force **T** is constant in the active zone. For the patterns B1 and B2, a small value of $(T_{max})_{B2}$ may result in either a value of $(T_{max})_{B1}$ larger than $(T_{max})_{A1}$ for a fixed potential failure plane (in the case of Fig. 5) (Jewell, 1990) or a longer retaining length **Lr** with a larger deformable active zone to increase $(T_{max})_{B2}$ to a value similar to $(T_{max})_{A2}$, mobilized at a backward location.

The results of a number of field and laboratory tests and numerical analyses and the behaviour of many full-scale walls have shown that the earthpressure on the back face of the facing increases with the increase in the facing rigidity (see, for example, Figs. 4.1 and 4.2 of Tatsuoka, 1993). **Fig. 6** shows the result of a relatively large-scale plane strain model test in the laboratory performed under well-controlled conditions (Tajiri et al., 1996). The wall was constructed on a concrete floor. The facing of concrete blocks was constructed simultaneously with backfilling and compacting cohesionless soil reinforced with long and short grid layers having T_R = 56.8 kN/m (5.8 tonf/m). It may be seen that the distribution of tensile force in the grid is similar to pattern A (Fig. 5b). This is due to the rigidity of the facing.

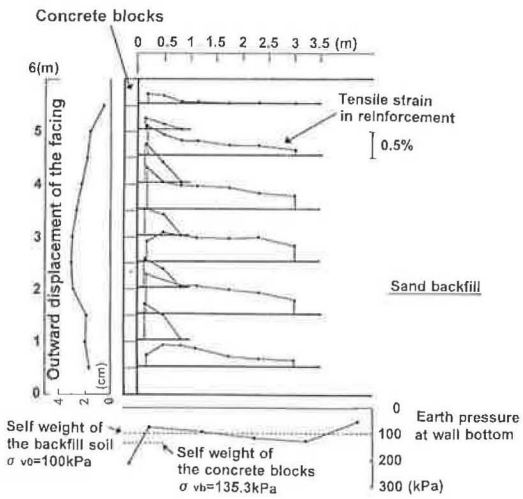

Fig. 6 Behaviour of a large model wall constructed in the laboratory (Tajiri et al., 1996)

Further, the use of a FHR facing is more effective to increase the wall stability and to reduce the wall deformation than the use of a relatively flexible facing such as a discrete panel facing and a wrapped-around facing (Tatsuoka et al., 1989, Tatsuoka et al., 1991, 1992). Tatsuoka (1993) classified the different types of facing rigidity and discussed their different contributions to the wall stability.

In the current design method for the GRS-RW system, a FHR facing is designed to support the earthpressure activated in the unreinforced backfill (Horii et al., 1994). However, both the moment and the shear forces in the facing and the overturning moment and the sliding force activated at the bottom of the facing can be very small because a FHR facing behaves as a continuous beam supported by a number of reinforcement layers with a very short vertical spacing (i.e., 30 cm) (Fig. 4b). Therefore, the facing can be very thin and the amount of steel-reinforcement required is very small as shown in Horii et al. (1994). The minimum facing thickness specified for the GRS-RW system is 30 cm to ensure the workability during concrete placing. The thickness is usually larger than structural requirement.

2.2 Should a wall in service be flexible ?

When the supporting subsoil is not very stiff, a conventional type reinforced concrete (RC) cantilever retaining wall usually needs to be supported by a pile foundation to prevent noticeable wall displacement to occur (**Fig. 7a**). On the other hand, **Fig. 8** shows an experimental 5 m-high cantilever RC wall constructed directly on an intact layer of volcanic ash clay (Kanto Loam). The backfill was a nearly saturated soil obtained from the nearby deposit of Kanto loam. This soil under the intact condition is stable due to slight cementation, but it becomes very soft by remolding due to a high natural water content (about 100 - 120 %) and a

high degree of saturation (85 - 90 %). The wall displaced about 10 cm at the top of the wall face during six months after the initiation of backfilling.

2.3 Advantages of staged construction procedures

It has been advocated that a reinforced soil retaining wall having a flexible or deformable facing can accommodate the deformation of the backfill and the underlying subsoil layer. However, a completed wall should be rigid and stable enough. This contradiction may be resolved by the staged construction method (Fig. 1), that is:

a) By the staged construction method, potential damage due to relative settlement between the rigid facing and the backfill to the connections between the facing and the reinforcements can be avoided.

b) In the staged construction procedures, good compaction of the backfill adjacent to the back face of the facing can be achieved by allowing relatively large outward lateral displacement to occur at the temporary wall face. Accordingly, sufficiently large tensile strains can be developed in the reinforcement. When a discrete panel or a full-height panel facing has been erected prior to soil compaction, the soil adjacent to the back face of the facing cannot be compacted sufficiently, to avoid large earthpressure on the facing, associated excessive lateral outward displacement of the facing, and the damage to the connection between the reinforcement and the back face of the facing. Accordingly, sufficiently large tensile strains may not be mobilized during backfilling.

c) When a full-height panel facing is propped during backfilling, some uncontrollable outward displacement of the facing may occur upon the removal of propping. On the other hand, when unpropped discrete-rigid panels are erected while the backfill is compacted, it may be difficult to ensure a good facing alignment and some post-construction displacement of the facing may also continue (Tatsuoka et al., 1994). In the staged construction procedures, as a FHR facing is cast-in-place after the major portion of the potential deformation of the backfill and the subsoil layer is over, a good alignment of the facing can be easily achieved. In addition, a pile foundation to support the facing becomes unnecessary, mainly because the facing is laterally supported with many reinforcement layers and therefore large moment and shear force is not mobilized at the bottom of the facing, (i.e., a self-

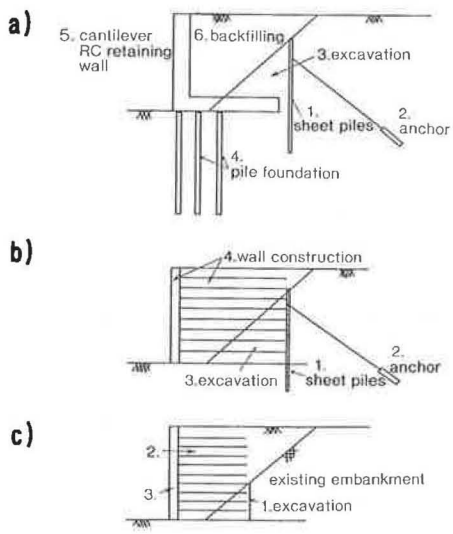

Fig. 7 Comparison of different retaining wall construction procedures for reconstructing an existing slope; a) a conventional RC cantilever RW, b) a reinforced soil retaining wall with relatively long reinforcement, and c) a reinforced soil retaining wall with relatively short reinforcement

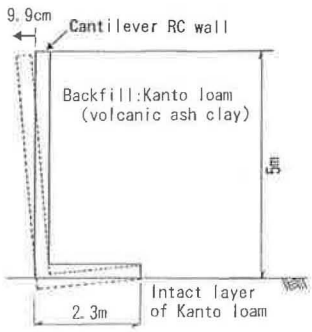

Fig. 8 Experimental 5 m - high cantilever RC wall at Chiba Experimental Station, IIS, University of Tokyo

supported structure; Fig. 4b). Particularly, in the staged construction method, the facing is free from the effects of earthpressure and associated vertical force caused by the downward force from the reinforcement layers that settle relative to the facing during and after the compaction of the backfill.

It is noted that in the staged construction procedures, the wall should be stable for some time before a FHR facing is cast-in-place. It has been confirmed from the full-scale behaviour of many walls that a wall without a FHR facing (i.e., before servicing to live load) is very stable, although some deformation of wall may occur. The authors consider that the use of gravel-filled bags at the shoulder of each soil layer (Fig. 1) together with a relatively small vertical spacing of reinforcement layers (i.e., 30 cm) contributes largely to the stability of a wall on a temporary basis.

2.4 Full-height rigid (FHR) facing

The use of a FHR facing has other advantages that include:

a) No major reinforced-soil retaining soil bridge abutments, including those of Terre Armee walls, had been constructed in Japan before seventeen GRS bridge abutments with FHR facings were constructed. They include the three abutments for railways of Seibu Railway Line at site No. 42 in Fig.3 (**Fig. 9**). During the commute time, passenger-full trains run on the bridge every three minutes at high

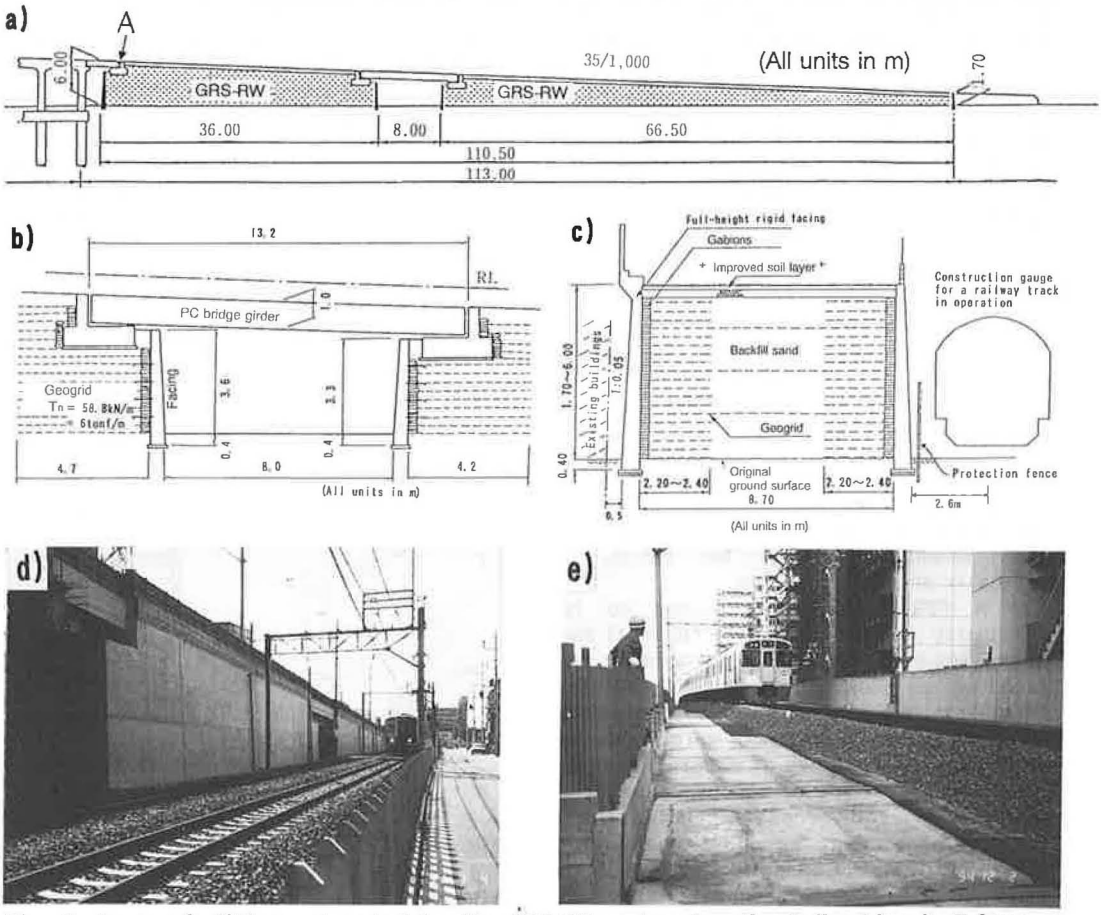

Fig. 9 Approach fill constructed by the GRS-RW system for the Seibu Line in Tokyo; a) general view, b) bridge abutment, c) cross-section, d) general view, and e) view at the crest

speed. The walls were constructed, without using a pile foundation, directly on a soil deposit of Kanto Loam. This soil deposit is similar to the one supporting the cantilever RC retaining wall shown in Fig. 8. Any problem due to the settlement of the bridge girder by train load has not been reported for these abutments. The use of a FHR facing made the construction of such GRS bridge abutments feasible. In particular, a GRS-RW with a FHR facing can effectively resist the seismic lateral load from a bridge girder. This was confirmed for the Seibu walls by applying a lateral outward force of 98 kN (10 tonf) to a RC block denoted by the letter A in Fig. 9a. The lateral movement at the top of the facing was only 0.9 mm for this relatively large load. Also, this case is characterized by very small construction space available; in Fig. 9c, the space between the wall face and the running trains is only 0.8 m and that between the wall face and the existing buildings is only 0.5 m. The facing was cast-in-place in the frame supported by steel bars anchored in the backfill soil without using any external support.

b) Laboratory tests (Tatsuoka et al., 1989) and full-scale loading tests (Tateyama et al., 1994a; Tamura et al., 1994) showed that a GRS-RW with a FHR facing can support very large vertical and lateral loads acting at and immediately behind the crest of the wall without exhibiting noticeable deformation (as explained above). Therefore, in many cases, FHR facings were constructed to support directly several types of structures (**Fig. 10a**). When the facing is deformable, a very complicated and expensive measures should be taken, as typically shown in **Fig. 10b**.

c) A FHR facing contributes to the durability and aesthetics of the wall face when compared with wrapped around wall face of a geosynthetic-reinforced soil retaining wall. In many walls constructed in urban areas, the facings were decorated as if they were stacks of natural stones.

d) It has been one of the major concerns among soil reinforcement engineers to make reinforcement as short as possible when the available space is limited. In the case shown in Fig. 7, when relatively long reinforcement is used (**Fig. 7b**), sheet piles and anchors may be needed to ensure the stability of the existing embankment during excavation. If so, a reinforced soil retaining wall may not be very cost-effective. Note that metal strips become indispensably relatively long to ensure the sufficiently large pull-out strength of reinforcement that is comparable with the tensile rupture strength. On the other hand, when the reinforcement can be shorter (**Fig. 7c**), slope excavation can be minimized without using sheet piles and anchors. This may result in a large cost reduction. The allowable reinforcement length can be made shorter by the use of not only planar reinforcement (i.e., geosynthetic sheets), but also a FHR facing. This means that, compared with metal strips, planar polymer reinforcement (e.g., grid) has a much shorter anchoring length (typically less than 30 cm) required to mobilize the anchoring strength T_{anchor} which is equivalent to the tensile rupture strength T_R (Fig. 5a). However, the use of shorter reinforcement may be penalized by larger outward shear deformation of wall (Jewell, 1990). The use of a FHR facing can make a reinforced zone behave like a monolith that can increase the wall stability and decrease the shear deformation of the wall. For the GRS-RW system, the allowable minimum length of the reinforcement is specified to be the larger

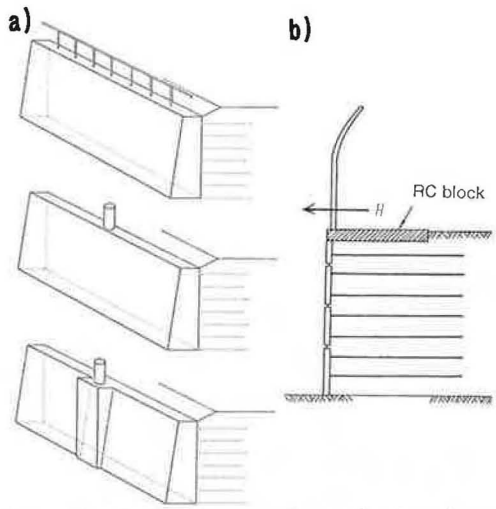

Fig. 10 a) Some examples of structures constructed directly on a full-height rigid facing, and b) a reinforced soil retaining wall with a deformable panel facing to accommodate a foundation of structure (Tatsuoka et al., 1994)

value of the 35 % of the wall height and 1.5 m. Furthermore, as the reinforcement becomes shorter, more load may be concentrated toward the bottom of the facing, particularly during an earthquake. Therefore, the contribution of the bearing capacity of the subsoil beneath the facing to the wall stability cannot be ignored, and this factor is considered in the current design.

Generally, the construction cost for the GRS-RW system (Fig. 1) is higher than that for conventional geosynthetic-reinforced soil retaining walls without rigid facings. It should be noted, however, that this GRS-RW system is much more cost-effective than conventional type retaining walls constructed as permanent important structures and, in many cases, conventional type metal-reinforced soil retaining walls. The authors believe that the use of a FHR facing is one of the major reasons why the GRS-RW system has been chosen by practicing engineers for a number of important permanent retaining walls and bridge abutments. Two recent case histories which typically show the cost-effectiveness of the GRS-RW system are shown below.

3 NAGANO WALL

3.1 General

This wall has a height of 2 m when completed for a total length of about 2 km (solid lines in **Fig. 11a**) for a yard for Shinkansen (bullet train) the north of Nagano City (No. 38 in Fig. 3). This case is one of the best examples showing the advantages of the staged construction method. The construction of the wall started in 1993 and completed in 1996. In addition, other GRS-RW's with FHR facings for a length of 100 m and two 3.4 m-high GRS-bridge abutments with FHR facings were constructed since 1995 as part of the approach fill to the yard (**Fig. 11b**). A very soft clay layer beneath the approach fill was improved by cement-mixing in-place to avoid intolerably large settlement relative to the adjacent RC bridge abutment supported by a pile foundation.

3.2 Preloading and settlement

For the walls at the yard, as the subsoil is a very thick deposit of very soft clay, a preload fill was placed on the embankment back of the GRS-RW's without a rigid facings. This resulted in a large settlement of the fill (about 1 m) (**Fig. 11c and d**). The initial wall height as constructed was 3 m to accommodate this large settlement. It is to be noted that no pile foundations were used, which should have been necessary if conventional cantilever RC retaining walls were constructed. A FHR facing was cast-in-place during the summer of 1996 about one year after about six months of preloading (**Fig. 11e**).

3.3 Clay backfill

The Nagano wall is one of the important cases also because of the use of a nearly saturated on-site clay as the backfill for the first time. The use of clay was decided based on very good performance of a series of full-height GRS-RW's with a backfill of nearly-saturated volcanic ash clay (Kanto Loam) (Tatsuoka et al., 1986, 1987, 1993; Yamauchi et al., 1987; Ling et al., 1995). The backfill soil for the Nagano wall is a highly weathered tuff which was available on-site. This soil became nearly saturated soft clay after compaction with an average water content of about 30 % and a degree of saturation of 70 % (**Fig. 11f**). The backfill soil was reinforced with a composite of non-woven and woven geotextiles, which has a rapture strength T_R of 35.3 kN/m (3.6 tonf/m) at a failure tensile strain of 7 % and a tensile rigidity of 490 kN/m (50 tonf/m) at an elongation of 5 %.

The use of low-quality on-site soil as the backfill soil results in a large cost reduction when compared to the case where the on-site soil is not used but treated for disposal and expensive borrowed cohesionless soil is used. The results of laboratory small model tests (Ling and Tatsuoka, 1994) showed that saturated clay can be effectively reinforced with a composite when the soil is anisotropically consolidated under field operational conditions. This result is consistent with the very good performance of the full-scale walls mentioned above.

3.4 Others

At this site, the importance of the use of a FHR facing was re-confirmed by comparing the behaviour of two experimental GRS-RW's with and without a FHR facing upon the construction of fill on top of the walls. That is, the deformation of the walls without a FHR facing was noticeably larger than that of the wall with a FHR facing (Tatsuoka et al., 1996d).

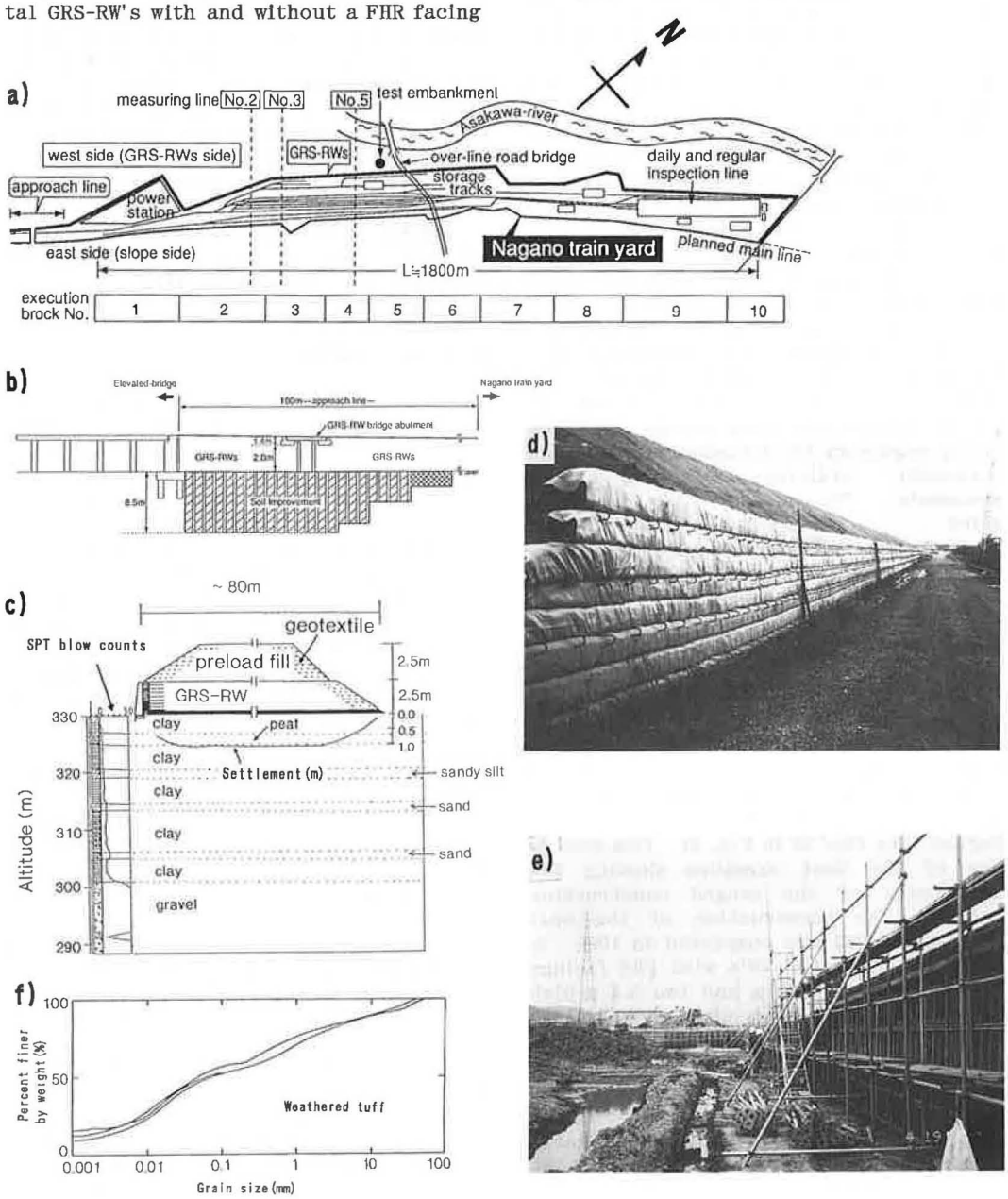

Fig. 11 a) Plan of the yard for Shinkansen in Nagano City and GRS-RW's with FHR facings (solid lines), b) approach fill to the yard, c) typical cross-section of embankment with GRS-RW, d) view of the wall during preloading e) view of casting- in-place of a FHR facing after the preload fill was removed, and f) some typical grading curves of the backfill

The success of Nagano wall shows that most types of on-site soils including those of "inferior" quality (e.g., sandy soils including a large amount of fines or even nearly saturated fine-grained soils) can be used for the backfill soil of the GRS-RW system. This is a special advantage over the conventional steel-reinforced soil retaining wall system (Zornberg and Mitchell,1994, Mitchell and Zornberg,1995).

4 RECONSTRUCTIONS OF RAILWAY EMBANKMENTS IN KYUSHU

4.1 Damage in 1989 and reconstruction

In the Mt. Aso area in the central Kyushu Island, a series of full sections of railway embankments of the Ho-hi Line located in narrow valleys were lost during the very heavy rainfall on 2 July 1989 (No. 8 in Fig. 3; see also **Fig. 12**). The damage was caused by dam-up of flood water in the upper reach of embankment due to the clogging of a drain pipe crossing each embankment. Six full sections of embankment were reconstructed as typically shown in **Fig. 13**. To reduce the amount of earthwork, a nearly vertical GRS-RW with a FHR facing was constructed at the downstream toe of each embankment, while the slope was reinforced with a grid (Tatsuoka et al., 1992). This remedial work is characterized by very large embankment heights and a large diameter drain pipe installed in each embankment.

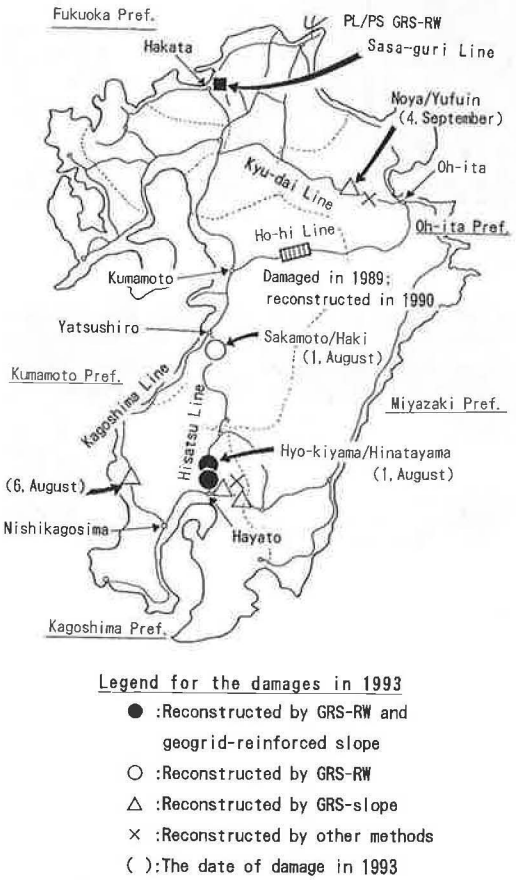

Fig. 12 Locations of the railway embankments seriously damaged by heavy rainfalls in 1989 and 1993 and that of the first PL/PS GRS bridge pier

4.2 Damage in 1993 and reconstruction

Again in June through September 1993, many sections of railway embankments at the sites indicated in Fig. 12 in the central and southern Kyushu were seriously damaged by a series of heavy rainfall. The scale of damage was much more vast than the previous case. The total precipitaion during these months in Kagoshima and Miyazaki Prefectures amounted to approximately 3,000 mm. Several damaged sections of embankment with a total soil volume of 18,700 m^3 at the sites, denoted by the symbol ● in Fig. 12, were reconstructed by November 1993 by the method similar to the previous case as typically shown in **Fig. 14**. The original fill material of this wall was a pumice called Shirasu. The fill soil was washed away from the embankment

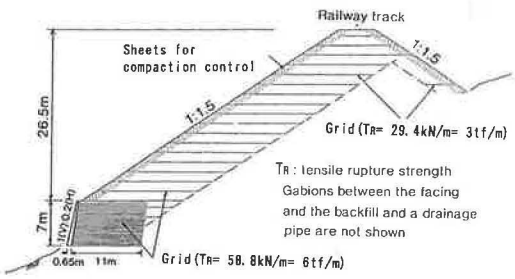

Fig. 13 Cross-section typical of the reconstructed railway embankment at Aso site, damaged in 1989, the Ho-Hi Line (No.8 in Fig. 3)

toe by flooding. The total volume of crushed stone gravel used for the reconstrution was 8,640 m^3. The reconstruction method was adopted based on the successful previous case of the Ho-hi Line while considering not only its low construction cost but also its relatively short construction period and relatively light construction machines required. The last two factors were particularly important, since as quick as possible remedial work was required, while most damaged embankments were located in remote mountain areas.

At the site denoted by the symbol ○, a conventional masonry retaining wall was totally lost for a length of abou 59 m by flooding of the adjacent river. The wall was reconstructed by the GRS-RW system

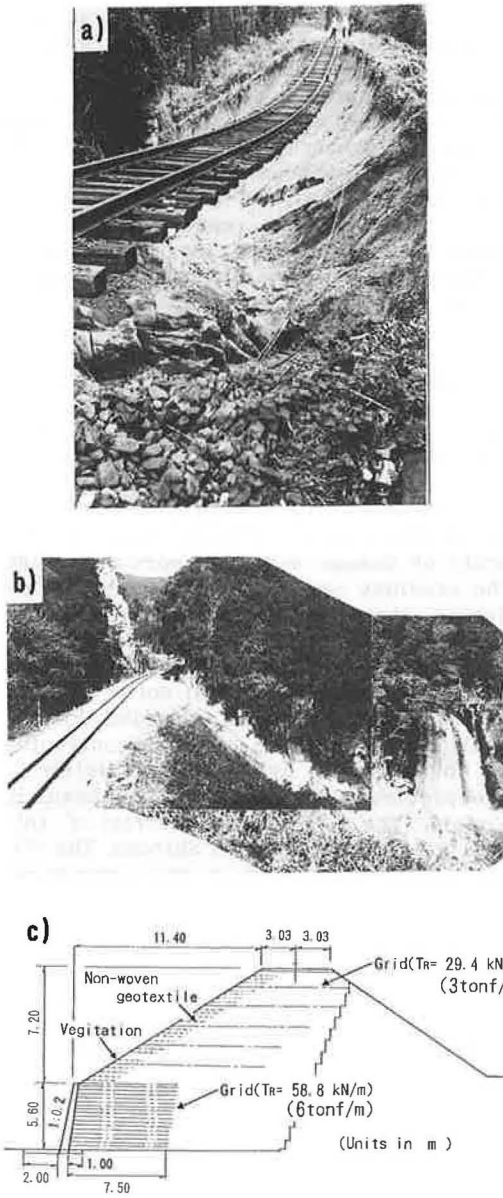

Fig. 14 a) View after damage, and b) view and c) typical cross-section of reconstructed embankment at Hyokiyama/ Hinatayama site, damaged and reconstructed in 1993, the Hisatsu Line (No. 35 in Fig. 3)

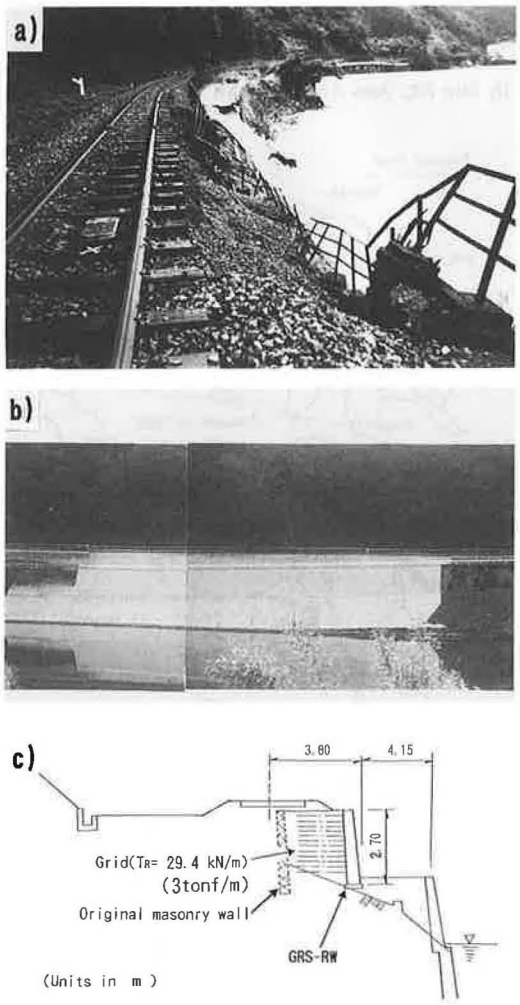

Fig. 15 a) View after damage, and b) view and c) typical cross-section of soil damaged retaining wall at Sakamoto/Haki site, damaged in 1993, the Hisatsu Line (No.35 in Fig.3)

(**Fig. 15**). At several other sites denoted by the symbol △, the slopes of damaged embankment with a total volume of 7,700 m^3 were reconstructed by reinforcing with geosynthetic sheets. The remaining part with a total volume of 3,600 m^3 was reconstructed without geosynthetics.

5 DEFORMATION OF WALL

5.1 Creep deformation

It is often claimed that most currently available geosynthetic reinforcing materials are too extensible and they exhibit larger creep deformation when compared with steel strip reinforcements (e.g., Schlosser et al., 1994). In most cases, the deformation of a wall during construction is not a serious problem. Therefore, creep deformation by dead and live loads and deformation by seismic loads while the wall is in service should be smaller than the specific allowable limit. However, there are no case of GRS-RW with a FHR facing that exhibited noticeable long-term creep deformation despite the use of so called extensible reinforcement (i.e., polymer grid). Probable reasons would include:

1) The GRS-RW's with FHR facings have a sufficient margin of safety resulting from the conservatism exercised at several design stages. In particular, the GRS-RW's are aseismic-designed by the pseudo-static limit equilibrium stability analysis using a horizontal pseudo-static seismic coefficient k_h of 0.2. This makes the safety factors of the GRS-RW's sufficiently large for static load.

2) In a reinforced soil retaining wall with a deformable facing, deformation occurs mainly at the wall face and in the backfill immediately behind the wall face. The use of a FHR facing can effectively restrain such deformation.

5.2 Preloaded and prestressed (PL/PS) GRS-RW

The longest bridge girder so far supported by GRS bridge abutments with FHR facings is still 13.2 m (Fig. 9b). To support a longer and heavier bridge girder, GRS bridge abutments should be stiffer than those constructed so far. As reinforcement works only after the surrounding soil extends sufficiently in the horizontal direction, it is very difficult to increase substantially the vertical stiffness of a reinforced soil mass against relatively small vertical load even when reinforced with densely spaced long inextensible reinforcement.

Working principles: Tatsuoka et al. (1996a, 1996b) and Uchimura et al. (1996) proposed a new construction method that aims at making a reinforced soil very stiff by preloading and prestressing (PL/PS) (**Fig. 16**). In this method, a GRS-RW is constructed by the staged construction method (Fig. 1), but with a pair of lower and upper reaction blocks connected to each other with four tie rods, or only with a top reaction block connected to tie rods that are anchored in the ground. Before a FHR facing is cast-in-place, the wall is preloaded and prestressed as follows:

a) Sufficiently large preload is applied by hydraulic jacks mounted on top of each tie rod. Relatively large preload can be applied without inducing the failure of the backfill because the backfill soil is reinforced. Since preloading and subsequent unloading bring the backfill to unloaded conditions, the wall exhibits nearly elastic deformation when external vertical load P_c is applied to the top of the wall.

b) After unloaded from the preload state to a certain level, the top ends of the tie rods are fixed to the upper reaction block and the hydraulic jacks are removed. Ten-

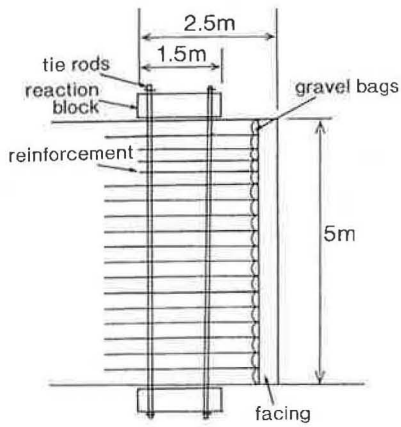

Fig. 16 Typical PL/PS GRS retaining wall

sion remained in the tie rods functions as 'prestress', which is balanced with the corresponding compressive force working on top of the backfill. The prestress keeps the elastic stiffness of the backfill very large when compared with the value in the case without prestress. Therefore, to take advantage of this mechanism, sufficiently large prestress should survive while the wall is in service.

c) When the load P_C is applied to the top reaction block, the tension in the tie rods decreases somewhat due to the compression of the backfill. By the same amount, the increase in the vertical load on top of the backfill by the load P_C becomes smaller when compared with the increase in the case without prestress. Therefore, tie rods work as a compressive structural component against P_C.

d) By preloading and unloading, prestress is induced also in the reinforcement members by an interaction between elasto-plastic properties of soil and rather elastic properties of reinforcement. This prestress can also contribute to maintain the integrity of the wall.

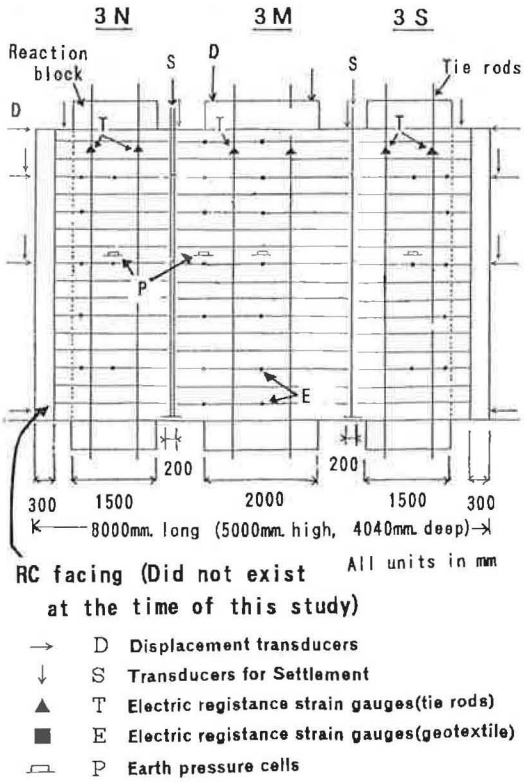

Fig. 17 Cross-section of the test section with three PL/PS GRS test segments

Field model tests: To validate this method, full-scale test walls were constructed in the beginning of 1995 at the Chiba Experiment Station, IIS, University of Tokyo (**Fig. 17**). The test sections have a width of 4 m under plane strain conditions. A well-graded gravel of crushed sandstone was compacted to a dry density ρ_d of 1.88

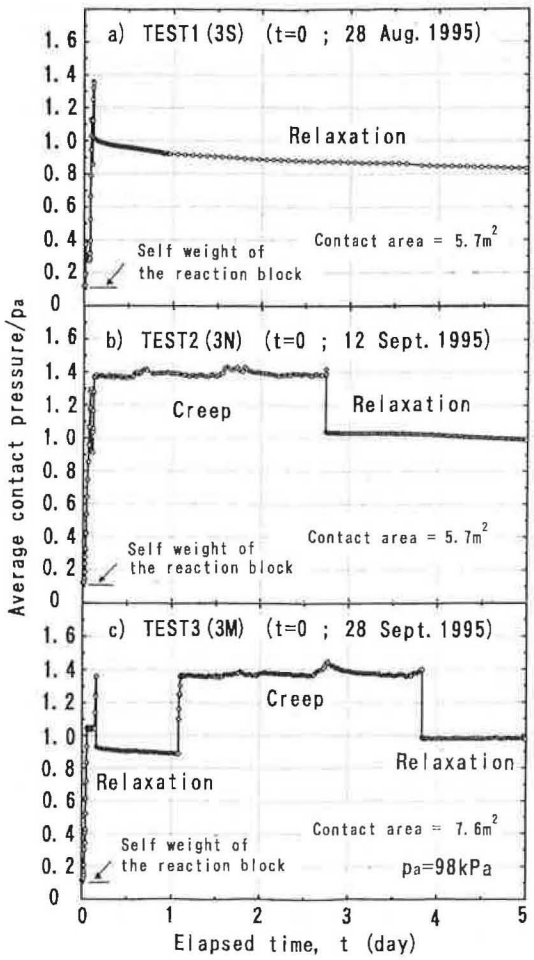

Fig. 18 Time histories of the average contact pressure $(P_O + P_C)/A$ at the top RC block from the end of August 1995; a) 3S, b) 3N and c) 3M; P_O is the weight of the top reaction block and P_C is the applied vertical load (Tatsuoka et al., 1996a)

ton/m^3 with an average water content of 7.0 %. The grid is the one made of PVA (polyvinyl alcohol with the trade mark 'Vinylon') having a nominal tensile rupture strength T_R of 73.5 kN/m (7.5 tonf/m). This type of grid is widely used in the actual construction projects of the GRS-RW system.

From the end of August 1995, test segments 3S, 3N and 3M were preloaded in this sequence. For segment 3S, the first preload with an average contact pressure of 123 kPa (12.6 tonf/m^2) was applied to the top reaction block having a base area of 5.7 m^2 for only about ten minutes (**Fig. 18a**). For segment 3N (**Fig. 18b**), creep deformation was allowed to occur for 63 hours with preload, during which the top block settled about 1.3 cm. For segment 3M having a top RC block with a larger base area of 2 m x 3.8 m = 7.6 m^2, after unloaded from the first short preloading, stress relaxation was allowed to occur for 22 hours (**Fig. 18c**). Then, the second preload was applied, which was maintained for 66 hours with the final creep settlement of about 0.7 cm.

The summary of the time histories of the contact pressure at the bottom of the top reaction block are shown in **Fig. 19**. **Fig. 20** shows the relationship between the average contact pressure and the average settlement of the top RC block in segment 3M. The following trends of behaviour may be noted in these figures:

1) When a noticeable amount of creep deformation of the backfill was not allowed to occur during preloading (i.e., segment 3S and segment 3M after the first preloading), the rate of stress relaxation immediately after unloading was very high.

2) By allowing some amount of creep deformation of the backfill to occur during preloading (i.e., segment 3N and segment 3M after the second preloading), the rate of stress relaxation became very low.

3) The relaxation rate increased suddenly by heavy rainfall with a total precipitation exceeding 130 mm due to Typhoon No. 12 on 17 September 1995. As the gravel had an initial water content of about 7.0 % with a fines content of 8.0 %, it seems that some suction was created during compaction, and part of the suction may have been lost by wetting.

4) When segment 3M was preloaded, a noticeable reduction in the tie rod tension in segments 3N and 3S occurred, due likely to the associated compression of the gravel in these segments.

5) The relaxation rate in segment 3M after the second preloading and in segments 3S and 3N after the preloading of segment 3M became very low. Subsequently, for more than eight months, prestress decreased only slightly.

6) The vertical stiffness of the wall during reloading (from point **a** to point **b** in Fig. 20) is considerably higher than that at the same load during primary loading, which is due to the effects of preloading. Furthermore, as it was reloaded from the prestressed condition (i.e., point a), it is very probable that the stiffness had become larger than that during reloading from a lower load level. This is due to the effects of prestressing.

The results shown above are quite encouraging. In particular, the finding that the relaxation rate of prestress becomes very low due to the creep deformation during the preloading stage is very important. Furthermore, in actual projects, backfill can be compacted to a denser state. In fact, in field compaction tests on the same type of gravel using a heavy compaction machine, a dry density of 2.09 ton/m^3 was easily achieved (Sekine et al., 1996) .

First project: The first application of the PL/PS method to an actual project is now under way at site No. 57 in Fig. 3 (PL/PS GRS-RW site in Fig. 12). In this project (**Fig. 21**), a temporary bridge will be constructed during the summer of 1996 and will be used for about two years, during which an existing bridge over a small river for a railway of the Sasa-guri Line, will be re-constructed. Two 16.5 m - long simple girders for a single track will be supported by a GRS bridge abutment at one end, a PL/PS GRS bridge pier at the center of the river channel, which will be instrumented, and a cement-treated columns at the other end. A soft clay deposit beneath the GRS pier was improved by creating in-place columns of cement-treated clay. Our main concern is the operational rigidity against dead and live loads of the GRS bridge pier, which are considered to be 196 kN (20tonf) and 1280 kN (130tonf) in the design, and effects of the live load on the relaxation

rate of prestress for a long duration. This case will be a great opportunity for examining the feasibility of the PL/PS method for permanent structures.

Other applications: The PL/PS method will be effective also in increasing the seismic stability of GRS-RW's. Furthermore, this method can be used to increase the vertical subgrade reaction of level ground to support a heavy structure which can tolerate limited displacements. In addition, this method is effective not only for compressive load but also for tensile load. Therefore, the rocking motion of a tall structure constructed on PL/PS reinforced soil could be effectively restrained (Tatsuoka et al., 1996a).

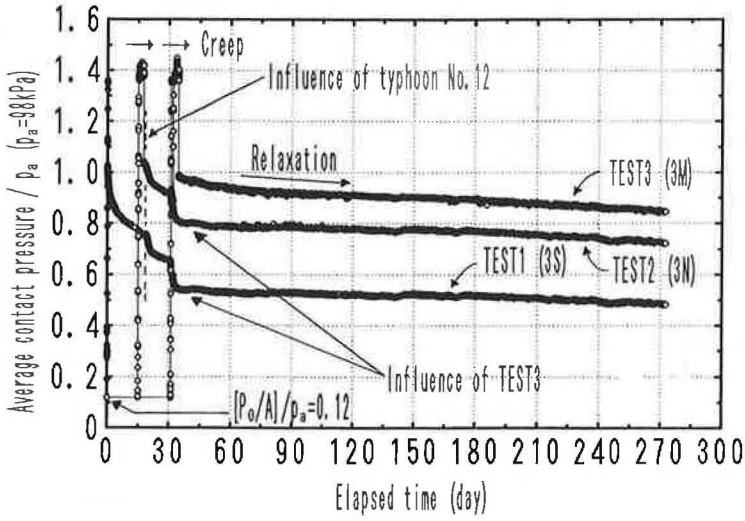

Fig. 19 Summary of the time histories of the average contact pressure ($P_O + P_C$)/A at the top RC blocks on three PL/PS GRS test segments from the end of August 1995, 3S, 3N and 3M

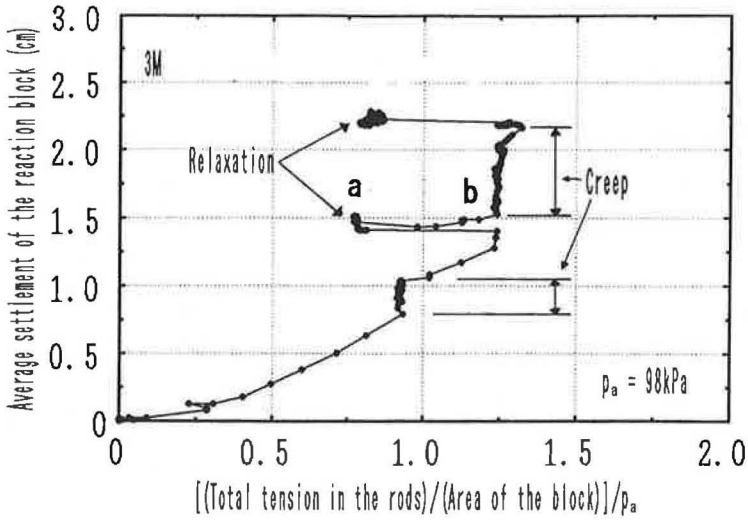

Fig. 20 Relationship between the average contact pressure ($P_O + P_C$)/A and the average settlement of the top RC block, segment 3M

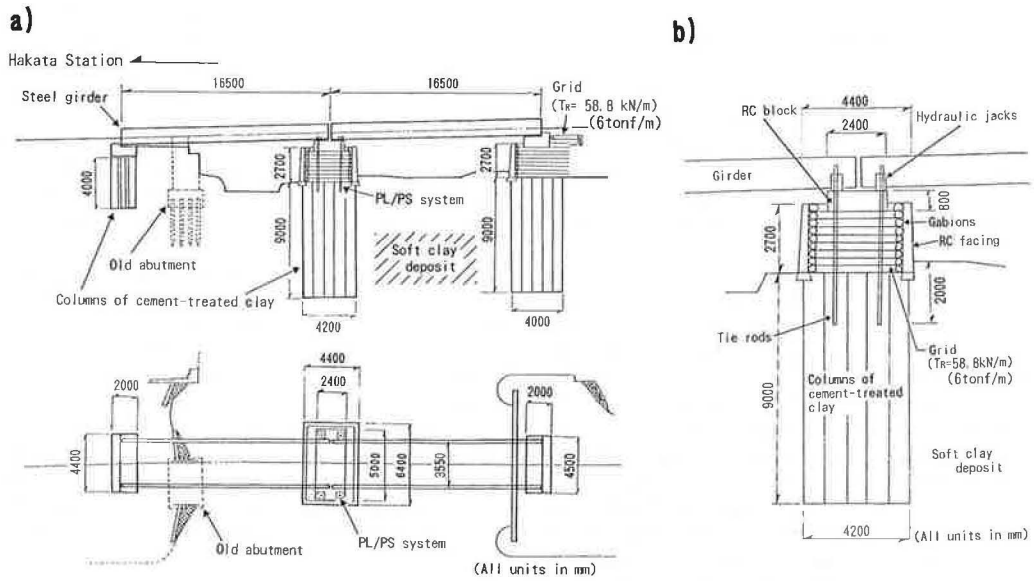

Fig. 21 First PL/PS GRS bridge pier; a) general plan and b) details of the pier, the Sasaguri Line (No. 57 in Fig. 3)

6 SEISMIC STABILITY OF WALL

6.1 General

At 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, including Kobe City and neighbouring urban areas. In the severely affected areas (**Fig. 22**), an extensive length of railway embankment had been constructed more than seventy years ago, which had a number of old retaining walls. Most of the walls were seriously damaged (Tatsuoka et al., 1996, 1996c). The walls can be categorized into the following five groups:

1) masonry retaining walls (denoted by MS in Fig. 22);

2) leaning-type (supported type) unreinforced concrete retaining walls (LT);

3) gravity-type unreinforced concrete retaining walls (GT); and

4) cantilever-type or inverted T-shaped type steel-reinforced concrete (RC) retaining walls (CT).

The first three types of retaining walls were most seriously damaged, while the damage to the last type was generally less serious.

6.2 Tanata wall

Compared with the above four types of retaining walls, the damage to a GRS-RW with a FHR facing at Tanata (GR1 in Fig. 22) was much less serious (**Fig. 23**). This wall was completed in February 1992 on the southern slope of the existing embankment for the JR Kobe Line to increase the number of railway tracks from four to five. The total wall length is 305 m and the largest height is 6.2 m. The surface layer in the subsoil consists of relatively stiff terrace soils (**Fig. 23**). The backfill soil is basically cohesionless soil with a small amount of fines. The reinforcement is a grid (Vinylon) coated with soft PVC for protection, having a nearly rectangular cross-section of 2 mm times 1 mm and an opening of 20 mm having a nominal tensile rupture strength T_R of 30.4 kN/m (3.1 tonf/m).

This wall deformed and moved slightly with the largest outward displacement occurred at the location with the largest height of wall in contact with a RC box culvert structure crossing the railway embankment. The displacement was 26 cm and 10 cm at the top of the wall and at the ground surface level, respectively (**Fig. 24a**). Despite these observations, the

performance of the GRS-RW was considered quite satisfactory based on the following facts:

a) The peak ground acceleration at the site is estimated to be more than 700 gals. That can be inferred also from a very high collapse rate of the Japanese wooden houses at the site (**Fig. 24b**). Many of the collapsed ones were constructed less than about ten years ago.

b) On the opposite side of the RC box structure, a RC retaining wall with the largest height of about 5.4 m (**Fig. 25**) was constructed concurrently with the GRS-RW. This wall is supported by a row of bored piles despite the similar subsoil conditions for the GRS-RW. Therefore, the construction cost per wall length of the RC retaining wall was nearly double to triple of that for the GRS-RW. In addition, a temporary cofferdam still existed at the time of the earthquake in front of the RC retaining wall. This may have somewhat contributed to the stability of the RC retaining wall during the earthquake. Despite these differences, the RC retaining wall displaced similarly to the GRS-RW (**Fig. 25b**); 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.

c) The length of geogrid reinforcement for GRS-RW's with FHR facings is generally shorter than that for most metal strip -reinforced soil retaining walls and other types of GRS retaining walls having deformable facings. For most of the GRS -RW's with FHR facings constructed to date, for conservatism, several reinforcement layers are made longer at higher levels (see Figs. 2a and 26). For the Tanata wall, the length of all reinforcement layers were truncated to nearly the same length due to construction restraints (Fig. 23d). This arrangement may have reduced the seismic stability of the wall; the tilting of the wall would have been smaller if the several top grid layers had been longer.

6.3 Other walls

In addition to the Tanata GRS-RW, the following GRS-RW's had been constructed at other three locations where the seismic intensity was fifth or sixth in the

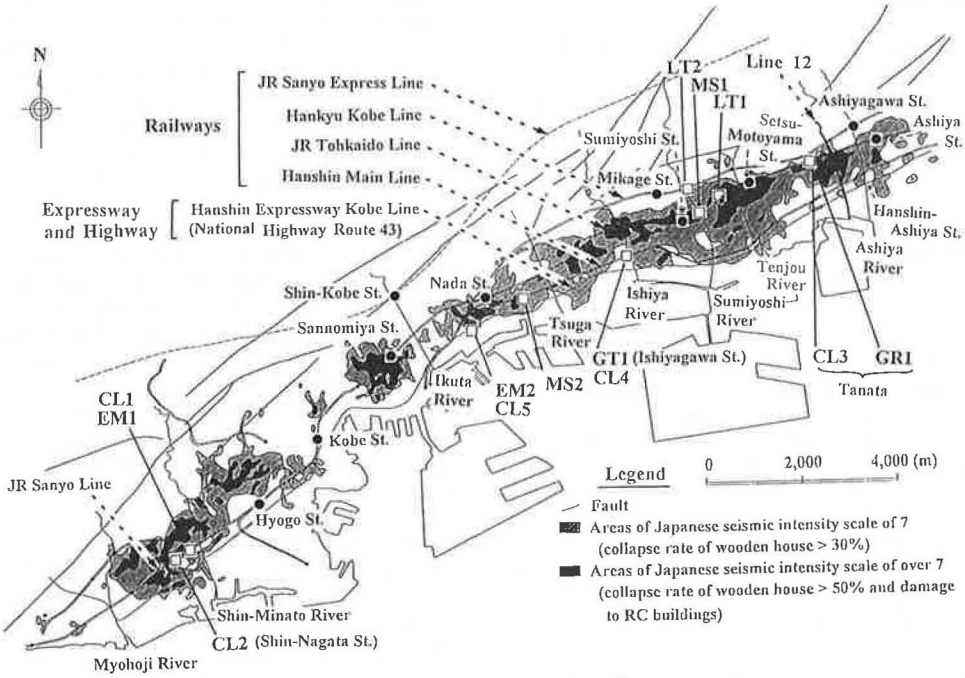

Fig. 22 Seriously damaged areas during the 1995 Great Hanshin Earthquake (modified from Chuo Kaihatsu Corp., 1995); **EM** means damaged embankments

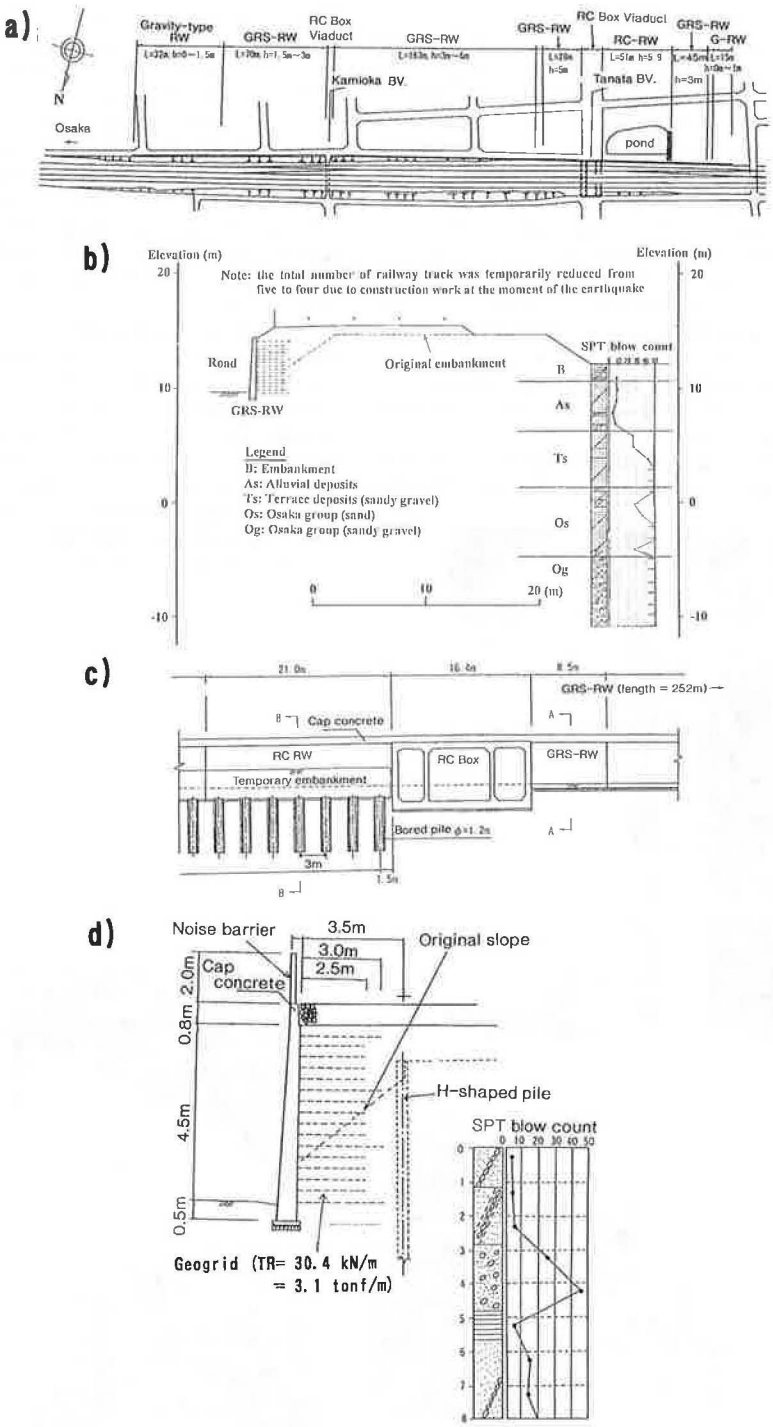

Fig. 23 a) Plan of the site, b) cross-section of embankment, c) front view from south of GRS-RW and RC retaining walls supported with a pile foundation, and d) typical cross-section of the GRS-RW, Tanata (No. 5 in Fig.3 and GR1 in Fig. 22)

Japanese scale and a number of wooden houses, railway and highway embankments and conventional types of retaining walls were seriously damaged. These GRS-RW's, however, were not damaged at all.

a) Amagasaki No.1 (Kanazawa et al., 1994), the first large-scale construction project of the GRS-RW system to support directly tracks of a very busy railway, Kobe Line (**Fig.26**). The average wall height is 5 m and the total length is 1,300 m. At some sections, the foundations for a steel frame structure for electricity supply were constructed inside the reinforced zone. Four pairs of GRS bridge abutments were also constructed to support bridge girders directly.

b) Walls with a largest height of 6.3 m for a total length of 120 m at Maiko site in Tarumi-ku, Kobe City, completed in May 1993 in order to expand the top of the road adjacent to one of the approach roads to the Akashi Kaikyo (Strait) Bridge, which is under construction. This site is located only 5 km from the epicenter.

c) Amagasaki No.2 with a height of 3 - 8 m and a length of about 400 m, located west of the Amagasaki No.1 GRS-RW. The wall was completed in March 1994 to support a new approach fill for a bridge of JR, the Fukuchiyama Line.

One of the mechanisms which make the GRS-RW with a FHR facing much more stable against seismic force than the conventional gravity-type retaining walls would be that the reinforced zone can behave as a relatively flexible monolith having a relatively large width/height ratio. More discussion on the seismic stability of the GRS-RW system is given in Tatsuoka et al. (1996c).

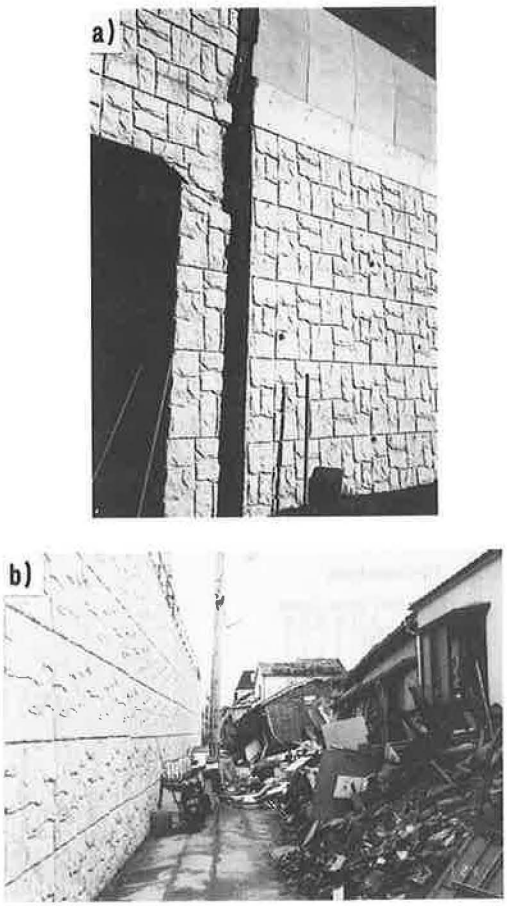

Fig. 24 a) View of the GRS-RW immediately after the earthquake and b) view in front of the GRS-RW at Tanata

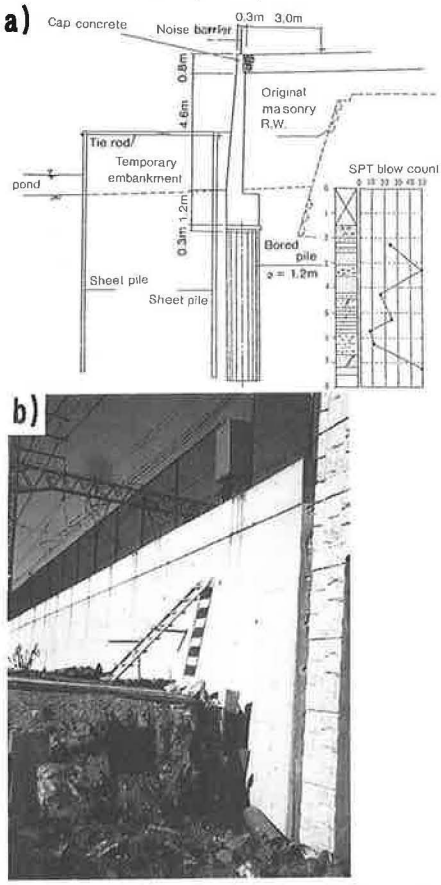

Fig. 25 a) Cross-section and b) view of the RC retaining wall at Tanata

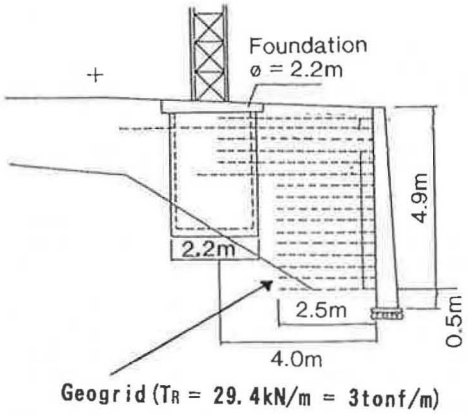

Fig. 26 Typical cross-section of GRS-RW's for the Kobe Line at Amagasaki (No. 7, Fig. 3)

7 SUMMARY

To a much greater extent and at a much higher rate than we anticipated when we started the study, the GRS-RW system reported in this paper has been used to construct important permanent retaining walls and bridge abutments, which are so far mainly for railways. The authors believe that this situation is due mainly to not only its high-cost performance, but also its performance being equivalent to, or even better than, that of other modern RC retaining walls and RC bridge abutments. One of the keys for the above is the use of a proper type of geosynthetic (grid for cohesionless soils or nonwoven/woven geotextile composite for nearly saturated cohesive soils), and another is the use of a full-height rigid facing cast-in-place by staged construction procedures.

It is also hoped that the new construction method by preloading and prestressing can explore new applications of GRS-RW's to structures such as bridge abutments and piers that allow very small displacements.

8 ACKNOWLEDGEMENTS

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