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Remedial treatment of soil structures using geosynthetic-reinforcing technology

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Abstract

The advantages of geosynthetic-reinforcing technology to construct new soil structures including; (a) a relatively short construction period; (b) small construction machines necessary; and (c) a higher stability of completed structures, all contributing to a higher cost-effectiveness, are addressed. A number of case successful histories of geosynthetic-reinforced soil retaining walls have been reported in the literature (e.g., [Tatsuoka, F., Koseki, J., Tateyama, M., 1997a. Performance of Earth Reinforcement Structures during the Great Hanshin Earthquake, Special Lecture. In: Proceedings of the International Symposium on Earth Reinforcement, IS Kyushu '96, Balkema, vol. 2, pp. 973–1008; Tatsuoka, F., Tateyama, M, Uchimura, T., Koseki, J., 1997b. Geosynthetic-reinforced soil retaining walls as important permanent structures, 1996–1997 Mercer Lecture. Geosynthetics International 4(2), 81–136; Tatsuoka, F., Koseki, J., Tateyama, M., Munaf, Y., Horii, N., 1998. Seismic stability against high seismic loads of geosynthetic-reinforced soil retaining structures, Keynote Lecture. In: Proceedings of the 6th International Conference on Geosynthetics, Atlanta, vol. 1, pp.103–142; Helwany, S.M.B., Wu, J.T.H., Froessl, B., 2003. GRS bridge abutments—an effective means to alleviate bridge approach settlement. Geotextiles and Geomembranes 21(3), 177–196; Lee, K.Z.Z., Wu, J.T.H., 2004. A synthesis of case histories on GRS bridge-supporting structures with flexible facing. Geotextiles and Geomembranes 22(4), 181–204; Yoo, C., Jung, H.-S., 2004. Measured behavior of a geosynthetic-reinforced segmental retaining wall in a tiered configuration. Geotextiles and Geomembranes 22(5), 359–376; Kazimierowicz-Frankowska, K., 2005. A case study of a geosynthetic reinforced wall with wrap-around facing. Geotextiles and Geomembranes 23(1), 107–115; Skinner, G.D., Rowe, R.K., 2005. Design and behaviour of a geosynthetic reinforced retaining wall and bridge abutment on a yielding foundation. Geotextiles and Geomembranes 23(3), 234–260]). Techniques for analyzing the seismic response of reinforced walls and slopes have also been developed (e.g. Nouri, H. Fakher, A., Jones, C.J.F.P., 2006. Development of horizontal slice method for seismic stability analysis of reinforced slopes and walls. Geotextiles and Geomembranes 24(2),175–187). Several typical cases in which embankments having a gentle slope and conventional-type soil retaining walls that were seriously damaged or failed were reconstructed to geosynthetic-reinforced steepened slopes or geosynthetic-reinforced soil retaining walls are also reported in this paper. It has been reported that the reconstruction of damaged or failed conventional soil structures to geosynthetic-reinforced soil structures was highly cost-effective in these cases. Rehabilitation of an old earth-fill dam in Tokyo to increase its seismic stability by constructing a counter-balance fill reinforced with geosynthetic reinforcement is described. Finally, a new technology proposed to stabilize the downstream slope of earth-fill dams against overflowing flood water while ensuring a high seismic stability by protecting the slope with soil bags anchored with geosynthetic reinforcement layers arranged in the slope is described.

1. Introduction

Based on the experiences on vast and serious damage to civil engineering structures during the 1995 Hyogo-ken Nambu earthquake (i.e., the 1995 Kobe earthquake), the seismic design codes for RC and steel structures were revised substantially so that the structures can withstand

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such a high level seismic load as experienced during the earthquake. On the other hand, a great number of conventional-type soil structures (e.g., embankments and soil retaining walls for railways, highways and residential areas) were seriously damaged during a number of recent major earthquakes, including the 1995 Kobe earthquake, and heavy rainfalls in Japan. The seismic design codes for soil structures that should be followed when designing new soil structures have also been revised. However, the revision was generally not as substantial and sufficient as those for steel-reinforced concrete and steel structures. Moreover, soil structures that are damaged or failed by earthquakes and heavy rainfalls are usually reconstructed to those of the original structural type, although it is anticipated that the reconstructed soil structures would not perform satisfactorily when subjected again to seismic load and rain fall of the similar level.

It has been often claimed, and it is still, that these situations for soil structures described above should be accepted for the following reasons:

(1) It is most urgent to recover as soon as possible the original function of damaged or failed structure even if the reconstructed soil structures may not have the same level of structural stability as before.
(2) The reconstruction of damaged or failed soil structures to those having stability significantly higher than the original ones, hopefully to the level to withstand the high-seismic design load against which RC and steel structures should be designed, is more time-consuming and more costly than the reconstruction to the original structural level.
(3) The number of soil structures that have been and will be constructed is so large that it is financially not feasible to rehabilitate old soil structures and to construct new soil structures so that they have a similar stability for seismic load and rainfall as RC and steel structures.
(4) It is generally much easier and faster to reconstruct damaged or failed soils structures when compared with the case of RC and steel structures. So, soil structures could be allowed to be seriously damaged or even fail in many cases.

However, many soil structures are as essential as RC and steel structures for the transportation system and other infrastructures for civil life. Moreover, the reconstruction of damaged or failed conventional-type soil structures (e.g., embankments having a relatively gentle slope and gravity-type or cantilever RC soil retaining walls supported with a pile foundation) to embankment with a geosynthetic-reinforced steepened slope and geosynthetic-reinforced soil retaining walls is more cost-effective, while the reconstructed soil structures have a stability substantially higher than that of damaged or failed soil structures. This high cost-effectiveness of reconstruction to geosynthetic-reinforced soil structures comes from a high structural stability of completed structure as well as a smaller amount of earthwork, a higher constructability under restricted conditions (e.g., a narrow space, a steep slope or a remote place) and a faster construction speed. Moreover, arrangement of a relevant drainage system into geosynthetic-reinforced backfill is usually much easier than with conventional-type soil structures. In addition, it is possible to construct geosynthetic-reinforced soil structures having a high resistance against water flow action. For these reasons, it can be said that it is not relevant to reconstruct damaged or failed conventional-type soil structures to those having the respective original structural form. Finally, the geosynthetic reinforcing technology is also useful to reinforce and rehabilitate old soil structures (e.g., embankments, soil retaining walls and earth fills) to have a higher stability against seismic load, rain fall (e.g., Tatsuoka and Yamauchi, 1986) and water flow.

In this paper, a geosynthetic-reinforced soil retaining wall (GRS-RW) technology that was developed to replace conventional-type soil retaining walls is first described. Then, several case histories (Tatsuoka et al., 1997a,b, 1998) in which conventional-type embankments and soil retaining walls that were seriously damaged or totally failed by heavy rainfall and seismic load were reconstructed to embankments having geosynthetic-reinforced steepened slopes and GRS-RWs having a full-height rigid (FHR) facing are reported. A case history in which the seismic stability of the down stream slope of an old earth-fill dam is being increased by constructing a counter weight fill having a steep slope reinforced with geosynthetic reinforcement is briefly described.

2. Geosynthetic-reinforced soil retaining walls with a FHR facing

A construction system for 'permanent' geosynthetic-reinforced soil retaining walls (GRS-RWs) (Fig. 1; Tatsuoka et al., 1997b) is now widely used in Japan. This system is characterized by the following features:

(1) The use of a FHR facing that is cast-in-place using staged construction procedures (Fig. 1a). The geosynthetic reinforcement layers are firmly connected to the back face of the facing.
(2) The use of a polymer geogrid reinforcement (Fig. 1b) for cohesionless soils to provide good interlocking with the backfill, and the use of a composite of non-woven and woven geotextiles for nearly saturated cohesive soils to facilitate both drainage and tensile reinforcement of the backfill. Low-quality on-site soil is used as the backfill, if necessary.
(3) The use of relatively short reinforcement.

The staged construction method (Fig. 1a) is one of the main features of the system, which consists of the

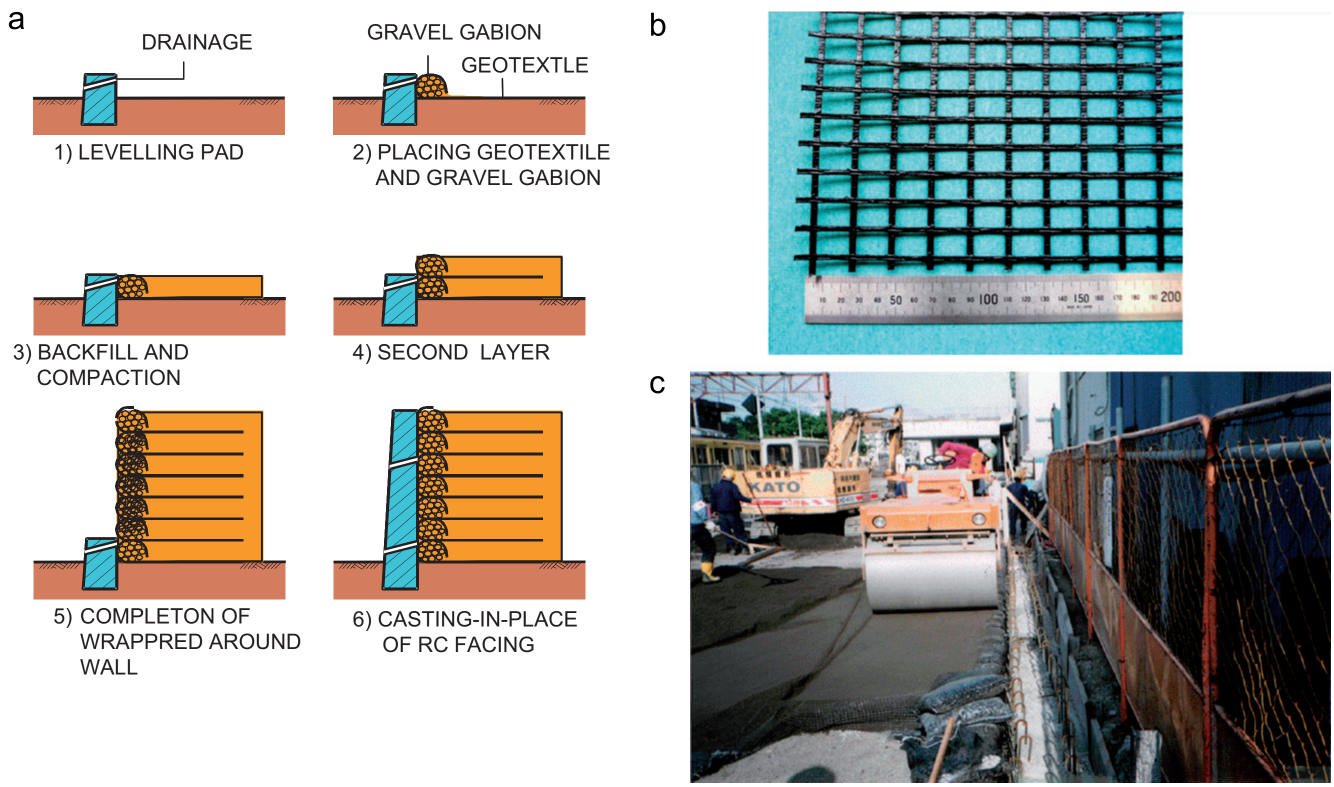

Fig. 1. (a) Standard staged construction procedure for a GRS-RW; (b) a typical geogrid (polyester); and (c) compaction of a soil layer close to the wall face.

following steps:

(1) A small foundation for the facing is constructed.
(2) A geosynthetic-reinforced soil wall with a wrap-around wall face is constructed using gravel-filled bags placed at the shoulder of each soil layer (Fig. 1c).
(3) A thin and lightly steel-reinforced concrete facing is cast-in-place directly on the wall face after the major part of ultimate deformation of the backfill and the subsoil layer beneath the wall has occurred. A good connection should be made between the facing and the main body of the wall.

The geosynthetic-reinforced soil retaining wall system is cost-effective, because the facing structure is much simpler than the conventional soil retaining wall systems, which results in lower construction cost, a higher construction speed and the use of lighter construction machines. Despite the above, the wall performance can be equivalent to, or even better than, conventional-type soil retaining walls. In addition, due to a flexibility of GRS-RW, the pile foundation that supports conventional-type soil retaining walls in usual cases becomes unnecessary, resulting in a more cost-effective system. This point is discussed below.

First, a conventional-type soil retaining wall is designed as a cantilever structure supported at its base resisting against the active earth pressure, P_A, from the unreinforced backfill (Fig. 2a). Therefore, large internal moment and

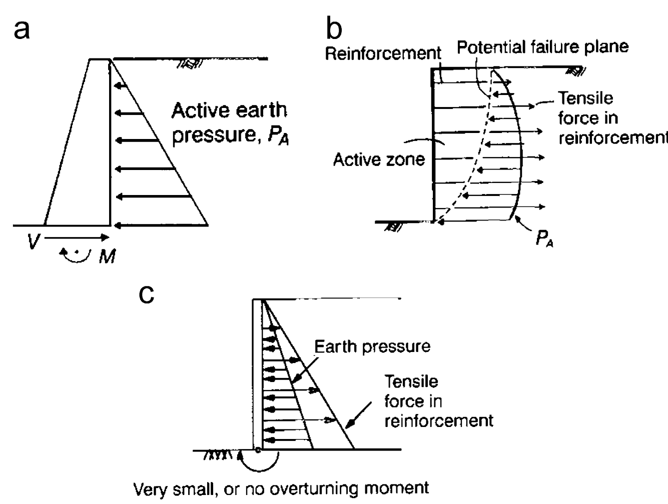

Fig. 2. Force equilibrium for: (a) conventional-type soil retaining wall; (b) reinforced soil retaining wall; and (c) FHR facing of a GRS-RW.

shear force are mobilized inside the facing structure, and large overturning moment and sliding force develop at the bottom of the wall structure (Fig. 3a). In the case of reinforced soil retaining wall, on the other hand, the backfill is retained by the tensile force in the reinforcement (Fig. 2b). The conventional explanation of the function of the facing, which is actually misleading, is that, because of tensile-reinforcement effects, only very small earth pressures act on the back of the facing, accordingly, only a light and flexible facing that can prevent the backfill soil from

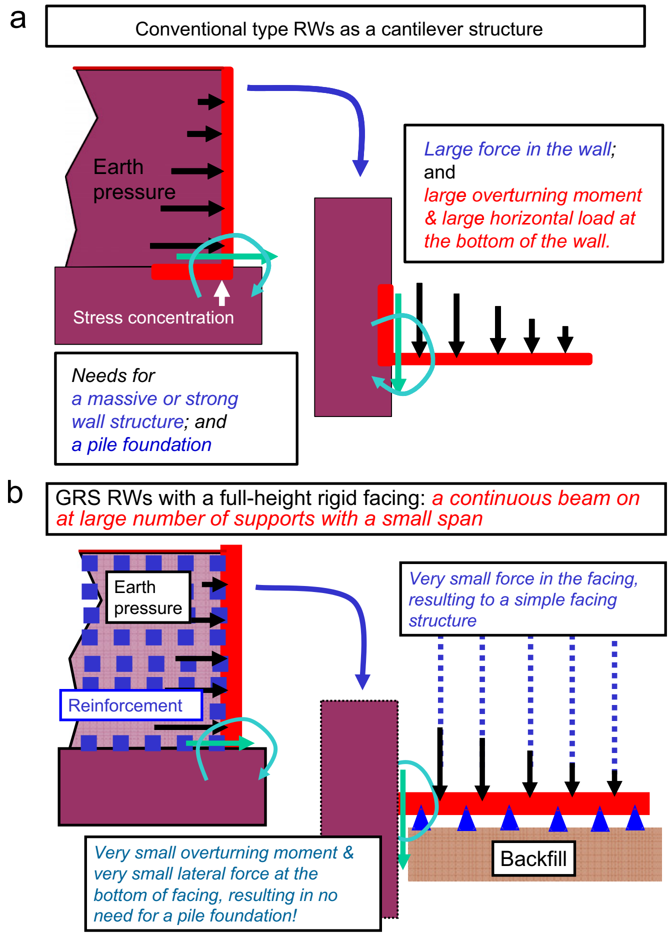

Fig. 3. Mechanisms of: (a) cantilever soil retaining wall; and (b) reinforced soil retaining wall with a FHR facing.

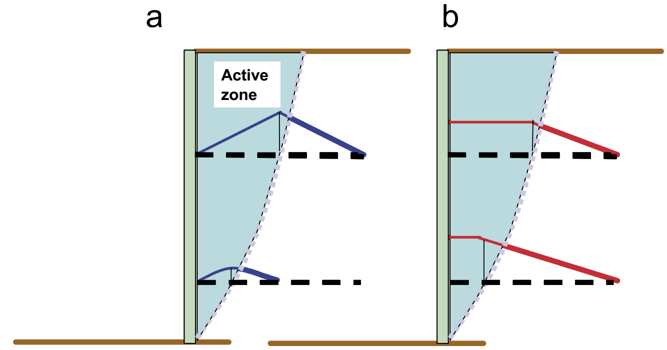

Fig. 4. Distribution patterns of tensile force in the reinforcement: (a) when no facing is used or when the facing and reinforcement are not connected; and (b) when a rigid facing and reinforcement are connected.

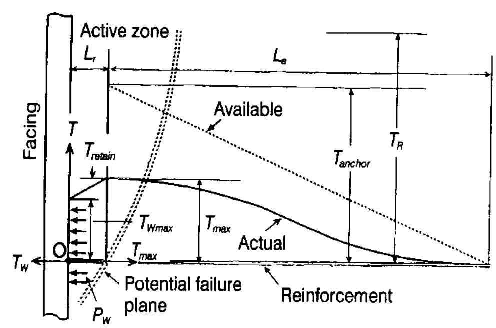

Fig. 5. Available maximum tensile force in the reinforcement arranged in a reinforced soil retaining wall (Tatsuoka, 1993 [6]).

spilling out is sufficient. However, if the earth pressure activated on the back of the facing is nearly zero, the tensile force at the connection between the reinforcement and the back of the facing becomes zero, which results in a large reduction of the soil retaining capability of the reinforcement (Fig. 4a) (Tatsuoka, 1993). As the lateral confining pressure on the soil in the active zone decreases, the active zone becomes more deformable and less stable, particularly when the backfill is cohesionless. On the other hand, when the reinforcement layers are connected to a rigid facing, the large earth pressure, as P_A, can act on the back of the facing (Fig. 2c), resulting into large connection force in the reinforcement (Fig. 4b). This situation, in which the confining pressure in the active zone is maintained to be high resulting in a high stiffness and small deformation of the active zone, is the preferred stable condition. Then, a FHR facing for a GRS-RW can behave as a continuous beam with a large number of supports with a small span (Fig. 3b). Then, only small force is activated inside the facing, resulting to a simple facing structure and insignificant overturning moment and lateral force activated at the bottom of facing resulting into no need for a pile foundation in usual cases.

With respect to the tensile force distribution of reinforcement, the maximum available tensile force, T_{max}, in each reinforcement member is obtained as (Fig. 5):

$$T_{max} = \text{Minimum of } [T_R,\ T_{anchor},\ T_{retain} + T_{W\,max}], \quad (1)$$

where T_R, the tensile rupture strength of each reinforcement layer; T_{anchor}, the anchorage capacity, which is approximately proportional to the anchorage length, L_a; T_{retain}, available retaining strength, which is approximately proportional to the retaining length, L_r; and, $T_{W\max}$, available tensile force at the connection between the reinforcement and the back of the facing, which increases with an increase in the available earth pressure on the back of the facing. When $T_{W\max}$ decreases to zero, the distribution of the reinforcement tensile force, T, becomes those presented in Fig. 4a. Thus, at lower levels in the wall, T_{max} cannot become large enough. On the other hand, a large $T_{W\max}$ value results in a large T_{max} value (i.e., Fig. 4b). It is assumed that, with this distribution pattern, the active zone is confined without exhibiting large strains, and therefore, large bond stresses are not mobilized at the surface of the reinforcement in the active zone. Consequently, the reinforcement tensile force, T becomes rather constant in the active zone.

The general trend of construction of elevated transportation structures in Japan is a gradual conversion from embankments having a gentle slope towards embankments supported with soil retaining walls (usually RC cantilever RWs with a pile foundation) or RC frame structures for higher ones (Fig. 6). Previously, the Terre Armee technique

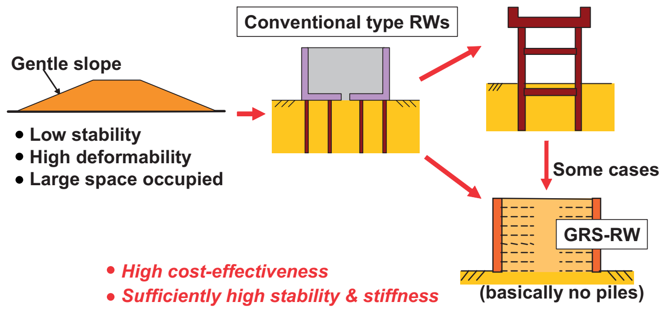

Fig. 6. History of elevated railway and highway structures in Japan.

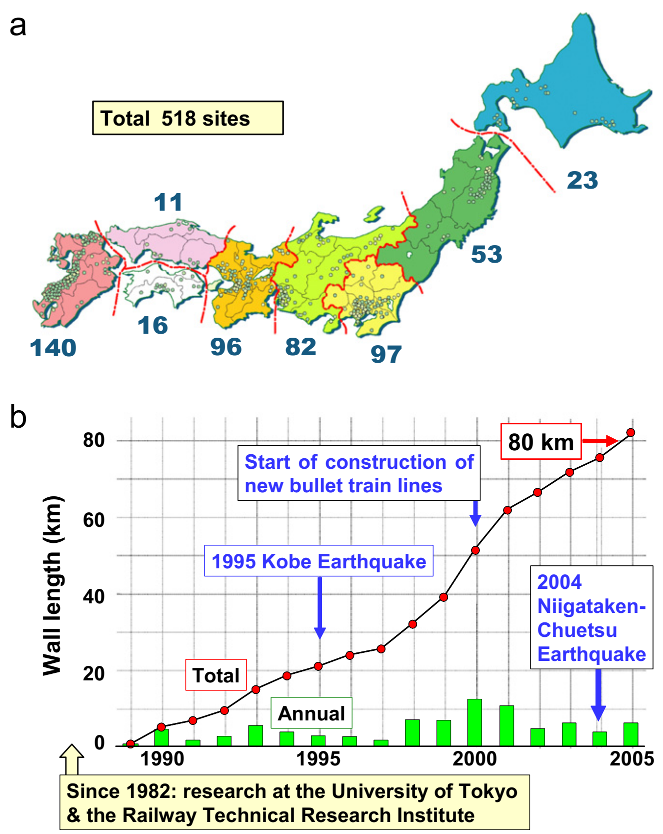

Fig. 7. (a) Locations of GRS-RWs projects in Japan; and (b) accumulated and annual lengths of constructed wall lengths as of the end of 2005 (by the courtesy of Dr. Tateyama, M.)

dominated the permanent reinforced-soil retaining wall market in Japan. Today, the use of the Terre Armee construction technique for railway support structures in Japan has declined to nearly zero. Recently, because of a higher cost-effectiveness and a sufficiently high stability and low deformability, the GRS-RW system (Fig. 1) is routinely adopted instead of conventional RC retaining wall techniques and the Terre Armee technique. Fig. 7 shows the locations of the major GRS-RW projects that have been constructed as of the end of 2005 and the length of the constructed walls. Two typical large-scale railways application projects are those in Nagoya (Fig. 8) and Amagasaki (Fig. 9). The stability of GRS-RW systems has been validated by the excellent post-construction performance of the constructed walls. It should be emphasized that no problematic cases, for example cases with too large wall deformation, have not been reported.

3. Damage by heavy rainfall and remedy works

3.1. Damage in 1989 and reconstruction

In the Mount Aso area in Kyushu Island, a series of railway embankments on the Ho-hi Line located in narrow valleys failed by a heavy rainfall on 2 July 1989 (Fig. 10). The damage was caused by the flow of flood water over the embankments due to the clogging of a drain pipe crossing

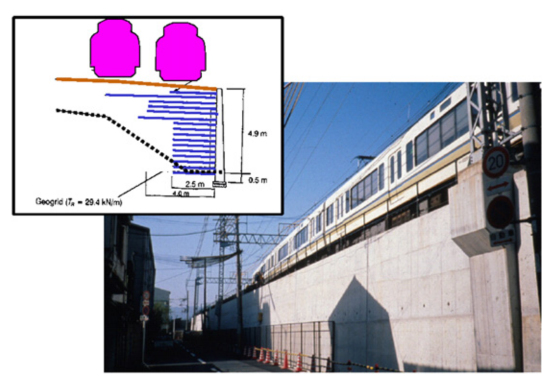

Fig. 9. GRS-RW with a FHR facing constructed to support rapid transit train tracks (Kobe Line), Amagasaki.

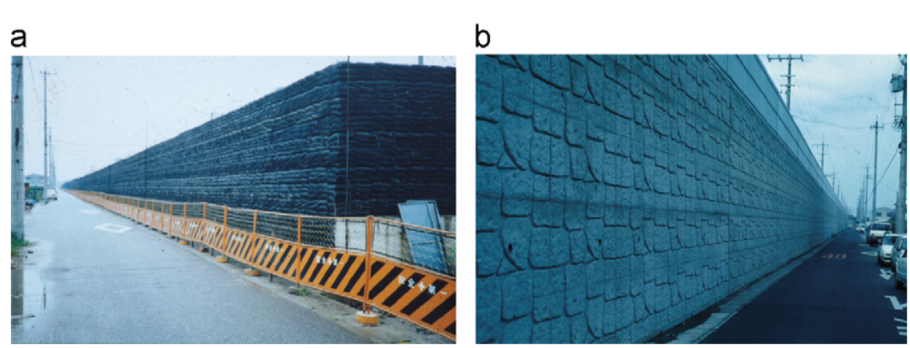

Fig. 8. GRS-RW with a FHR facing constructed to support a yard for Bullet train (Shinkansen), Nagoya: (a) wall under construction (at stage 5 in Fig. 1a); and (b) completed wall (at stage 6).

the respective embankments. The entire sections of six embankments were reconstructed (Fig. 11). To reduce the amount of earthwork, a nearly vertical GRS-RW with a

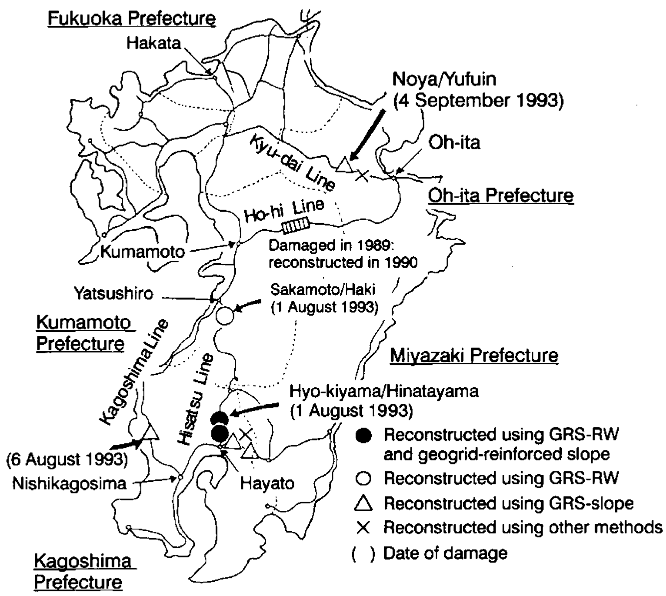

Fig. 10. Locations of railway embankments seriously damaged by heavy rainfalls in 1989 and 1993.

FHR facing was constructed at the downstream toe of the respective embankments, and the slope was steepened by being reinforced with geogrid layers. This remedial work is characterized by large embankment heights and a large diameter corrugated steel drain pipe installed in the respective embankments.

3.2. Damage in 1993 and reconstruction

From June to September 1993, many railway embankment sections in the central and southern Kyushu were seriously damaged or destroyed by a series of heavy rainfalls (see Fig. 10 for the locations of the sites). The scale of damage was significantly greater than the one in 1989. The total precipitation during these months in Kagoshima and Miyazaki Prefectures amounted to approximately 3000 mm. By November 1993, damaged embankment sections (solid circles in Fig. 10) were reconstructed using a method similar to the one used in 1989. The total volume of the damaged embankments that were reconstructed was 18,700 m^3. At Hyo-kiyama/Hinatayama site (Fig. 12), the backfill material, which was a pumice called Shirasu, was washed away from the embankment toe by a flood. The total volume of crushed gravel used for the reconstruction was 8640 m^3. This reconstruction method was adopted

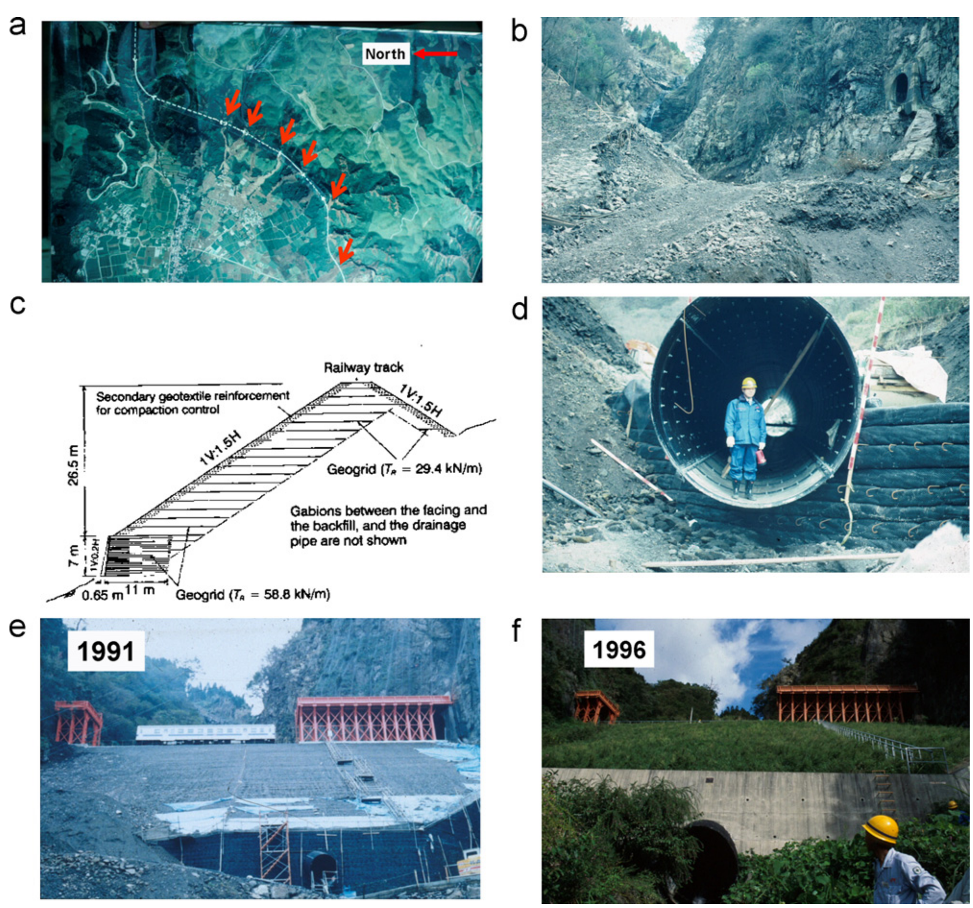

Fig. 11. Mount Aso site of Ho-hi Line: (a) air view of the site; (b) typical failed embankment; (c) typical cross section of reconstructed embankment; (d) a large-diameter corrugated steel drainage pipe installed in the reconstructed embankment; and (e) and (f) views during and after reconstruction.

based on the successful previous construction case history for the Ho-hi Line described above (i.e., a low construction cost, a relatively short construction period and relatively light construction equipment required). The last two factors are particularly important because rapid remedial work was required, and most of the damaged embankments were located in remote mountainous areas. At the site denoted by a hollow circle in Fig. 10 (Sakamoto/Haki site), a conventional masonry retaining wall was completely destroyed for a length of approximately 59 m by flooding of the adjacent river. The wall was reconstructed to a GRS-RW. At several other sites denoted by hollow triangles in Fig. 10, the slopes of the damaged embankments (total soil volume of 7700 m^3) were reconstructed being reinforced with geosynthetic reinforcement.

4. Damage by earthquakes and remedy works

4.1. 1995 Kobe earthquake

At 5:46 a.m. on 17th January, 1995, a devastating earthquake measuring 7.2 on the Richter scale occurred in

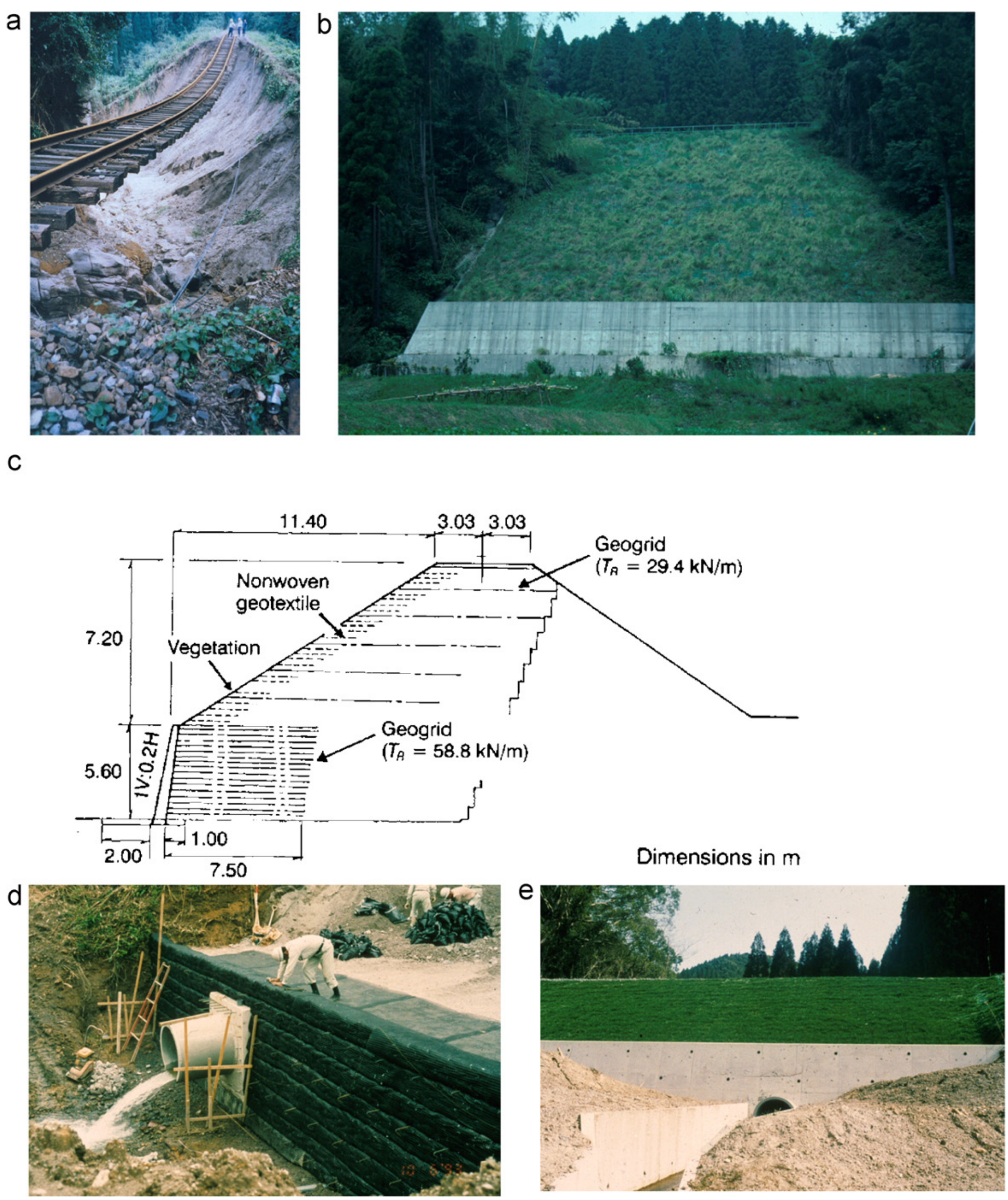

Fig. 12. Reconstruction of railway embankment failed in 1993 at Hyo-kiyama/Hinatayama site, the Hisatsu Line: (a) failed embankment; (b) reconstructed embankment; (c) typical cross section of reconstructed embankment; and (d) and (e) a drainage pipe installed at another site.

the southern section of Hyogo Prefecture, including Kobe City and neighbouring urban areas. In the severely affected areas (Fig. 13), an extensive length of railway embankments had been constructed more than 70 years

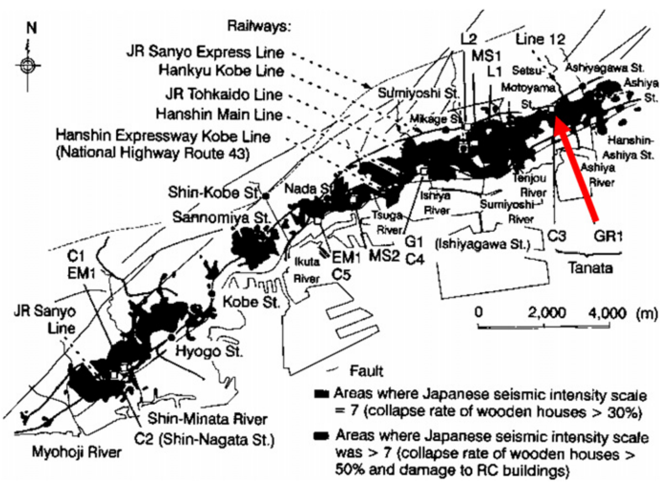

Fig. 13. Seriously damaged areas during the 1995 Great Hanshin earthquake (modified from Chuo Kaihatsu Corporation, 1995). *Note*: EM = damaged embankments.

before the earthquake. Most of the many old and new retaining walls for these embankments were seriously damaged (Fig. 14; Tatsuoka et al., 1996, 1997a, b, 1998). The conventional soil retaining walls can be categorized into four groups: (1) masonry soil walls (denoted by MS in Fig. 13); (2) leaning (supported), unreinforced concrete walls (L); (3) gravity, unreinforced concrete walls (G); and (4) cantilever or inverted T-shaped steel-reinforced concrete (RC) walls (C). The first three types of soil retaining walls were the most seriously damages. The damage to cantilever walls was also very serious although it was generally less serious than the two conventional types of RWs described above. Many of the damaged conventional type RWs were reconstructed to GRS-RWs with a FHR facing.

The damage to a GRS-RW with a FHR facing that had been constructed at Tanata site (GR1 in Fig. 13) was substantially less serious when compared to the four conventional types of soil RWs described above. The Tanata wall was completed in February 1992 on the southern slope of an existing embankment for the JR Kobe Line to increase the number of railway tracks from four to five (Fig. 15a). The total length of the GRS-RW is 305 m and the greatest height is 6.2 m. The surface layer in

Fig. 14. Typical damage to conventional type RWs; (a) gravity-type wall at Ishiyagawa Station, Hanshin Line (site G1 in Fig. 13); and (b) and (c) gravity (leaning type) and masonry walls (sites L1 and MS1 in Fig. 13) and reconstruction to GRS-RWs with a full-height rigid facing at Sumiyoshi, JR Kobe Line.

the subsoil consists of relatively stiff terrace soils. The backfill soil is basically cohesionless with a small amount of fines. The reinforcement is a geogrid made of PVA (polyvinyl alcohol) coated with soft PVC (polyvinyl chloride) for protection having a nearly rectangular cross section of 2 mm by 1 mm and an aperture size of 20 mm with a nominal tensile rupture strength $T_{\mathrm{R}} = 30.4\,\mathrm{kN/m}$. The wall deformed to a limited extent and moved slightly laterally in an outward direction. The largest outward displacement occurred at the highest wall location, which is in contact with a RC box culvert structure crossing the railway embankment. The displacement was 260 and 100 mm at the top of the wall and at ground level. Despite the above, the performance of the GRS-RW was considered very satisfactory because of the following facts:

1. The peak ground horizontal acceleration at the site was estimated to be more than 700 gals (i.e., 700 cm/s^2; or 0.7 g, where g is the gravitational acceleration). This is

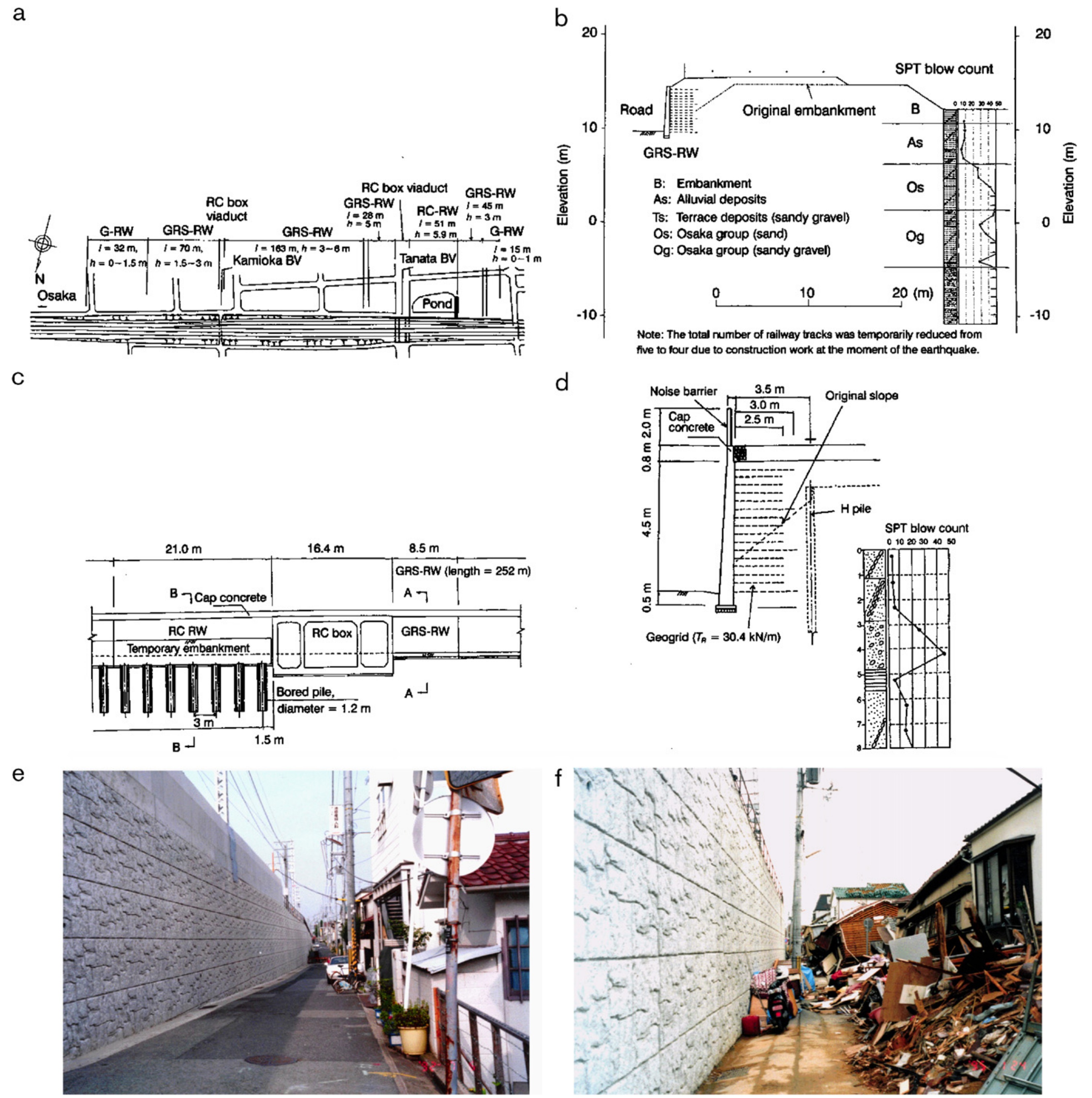

Fig. 15. Walls at Tanata: (a) plan view (G-RW, gravity soil retaining wall); (b) cross section of the embankment; (c) front view (from the south); (d) typical cross section of the GRS-RW; (e) completed wall (July 1992); and (f) a week after the earthquake.

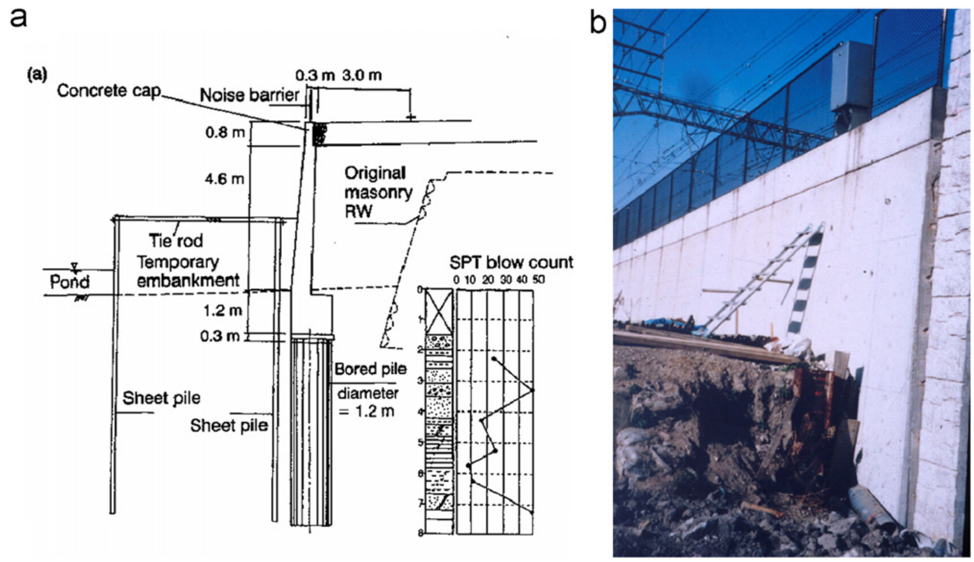

Fig. 16. Cantilever RC soil retaining wall at Tanata: (a) cross section; and (b) front view.

consistent with a high collapse rate of wooden houses at the site (Fig. 15f). Many of the collapsed houses were relatively new, constructed less than approximately 10 years ago.

2. On the opposite side along the railway of the RC box structure, a RC retaining wall with a maximum height of approximately 5.4 m (Fig. 16) had been constructed at the same time as the GRS-RW. This wall is supported by a row of bored piles despite the similar subsoil conditions as the GRS-RW. Therefore, the construction cost per wall length of the RC retaining wall was approximately from double to triple to that of the GRS-RW. In addition, a temporary cofferdam still existed in front of the RC retaining wall at the time of the earthquake, which should have contributed to the stability of the RC retaining wall during the earthquake. Despite these differences, the RC retaining wall displaced similarity to the GRS-RW: i.e., at the interface with the RC box structure, the outward lateral displacement of the retaining wall was 215 mm at the top and 100 mm at ground level.
3. The length of geogrid reinforcement used in GRS-RWs with a FHR facing is generally much shorter than that for most metal strip-reinforced soil retaining walls and other types of GRS retaining walls with a deformable facing. For conservatism, most of the GRS-RWs with a FHR facing that have been constructed to date have several longer reinforcement layers at higher levels of the wall (Fig. 9). For this GRS-RW at Tanata, the length of all reinforcement layers was truncated to approximately the same length due to construction restraints at the site (Fig. 15d). This arrangement may have reduced the seismic stability of the wall; the wall would have tilted less if the top geogrid layers had been made longer.

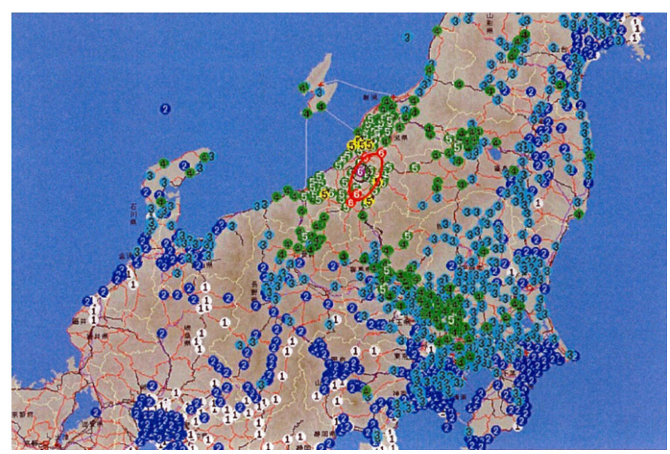

Fig. 17. Distribution of the JMA-scale seismic intensity.

4.2. 2004 Niigata-ken Chuetsu earthquake

An intense earthquake of magnitude 6.8 hit the mid Niigata Prefecture, on the West coast of Honshu, Japan, at 17:56 JST on October 23, 2004. The hypocenter of the main shock was located at 37.3 N and 138.8 E with a depth of 13 km, 195 km (120 m) North-Northwest of Tokyo (Fig. 17). The maximum intensity of 6+ on the 7-grade Japanese intensity scale was reached. The main shock was followed by a series of strong aftershocks in a rapid succession, including four of magnitude 6 or greater within 38 min after the main shock, which should also have damaged the area. The Geographical Survey Institute (GSI) of the Government of Japan published preliminary estimates that a fault having a length of 22 km and a width of 17 km moved approximately 1.4 m.

This was the deadliest earthquake to strike Japan since the 1995 Kobe earthquake. The maximum acceleration of 1500 gal was recorded at Ojiya station, the nearest K-NET site to the hypocenter. This acceleration was much greater than the one recorded during the 1995 Kobe earthquake. As of November 5, 2004, 40 people were reported killed and 2900 injured. There were 395 buildings destroyed and 3473 damaged in the Niigata Prefecture in addition to serious damage done to a large number of embankments and soil retaining walls for railways, highways and residential areas. Compared to the damage to steel-reinforced concrete structures, the scale and extent of the damage to soil structures and its effect on the transportation system and associated civilian life were much more extensive. In particular, a number of embankments that had been constructed in narrow valleys and on slopes (Fig. 18), where ground and surface water tends to concentrate, experienced the greatest damage. It is likely that the effect of heavy rainfall two days before the earthquake resulted in the backfill becoming more saturated than under typical conditions.

Figs. 19–21 show the damage to four embankments constructed within a length of 2 km on the slope of the right bank of Shinano River (Fig. 18), which are typical of the damage for highways, railways and residential areas by the earthquake. The failure of these three embankments delayed the reopening of the railway line. These embankments were constructed in eroded depressions in the river terrace that had up-heaved tectonically of Shinano river. The failure surfaces were located in weathered sedimentary softrock of late Tertiary to early Pleistocene (Uonuma group) or the overlying backfill of gravelly soil or in both. At site 1 (220.3 km of Jo-Etsu Line from Tokyo, Fig. 19), the railway embankment for Jo-Etsu Line was supported by a gravity-type soil retaining wall at its slope toe. The embankment totally failed for a length of about 90 m in the railway direction. The depth of the failure surface was about 7 m (Morishima et al., 2005). The total volume of

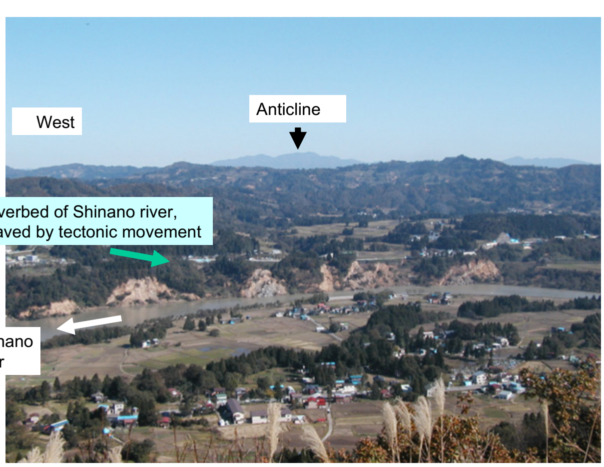

Fig. 18. View of the right bank of Shinano river between Ojiya and Kawaguchi, where embankments for the Jo-Etsu railway line and National Highway Route 17 were severely damaged.

failed soil was about 9900 m^3. The failed soil mass reached the river. At site 2 at Ten-no, Ojiya city (221.000 km, Fig. 20), an embankment for National Highway Route 17, which was retained by a 2 m high gravity-type soil retaining wall, failed. Buried gas and water pipes and information cables were also damaged. The embankment of Jo-Estu Line, located at the lower level, failed together. The toe of the slope of the railway embankment had been supported by a 4 m high gravity-type soil retaining wall. The total amount of failed soil reached about 13,000 m^3. At site 3 (222.000 km, Fig. 21), the embankment that was constructed in a depression that had been made by erosion of the river totally failed. The depth of the failure surface below the two tracks was 13–15 m. The embankment failed for a length of about 40 m with the maximum depth of failure surface of about 9 m. The volume of the failed soil was about 6500 m^3.

The three failed embankments for Jo-Etsu Line at these three sites were reconstructed to three GRS-RWs (illustrated in Fig. 1) within *two* months after the failure. The failed gravity-type soil retaining wall for National Highway Route 17 at site 2 was reconstructed to another type of GRS-RW. At site 1 (Fig. 22), a set of ground anchor was arranged to prevent a failure along inclined bedding planes in the surface weathered sedimentary soft rock layer (Fig. 19b). The base ground for the GRS-RW was improved to a depth of 1 m by cement mixing with a cement weight of 150 kg/m^3, which was then covered with a drainage layer consisting of crushed gravel. Geogrid reinforcement layers were arranged at a vertical spacing of 30 cm following the construction standard. The total volume of backfill is 1800 m^3, which is substantially smaller than the volume of the failed soil (i.e., about 9900 m^3). A FHR facing of concrete with a thickness of 30 cm is 6.9 m high was subsequently constructed. At site 2 (Fig. 23), a GRS-RW with the largest height of 13.2 m was constructed. The volume of the backfill is equal to 4600 m^3, which is also substantially smaller than the volume of the failed soil (i.e., about 13,000 m^3). A drainage layer and drainage piles were arranged at, respectively, the back and the bottom of the reconstructed embankment. Fig. 23c shows the first train running on the wall at site 2. The 4.6 m high GRS-RW at site 3 was completed first (Fig. 24). The backfill was a crushed gravel (C40, with a maximum particle diameter equal to 40 mm). A 3.95 m high geosynthetic-reinforced fill with a slope of 1:1.5 (V:H) was constructed on the GRS-RW. A set of gravel-filled steel mesh gabions were arranged at the toe of the fill in front of the GRS-RW for a better drainage from the supporting ground (Fig. 21b).

This type of GRS-RW with a FHR facing was chosen to reconstruct the failed embankments to stable permanent (not temporary) structures for the following reasons:

(1) Several GRS-RWs of this type performed very satisfactorily during the 1995 Kobe earthquake. A number of conventional-type retaining walls (gravity, leaning,

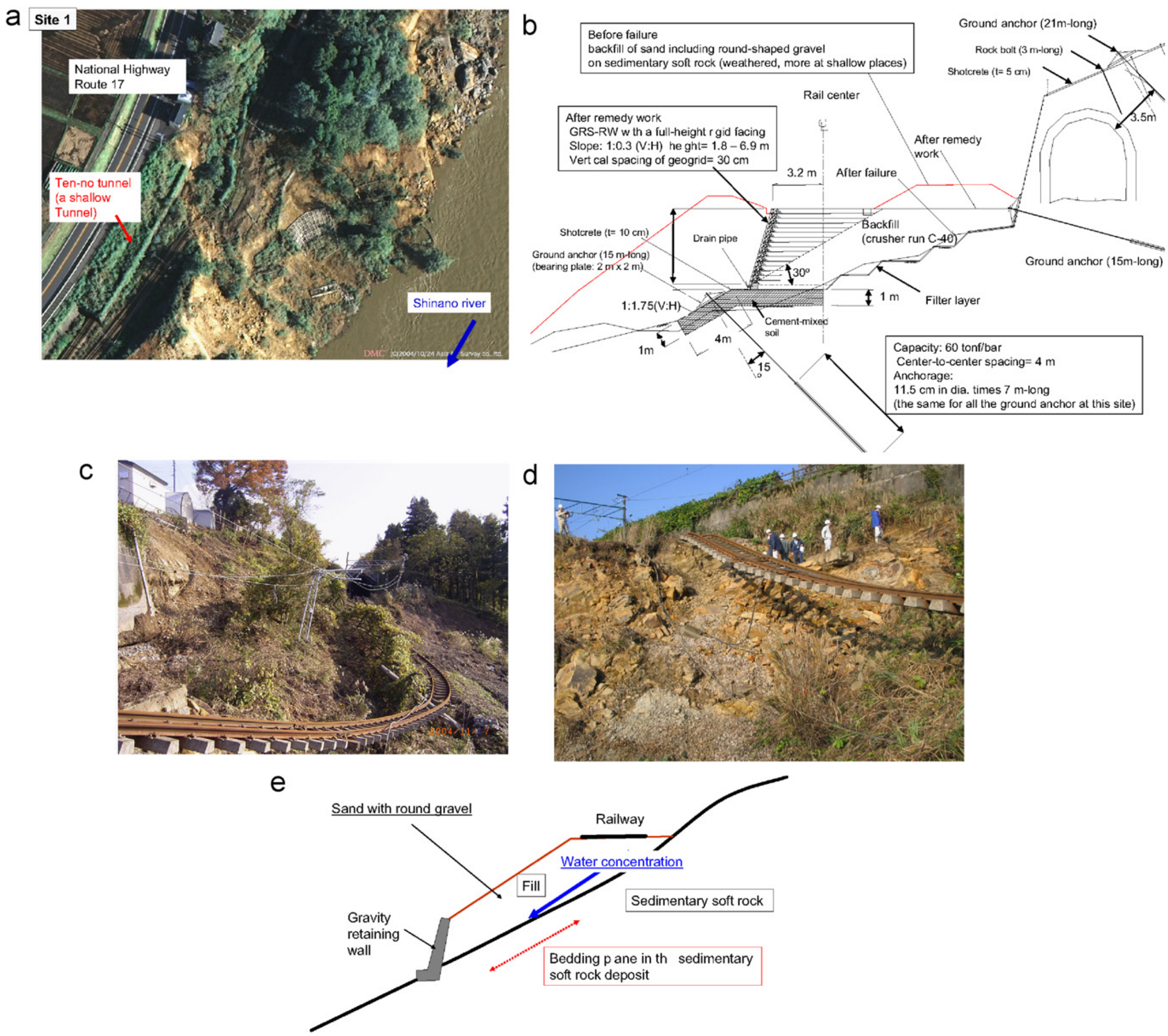

Fig. 19. Failure of a railway embankment at site 1: (a) air view (by the courtesy of Asia Air Survey Co. Ltd.); (b) cross section of the embankment after failure and reconstruction (Morishima et al., 2005); and (c) and (d) close views of the failure; and (e) failure of a gravity-type soil retaining wall at the embankment toe.

masonry, and cantilever RC) that failed during the Kobe earthquake were reconstructed to GRS-RWs having a FHR facing within a short period of time (Tatsuoka et al., 1996, 1997a, b, 1998).

(2) In this case, it was estimated that this type of GRS-RW is much superior to other types of structures (i.e., conventional gently sloped embankments, RC cantilever walls supported with a pile foundation, bridges, etc.) in terms of: (a) construction cost; (b) construction period; and (c) high performance in terms of deformation and ultimate stability. Reconstruction to GRS-RWs with a FHR facing is much more cost-effective due to factors (a) & (b) when compared to reconstruction of bridges. A significant reduction of earthwork is the important factor when compared to reconstruction to the original embankment.

This case history also validates that the GRS-RW with a FHR facing (Fig. 1) can be a very competitive construction method for critical wall structures, such as those for railways and highways.

5. Rehabilitation works of old earth-fill dams

5.1. Earth-fill dam for water supply in Tokyo

The earth-fill dam for Murayama Lower Reservoir (called Tama Lake, Fig. 25a) for water supply to the metropolitan, located in the north of Tokyo, was constructed about 80 years ago (completed 1927). When constructed, the earth dam, having a crest length of 587 m and a height of 33.6 m, was the largest one in Japan. The reservoir is exclusively for water supply in Tokyo, which

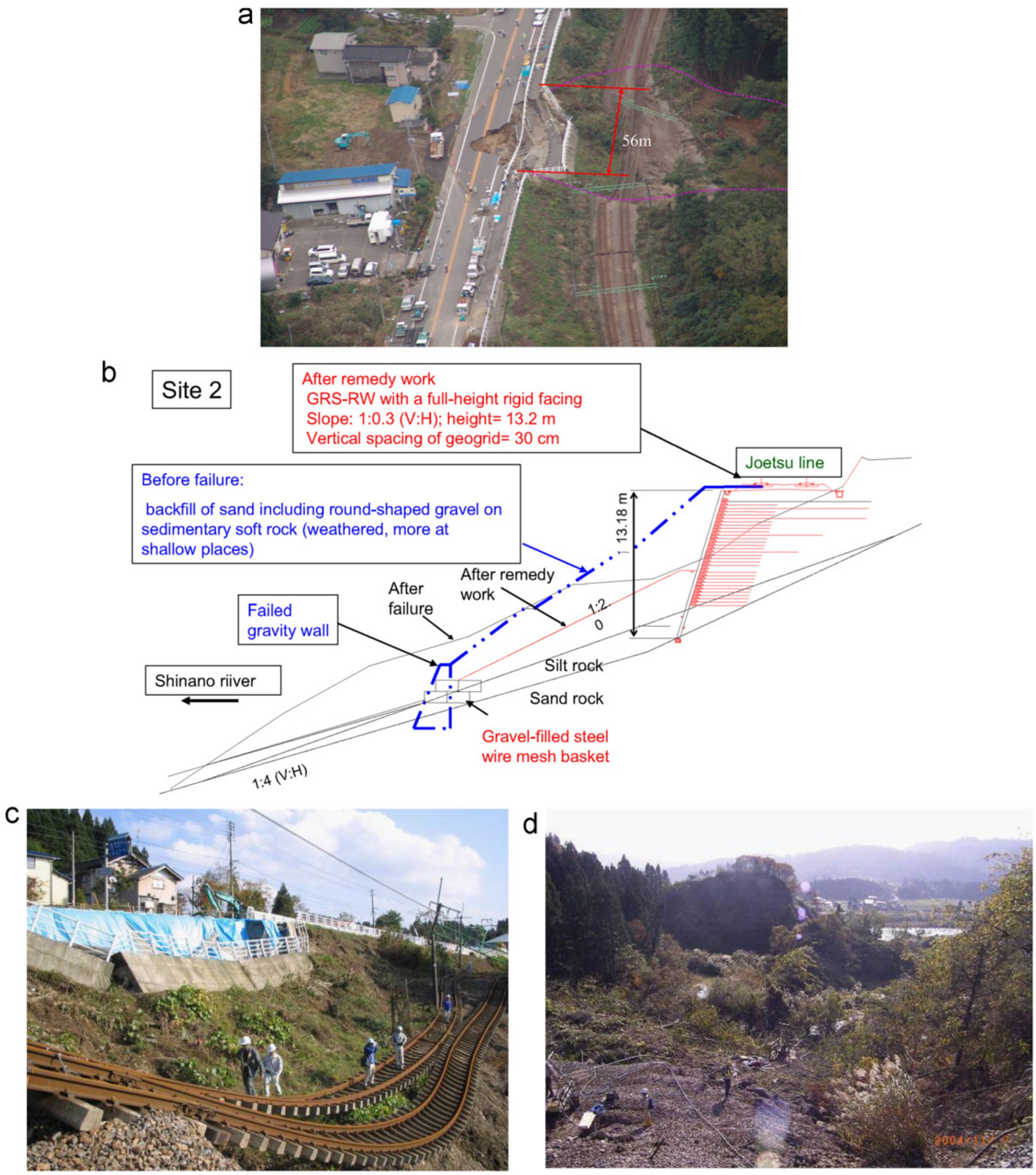

Fig. 20. Failure of highway and railway embankments at site 2 (Ten-no): (a) air-view (by the courtesy of Ministry of Land, Infrastructure & Transport); (b) cross section of the railway embankment after failure and reconstruction; and (c) and (d) close views.

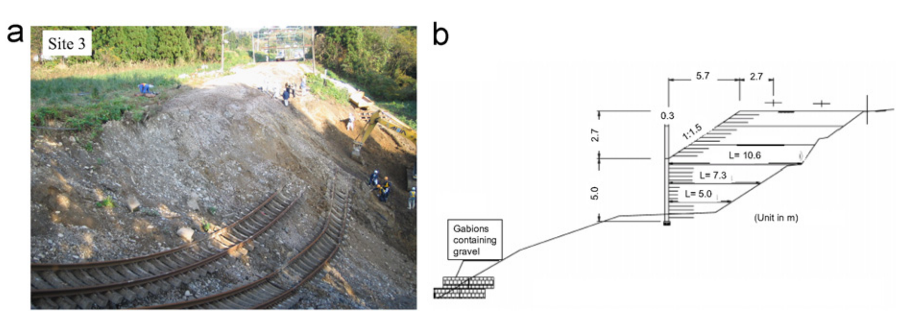

Fig. 21. Failure of railway embankment at site 3: (a) (left) close view; (b) (right) cross section of the embankment after failure and reconstruction (Morishima et al., 2005).

Fig. 22. Reconstruction to a GRS-RW with a full-height rigid facing, site 1 (cf. Fig. 20b).

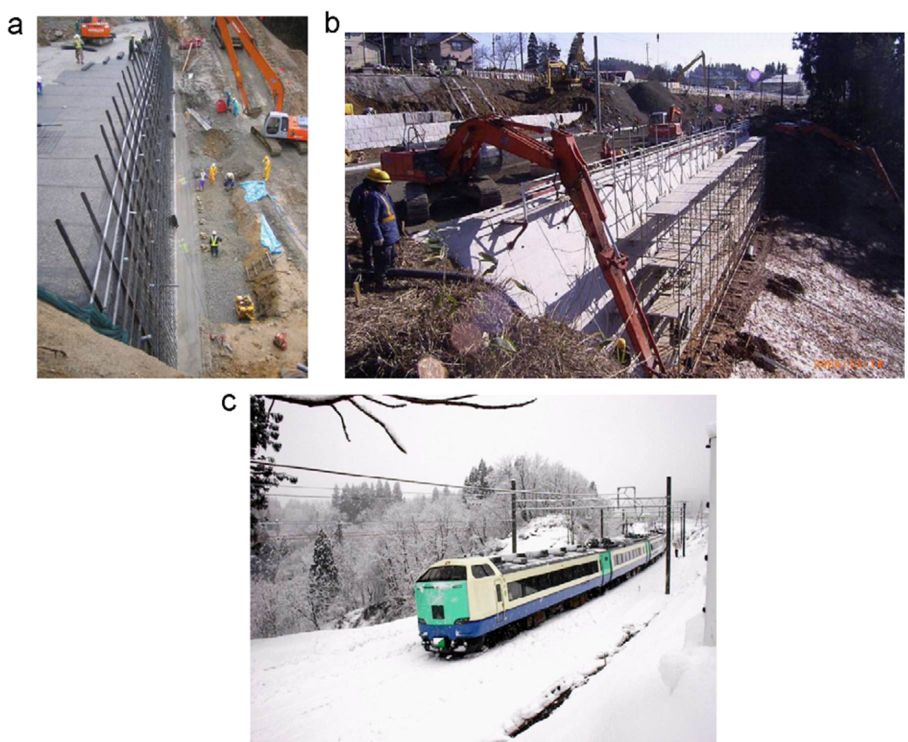

Fig. 23. Reconstruction to a GRS-RW with a full-height rigid facing, site 2 (cf. Fig. 19b).

will become extremely important in supplying water at the time of disaster, like seismic ones, because of its ability of sending raw water in gravity flow to several water purification plants on the downstream. The dam is now being rehabilitated by stabilizing the down-stream slope aiming at a substantial increase in the seismic stability to remove a possibility of vast disaster to a heavily populated residential area that had been developed recent years in front of the dam. As illustrated in Figs. 25c and d, to this end, a 17 m high counter-weight fill having a 1:1 steep slope was constructed due to a severe space restriction. To ensure a high seismic stability, the slope is reinforced with

Fig. 24. Reconstruction to a GRS-RW with a full-height rigid facing, site 3 (cf. Fig. 21b).

geosynthetic reinforcement layers. A HDPE geogrid was used. The total area of the geogrid layers stalled in the fill is 28,500 m^2.

5.2. A new technology to stabilize the down stream slope of earth-fill dam by placing soil bags allowing overflow in emergency

Ever year a great number of agricultural irrigation earth dams are seriously damaged or totally failed by overflow in events of flooding that exceeds the drainage capacity of a flood discharge system. It is not cost-effective to increase the drainage capacity of a flood discharge system of reinforced concrete (RC) structure so that it can discharge the design flood that might take place once every 100 years. Mohri et al. (2005) proposed to protect the downstream slope of such earth-fill dams by protecting using soil bags that are anchored with geosynthetic reinforcement layers arranged inside the slope (Fig. 26). This is a realistic cost-effective method to rehabilitate a great number of old earth-fill dams without increasing the capacity of a flood discharge RC structure. This is because this new method requires a much shorter construction period when compared with the one necessary to increase the capacity of an existing flood discharge system of RC structure. Moreover, the slope constructed or reconstructed by the new technology becomes more stable against seismic load.

The protected downstream slope should have a sufficiently high resistance against flood overflow. To validate the above, a series of model tests were performed. Six small models having different arrangements, listed in Table 1, were prepared. The model consisted of a 20 cm thick base ground, on which a 50 cm high model earth-fill dam with a slope of 1:2 (V:H) assuming a scale of $\frac{1}{10}$ of an assumed prototype was constructed (Fig. 27a). The fill material was Hokota sand (specific gravity $G_s = 2.676$, mean diameter

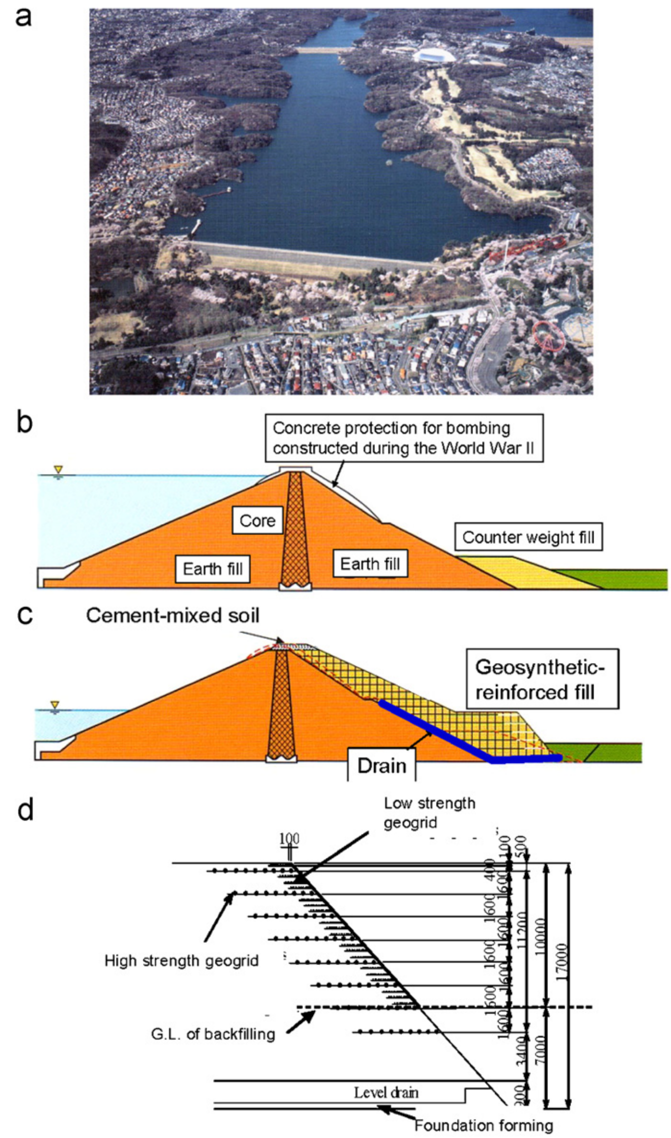

Fig. 25. Shimo-Murayama dam: (a) Tama lake and the dam; (b) and (c) dams before and after the rehabilitation work; and (d) geogrid-reinforced counter-weight embankment (Maruyama et al., 2006 [2]).

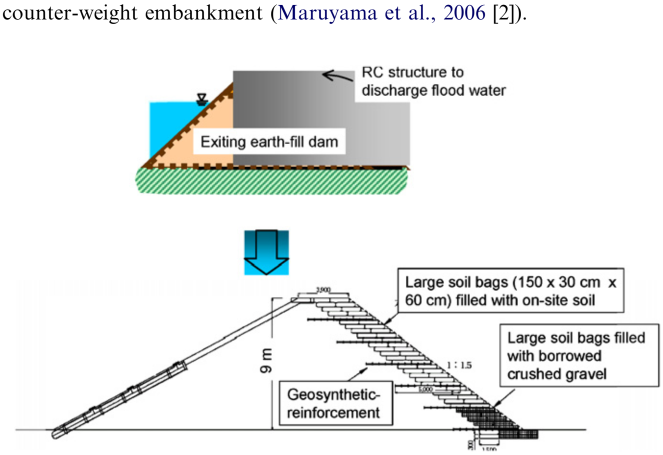

Fig. 26. A new technology to rehabilitate existing old earth-fill dams to have a high flood discharge capacity and a high seismic stability (when applied to a 9 m high typical earth-fill dam; Mohri et al., 2005; Matsushima et al., 2005).

Table 1
Model test cases (Matsushima et al., 2005)

	Slope protection	Soil bag type and dimensions (thickness × width × length) (cm)	Over-lapping (cm)
Case 1	Covered with non-woven geotextile sheets	—	—
Case 2	Covered with soil bags of type A without over-lapping	A: 2 × 5 × 6	—
Case 3	Covered with soil bags of type A with over-lapping	A: 2 × 5 × 6	2
Case 4	Covered with soil bags of type B with over-lapping	B: 2.5 × 8 × 8	3
Case 5	Covered with soil bags of type C with over-lapping	C: 2 × 8 × 12	8
Case 6	Covered with soil bags of type D with over-lapping	D: 2 × 5 × 18	14

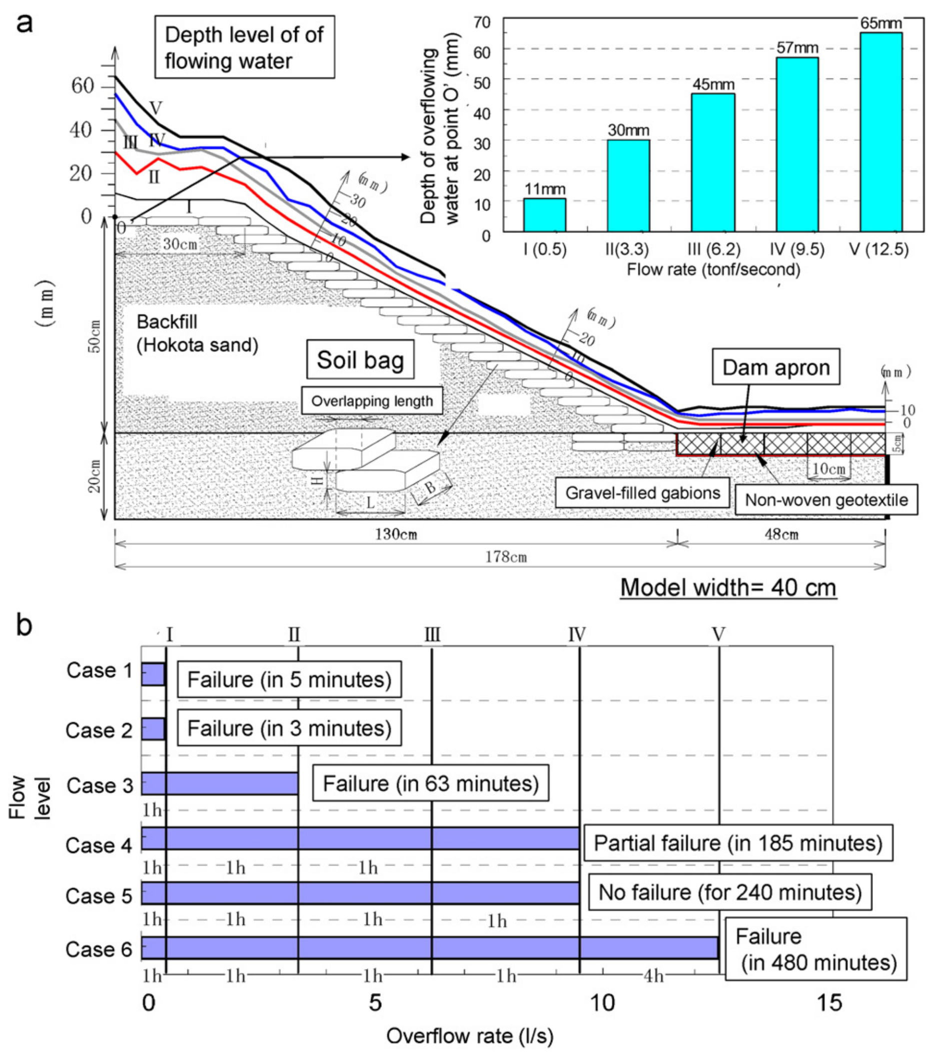

Fig. 27. Model tests to validate the new technology described in Fig. 26 (Matsushima et al., 2005); (a) structure of the model; and (b) summary of the test results.

D_{50} = 0.184 cm; coefficient of uniformity U_c = 5.82; maximum dry density $\rho_{dmax} = 1.517\,g/cm^3$ and optimum water content w_{opt} = 14.3%), compacted at water content w = 11.5% to a relative density D_r equal to 85% (not very dense). The soil bags were made of polyester. With the respective model, the water flow was continued increasing the flow rate every 1 h as shown in Fig. 27b.

The test results (Fig. 27b) showed that the stability against overflow increased by protecting the slope by using soil bags with sufficient overlapping between the vertically adjacent soil bags. That is:

(1) *Case* 1: The slope soil was eroded gradually with time by flowing water that passed below the non-woven

geotextile sheets, finally resulting into a sudden shallow failure of the slope.

(2) *Case* 2: The slope was protected with soil bags with no overlapping between vertically adjacent soil bags. Sliding at the interface between the slope and the soil bags was induced by flow water that reached the interface when the flow rate was still low. The sliding described above resulted in a sudden slide down of all the soil bags.

(3) *Cases* 3, 4 *and* 5: By using longer soil bags with larger overlapping between the vertically adjacent bags, the slope became more stable with a less amount of erosion of slope soil.

6. Conclusions

Some experiences from serious damage and failure of conventional-type embankments and soil retaining walls during recent several severe earthquakes and heavy rainfalls in Japan and their reconstruction to geosynthetic-reinforced soil structures (i.e., embankments with steepened slopes reinforced with geosynthetic reinforcement and geosynthetic-reinforced soil retaining walls) are described. In these case histories for railways, highways and earth dams, the geosynthetic-reinforcing technology for soil structures that is described in this paper was adopted based on judgements that this technology is highly cost-effective and efficient not only in constructing new soil structures, but also in rehabilitating old soil structures and reconstructing damaged and failed soil structures. Finally, new methods to rehabilitate old earth-fill dams, which were either actually used or is being investigated, are introduced.

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