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Lessons from the Failure of Full-Scale Models and Recent Geosynthetic-Reinforced Soil Retaining Walls

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ABSTRACT

Geosynthetic-reinforced soil retaining walls having a full-height rigid have been constructed for a total wall length of about 35 km as permanent important railway and highway soil retaining structures in Japan. The walls are first constructed by placing gabions filled with crushed gravel on the shoulder of each soil layer. A full-height rigid facing is cast-in-place on a wrapped-around wall that has been constructed to its full-height. The construction procedure and design methodology has been developed based on several lessons learned from the failure of a number of full-scale geosynthetic-reinforced soil retaining wall models. The backfill soil was a volcanic ash clay with a water content of about 100 % for four embankments, a highly weathered tuff for one embankment and a cohesionless soil including some amount of fines for the other. Failure was caused either by allowing natural rain percolating through the backfill for a long duration, by providing a large amount of water from the crest, by loading vertically with a footing placed on the crest of a wall, or by leaving unreinforced or wrapped-around flat wall faces under unsupported conditions. It was learned that a good drainage function is essential for geosynthetic reinforcement to be used for a nearly saturated clay backfill, while facing rigidity contributes significantly in several different ways to the wall stability for any type of backfill soil.

INTRODUCTION

The type of elevated railway and highway structures in Japan has gradually shifted from embankment with gentle slopes toward backfill with vertical or nearly vertical retaining walls, and then toward steel-reinforced concrete frame structures (**Figure 1a**). This shifting is due largely to several drawbacks inherent to embankments with gentle slopes, including the followings:

1) the slope of embankment is usually difficult to be well compacted, and therefore it may become unstable during a heavy rainfall and an earthquake, exhibiting too large deformation; and

2) embankment with gentle slopes occupies a too large base area, and therefore the cost-effectiveness of its construction in urban areas may become too low.

On the other hand, when the space below an elevated railway or highway structure is not utilised for other purposes and if the following conditions are satisfied, soil structures having a vertical or nearly vertical wall face could be more cost-effective than RC frame structures:

a) the soil structure is stable, rigid and durable enough when compared with equivalent RC frame structures; and

b) the construction cost of soil structure is markedly lower than that of equivalent RC frame structures.

The factor b) is relevant particularly when a RC frame structure should be supported by a long pile foundation while it is not the case with the soil structure, and/or when on-site soil is available for the backfill.

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Despite that backfill supported by conventional type cantilever RC retaining walls (**Figure 1b**) may satisfy the above condition a), this type of retaining structure should usually be supported by a pile foundation for the following reasons and therefore its construction may not be cost-effective:

1) Backfill having stable vertical wall face cannot be constructed without using a massive propping, of which the use is however not cost-effective. Therefore, the backfill should be placed on the back face of a RC retaining wall that has been constructed in advance.
2) Large settlement and lateral flow of the supporting ground may take place due to the backfill weight. As RC retaining wall structures are rigid, if it is not supported by a deep foundation, the allowable deformation of supporting subsoil is usually very small. Therefore, it is usual that the wall is supported by a pile foundation. In particular, when a RC retaining wall is constructed on a relatively thick soft subsoil layer, the total amount of necessary piles may increase due to large negative friction and bending force that may be exerted to the piles. In addition, the sliding force at the base of retaining wall by the lateral earth pressure from the backfill soil is approximately in proportional to the square of wall height H and the overturning moment about the toe is approximately in proportional to H^3. So, the construction cost of conventional cantilever retaining wall increases at an increasing rate with the wall height, particularly when the walls become higher than, say, 5 m.

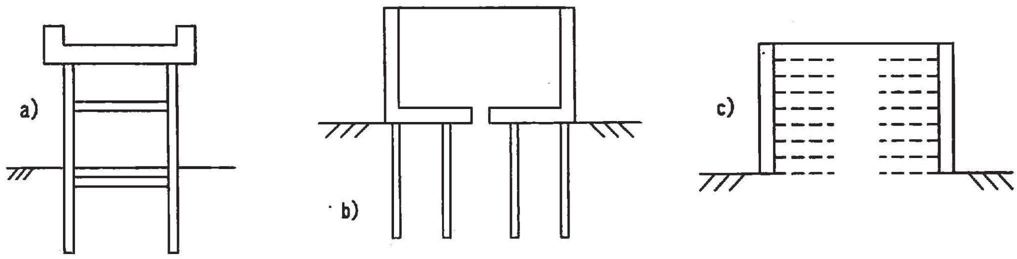


Figure 1. a) Elevated RC framework structure; b) backfill supported by conventional type cantilever RC retaining walls; and c) GRS-RW with a full-height rigid facing.

Geosynthetic-reinforced soil retaining walls with a full-height rigid facing

On the other hand, it has been shown that geosynthetic-reinforced soil retaining walls (GRS-RWs) having a full-height rigid facing can be more cost-effective, satisfying the conditions a) and b) above. **Figure 2** shows the standard staged-construction procedure for GRS-RWs, which is described below;

1) a small foundation pad for full-height rigid facing is first constructed;
2) a full-height wall is constructed with a help of gravel-filled bags placed at the shoulder of each soil layer, which is wrapped-around with a reinforcement geosynthetic sheet; and
3) after having confirmed that most of possible deformation of the backfill and supporting subsoil layers has taken place, a thin lightly steel-reinforced concrete facing is cast-in-place on the wall face in such a way as that it is firmly connected to the reinforcement layers and the wall face of backfill.

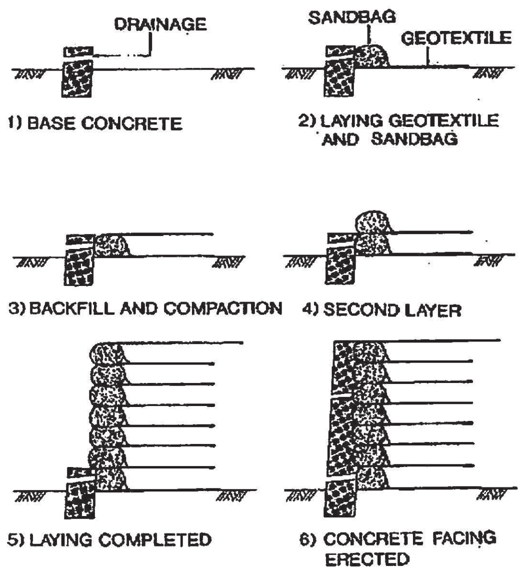


Figure 2. Standard staged-construction procedure for GRS-RWs with a full-height rigid facing.

The main features of the GRS-RW system can be summarised as follows (Tatsuoka 1993, Tatsuoka et al., 1994, 1997a):

1) Before casting-in-place a full-height rigid facing, GRS-RWs are essentially flexible during the filling-up and compaction of backfill and therefore, free from several problems arising from the interaction between a rigid facing and deformable backfill. Therefore, the walls can accommodate relatively large deformation of supporting ground without serious problems.
2) Actually, all the GRS-RWs that have been constructed so far (**Figure 4**) are not supported with a pile foundation. In case the supporting ground is too soft and weak, the surface soil layer is usually improved by in-situ cement mixing. After casting-in-place a full-height rigid facing, the GRS-RWs becomes stable, rigid, durable and aesthetically acceptable enough and could be equivalent to corresponding RC cantilever soil retaining structures. In particular when concentrated load is to be applied on the top of facing or the crest of wall, the use of full-height rigid facing can increase the wall stability and decrease the wall deformation.
3) By using a full-height rigid facing, a GRS-RW can become stable and rigid enough even with relatively short reinforcement. This feature becomes advantageous particularly when reconstructing a gentle slope of existing embankment to a vertical wall (**Figure 3**) by reducing the amount of soil to be excavated and to be filled, in addition to a reduction in the number of construction stage.
4) Inferior on-site soil such as sand including a large amount of fines is not allowed to use as the backfill when conventional steel strip-reinforcement is used (e.g., Terre Armee retaining walls). On the other hand, a polymer grid is much better in reinforcing such a type of soil. Even a nearly saturated clay can be used as the backfill soil for a GRS-RW by using a composite geosynthetic, which has a drainage function of a non-woven geotextile component and a large tensile stiffness by a woven geotextile component.

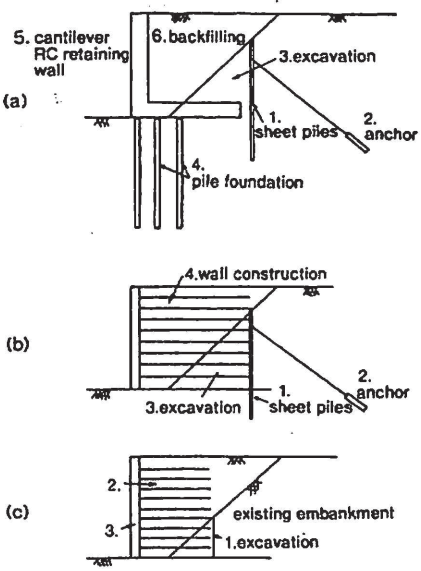


Figure 3. Comparison of; a) conventional RC cantilever RW; b) conventional steel-reinforced soil RW having long reinforcement; and c) GRS-RW having short reinforcement, for reconstructing an existing slope; the numbers mean the construction sequences.

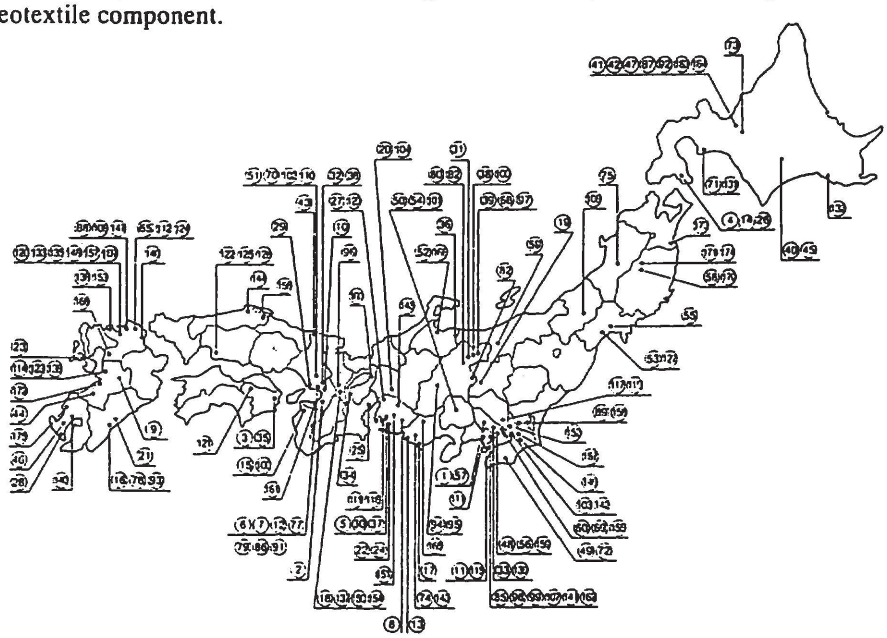


Figure 4. GRS-RWs with a full-height rigid facing constructed by the staged construction procedure shown in Figure 2 (as of the year 2000); the numbers mean the chronological order.

A large length of elevated RC frame structures for railways and highways was seriously damaged or totally collapsed during the Hyogo-ken-Nambu Earthquake of January 17, 1995. On the other hand, all the GRS-RWs with a full-height rigid facing that had been constructed for a length of about 2 km in the affected areas performed very well (Tateyama et al., 1995, Tatsuoka et al., 1995, 1997b). Based on the experience mentioned above, a number of conventional type retaining walls, mostly gravity type, that were seriously damaged during the earthquake were re-constructed to this type of GRS-RW. This fact is another factor that has enhanced the trend of "returning" to soil structures (not to conventional types but to the new type, GRS-RW having a full-height rigid facing constructed as shown in Figure 2). The mechanism of high seismic stability of this type of GRS-RWs was discussed in detail by Tatsuoka et al. (1998). Very recently, the importance of a high facing rigidity and a firm connection between the facing and the reinforcement layers was proven again by the failure of a couple of modular block geogrid-reinforced walls in Taiwan during the 1999 ChiChi Earthquake (Huang, 2000).

This type of GRS-RW has already been constructed to support railway and highway embankments for a length of about 35 km (Tatsuoka et al., 1991, 1997a, Emura et al., 1994, Doi et al., 1994, Kanazawa et al., 1994). **Figure 4** shows the locations of the main projects. It is very important to note that there has not been any problematic case with this type of GRS-RW, and the staged construction procedure of GRS-RW (Figure 2) is now the most popular wall construction procedure for railways, replacing the conventional wall construction procedures.

The study into this type of GRS-RW started early 1980's by, in parallel, full-scale model tests in the field and small-scale model tests in the laboratory. In the laboratory tests, small models were brought to failure by static loading tests using a footing (Tatsuoka et al., 1989) and by shaking table tests on large and small models (Murata et al., 1994). These model tests were then numerically analysed (Murata et al., 1992, Ling et al., 1995). However, this type of GRS-RW was adopted for the actual construction projects for important permanent civil engineering structures, such as railways and highways, only after this construction system was validated by a series of full-scale failure tests. In the tests, the full scale model walls were brought to failure by several different methods to get a better insight into the failure mechanism of reinforced soil retaining walls, as summarised below, which was essential for the development of cost-effective GRS-RW structure, appropriate construction procedure and rational design method.

FIELD TEST PROGRAM

The following three groups of full-scale test walls reinforced with geosynthetic reinforcement, according to the soil type for the backfill, were constructed since 1982.

Nearly saturated volcanic ash clay (Kanto loam by local naming) that was available on site

<u>1) Chiba No. 1 embankment</u> (**Figure 5** and **Table 1**; Tatsuoka et al., 1986, 1991, Tatsuoka and Yamauchi, 1986, Nakamura et al., 1988): This test embankment was constructed in 1982 at Chiba Experiment Station, Institute of Industrial Science, the University of Tokyo, to examine whether nearly vertical stable walls can be constructed by reinforcing a nearly saturated clay with a geotextile. Because of its drainage function, a non-woven geotextile (spun-bonded 100 % polypropylene) was selected as the reinforcement. This material is however not used at present as tensile reinforcement for

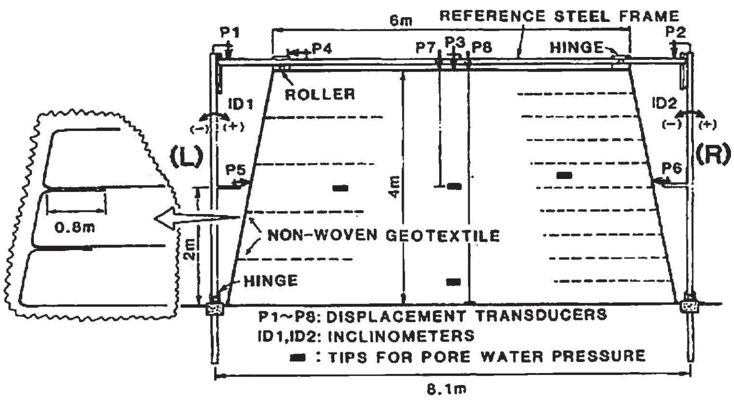


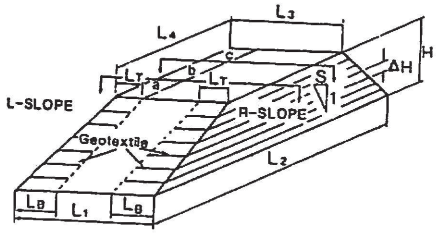
Figure 5. Initial cross-section of Chiba test embankment No. 1.

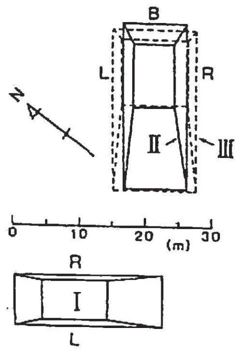
Embankment		No. 1	No. 2	No. 3	Kami-Onda
Properties of the backfill (Kanto loam)					
Water content(%)	at filling at demolishing	100 93[1]	120 104-119[1]	110 - [2]	116 123
Dry density (g/cm^3)	at filling at demolishing	- [2] 0.69[1]	0.55-0.65 0.60-0.67	0.60 - [2]	0.58 0.58
Degree of saturation	at filling at demolishing	- [2] 85%[1]	90% 78-85%[1]	85% - [2]	83% 87%
q_c [5] (kgf/cm^2)	at filling at demolishing	2-10 2-10	5-10[3] and 2- 7[4] 4-12[3]	5-10 - [2]	- [2] 2-4.5
Dimensions of embankment and arrangements of geotextile					
Height at filling (m)		4.0	5.2	5.5	3.0, 5.4[6]
Vertical spacing of geotextole sheets (m)		L[7]: 0.8 R[8]: 0.4	0.4	0.5	0.5
Length of geotextile, from the lowest layer to the top layer (m)		2.0 (constant)	L:2.24-3.8 R:1.24-2.8 B:1.0-3.4	2.5-5.0	2.0-3.0
Slope in horizontal : vertical		0.2 : 1.0	L and R: 0.3:1.0 B: 0.5:1.0	0.2 : 1.0	0.2 : 1.0
Facing structure:		Flexible[10]	Gabions[11]	L: G+S[12] R: Panels[13] B:Flexible	Lower wall : Panels Upper wall : Gabions

1) Averaged except for a slightly dried thin layer near the crest.
2) not measured.
3) For the mechanically compacted central places of embankment.
4) For the manually compacted places near the slope face.
5) Cone penetration resistances obtained with a cone having a cross-sectional area of 6.45 cm2 and an apex angle of 30 degrees.
6) The height of the lower wall and the height in total after the upper wall was constructed, respectively.
7) L: left-hand slope, 8) R: right-hand slope and
9) B: back-side slope.
10) The slope face of each soil layer was wrapped around with a geotextile sheet.
11) Gabions were placed at the shoulder of eash soil layer.
12) A shotcrete layer was placed on the slope face which had been constructed with the aid of gabions.
13) The facing stucture consisted of discrete concrete panels.

Table 1. Description of test embankments, Chiba Nos. 1, 2 and 3 and Kami-Onda

prototype GRS-RWs because of its low stiffness and low rupture strength. Appropriate composites consisting of non-woven/woven geotextiles having both a proper drainage function and high tensile stiffness and strength were not available at that time. Nearly vertical flat face of each soil layer was wrapped-around with each non-woven geotextile reinforcement sheet. The strength and deformation characteristics of undisturbed soil samples obtained from compacted soil layers and those of the non-woven geotextile were studied by, respectively, triaxial compression tests and tensile tests (Tatsuoka et al., 1986, Tatsuoka and Yamauchi, 1986, Yamauchi, 1986). The wall exhibited large deformation already during construction and also for a long period, particularly by heavy rainfalls, after construction, showing that a high deformability of such a wrapped-around wall having flat wall face is a real serious problem.





	Embankment I		Embankment II		Embankment III
Slope	R	L	R	L	R, L, B
L_B	2.0	2.0	2.8	3.8	2.5, 5.0
L_T	2.0	2.0	1.24	2.24	2.5, 5.0
L_1	7.6		10.0		12.2
L_2	22.0		23.4		21.15
L_3	8.0		6.48		10.0
L_4	10.0		12.0		10.55
H	4.0		5.2		5.5
ΔH	0.4	0.8	0.4	0.4	0.5
S	0.2		0.3		0.2
a	2.5		4.0		
b	5.0		4.0		
c	2.5		4.0		

UNIT: m

MASS PER UNIT AREA OF NON-WOVEN GEOTEXTILE

u (g/m²)	400*)	300*)	300*)

*: NOMINAL

Table 1 (continued). Dimensions and relative locations of Chiba Nos. 1, 2 and 3 embankments.

2) Chiba No. 2 embankment (**Figure 6** and Table 2; Tatsuoka et al., 1986, 1991, Tatsuoka and Yamauchi, 1986, Yamauchi et al., 1987, Nakamura et al., 1988): This test embankment, which was slightly larger than Chiba No. 1 embankment, was constructed in 1984, using the same types of backfill soil and reinforcement as Chiba No. 1 embankment, to confirm the lessons obtained from the test described above and also to investigate the effects of gabions placed at the wall face on the wall stability. That is, gabions filled with Kanto loam were placed at the shoulder of each soil layer to confine backfill soil behind gabions during compaction and also during and after wall construction. Despite its low stiffness of the non-woven geotextile reinforcement, the reinforced clay walls performed very well.

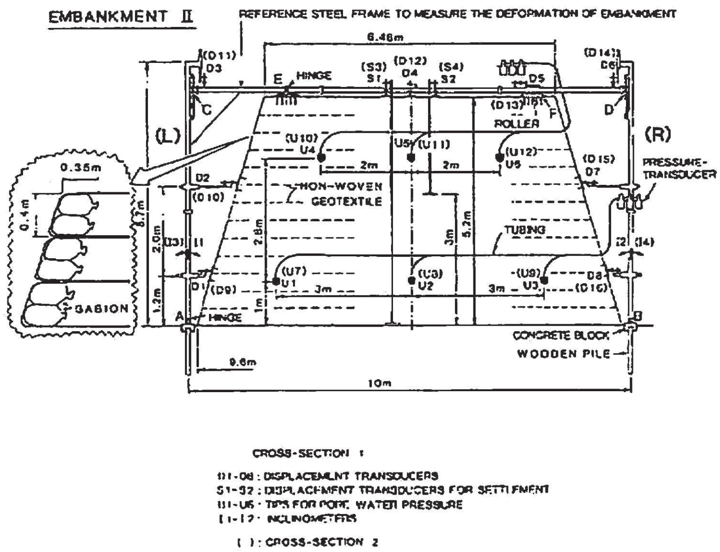


Figure 6. Initial cross-section of Chiba test embankment No. 2.

3) Kami-Onda embankment (**Figure 7** and Table 1; Tatsuoka et al., 1987, 1989, Nakamura et al., 1988): This was constructed in Yokohama City in 1985 to confirm the lessons obtained from the behaviour of Chiba Nos. 1 and 2 embankments and also to investigate into the effects of discrete precast concrete panels (**Figure 8a**) at the wall face on the wall stability. Using nearly the same types of backfill soil and reinforcement, Chiba Nos. 1 and 2 embankments were constructed on stiff intact Kanto loam ground, while Kami-Onda embankment was constructed on a 7 m-thick fill of very soft remoulded Kanto loam.

For this reason, Kami-Onda embankment settled down largely during and after construction.

4) Chiba No. 3 embankment (**Figure 9** and Table 1; Tatsuoka et al., 1987, 1989, Nakamura et al., 1988): This was constructed in 1986 to confirm the lessons obtained from the previous three tests by comparing the behaviours of three walls having different facing types; wrapped-around (without gabions), discrete concrete panels (**Figure 8b**), wrapped-around (with gabions) covered with a 8 cm-thick shotcrete layer. The behaviours of these three walls were very different according to very different facing rigidities.

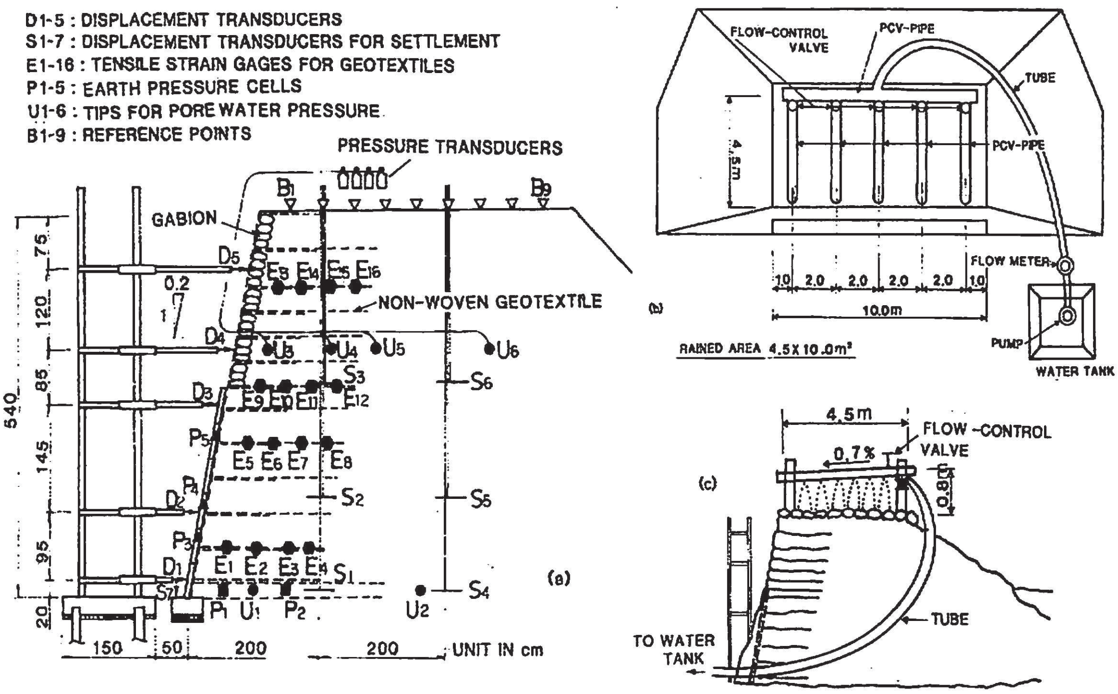


Figure 7. Kami-Onda test embankment; a) initial cross-section; b) plan; and c) configuraion of rainfall test.

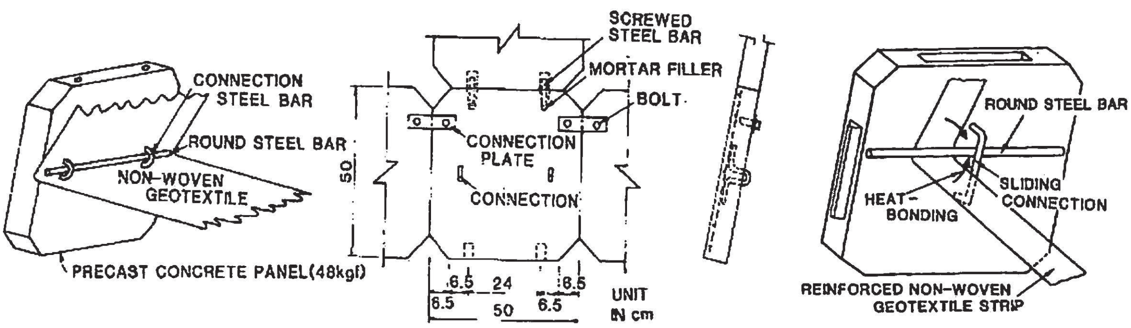


Figure 8. Precast concrete panel (50 cm x 50 cm x 5 cm thick and a weight of 34 kgf); a) the one for Kami-Onda embankment; and b) the one for the Chiba No. 3 embankment (the geotextile sheet or strip connected to each panel was sandwiched between non-woven geotextile sheets used to reinforce the backfill).

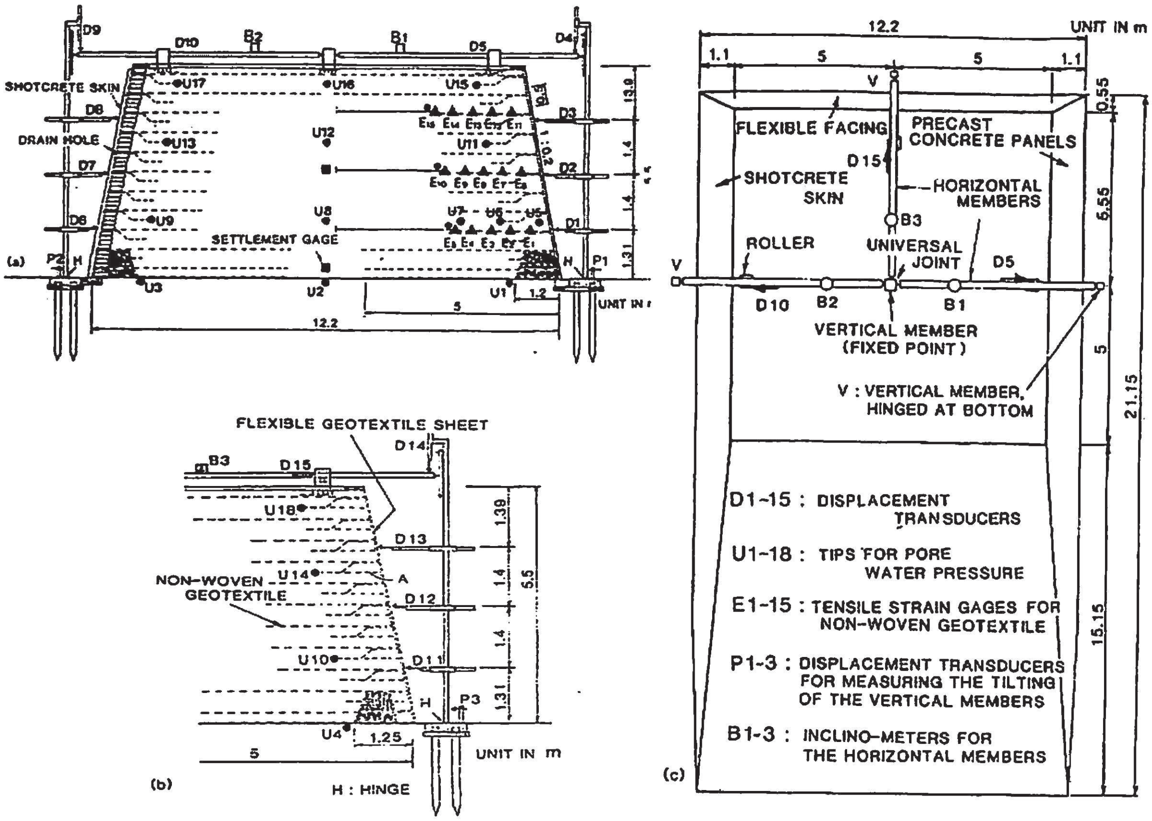


Figure 9. Initial cross-sections of; a) left-hand and right-hand walls; and b) backside wall; and c) plan of Chiba test embankment No. 3.

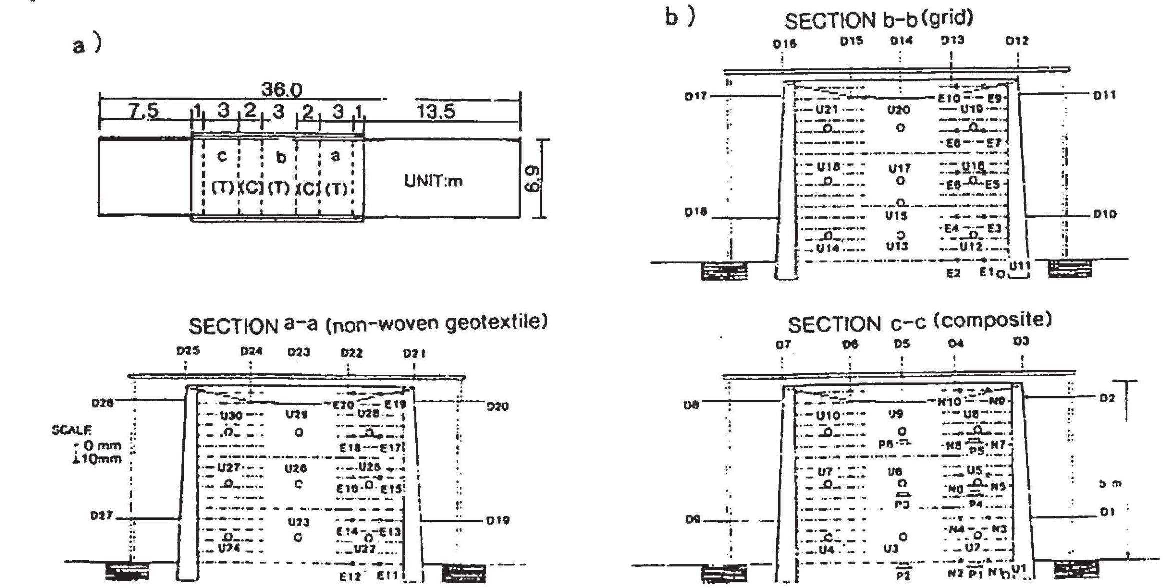


Figure 10. a) Plan; and b) initial cross-sections (gabions at the wall face are not shown), JR test embankment No. 2.

5) JR No. 2 embankment (**Figure 10**; Murata et al., 1991, 1992, Tatsuoka et al., 1992): This was constructed at Experiment Station of Japan Railway (JR) Technical Research Institute in the beginning of

1988 to examine the stability of proto-type full-scale GRS-RWs having a full-height rigid facing. The backfill soil was also Kanto loam with an initial water content of 120 - 130 %, an initial degree of saturation of about 90 % and an initial dry density of 0.55 - 0.60 g/cm^3. The results from plane strain compression tests on undisturbed samples retrieved from the compacted soil layers are described in Kato et al. (1991). Three types of reinforcement were used; a non-woven geotextile as used for Chiba Nos. 1, 2 and 3 and Kami-Onda embankments in section a-a; grid sheets with each being sandwiched between two gravel drainage layers in section b-b; and a composite consisting of non-woven/woven geotextiles in section c-c. The hydraulic and mechanical properties of these geosynthetics are described in detail in Ling et al. (1992, 1993, 1994, 1995). The three test sections, having in total six wall segments reinforced with different reinforcement types, exhibited similarly very well for a long duration, which reconfirmed that the effects of facing type could be much more important than the stiffness of reinforcement on the stability of reinforced soil retaining wall.

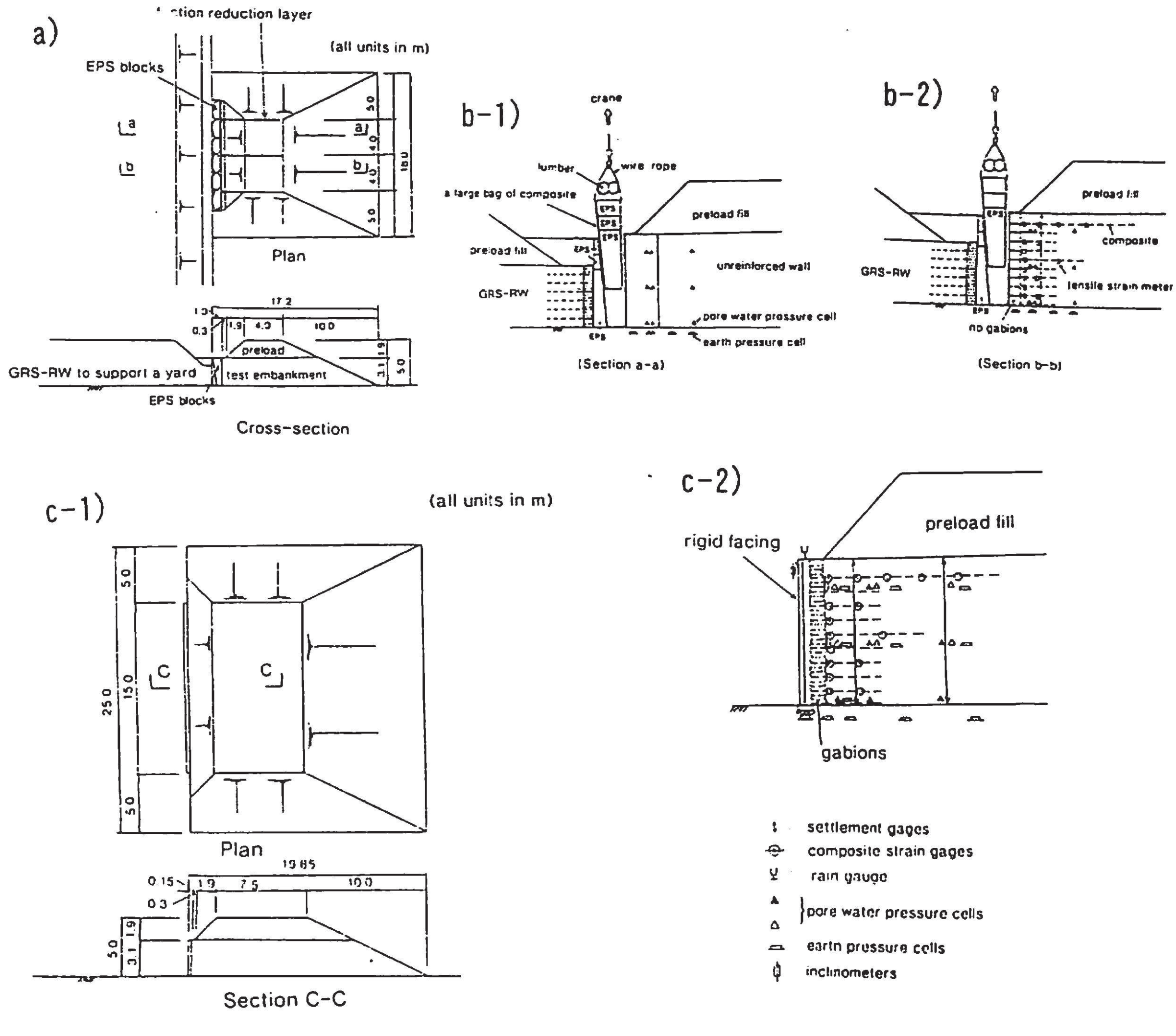


Figure 11. a) Overall dimensions of test embankment; b-1) unreinforced clay wall (section a-a in Figure a); b-2) wrapped-around clay wall (section b-b in Figure a); c-1) experimental part of the GRS-RW for Nagano Shinkansen (bullet train) yard; and c-2) experimental GRS-RW (section c-c in Figure c-1).

Nearly saturated highly weathered tuff

This type of soil, which was available on site, was used to construct the test embankment in 1994 (**Figure 11a**) to reconfirm the function of full-height rigid facing. The soil became a nearly saturated soft clay by compaction, having an average water content of about 30 % and a degree of saturation of 70 % (**Figure 12**). This field test was performed in conjunction of the construction for a period from 1993 to 1994 of prototype GRS-RWs with a complete height of 2 m having a total length of about 2 km (**Figure 13**). The backfill was the same type of soil as the test embankment and was reinforced with a non-woven/woven geotextile composite. The prototype GRS-RWs, which are supporting a yard for Shinkansen (bullet train) in the north of Nagano City, are the first actual clay walls for a railway structure in Japan. As the prototype walls were constructed on a thick very soft clay layer (Figure 13), a preload fill was placed on the embankment before casting-in-place a rigid facing on the wrapped-around wall face (Figure 2), which resulted in a large ground settlement of about 1 m. . The initial wall height as constructed was about 3 m to accommodate this expected large ground settlement. The walls were not supported by any pile foundation, which would have been necessary if conventional cantilever RC retaining walls were constructed. A full-height rigid facing was cast-in-place in the summer of 1995 after a preload fill had been removed, without any problem by relative settlement between the backfill and the rigid facing.

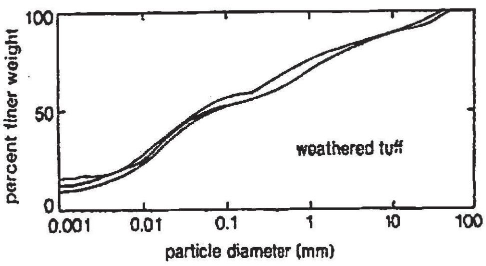


Figure 12. Typical grading curves of the backfill soil for the GRS-RWs supporting Nagano Shinkansen yard.

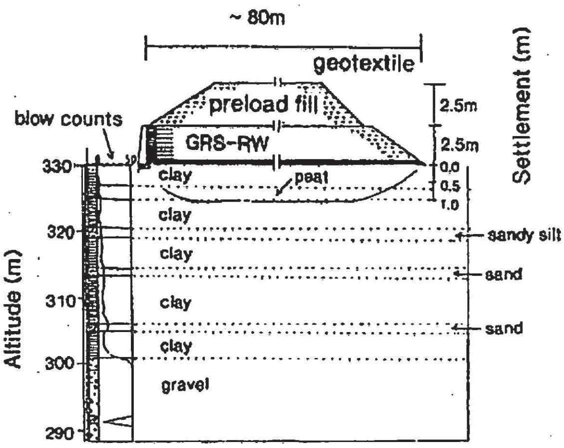


Figure 13. Typical cross-section of Nagano Shinkansen yard with a GRS-RW on one side (n.b., full-height full-height rigid facing was cast-in-place after the preload fill had been removed).

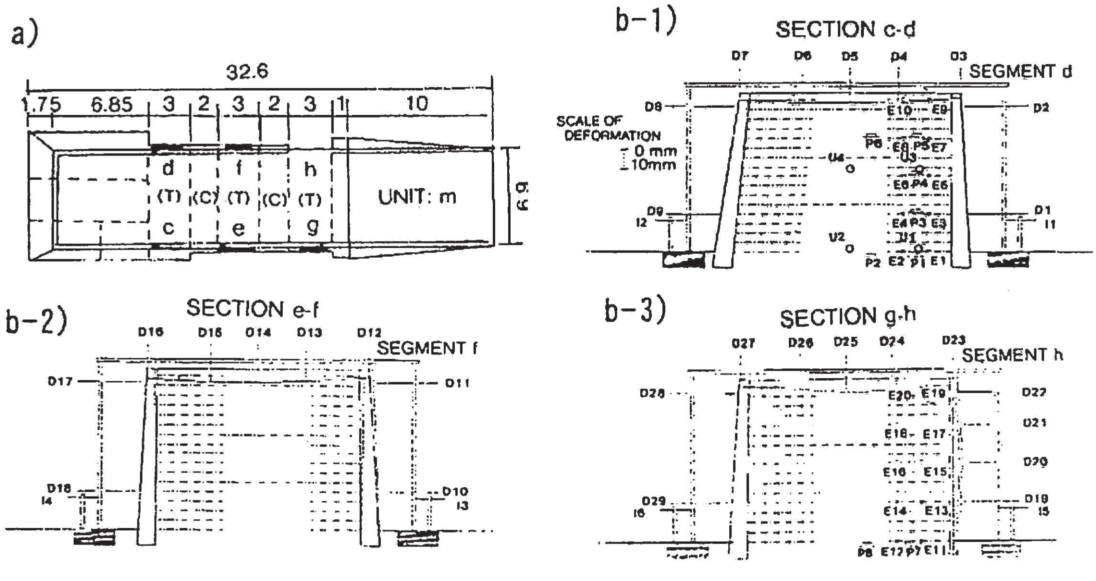


Figure 14. a) Plan; and b) initial cross-sections (gabions at the wall face are not shown) of JR test embankment No. 1.

The three test walls were; two walls of the test embankment (**Figure 11a**), consisting of an unreinforced wall without gabions (section a-a; Figure **11b-1**) and a wrappted-around wall reinforced with a composite geotextile (section b-b; **Figure 11b-2**); and a part of the prototype GRS-RW with a full-height rigid facing (**Figure 11c**). As the two test wall sections of the test embankment (Figures 11a) were faced to the prototype GRS-RW (Figure 11c) with a separation of about 1.0 m, the test embankment was constructed by using large pieces of EPS installed in the separation as a temporary support.

Cohesionless soil

JR No. 1 test embankment (**Figure 14**; Murata et al. , 1992, 1991, Tatsuoka et al., 1992) was constructed for a period from the end of 1987 to the beginning of 1988 to examine the stability of prototype GRW-RWs with sand backfill having a full-height rigid facing (Figure 2). For comparison purposes, one test wall segment (segment h) was constructed to have a discrete panel facing, as used for Chiba No. 3 embankment. Each panel was fixed to gabions filled with gravel that had been placed on the shoulder of each soil layer. The backfill soil was Inagi sand, with D_{50}= about 0.2 mm and a fines content= 16 %. Plane strain compression tests were performed on specimens prepared in the laboratory to have nearly the same averaged density (= 1.493 g/cm^3) and water content (15.3 %) as the backfill (Park and Tatsuoka, 1989). The reinforcement was a grid consisting of longitudinal and transversal members with a rectangular cross-section of 0.9 mm x 3 mm and an aperture of 20 mm. The tensile rapture strength was 2.8 tonf/m and a secant tensile modulus at an elongation of 5 % was 1.0 tonf/m, both measured at a strain rate of 5 %/min. The grid was polyester fibers coated with PVC protection. For most of the prototype GRS-RWs that were subsequently constructed (Figure 4), a grid of Vinylon fibers coated with PVC protection (Vinylon is the trade mark of polyvinyl alcohol) was used because of its high resistance against the effects of alkaline, which is important when used in contact with concrete (Tatsuoka et al., 1994).

TEST DESCRIPTIONS

The test walls were brought to failure by the following different methods.

Chiba Nos. 1, 2 and 3 test embankments

The following test wall sections were designed to be purposefully weak as follows:

1) The left-hand side wall of Chiba No. 1 embankment (denoted as L in Figure 6) had a vertical spacing of as large as 80 cm between reinforcement layers.

2) The right-hand side wall of Chiba No. 2 embankment (denoted as R in Figure 7) had very short reinforcement layers with a length of only 1.24 m at the top layer.

3) The backside wall of Chiba No. 3 (Figure 9b) had a flat wrapped-around wall face without using gabions despite a relatively large wall height of 5.5 m.

The long-term behaviour of these walls was continuously and automatically monitored, expecting that these walls would exhibit large deformation, particularly during natural heavy rainfalls (as they did).

Chiba No. 2 and Kami-Onda embankments

To bring Chiba No. 2 embankment to failure, for a period of eight days in October 1985, about 70 m^3 of water, equivalent to as much as of a 900 mm precipitation for the whole crest area, was supplied from a pond made on the crest (**Figure 15**). Similarly, for Kami-Onda embankment, for a period of four days in

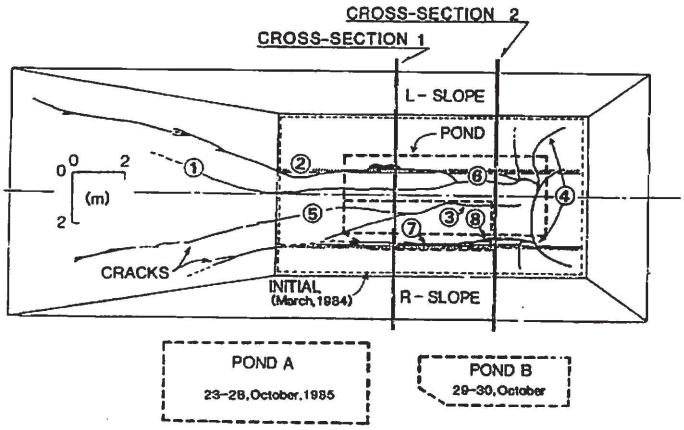


Figure 15. Plan of Chiba No. 2 embankment and cracks developed by water supply from the pond.

December 1985, an artificial rainfall, equivalent to as much as of a 620 mm precipitation, was supplied from many small holes made in four PVC pipes on the crest (Figures 7b and c).

JR Nos. 1 and 2 embankments

Any sign of instability of the walls was not observed for a long period of nearly two years after construction. From the end of 1989 to the beginning of 1990, three of the six test wall segments of JR No. 1 embankment and one of the six test wall segments of JR No. 2 embankment were brought to failure by loading vertically with a footing placed on the crest of the wall. For JR No. 1 embankment, a footing with a base area of 2 m times 3 m was placed nearly at the center of the crest, immediately behind the reinforced zone (**Figure 16**). For JR No. 2 embankment, the first loading test was similar to the one shown in Figure 16, which did not bring the wall to ultimate failure. The second loading test, is shown in **Figure 17**, in which a smaller footing with a base of 1 m times 3 m was used to bring the wall to failure.

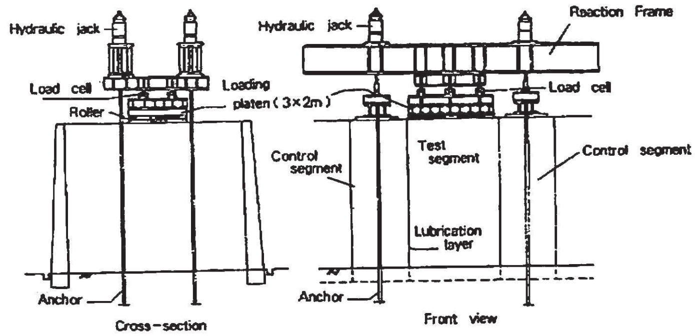


Figure 16. Loading method for JR No. 1 sand embankment (n.b., reinforcements are not shown for simplicity).

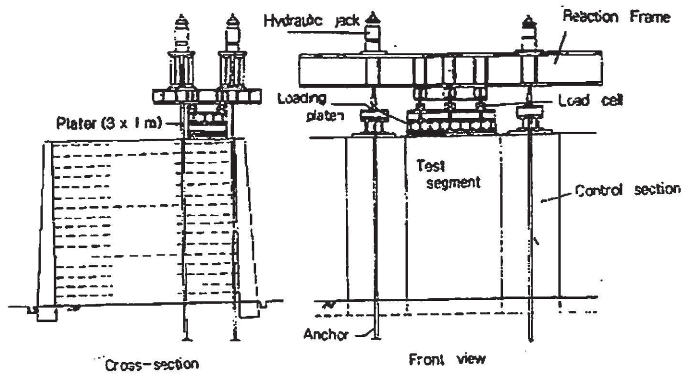


Figure 17. Front loading method for JR No. 2 clay embankment (n.b., the back loading method is the same with the one shown in Figure 16).

Nagano test embankments

Expecting that the unreinforced wall and the wrapped-around wall (without gabions) would exhibit large deformation under unsupported conditions, the temporary support of EPS pieces was removed for a very short duration to expose the wall faces (Figure 11b).

FAILURE AND LESSONS

Chiba No. 1 test embankment

Figure 18 shows two of the cross-sections that were exposed when the embankment was demolished in 1985 after a continuous observation of its behaviour for three years and four months (Tatsuoka et al., 1986, 1989, Tatsuoka and Yamauchi, 1986, Nakamura et al., 1988). **Figure 19** shows the deformation of the cross-section 2 for a) the first year; and b) the subsequent three years. The left-hand side wall, in which the vertical spacing between reinforcement layers was as large as 80 cm, deformed very largely mostly by a series of heavy rainfalls in the first year after construction (see Fig. 6 of Tatsuoka et al., 1986). The failure was triggered by a local compressive failure that occurred immediately behind the wrapped-around wall face in the bottom soil layer as follows (**Figure 20**):

a) Rain water percolated through the backfill and accumulated in the bottom soil layer, which led to a reduction in the soil suction (negative pore water pressure) and further an increase in the positive pore water pressure, making the soil very weak, despite a relatively large vertical pressure in the bottom soil layer.
b) The geotextile wrapping the nearly flat wall face could not restrain effectively the deformation of the soil immediately behind the wall face until the soil deformed largely and the deformed wall face became round sufficiently. It seems that the vertical spacing of 80 cm was too large for an effective

arch action to be activated in the soil layer to prevent such deformation as above to occur. It seems that the compressive failure, which first took place behind the wall face in the bottom soil layer, proceeded towards deeper places.

c) Consequently, the reinforced soil zone locating above the bottom soil layer settled down and displaced outward as a monolith, creating a shear zone between the reinforced zone and the unreinforced zone behind. The deformation in the right-hand wall was smaller, but it is too large for the wall to be used as an important permanent structure.

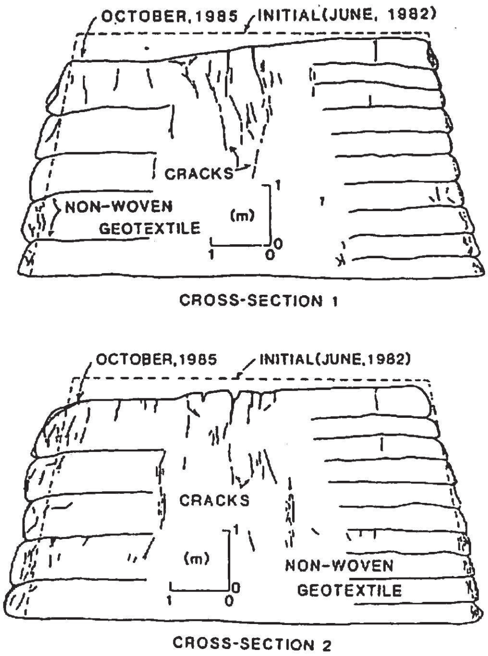


Figure 18. Two exposed cross-sections of Chiba No. 1 embankment.

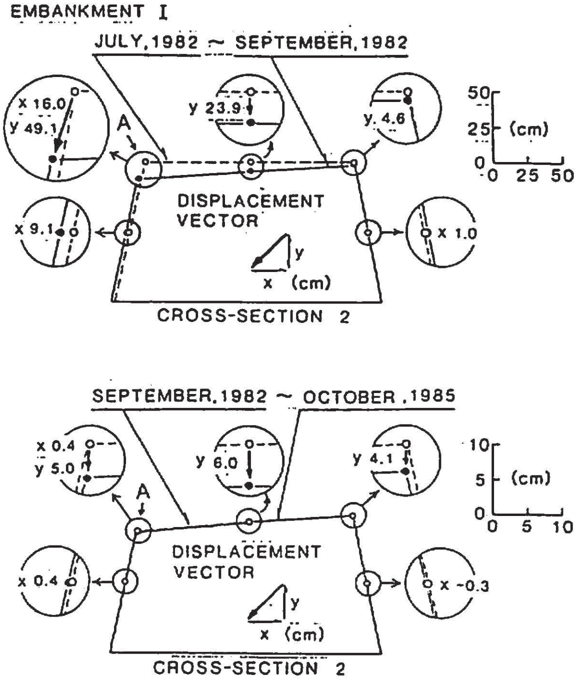


Figure 19. Deformation of Chiba No. 1 embankment about three years and four months after construction.

We can learn the following several lessons from this failure:

1) The fact that a local soil failure triggered the overall failure of wall shows that it could be more important for the wall stability to prevent a local failure in the backfill immediately behind the wall face than to attempt to restrain the lateral deformation of backfill soil by using very stiff reinforcement.
2) Flat wrapped-around wall face without using gabions cannot confine effectively the deformation of soil behind the wall face. This type of wall face cannot be recommended unless the structure is secondary and temporary. In case wrapped-around wall is to be constructed without gabions, the wall face of each soil layer should be constructed round.
3) For clay backfill, a vertical spacing of 80 cm between geotextile layers is too large;
   i) to effectively drain water from the inside of each soil layer and to maintain a high suction in the soil layer:- the importance of keeping a sufficiently high suction in the backfill soil for the stability of clay wall cannot be over-emphasised; and
   ii) to effectively confine the back-fill soil, in particular at and near the wall face, when the wall

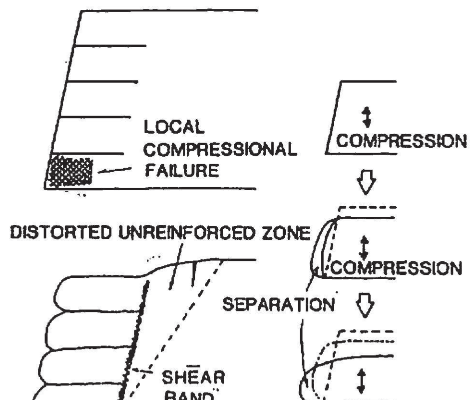


Figure 20. Schematic figures showing local compressive failure immediately behind the wall face in the bottom soil layer and associated overall failure of wall.

face is a wrapped-around one without gabions.

4) Although large deformation occurred in the lowest soil year in the first year after construction, the walls were very stable in the subsequent years (see Figure 19b). Further, the main body of the reinforced zone (excluding the largely deformed bottom soil layer) behaved like a monolith without including major failure planes inside, in particular in the right-hand wall (Figure 18). This result indicates that stable reinforced soil walls can be constructed using nearly saturated clay if the backfill is effectively reinforced with reinforcement having proper functions of drainage and tensile-reinforcing and a proper rigid facing is used.

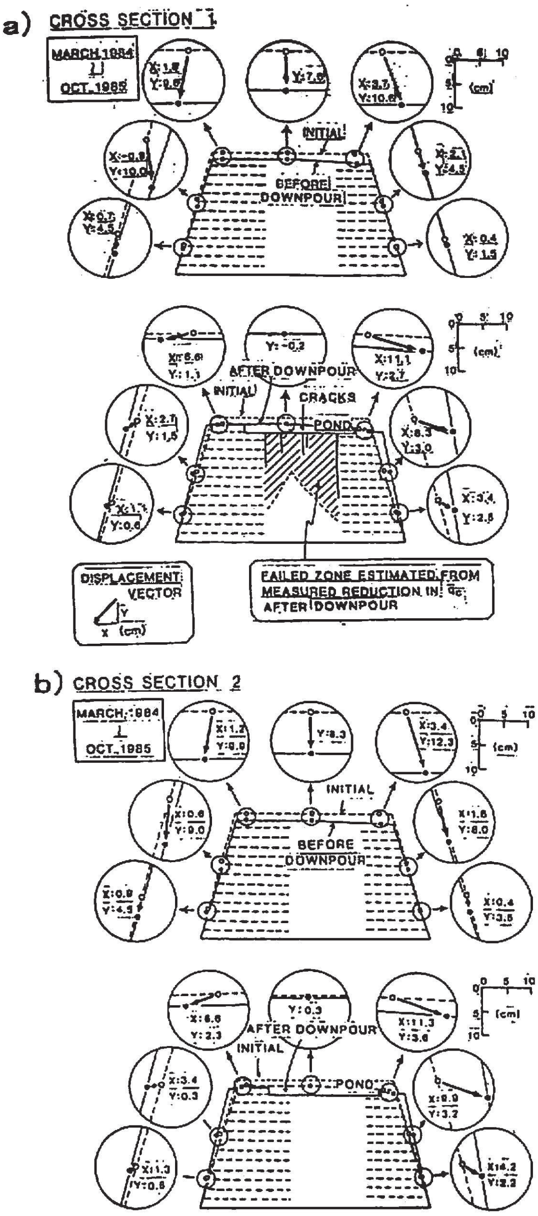


Figure 21. Accumulated deformation of Chiba No. 2 embankment for a period from the end of construction (May 1984) and an artificial heavy rainfall test (October 1985) and the deformation by the rainfall test: a) cross-section 1; and b) cross-section 2 (see Figure 15 for the locations of cross-sections 1 & 2).

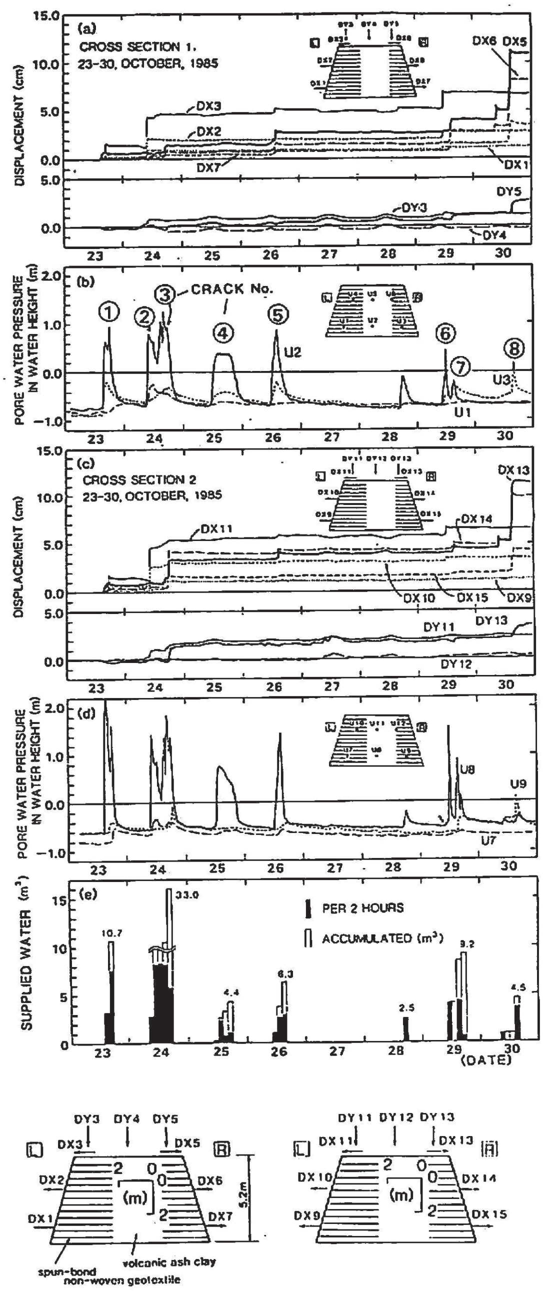


Figure 22. Time histories of lateral displacements at the wall face and pore water pressure in the backfill at cross-sections 1 & 2 and supplied water during the artificial heavy rainfall test, Chiba No. 2 embankment.

Chiba No. 2 test embankment

This embankment was demolished in October 1986 after a continuous observation of its behaviour for about two and a half years with an artificial heavy rainfall test at an intermediate stage (Tatsuoka and Yamauchi, 1986, Tatsuoka et al., 1987, Yamauchi et al., 1987). Both side walls deformed noticeably by rainfalls in the first year after construction (see the top figures of **Figures 21a** and **b**). The wall deformation could be well correlated to the decrease in the soil suction and the increase in the positive pore water pressure. It is to be noted that by using gabions, the wall deformation became much smaller than that of Chiba No. 1 embankment. In particular, outward inclination of wall face, which is a sign of wall instability, was practically a nil.

In the heavy rainfall test, water was supplied first from Pond A at the center on the crest (Figure 15), starting in 23rd October 1986 (**Figure 22**). The moments when the cracks, numberd 1~8 in Figure 15, appeared are indicated in Figure 22b. By 24th, two major cracks, Nos. 1 and 2, appeared on the crest of the central unreinforced zone immediately behind the left-hand reinforced zone, despite that the reinforcement was much longer in the left-hand wall. This was because Pond A was located above the back face of the left-hand reinforced zone, but not above the back face of the right-hand reinforced zone, while vertical cracks can develop most easily along the back face of reinforced zone. Therefore, the wall deformation was much larger in the left-hand wall than in the right-hand wall (see Fig.22). From the noon in 29th October, water was supplied from Pond B, which had been relocated closer to the right-hand wall face. As a consequence, other cracks, Nos. 7 and 8 in Figure 15, appeared on the crest along the back face of the right-hand reinforced zone, resulting into large deformation of the right-hand wall (see Figure 22). The bottom figures of Figures 21a and b show the total deformation caused by this artificial rainfall test. It may be seen that the total deformation was larger in the right-hand wall than in the left-hand wall, while noticeable outward inclination occurred in both the walls.

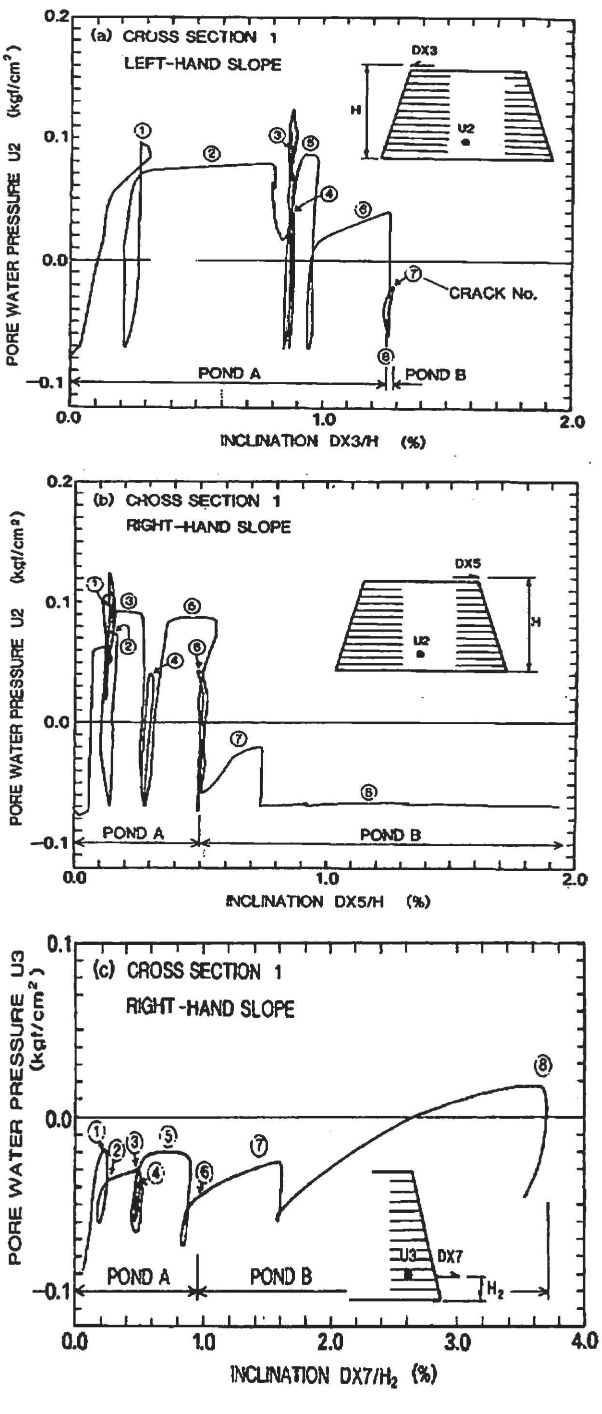


Figure 23. Relationships between wall face inclination and pore water pressure observed during the artificial rainfall test, Chiba No. 2 embankment.

The pore water pressure that was recorded in the reinforced zones during the rainfall test was much smaller than that in the central unreinforced zone (**Figure 22b and d**). This was mainly due to the fact that major cracks developed in the unreinforced zone, but not in the reinforced zones, and perhaps due partly to the drainage function of the non-woven geotextile. It is very likely that the pore water pressure controlled the wall deformation, as the wall deformation is well correlated to the variation in the pore water pressure (**Figure 23**). **Figure 24** shows one of the cross-sections that were exposed at demolishing. Several vertical tension cracks were observed only in the central unreinforced zone. Two shear zones appeared, starting from Points B and C at the crest and developing downwards. In the right-hand wall, the direction of the shear zone changed at Point D from the vertical direction to a more horizontal one towards the toe of the wall face (Point E).

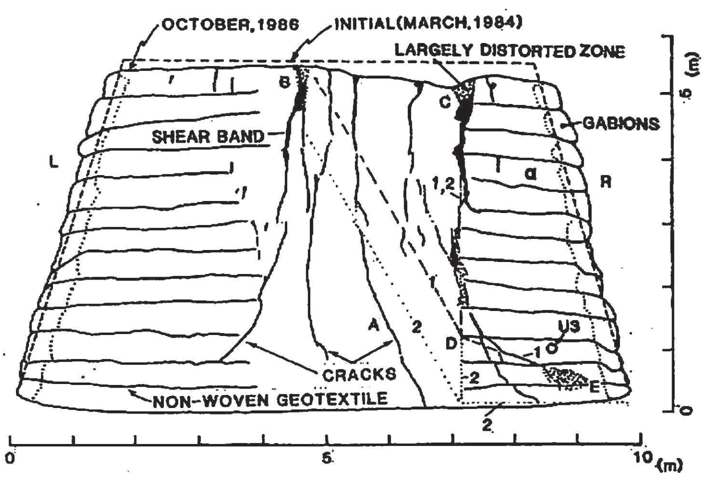


Figure 24. Cross-section of Chiba No. 2 embankment exposed at demolishing; lines numbered 1 & 2 are the critical failure surfaces obtained by the limit equilibrium stability analysis withut and with taking into account the positive pore pressure in cracks (Tatsuoka et al., 1987).

We learned the following several lessons from this failure case history:

1) The use of gabions at the shoulder of each soil layer is very effective not only for a good compaction of backfill, but also for confining the soil near the wall face, so for maintaining a high soil strength. Although gabions were filled with the backfill soil (i.e., Kanto loam) for this embankment, it was found later that gabions should be filled with gravel for a much better function while not losing cost-effectiveness. It was further considered that if the wall face were made more rigid than it was, the wall deformation had been smaller.

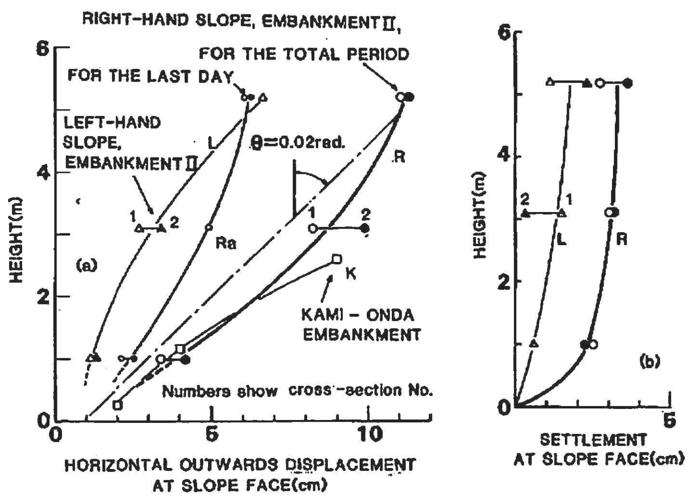


Figure 25. a) Horizontal outwards displacement; and b) settlement at the wall face, plotted against the height from the wall bottom, caused by the artificial rainfall test, Chiba No. 2 and Kami-Onda test embankments.

2) The continuous record of pore water pressure at several representative points inside the backfill reconfirmed that the major cause for the wall deformation was the decrease in the suction and the increase in the positive pore water pressure. The lines numbered as 1 and 2 in Figure 24 are the critical failure planes obtained by the limit equilibrium stability analysis, respectively, without and with taking into account the positive pore water pressure in the cracks. The result from the analysis indicated that the failure of the wall by the rainfall test can be explained much better by taking into account the effects of positive pore water pressure (Tatsuoka et al., 1987).

3) The relationships denoted as *R* and *L* in **Figure 25** show; a) horizontal outward displacements; and b) settlements at the right-hand and left-hand wall faces that occurred during the total period of rainfall test. The relationship denoted as *Ra* shows the wall displacements that took place only in the last day of the test (30th October 1985). Similar data obtained from the rainfall test on Kami-Onda embankment is also shown (denoted as *K*). It was found that the wall deformation consisted of those by the following three failure modes (**Figure 26**); a) rotation (or over-turning) of reinforced zone about the wall face toe; b) sliding out of reinforced zone along a failure plane; and c) local compressive

failure near the wall face in the several bottom soil layers. This result shows that safety factors for all these three failure modes should be examined in design. The failure mode a) was observed to a much less extent in the left-hand wall because of using longer (and sufficiently long) reinforcement. It is likely that the failure made c) would have been prevented if the wall face were more rigid.

4) Despite the use of so-called relatively extensible reinforcement (i.e., a non-woven geotextile), no failure plane and tension cracks were observed in the reinforced zones, except for in the several bottom soil layers in the right-hand wall. (Figure 24), as with Chiba No. 1 test embankment.

5) Practically no creep deformation of the embankment took place in the subsequent years after large wall deformation took place by the rainfall test in the second year (1985) (**Figure 27**). It is likely that this behaviour is due to the effects of preloading on the backfill and pretensioning of the reinforcement applied associated with the previous relatively large wall deformation. This result indicates that by effects of preloading by external loading, heavy rainfall, seismic load and so on, only small creep deformation may take place under ordinary static load conditions for the subsequent periods. Typical creep test results that support this inference are presented in **Figure 28**. The specimen was cylindrical with a periphery length of 30 cm and a length of 8 cm of a 100 % polypropylene spun-bond (needle-punched) non-woven geotextile with a mass per unit area of 400 g/m^2, as used for Chiba No. 1 embankment (see Figure 3 of Tatsuoka and Yamauchi 1986). The test results obtained by this testing method are similar to those obtained under plane strain conditions, which are operational conditions in the embankment (Yamauchi, 1986). The ultimate tensile rupture strength was about 3 tonf/m. A constant tensile load was applied to the specimen stepwise in the sequence of 50 kgf, 100 kgf, 150 kgf, 200 kgf, 250 kgf and 300 kgf, and then 250 kgf, 200 kgf, 150 kgf and finally 75 kgf. It may be seen that creep deformation did not occur in the creep tests after the load level was decreased from the maximum load; even delayed contraction took place (i.e., the behaviour

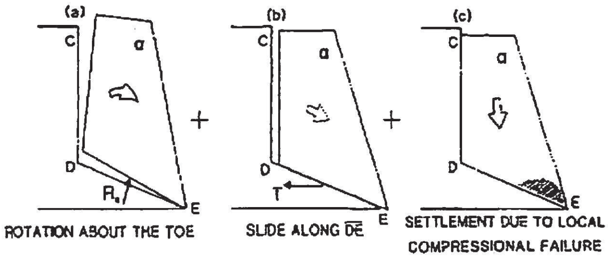


Figure 26. Schematic diagrams showing three typical failure modes observed in the right-hand wall of Chiba No. 2 embankment.

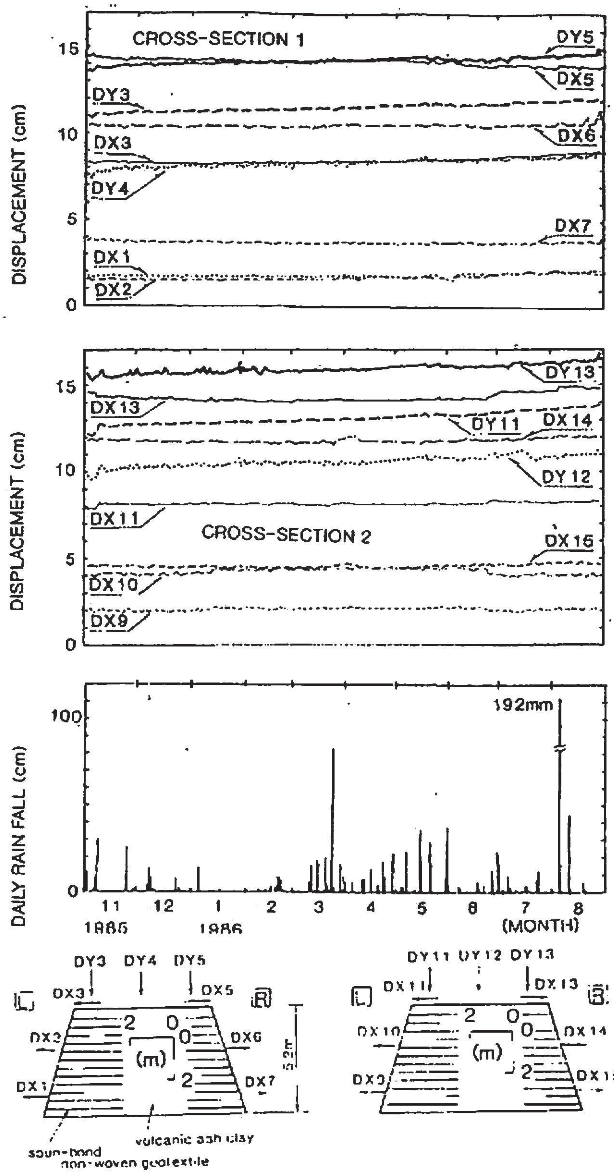


Figure 27. Long-term time histories of deformation and daily rainfall after the artificial rainfall test until its demolishing, Chiba No. 2 embankment.

opposite to tensile creep deformation, known as creep recovery). Similar test results are reported in Yamauchi (1986).

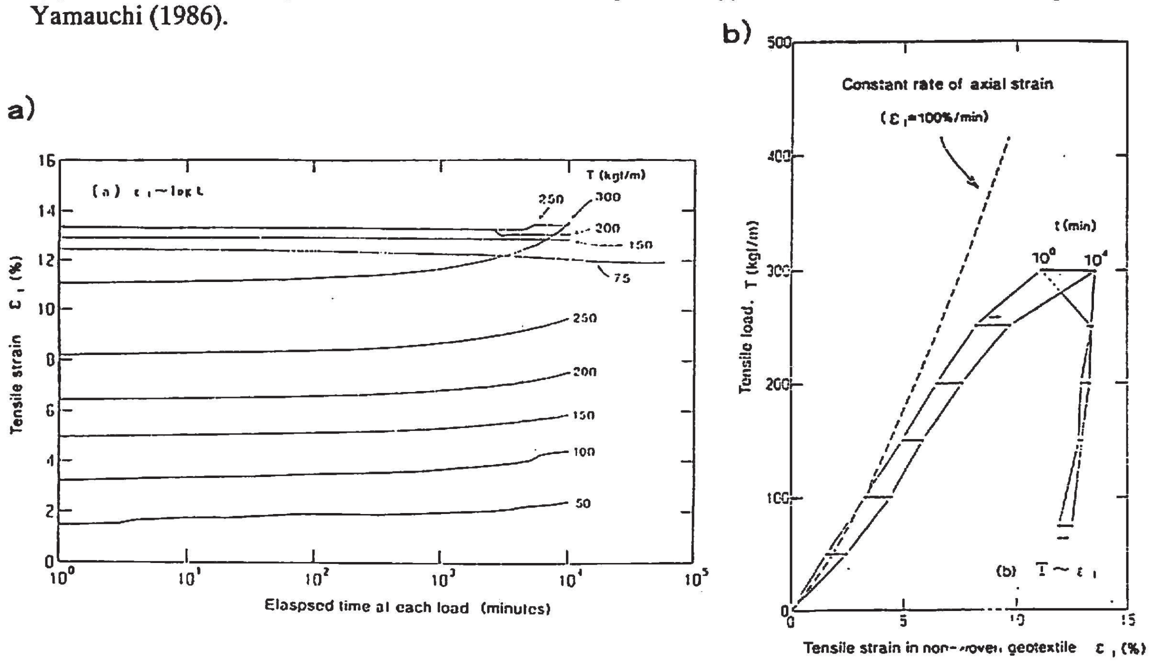


Figure-28. Typical results from a creep test of a non-woven geotextile; a) relationships between tensile strain and log. of elapsed time in each creep test-stage; and b) relationships between tensile load and tensile strain (the broken line is the result from a tension test at a constant axial strain rate of 100 %/min.) (Yamauchi, 1986).

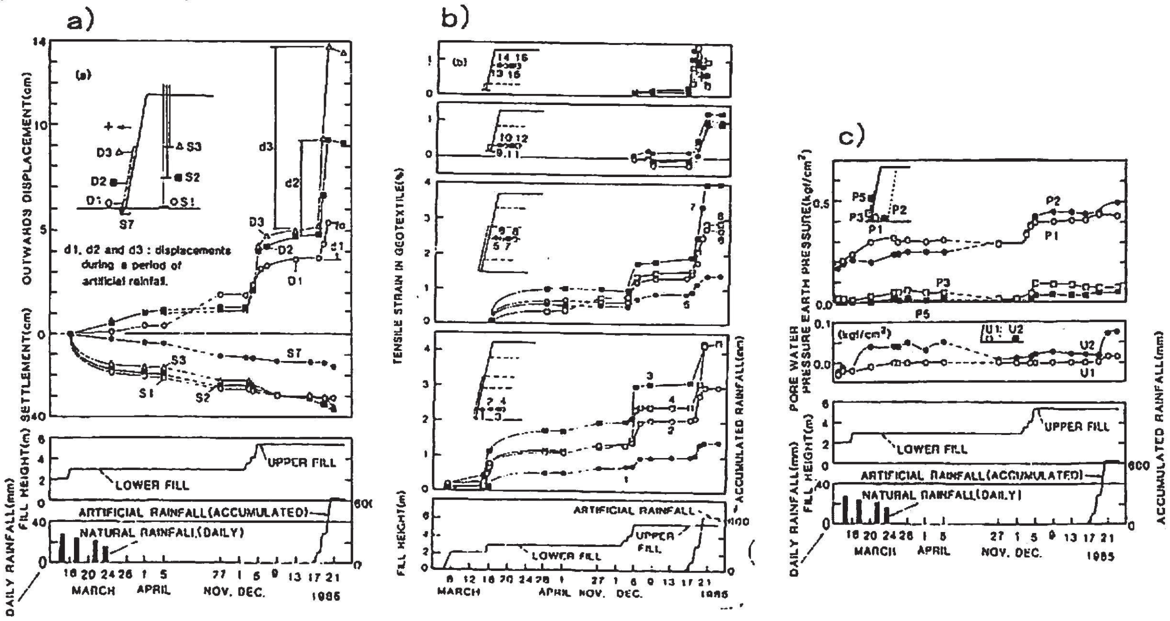
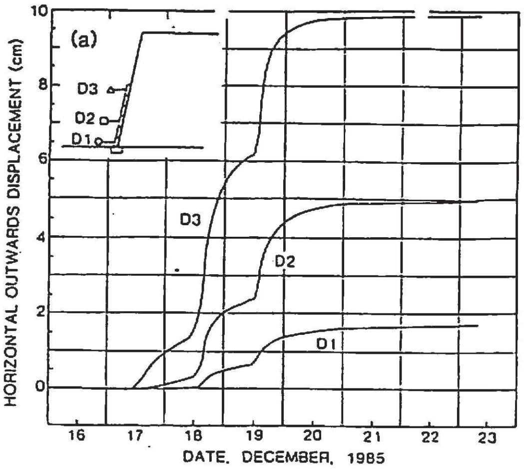


Figure 29. Time histories of; a) wall deformation, fill height and daily rainfall; and b) tensile strains in the reinforcement; and c) earth pressure at the back of facing and pore water pressure in the backfill, during and after construction, Kami-Onda embankment.

Kami-Onda embankment

The details of the post-construction behaviour for about a half year of this embankment and the behaviour during a heavy rainfall test was reported by Tatsuoka et al. (1987). The lessons obtained from the previous tests described above were reconfirmed by the results from this test, as follows:



1) The effects of suction and positive pore water pressure could be significant on the stability of clay wall. That is, the post-construction behaviour (**Figure 29**) showed that the deformation of the lower part of the wall having a discrete panel facing during the rainfall test was much larger than that induced by the weight of the upper part of the wall. In addition, it can be seen from the behaviour during the rainfall test (**Figure 30**) that the deformation of the wall was associated with the decrease in the suction and the increase in the positive pore water pressure. As seen from **Figure 31**, the geotextile tensile strains are well correlated to the increase in the pore water pressure.
2) When demolishing the embankment, cracks were observed only in the unreinforced zone (**Figure 32**). As a whole, despite large deformation of the wall and the supporting ground and the use of a so-called extensible reinforcement (i.e., a non-woven geotextile), the reinforced zone behaved like a monolith.

Chiba No.3 embankment

This embankment was constructed in October 1986 and demolished in October 1994. The essential role of facing rigidity for wall stability was reconfirmed by its behaviour. That is, already during construction, the back-side wall having a flat wrapped-round face without gabions exhibited large deformation, which was much larger than that of the other two walls, having more rigid facing (see **Figures 33** and **34**). About a half year after construction, a counter-weight fill was constructed to restrain further large deformation of the backside wall, which may damage seriously the other walls (Figure 34b). In the second and subsequent years after construction, the deformation of the backside wall still continued, while the deformation of the

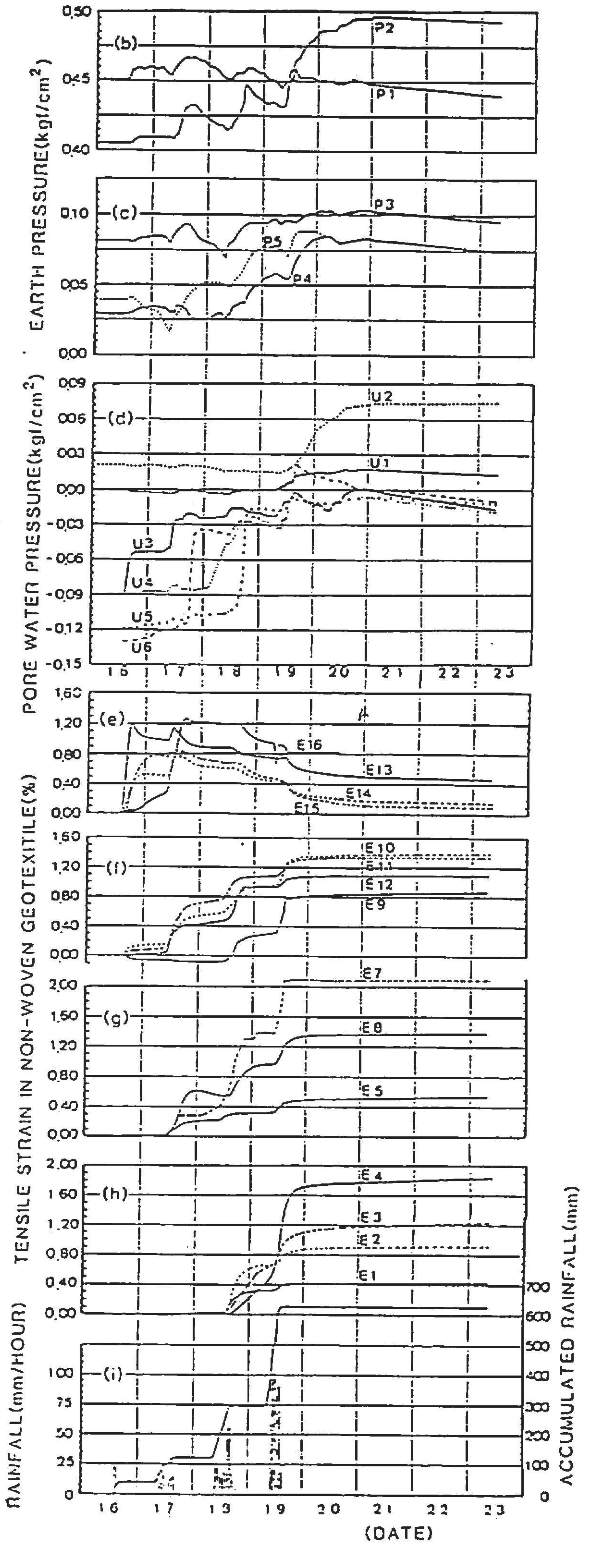


Figure 30. Time histories of; a) lateral displacement at the wall face; and b) earth pressure, pore water pressure, reinforcement tensile strains and daily rainfall, during the artificial rainfall test, Kami-Onda embankment.

other two walls became very small. It may be seen from **Figure 35** that the major increase in the tensile strain in the reinforcement (i.e., a non-woven geotextile) occurred only in the first year after the wall construction. The connection force at the back of facing was very large, indicating its importance for wall stability.

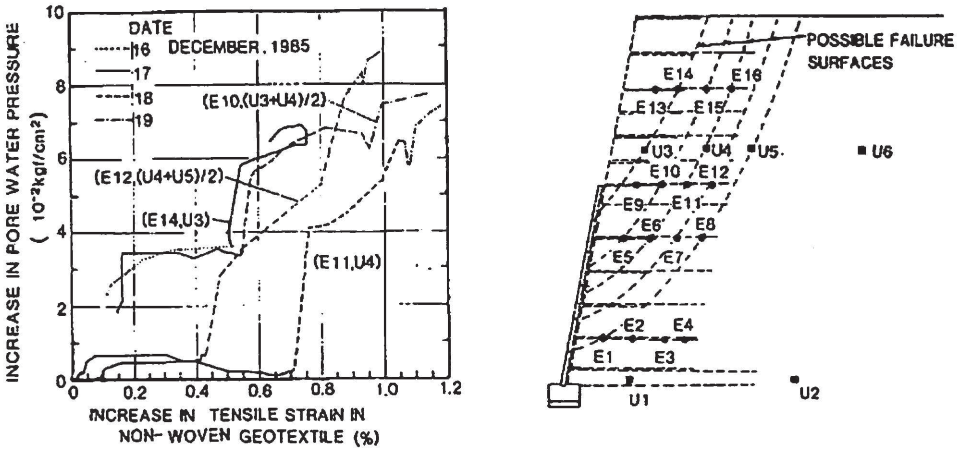
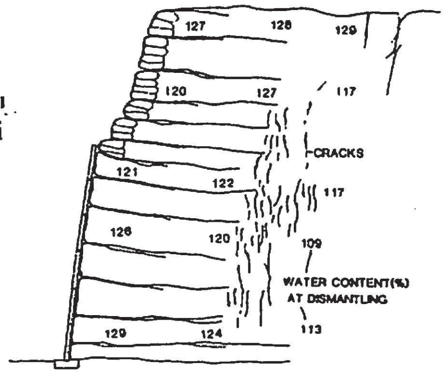
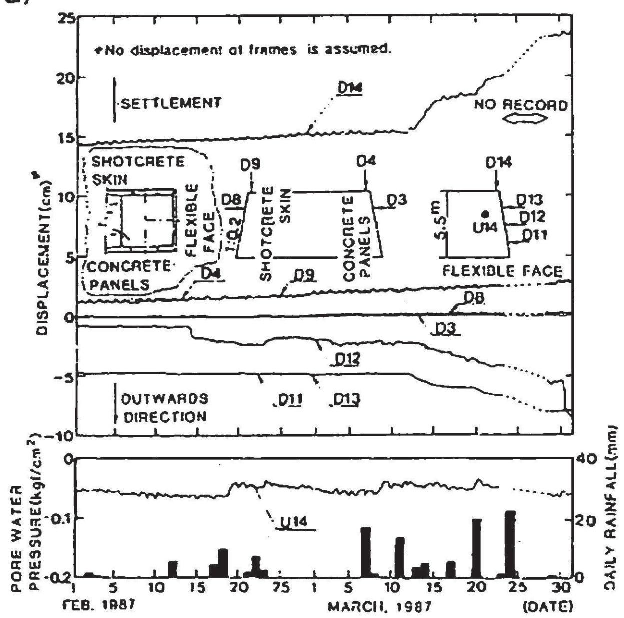


Figure 31. Correlation between pore water pressure and geotextile tensile strains during the artificial rainfall test, Kami-Onda embankment.

Figure 32. Cross-section exposed at demolishing, Kami Onda embankment.



a)



b)

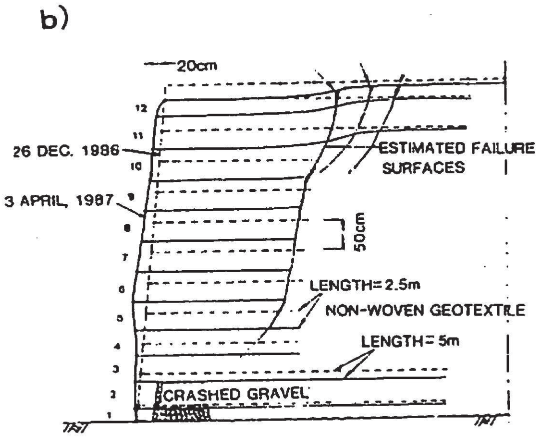


Figure 33. Time histories of wall deformation, pore water pressure in the backfill and daily rainfall in the three walls immediately after its construction; and b) estimated deformation of the backside wall, Chiba No. 3 embankment.

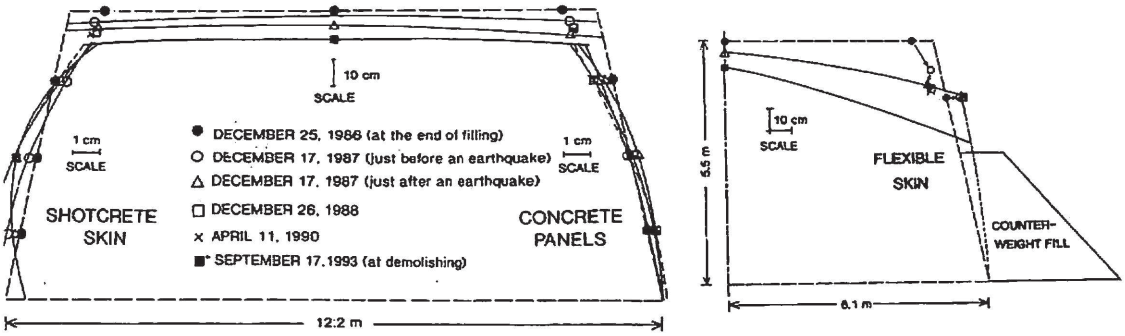


Figure 34. Deformation of; a) right-hand and left-hand walls; and b) backside wall, Chiba No. 3 embankment.

The wall deformation was carefully observed at demolishing in 1994 (**Figure 36**). It was found that the major part of the backfill of the right-hand and left-hand walls, having a discrete panel facing and a gabions covered with a shotcrete facing, respectively, remained essentially intact with no cracks, despite that the backfill was reinforced with a non-woven geotextile. The cone penetration values q_c measured when demolishing the embankment (**Figure 37a**) were similar to those measured during construction (see Fig. 10 of Nakamura et al., 1988). On the other hand, the q_c values measured in the backfill of the wrapped-around back-side wall were very small in several zones (**Figure 37b**). This reduction may be due to significant disturbances caused by large deformation of the wall. The water content measured when demolishing the embankment (**Figure 38**) was similar to the values measured during construction.

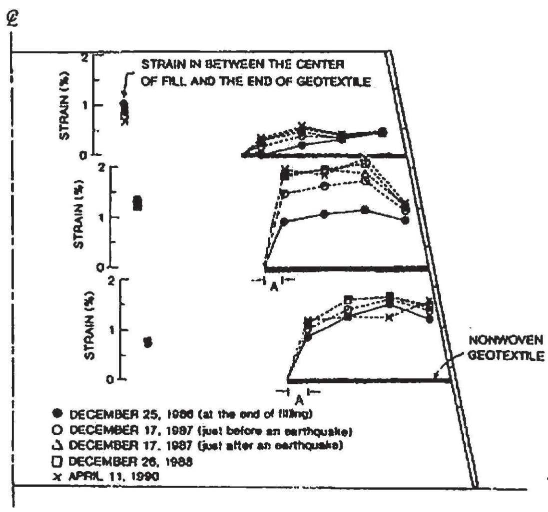


Figure 35. Distributions of tensile strains in reinforcement layers in the wall having a discrete panel facing, Chiba No. 3 embankment.

The major lessons obtained from this case are as follows:

1) It was reconfirmed that the deformations of flat wrapped-around face without gabions of clay wall starts from the wall face and the deformation would become too large. So this type wall cannot be used as important permanent structures.
2) Even nearly saturated clay such as Kanto loam can be used as the backfill when reinforced with a proper reinforcement having a proper drainage function

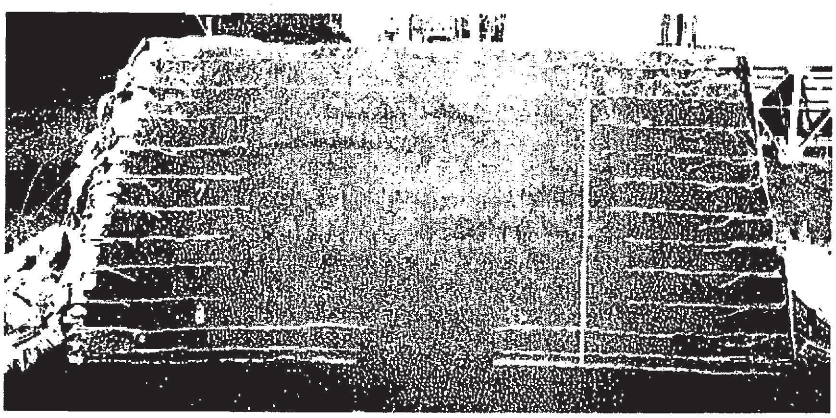
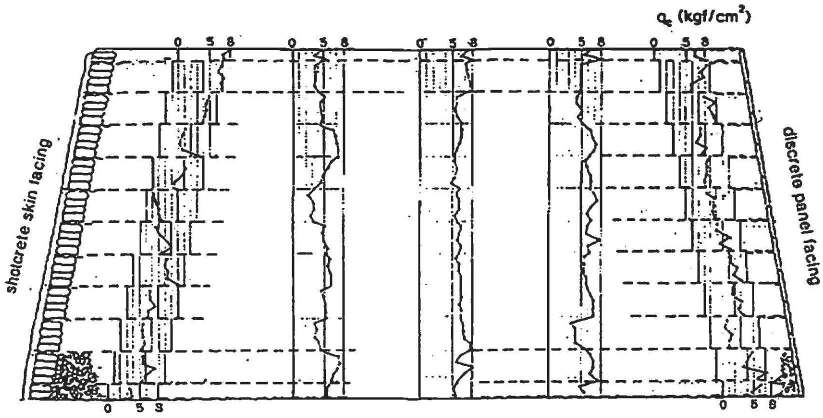


Figure 36. Cross-section including the right-hand and left-hand walls, exposed at demolishing, Chiba No.3 embankment.

and a tensile rigidity and a proper rigid facing is used.

3) Facing of relatively small discrete panels, as used for the right-hand wall, is not rigid enough, but the use of larger panels may be better. It was also found that it is very difficult to compact soil immediately behind such a panel without displacing outwards the panel. For this reason, it was very difficult to achieve a good wall face alignment. This problem has been appreciated also with Terre Armee walls, using a discrete panel facing (Tatsuoka et al., 1994).

4) The left-hand wall was constructed by a staged construction method, similar to that described in Figure 2. That is, a full-height wall was first constructed by a help of gabions placed the shoulder of each soil layer. After the deformation of the wall and the supporting ground became small enough, a shotcrete layer with a thickness of about 8 cm was cast-in-place on the wrapped-around wall face. The construction efficiency with this facing type was found much better than that with discrete panel facing. It was found, however, that this about 8 cm-thick shotcrete layer was not rigid enough to keep the wall face deformation to a very small value. Moreover, shotcrete facing may not be aesthetically acceptable in many features, in particular when constructed in urban areas.



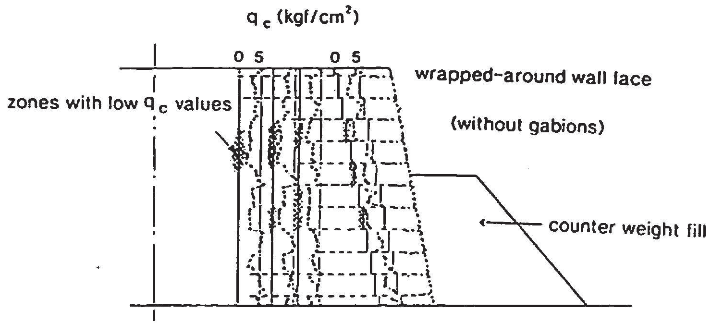


Figure 37. Cone penetration resistance q_c measured at demolishing, Chiba No.3 embankment.

Based on the experiences described above, the following studies were performed subsequently;

1) a series of laboratory model tests to examine the effects of facing rigidity on the deformability and stability of geotextile-reinforced soils retaining wall under static loading conditions (Tatsuoka et al., 1988) and under dynamic loading conditions (Murata et al., 1988);

2) two full-scale model tests on prototype walls (i.e., JR Nos. 1 and 2 tests embankments), considering the use of this type of GRS-RW as actual permanent important railway structures; and

3) the development of the design method for this type of GRS-RWs having a full-height rigid facing (Horii et al., 1994).

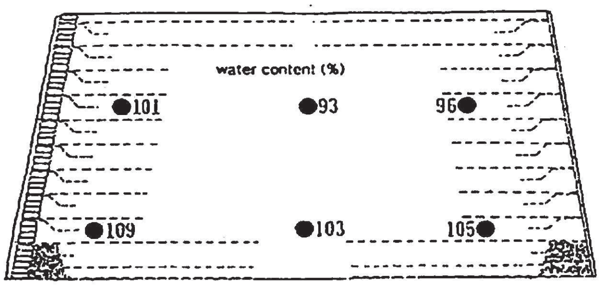
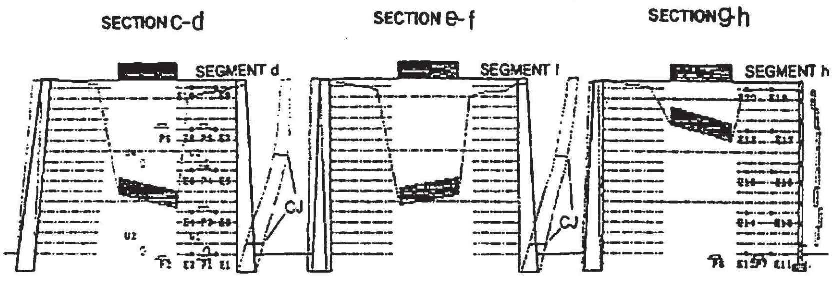


Figure 38. Water contents at demolishing in the right-hand and left-hand walls, Chiba No.3 embankment.

JR Nos. 1 and 2 embankments

The JR Nos. 1 and 2 embankments, with sand and clay backfill, were constructed by the staged construction method (Figure 2). In place of a shotcrete layer, an unreinforced concrete facing, with a thickness of 55 cm at the base and 30 cm at the top, was cast-in-place on the wrapped-around · wall face (with gabions) so that the facing became more rigid and the finished wall face became aesthetically more acceptable. The gabions were filled crushed gravel, in place of Kanto loam. The long-term behaviours of the walls for about two years after construction until loading tests, revealed the followings (Tatsuoka et al., 1992):



SEGMENT	d	f	h
SETTLEMENT OF FOOTING (cm)	69.4	68.3	29.8
FOOTING PRESSURE (tonf/m²)	57.5	50.0	37.5

Figure 39. Deformation by loading of three cross-sections of JR No. 1 embankment.

a) The test wall segment *h* of JR No.1 sand embankment, having a discrete panel facing, exhibited deformation that was much larger than that of the other walls (Figure 14b).
b) For the other two test wall segments, having a full-height rigid facing, the outward displacement of the wall face was practically a nil, and the settlement at the crest was also very small (Figures 10 and 14).

The following lessons were obtained from the results of the loading tests (Tatsuoka et al., 1990, 1992, Murata et al., 1991, 1992):

1) The test wall segment *h*, having a discrete panel facing, was most deformable and weakest among the three test segments of No. 1 embankments that were loaded at the crest (**Figure 39**), showing that this type of discrete panel facing cannot be used for permanent important structures.
2) Having the same type of full-height rigid facing, the grid reinforcement for segment *f* was shorter than that of segment *d* (1.5 m vs. 2.0 m). The effect of reinforcement length was found noticeable. A FEM analysis (Ling et al., 1995) showed that the three reinforcement layers placed over the embankment width contributed only partly to the stability of the wall. Undoubtedly, the stability of GRS-RW increases with the increase in the length of reinforcement.
3) When reconstructing a gentle slope of existing embankment to a vertical wall so as to widen the crest, the use of long reinforcement may require a large excavation into the slope, which may not be cost-

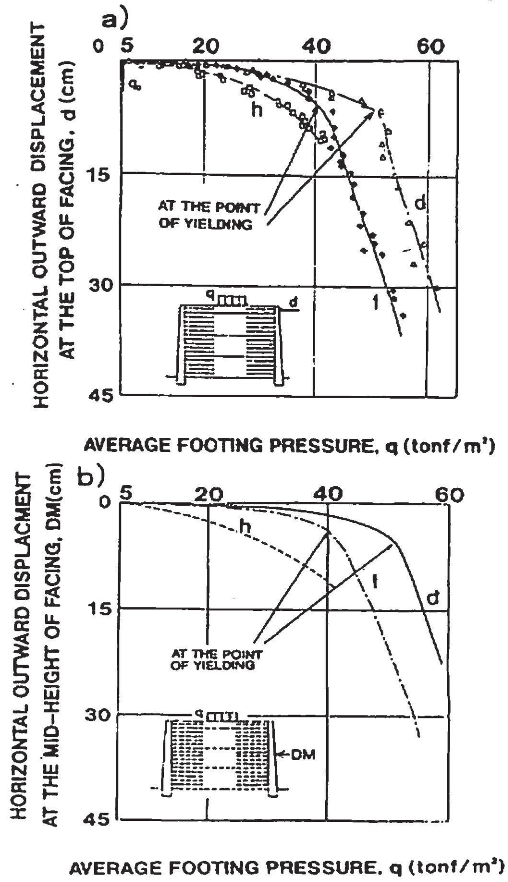


Figure 40. Outward horizontal displacements at; a) the top of facing; and b) the mid-height of facing, plotted against the average footing pressure, three wall segments of JR No. 1 embankment.

effective and also may unstabilize the slope (Figure 3b). On the other hand, the use of short reinforcement may eliminate or minimise excavation into an existing slope during reconstruction. In this test program, it was examined whether sufficiently stable GRS-RWs can be constructed using very short reinforcement. The wall segment *f*, with reinforcement having a length of only 1.5 m, was stable enough against ordinary design loads. Based on this fact and associated numerical simulation of the test result, the current design method specifies that the minimum allowable reinforcement length is equal to the smaller value of 1.5 m and 35 % of the wall height, on the premise that the wall stability is ensured by proper stability analysis, as described in Murata et al. (1992) and Horii et al. (1994).

4) The yielding of the test wall segments *f* and *d* upon vertical loading on the crest is described in **Figure 40**, which was controlled by the failure at the construction joint in the unreinforced facing, as shown in Figure 39. Apparently, the walls would have been stronger if the construction joints had been reinforced with steel reinforcement. Based on this experience, the facing used for prototype GRS-RWs is lightly steel-reinforced so that it can withstand the design earth pressure, which is equal to the active earth pressure when the backfill soil is not reinforced.

5) The unreinforced concrete facing of the test wall segment of JR No. 2 clay embankment was made stiffer and stronger by attaching a pair of H-shaped steel beam as a stiffener on the wall face before the loading test. The clay backfill of the test segment was reinforced with a composite geotextile. A footing, which was 2 m wide and 3 m long (in the direction orthogonal to the plane of this page), was used for back loading (**Figure 41a**), while another one, which was 1 m wide and 3 m long, was used for front loading (**Figure 41b**). Under nearly or fully drained conditions, the

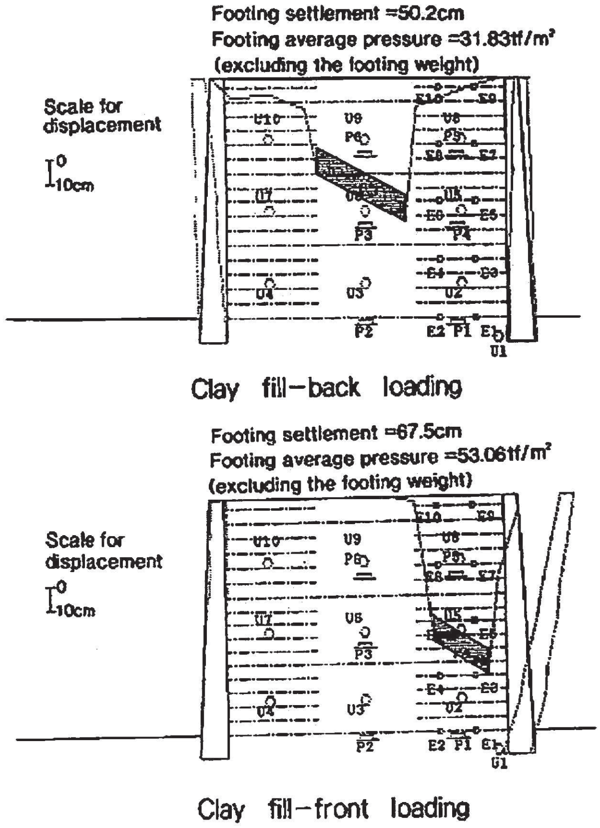


Figure 41. Deformation by loading of three cross-sections of JR No. 2 embankment.

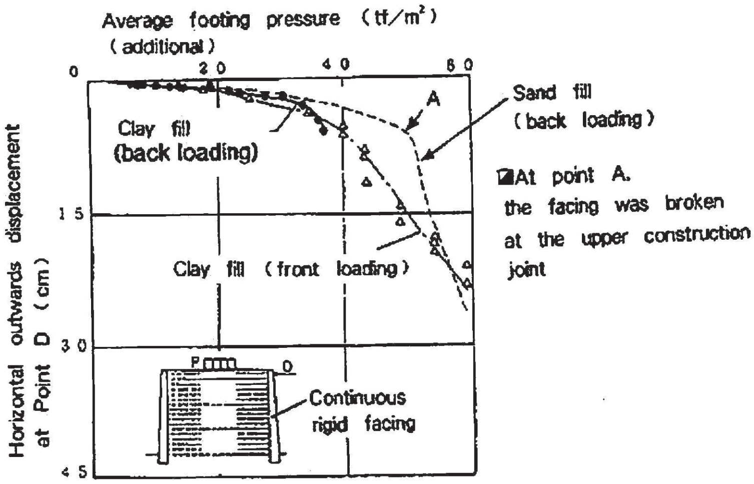


Figure 42. Relationships between footing pressure and outward horizontal displacement at the mid-height of facing by back-loading and front loading of JR No. 2 embankment, compared with that of wall segment *d* of JR No. 1 embankment (denoted by the letter A).

bearing capacity in terms of the maximum footing pressure decreases with the decrease in the footing width. Despite this factor and the fact that the 1 m-wide footing was located much closer to the wall face, the bearing capacity in terms of footing pressure was larger in the front loading test (**Figure 42**). It is likely that the effects of reinforcing the backfill zone immediately below he footing using a composite geotextile overwhelmed the effects of the above two factors. In addition, the deformation of the clay wall (JR No. 2) at a average footing pressure of, say, 20 tonf/m^2, was only slightly larger than that of the sand wall (JR No. 1). An important lesson is, therefore, that a properly designed and constructed GRS-RW using a nearly saturated clay backfill can be constructed as an important permanent structure that supports large concentrated load on the crest.

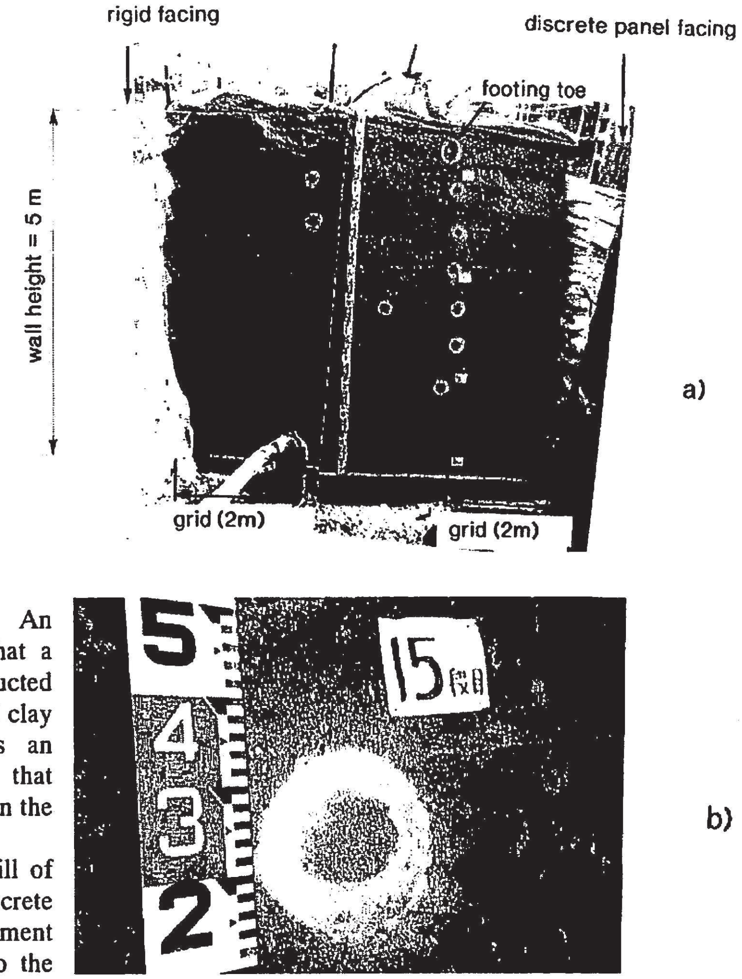
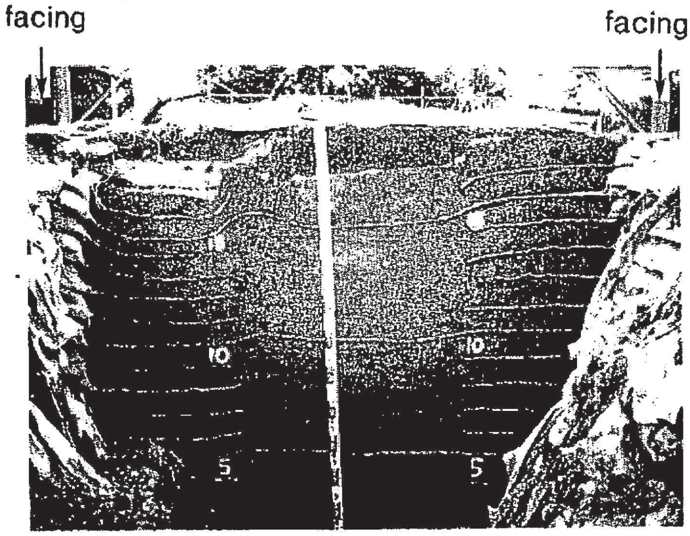


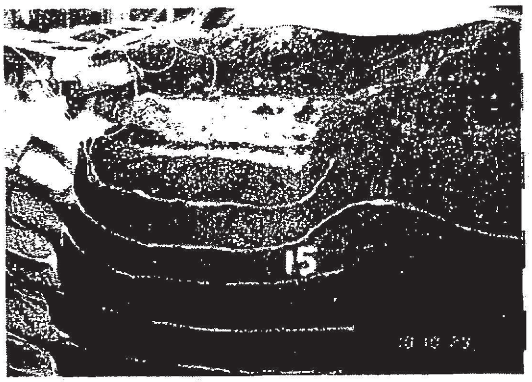
Figure 43. Shear zone observed in segment *h* having a discrete panel facing of JR No. 1 embankment.

6) After the loading test, the backfill of the wall segment *h*, having a discrete panel facing, of JR No. 1 embankment was excavated to investigate into the failure mode (**Figure 43a**). In this figure, white circles denote the places where shear bands were identified. As seen from this figure, two failure planes (or shear bands) developed from the toe and heel of the footing. **Figure 43b** shows the details around the second circle from the crest along the failure plane extending at the back face of the reinforced zone. The failure plane starting from the footing toe did not penetrat into the reinforced zone, but developed downward vertically along the back face of the reinforced zone. Near the wall bottom, the two failure planes joined into a single failure plane, which developed towards the wall face, penetrating the reinforced zone. This failure mode indicates how difficult a failure plane develops in a reinforced zone even when the reinforcement is a so-called extensible one. It was also noted that this failure mode is very similar to the one that had been observed in the corresponding small model tests at a scale of one tenth (see the failure modes of the models Types B and B' shown in Fig. 9 of Tatsuoka et al., 1988). These observations validate the two-wedge failure mechanism that is assumed in the current design method employed for GRS-RWs (Horii et al., 1994).
7) The section c-c of JR No.2 clay embankment was also excavated after the loading tests (**Figure 44**). It may be seen that in the front loading test, the gabions functioned as a buffer for the relative settlement between the rigid facing and the backfill soil, preventing serious damage to the connection between the facing and the reinforcement. Further, any clear failure plane was not observed in the reinforced zone subjected to vertical loading at the crest. This is another result showing how difficult a shear band (or failure plane) develops in a reinforced soil.

The stability of the failed walls of JR Nos. 2 and 3 embankments was analysed by the limit equilibrium



a)



b)

Figure 44. Cross-section of section c-c after loading test, JR No. 2 embankment.

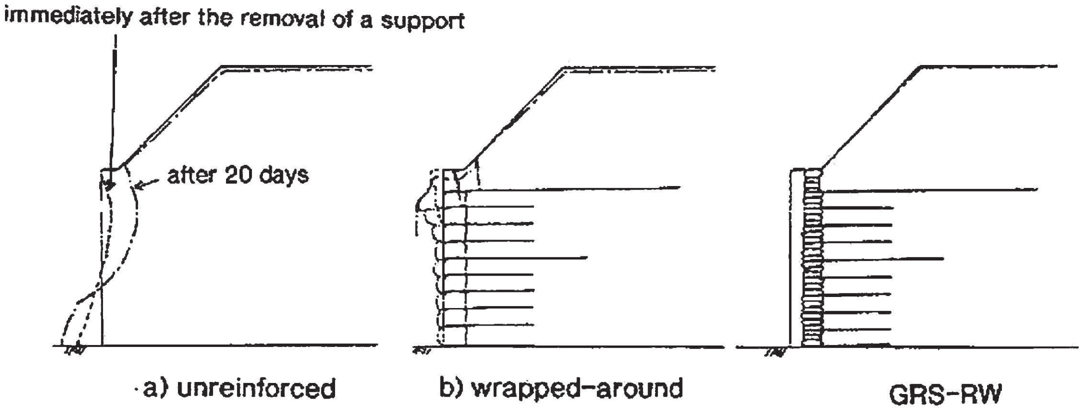
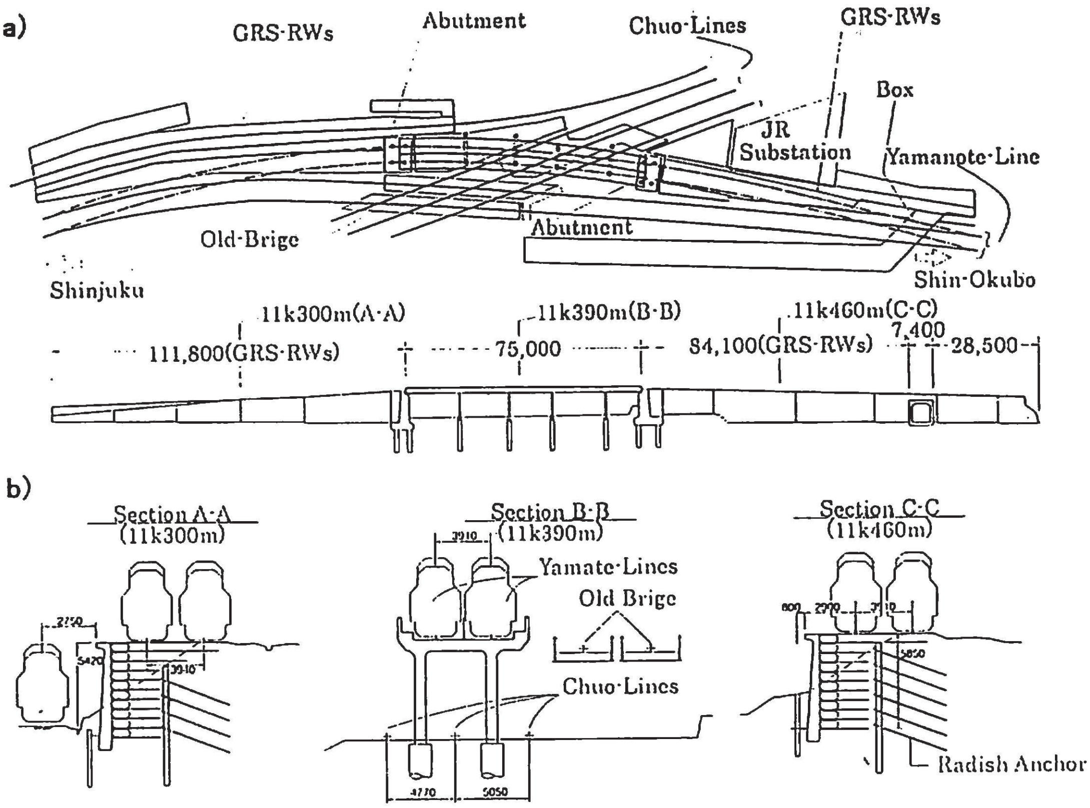
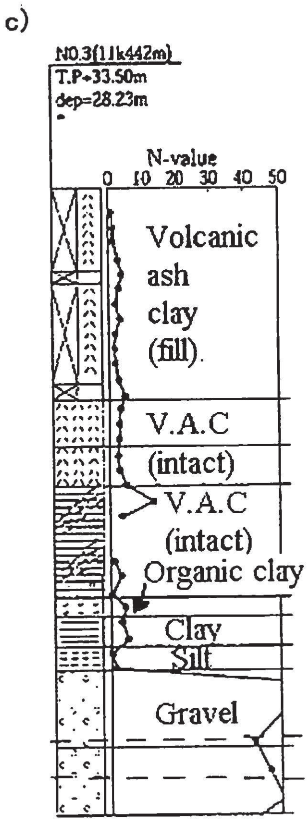


Figure 45. Failure and deformation of test walls at Nagano; a) unreinforced wall; b) wrapped-around reinforced wall (without gabions); and c) GRS-RW having a full height rigid facing.

method, taking into account the effects of facing rigidity (Murata et al., 1992). The deformation of JR No. 2 clay embankment was analysed by FEM analysis (Ling et al., 1995). The results from these studies contributed to the decision of adopting this type of GRS-RWs as important permanent structures for railways (see Figure 4).





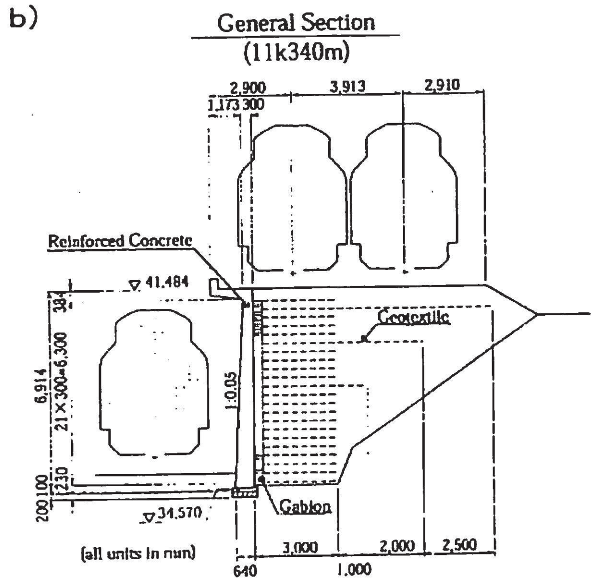


Figure 46 GRS-RW having a full-height rigid facing in a latest project close to Shinjuku station in Tokyo; a) general plan and general side view; b) typical cross-sections; c) ground condition; (see figure a) (to continue).

Nagano Test Embankment

Figure 45 shows; a) the failure of the unreinforced clay wall; b) large deformation of the wrapped-around reinforced clay wall (without gabions); and c) the deformation (actually no observable deformation) of the GRS-RW. Large deformation of the unreinforced and wrapped-around clay walls started immediately after the removal of a support of EPS piece (see Figure 11b). Without having received any heavy rainfall, the walls exhibited deformation continuously for a subsequent period of 20 days. The failure of the unreinforced clay started form ripping off of small soil pieces from the wall face, which triggered the failure of the overlying soil mass. The failure in the similar manner continued progressively towards deeper places of the wall. Large outward deformation of the wrapped-around reinforced wall started at the wall face of the top three layers, which was followed by the deformation of the lower part caused by a local compressive failure of soil at the wall face. In fact, the stability at the wall face of the wrapped-around wall was found not much better than that of the unreinforced wall. The results showed again that the weakest part of wrapped-around clay wall is facing (in particular when the wall face is flat not using gabions).

TYPICAL RECENT CASE HISOTRY OF GRS-RW

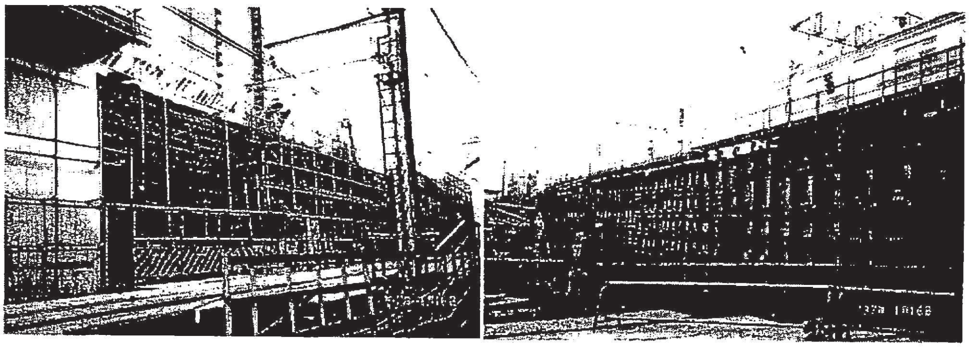
Based on the lessons obtained from a series of full-scale failure tests described above and results from a comprehensive series of laboratory model tests and numerical analysis, the staged construction procedure of GRS-RW having a full-height rigid wall (Figure 2) was developed. Tatsuoka et al. (1997a) reported a number of case histories of GRS-RWs that had been constructed by the staged construction procedure (Figure 2) as important permanent structures for railways and highways. The sites of these projects are numbered up to around 100 in the chronological order in Figure 4. **Figure 46** shows one of the latest typical projects, located close to Shinjuku station in Tokyo, which is the busiest station in Japan. The project is the reconstruction of a very old bridge for Chuo Line and an associated relocation of two railway tracks for Yamanote Line, as shown in **Figure 46a**. Figure 46b shows three typical cross-sections. As Chuo and Yamanote Lines are the busiest and most important rapid transits in Japan, this project has been considered as one of the most critical and challenging geosynthetics engineering case projects. The project started five years ago and completed in the beginning of this year (2000).

It was decided to adopt GRS-RWs having a full-height rigid facing for this project because of the following advantages over the conventional retaining wall types:

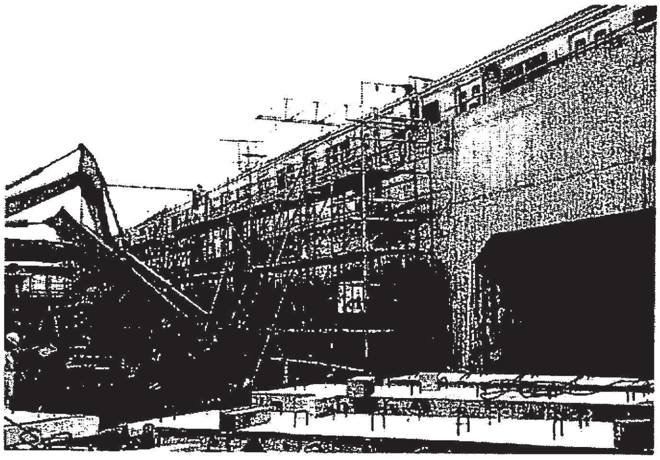
1) The subsoil consists of several lightly cemented volcanic ash clay layers (locally called Kanto loam), which deposited at different epochs during the Pleistocene period, underlain by an older dense gravel deposit (**Figure 46c**). The surface Kanto loam layer can support a two or three story wooden residential house without using a pile foundation, and it is also a usual practice that embankment of about 5 m high is constructed directly on this soil deposit without any serious problem of excessive settlement. In comparison, conventional cantilever RC soil retaining walls, if constructed at this site, should be supported by a pile foundation that is longer than 15 m, reaching to the Pleistocene gravel layer (see Figure 46c). On the other hand, reinforced backfill can be constructed without using a pile foundation. In fact, the GRS-RWs were constructed without using a pile foundation, exhibiting only a small ground settlement.
2) Due to a very severe space restriction at the site, large construction plants cannot be used. The construction of GRS-RWs needs only small construction plants.
3) The GRS-RWs becomes stable and stiff enough after a full-height rigid facing is cast-in-place.
4) The GRS-RWs are therefore much more cost-effective than conventional type soil retaining walls.

In this project, as shown in **Figure 46b**, the existing railway embankment was fist excavated after it had been reinforced by; a) pre-propping by using vertical sheet piles; and b) nailing using large diameter (about 40 cm) in-situ cement-mixed columns with central reinforcement of fibre-reinforced plastic (FRP). The nail is denoted as Radish Anchor in Figure 46b (Tateyama et al., 1992 and Tatsuoka, 1993). GRS-RWs were then constructed in front of the nailed walls. **Figure 46d** shows the details of the GRS-RW, **Figures 46e** and **f** show views during construction, and **Figure 46g** shows views of the completed GRS-RWs.

d)



e)



f)

g)

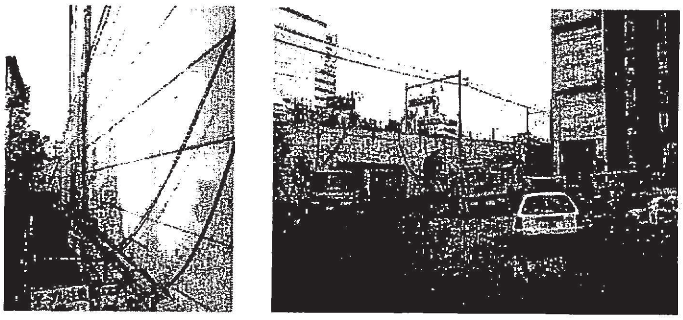


Figure 46 (continued). d) details of GRS-RW and e) a view during wall construction; f) a view during finishing the wall; and g) views of completed GRS-RW.

CONCLUDING REMARKS

Among those learned from the failure of a series of full-scale test walls described in this report, the following lessons are important:

1. Wrapped-around walls are generally too deformable, particularly when the wall face is finished flat without using gabions. Therefore, this type of reinforced soil retaining walls may not be used as important permanent structures that allow a limited amount of wall deformation. As discussed in detail by Tatsuoka (1993), a rigid facing, in particular a full-height continuous rigid facing, to which reinforcements are fixed helps in substantially increasing the stability of wall and in decreasing the deformation of wall.
2. The construction of sufficiently stable and rigid clay walls as important permanent structures is quite feasible by reinforcing the backfill with a proper composite geotextile having sufficiently high drainage function and tensile rigidity and using a full-height rigid facing. In the design of GRS-RWs having clay backfill, due consideration of drainage and consolidation of clay soil layers between

geosynthetic reinforcement layers is essential (e.g., Giroud, 1983).

3. For the last decade in Japan, a large number of prototype GRS-RWs, with a total wall length being about 35 km, have been constructed by the staged construction procedure shown in Figure 2. It is significant that so far any problematic case has been reported.

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