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Title: REINFORCING STEEP CLAY SLOPES WITH A NON-WOVEN GEOTEXTILE

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Six full-scale test embankments having near-vertical slopes were constructed, five of them by using a near-saturated clay, while one by using sand. Based on the observation of their behaviour, a reinforcing method suitable for most types of soils is proposed. In this method, relatively short planar sheets of geotextile having a function of drainage are used together with a continuous rigid facing structure placed after the filling of backfill is completed.

INTRODUCTION

1. Soil reinforcing will find more increasing popularity in stabilizing a steep slope of embankment if the following requirements are satisfied: ① For a large cost saving, most on-site soils including near-saturated cohesive soils can be used as the backfill, which are often available from excavation work in inland areas in Japan. On the other hand, the usage of cohesionless soils would be much more expensive. However, when using such cohesive soils, appropriate measures for the drainage should be taken for compacting the backfill effectively and for ensuring the stability of the slope. ② It should be reasonably inexpensive so that it could be used for large length as, for example, railway or highway embankments. ③ It can be used for the re-construction of embankment for making the slope steeper (Fig. 1). In this case, the use of long reinforcement needs a large amount of excavation of the existing gentle slope. ④ The deformation of slope, especially the settlement at the crest, should be small, particularly when used for railway embankments.

2. A reinforcing method for earth retaining walls which can satisfy the above requirements has been studied for the last decade. So far, six 4.0 to 5.4 m-high test embankments reinforced with various types of geotextile and facing structures were constructed. A high-water content, very problematic volcanic ash clay called Kanto loam was used for five of them, and a cohesionless soil for the other one. Their post-construction behaviour was continuously observed.

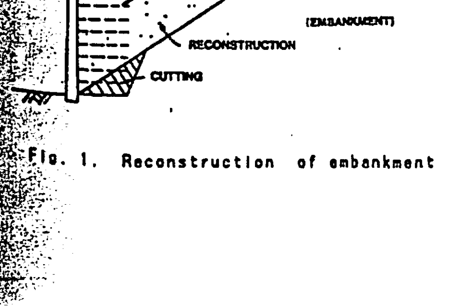


Fig. 1. Reconstruction of embankment

Table 1. Description of test embankments

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 3)	110 - 2)	116 123
Dry density (g/cm³)	at filling at demolishing	- 2) 0.69 1)	0.55-0.65 0.60-0.67	0.60 - 2)	0.68 0.66
Degree of saturation	at filling at demolishing	- 2) 85% 1)	90% 78-85% 3)	85% - 2)	93% 87%
q_c 5) (kgf/cm²)	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	5.0, 5.4 6)
Vertical spacing of geotextile sheets (m)		L 7): 0.6 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 each 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.

BEHAVIOUR OF TEST EMBANKMENTS

3. This research consisted of three stages.

First stage:

4. Three embankments (Nos. 1 and 2 and Kami-Onda) were constructed from 1982 (Table 1 and Fig. 2, refs 1-2, 4-6). For these three and the next (No. 3) clay embankments, a spun-bond non-woven geotextile of 100 % polypropylene was used. It had force per unit width (tonf/m) at 15 % elongation was $0.435 + 0.00675 \cdot \sigma_o$, where σ_o is the normal force between 0 and 32 tonf/m^2.

5. Artificial heavy rainfall tests were performed on No.2 and Kami-Onda embankments to critically evaluate their stability. They were dismantled to closely observe the inside failure modes (Fig. 2). The following points were found:

6. (1) The slopes of No. 2 (Fig. 2b) and the upper wall of Kami-Onda (Fig. 2c) were constructed with the aid of gabions placed at the shoulder of each soil layer. The lower wall of Kami-Onda (Fig. 2c) was constructed with the aid of discrete concrete panels. Despite their steep slope of 0.3 or 0.2 in horizontal to 1.0 in vertical, all these slopes were very stable against natural heavy rainfalls, due mainly to a high degree of compaction of the backfill behind the slope face and the confinement to the soil layer by the facing structure.

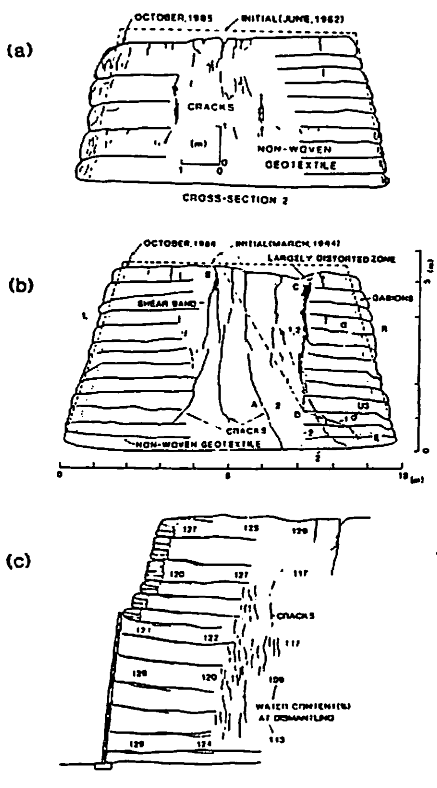


Fig. 2. Cross-sections of test embankments, (a) No.1, (b) No.2 (c) Kami-Onda (refs 1, 2, 5, 6)

Note: Lines denoted by 1 and 2 in Fig. 2b are the failure surfaces by the stability analysis by the limit equilibrium method with and without water pressure in the cracks, respectively.

7. The slope face of each soil layer of No. 1 (Fig.2a) was just wrapped around with a geotextile sheet. The right-hand slope having a vertical spacing of geotextile sheet was 40cm was very stable. However, in the left-hand slope in No. 1 embankment (Fig. 2a), large compressive deformation was caused in the lowest soil layer near the slope face by natural rainfalls of an anticipation of 30 mm/day or more. This was due to a too large vertical spacing of 80 cm between the geotextile layers in addition to the lack of the confinement to the soil by the non-woven geotextile sheet wrapped around the slope face of each soil layer.

8. (2) The non-woven geotextile functioned very well as a drainage both during the filling of backfill and against heavy rainfalls in maintaining a high degree of suction (i.e., negative pore water pressure) in soil layers between geotextile sheets. In contrast, the pore water pressure in the zones remote from geotextile sheets became positive occasionally during heavy rainfalls.

9. Fig. 3 shows the safety factors defined as the ratio of the failure height to the actual height of slope, plotted against the suction in soil, for the two slopes of No. 2 embankment. These values were calculated by the stability analysis by the two-wedge limit equilibrium method, by using the estimated shear strength of soil: τ = (total normal stress + suction)$\cdot$tan ϕ' (ϕ'= 30 degrees) (ref. 6). This result suggests that the suction contributes significantly to the stability of clay slopes.

10. (3) Fig. 4 shows the deformation of the three slopes caused by the artificial rainfall tests (ref. 6). They are simplified as illustrated in Fig. 5. It may be seen from Fig. 4a that the deformation due to the sliding along a failure surface (DE in Fig.5) was relatively small when compared to that due to the over-turning about the toe of slope (E in Fig. 5). This was due to the use of relatively short geotextile. In this case, the over-turning mode was accompanied with the local compression in the lower soil layers (see Fig. 4b).

Second stage:

11. Based on the observation described in the above, it was considered that the use of continuous rigid facing can increase the resistance of

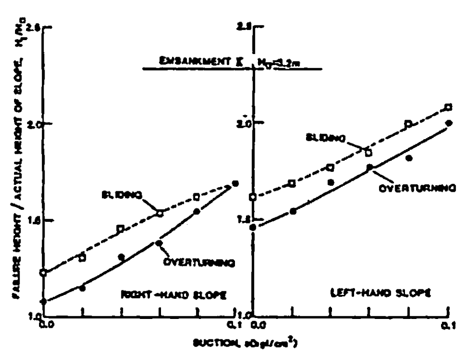


Fig. 3. Safety factors of slopes of No. 2 embankment assuming no water pressures in vertical cracks (ref. 6)

wall against the over-turning by increasing (1) the strength of soil behind the facing by the confinement of facing, (2) the overall bending rigidity of wall and (3) the vertical pressure developed on the bottom of facing.

12. Then, a 5.5 m-high No. 3 clay test embankment was constructed in 1986 (Fig. 6), which had three test slopes: (a) the back-side slope (Fig. 6b), with the slope face wrapped around with non-woven geotextile as the left-hand slope of No. 1 embankment, (b) the right-hand slope (Fig. 6a), covered with discrete concrete panels as used for the lower wall of Kami-Onda embankment, and (c) the left-hand slope (Fig.6a), constructed with the aid of gabions as the slopes of No.2 embankment, but with an about 8 cm thick shotcrete layer subsequently placed on it. Fig. 7 shows the deformation of the walls for three and a half years. Note that in Fig. 7, two different scales are used for deformation.

13. The embankment experienced several times of natural heavy rainfalls. The deformation in the back-side slope was much larger than that in the other two. Therefore, a counter-weight fill was constructed to stabilize it so that its deformation did not damage the other two stable slopes. The back-side slope further deformed slightly by a relatively large earthquake in December 1987, which had a maximum horizontal acceleration of 326 gals at the site. However, the deformation of the other two walls was negligible. After that event, all the slopes have been very stable.

14. Fig. 8 shows tensile strains developed in the geotextile in the right-hand slope. The development of some strains at the rear of facing indicates a degree of confinement from the facing applied to the soil layer. It may also be seen that the anchorage length, as denoted by A, was small, which is one of the characteristic features for such planar reinforcement.

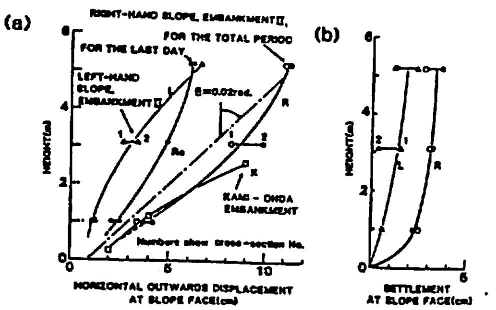


Fig. 4. (a) Horizontal outward displacement and (b) settlement at the slope face caused by the artificial rainfall tests, plotted against the height from the bottom of slope (ref. 6)

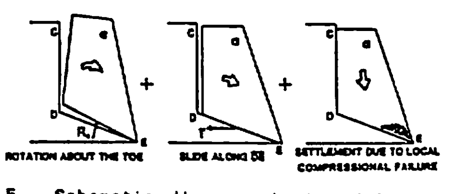


Fig. 5. Schematic diagram showing deformation modes shown in Fig. 4.

15. Any clear difference in the behaviour was not observed between the left-hand slope having continuous and relatively rigid facing and the right-hand slope having discrete-panel facing. The difference, however, was very clear in the laboratory tests in which model walls of sand with a wall height of 52 cm, having different

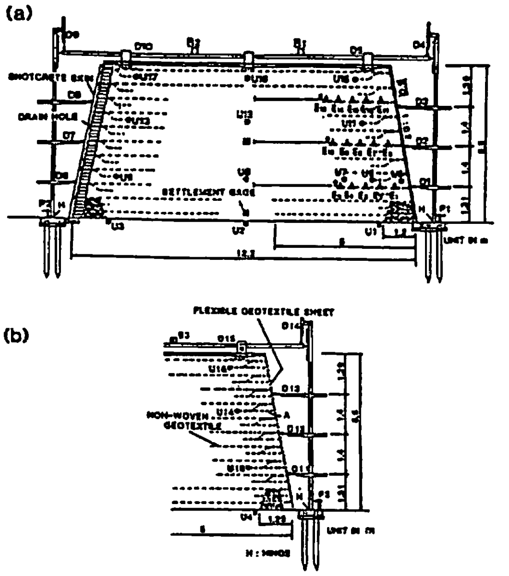


Fig. 6. (a) Right-hand and left-hand slopes and (b) back-side slope of No. 3 embankment (U: pore-pressure meters, E: tensile strain gauges for geotextile)

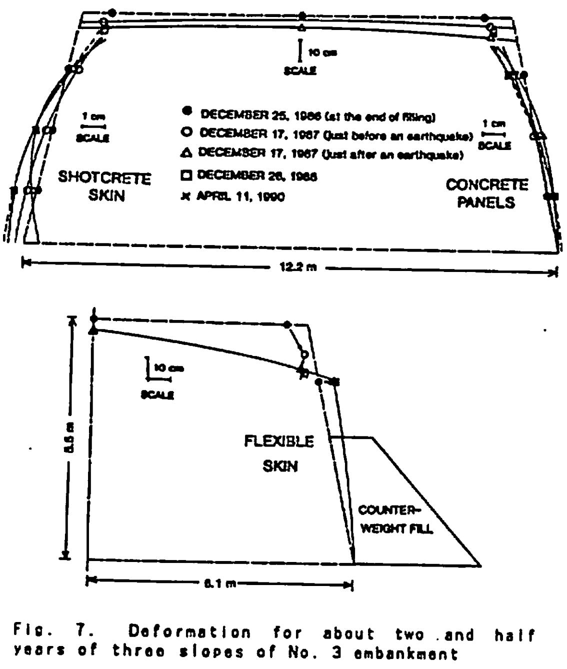


Fig. 7. Deformation for about two and half years of three slopes of No. 3 embankment

types of facing were brought to failure by loading at their crests (ref. 3). Namely, the facing of discrete concrete panels was simulated by piled-up wooden blocks with a rough back face. This model wall was weaker than another model wall having continuous rigid model facing. This model wall was very stable, despite the use of short (30 % the wall height) reinforcement of grid-type. Further, a model wall with facing of piled-up wooden blocks having a smooth back face together with a layer of soft material between the blocks was weaker than the above two model walls. The flexible facing wrapped around with geotextile sheets was simulated by a model facing structure made of a rubber membrane. This model wall was extremely weak.

Third stage:

16. To confirm the findings described above, other two test embankments were constructed from the end of 1987 through the beginning of 1988. The first one (JR No. 1, Fig. 9a) used a sand backfill having a mean diameter of 0.2 mm and a fines content of 16 %. It was reinforced with a grid-type reinforcement made of polyester dipped with PCV liquied, having an ultimate strength of 3.0 tonf/m. Among its six test segments, five had continuous rigid facing of unreinforced concrete slab. The other one had discrete-panel facing as used for the lower wall of Kami-Onda and the right-hand slope of No. 3.

17. The second embankment (JR No. 2, Fig. 9b) used a volcanic ash clay (Kanto-loam) backfill having a water content of about 120 % and a degree of saturation of about 90 %. Two of the six test segments were reinforced with a grid--type geotextile sandwiched between gravel drainage layers, two others with a non-woven geotextile and the other two with a composite of a sheet of high tensile-rigidity woven geotextile sandwiched between non-woven geotextile sheets. The ultimate strength was 1.8 tonf/m. All the six test segments had continous rigid facing.

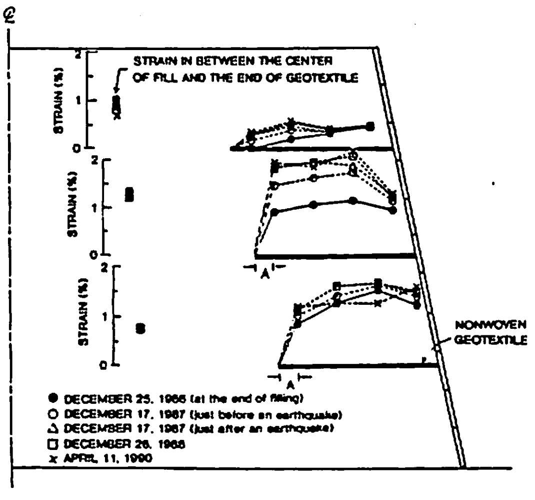


Fig. 8 Strains in non-woven geotextile layers at the right-hand slope of No. 3 embankment

18. All the test segments having continuous rigid facing of the two JR embankments exhibited a very small settlement on the crest of only about 1 cm about two years after the construction (Fig. 9). On the other hand, the test segment with discrete-panel facing exhibited a relatively large deformation. These results are well in accordance with those of the laboratory model wall tests (ref. 3). These embankments has also been very stable against several times of natural heavy rainfalls.

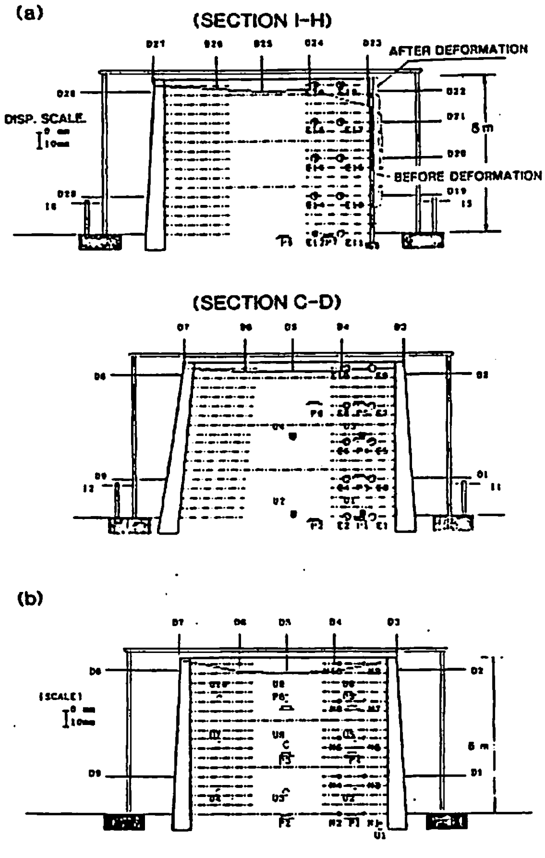


Fig. 9. Deformation for about two years of typical segments of JR embankments: (a) No. 1 (sand) and (b) No. 2 (clay)
Note: Scale is different for embankment and deformation.

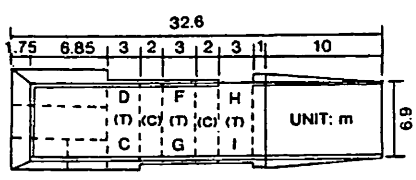


Fig. 10. Plan of JR No.1 embankment (T: test section, C: control section)

19. Three test sections of JR No.1 embankment were loaded at their crests by using a footing with a 2 m x 3 m base in January, 1990 (Figs. 10 and 11). The footing was located at the center of the crest, at a distance of 2 m from the rear of facing. The test sections were separated to each other by a control section in between. A layer of two plywood sheets with a grease layer in between had been placed between each test section and the adjacent control sections to achieve plane strain conditions. Of two segments in each section, the one which had been considered weaker in advance failed (Fig. 11).

20. Fig. 12 shows the relationships between the average footing pressure and the outward lateral movement at the top of wall. The effect of the different length of geotextile between Segments D (2 m) and F (1.5 m) and that of the different facing types between Segments D and H may be seen. As indicated in Fig. 12, Segments D and F yielded when a crack appeared in the upper one of the two construction joints, which were very lightly steel-reinforced, in each facing. If the construction joint had been stronger, the footing load which the walls could sustain would have been larger. Indeed, this result is one of the evidences for the contribution of the overall rigidity of facing to the stability of the walls. It is also found that each pattern of deformation of facing as seen in Fig. 11 was very similar to that observed in the corresponding model test in the laboratory. JR No.2 clay embankment will be loaded in 1990.

PROPOSED REINFORCING METHOD

21. Based on the above, the following reinforcing method can be proposed as the one which satisfies the requirements described in INTRODUCTION: ① Planar geotextile sheets are used to increase the contact area with the backfill, thus, to reduce the required anchorage length. For a cohesive soil backfill, a composite of non-woven and woven geotextiles is used to facilitate drainage and to ensure high tensile rigidity and strength. ② Continuous rigid facing is used to stabilize the wall and to reduce the deformation of wall, particularly the settlement at the crest of backfill, by enhancing the reinforced zone to behave like a monolith. Its use is also to increase the resistance against mechanical damage and fire and to prevent the deterioration of geotextile which occurs when exposing to sun light. ③ Relatively short reinforcement is used so as to be applicable to the reconstruction of embankment (Fig. 1). It is considered that the reduction in the stability of slope by using shorter reinforcement can be compensated by using a planar geotextile (Item ①) and using continuous rigid facing (Item ②). ④ A stage construction method (Fig. 13) is used. Namely, first the wall is constructed with the aid of gabions placed at each construction lift. After the major compression of the backfill and/or the supporting ground has occurred, the facing is placed by either of the following methods or another. (1) A lightly steel-reinforced concrete slab is placed direct-

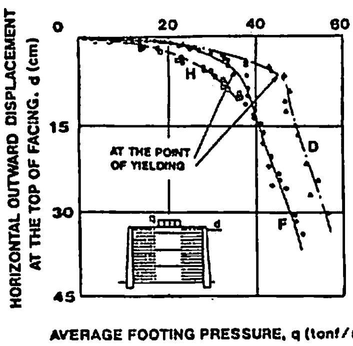


Fig. 12. Relationship between average footing pressure and horizontal outwards displacement of at the top of wall (JR No. 1)

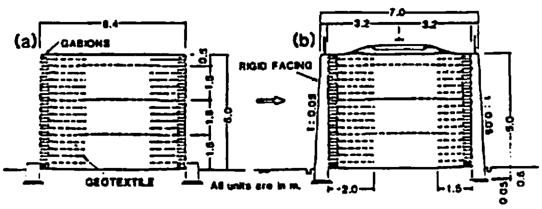
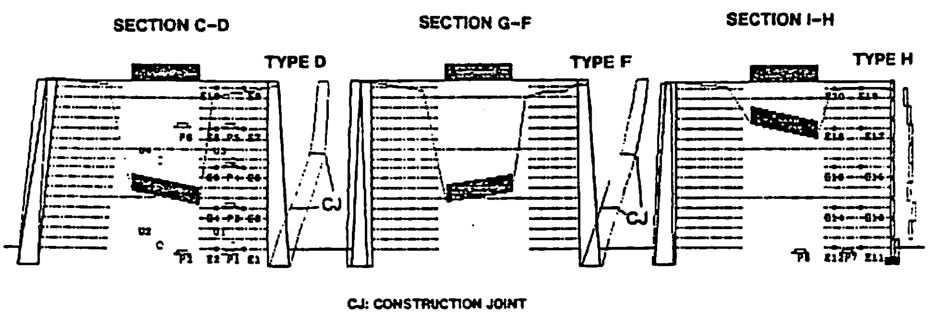


Fig. 13. Stage construction method



TYPE	D	F	H
SETTLEMENT OF FOOTING (cm)	69.4	68.3	29.8
FOOTING PRESSURE (tonf/m²)	54.8	50.2	36.0

Fig. 11. Deformation of test section by loading test

ly on the slope face wrapped around with geotextile sheets so that the slab does not separate from the slope face. (2) A precast continuous lightly steel-reinforced concrete plate is placed on the slope face, leaving some space between its rear and the slope face, which is subsequently filled with fresh concrete.

22. There are several other reinforcing methods which use continuous rigid facing. Some characteristic disadvantages when using it, however, are known as discussed below.

23. First, when the backfill is compacted using a fixed facing structure, only small amount of tensile strain may be developed in reinforcement, while larger earth pressure may be activated on the back face of facing. For the efficient usage of reinforcing members, however, larger tensile stress should be developed in them, with smaller earth pressure on the back face of facing. When the support of the facing is removed after the full height of wall is completed, tensile strain in reinforcement may increase by the outward movement of facing, but may not be much in the lower parts of wall since the facing would rotate about its bottom end. Further, in order to develop larger tensile strains in reinforcement, larger displacement of facing may be preferable. It would, however, be rather difficult to control and also it may induce some unpreferable relative settlement between the backfill and the facing. On the other hand, in the stage construction method, sufficient amount of tensile strain can be developed at the filling of backfill, without activating large earth pressure on the back face of facing.

24. Second, with the reinforcement members connected to the back face of continuous rigid facing when filling the backfill, the relative settlement between the facing and the backfill due to the compression of the backfill and/or the supporting ground may damage the connections of the reinforcing members with the back face of facing. To eliminate this problem, several methods have been developed; e.g., the reinforcing members are permitted to slide relative to the facing by means of slideable attachments. This problem, however, is not serious in the stage construction method.

25. In the stage construction method, there may be a limit of the height of wall because of the construction of wall without using a continuous rigid facing structure. While it is known that the limit is larger than 5.5 m, this issue is to be studied.

26. A series of shaking table tests of the scaled models also were performed for the a-seismic design. The design method including the stability analysis of wall and the structural design of facing are now being undertaken. Also, the construction of the wall of this type has been started at several sites for railway embankments.

CONCLUSIONS

27. The following points were found:

(1) A function of drainage of geotextile is essential when reinforcing a clay backfill.

(2) The use of continuous rigid facing is very effective to stabilize a reinforced wall and to reduce its deformation.

(3) The reinforcement can be relatively short (say, only 30 % the slope height) without losing its stability when a planar geotextile and continuous and rigid facing are used. However, the stability of wall may be more critical against the over-turning about the toe of wall than against that the translatory sliding-out.

(4) The reinforcing method proposed in this paper is very economical, particularly when used for a near-saturated cohesive soil.

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