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Roles of facings in reinforcing steep clay slopes with a non-woven geotextile

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ABSTRACT: Various roles of facing structures for stabilizing steep reinforced slopes of clay embankments are classified and defined. These roles are examined based on the behaviors of four full scale clay test embankments reinforced with relatively short non-woven geotextile sheets. The behavior clearly shows that a slope covered with a rigid facing structure can be very stable, whereas a slope covered with a flexible facing exhibits a relatively large deformation.

1 INTRODUCTION

Metal strips as used in the Reinforced Earth retaining walls are inadequate as a reinforcement for clays, because they lack the drainage function. For cohesive soils, planar geotextile sheets having a function of drainage such as non-woven geotextile sheets are adequate, despite their relatively lower tensile strength. Further, planar geotextile sheets have another advantage of a larger contact area with soil. Consequently, theirs ratio of the pull-out resistance to the tensile strength becomes much larger than that of metal strips. This results in a much smaller anchoring length needed for geotextile sheets than metal strips.

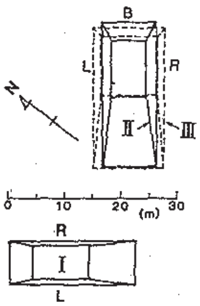
So far, four full-scale clay test embankments reinforced with a non-woven geotextile have been constructed by the authors (Tatsuoka and Yamauchi 1986, Tatsuoka et al. 1987, Yamauchi et al. 1987). Fig.1 illustrates three of them. Non-woven geotextile sheets were used as (1) a drainage material, (2) a tensile reinforcement, and (3) a material to facilitate better compaction. The length of non-woven geotextile sheets was made so short that some sheets placed at higher levels did not extend beyond the potential failure plane. It has been found from the long-term behavior and the behavior during artificial heavy rain-fall tests that steep clay slopes can be made very stable by using both relatively short non-woven geotextile sheets and an adequate facing structure.

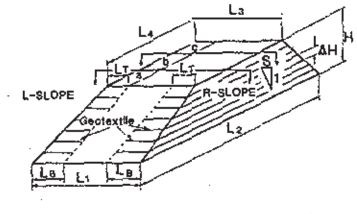
2 ROLES OF VARIOUS KINDS OF FACING RIGIDITY

When a short reinforcement is used for a steep slope, the following different kinds of facing rigidity, as illustrated in Fig.2, contribute to the stability in various ways (Tatsuoka et al. 1987).

When the facing is flexible without the local rigidity in the sense that large earth pressures are not activated on the back face of facing, the local compressional failure in soil near the facing tends to occur, in particular near the toe, as illustrated in Fig.3. This type of local failure can induce the three failure modes illustrated in Fig.4. Such type of failure has been observed for Test Embankment I as reported in details by Tatsuoka and Yamauchi (1986). In particular with a shorter reinforcement, the tendency of rotation of the reinforced zone about the toe can be larger.

The two slopes of Embankment II as shown in Fig.5 had partially the local rigidity (Type B-2 in Fig.2). When compared to the facing type B-2, the type B-1 has a worse degree of the local rigidity, whereas the type B-3 has a better degree. However, these facing types B-1, B-2 and B-3 lack the overall axial rigidity in the sense that a sufficient amount of the weight of back fill is not transmitted to the base ground through the facing. The facing type C has this function. However, the facing type C lacks the overall bending rigidity, thus such failure modes as illustrated in Fig.6 are not





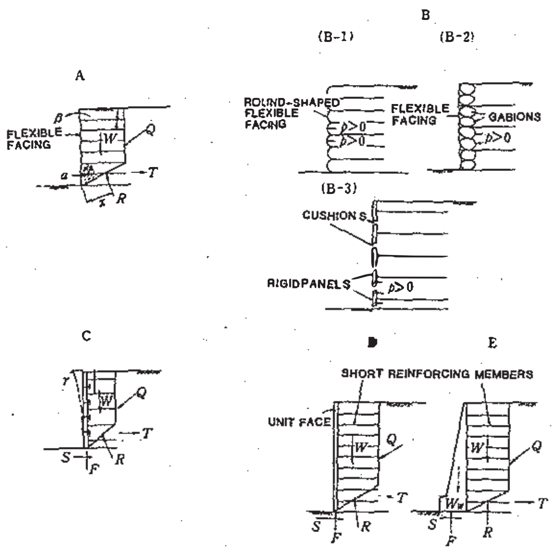
	Embankment I		Embankment II		Embankment III
Slope	R	L	R	L	R, L, B
L_B	2.0	2.0	2.8	3.8	2.5, 5.0
L_T	2.0	2.0	1.24	2.24	2.5, 5.0
L_1	7.6		10.0		12.2
L_2	22.0		23.4		21.15
L_3	8.0		6.48		10.0
L_4	10.0		12.0		10.55
H	4.0		5.2		5.5
ΔH	0.4	0.8	0.4	0.4	0.5
S	0.2		0.3		0.2
a	2.5		4.0		UNIT:m
b	5.0		4.0		
c	2.5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

MASS PER UNIT AREA OF NON-WOVEN GEOTEXTILE

(g/m²)	400[a)]	300[a)]	300[a)]

a.NOMINAL

Fig.1. Test clay embankments constructed at Chiba Experiment Station, Institute of Industrial Science, Univ. of Tokyo.



FUNCTION \ FACING TYPE	A	B-1	B-2	B-3	C	D	E
LOCAL RIGIDITY	×[1)]	△[2)]	□[3)]	○[4)]	○	○	○
OVERALL AXIAL RIGIDITY	×		×		○	○	○
OVERALL BENDING RIGIDITY	×		×		×	○	○
GRAVITY REGISTANCE	×		×		×	×	○

Note : 1) has not this function.
2) has this function only to a limited extent.
3) has this function to a large extent.
4) has this function sufficiently.

Fig.2. Illustration of various facing structures and their functions.

effectively restrained. The facing type D has this kind of rigidity.

When a long reinforcement is used, as illustrated in Fig.7, the reinforcement in each soil layer is designed so

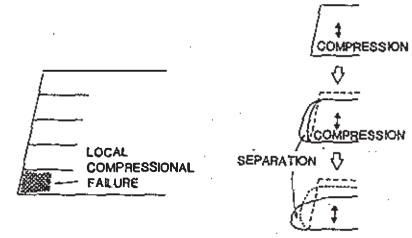


Fig.3. Local compressional failure of soil by using a flexible facing (Type A).

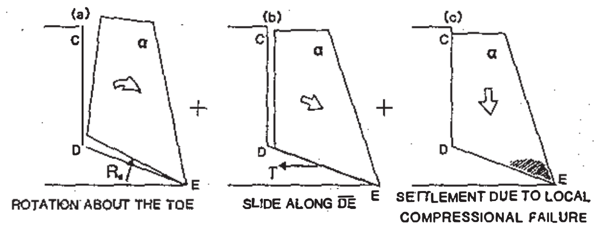


Fig.4. Three major failure modes of slope reinforced with a short reinforcement.

as to resist againt the earth pressure acting to each soil layer. However, in the reinforcing method using a short reinforcement and a type D facing, the reinforced zone together with the facing are expected to behave more or less as a monolith. The degree of resembling a monolith depends on the degree of the rigidity of facing and the degree of reinforcing.

The facing type E shown in Fig.2 has further a gravity resistance. This type of facing may be very practical. In order to avoid the damage to the reinforcement at the connection to the rigid facing during the filling of embankment, the following method is effective; i.e., a slope having a type B-2 facing is first constructed. After an initial settlement has sufficiently occurred, a rigid facing structure is placed on the existing slope surface. In fact, this method was adopted for a test slope of Test Embankmnet III.

3 CONSTRUCTION OF TEST EMBANKMENT III

Based on both the above consideration and the experiences with Test Embankments I and II and the other similar one constructed at another place (Kami-Onda Embankment, Tatsuoka et al., 1977), Test Embankment III having three test slopes with different facing structures was constructed in October

1986 (see Fig.8). A volcanic ash clay called Kanto loam was used. At filling, the average values of the water content w, the degree of saturation S_r, and the dry density γ_d were 110%, 85% and 0.6gf/cm^3, respectively. A spun-bond 100% polypropylene non-woven geotextile was used as reinforcing sheets as for Embankment Ⅱ. The force per unit width at 15% elogation $\alpha_{0.15}$ and those at peak α_f are a function of normal pressure σ_0(tf/m^2):i.e., $\alpha_{0.15}$(tf/m)=0.453+0.00675σ_0 and α_f(tf/m)=1.5+0.017σ_0 (Tatsuoka et al., 1987). The lowest two sheets were made longer as 5m in order to drain better the soil at the lower levels in the embankment. The top two sheets were also made longer as 5m, in order to prevent the possible development of cracks from the crest and also to increase the resistance against the overturning of the reinforced zone. A mass of crushed gravel was placed near the toe in order to collect water effectively from the interior of embankment and also to increase the resistance against overturning by increasing the strength of the toe of the slope. The three test slopes have three different facings as follows:

(1) Precast-concrete panel facing (as Type C in Fig.2). A facing structure, consisting of precast concrete panels, as shown in Fig.9, was placed on a foundation which had been made to avoid its settlement. This type of facing has been used also for Kami-Onda Embankment. Each panel has a dimension of 50cm×50cm×5cm and a weight of 34kgf for easier manual handling. Further, projections and grooves were provided at its four edge surfaces for easier connection to other panels. A 10cm-wide strip of non-woven geotextile reinforced with a polypropylene sheet having a high tensile stiffness was connected to each panel by means of a round steel bar and a sliding connection embedded in the panel. The reinforced strip geotextile has a yielding load of 1.18 tonf/m at a tensile strain of 4.7%. The sliding connections allow the settlement of the strip relative to the face. One end of the strip was heat-bonded to another part of the strip over 15cm to make a hoop to hook the steel bar. The strip was placed between two planar non-woven geotextile sheets so that eventually the geotextile sheets were to be connected to the concrete panels. Strains in geotextile sheets were measured only in this slope.

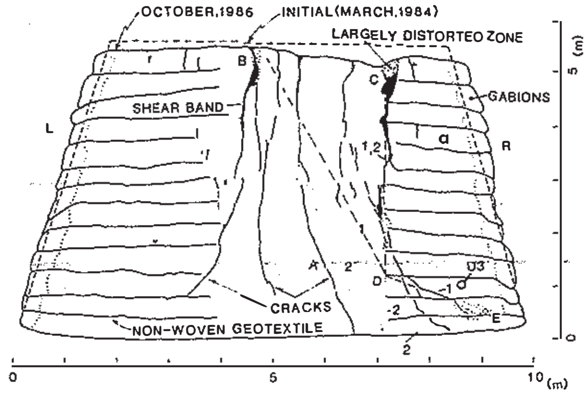


Fig.5. Cross-section of Embankment Ⅱ exposed one year after relatively large deformation by an artificial heavy rainfall test; Lines denoted by 1 and 2 are failure surfaces by the analyses of the limit equilibrium method with and without water pressures in cracks (Yamauchi et al., 1987)

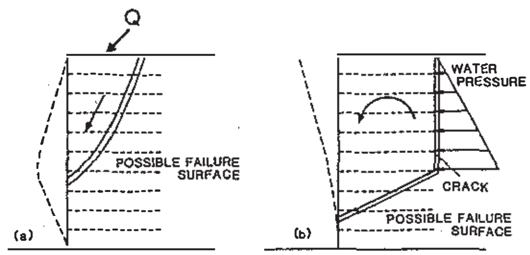


Fig.6. Possible failure mechanisms due to lack of overall rigidity in facing.

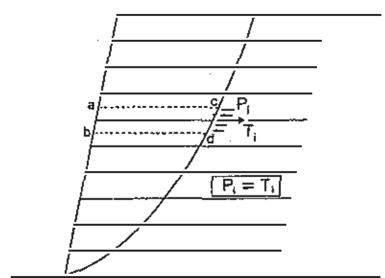


Fig.7. Assumed horizontal force equilibrium for a long reinforcement.

(2) Shotcrete skin facing (as Type D in Fig.2). After the slope had been completed by using gabions at the face as the slopes of Embankment Ⅱ (see Fig.5), a skin of shotcrete reinforced with wire mesh, having a thickness of about 8cm, was placed on the existing slope face as shown in Fig.8a. The shotcrete skin was anchored with 10cm-wide reinforced non-geotextile strips as mentioned above to the main body of the embankment. Drain holes were placed as needed. It was expected for

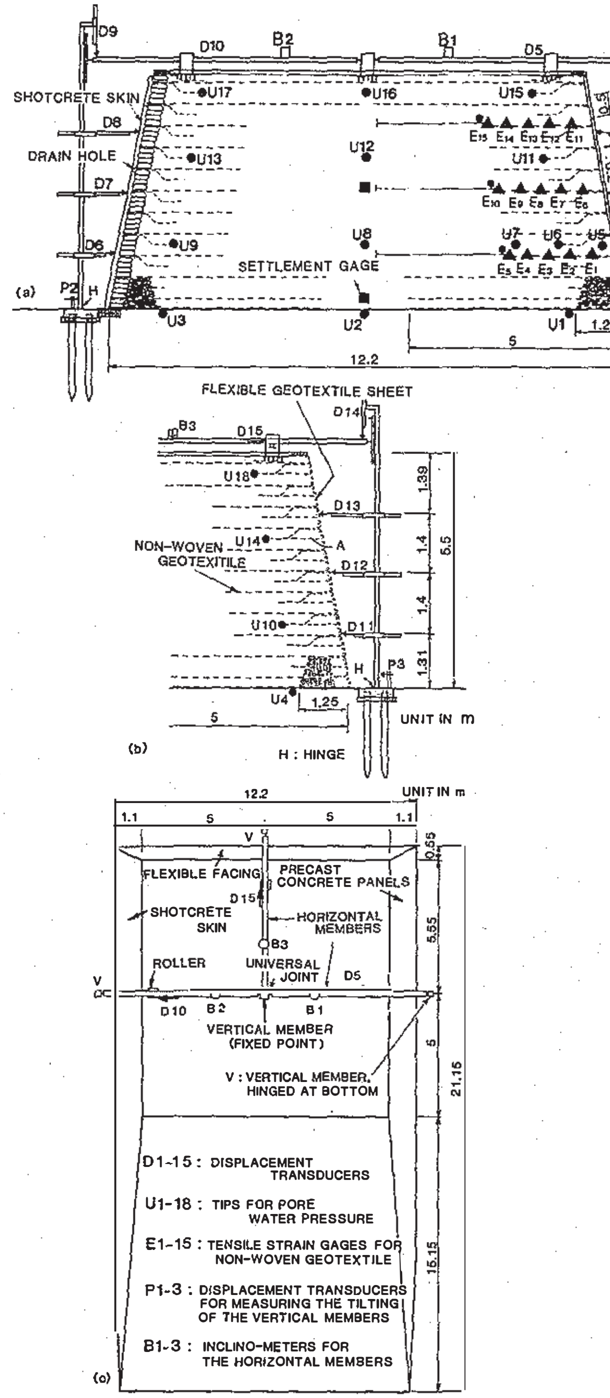


Fig.8. Cross-sections and plan of Embankment Ⅲ.

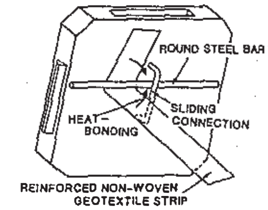


Fig.9. Precast concrete panel.

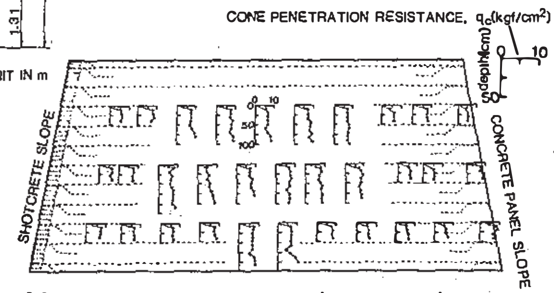


Fig.10. Cone penetration resistances of soils as compacted (Embankment Ⅲ).

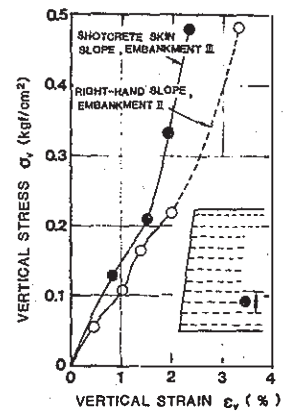


Fig.11. Compression of soil near slope face during filling.

the gabions to work as buffers for possible further relative settlements between the shotcrete skin and the embankment.

(3) Flexible facing (as Type A in Fig.2). The flat slope faces of soil were made and they were wrapped around with geotextile sheets (Fig.8b). No structural measures was used. A thin non-woven geotextile sheet having a nominal thickness of 1.5mm (which is half that of the non-woven geotextile used as the reinforcing material) and a length of 1m, as denoted by the letter A in Fig.8b, was placed horizontal at the intermediate height of each soil layer for better compaction of the soil near the face. This slope was expected to exhibit the worst performance among the three slopes, providing a good evidence for the importance of facing structural measures for such steep clay slopes.

Each soil layer was compacted by a heavy compaction plant with a weight of 12tf. A light compaction plant with a weight of 90kgf was used to compact the soil within about 80cm from the slope face. By this mechanical compaction, the soil near the face was well compacted, as seen from the result shown in Fig.10: i.e., the cone penetration resistances near the face were almost the same as those for the main body of the embankment. Fig.11 shows the relationship between the overburden and the vertical compressional strain at the slope face in each soil layer observed during filling in two slopes constructed using gabions, as indicated in the inset figure. The shape of the curve resembles the one under the one-dimensional compression of soil in the sense that the rate of axial compression decreases as the overburden increases. This implies that the geotextile restrained the horizontal tensile strains in the soil. It may also be seen that the vertical compression of soil near the face is smaller for this slope than that for Embankment Ⅱ, where manual compaction was employed near the slope face.

Fig.12 shows the tensile strains in the non-woven geotextile sheets for the concrete panel slope which occured during further filling over each soil layer. It may be seen that the geotextile functioned as a tensile reinforcement during filling. These data also suggest that earth pressures were activated at the back face of the facing, indicating that the precast concrete panels confined the back fill.

4 POST CONSTRUCTION BEHAVIOR OF EMBANKMENT Ⅲ

Fig.13 shows the time histories of the displacements at some representative points for a period from about two months after the construction. Fig.14 compares the displacements of the three slopes for about a year after the construction. Fig.15 shows the deformation of the flexible facing slope. It may be clearly seen from

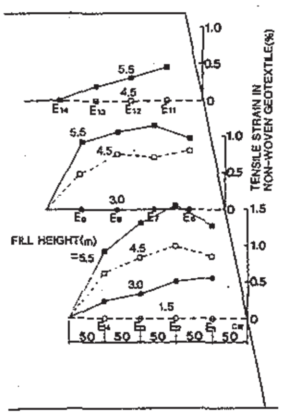


Fig.12. Tensile strains in geotextile sheets in concrete panel slope during filling (Embankment Ⅲ).

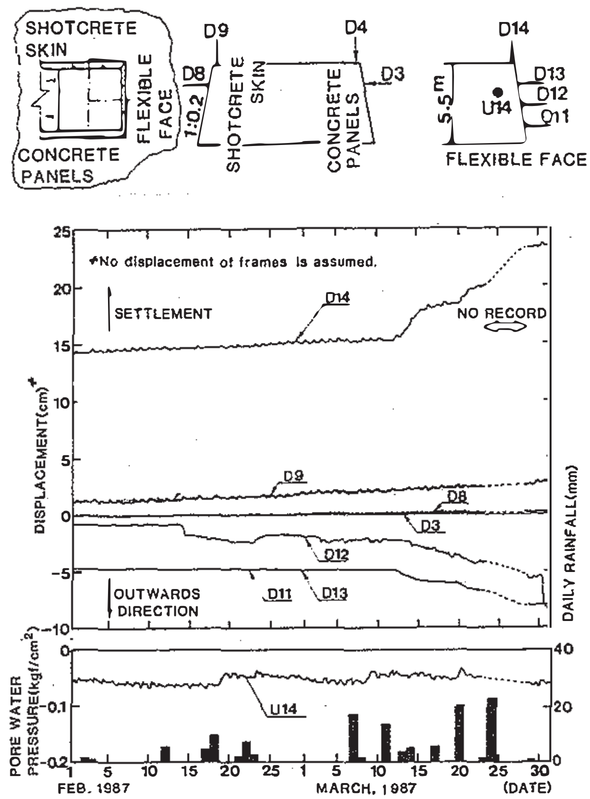


Fig.13. Behavior of Embankment Ⅲ during 1 Feb. ~ 31 March, 1987.

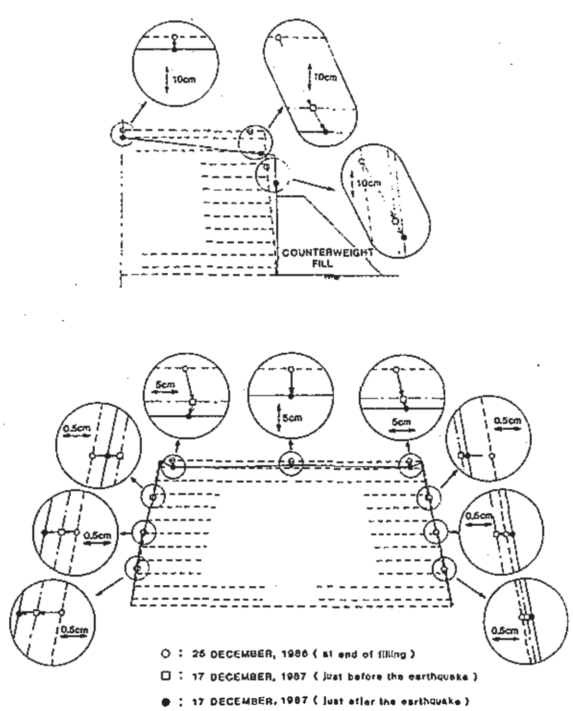


Fig.14. Total displacements at representative places at crest and slope faces, Embankment Ⅲ.

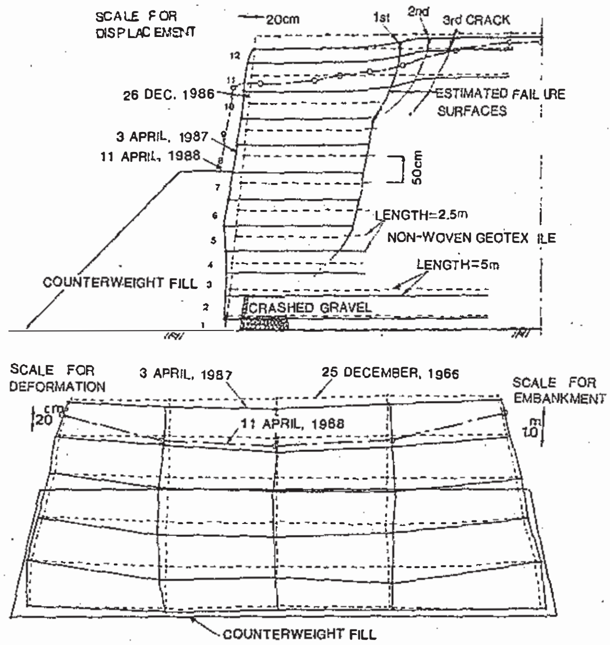


Fig.15. Deformation of flexible facing slope, Embankment Ⅲ (note that the scale for displacement is 2.5 times the scale for embankment).

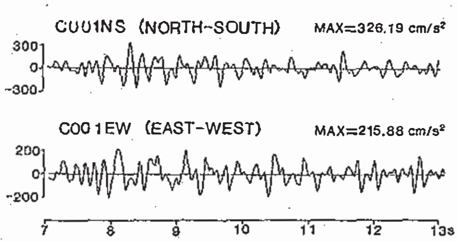


Fig.16. Earthquake motions at 1m-underground on 17 Dec. 1987, recorded at a distance of 210m from Embankment Ⅲ.

these results that the displacements of the flexible facing slope were much larger than those of the other two slopes, the one having gabions and a shotcrete skin and the other one having precast concrete panels. It may also be seen from Fig.13 that the displacements at the flexible facing slope were induced by rainfall. To avoid too large deformation of the flexible facing slope, a counterweight fill as shown in Fig.14 and Fig.15b was constructed in May, 1987.

The total rainfall for thirteen months after the construction to the end of 1987 is 1400mm. For this amount of rainfall, the displacements at the two rigid facing slopes were very small. In particular, it is to be noted that the horizontal outwards displacements near the crest (D3 and D8), which are the good indicators of the instability of the steep slopes, were remarkedly small.

The test embankment experienced a relatively large earthquake motion on 17 Dec. 1987 (Fig.15). The deformation by the earthquake was the largest also for the flexible facing slope despite the counterweight fill (see Fig.14), and the deformations of the other two slopes were very small. This behavior suggests that also for increasing the resistance against seismic effects, the use of facing structures having various kinds of rigidity is effective.

From the field observation, a clear difference was not found in the behavior of the two slopes with facings having some degrees of rigidity. However, a clear difference has been observed between these two facing types C and D in the laboratory model tests in which the model slopes were brought to failure by loading them at their crests. The results will be reported in the future.

5 CONCLUSIONS

The behavior of full-scale test embankments of clay reinforced with a non-woven geotextile during natural rainfall, an artificial heavy rainfall test and a relatively strong earthquake motion shows that the facing structure having various kinds of rigidities should be used for increasing the stability of the steep slopes. These various kinds of rigidity can be classified into the local rigidity, the overall axial rigidity and the overall bending rigidity.

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