Earth Reinforcement, Ochiai, Yasufuku & Omine (eds) © 1996 Balkema, Rotterdam. ISBN 90 5410 833 9

In situ failure test of high water content soft clay embankments reinforced by GHDs

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ABSTRACT: To apply newly developed geosynthetic horizontal drains (GHDs) to reinforced embankment, full scale in situ failure tests were carried out. The GHDs consist of geosynthetic materials which have improved fabric parameters for permeability and reliable strength. Four kinds of GHDs were selected and a very soft clay with a high water content was used for the filling material. In this paper, the properties of GHDs used were introduced and the design procedure for the reinforced embankment including drainage by GHDs was discussed. Highly critical values for safety of the embankments were selected. The filling frame was made with sandy soil and the soft clay soil embankments was designed to be 3m high with a 1:0.5 slope angle.

Execution steps of the embankment and failure patterns of the test embankments are summarized. The induced excess pore water pressure and deformation of the embankment, etc. were carefully monitored during the embankment works. The results obtained reveal that the assumed critical safety factor of the slope closely coincided with the limiting loading height and the GHDs provide sufficient reinforcement.

1 INTRODUCTION

Due to the reduction of disposal areas and with consideration of environmental preservation, good use of inferior quality soil such as surplus soil from construction is needed. When soft clay of low strength is used as a filling material for embankments, the filling is expected to increase in strength owing to consolidation by dewatering effects. Therefore, we can expect to develop into a reinforcing material that has not only superior strength but also transmissivity.

Recently, the reinforced embankment using geosynthetic materials has been used as one of the general construction methods for stabilizing embankments. Despite the good mechanical properties of the geogrid, which is a typical reinforcing material, it had been suited to reinforced embankments filled by sandy soils only because of lack of drainage. This paper describes the results of drainage studies involving newly developed GHDs examined by in situ failure test of full scale reinforced embankments filled using soft clay with a high water content.

2 BASIC PROPERTIES OF GHDS

Fifteen kinds of GHDs which had sufficient strength and transmissivity were proposed for the construction of reinforced embankment (Kamon et al, 1994). Almost all of these GHDs were newly developed and they were evaluated for mechanical characteristics, pull-out strength and transmissivity. Four materials were selected from the fifteen GHDs

Table 1 Basic properties of GHDs

Materials	Thickness (mm)	Unit density (g/m^2)	Tensile strength (kN/m)	Elongation (%)	In-plate permeability (cm/s) Normal stress at 98.0kPa	Normal stress at 294.0kPa
Nonwoven fabric	5.6	454	41.0	82.5	1.6×10^{-1}	6.2×10^{-2}
Reinforced nonwoven fabric	8.7	1581	72.9	11.4	3.2×10^{-1}	1.0×10^{-1}
Warp knitted fabric	4.9	708	79.9	18.1	9.4×10^{-1}	3.1×10^{-1}
Plastic core covered by nonwoven fabric	3.6	1636	82.8	32.1	1.6×10^{1}	1.6×10^{1}

after considering their functions and characteristics and they were applied to the slope failure test of reinforced embankments.

The basic material properties of these GHDs are summarized in Table 1, and cross-sections are shown in Fig. 1. They are classified as (1)nonwoven fabric (commonly used), (2)reinforced nonwoven fabric (reinforced with high tensile strength yarn), (3)warp knitted fabric and (4)plastic core covered by nonwoven fabric (improved mechanical characteristics). The width of GHDs (2) to (4) was 0.3m each.

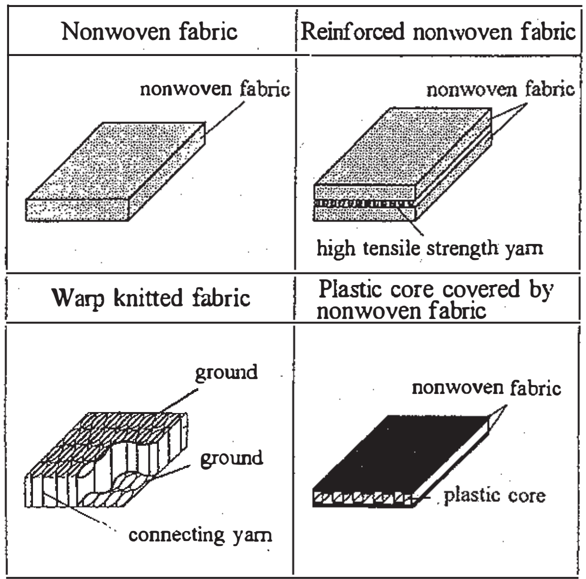
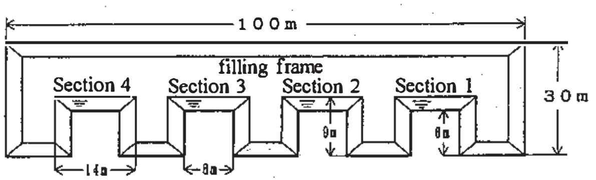


Fig. 1 Cross-section of GHDs



Section 1 : nonwoven fabric
Section 2 : reinforced nonwoven fabric
Section 3 : warp knitted fabric
Section 4 : plastic core covered by nonwoven fabric

Fig. 2 Filling frame of test embankment site

3 DESIGN OF SOFT CLAY EMBANKMENT REINFORCED BY GHDS

In situ failure tests of reinforced embankments were carried out to evaluate the applicability of GHDs. Soft clay (alluvial clay) with a high water content was used as filling material for the embankment. It was picked up at a disposal area in the Sakai 7-3 industrial disposal area of Osaka prefecture. The physical properties of the soil are indicated in Table 2. The water content was approximately 85% and equal to its liquid limit.

The embankments in four separated sections were 3m high with a 1:0.5 slope angle. It was too difficult to fill the embankment using ordinary methods of construction because the soil was extremely soft. Therefore, a filling frame was made with sandy soil and soft clay filling was added to the area between frames as shown in Fig. 2. Filling performed layer by layer (the thickness of each layer was 0.3m) and the surface of slope was protected by sandbags together with a nonwoven fabric (200g/m^2). To increase the strength of the soil owing to consolidation, only two layers were added each day. After the 8th layer of filling work was completed, the embankment was allowed to settle for two days, then the 9th and 10th layers were added. The embankments were left for a certain period until their degree of consolidation reached 90% or more before further loading was added. Nonwoven fabric reinforcement was laid over the whole surface of the embankment at Section 1 and 0.3m wide GHDs were placed over 50% of the embankment with staggered arrangement at Sections 2, 3 and 4 as shown in Fig. 3. For the design, it was considered that the soft clay embankment was supported by 5m long GHDs and additional loading for the failure test was supported by 10m long GHDs as indicated at the cross-section of Fig. 3. The design safety factor of the reinforced embankment was fixed at 1.0 for the filling of soft clay and 1.0 for the failure test of embankment with 1.5 meter thick additional loading modified by the Japanese design manual (1992).

The consolidation effects by GHDs were considered one-dimensionally in the design. The cohesion of the soft clay filling was assumed as follows; The embankment load (deadweight): height of embankment h(m) which corresponded to the initial cohesion (c_0:4.2kPa) was regarded as the consolidation yield stress, and the increase of cohesion was postulated when the deadweight exceeded that value.

Table 2 Physical properties of filling material used

Unit weight γ_t (kN/m^3)	Internal friction angle ϕ(°)	Cohesion c_u(kPa)	Rate of strength increase α(c_u/p)	Coefficient of consolidation c_v(cm^2/day)
14.7	0	4.2	0.3	40

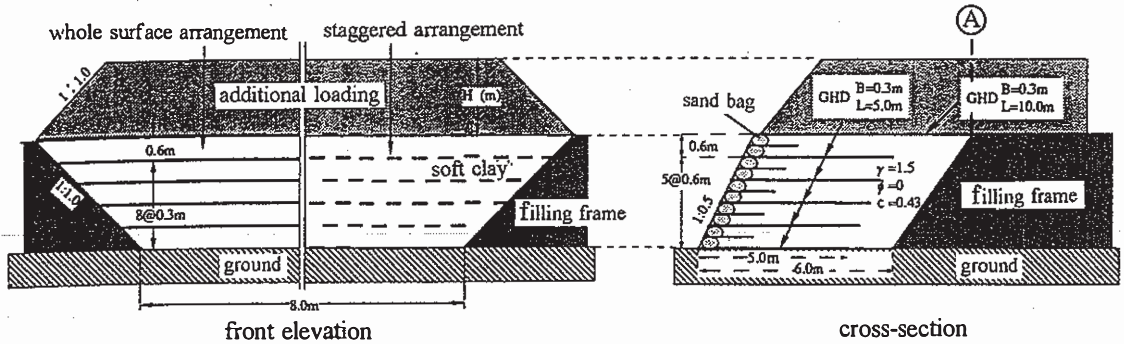


Fig. 3 Section of in situ failure test embankment

The height of the embankment was calculated by equation (1):

$$h = c_0\gamma/\alpha = 4.2/0.3 \times 14.7 = 1.0\ (m) \qquad (1)$$

where; α: Strength increasing ratio (0.3), γ: Unit weight of soil (14.7kN/m^3)

Therefore, the strength increase due to consolidation at the position from 1m depth to the bottom of the embankment was calculated by equation (2):

$$c = c_0 + U \cdot \alpha \cdot \gamma \cdot (H - h) \qquad (2)$$

where; U: Degree of consolidation, H: Total height of embankment

The stability of the embankment was calculated according to the following rotational slip method (equation (3)).

$$Fs = \frac{R \cdot \Sigma\{ci \cdot li + (Wi \cdot \cos\theta i + Ti \cdot \sin\theta i) \cdot \tan\phi i + Ti \cdot \cos\theta i\}}{R \cdot \Sigma(Wi \cdot \sin\theta i)} \qquad (3)$$

where; ci: Cohesion of soil (kPa), wi: Clod weight,

ϕi: Angle of internal friction (degree), Ti: Tensile strength of material (kN/m)

The smaller value between the ultimate tensile strength T_a and the pull out resistance T_{pi} of GHDs. T_{pi} was calculated by equation (4):

$$T_{pi} = 2 \cdot (0.5ci + 1.0 \cdot \sigma_v \cdot \tan\phi i) \cdot L_{ei} \qquad (4)$$

where; σ_v: Vertical earth pressure which works upon the anchorage part of material at each layer, L_{ei}: Anchorage length of the material at each layer, Fs: Safety factor, R: Radius of the rotational slip surface, li: Arch length of the slip surface which was divided into sections, θi: Sliding angle at the center point of the slip surface which was divided into sections

4 EXECUTION OF SOFT CLAY EMBANKMENT

To prevent the outflow and/or inflow of water during the measurement of transmissivity, the side slopes and bottom edges of filling frames were covered by impermeable sheets as shown in Photo 1. The sampling of soil to measure the water content, portable cone penetration test and mini-vane shear test were applied to every layer of filling to investige the status of the embankments. Laser

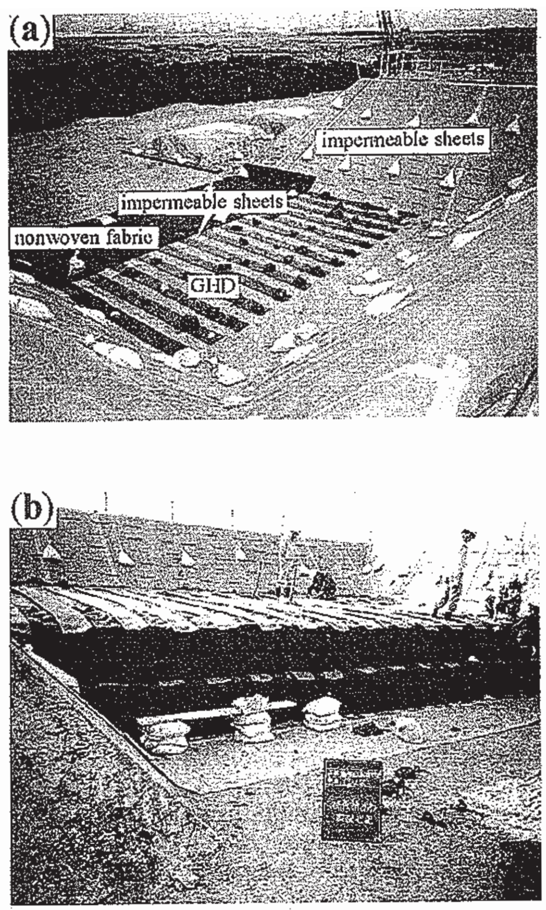
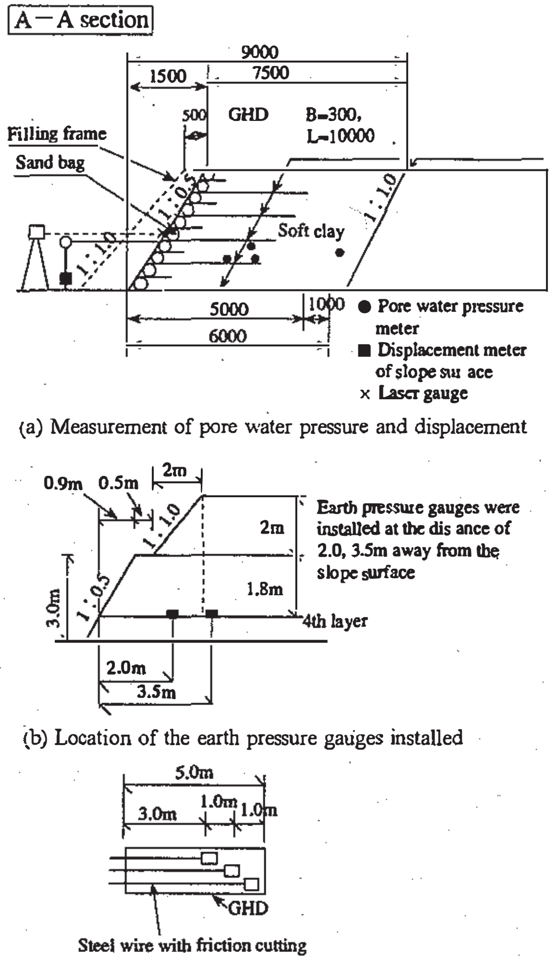


Photo 1 Filling work in the filling frame

distance meters, pore water pressure meters, earth pressure gauges, relative displacement meters and pipe strain meters were installed in the embankments in order to certify deformation of the soft clay and the status of the GHDs themselves. These monitoring instruments with automatic measurement systems were mainly installed on the slope and around expected sliding surfaces. The basic arrangement of measuring instruments actually installed is indicated in Fig. 4.



(a) Measurement of pore water pressure and displacement

(b) Location of the earth pressure gauges installed

(c) Measurement of elongation of GHD

Fig. 4 Monitoring system of the embankment

The filling works were carried out smoothly in Sections 1 and 2. However, in Sections 3 and 4, the slope displacement increased during filling of the 8th layer, and especially in Section 4 cracks appeared on the crown of embankment (at a point approximate 5m from the slope shoulder) immediately after completion of the 8th layer. Therefore, filling was stopped at the 8th layer due to the visibility of cracks at the crown. Photo 2 shows the embankment after the completion of filling in Section 2.

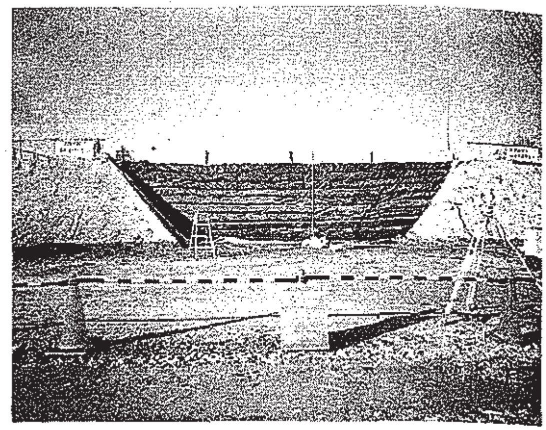


Photo 2 Completed filling in Section 2

5 RESULTS OF FAILURE TESTS

After the completion of filling works, 10m long GHDs were placed at the crown of each embankment (on top of the 10th layer) and the failure tests were conducted by the additional loading of sandy soil. In Section 4 after completing the 0.6m thick filling from the 8th to the 10th layer using sandy soil, 10m long GHDs were set up in the same way. This 0.6m thick sand layer was considered a part of the additional loading.

The failure of test embankments in each section occurred under different thicknesses of additional load. In Section 1, where nonwoven fabric reinforcement was laid over the whole surface of embankment, displacement occurred across the whole slope as if it was pushed out forward and collapsed after the second layer covered the first layer. In Sections 2 to 4 where GHDs were set up using a staggered arrangement, the center part of the slope was pushed out as the thickness of the additional load increased and collapsed after the higher layer covered the lower layer. Afterwards, cracks occurred at the wrapped part of the covering

Photo 3 Failure pattern in Section 1

Table 3 Additional loading and the height of the main scarp in each Section

Sections	Thickness of additional loading	Height of main scarp
1	2.5m(failure occurred during the filling)	1.5~2.0m
2	2.0m(failure occurred after the filling)	1.25m
3	2.0m(failure occurred during the filling)	0.8m
4	3.1m(failure occurred after the filling)	1.2m

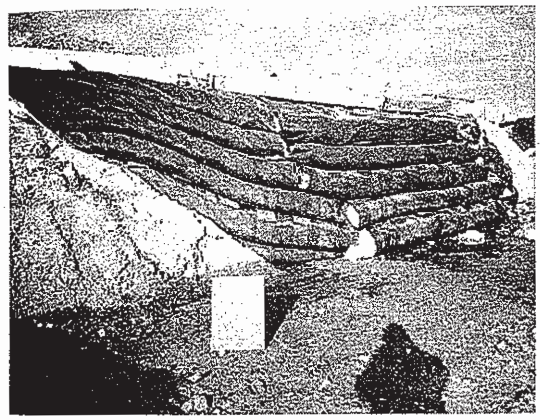


Photo 4 Failure pattern in Section 2

material. Photos 3 and 4 show the failure patterns of test embankments in Sections 1 and 2. On the surface of the additional load in all Sections after the collapse, the main scarps appeared at cross section Ⓐ as indicated in Fig. 3. The thickness of additional load and the height of the main scarp are summarized in Table 3. The deformation of 10m long GHDs which were placed on top of the 10th layer was observed in all Sections after the additional loading was removed when the failure test was completed. By this observation, it was confirmed that GHDs were broken in Sections 1, 3 and 4 at the filling frame right under position Ⓐ as indicated in Fig. 3. GHDs in Section 2 were pulled out without any breakage.

Figure 5 indicates all the results of measurement of excess pore water pressure which were obtained in soft clay and were attached to GHDs. The pore water pressure meter attached to GHDs was measured at an approximate maximum of 19.6kPa pore pressure, and maintained a definite and unchanging value with more than 29.4kPa of additional loading. In comparison with these values, pore water pressure in soft clay did not dissipate due to that approximate 70 to 80% of earth pressure. However, pore water pressure in soft clay between GHDs demonstrated values similar to approximately 50 to 60% of the value of earth pressure. The differences in pore water pressure between the two areas can be considered due to the drainage by the GHDs.

*G shows the sensors attached the surface of GHD .
*C shows the sensors installed in the clay layer
*2, 3.5, 5 mean the distance from the surface of slope.
Unit; m

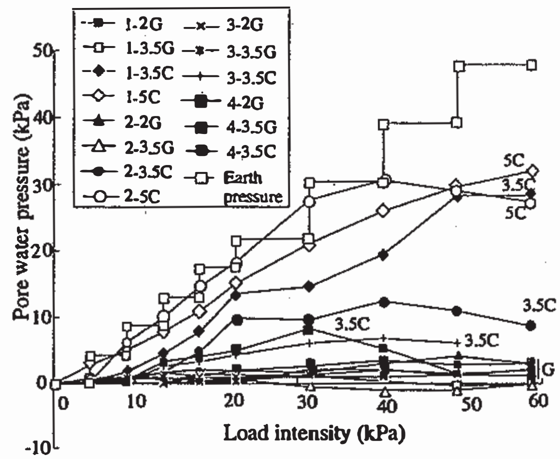


Fig. 5 Behavior of pore water pressure

Figure 6 indicates a change over elapsed time in the relative displacement around sliding surfaces and horizontal displacement at the slope of the embankment. Although the horizontal displacement at the slope came to more than 1m, the elongation of GHD showed only 1 to 2cm of relative displacement. This result clearly indicates the

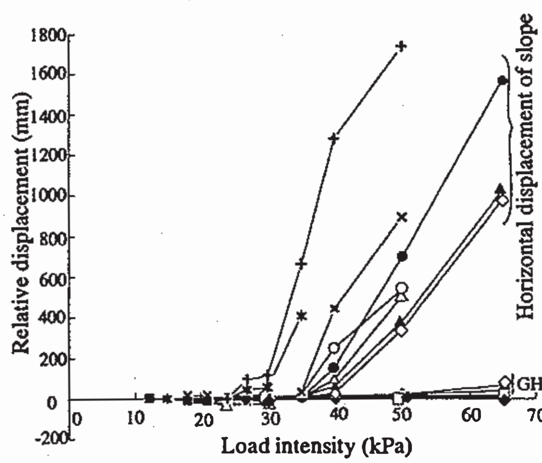


Fig. 6 Relative displacement of embankments

sliding surfaces in soft clay where outside of the GHD area as expected in our design.

Figure 7 shows the strength increase in the clay due to GHDs. Cohesion (c_u) at the time of filling was assumed to be 2.9 to 4.9kPa, and these values were slightly larger at the upper part of the embankment because of the lower water content. The value of c_u after the failure test became 9.8 to 21.6kPa and was relatively small at the lowest layers in Sections 3 and 4 although the cu was calculated as 14.7kPa based upon the strength increasing ratio ($c_u/p=0.3$). This finding explains the relatively large deformation in Sections 3 and 4.

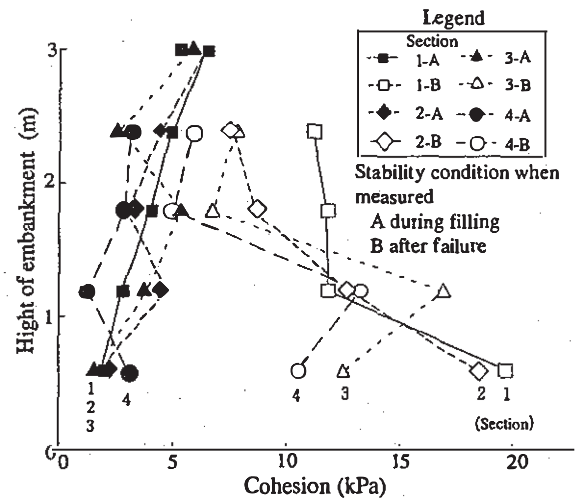


Fig. 7 Strength increase promoted by GHDs

6 CONCLUSIONS

Newly developed GHDs, that showed sufficient tensile strength and transmissivity, were tested in situ. Full-scale reinforced embankments filled by soft clay with a high water content were designed under critical safety condition and slope failure tests were carried out. The reinforcing and drainage effects of GHDs can be estimated according to these test behaviors. From the results obtained, GHDs can be used practically in designing reinforced embankments filled using very soft clay which is usually disposed of as construction waste.

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