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Effect evaluation for the geocomposite reinforced embankment of cohesive soil

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ABSTRACT: In recently years, the low quality soft clay from construction site increases rapidly, due to the development of urban and underground space. Combining the geosynthetics reinforcement with this surplus soil is one of the effective recycling methods. Considering a steep slope banking with high water content and low quality soil, the reinforcement effect of geocomposite is evaluated by using the finite difference analysis. In this study, it is clarified that the reinforcement mechanics of geocomposite for the improvement on the tension and drainage capability of embankment, and also some suggestions for the design of geocomposite reinforced embankment are proposed.

1 INTRODUCTION

Recently, the surplus soils, especially the low quality soft clay from construction sites, rapidly increase due to the development of urban area and underground space. In the metropolitan area, about 120 million m^3 surplus soils are generated per year. 70% among them are high water content and low quality soil such as silt, Kanto Loam and other clays, which are difficult to be disposed properly and economically. On the other hand, environment protection has attracted more and more society concern and become an indispensable issue for the construction industry. Therefore, it raises an important problem to reuse such low quality surplus soils properly.

Combining with geosynthetics reinforcement, the surplus soils can be reused as an embankment material. The reinforcement material, geocomposite can greatly improve the tension strength and drainage capability of the embankment, which makes a steep slope embankment feasible and more economic. In *Design and Construction Manual for Geotextile Reinforced Embankment* (Civil Engineering Research Center, 1993), the reinforcement evaluations for drainage and tension are introduced, however, there is no methodology that can take both the drainage and tension-reinforcement effects into account.

Saving the issues mentioned above, a steep slope embankment with high water content and low quality soil is applied for evaluating the reinforcement effect of geocomposite by using the finite difference analysis. Some suggestions for the design of geocomposite-reinforced embankment are also proposed in this study.

2 STABILITY ANALYSIS OF VOLCANIC ASH CLAY BANKING

2.1 *The outline of analyses model and method*

Using the finite difference analysis (simulation codes: $FLAC^{3D}$), several cases with different reinforcement layers, embankment heights and consolidation periods, as list in Table 1, are studied. The constitutive law employed for the banking soil, i.e. the Kisarazu Kanto loam with 98% water content, is the Mohr-Coulomb model. The geocomposite TRF-31 (abbreviated as GC hereafter) is selected as the reinforcement material, and the drainage and tension strength are focused in these case studies. The embankment is supported by a drainage foundation with enough loading capability.

The properties of the banking material and the reinforcement material that employed in these studies are listed in Table 2, which are partly referenced

Table 1. The study cases.

Height of embankment (m)	8	12	16
Interval of laying reinforced material (cm)	No reinforcement(N), 45(GC45 cm) 90(GC90 cm)		
Speed of work (hour)	240, 480, 720	480, 720	720, 960

Table 2. The properties of banking soil and GC.

Value of banking material (kanto lome)
Volumetric elastic coefficient bulk modulus: K(kPa) = 500
Cohesion: c (kPa) = 19.6
Density: ρ(g/cm^3) = 1.366
Angle of internal friction: ϕ_{cu}(deg) = Formula (1)
Dilatancy angle (deg) = 0
Limit of tensile stress: σ'(kPa) = 19.6
Value of geocomposite
Cohesion of interface: c_{cus}(kPa) = 4.41
Angle of internal friction of interface: ϕ_{cus}(deg) = Formula (2)
Rotation of elastic modulus: (kPa) = 28000

to the former researcher's studies (Y.Tanabashi & H.Nagashima, 2002). The strength characteristic in each layer differs from each other, since they are banked step by step at different periods. Through a series of constant volume simple shear tests, an empirical equation (1) is proposed to generalize the relation between the friction angle of each banking layer and its consolidation period. For the same method, the friction angle of GC-soil interface is formulated as empirical equation (2). As for the cohesion, it can be regarded as a constant during all consolidation periods.

$$\phi_{cu}(t_c) = 20.2 - \frac{1}{\exp\left(-3.00 + 0.941\sqrt{t_c}\right)} \quad (1)$$

$$\phi_{cus} = 22.0 - \frac{1}{\exp\left(-3.09 + 0.944\sqrt{t_c}\right)} \quad (2)$$

Here, t_c is the consolidation period (unit is hour), ϕ_{cu} and ϕ_{cus} are the friction angles of the banking material and the GC-soil interface, respectively.

The membrane element is selected to simulate the GC, since they cannot subject to bending moment. And the tension capability of GC is set to 40% of its designation capability due to the embankment's creep effect during its lifetime. The consolidation period for each layer is assumed to be same, as illustrated in Fig. 1. After banking, a footing loading is loaded at every 5–10 kPa step to the crown surface of the embankment, as illustrated in Fig. 2, and the embankment's deformation behavior is evaluated in these studies.

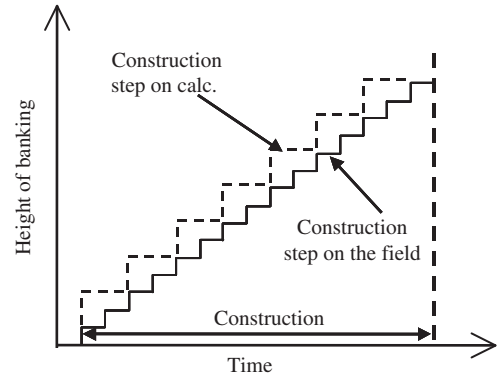


Figure 1. The relationship between construction process and construction period of the banking.

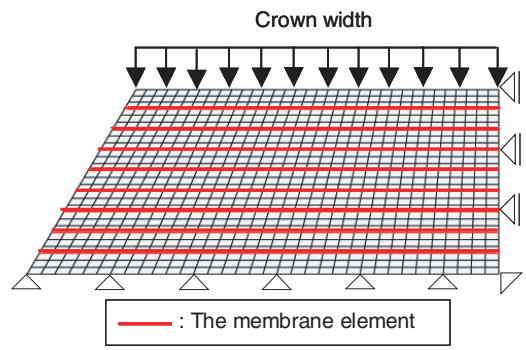


Figure 2. Analysis model.

2.2 *Conversion for the banking consolidation period*

The conversion consolidation time was introduced in order to handle the condition of the consolidation in the banking as a result of different layers. Corresponding to different consolidation coefficient and consolidation period at every 90 cm/layer, the frictional angle of Kanto loam, as well as the friction angle of GC-soil interface, was evaluated. From layer 1, the subsequent layers were heaped step by step, and actual consolidation period for every layer was calculated. In other words, various stress history is converted into a unique stress history, then the properties employed in each layer can be calculated according to its consolidation period. Some constants used for consolidation period conversion could be obtained from simple shear tests.

$$k = 0.0042(m/\min)$$
$$c_v = 8.25\times10^{-6} + 2.72\times10^{-6}\ln(\sigma_c/9.8)(m^2/\min)$$

where k is coefficient permeability, c_v is coefficient consolidation and σ_c is the consolidation pressure.

In the GC45 cm case, a unique c_v is used for every two soil layers. And the conversion of the time factor to

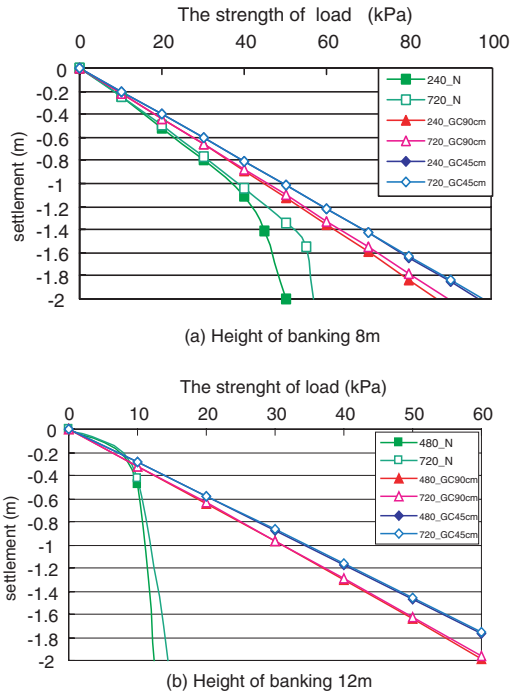


Figure 3. Loading – settlement curves.

real time is required, since strength characteristics of soil and the frictional property between GC (made of no woven fabric commonly) and soil are experimental parameter. They are related to the consolidation degree of soil. Then in a real ground or a test specimen, a unique consolidation degree corresponds to a unique consolidation period. Therefore, they can be converted interchangeably by using the Terzaghi's square law, and the conversion is carried out in the real time.

3 ANALYSIS RESULTS AND DISSCUSIONS

3.1 *Loading-settlement curve*

Loading-settlement curves of banking height of 8 m and 12 m are shown in Fig. 3, respectively. In the cases without geocomposite reinforcement, there is the depression effect of the settlement by lengthening the construction period, as shown in Fig. 3a. It is proven that the loading capability of embankment increases with the consolidation degree. In the 720_N case, the settlement developed rapidly after it was subjected to a surface loading of 50 kPa. It can be inferred that there is a limit in the strengthening effect, even if the construction period is lengthened in the non-reinforced banking.

In the cases with GC reinforcement, the loading-settlement curves are almost the same with those

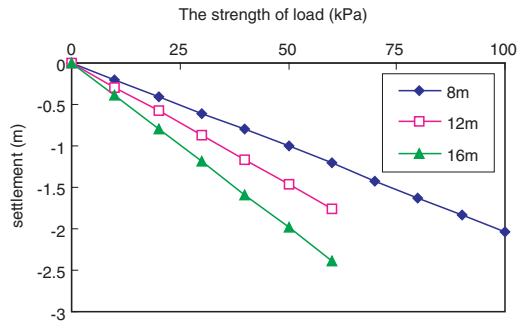


Figure 4. Crown settlements on each embankment heights.

without reinforcement before a loading threshold of 30 kPa. And the slope of loading-settlement curve remained constant even the embankment was subjected to a load of 50 kPa.

It can be inferred from Fig. 3b that the embankment tends to fail after the 10 kPa loading on the cases with reinforcement. From this result, near 12 m seems to be the limitative banking height.

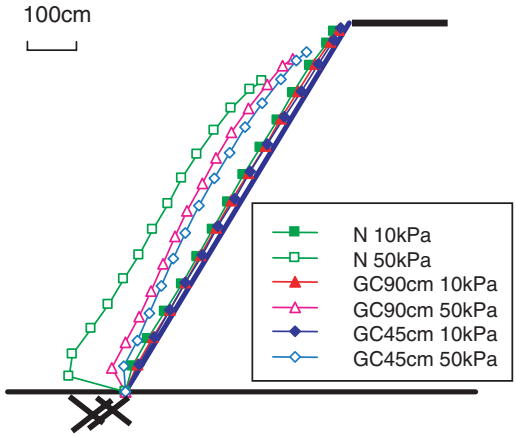
In cases of GC90 cm and GC45 cm, the difference between the reinforcement effects of the settlement is remarkably observed in comparison with those cases with a banking height of 8 m. It can be inferred that there is the strengthening effect from the case in which the load strength is smaller, as the banking rises. The effect of consolidation period is not remarkable, as illustrated in both Fig. 3a and Fig. 3b.

The crown settlements of different height cases with GC45 cm reinforcement are illustrated in Fig. 4. At a same loading, the settlement of the higher embankment is larger.

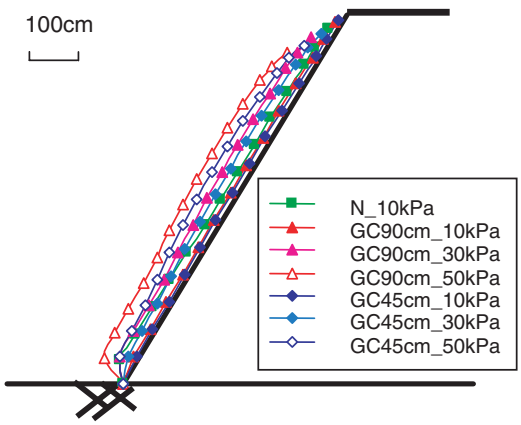
3.2 *Load strength-deformation of slope*

The curves of loading versus slope deformation for the case with an embankment height of 8 m and 16 m are shown in Fig. 5. In the cases without reinforcement, a large deformation occurred under a surface load of 50 kPa. In the cases with GC90 cm or GC45 cm reinforcement, the horizontal displacement of the slope is suppressed about 50∼65%, the crown settlement is also suppressed about 50∼65% comparing to those cases without reinforcement. Moreover, there is no large deformation over the foot of the embankment, and it can be inferred that the whole embankment is stable. In the cases without reinforcement, large displacement occurred at the foot of the embankment, while in the cases with reinforcement, the displacement at the middle slope is the largest, which is quite differed from the former ones.

The analyses results of embankment with the height of 12 m are shown in Fig. 5b. In cases without reinforcement, the embankment failed under the surface



(a) Height of banking 8m (720hours)



(b) Height of banking 12m (720hours)

Figure 5. Load strength-deformations of slope.

load of 15∼20 kPa. Therefore, there were no more displacement records in following the deformation of the slope. In the cases with reinforcement, it is effective to restrain the deformation by the difference between the laying intervals. In case GC45 cm, the deformation is small, and the deformation at the foot of the embankment is restrained apparently comparing with the case of laying interval 90 cm. The tendency that the stress is concentrated at the foot of slope is observed for the embankment. It seems that the stress concentration at the foot of embankment could be reduced by increasing the laying interval dense. Without a rapid collapse the 12 m high embankments with GC reinforcement can be regarded as stable under a maximum surface loading of 50 kPa.

3.3 *Shear failure region and displacement vector*

The displacement vector and the shear failure region distribution of the 8 m high embankments after a construction period 720 hours are shown in Fig. 6, where the difference can be confirmed from the displacement vector figures. In the case without reinforcement, excessive deformation and plasticity occurred near the slope at the load strength of 50 kPa. Then the embankment failed along this sliding surface. The shear fracture region figure is also depicted in this figure. In comparison with cases of GC90 cm and 45 cm, it is shown that the displacement of the whole embankment is suppressed in the GC45 cm case. In both cases with reinforcement, the displacement at the middle and the foot of embankment is restrained, due to the effect of GC reinforcement.

There is a small failure region at the foot of slope on both cases. The stress concentrates in this part subject to the effect of the load, and the destruction seems to have locally been generated. However, there is no progress of the breakdown region to banking upper part. It is considered that the reinforcement near the foot of slope must be sufficiently considered from this result, when the banking is constructed in the field.

3.4 *Tensile stress the reinforcement material*

The resultant tensile stress in the reinforcement after construction period 720 hours is shown in Fig. 7. The legend is the distance from the embankment bottom. It is proven that the stress is also concentrating each GC near 4∼6 m from the slope, which indicates the position of the sliding surface. Especially, a larger tensile stress occurred near 270∼540 cm from the embankment bottom, which agrees with the behavior of the displacement vector figure mentioned above. Moreover, the tensile stress locally increases at the foot of embankment right after the slope. This seems to be the effect by the stress concentration to the top of slope, as shown in the distribution diagram of shearing stress. In comparison with both cases, the tensile stress in GC45 cm case is smaller than that in the GC90 cm cases. It tends to be similar even in the case of high fill (12 m, 16 m), and load diffusion effect of each GC seems to stabilize the whole embankment.

3.5 *Limit of embankment height without reinforcement*

The smallest safety factor for the no reinforcement case after a construction period of 720 hours can be calculated and depicted in Fig. 8, which is derived from exponent extrapolation. For example, a height of 11.3 m corresponds to a design safety factor of 1.2. The Limit of embankment height (corresponding to the

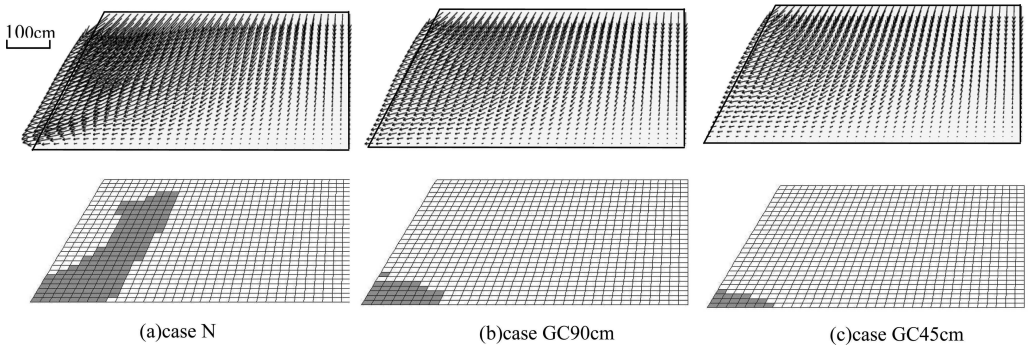


Figure 6. Displacement vector and shear failure region.

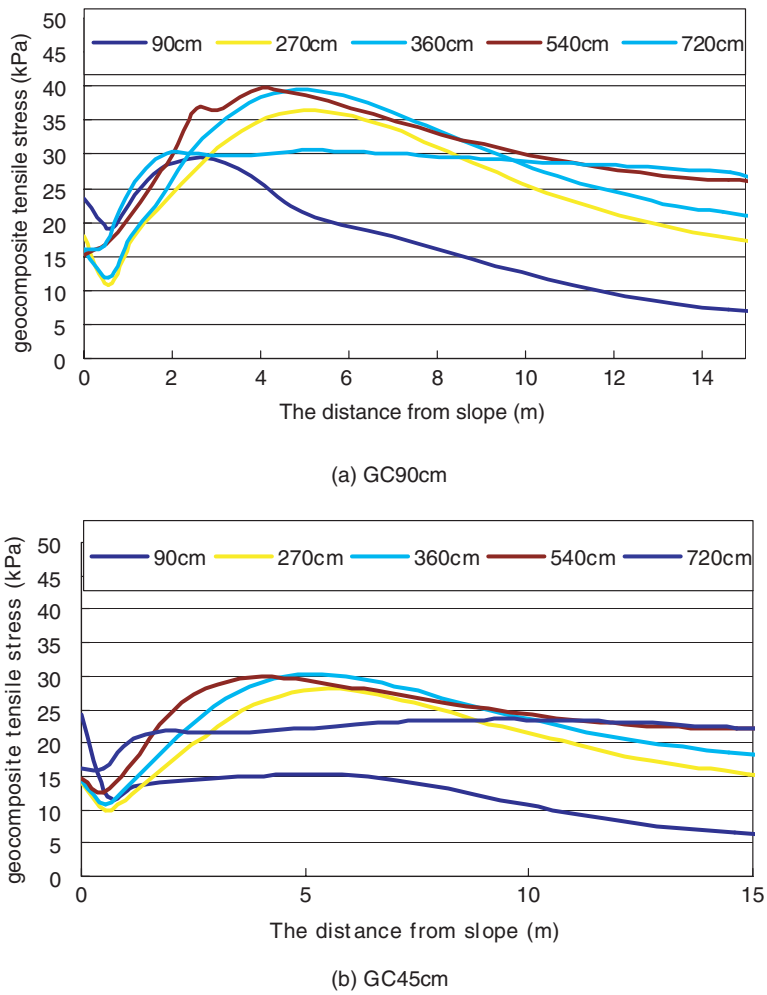


Figure 7. Tensile stresses in the reinforcement material.

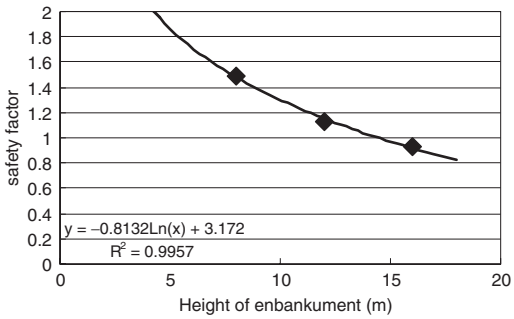


Figure 8. Safety factor against embankment height with out reinforcement material.

safety factor of 1.0) is 14.4 m, which approximately agrees with the results from 12 m high cases.

4 CONCLUSIONS

Considering a steep slope banking with high water content low quality soil, the reinforcement effect of geocomposite is evaluated by using finite difference analysis. The settlement at the crown, the displacement at the slope and the resultant stress in geocomposite are studied in this paper. Some suggestions for the design of geocomposite-reinforced embankment are also proposed. It has been clarified as follows.

This analysis method was able to simulate the increase of strength that related to the consolidation of embankment itself and the consolidation construction period; therefore, it can approximate a real embankment by the analysis.

In case of the reinforced embankment, the effect of restraining the displacement of the slope is much greater than the effect of restraining the settlement of the crown.

In the cases without reinforcement, large displacement occurred at the foot of the embankment, while in the cases with reinforcement, the displacement at the middle slope is the largest, which is quite differed from the former one.

The characteristics of resultant tensile stress in the reinforcement and the stress concentration in the embankment are analyzed, which can help the design of geocomposite-reinforced embankment.

In the future, this analysis method can be verified and generalized by comparing the analyses results with construction case in the field.

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