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Embankment Construction with Saturated Clayey Fill Material Using Geocomposites

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ABSTRACT: To design an embankment using saturated clayey backfill with a dual function (drainage and reinforcement) geocomposite, predicting the undrained shear strength (S_u) of the backfill during construction is an essential requirement. A method of predicting the S_u value has been proposed, in which the effects of discharge capacity (Q_w) of the geocomposite, spacing ($2B$) between geocomposite layers, construction speed (V), and the coefficient of consolidation (C_v) of the backfill are considered. Then Q_w values of four geocomposites were measured by laboratory tests under the confinement of clayey soils. The test results indicate that to maintain a higher long-term (about 1 month) Q_w value (> 100 m^3/year/m), for a geocomposite confined in clayey soil, it must have a drainage core and a strong filter. Finally, example analysis was conducted on predicting S_u values and evaluating the factor of safety (FS) of an assumed geocomposite reinforced 5 m height embankment with saturated clayey backfill. A parametric study was carried out to investigate the effects of Q_w, $2B$, as well as V.

1. INTRODUCTION

How to effectively treat the waste clayey soils generated from construction sites or dredged from ports is one of the geoenvironmental problems. On the other hand, there is a shortage of granular materials for embankment construction in Japan. Therefore, it is desirable to use waste clayey soils as embankment fills. Since the strength of the waste clayey soil is low, in many cases, it cannot be directly used as a construction material. A commonly used method is to mix cement or lime into the waste clayey soils to improve its engineering properties first, and then use them in embankment and/or land reclamation constructions. There are reports of using this kind of method for airport construction (Tsuchida and Kang 2003). As for embankment construction, this method is not widely used because the resulting material may have sufficient compression strength, but it has a high void ratio and is brittle and it may cause long-term maintenance problems of a road embankment. Also, cement treatment may result in higher pH values of the treated soil, which may have a negative impact on geo-environment. The Institute of Advanced Construction Technology (IACT), Japan (1999) proposed that for the waste clayey soils with a cone resistance of more than 200 kPa can be used as fill material of part of an embankment. The cone resistance refers the value measured by a portable cone with a cross-sectional area of 324 mm^2 and an apex angle of 30° and the penetration speed of 10 mm/s. Recently, a project of constructing a highway around Ariake Sea, Japan, is under way. In this project, clayey soils with a cone resistance of more than 500 kPa were accepted as part of the embankment fill.

Another method of embankment construction with saturated clay backfill is to use dual function (reinforcement and drainage) geocomposites. The drainage effect of the geocomposite accelerates the self-weight induced consolidation and increases the undrained shear strength (S_u) and the density of the clayey soils, and the mobilized tensile force in the geocomposite further increases the stability of the embankment. There are reported case histories of this method (e.g. Tatsuoka and Yamauchi 1986). However, for the design of a dual function geocomposite reinforced embankment with saturated clayey backfill, there are still issues to be resolved. One of the issues is how to predict the distribution of S_u of the clayey backfill during an embankment construction, and another one is what kind of geocomposite can be used in term of long term discharge capacity for accelerating the consolidation and tensile strength for embankment stability.

In this paper, firstly, a method of predicting S_u values within a dual function geocomposite reinforced embankment with saturated clayey backfill is described. Then the results of clay-confined discharge capacity tests of four geocomposites are presented. Finally, the factors influencing S_u values within an assumed 5 m high embankment and therefore the factor of safety (FS) of the embankment are investigated numerically.

2. UNDRAINED SHEAR STRENGTH OF SATURATED CLAYEY BACKFILL (S_u)

2.1 Method for Calculating S_u

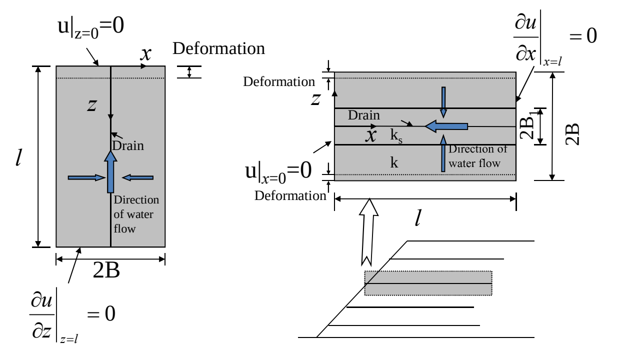
To design an embankment with saturated clayey backfill and reinforced by a dual function geocomposite, one of the tasks is to predict S_u values within the embankment during the embankment construction. The value of S_u of a soil is a function of effective stress, stress history and the mechanical properties of the soil. Ladd (1991) proposed an empirical equation to calculate S_u value of clayey soils as follows:

$$S_u = S \cdot \sigma_v' \cdot (OCR)^m \qquad (1)$$

where σ'_v is vertical effective stress, OCR is over-consolidation ratio, and S and m are constants. Ladd (1991) proposed that the range for S is 0.162 to 0.25 and for m is 0.75 to 1.0. For an embankment construction, OCR = 1.0, and only S is needed. Therefore, if the value of σ'_v can be evaluated and S value can be determined, S_u can be predicted.

2.2 Consolidation Theories

Regarding the theory (equation) for calculating the self-weight induced degree of consolidation (U), two possible cases can be considered. One is for using a continuous sheet geocomposite, and another is using a strip geocomposite (i.e. placed in a discontinuous fashion). It is suggested that for the continuous sheet geocomposite case, a solution proposed by Hird et al. (1992) can be used. Hird et al. (1992) extended Hansbo′s solution (1981) for vertical drain consolidation under an axisymmetric condition to a plane strain condition. A vertical plane strain unit cell adopted by Hird et al. (1992) is shown in Fig. 1(a). In the case of embankment construction, the geocomposite serving as a drain is placed horizontally. Figure 1(b) shows a horizontal plane strain unit cell. Although the deformation patterns in Figs. 1(a) and (b) are different, both of them satisfy the equal strain assumption, a basic assumption of Hansbo′s solution. Therefore, it is proposed that equations derived for Fig. 1(a) case can be used for Fig. 1(b) case. With the notations in Fig. 1(b), the average degree of consolidation at a distance of x from the drainage surface (x = 0) is as follows:



(a) Plan strain vertical drain drain (b) Plane strain horizontal

Figure 1. Plane strain unit cells

$$U = 1 - exp(\frac{-8T}{\mu}) \quad (2)$$

The expressions for T and μ are as follows:

$$T = \frac{C \cdot t}{4B^2} \quad (3)$$

$$\mu = \frac{2}{3} + \frac{2k}{B \cdot Q_w}(2 \cdot l \cdot x - x^2) + 2 \cdot (\frac{k}{k_s} - 1)(b_s - b_s^2 + \frac{b_s^3}{3}) \quad (4)$$

where C is the coefficient of consolidation of clayey soil, t is time, B is the half width of a plane strain unit cell, k and k_s are the hydraulic conductivities of clayey soil and the smear zone around a drain, respectively, Q_w is the discharge capacity of a geocomposite per unit width, l is the drainage length, x is the distance from the drainage surface, and b_s=B_1/B (B_1 is the half width of the smear zone). Equation (4) was derived by Chai and Miura (2002).

In the case of the strip geocomposite, as shown in Fig. 2, for an embankment construction, the horizontal and vertical spacings between geocomposite strips may not be equal. The influencing area of a strip may be a rectangle rather than a square. It is proposed to represent the rectangle by a circle with the same area and using Hansbo's (1981) solution to calculate U value. To investigate the effect of the shape of the smear zone on the average degree of consolidation, Chai and Miura (1999) conducted finite element analyses with rectangular and circular shaped smear zone of the same cross-sectional area, and the results indicate that the shape does not make a significant influence on the average degree of consolidation. The U value can still be calculated by Eq. (2), but the expressions for T and μ become:

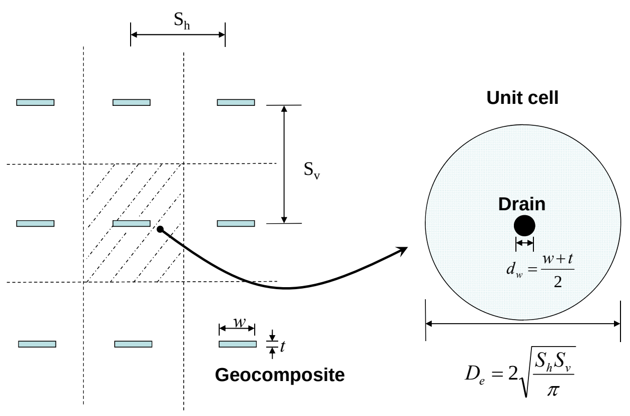


Figure 2. Unit cell model for strip geocomposite case

$$T = \frac{Ct}{D_e^2} \quad (5)$$

$$\mu = \ln\frac{D_e}{d_s} + \frac{k}{k_s}\ln\frac{d_s}{d_w} - \frac{3}{4} + \pi\frac{2l^2k}{3q_w} \quad (6)$$

where D_e is the diameter of unit cell, d_w is the equivalent diameter of a drain, d_s is the diameter of the smear zone, and q_w is the discharge capacity of a drain (here a geocomposite strip).

For embankment construction, it can be considered that there is no smear zone (all the soil is remolded). Then with the method presented above, there are totally six parameters needed: Q_w (or q_w); C; k; B (or D_e); construction speed (V), and the constant S in Eq. (1). Further V and B (or D_e) can be specified by design, the remaining four parameters are, Q_w (or q_w), C, k, and S. S is an empirical parameter and can be determined based on the local experience. Finally, Q_w (or q_w), C, and k must be measured by laboratory or field tests.

2.3 Method for Calculating Average Degree of Consolidation

During an embankment construction, backfill is placed layer by layer. To predict the vertical effective stress variation in each layer during the embankment construction, the following assumptions are made.

(1) Approximate the construction process by stepwise loads (Fig. 3).

(2) Considering the condition at the base of an embankment, take the total load at i step as p_i, and the degree of consolidation at time t_i as U_i. At t_i, incremental load of j step Δp_j is applied, then for a total load $p_j = p_i + \Delta p_j$, the degree of consolidation (U_{jo}) at t_i can be calculated as (Chai and Miura 2002):

$$U_{jo} = \frac{U_i \cdot p_i}{p_i + \Delta p_j} \quad (7)$$

An imaginary time corresponding to U_{jo} (under load p_j) is:

$$t_{jo} = -\frac{B^2}{2 \cdot C}\mu \cdot ln(1 - U_{jo}) \quad \text{(geocomposite sheet)} \quad (8a)$$

$$t_{jo} = -\frac{D_e^2}{8 \cdot C}\mu \cdot ln(1 - U_{jo}) \quad \text{(geocomposite strip)} \quad (8b)$$

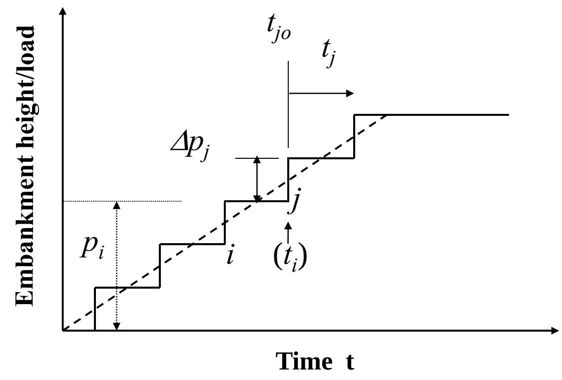


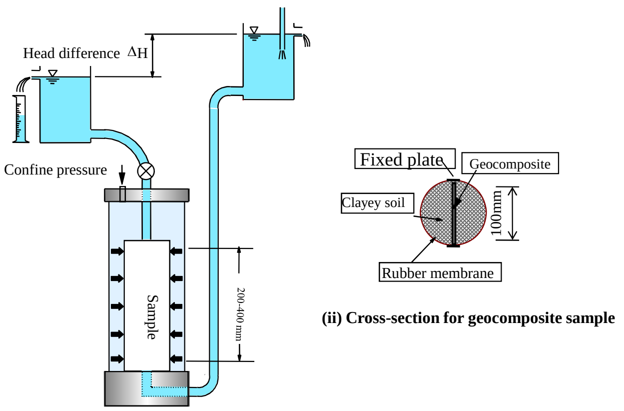
Figure 3. Stepwise loading

Using the moment of applying Δp_j as a new origin for time, if the time from the new origin is t_j, then the time for calculating the degree of consolidation at time t_j will be (t_{jo}+t_j). When the average degree of consolidation of each layer of an embankment at a given time is known, σ'_v and therefore S_u can be calculated.

3. LABORATORY TEST RESULTS ON Q_w

3.1 Test Equipment

Q_w values provided by manufacturers are normally under the condition that the geocomposites are confined by rubber membrane or between two parallel plates (ASTM 2003). However, if using a geocomposite for constructing an embankment with saturated clayey backfill, there is a possibility that clayey particles enter the opening of the geocomposite or the drainage channel through the filter thus influencing the Q_w value. The flow rates of four geocomposites were measured under the confinement of clayey soils using a triaxial type discharge capacity test device (Fig. 4). During the tests, tap water was used and re-circulated by a micro-pump. The test procedure was described elsewhere (e.g. Chai and Miura 2002).



(i) Set-up

Figure 4. Illustrations of test set-up

3.2 Materials

Four geocomposites (A, B, C and D) were used and their structures and some of index properties are given in Fig. 5. Two types of clayey soils were used as confinement materials. One was remolded Ariake clay. Its liquid and plastic limits were 115% and 54%, respectively. The clay content (< 2 μm) was about 48%. Another one was a mixture of the Ariake clay and decomposed granite passing through 1.2 mm sieve with a ratio of 1:2 by dry weight designated as the mixed soil. The liquid and plastic limits of the mixed soil were 61.6% and 33.7% respectively.

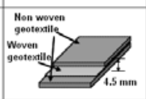
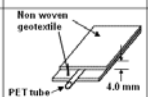
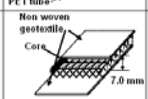
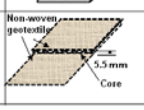
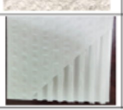
Type	Structure (sketch)	Structure (photo)	Material	Mass per unit area (g/m²)	Tensile strength (kN/m)
A			Polypropylene	678	49.3
B			Non-woven geotextile: polypropylene Tube: PET filament spacing: 100 mm, inside diameter: 2 mm	281	11.0
C			Non-woven geotextile: polypropylene Core: high density polyethylene	1010	18.0
D			Filter: polyester Core: polyolefin resin	1170	52.7

Figure 5. Structures and index properties of geocomposites

3.3 Test Results

For geocomposites A and B, the tests were conducted under the confinement of both the Ariake clay and the mixed soil, for geocomposite C, only with the mixed soil confinement, and for geocomposite D only with the Ariake clay confinement. All the tests were conducted with a hydraulic gradient (i) of about 0.1 (the results were linearly converted to i = 0.1). The confining pressures (σ) adopted were 10, 50, and 100 kPa. The test results are given in Figs. 6 to 9 for geocomposites A, B, C and D respectively. Generally, the flow rates reduced with the elapsed time and the increase of σ. It is considered that the reduction on the flow rate with the elapsed time is mainly due to the clogging of the drainage paths caused by the soil particles entered the openings of the geocomposites. Geocomposites A and B had lower to zero long-term (about 1 month) flow rates and geocomposites C and D had higher flow rates.

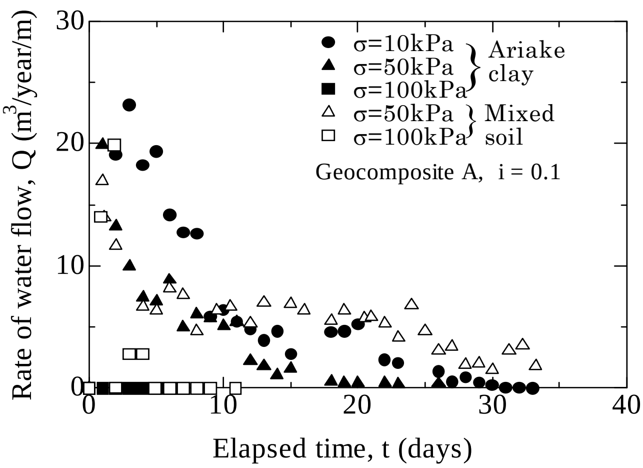


Figure 6. Rate of flow of geocomposite A

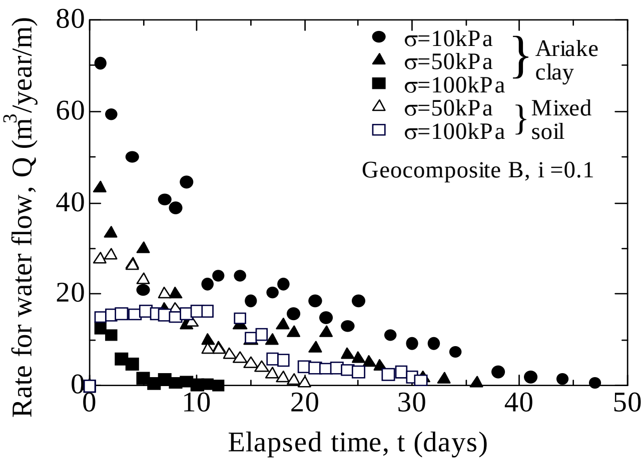


Figure 7. Rate of flow of geocomposite B

For geocomposite A (Fig. 6) at 1 month elapsed time, when σ = 100 kPa, the flow rate was practically zero. Under σ = 10 and 50 kPa, it was 0 to 2 m³/year/m. The test with the mixed soil confinement showed a higher long-term (more than one month) flow rate. Geocomposite B has one drainage tube per 0.1 m width and had higher flow rate than geocomposite A. At one month of elapsed time, the flow rate is 0 to 10 m³/year/m under σ = 100 to 10 kPa (Fig. 7). Geocomposites C and D have a drainage core and the reduction on flow rates with the elapsed time is less significant than geocomposites A and B, and under σ = 100 kPa, within one month, the flow rate almost did not decrease (Figs. 8 and 9). Regarding the effect of confining pressure, short-term (lasted for about 3 hours) test results indicate that for geocomposite C, when the confining pressure increased to more than 150 kPa, obvious reduction of the flow rate was observed and the flow rate of σ = 200 kPa was about 70% of that of σ = 100 kPa. For geocomposite D, the flow rate at σ = 200 kPa was about 90% of that at σ = 100 kPa. These results indicate that the filters of geocomposites C and D are relatively strong.

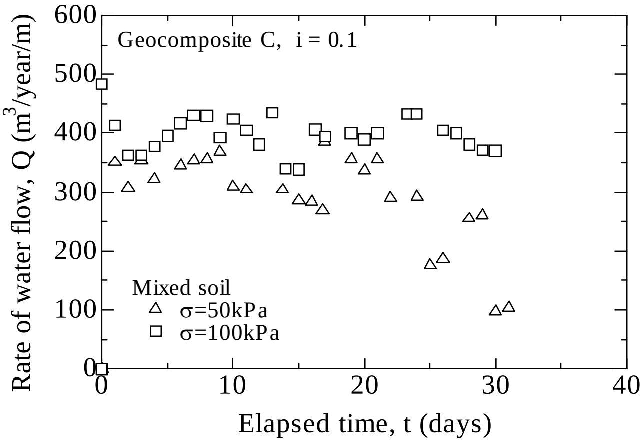


Figure 8. Rate of flow of geocomposite C

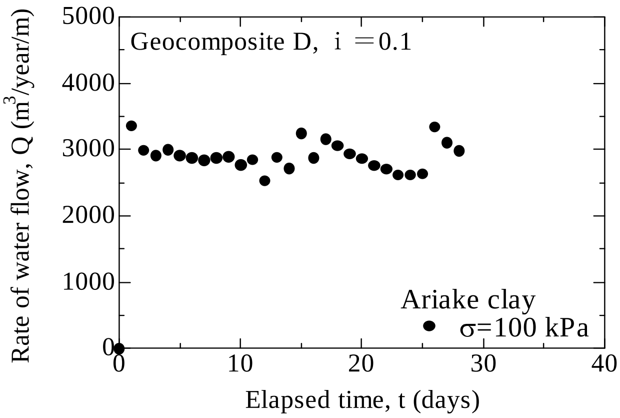


Figure 9. Rate of flow of geocomposite D

Above discussions clearly indicate that to maintain a higher long-term (about one month) flow rate under clayey soil confinement, a geocomposite must have a drainage core and a strong filter. The geocomposites A and B may be not suited for constructing embankments with saturated clayey fill where the long-term drainage capacity is required. From laboratory model test results, Yasuhara et al. (2003) suggested that in case of using clayey backfill, a geocomposite should be sandwiched into a sand layer to enhance its drainage capacity. Chai et al. (2004) reported that the long-term confined in clay flow rate of a prefabricated vertical and/or horizontal drain is a function of hydraulic radius (R) of its drainage channel. R is defined as the cross-sectional area of a drainage channel divided by its periphery length. For geocomposites A and C, it is difficult to calculate R values because of irregular shape of the drainage channels. For geocomposite B, the drainage tube has an R value of 0.5 mm, and for geocomposite D, an R value of about 0.93 mm can be estimated. For geocomposite D, the drainage channel has a trapezoidal shape and a cross-sectional area of about 14 mm^2, and total number of the channels of about 202 per meter width. By further assuming that the flow capacity of geocomposite B is mainly from the tube, the flow rates per unit drainage area of geocomposites B and D at about 1 month elapsed time can be evaluated and the results are shown in Fig. 10 together with the results reported by Chai et al. (2004). The results from this study confirm the finding by Chai et al. (2004) about the effect of R on flow rate. Figure 10 also indicates that a geocomposite with a higher R value can have sufficient confined in clay discharge capacity without being sandwiched into a sand layer.

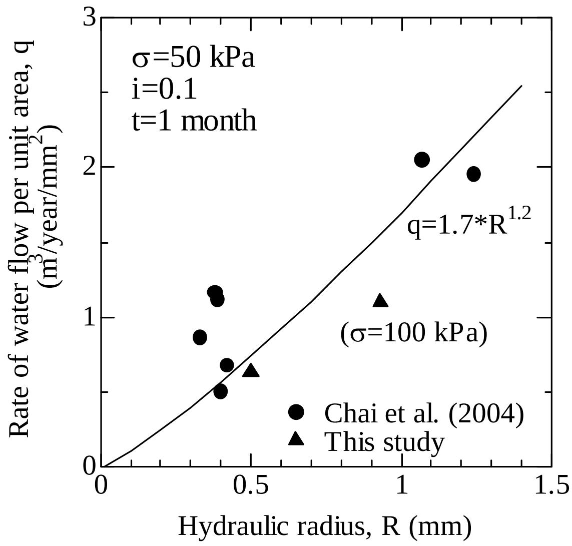


Figure 10. Effect of hydraulic radius on the flow rate

The discharge capacity (Q_w) is defined as the flow rate under a hydraulic gradient (i) of 1.0. For geocomposites C and D, if linearly converting the results given in Figs. 8 to 9 to i = 1.0, at about one month elapsed time, the range of Q_w will be about 1,000 – 25,000 m^3/year/m.

4. STABILITY ANALYSIS

4.1 Conditions Assumed for Analysis

The geometry of a 5 m high embankment assumed for stability analysis is shown in Fig. 11. It is further assumed that a sheet type geocomposite is used, and the potential failure surface will not pass through the foundation. As shown in Fig. 11 the geocomposite is discontinued at the middle of the embankment. Although whether the geocomposite will be continued through the whole width of an embankment depends on the construction procedure, discontinuous assumption is in the safe side in term of stability analysis, in which the possible pullout failure of the geocomposite can be considered. Other assumed conditions are listed in Table 1. The Q_w values are selected based on the laboratory test results presented above but much lower than that of geocomposites C and D. Regarding the allowable tensile force (T_a) of the geocomposites, the ultimate tensile strengths provided by the manufacturers are given in Fig. 5. For geocomposite A, the test was conducted using a 200 mm wide strip sample with a strain rate of 1%/min. The failure strain was about 10%. For geocomposite B, the test was conducted using the sample of 100 mm in width and 200 mm in length (between clamps) with a strain rate of 100%/min. The failure strain was about 40%. For geocomposite C, the test was conducted per ISO 10319 (ISO 1993) and the failure strain was 60%. For geocomposite D, the test was conducted using a strip of 94 mm in width and 250 mm in length with a strain rate of 20%/min. The peak value was mobilized at a strain of about 10% and the sample failed at a strain of about 50%. Based on the above information, T_a values of 0, 5 and 10 kN/m are assumed. It is not intended to use Q_w and T_a values of a specific geocomposite, rather the analysis tries to provide some general information on the effect of Q_w and T_a on the stability of a dual function geocomposite reinforced embankment with saturated clayey backfill.

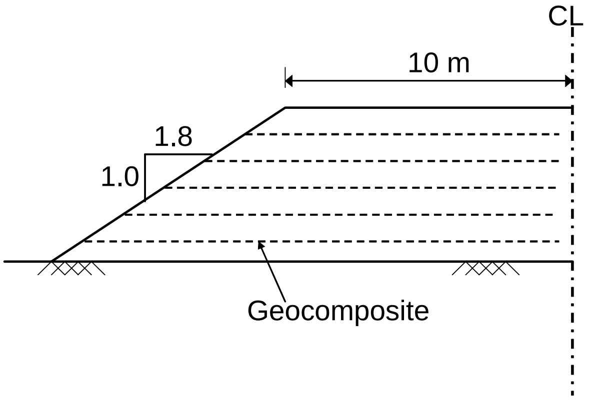


Figure 11. Geometry of an assumed embankment

Table 1 Parameters Used for Stability Analysis

Parameters	Basic value	Range of variation
Discharge capacity of geocomposite, Q_w (m^3/m/year)	20	10 – 100
Spacing between geocomposites, $2B$ (m)	0.5	0.5 – 1.0
Coefficient of consolidation of backfill, C (m^2/day)	0.1	–
Speed of construction, V (m/day)	0.05	–
Hydraulic conductivity of backfill, k (m/day)	0.004	–
Total unit weight of backfill, γ_t (kN/m^3)	15.0	15.0 and 20.0
Allowable tensile force in geocomposite, T_a (kN/m)	5	0 – 10

4.2 Predicted S_u Values Within the Embankment

With the method presented above and the values of parameters listed in Table 1, the distribution of S_u within the assumed embankment can be predicted using Eq. (1). In the calculation, the adopted value of the constant S in Eq. (1) is 0.25.

Figure 12 shows the effect of Q_w and spacing ($2B$) on S_u distribution at the end of the embankment construction. It can be seen that Q_w had a significant influence on S_u value. S_u increased with Q_w, but the increase rate decreased with the increase of Q_w. Increasing in spacing of geocomposites reduced S_u values. However, with the increase of Q_w value, the difference between $2B$ = 0.5 m and 1.0 m cases gradually decreased, which means that using a geocomposite with a higher Q_w value, a relatively larger spacing (e.g. $2B$ = 1.0 m) can be adopted and it may result in a more economic design.

Figure 13 shows the distributions of S_u at the end of the construction with three different construction speeds (V). Reducing in V increased the time for self-weight induced consolidation and resulted in higher S_u values.

4.3 Factor of Safety (FS)

With the predicted S_u values given in Figs. 12 and 13, and T_a values in Table 1, FS of the assumed embankment (rigid foundation) was analyzed by using Bishop's slip circle method (Bishop 1955). The program used is ReSSA (2.0) (Leshchinsky 2001). Failure mechanisms of the reinforcement considered are (1) rupture and (2) pullout, i.e. the mobilized tensile force is the minimum of the pullout and rupture failures. In case of pullout failure, the interface shear resistance between the geocomposite and backfill soil was 80% of the corresponding shear strength of the backfill soil.

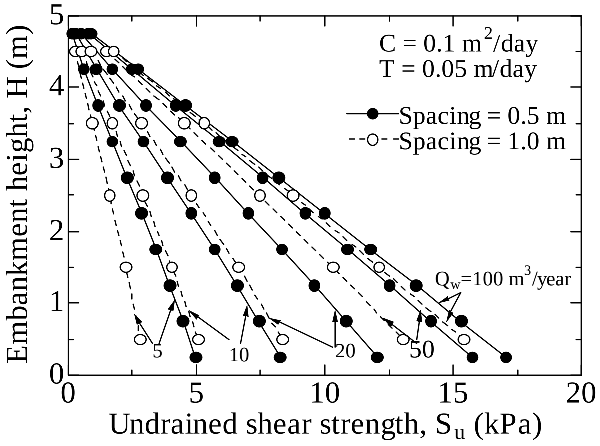


Figure 12. Effect of Q_w on predicted S_u values

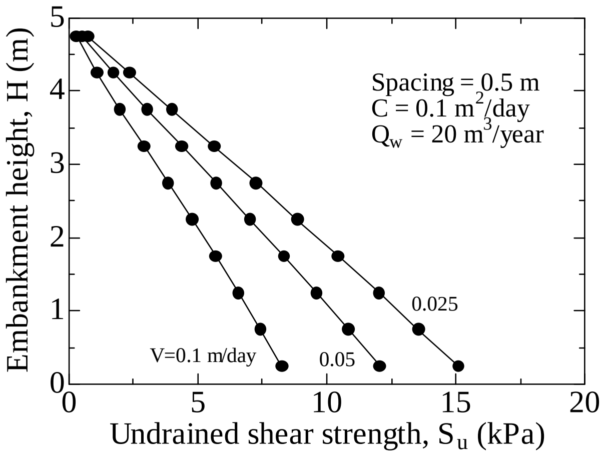


Figure. 13 Effect of construction speed on S_u values

Figure 14 shows the variation of FS with Q_w for $2B$ = 0.5 m (solid lines) and 1.0 m (dashed lines) cases. It can be seen that FS increases with the increase of Q_w, but the increment rate is gradually reduced. When Q_w > 50 m^3/year/m, the increase rate is small. If taking FS > 1.2 as a basic requirement (JRA 1999), without considering the reinforcement effect (T_a = 0), the requirement can be satisfied for $2B$ = 0.5 m and $Q_w \geq 50$ m^3/year/m, and $2B$ = 1.0 m and $Q_w \geq 100$ m^3/year/m cases. Comparing the FS of $2B$ = 0.5 m and 1.0 m cases, two points can be made. The first one is that with the increase of Q_w, the difference between FS values of $2B$ = 0.5 m and 1.0 m becomes smaller (the difference on S_u values becomes smaller, Fig. 12). The second one is that for $2B$ = 1.0 m case, the effect of T_a on FS is smaller than that of $2B$ = 0.5 m case because $2B$ = 1.0 m case has less number of the reinforcement (geocomposite) layers.

The effect of construction speed on FS is shown in Fig. 15. The slower the construction speed, the higher the FS value. For Q_w = 20 m^3/year/m, without reinforcement effect, FS > 1.2 requirement cannot be satisfied even for V = 0.025 m/day case. FS > 1.2 can be satisfied for V = 0.05 m/day and T_a = 5 kN/m case.

Theoretically, the degree of self-weight induced consolidation is not influenced by unit weight of the backfill (γ_t) (magnitude of load). Therefore, the vertical effective stress (σ'_v) within the embankment, and therefore S_u values (by Eq. (1)) of the embankment are linearly proportional to effective unit weight of the backfill. However, the increase in γ_t will increase driven force for slip failure. As a result, if the reinforcement effect is not considered, γ_t will not influence FS values. In case of considering the reinforcement effect, since the allowable tensile force (T_a) in the geocomposite does not increase with the increase of γ_t, the corresponding FS value will decrease. Figure 16 compares FS values of γ_t = 15 and 20 kN/m^3 cases. For T_a > 0 cases, FS of γ_t = 20 kN/m^3 case is lower. The larger the T_a value considered, the larger the difference on FS values.

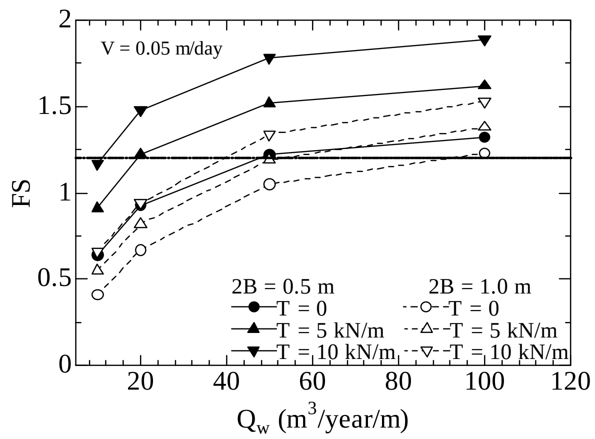


Figure 14. Effect of discharge capacity (Q_w) and spacing on FS

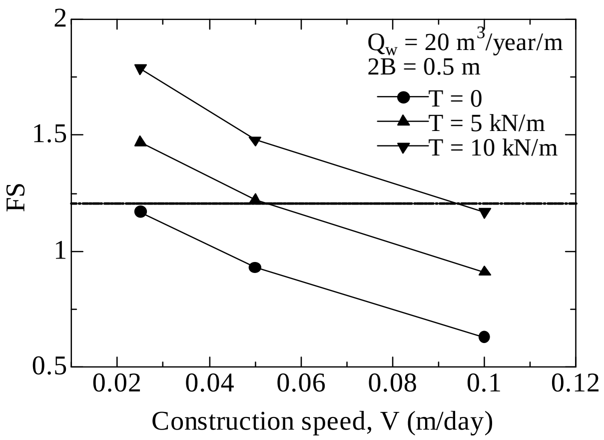


Figure 15. Effect of construction speed on FS

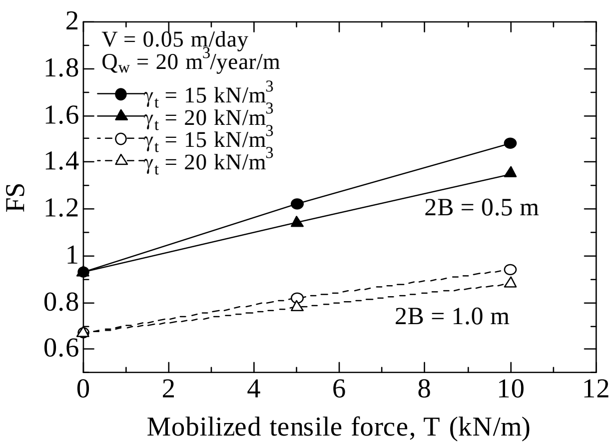


Figure 16. Effect of total unit weight (γ_t) of the backfill on FS

The results of slip circle analysis indicate that using dual function (drainage and reinforcement) geocomposites, an embankment with a saturated clayey backfill can be successfully constructed.

5. SUMMARY AND CONCLUSIONS

It is proposed that saturated clayey soils can be used as embankment backfill with combination of a dual function (drainage and reinforcement) geocomposite. A method for predicting the undrained shear strength (S_u) of the clayey backfill within the embankment has been proposed which considers the effects of the spacing (2B) and discharge capacity (Q_w) of a geocomposite, construction speed and the coefficient of consolidation of the backfill. The results of clay-confined discharge capacity test of four geocomposites are reported. Finally an example analysis on the S_u distribution and the factor of safety (FS) of an assumed 5 m height embankment was conducted with parametric study. Based on the analysis and test results, the following conclusions can be drawn.

(1) Q_w values of geocomposites. The flow rates of the geocomposites decreased with the increase of the confining pressure and the elapsed time. To maintain a higher long-term (about one month) Q_w value, a geocomposite must have a drainage core and a strong filter.

(2) Undrained shear strength (S_u). S_u in an embankment at the end of the construction is a function of the spacing (2B) between geocomposites, the discharge capacity of a geocomposite as well as the construction speed. For the conditions investigated, it shows that when $Q_w \geq 100$ m^3/year/m, the difference on S_u values for 2B = 0.5 m and 1.0 m became small, which implies that using a geocomposite with a higher Q_w value, a larger spacing (e.g. 2B = 1.0 m) can be adopted.

(3) Factor of safety (FS). The analysis results indicate that with a requirement of FS > 1.2, for the conditions considered, even without considering the reinforcement effect (tensile force T_a = 0), 2B = 0.5 m and $Q_w \geq 50$ m^3/year/m, and 2B = 1.0 m and $Q_w \geq 100$ m^3/year/m cases can satisfy the requirement. In term of stability, using a saturated clayey backfill, an embankment can be successfully constructed with a combination of geocomposites.

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