Behavior of a Reinforced and an Unreinforced Test Embankment: A Comparison

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ABSTRACT: The observed behaviour of a reinforced and an unreinforced embankment constructed on a soft compressible organic clayey silt deposit is compared in this paper. A relatively high strength polyester woven geotextile was used as the reinforcement which was instrumented with three types of strain gauges. The field investigation indicates that considerable caution is required for the design of embankments on soft organic clayey silt deposits if such design is based on vane strength and on the use of simple limit equilibrium design procedures. The observed failure thickness of the unreinforced embankment was below the predicted range and that of the reinforced embankment was well within the predicted range. This suggests that the relative benefits of reinforcement may be even greater for the type of soil investigated in this test embankment (soft compressible organic clayey silt) than they are for conventional perfectly plastic or work hardening soils.

INTRODUCTION

Construction of embankments on soft compressible organic clayey silt/silty clay deposits often require special measures, such as the use of geotextile reinforcement, to satisfy stability requirements. Despite the research that has been conducted into the stability of fills on soft clays there are still many unanswered questions concerning the behaviour of unreinforced and geotextile reinforced embankments on soft compressible organic clayey silt deposits.

To provide a case record of deformation and progressive failure of both a geotextile reinforced and unreinforced embankment section, an instrumented test embankment was constructed on a soft compressible organic clayey silt deposit at Sackville, New Brunswick, Canada. A relatively high strength polyester woven geotextile (ultimate strength of 216 kN/m) was used as reinforcement. This paper focuses on the comparison of the observed behaviour between the unreinforced and reinforced embankments. Complete details regarding the performance of the unreinforced and reinforced embankments are given by Rowe et al., 1991a and 1991b respectively.

SOIL PROFILE AND TEST EMBANKMENT CONFIGURATION

The geometry of the reinforced and unreinforced sections are shown in Figs. 1 and 2 respectively. To minimize the fill required to cause failure, the root mat on the northern side of the embankment was cut to a depth of 1 to 1.2 m on an approximately 1.3 to 1.8 m square grid. Atterberg limits and organic content of the clayey silt deposit, data from two static cone penetration tests and data from the field vane tests performed at the reinforced and unreinforced sections are presented in Fig. 3. It is noted that the liquidity index of the soil exceeds unity at depths ranging from 1 to 6 m. Additional field tests such as pressure meter and piezo cone tests were also performed for this project but are not presented due to lack of space.

A locally available fill material (gravelly silty sand with some clay, unit weight = 19.6 kN/m^3) was used for most of the construction work. During the construction of the embankments and the berms, the fill was found to stand free at a slope of about 45° (see Rowe et al., 1991a for additional details regarding the properties of the fill). to ensure adequate interaction between the geotextile and the surrounding soil, a 0.3 - 0.5 m thick layer of good quality granular fill material (unit weight = 18 kN/m^3, $c' = 0$, $\phi' = 42.3°$) was used for the first 0.7 m thickness of the reinforced embankment.

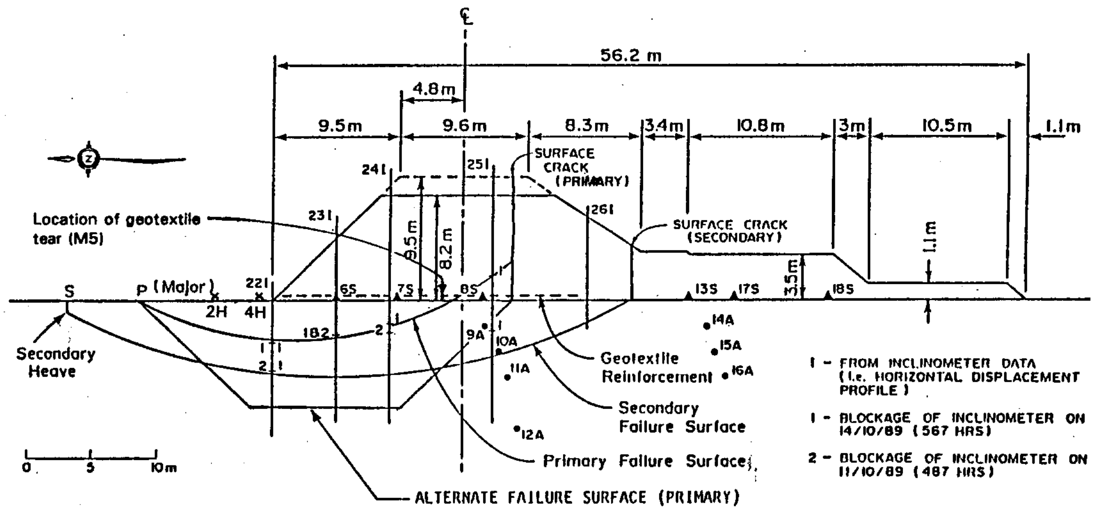

FIG. 1 EMBANKMENT GEOMETRY AND INFERRED FAILURE SURFACES: REINFORCED SECTION

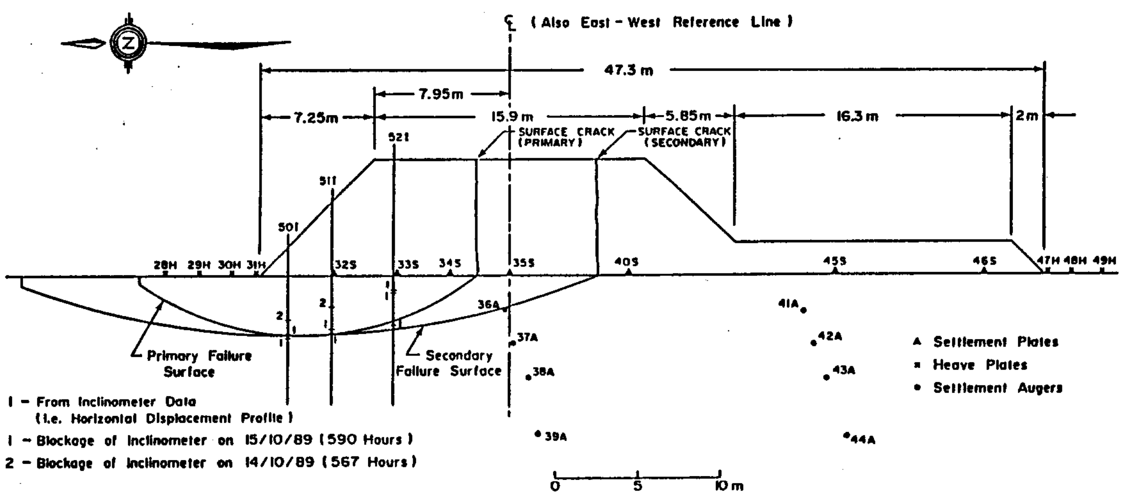

FIG. 2 EMBANKMENT GEOMETRY AND INFERRED FAILURE SURFACES: UNREINFORCED SECTION

The reinforcement used in this project was a Nicolon (style 68300) polyester woven geotextile with relevant engineering properties as given in Table 1. The geotextile was factory sewn.

Table 1: Properties of the geotextile

Mass	631 g/m^2
Tensile strength (wide width)	216 kN/m
Failure strain (wide width)	13%
Initial modulus (wide width)	257 kN/m
Elastic modulus (wide width)	1920 kN/m
Secant modulus(0-5% strain)	1466 kN/m
Secant modulus (0-10%strain)	1678 kN/m

Geotextile strains were monitored both in the transverse and longitudinal directions with a total of 38 electrical resistance, 7 electromechanical and 7 mechanical strain gauges (see Rowe and Gnanendran, 1994).

Both the piezo-cone data and vane data indicated that the soil strength varied over the site - being approximately 30% higher beneath what was selected to be the unreinforced section than beneath the reinforced section. Based on this information, it was considered best to arrange the sections such that the stronger soil was beneath the unreinforced section thereby minimizing the amount of fill required to fail the reinforced section. Limit equilibrium analyses performed on the basis of vane strength indicated that the failure height of the unreinforced section range between 7 and 11.4 m and that of the reinforced section range between 6.6 and 11.1 m. The range represents a reasonable interpretation of the vane data. The predicted failure heights were indicated as 9.2 and 8.8 m respectively for the unreinforced and reinforced sections based on an average vane strength profile beneath each section (see Fig. 3 for vane strength profiles) and limit equilibrium.

FAILURE OF REINFORCED EMBANKMENT

The settlement of the reinforced embankment at settlement plates 7S and 8S (see Fig. 1 for their location) is plotted

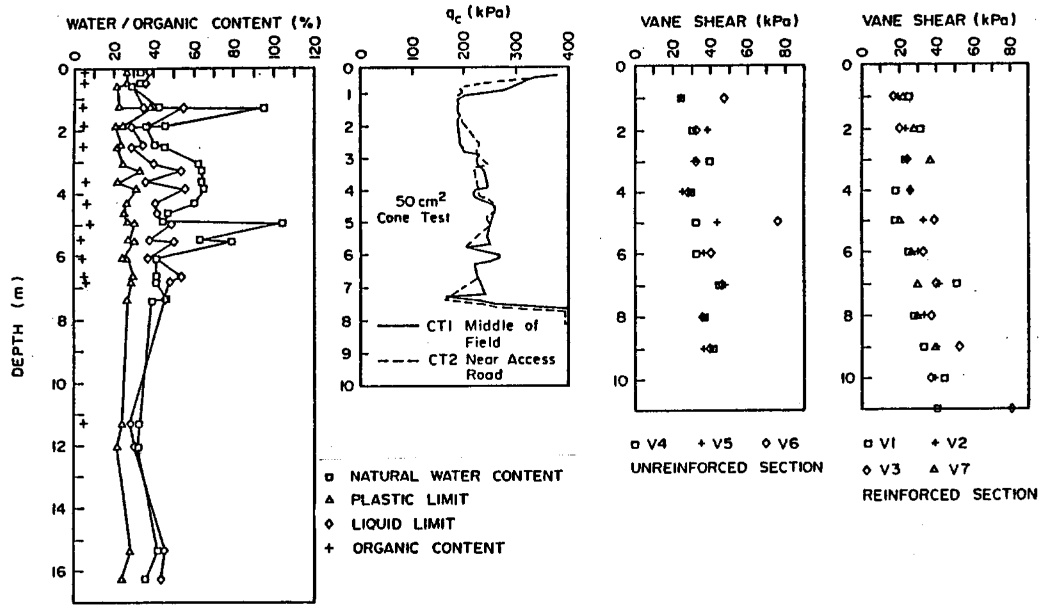

FIG. 3 SOIL PROFILE

against embankment thickness in Fig. 4. For convenience and comparison purposes, the similar data for the unreinforced embankment is also shown in this figure at settlement plates 33S and 35S (see Fig. 2 for their location). A reasonably linear load settlement relationship was indicated during the early stages of construction up to about 3.4 m. There was a gradual increase in the rate of settlement for embankment thicknesses between 3.4 and 5.5 m. The variation of heave of the ground north of the toe of the embankment (i.e. at heave plates 2H and 4H) with embankment thickness shown in Fig. 5 indicated a somewhat similar behaviour. The linear response during the early stages of construction was evident only up to about 2.4 m thickness, apparently indicating an elastic behaviour of the foundation soil. Flattening of the heave curves could be observed between 5.5 and 5.7 m thickness.

Based on the pore pressure data, the settlement and heave response, the large increase in horizontal displacement at inclinometers 22I, 23I and 24I, and the rapid increase in geotextile strain as the embankment thickness was increased from 5 to 5.7 m, Rowe et al. (1991b) inferred that the shear strength of the soil was reached when the embankment thickness approached 5.7 m and that the geotextile was all that prevented embankment failure at this fill thickness. Fig. 6 shows a plot of net embankment height (i.e. the difference in elevation between the embankment crest and the ground near the embankment toe) against fill thickness. It can be seen that the relationship between net embankment height and thickness was essentially linear and identical for both the reinforced and unreinforced embankments up to a fill thickness of about 2.4 m. Allowing for some consolidation settlement that occurred while the embankment was held at 2.4 m thickness for some time, the net height - thickness curve continued at the same slope between 2.4 and 3.2 m thickness after which non-linear behaviour was evident for the reinforced embankment. At a fill thickness of 5.7 m, there was considerable soil movement (without significant pore

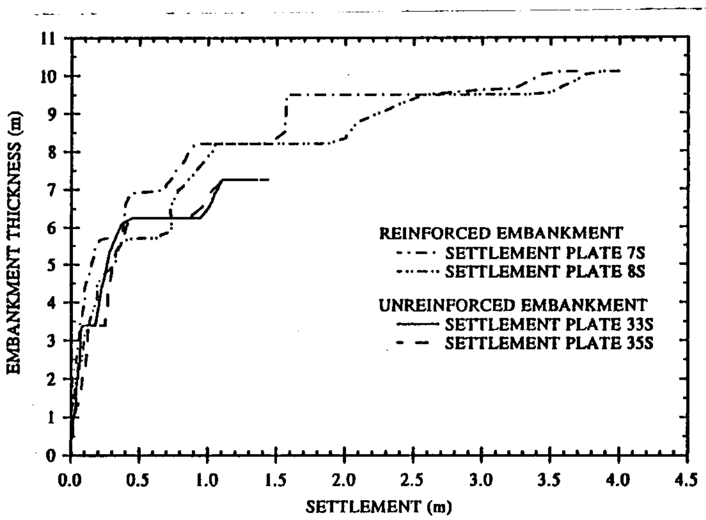

FIG. 4 VARIATION OF SETTLEMENT WITH EMBANKMENT THICKNESS AT SETTLEMENT PLATES 7S, 8S, 33S AND 35S

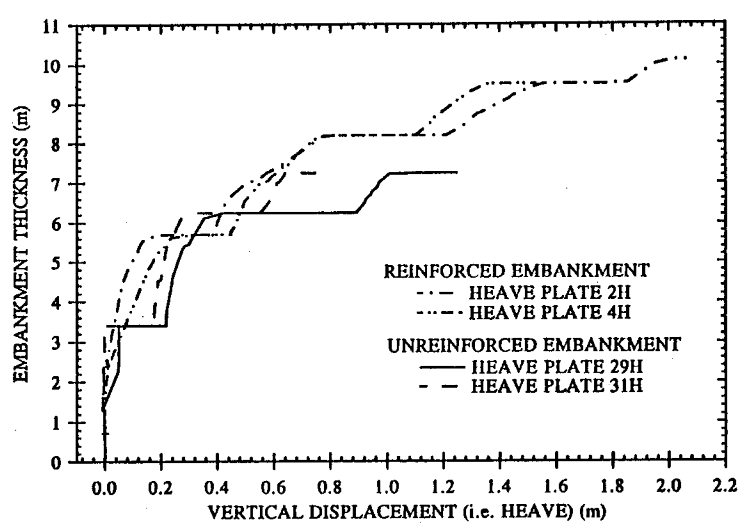

FIG. 5 VARIATION OF VERTICAL DISPLACEMENT WITH EMBANKMENT THICKNESS AT HEAVE PLATES 2H 4H, 29H AND 31H

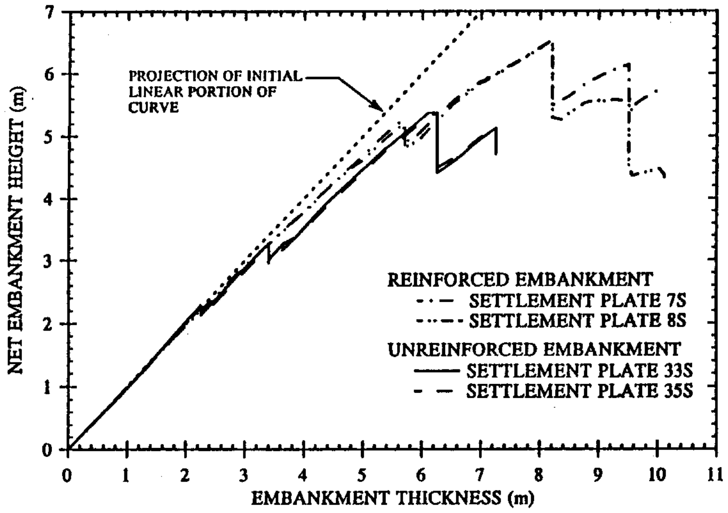

FIG. 6 VARIATION OF NET EMBANKMENT HEIGHT WITH EMBANKMENT THICKNESS AT SETTLEMENT PLATES 7S, 8S, 33S AND 35S

pressure dissipation) during a brief work stoppage. There was rapid development of strain from a typical value of about 2% at 5 m thickness to about 5% at 5.7 m thickness indicates that the geotextile began to play a significant role in providing embankment stability during this period and, as indicated above, the evidence suggest that the soil shear strength was reached along a potential failure surface at 5.7 m thickness. The authors have concluded that it is likely that it was only possible to increase the fill thickness above about 5.7 m at the reinforced section because of the geotextile reinforcement. Thus, although there was strong evidence of failure in the soil there was no evidence that the embankment itself had failed at a thickness of about 5.7 m since additional fill could be placed without any significant change in the rate of increase in net embankment height with fill thickness until the fill thickness reached about 6.5 m (i.e. a net embankment height of about 5.5 m) at which point there was a change in slope of the net height versus fill thickness response (see Fig. 6). However, the embankment continued to stand intact until the thickness was increased to 8.2 m. There was a rapid increase in the geotextile strain during the construction of the embankment from 5.7 m to 8.2 m.

Based on the geotextile strain measurements, it would appear that the geotextile reinforcement tore at about 13 m from the toe of the embankment at a fill thickness of 8.2 m and hence its contribution to stability was largely reduced. Corresponding to the failure of the geotextile, cracks opened up on the ground at the outer limit of a major heave zone which developed north of the embankment. Cracks also developed along the crest of the embankment. However, the failure of the reinforcement did not result in a dramatic collapse of the embankment. It would appear that the rheological properties of the soil were such that the stress redistribution resulting from the tearing of the reinforcement was largely absorbed by the soil in the short term. These stresses were redistributed in the soil with time resulting in a decrease in the net embankment height to about 5.2 m. The placement of additional fill increased the net fill height to 5.6 m at a fill thickness of 8.75 m. However, beyond this, there was no gain in net fill height as more fill was placed. When loading ceased at a fill thickness of 9.5 m, the net fill height was still only 5.6 m. The embankment continued to deform with no significant decrease in excess pore pressures, and the net height was reduced to about 4.4 m. An attempt to increase the fill thickness to about 10 m did not result in any significant change in net embankment height.

The response of the embankment after failure of the reinforcement (i.e. at 8.2 m fill thickness) was quite different to that prior to failure of the reinforcement and is consistent to the behaviour of the unreinforced embankment to be discussed below. It is evident that the reinforcement had played a significant role in improving embankment stability.

UNREINFORCED EMBANKMENT FAILURE

The variation of settlement of the unreinforced embankment with thickness was reasonably linear during the early stages of construction up to 3.4 m; probably due to "elastic" behaviour of the foundation soil (see Fig. 4). This is consistent with the finding of Tavenas et al. (1974) who suggested that foundations behave elastically up to a critical height approximately equal to about 50% of the failure height (of 6.1 m, as discussed subsequently in this paper). The heave of the ground north of the toe of embankment was negligible until the embankment was constructed up to 3.4 m thickness (see Fig. 5). Accordingly, the net embankment height of the unreinforced embankment increased linearly with embankment thickness during the early stages of construction up to 3.4 m thickness as evident from Fig. 6 (i.e. settlement plates 33S and 35S).

When the embankment was increased from 3.4 m to 5.4 m there was initially a thickness - settlement response similar to that in the initial phase of loading. From 5.4 m there was an increased rate of settlement. At a thickness of 6.1 m, the rate of deformation became sufficiently large that there was no gain in net embankment height as the fill thickness was increased to 6.25 m. When filling ceased at 6.25 m thickness, the embankment continued to deform rapidly as is evident from Fig. 6 and from the settlement and heave responses shown in Figs. 4 and 5. At this thickness, the soil continued to deform in a viscous manner without significant dissipation of pore pressures and over the next 42 hours the net embankment height was reduced from 5.4 m to about 4.5 m. The failure was of a visco-plastic nature. The failure thickness is interpreted to be 6.1 to 6.25 m, with the maximum net fill height achieved being 5.4 m.

To confirm the visco-plastic nature of the failure, more fill was added and the movement of the embankment closely monitored. The embankment height increased gradually again, but at a lesser gradient up to a maximum of about 5.2 m (see Fig. 6). The construction was stopped at 7.25 m thickness when very rapid settlement and heave was observed (see Figs. 4 and 5). The net embankment height dropped from 5.2 m to about 4.8 m within 2 hours. This response is consistent with a visco-plastic failure of the soil and it was evident in the field that the embankment could not be constructed above the net height of 5.4 m, which was reached first at a fill thickness of 6.1 m.

GENERAL OBSERVATIONS AND DISCUSSION

As discussed previously, the reinforced embankment failed at a thickness of 8.2 m. Failure of the reinforcement, cracking along the embankment crest and heave outside the embankment was observed at this thickness. The rapid construction of the embankment from 8.2 to 9.5 m, gave rise to a rotational type deformation of the crest, with maximum depression closer to settlement plate 8S, and severe cracking of the ground north of the embankment toe. Two distinct heave zones were evident during this phase of construction as indicated on Fig. 1. An apparent circular failure surface inferred from the inclinometer data (i.e. from both the horizontal displacement and blockage of the probe in the inclinometer casings) and the crack patterns, is shown in Fig. 1 as the primary failure surface. This surface intersected the reinforcement very close to the location of the failure of the reinforcement (near monitoring point M5). A secondary failure surface is shown passing close to the edge of the reinforcement. A potential deep seated non-circular failure mechanism is also shown.

A limit equilibrium analysis performed on the basis of average vane strength profile for the soil beneath the reinforced section (for a 8.2 m thick embankment) indicated factors of safety of about 1.46 and 1.21 respectively for the above primary and secondary failure surfaces inferred from the field investigation (see Fig. 1). The factor of safety for the deep seated wedge mechanism estimated on the basis of the same strength profile is about 1.54. The plasticity index of the soil ranged between 9 and 19% with an average of about 14% and the corresponding Bjerrum's correction for the vane strength will not have a significant effect on the factor of safety (see Bjerrum, 1973).

The apparent circular type failure surfaces established from the cracks and the inclinometer data for the unreinforced embankment are presented in Fig. 2. A limit equilibrium analysis performed on the basis of the same average vane strength profile for the foundation soil (for a 6.1 m thick embankment) indicated factors of safety of about 1.59 and 2.34 respectively for the above primary and secondary failure surfaces inferred from the field investigation. The failure thickness of 6.1 m indicated by this field investigation is significantly lower than the 9.2 m failure thickness indicated by limit equilibrium analysis performed on the basis of average vane strength profile (and below the range 7.0 - 11.4 m, determined for the range of vane strengths) of the soil beneath the unreinforced section. It would appear that the rate of loading has an effect on the apparent undrained shear strength of the soft organic clayey silts and that the strength that is evident from vane tests over estimates the strength available during embankment construction. Similarly the strength available on first loading of the soil decreases with time and the consequent progressive failure is likely to have contributed to the discrepancy between predicted and observed lower failure thickness for this embankment. It is noted that considerable caution is required for the design of embankment on soft organic clayey silt deposits if such design is based on vane strength and on the use of simple limit equilibrium design procedures.

It is noted that the failure thickness of 8.2 m indicated by this field investigation for the reinforced embankment is about 7% lower than the 8.8 m failure thickness determined from limit equilibrium analysis and average vane strength profile (and well within the range of 6.6 - 11.1 m determined from the range of vane strength profile) beneath the reinforced embankment. However, the observed failure thickness for the unreinforced embankment of 6.1 m is about 34% lower than the 9.2 m calculated using a similar limit equilibrium analysis and the average vane strength beneath the unreinforced embankment. This observation coupled with the pore pressure and geotextile strain responses (discussed earlier) suggest that the relative benefits of reinforcement may be greater for the type of soils (soft compressible organic clayey silt) investigated in this test embankment than they are for perfectly plastic or work hardening soils. It should be noted, however, that the proximity of the reinforced and unreinforced sections and the prior failure of the reinforced section may have had some effect on the failure height for the unreinforced embankment. Extrapolating on the basis of the unreinforced embankment and adjusting for the 30% difference in the vane strength of the foundation; one may have expected the reinforced section to have failed at about 4.7 m if no reinforcement had been used. In the reinforced section, significant deformation of the embankment did occur as the embankment was constructed from 3.4 m to 5.7 m and it is likely that if there had not been any reinforcement the embankment would certainly have failed at some thickness between 5 m and 5.7 m.

SUMMARY AND CONCLUSIONS

The observed behaviour of a geotextile reinforced and unreinforced embankment sections constructed on a soft compressible organic clayey silt deposit have been compared in this paper. The soil strength varied over the site with the average vane strength beneath the unreinforced embankment being about 30% higher than that beneath the reinforced embankment. The reinforced embankment behaved elastically up to about 2.4 m thickness as evident from settlement and heave responses. The inclinometer data and the geotextile strains indicated significant plastic deformation in the soil during the construction of the embankment from 5 to 5.7 m. Based on the excess pore pressure responses and soil deformations, it is considered that the foundation soil approached failure at a thickness of about 5.7 m. It was possible to construct the embankment above 5.7 m thickness because of the geotextile and the reinforced embankment failed at a thickness of about 8.2 m (i.e. a net height of about 6.6 m) when the tensile capacity of the geotextile was exceeded and the geotextile tore. The failure was of a plastic type and no classical-type of abrupt failure was encountered during the construction of this embankment.

The unreinforced embankment behaved elastically up to a thickness of about 3.4 m. The settlement and heave responses suggested that the embankment failed at a thickness of about 6.1 m and a corresponding net height of 5.4 m. The failure was of a plastic (or visco-plastic) type and no classical-type abrupt failure was encountered during the construction of this embankment. The height to which the unreinforced embankment could be constructed was substantially less than would be expected from conventional limit equilibrium analysis based on vane strength data. It is suggested that progressive failure may have been a significant factor affecting the performance of this embankment and considerable caution is required for the design of embankment on soft organic clayey silt deposits if such design is based on vane strength and on the use of simple limit equilibrium design procedures.

It was noted that the soil strength varied over the site - being approximately 30% higher beneath the unreinforced section than beneath the reinforced section. The fact that the failure thickness of the unreinforced embankment was below the predicted range and that the reinforced embankment was well within the predicted range (and significantly above the predicted thickness for the average strength profile) suggests that the relative benefits of reinforcement may be even greater for the type of soil investigated in this test embankment (soft compressible organic clayey silt) than they are for perfectly plastic or work hardening soils. Allowing for the 30% difference in shear strength beneath the two embankment sections, it would appear that the reinforcement permitted the construction of the reinforced embankment to fill thicknesses between 45 and 75% greater than would have been expected without reinforcement.

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