A SIMPLE SERVICEABILITY CRITERION FOR REINFORCED EMBANKMENT ON SOFT CLAY

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ABSTRACT
In this paper, the behaviour of reinforced embankment on soft clay is explored parametrically using a non-linear finite element program. The objective of the parametric study was to relate the lateral deformation of the clay layer to the amount of tension mobilized in the reinforcement. It was found that large tension in the reinforcement could only be mobilized by inducing excessive lateral deformation in the clay layer. A novel serviceability criterion is proposed based on the results of the parametric study that aims to limit the lateral deformation of the clay foundation at the toe of the embankment by limiting the allowable mobilized tension in the reinforcement. Based on this serviceability criterion, a simple procedure for the evaluation of the efficiency of soil-reinforcement interface for reinforced embankments on soft clays is proposed.

RÉSUMÉ
Dans cet article, le comportement de remblai renforcé sur argile molle est exploré paramétriquement en utilisant un programme d'élément fini non linéaire. L'objectif de l'étude paramétrique était de relier la déformation latérale de la couche d'argile à la quantité de tension mobilisée dans le renforcement. Il a été trouvé que la grande tension dans le renforcement ne peut être mobilisée qu'en induisant une déformation latérale excessive dans la couche d'argile. Un critère original de servisabilité est proposé en se basant sur les résultats de l'étude paramétrique qui vise à limiter la déformation latérale de la fondation d'argile à l'orteil du remblai en limitant la tension mobilisée admissible dans le renforcement. En se basant sur ce critère, on propose un procédé simple pour l'évaluation de l'efficacité de l'interface de sol- renforcement pour les remblais renforcés sur les argiles molles.

1. INTRODUCTION

Without doubt the failure of a structure marks the end of its life but another limit worthy of consideration is the state of the structure, which marks the end of its useful life, even though the failure is not reached. Consideration should be given to the serviceability limit state of a structure beyond which it ceases to be capable of carrying out the function for which it was designed. A novel serviceability criterion is proposed for reinforced embankments on soft clay that aims to limit the lateral deformation of the clay foundation at the toe of the embankment by limiting the allowable mobilized tension in the reinforcement and ensure serviceability of the embankment. Numerous limit equilibrium-based design methods have been proposed by various authors to assess the stability of geotextile-reinforced embankments founded on soft clayey deposit (Low and Tang 1997, Hird et al. 1997; Palmeira et al. 1998). However, limit equilibrium-based design methods generally do not take into account the effect of system deformation on the soil-reinforcement interaction and neglect the redistribution of stresses in the embankment caused by the inclusion of reinforcement at the base of the embankment. A few researchers have proposed methods to limit the maximum allowable tensile strain in the reinforcement to ensure serviceability of the embankment (Rowe and Soderman 1985; Mylleville and Rowe 1991; Hinchberger and Rowe 2003). The maximum allowable tensile strain approach proposed by Rowe and Soderman (1985) has found widespread acceptance in the geotechnical community. The new design procedure based on the proposed serviceability criterion aims to improve upon the approach proposed by Rowe and Soderman (1985) by taking into account the rate of lateral deformation of the clay layer vis-à-vis the rate of embankment construction.

2. FINITE ELEMENT ANALYSES

The mechanism of soil-reinforcement interaction for a reinforced embankment on soft clay has been explored by conducting a parametric study using a coupled non-linear elasto-plastic finite element program (Crisp Consortium 2003). Figure 1 shows the details of the 2-D plane strain mesh used for the analyses. The elements representing the embankment were added in-place in six layers, each of 1 m thickness, to simulate its construction over a clay foundation. Beyond a height of 6 m, the increase in height of the embankment was simulated by placing a normal surcharge at the top of the embankment until failure of the clay foundation was achieved. The reinforcement was discretized using three-noded bar elements and interface elements were used to simulate clay-reinforcement and embankment-reinforcement interfaces. Interface elements were also used at the bottom horizontal boundary of the clay foundation as shown in Figure 2.

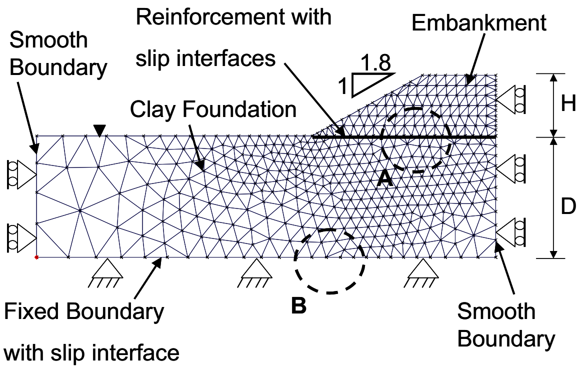

Figure 1. Finite element mesh and boundary conditions

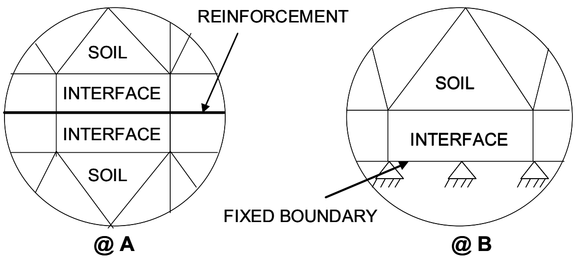

Figure 2. Finite element mesh - Details at the interfaces

2.1 Material Modelling

2.1.1 Clay Foundation

The clay foundation was modelled as an elastic-perfectly plastic material with a Tresca yield criterion and associated flow rule. Table 1 gives all the input parameters for this soil model. The value of undrained shear strength S_U required by this soil model can be obtained from simple shear test or in-situ vane shear test. Sharma (1994) used an S_U value equal to 0.75 times the S_U value obtained from a triaxial consolidated undrained (CU) test and was successful in obtaining reasonably accurate predictions of lateral and vertical deformation as well as excess pore pressure within the clay foundation. The stiffness of the clay foundation is represented by the undrained Young's modulus E_U that can be specified using the E_U/S_U ratio for the clay layer.

2.1.2 Sand Embankment

The sand embankment was modelled as elastic-perfectly plastic material with a Mohr-Coulomb yield criterion and associated flow rule. Table 1 gives all the parameters specified for this soil model. A small value of effective cohesion c' = 1 kPa was specified in order to avoid numerical instabilities at the toe of the embankment. The stiffness of the embankment was modelled as linearly increasing with the depth of the embankment with maximum stiffness E_o = 10000 kPa at the base of the embankment. No testing was done to establish the stiffness parameters in view of the fact that most of the embankment reaches yield condition when approaching full height (Hird et al. 1990).

2.1.3 Reinforcement and its interfaces with the soil

For all the analyses, a high stiffness geotextile was taken as the reinforcement. Linear elastic bar elements that are capable of taking only a tensile axial force were used to model the geotextile reinforcement. The soil-reinforcement interfaces were modelled using the six-noded quadrilateral interface elements based on an elastic-perfectly plastic model. The clay-reinforcement interface was considered undrained with shear strength equal to 0.8 times the undrained shear strength of clay (0.8 x S_{UO}). The sand-reinforcement interface was considered purely frictional, with friction angle equal to that of sand embankment. Table 1 gives the parameters specified for the reinforcement and Table 2 gives the interface parameters.

Table 1. Material parameters

Clay Foundation	Sand Embankment	Geotextile
E_U = 1425	E_o = 10000	E_r = 2.7E+6
ν = 0.49	ν = 0.3	ν = 0.35
S_{UO} = 8 to 20	c' = 1	A_r = 0.00212
γ = 16.3	γ = 16.0	
m_E = 425	Φ = 35	
m_C = 0 and 1		

Legend: E_U – Young's modulus of the clay foundation (kPa); ν – Poisson's Ratio; S_{UO} – Undrained Shear Strength of clay foundation at surface (kPa); S_U – Undrained Shear Strength of clay foundation (kPa); γ – Saturated unit weight (kN/m^3); m_E – Rate of increase of Young's modulus with depth; m_C – Rate of increase of cohesion with depth; E_o – Young's modulus of the sand embankment at the base of the embankment (kPa); Φ – angle of friction (degree); E_r – Young's modulus of the reinforcement (kPa); A_r – area of cross-section per meter width of the reinforcement (m^2/m); c' – effective cohesion (kPa)

Table 2. Interface parameters

Clay-geotextile	Sand-geotextile	Clay-bottom boundary
c = 0.8*S_{UO}	c = 0	c = 0.8*S_U
Φ = 0	Φ = 35	Φ = 0
K_n = 3500	K_n = 13000	K_n = 3500
K_s = 1000	K_s = 3800	K_s =1000
K_{sres} = 10	K_{sres} = 40	K_{sres} = 10
t = 0.04	t = 0.04	t = 0.04

Legend: c – cohesion (kPa); Φ – angle of friction (degree); K_n – normal stiffness of the slip element (kPa); K_s – shear stiffness of the slip element (kPa); K_{sres} – residual stiffness of the slip element after slip (kPa); t – thickness of slip element (m).

2.1.4 Interface at the Bottom Horizontal Boundary

The purpose of using interface elements at the bottom boundary was to limit the available shear strength in case of localization of shear strains at this boundary. True deformation mechanism is difficult to capture if the elements at the bottom of clay foundation are fixed. This is especially true in case of a shallow clay layer. In reality there exists an interface between the hard strata (or bed rock) and the clay foundation. This interface cannot possibly have strength greater than the undrained shear strength of clay at the corresponding depth. Including slip elements at this interface ensures limited shear strength should the failure surface goes through this interface. Hird et al. (1990) also used the similar interface elements at the boundary to model the collapse of reinforced embankment on soft clay. This interface is considered undrained with shear strength equal to 0.8 times the undrained shear strength of clay at the bottom. Table 2 lists the parameters specified for this interface.

2.1.5 Model Calibration

The initial model was calibrated by back analyzing the results of centrifuge tests conducted by Sharma (1994). The initial model was similar to the model shown in Figure 1. However, the left hand side boundary was closer to the toe of embankment to reflect the boundary conditions prevailing in the centrifuge models. The calibration was started by back-analyzing the control centrifuge test in which failure of an unreinforced embankment was observed at an embankment height of 5.1 m. The S_U value was adjusted in the back-analysis until a 5.1 m high unreinforced embankment was brought on the verge of failure. This S_U value was then used for other back-analyses involving reinforced embankments. It was found that the model gave reasonably accurate estimates of the deformation of the clay foundation as well as the magnitude and the distribution of excess pore water pressures in the clay foundation accurately.

3. DETAILS OF THE PARAMETRIC STUDY

In order to evaluate the effect of system deformation on soil-reinforcement interaction and the factors that affect the magnitude of mobilized tension in the reinforcement, the following parameters were studied:

– Depth of clay layer D, varied from 3 m to 24 m
– Undrained shear strength S_{UO} at the top of the clay layer, varied from 8 to 20 kPa
– Rate of increase of S_U with depth, m_C, taken either 0 (corresponding to no increase in S_U with depth) or 1 (corresponding to an increase in S_U at a rate of 1 kPa per meter with depth)

All of the above analyses were given a unique identifier. For example, analysis 8-14-1 corresponds to a model with D = 8 m, S_{UO} = 14 kPa and m_C = 1 kPa per meter depth.

3.1 Serviceability Criterion

The embankment height H was increased up to the point of failure for each of the analyses. For all the analyses, it was found that after a certain height of embankment (denoted by $H_{CRITICAL}$) is reached, onset of plastic yielding in clay foundation takes place, resulting in increased lateral deformation of the clay foundation near the toe of the embankment. If the embankment construction was continued beyond $H_{CRITICAL}$, the embankment approached failure rapidly. To capture the above-mentioned behaviour of the embankment, a criterion that characterizes the serviceability-limit state of the embankment is proposed. The basic idea behind the proposed serviceability criterion is that if the embankment is to remain serviceable, the rate of lateral displacement at the toe of the embankment with respect to the rate of embankment construction must be kept below a certain value. The serviceability criterion is defined in terms of a dimensionless parameter R:

$$R = \Delta (\delta_h) / \Delta H \qquad [1]$$

In Equation 1, $\Delta (\delta_h)$ is the change in the lateral displacement at the toe in response to a change in the height of the embankment (ΔH). Typical plot between H and R can be seen from Figure 3 showing the result of the analysis 14-14-1. It can be seen from Figure 3 that beyond an R-value of 0.1, there is no appreciable change in the height of the embankment but there is a significant increase in the horizontal displacement at the toe. For the analysis 14-14-1, for R = 0.1, $H_{CRITICAL}$ is equal to 7 m and mobilized tension in the reinforcement (T_{MOB}) is equal to 125 kN/m. It is proposed that the embankment remains serviceable at R-values of $\leq$ 0.1.

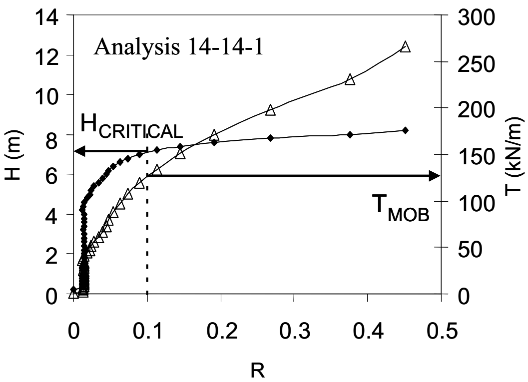

Figure 3. $H_{CRITICAL}$ and T_{MOB} vs. R

4. DEFORMATION MECHANISMS

4.1 Deformation Mechanism for Deep Clay Layer

Figures 4 and 5 show the displacement vectors and the contours of maximum shear strain for the analysis 24-14-1 with embankment constructed up to a height of 8 m. It can be seen from Figures 4 and 5 that the clay foundation has nearly failed by undergoing a rotational slip due to the construction of the embankment. Shear strains in excess of 30% are seen to occur along the circular slip surface.

The distribution of shear stresses at the clay-reinforcement interface for different $H/H_{CRITICAL}$ ratios is shown in Figure 6. A $H/H_{CRITICAL}$ ratio greater than 1 signifies that the embankment has been constructed to height beyond the critical height of the embankment. Such a failure mechanism is less than ideal for the mobilization of tension in the reinforcement as only the portion of the reinforcement closer to the centre of the embankment outside the slip surface mobilizes significant tension. It can be clearly seen from Figure 6 that the shear stresses at the interface were much less than the available shear strength at the interface.

4.2 Deformation Mechanism for Shallow Clay Layer

Figures 7 and 8 show the displacement vectors and the contours of maximum shear strain for the analysis 3-14-1 with embankment constructed up to a height of 11 m. It can be seen from Figures 7 and 8 that the developing rotational slip failure is interrupted by the boundary between the clay foundation and the stiff bedrock. As a result, the mode of failure of the clay foundation is similar to that observed in a direct simple shear test.

The distribution of shear stresses at the clay-reinforcement interface for different $H/H_{CRITICAL}$ ratios is shown in Figure 9. The clay layer is getting squeezed from underneath the embankment and therefore, it is imposing considerable frictional drag on the reinforcement at the clay-reinforcement interface. It can be clearly seen from Figure 9 that the available shear strength at the interface has been completely mobilized on the completion of embankment. Such a mechanism is ideal for efficient mobilization of tension in the reinforcement as almost the entire length of the reinforcement participates in the tension mobilization process. For such cases, it is generally not a problem to keep the embankment serviceable. Therefore, only the cases showing rotational failure of clay foundation will be considered for subsequent presentation and analysis of results.

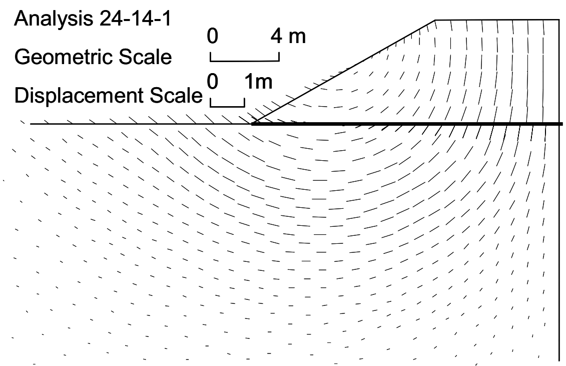

Figure 4. Deep Clay Layer – Displacement vectors

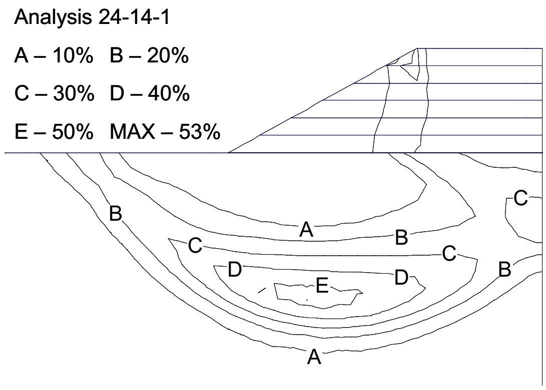

Figure 5. Deep Clay Layer – Shear Strain Contours

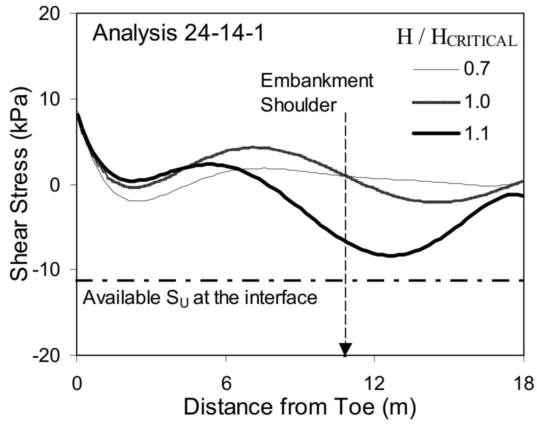

Figure 6. Deep Clay Layer – Shear stresses at clay-reinforcement interface

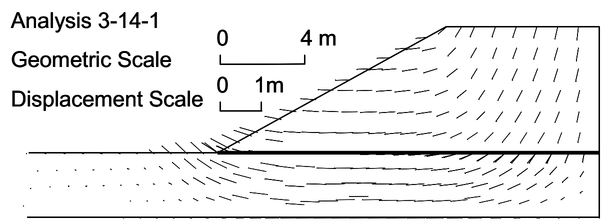

Figure 7. Shallow Clay Layer – Displacement vectors

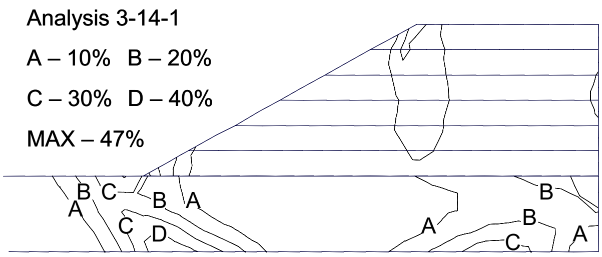

Figure 8. Shallow Clay Layer – Shear Strain Contours

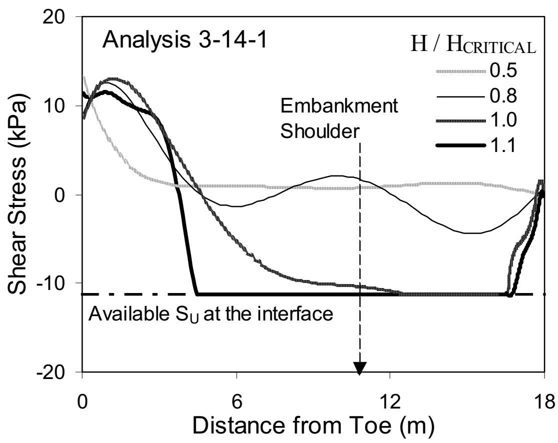

Figure 9. Shallow Clay Layer – Shear stresses at clay-reinforcement interface

5. SERVICEABILITY-BASED DESIGN PROCEDURE

The critical height of the embankment ($H_{CRITICAL}$) was obtained for the various parametric analyses, by employing the previously discussed serviceability criterion i.e., limiting the parameter R to a value of 0.1. This $H_{CRITICAL}$ corresponds to the height of reinforced embankment which can be constructed over a clay deposit of a given undrained shear strength, without causing serviceability problems. Further for each of the parametric analysis T_{MOB} corresponding to $H_{CRITICAL}$ was obtained.

5.1 Estimation of $H_{CRITICAL}$

Figures 10 and 11 show the plot between S_{UO} and $H_{CRITICAL}$ for the cases m_C = 0 (undrained shear strength constant with depth) and m_C = 1 (undrained shear strength increasing linearly with depth), respectively. Figures 10 and 11 can be used as design charts. Based on the known undrained shear strength of the clay foundation at the clay-reinforcement interface and the rate at which undrained shear strength increases with depth, corresponding $H_{CRITICAL}$ can be obtained from either Figure 10 or Figure 11.

5.2 Estimation of T_{MOB}

Figures 12 shows the plot between the critical height of the embankment ($H_{CRITICAL}$) and the tension mobilized in the reinforcement (T_{MOB}) for both cases m_C =0 and m_C = 1. Interestingly, a unique relationship was obtained between $H_{CRITICAL}$ and T_{MOB}, which can be attributed to the limited available shear strength at the clay-reinforcement interface. Figure 12 can also be used as a design chart. For the critical height of the embankment estimated using Figure 10 or Figure 11, one can estimate the tension that can be mobilized safely in the reinforcement for a clay foundation depth greater than or equal to 4 m using Figure 12.

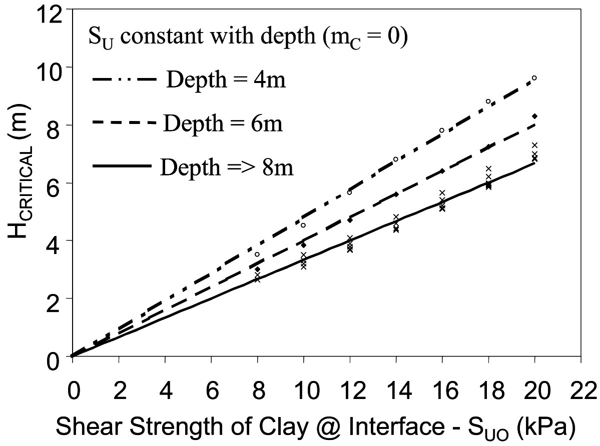

Figure 10. S_{UO} vs. $H_{CRITICAL}$ for case m_C = 0

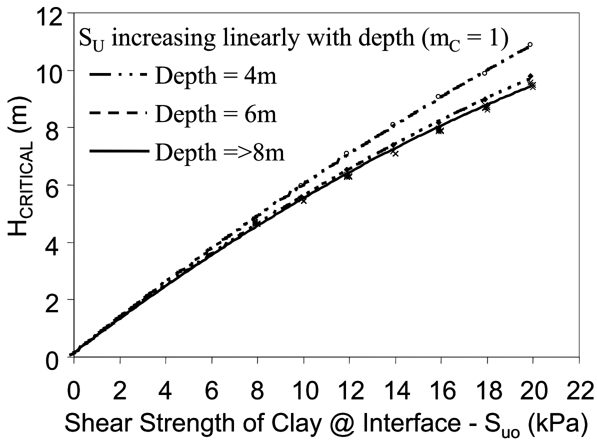

Figure 11. S_{UO} vs. $H_{CRITICAL}$ for case m_C = 1

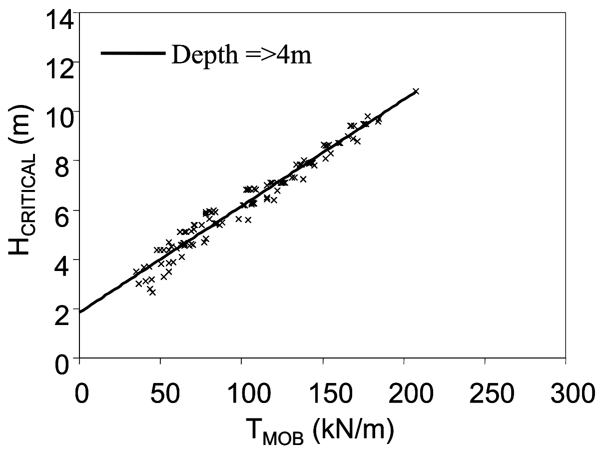

Figure 12. $H_{CRITICAL}$ vs. T_{MOB} (for both m_C = 0 and m_C = 1)

5.3 An Example

Suppose an 8 m high embankment with 1.8H: 1V is to be constructed over an 18 m deep clay deposit using high stiffness geotextile reinforcement. The undrained shear strength of clay deposit at the surface is 12 kPa and the rate of increase of undrained shear strength with depth is 1 kPa/m. The fill parameters are γ = 16 kN/m^3 and Φ = 35^0. Following the serviceability-based design approach,

– Obtain $H_{CRITICAL}$: From Figure 11, for the given undrained shear strength, $H_{CRITICAL}$ is equal to 6.5 m.
– Estimate T_{MOB}: Corresponding to $H_{CRITICAL}$ = 6.5 m, the T_{MOB} can be estimated from Figure 12, which is 110kN/m.

Hence to ensure serviceability of the embankment, the embankment height should not be increased beyond 6.5 m, which is the critical height of the embankment. Even with a high stiffness geotextile in place, maximum safe mobilized tension in the reinforcement would only be 110kN/m, which can be attributed to limited available shear strength at the clay-reinforcement interface. Hence to achieve the desired embankment height, one would first need to incorporate a ground improvement technique that would increase the available shear strength of clay foundation (Leroueil et al. 1990).

6. CONCLUSIONS

The parametric study has revealed that the soil-reinforcement interaction mechanism depends on the ratio of embankment height to the depth of the clay layer (H/D ratio). In the case of an embankment constructed over a deep clay deposit, the failure mechanism is more like a slip circle and there is poor mobilization of tension in the reinforcement. However, if the embankment is constructed over a shallow clay layer, the failure mode is that of direct simple shear and almost the entire length of the reinforcement participates in the mobilization of tension. In this case, the tension mobilized in the reinforcement is higher and therefore, the contribution of the reinforcement towards overall stability of the embankment is greater. The results presented in the paper have shown that there is a direct relationship between the increment in embankment height and the rate of increase in horizontal displacement near the toe. A unique relationship can be established between the critical height of the embankment and the tension mobilized in the reinforcement, which can be attributed to the limited available shear strength at the clay-reinforcement interface. Based on a serviceability-based criterion, simple and versatile design procedure for reinforced embankments on soft clay has been proposed for a high stiffness geotextile reinforcement and for a given embankment geometry. The reader should note that the design procedure is not yet complete. The authors are conducting further analyses to establish the effect of reinforcement stiffness and embankment geometry on $H_{CRITICAL}$ and $T_{MOB.}$

7. REFERENCES

CRISP Consortium (2003). SAGE CRISP Version 4.0 – User's and Technical Manuals. CRISP Consortium Ltd., http://www.mycrisp.com/.

Hird C.C., Pyrah I.C. and Russell D. 1990. Finite element analysis of the collapse of reinforced embankments on soft clay. Géotechnique, Vol. 40(4), pp. 633-640.

Hird C.C., Russell D. and Jewell R.A. 1997. Stability of single-stage constructed reinforcement embankment on soft clay. Proceedings of the Institution of Civil Engineers, Geotechnical Engineering, Vol. 125(4), pp. 191-205.

Hinchberger S.D. and Rowe R.K. 2003. Geosynthetic reinforced embankments on soft clay foundations: Predicting reinforcement strains at failure. Geotextiles and Geomembranes, Vol. 21(3), pp. 151-175.

Leroueil S., Magnan J.P, and Tavenas F.1990. Embankments on soft clay. Ellis Horwood Limited, U.K.

Low B.K. and Tang W.H. 1997. Reliability analysis of reinforced embankments on soft ground. Canadian Geotechnical Journal, Vol. 34(5), pp. 672-685.

Mylleville R.K. and Rowe R.K. 1991. On the design of reinforced embankments on soft brittle clays. Geosynthetics '91 Conference, Atlanta, USA.

Palmeira E.M., Pereira J.H.F. and da Silva A.R.L. 1998. Back analyses of geosynthetic reinforced embankments on soft soils. Geotextiles and Geomembranes, Vol. 16, pp. 273-292.

Rowe R.K., and Soderman K.L. 1985. An approximate method for estimating the stability of geotextile-reinforced embankments. Canadian Geotechnical Journal, Vol. 22, pp. 392-398.

Sharma J.S. 1994. Behaviour of reinforced embankments on soft clay. Ph.D. thesis, Cambridge University, U.K.