THE EFFECT OF PVDs AND REINFORCEMENT ON THE BEHAVIOUR OF EMBANKMENTS ON SOFT RATE-SENSITIVE SOILS

C. Taechakumthorn and R. Kerry Rowe
GeoEngineering Centre at Queen's – RMC
Department of Civil Engineering, Queen's University, Canada

ABSTRACT
The effect of the use of prefabricated vertical drains (PVDs) and geosynthetic reinforcement on the behaviour of reinforced embankments over rate-sensitive soil are examined. An elasto-viscoplastic constitutive model coupled with Biot consolidation theory is utilized. For rate-sensitive soil, the faster the construction, the higher the short-term mobilized strength of the soil; however, this strength is lost with subsequent time. Geosynthetic reinforcement improves the stability and minimizes the shear induced deformation of the reinforced embankment and foundation soil. PVDs not only reduce the long-term mobilized strain but also work together with reinforcement to minimize differential settlement. The rate-sensitive shear strength and the increase in shear strength due to pore pressure dissipation are discussed.

RÉSUMÉ
L'effet de l'utilisation de la verticale préfabriquée vidange (PVDs) et le renfort geosynthetic sur le comportement du sol rate-sensitive renforcé d'excédent de remblais sont examinés. Un modèle constitutif elasto-viscoplastic couplé à la théorie de consolidation de Biot est utilisé. Pour le sol rate-sensitive, plus la construction est rapide, plus la force mobilisée à court terme du sol est haute ; cependant, cette force est perdue avec du temps suivant. Le renfort de Geosynthetic améliore la stabilité et réduit au minimum la déformation induite par cisaillement du sol renforcé de remblai et de base. PVDs réduisent non seulement la contrainte mobilisée à long terme mais collaborent également avec le renfort pour réduire au minimum le règlement différentiel. La résistance au cisaillement rate-sensitive et l'augmentation de la résistance au cisaillement due à la dissipation de pression de pore sont discutées.

1 INTRODUCTION

The behaviour of reinforced embankment constructed on typical soft soils has been extensively studied. However, the effects of the viscous behaviour of a rate-sensitive foundation on the short-term and long-term performances of reinforced embankments has only received limited attention. A study by Rowe et al. (1996) of the time-dependent behaviour of the Sackville test embankment showed that in order to accurately predict the response of the reinforced embankment on this rate-sensitive soil, it was necessary to use a constitutive model that considers the viscous behaviour of the soft soil deposit. Rowe and Hinchberger (1998) proposed an elasto-viscoplastic constitutive model and demonstrated that the proposed model could adequately describe the field behaviour of the test embankment. Rowe and Li (2002) showed that the long-term stability of reinforced embankments on rate-sensitive soil decreased after the end of construction because of the delayed excess pore water pressure build up as a result of soil viscosity. The installation of prefabricated vertical drains (PVDs) has the potential to reduce the effects of delayed excess pore water pressure. However, the effects of PVDs on the performance of reinforced embankment on rate-sensitive soils have not been studied in the literature

The objective of this paper is to perform a parametric study of the combined effects of reinforcement and PVDs on the behaviour of embankments on soft rate-sensitive soil. The short-term stability of the embankments is compared with the result from a conventional elasto-plastic constitutive model. The influence of factors such as the stiffness of reinforcement, rate of construction and spacing of PVDs are examined with respect to the time-dependent responses of reinforcement strains, differential settlement and lateral deformation of the embankment.

2 FINITE ELEMENT MODELING

The finite element program AFENA (Carter and Balaam, 1990), previously modified by Rowe and Hinchberger (1998) to incorporate the elasto-viscoplastic constitutive model, was adopted in this study. Drainage elements (Russell, 1990) and implemented by Li and Rowe (2001) was utilized to investigated the effect of PVDs. The results presented herein were obtained from embankments with 2H:1V side slopes overlaying 15 m of soft rate-sensitive clay above the rigid and permeable sand layer. A typical mesh is shown in Figure 1.

2.1 Details of Mesh Discretisation

The finite element mesh included a total of 1815 six-noded linear strain triangular elements, with 4003 nodes to discretise the embankment and foundation soils. Two-noded bar elements were used to model the reinforcement and two-noded interface joint elements (Rowe and Soderman, 1985) were used to model the fill/reinforcement and fill/foundation interfaces. For PVDs modeling, two-noded drainage elements (Li, 2000) were utilized.

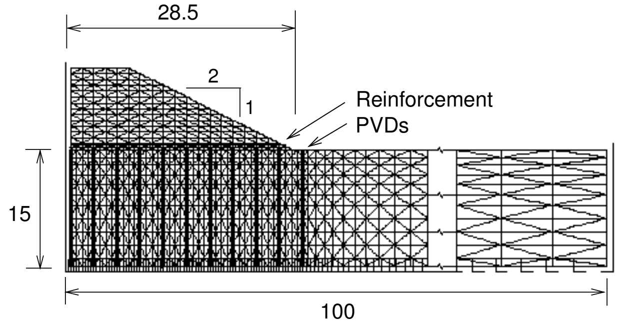

Figure 1. Finite elements mesh discretisation.

The centerline of the embankment and far field boundary, located 100 m away from centerline, were taken to be smooth-rigid boundaries while the bottom boundary of the finite element mesh was assumed to be free draining and rough-rigid. The embankment construction was simulated by gradually turning on the gravity of the embankment in 0.75 m thick lifts at a rate corresponding to the construction rate of the embankment. The PVDs were fully penetrating in a square pattern with three different spacings S = 1, 2, and 3 m and zero excess pore water pressure were assumed along drainage elements.

2.2 Elasto-Viscoplastic Constitutive Model

The elasto-viscoplastic model, proposed by Rowe and Hinchberger (1998), is fully coupled with Biot consolidation (Biot, 1941). This model based on Perzyna's theory of overstress viscoplasticity (Perzyna, 1963), an elliptical yield cap model (Chen and Mizuno, 1990), a Drucker-Prager failure envelope, and concepts of the critical state framework (Roscoe and Schofield. 1963). The governing equation was expressed in terms of the strain rate tensor:

$$\dot{\varepsilon}_{ij} = \frac{\dot{S}_{ij}}{2G} + \frac{1}{3K}\dot{\sigma}_{ii} + \gamma^{vp}\langle\phi(F)\rangle\frac{\partial f}{\partial \sigma_{ij}} \quad [1]$$

where S_{ij} is deviatoric stress; G is shear modulus; σ_{ii} is sum of principle stresses; K is bulk modulus; γ^{vp} is the viscoplastic fluidity parameter and ø(F) is a flow function that can be expressed in term of overstress as:

$$\phi(F) = \left(\frac{\sigma'^{(s)}_{my} + \sigma'^{(d)}_{os}}{\sigma'^{(s)}_{my}}\right)^n - 1 \quad [2]$$

where $\sigma'^{(d)}_{os}$ is overstress; n is strain rate exponent and f is plastic potential function. The elastic bulk modulus K and shear modulus G are functions of mean effective stress as shown below:

$$K = \frac{1+e}{\kappa}\sigma'_m \quad [3]$$

$$G = \frac{3(1-2v')K}{2(1+v')} \quad [4]$$

where e is the void ratio; κ is the recompression index; σ'_m is mean effective stress and v' is Poisson's ratio.

2.3 Drainage Elements

The capture all the details of a system involving PVDs a full 3-D analysis would be required. However it has been shown that with suitable approximations to model the vertical drains, a 2D analysis can give good results. Specifically, a technique for using the average degree of consolidation on the horizontal plane, U_h, at depth z and time t under instantaneous loading was proposed by Hansbo (1981) and Hird et al. (1992) for axisymmetric and plane strain conditions, respectively.

Hird et al. (1992) suggested that in order to match the average degree of consolidation at any time and depth in these two conditions, the average degree of consolidation can be simply set equal to each other. This can be achieved by geometric matching, permeability matching or a mix of geometric and permeability matching. In previous work by Li and Rowe (2001), the permeability matching scheme was adopted by modifying the horizontal hydraulic conductivity of soil and equivalent discharge capacity of vertical drains in plane strain conditions as:

$$k_{pl} = \frac{2k_{ax}}{3\left[\ln\left(\frac{n}{s}\right) + \left(\frac{k_{ax}}{k_s}\right)\ln(s) - \frac{3}{4}\right]} \quad [5]$$

$$Q_w = \left(\frac{2}{\pi R}\right)q_w \quad [6]$$

$$n = \frac{R}{r_w}, \quad s = \frac{r_s}{r_w} \quad \text{and} \quad q_w = \pi k_w r_w^2 \quad [7]$$

where: k_{pl}, k_{ax}, k_s and k_w are the hydraulic conductivity of soil in the horizontal direction for plane strain condition, for axial symmetric condition, soil in the smear zone, and the vertical drain respectively; r_w, r_s and R are the radius of the vertical drain, smear zone and influence zone, respectively; Q_w and q_w are the equivalent discharge capacity for the plane strain and axisymmetric unit cell, respectively.

3 CONSTITUTIVE PARAMETERS

3.1 Soft rate-sensitive soil properties

The soft rate-sensitive soil examined is denoted here as soil CR1. Constitutive parameters used for soil CR1 are similar to the estimated soil properties at the Sackville test embankment (Rowe and Hinchberger, 1998). The various parameters for CR1 are listed in Table 1.

The hydraulic conductivity of soft rate-sensitive clay was taken to be a function of void ratio as:

$$k_v = k_{vo} \exp\left(\frac{e - e_o}{C_k}\right) \qquad [8]$$

where k_{vo} is the reference hydraulic conductivity (2×10^{-9} m/s); e_o is the reference void ratio (e_o = 1.5) and C_k is hydraulic conductivity change index (C_k = 0.2). The hydraulic conductivity was considered to be anisotropic with k_h/k_v = 4.

Table 1. Details of Foundation Soil Properties.

Soil Parameter	Soil CR1
Failure envelope, $M_{N/C}$ (ø')	0.96 (29^o)
Cohesion intercept, c_k (kPa)	0
Failure envelope, $M_{O/C}$	0.75
Aspect ratio, R	1.25
Compression index, λ	0.16
Recompression index, κ	0.034
Coefficient of at rest earth pressure, K_o'	0.75
Poisson's ratio, υ	0.3
Unit weight, γ (kN/m^3)	17
Initial void ratio, e_o	1.50
Viscoplastic fluidity, γ^{vp} (1/hour)	2.0 x 10^{-5}
Strain rate exponent, n	20

3.2 Backfill Properties and Construction Rate

The purely frictional granular soil is used to model the embankment fill. The assumed properties are friction angle ø' = 37^o, dilation angle ψ = 6^o and unit weight γ = 20 kN/m^3. The non-linear elastic behaviour of the fill was modeled using Janbu's (1963) equation:

$$\frac{E}{P_a} = K\left(\frac{\sigma_3}{P_a}\right)^m \qquad [8]$$

where E is the Young's modulus; P_a is the atmospheric pressure; σ_3 is the minor principle stress and K and m are material constants selected as 300 and 0.5, respectively. The construction rates of two cases examined in this study were 2 m/month and 6 m/month.

3.3 Interface parameters

Rigid-plastic joint elements (Rowe and Soderman, 1985) were used to model the fill/reinforcement and fill/foundation interface. The fill/reinforcement interface was assumed to frictional with ø' = 37^o. The fill/foundation interfaces had the same shear strength as the foundation soil at ground surface.

Reinforcement with tensile stiffness, J, of 0 (no reinforcement), 250, 500, 1000 and 2000 kN/m were examined

3.4 PVDs properties

The PVDs in this study are modeled to have typical rectangular cross section of 100 mm × 4 mm (Holtz, 1987) and that equivalent to a 66 mm circular drain. The PVDs spacing, S, of 1, 2 and 3 examined are in the typical range used in practice (Holtz, 1987). In this study, the hydraulic conductivity of soil in smear zone was assumed to be isotropic and same as vertical hydraulic conductivity. The effect of this assumption will be examined in a subsequent paper.

4 RESULTS AND DISCUSSIONS

The effects of various parameters such as soil viscosity, construction rate, reinforcement stiffness and PVDs spacing on short-term and long-term behaviour of the reinforced embankment on the soft rate-sensitive soil are presented in the following subsections.

4.1 The effects of soil viscosity and construction rate on the short-term stability of embankments

The short-term stability of an embankment can be assessed in terms of failure height of the embankment. The failure height of (reinforced and unreinforced embankment) can be defined as the height of fill at which any attempt to increase the fill materials will not result in an increase in the net embankment height (actual height above the original ground surface). This failure height is obtained by plotting the relationship between the net embankment heights (fill thickness minus settlement) against the fill thickness.

Figure 2 shows the effect of rate-sensitive soil on the stability of an embankment. Case I results are for a non-viscous soil (NV Soil) and were obtained using a conventional elasto-plastic model. Case II results show the effect of soil viscosity and were obtained using the elasto-viscoplastic model. The results show that the rate-sensitive soil (all other things but viscosity being equal) had significant higher short-term failure height (5.8 m for

Case II compared to 3.6 m for Case I) because the viscosity of the soil provided extra short-term strength at the initial stage of loading.

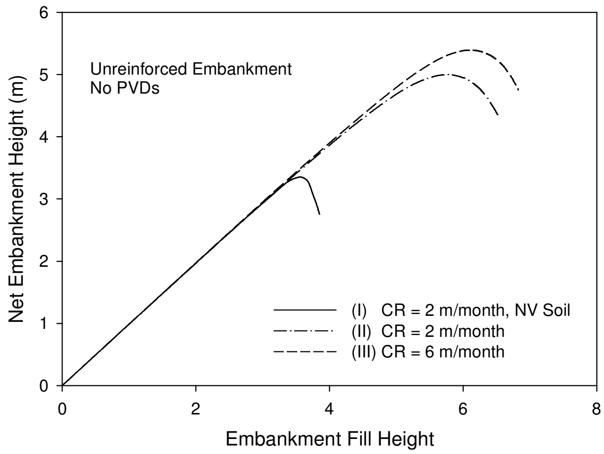

Figure 2. The effect of soil viscosity and construction rate on the short-term embankment stability

For the rate-sensitive soil, the faster loading rate (6 m/month) resulted in higher short-term shear strength of the soil and hence high short-term stability than the slower (2 m/month) loading rate (Figure 2). The short-term failure height in Case III (6.08 m), was almost 0.3m higher than that for Case II. However, this short-term strength decreased with time as will be discussed later.

4.2 The effects of reinforcement stiffness on the short-term stability of the embankment

The main function of geosynthetic reinforcement is to reduce outward shear stress on the foundation soil, and if stiff enough, induce inward shear stresses of the soil and in so doing increase the stability of the embankment.

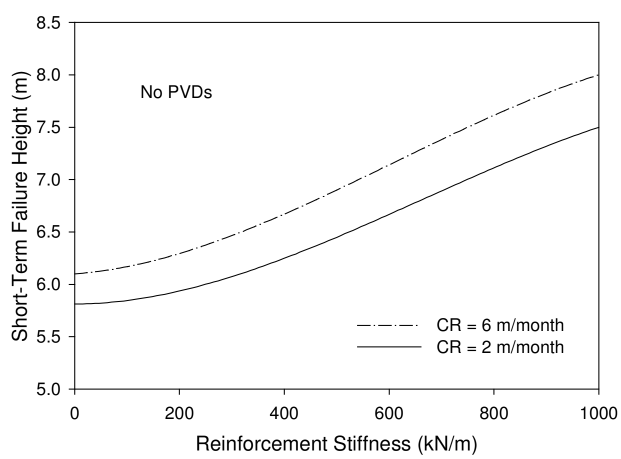

Figure 3. The effect of reinforcement stiffness on the short-term embankment stability

As shown in Figure 3, the effects of the short-term failure height increased with increasing the reinforcement stiffness. The benefit of the reinforcement was very small for stiffnesses less that 200 kN/m but was quite significant for reinforcement stiffnesses greater than 400 kN/m. The effects of construction rate are also evident from Figure 3 with the faster construction rate giving a higher short-term embankment failure height for all cases.

4.3 The effects of prefabricated vertical drains on the short-term stability of the embankment

The presence of PVDs provides a short horizontal drainage path for the soft clay and takes the advantage of higher hydraulic conductivity in horizontal direction to accelerate the dissipation of the excess pore water pressures in the foundation. This can be expected to increase the rate of strength gain due to the consolidation in the normally consolidated soil.

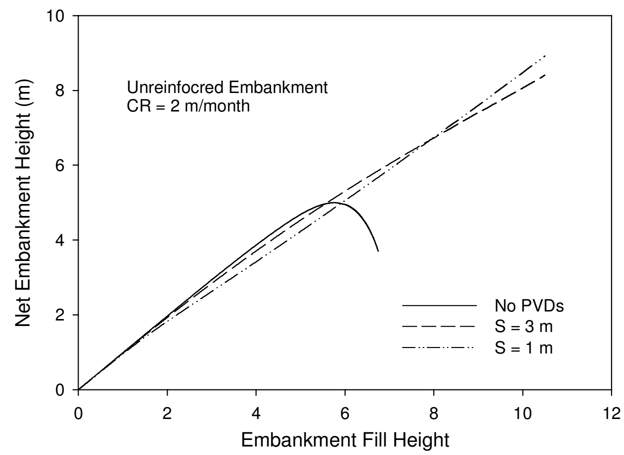

Figure 4. The effect of PVDs on the short-term embankment stability

Figure 4 demonstrates the effect of PVDs and PVD spacing. The short-term failure of the reinforced embankment with the reinforcement stiffness of 500 kN/m without the use of PVDs (Case I) was 5.8 m. When PVDs are employed, the short-term stability of the reinforced embankment was improved substantially and there was no failure even when the fill was built up to 10.5 m.

During the initial stage of construction, the smaller PVD spacing (S = 1 m) gave larger settlement due to the higher degree of partial consolidation during the construction. However, this higher degree of partial consolidation resulted in smaller overstress in the soil and accordingly less viscoplastic deformation was generated. Thus for S = 1 m, as the fill thickness approached 8.0 m, the net embankment became higher than that for S = 3 m.

4.4 The effects of soil viscosity on the long-term reinforcement strain

Three 5 m high reinforced embankments were numerically constructed on the rate-sensitive soil to illustrate the effect of viscosity on reinforcement strain both with and without PVDs and the effect of construction rate (Figure 5).

As mention earlier, a rate-sensitive soil will exhibit higher short-term strength as the rate of loading is increase, however, the soil strength will decrease with time. The results from Case I and Case II (Figure 5) show the effect of the construction rate on the long-term reinforcement strain. The reinforcement strains at the end of the construction were 2.6% and 1.9% for Cases I and II, respectively. At the end of the construction, the reinforcement strain in Case II (slower construction rate) was higher because soil exhibited lower short-term strength and transferred more load to the reinforcement. In the longer-term, however, this slower construction rate also allowed a higher degree of partial consolidation and reduced the amount of overstress in the soil. Consequently there was less creep and stress relaxation in the soil following construction and this resulted in smaller long-term reinforcement strains.

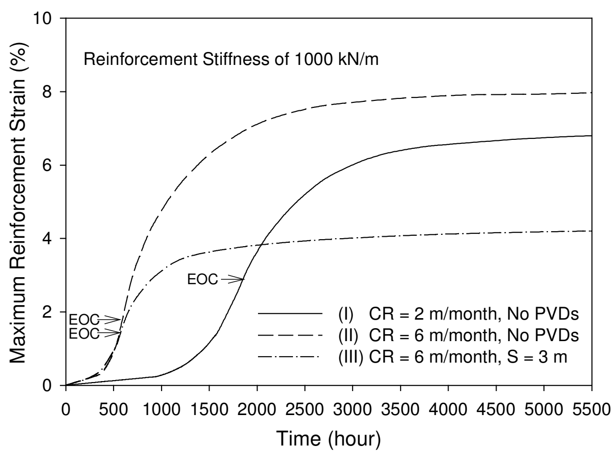

Figure 5. The effect of construction rate and PVDs on the long-term mobilized reinforcement strain

Designers commonly aim to limit reinforcement strains to 5% - 6% (Rowe and Li 2005). The results for Cases I and II correspond to long-term reinforcement strains of 6.9% and 8.0%, respectively and hence exceed typical desirable limits. The rate of excess pore water dissipation and the consequent the rate of shear strength gain in the soil can be increased using PVDs. The result for Case III show that with the use of PVDs, the long-term reinforcement strain could be limited to only 4.2% even at the fastest construction rate. Thus either a higher reinforced embankment could be achieved or reinforcement with less stiffness could be used while maintaining a reinforcement strain below 5.0 %.

4.5 The effects reinforcement and PVDs on the differential settlement and lateral deformations

Reinforcement can significantly reduce differential settlement and heave of the foundation for embankments on rate-sensitive soil. Figure 6 shows the ground surface profiles for embankments with different reinforcement stiffnesses at 27 days after the end of construction. For the case of an unreinforced embankment (J = 0 kN/m), the differential settlement between center and crest of the embankment was 0.8 m but for the reinforced embankment, this was reduced to 0.5 and 0.4 m for reinforcement stiffness of 500 and 1000 kN/m (Case II and III) respectively. The maximum calculated heaves were 1.5, 0.9, and 0.7 m for the unreinforced embankment and for the reinforcement stiffness of 500 and 1000 kN/m, respectively. The presence of PVDs considerably reduced the differential settlement of the foundation. The results from Case IV show that with the use of PVDs, even with the less stiff reinforcement, the differential settlement was reduced to 0.2 m.

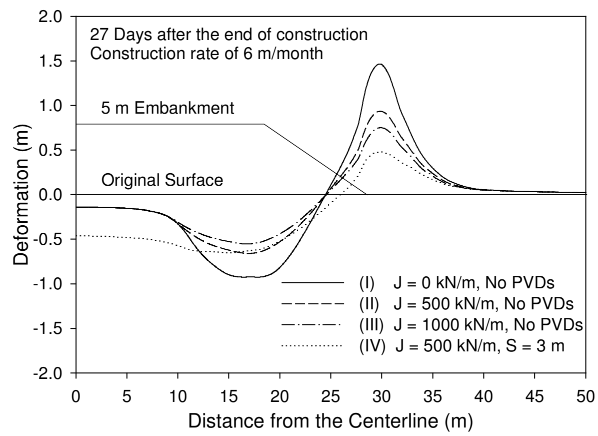

Figure 6. The effect reinforcement stiffness and PVDs on differential settlement

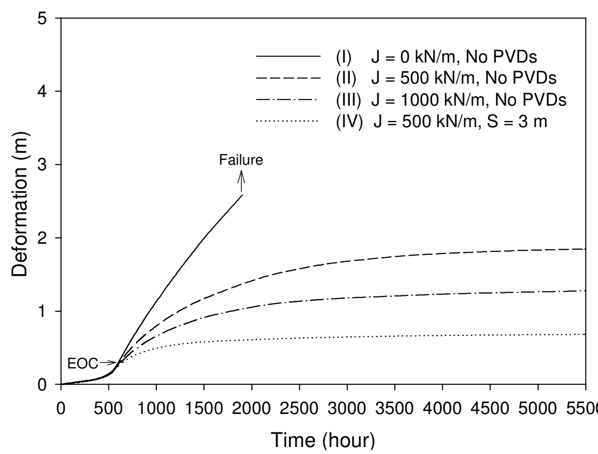

Figure 7. The effect reinforcement stiffness and PVDs on the lateral deformation at the embankment toe

Reinforcement also had a benefit on reducing the lateral deformation of the embankment. Figure 7 shows that without reinforcement, there was excessive movement at the toe of the embankment and eventually failure. The maximum lateral toe deformations of the embankment was reduced from 1.9 m, for reinforcement stiffness of 500 kN/m, to 1.2 for reinforcement stiffness of 1000 kN/m. Figure 7 also shows the effect of PVDs on the lateral deformation. With a reinforcement stiffness of 500 kN/m combined with the use of PVDs there was only 0.7 m lateral toe movement. This was smaller than obtained for Case III using a reinforcement stiffness of 1000 kN/m because of higher pore pressure dissipation and consequently higher soil strength and less overstress of the soil that occurs with the use of PVDs.

5 SUMMARY AND CONCLUSIONS

The faster the rate of construction, the higher the short-term stability of embankments on a rate-sensitive foundation soil. However the consequent larger overstress generated in the soil resulted in larger viscoplastic deformations. Geosynthetics reinforcement reduces the shear stress in the foundation and hence improves embankment stability. The use of PVDs increased the degree of partial consolidation during the construction and the short-term stability of the embankment. The long-term reinforcement strain was affected by the rate of construction. The slower construction rate resulted in less overstress in the soil for given embankment fill thickness and this resulted in less viscoplastic deformation of the soil. The use of PVDs not only increased the rate of excess pore water pressure dissipation but also decreased the effects of overstress and the long-term reinforcement strain. The differential settlement and the toe lateral deformation of the embankment were minimized by using reinforcement and the effect could be enhanced using PVDs.

This study demonstrates that the behaviour of rate-sensitive soil can have significant effects on the engineering behaviour of the reinforced embankment, especially after the end of construction. Therefore, the viscosity and viscoplastic characteristic of the soil should be considered in the design and construction of earth structures on the rate-sensitive soil.

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REFERENCES

Biot, M.A. 1941. General theory of three-dimensional consolidation. Journal of Applied Physics. 12: 155-164.

Carter, J.P., & Balaam, N.P. 1990. *AFENA– A general finite element algorithm: Users manual.* N.S.W: School of Civil Engineering and Mining Engineering, University of Sydney, Australia.

Chen, W.F., and Mizuno, E. 1990. Nonlinear Analysis in Soil Mechanics: Theory and Implementation. Elsevier Science Publishing Company Inc., New York, U.S.A.

Hansbo, S. 1981. Consolidation of fine-grained soils by prefabricated drains. Proceedings of the 10th International Conference on Soil Mechanics and Foundation Engineering, Stockholm, 3: 677-682.

Hird, C.C., Pyrah, I.C., Russell, D., and Cinicioglu, F. 1995. Modelling the effect of vertical drain in two-dimensional finite element analyses of embankment on soft ground. Canadian Geotechnical Journal, 32: 795-807.

Holtz, R.D. 1987. Preloading with prefabricated vertical strip drains. *Geotextiles and Geomembranes* 6(1-3): 109-131.

Janbu, N. 1963. Soil compressibility as determined by oedometer and triaxial tests. *Proc. of the European Conf. on Soil Mechanics and Foundation Engineering*, Wiesbaden, Germany. 1:19-25.

Li, A.L. 2000. *Time dependent behaviour of reinforced embankments on soft foundations*. Ph.D. thesis, London: The University of Western Ontario, Canada.

Li, A.L., & Rowe, R.K. 2001. Combined effects of reinforcement and prefabricated vertical drains on embankment performance. *Canadian Geotechnical Journal* 38: 1266-1282.

Roscoe, K.H., and Schofield, A.N. 1963. Mechanical behaviour of an idealised "wet clay". Proceedings of the Second European Conference on Soil Mechanics: 47-54.

Rowe, R.K. & Soderman, K.L. 1985. An approximate method for estimating the stability for geotextiles reinforced embankments. *Canadian Geotechnical Journal* 22(3): 392-398.

Rowe, R.K., & Hinchberger, S.D. 1998. The significance of rate effects in modelling the Sackville test embankment. *Canadian Geotechnical Journal* 33: 500–516.

Rowe, R.K., & Li, A.L. 2002. Behaviour of reinforced embankments on soft rate sensitive soils. *Geotechnique* 52(1): 29-40.

Rowe, R.K. & Li, A.L. 2005. Geosynthetic-reinforced embankments over soft foundations, *Geosynthetics International* 12(1):50-85.

Rowe, R.K., Gnanendran, C.T., Landva, A.O., & Valsangkar, A.J. 1996. Calculated and observed behaviour of a reinforced embankment over soft compressible soil. *Canadian Geotechnical Journal* 33: 324–338.

Russell, D. 1990. An element to model thin, highly permeable materials in two dimensional finite element consolidation analyses. *Proc. of the 2nd European Specialty Conf. on Numerical Method in Geotechnical Engineering*, Santander, Spain: 303-310.