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The effect of soil viscosity on the behaviour of reinforced embankments

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ABSTRACT: For conventional soils, a slower construction rate leads to higher embankment stability. In contrast, rate-sensitive soils show higher short-term embankment stability with a faster construction rate; however this may also give rise to long-term problems. For example, rate-sensitive soils exhibit significant creep induced deformations and this can contribute to the development of excessive long-term reinforcement strains. Delayed creep induced excess pore water pressures may also give rise to minimum stability some time after the end of construction and post-construction unreinforced embankment failures have been reported in the literature as a result of soil viscosity. Prefabricated vertical drains (PVDs) have been shown to substantially reduce the effect of creep induced excess pore pressures and increase the rate of strength gain due to consolidation of the soil. It is shown herein that PVDs may also enhance the effectiveness of geosynthetic reinforcement at minimizing differential settlement and lateral deformations of the foundation soils. This paper highlights the findings from earlier work by the authors by presenting a numerical study of the behaviour of geosynthetic reinforced embankments constructed over rate-sensitive soils. The effect of soil viscosity is addressed using a elasto-viscoplastic constitutive model. The effect of various factors such as construction rate, reinforcement stiffness, and PVD spacing is investigated, both during and following construction. The benefit of using PVDs combined with the use of geosynthetic reinforcement to improve the performance of reinforced embankments is highlighted.

1 INTRODUCTION

Stability and the time required for consolidation are two key considerations with respect to the construction of embankments over soft clay deposits. Several techniques have been developed to improve embankment stability and accelerate consolidation. These techniques include the use of geosynthetic basal reinforcement to reduce the outward shear force on the foundation soil (e.g. Rowe, 1984; Fowler and Koerner, 1987; Jewell, 1987; Rowe and Soderman, 1987; Rowe and Li, 1999; Bergado et al., 2002; Shen et al., 2005; Rowe and Taechakumthorn, 2007a; Kelln et al., 2007 and many others) and the use of prefabricated vertical drains (PVDs) to accelerate pore pressure dissipation in the clay (e.g. Crawford et al., 1992; Bergado et al., 1997; Chai and Miura, 1999; Li and Rowe, 1999; Indraratna and Redana, 2000; Bo, 2004; Chai et al., 2004; Rujikiatkamjorn et al., 2007 amongst many). For the case of a conventional soft clay deposit, the combined use of geosynthetic reinforcement with PVDs has been studied by several researchers and been shown to allow the rapid construction of higher embankments than would have been possible with the use of either method alone (Li and Rowe, 2001).

There have been many studies of the time-dependent behaviour of rate-sensitive soils (e.g. Lo and Morin, 1972; Vaid and Campanella, 1997; Vaid et al., 1979; Graham et al., 1983; Kabbaj et al., 1988; Leroueil, 1988). The performance of reinforced and unreinforced embankments constructed over rate-sensitive soil has also been investigated by both field studies (Rowe et al., 1995, 1996) and numerical analysis (Hinchberger and Rowe, 1998; Rowe and Hinchberger, 1998; Rowe and Li, 2002; Rowe and Taechakumthorn, 2007a). A study of a test embankment on a rate-sensitive soil at Sackville, New Brunswick (Rowe et al., 1996) showed that in order to accurately capture the time-dependent behaviour of the test embankment, the constitutive model need to consider the effect of soil viscosity. Rowe and Hinchberger (1998) proposed an elasto-viscoplastic constitutive model and demonstrated that the model can adequately describe the field behaviour of the test embankment.

Rowe and Li (2002) showed that the long-term stability of a reinforced embankment on rate-sensitive soil decreases after the end of construction due to the generation of delayed creep induced pore pressures. Rowe and Taechakumthorn (2007b) demonstrated the significance of PVDs in reducing the effect of delayed creep induced pore pressures and improving the stability of reinforced embankments over rate-sensitive soil. However, there has been very little research into the effects of the combined use of geosynthetic reinforcement and PVDs on the performance of embankments on rate-sensitive soil.

This paper uses the results of finite element analyses to illustrate a number of important issues related to the behaviour of the reinforced embankments on rate-sensitive soil deposits, both during and following the construction. In doing so it builds on a number of recent papers by the authors (Rowe and Taechakumthorn 2007a,b, 2008 and Taechakumthorn and Rowe 2007). The effect of the presence of PVDs combined with the use of geosynthetic reinforcement will be discussed. The interacting effects of soil viscosity, reinforcement stiffness, construction rate and PVD spacing will also be examined with respect to the time-dependent responses of the excess pore pressures and reinforcement strains. Finally, the effect of the construction rate on the surface settlement, heave and lateral deformation of the foundation will be highlighted.

2 FINITE ELEMENT MODELING AND MODEL PARAMETERS

2.1 *Mesh discretisation*

To simulate the construction of the reinforced embankment, a version of finite element program AFENA (Carter and Balaam, 1990), as adapted by Rowe and Hinchberger (1998), was adopted. A series of small strain finite element analyses were conducted. The effect of PVDs was modeled using drainage elements (Russell, 1990) as implemented by Li and Rowe (2001).

The results presented herein were obtained for embankments with the 2H:1V side slopes overlaying a 15 m soft, rate-sensitive, clay deposit above a rigid and permeable layer. A typical finite element mesh (Fig. 1) comprised 1815 six-noded triangle elements discretizing the embankment and the subsoil as well as a series of bar elements to discretize the geosynthetic reinforcement and two-noded rigid-plastic interface joint elements (Rowe and Soderman, 1985) to model fill/reinforcement and fill/foundation interfaces. PVDs were modeled using two-noded drainage elements (Li and Rowe, 2001).

The embankment centerline and far field boundary, located 100 m away from the centerline, were taken to be smooth-rigid boundaries. The bottom boundary was assumed to be rough-rigid with a free drainage boundary. The construction of reinforced embankments was simulated by gradually turning on the gravity in the fill material in 0.75 thick lifts at a rate corresponding to the construction rate of the embankment. The PVDs were fully penetrating in a square pattern. The effect of the smear zone was addressed by assuming that the ratio of equivalent radian of smear zone and vertical drains, s, was equal to 4. Details regarding modelling of PVDs and smear zone are provided in a subsequent sub-section.

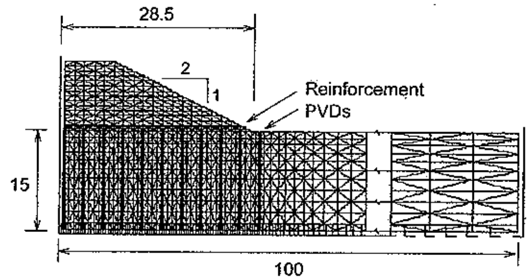

Figure 1. Finite element mesh discretisation.

2.2 *Constitutive model for rate-sensitive soils*

The elasto-viscoplastic constitutive model utilized herein (Rowe and Hinchberger, 1998), is fully coupled with Biot consolidation theory (Biot, 1941). This model also incorporates the Perzyna's theory of overstress viscoplasticity (Perzyan, 1963). The model is based on an elliptical yield cap model (Chen and Mizuno, 1990), a Drucker-Prager failure envelope and concepts drawn from critical state soil mechanics (Roscoe and Schofield, 1963). The constitutive model and computer program adopted in this study were successfully verified with the results from test embankments constructed over soft rate-sensitive soil in Sackville (Rowe and Hinchberger, 1998) and Gloucester (Hinchberger and Rowe, 1998). The main features of the model are summarized below. Additional details regarding the model were provided by Hinchberger (1996) and Rowe and Hinchberger (1998).

According to Perzyna's overstress theory of viscoplasticity, the governing equation was expressed in terms of strain-rate tensor:

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

where: S_{ij} is deviatoric stress; G is shear modulus; σ_{ij} is sum of principle stress stresses; K is bulk modulus and γ^{vp} is viscoplastic fluidity parameter. The terms $\phi(F)$ and f are flow and an elliptical plastic potential functions in $\sigma'_m - \sqrt{2J_2}$ stress space (where σ'_m is mean effective stress and J_2 is second invariant of deviatoric stress tensor), respectively which can be expressed as:

$$\phi(F) = \left(\frac{\sigma_{my}^{\prime(s)} + \sigma_{os}^{\prime(d)}}{\sigma_{my}^{\prime(s)}}\right)^n - 1 \quad (2)$$

$$f = (\sigma_m' - l)^2 + 2J_2R^2 - (\sigma_{my}' - l)^2 = 0 \quad (3)$$

where: $\sigma_{my}^{\prime(s)}$ or σ_{my}' is the intercept of the elliptical cap yield surface with the σ_m' axis ; $\sigma_{os}^{\prime(d)}$ is overstress; n is strain-rate exponent; l is mean effective stress corresponding to the center of the ellipse and R is ratio between major and minor of the ellipse.

The elastic bulk modulus, K, and shear modulus, G, are function of the mean effective stress given by:

$$K = \frac{1+e}{\kappa}\sigma_m' \quad (4)$$

$$G = \frac{3(1-2\nu')K}{2(1+\nu')} \quad (5)$$

where: e is void ratio; κ is recompression index and ν is Poisson's ratio.

2.3 *Drainage elements*

Previous research has shown that the 3-D system of a soil foundation with PVDs can be reasonable modeled using a 2-D plane strain approximation. Hird et al. (1992) demonstrated that the average degree of consolidation on the horizontal plane, U_h, at depth z and time t under instantaneous loading obtained from an axisymmetric analysis could be matched to that obtained in a plane strain analysis by geometric matching, permeability matching or a mix of these two techniques. Li and Rowe (2001) adopted the permeability matching scheme by modifying the horizontal hydraulic conductivity of the soil and the equivalent discharge capacity of vertical drains under plane strain conditions. The same matching scheme was also adopted herein, viz:

$$k_{pl} = \frac{2k_{ax}}{3\left[\ln\left(\frac{n}{s}\right) + \left(\frac{k_{ax}}{k_s}\right)\ln(s) - \frac{3}{4}\right]} \quad (6)$$

$$Q_w = \left(\frac{2}{\pi R}\right)q_w \quad (7)$$

$$n = \frac{R}{r_w}, \quad s = \frac{r_s}{r_w} \quad and \quad q_w = \pi k_w r_w^2 \quad (8)$$

where: k_{pl}, k_{ax}, k_s and k_w are the hydraulic conductivities of: the bulk soil in the horizontal direction for plane strain conditions, the bulk soil under axial symmetric conditions, the soil in the smear zone (assumed to be isotropic and the same as the vertical hydraulic conductivity of soil), and the vertical drain, respectively. r_w, r_s and R are the equivalent radius of the vertical drain, smear zone and influence zone, respectively. Q_w and q_w are the equivalent discharge capacities for the plane strain and axisymmetric unit cells, respectively. Details of the drainage elements have been provided by Li and Rowe (2001).

Table 1. Details of foundation soil properties.

Soil Parameter	Soil RS1	Soil RS2
Failure envelope, $M_{N/C}$ (ϕ')	0.96 (29°)	0.96 (29°)
Cohesion intercept, c_k (kPa)	0	0
Failure envelope, $M_{O/C}$	0.75	0.75
Aspect ratio, R	1.25	1.25
Compression index, λ	0.16	0.16
Recompression index, κ	0.034	0.034
Coefficient of at rest earth pressure, K_o'	0.75	0.75
Poisson's ratio, υ	0.3	0.3
Reference hydraulic conductivity, k_{vo} (m/s)	2×10^{-9}	2×10^{-9}
Hydraulic conductivity ratio, k_h/k_v	4	4
Unit weight, γ (kN/m^3)	17	17
Initial void ratio, e_o	1.50	1.50
Viscoplastic fluidity, γ^{vp} (1/hour)	2.0×10^{-5}	1.0×10^{-7}
Strain rate exponent, n	20	30

3 MODEL PARAMETERS

3.1 *Foundation soils properties*

As in a number of previous papers by the authors (Rowe and Li, 2002; Rowe and Taechakumthorn, 2007a,b), the two soft rate-sensitive soils examined in this study, denoted as RS1 and RS2, have parameters as listed in Table 1 together with preconsolidation and initial vertical stress profiles as shown in Figure 2. The constitutive parameters used for soil RS1 are similar to those for the subsoil below the Sackville test embankment (Rowe and Hinchberger, 1998). Soil RS2 was assumed to have the same properties as Soil RS1 except for the rate-sensitive parameters (the viscoplasticity and the strain-rate exponent).

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

$$k_v = k_{vo}\exp\left(\frac{e-e_o}{C_k}\right) \quad (9)$$

where: C_k is hydraulic conductivity change index ($C_k = 0.2$). The hydraulic conductivity was considered

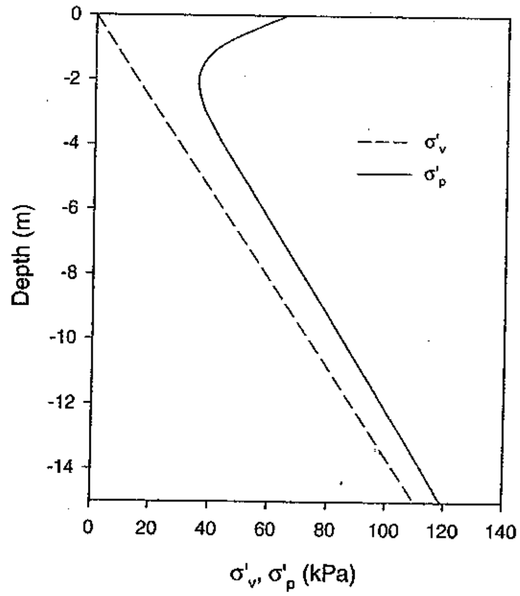

Figure 2. Preconsolidation pressure and initial vertical stress profiles (based on Rowe and Li, 2002).

to be anisotropic with the ratio of horizontal to vertical hydraulic conductivity (k_h/k_v) of 4.

3.2 *Embankment fill properties and rate of construction*

The embankment fill was assumed to be a purely frictional granular soil with a friction angle of 37°, a dilation angle of 6°, and a unit weight of 20 kN/m^3. The nonlinear elastic behaviour of the fill was model using Janbu's equation (1963) as:

$$\frac{E}{P_a} = K\left(\frac{\sigma_3}{P_a}\right)^m \qquad (10)$$

where: E is Young's modulus; P_a is the atmospheric pressure; σ_3 is the minor principle stress and K and m are material constants selected to be 300 and 0.5, respectively.

The construction rates of the cases examined in this study were 2, 4, 6, 8 and 10 m/month.

3.3 *Interface parameters and stiffness of reinforcements*

The interactions between fill/reinforcement and fill/foundation were assumed to have frictional angle of 37°. The axial tensile stiffness of the geosynthetic reinforcement was varied with values of 0 (no reinforcement), 500, 750, 1000, 2000, 4000 and 8000 kN/m being considered.

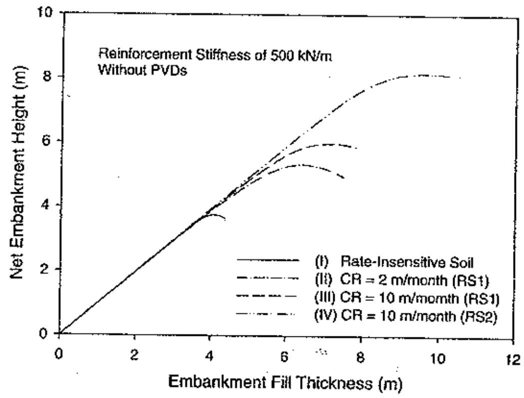

Figure 3. The effect of soil viscosity and construction rate on the short-term stability of reinforced embankments.

3.4 *PVDs properties*

PVDs with a typical rectangular cross section of 100 mm × 4 mm (Holtz, 1987) were considered. The equivalent diameter (d_w) of a circular drain is equal to 52 mm based on Rixner et al. (1986): $d_w = (b+t)/2$, where: b and t are width and thickness of PVDs, respectively. The PVD spacings (S) of 1, 2 and 3 m examined in this paper are in the typical range used in practice (Holtz, 1987). The discharge capacity of the PVDs was taken to be a 100 m^3/year (Rixner et al., 1986).

4 RESULTS AND DISCUSSIONS

4.1 *Effects on short-term stability of the reinforced embankment – No PVDs*

A series of finite element analyses were conducted to simulate the construction of reinforced embankments on different rate-sensitive soils. The effects of different reinforcement stiffness, construction rate, and PVD spacing were investigated.

The short-term failure height of a reinforced embankment can be assessed in terms of the failure height which is defined as the height of the embankment fill at which any further attempt to add fill material will not result in an increase in the net embankment height (i.e. the height above the original ground surface). The failure height can be obtained by plotting the relationship between the net embankment height (fill thickness minus settlement) against the fill thickness as shown in Figure 3.

The results in Figure 3 show the effect of soil viscosity and construction rate on the stability of the reinforced embankment examined. For Case I which represents a rate-insensitive (non-viscous) soil, a conventional elasto-plastic constitutive model was employed having the same soil properties as the other

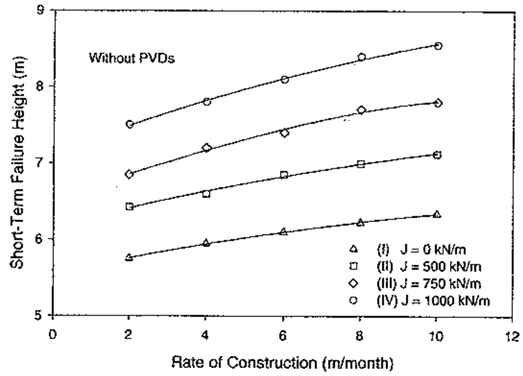

Figure 4. The effect of reinforcement stiffness and construction rate on the short-term embankment stability (Soil RS1).

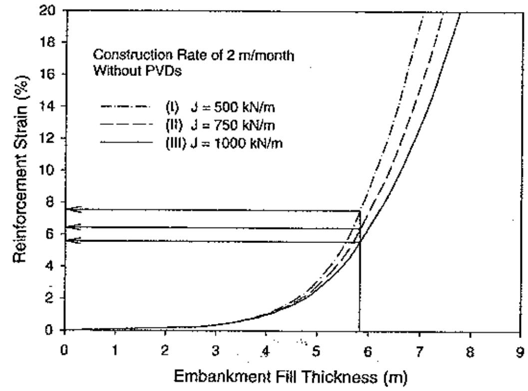

Figure 5. Short-term reinforcement strain versus fill thickness (Soil RS1).

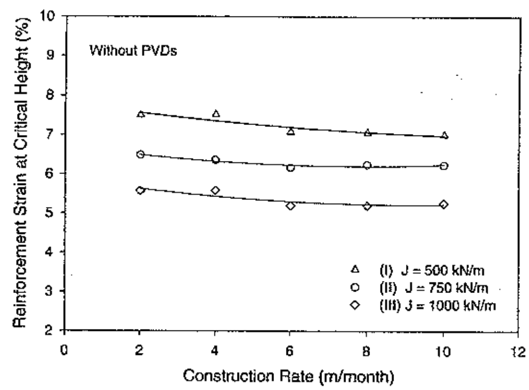

Figure 6. The effect of reinforcement stiffness and construction rate on the short-term reinforcement strains (Soil RS1).

soils except for soil viscosity. In all cases there was a linear initial response followed by non-linear response until the calculated failure heights of 4.05, 6.45, 7.20, and 9.45 m were reached for Cases I, II, III and IV, respectively. The differences in short-term failure height between Case I, III and IV illustrate the effect of soil viscosity. The short-term strength of the viscous soil allowed the soil to carry extra load with the more viscous soil (RS2) exhibiting the higher short-term undrained shear strength and hence a greater short-term failure height than the less viscous soil (RS1, Case III) or inviscous soil (Case I). Figure 3 also demonstrate the effect of construction rate with the faster construction rate of 10 m/month (Case III) resulting in a 1.75 m higher failure height (7.2 m) than obtained at the slower rate of 2 m/month (6.45 m-Case II). However, the shear strength decreased with time as will be discussed later.

The main function of geosynthetic reinforcement is to reduce outward shear stress on the foundation soil, and if stiff enough, induce inward shear stress on the foundation soil. In so doing, the reinforcement increases the stability of the embankment. Figure 4 demonstrates the role of reinforcement in improving the short-term stability of the embankment. A series of unreinforced and reinforced embankments were numerically constructed at different construction rates until failure. The stiffer reinforcement resulted in higher failure height. The effect of construction rate on the rate-sensitive soil was also confirmed as the faster construction rate led to higher short-term embankment stability.

Case histories (Rowe et al., 1995, Rowe and Hinchberger, 1998 and Bergado et al., 2002) indicate that a geosynthetic reinforced embankment can be constructed to a height well beyond the point where there is significant plastic failure in the foundation soil and hence the point where an unreinforced embankment would fail.

Bergado et al. (2002) defined the term critical height as the failure height of the unreinforced embankment on the same soil foundation. Figure 5 shows the relationship between embankment fill thickness and corresponding reinforcement strain for the case of 2 m/month construction rate. The critical height of 5.75 m for the case in Figure 5 was obtained from the short-term failure height of the unreinforced embankment in Figure 4. The compatible reinforcement strains at the critical height were 7.5, 6.5 and 5.5% for reinforcement stiffness of 500, 750 and 1000 kN/m, respectively. The reinforcement strains calculated for the different reinforcement stiffnesses at the critical height were plotted against the corresponding construction rate in Figure 6. In these cases the effect of construction rate is small as the reinforcement strains are about 7.5, 6.5, and 5.5% for reinforcement stiffness of 500, 750 and 1000 kN/m, respectively regardless of applied construction rate.

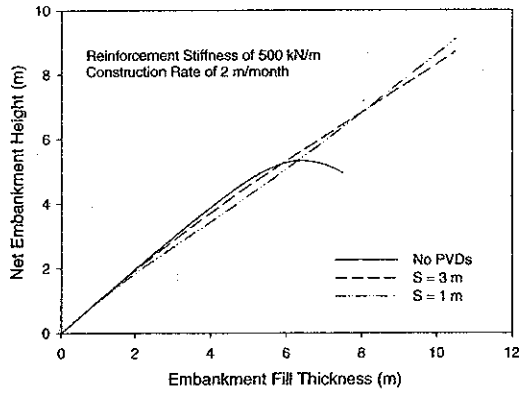

Figure 7. The effect of PVDs on the short-term stability of the reinforced embankments (Soil RS1).

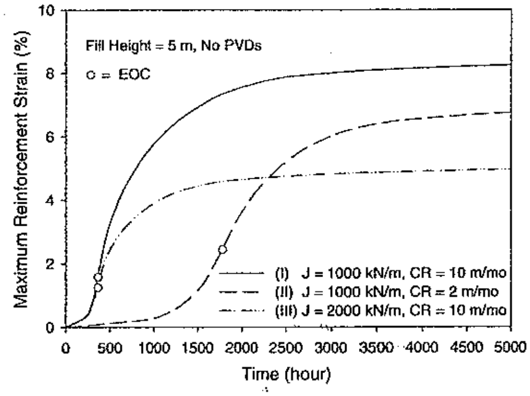

Figure 8. Effect of reinforcement stiffness and construction rate on the long-term reinforcement strains (Soil RS1).

4.2 *Effects of PVDs on short-term stability*

PVDs have been being used to accelerate the construction of high embankments on soft ground by accelerating pore pressure dissipation and hence the gain in shear strength of the foundation with time. As shown in Figure 7, the short-term stability of the reinforced embankments was significantly improved when PVDs were installed. The results showed no failure even when the embankment fill thickness was increased up to 10.5 m. During the initial stage of construction, the case of smaller PVD spacing (S = 1 m) showed larger settlement because the subsoil had a higher degree of partial consolidation compared to the case of larger PVD spacing (S = 3 m). However, the higher degree of partial consolidation resulted in less overstress remaining in the soil and accordingly, less viscoplastic deformation was developed. As a result, the case of smaller PVD spacing showed smaller settlement after the fill thickness exceeded 8.5 m.

4.3 *Effects on the long-term reinforcement strains*

As noted above, even though the viscoplastic characteristics of a rate-sensitive soil allowed the construction of higher embankments in the short-term, the overstress and creep deformation associated with the construction can be expected to cause long-term deformation and, potentially, stability problems. To investigate this problem, a conventional embankment design was performed using limit equilibrium analysis. The undrained shear strength profile of the soft clay deposit used in the design calculations corresponded to the plane strain shear strength established form numerical tests at a typical recommended strain rate of 0.5–1.0%/hour (Germaine and Ladd, 1988). The analysis shows that using the reinforcement force of 100 kN/m (i.e. $J = 2000$ kN/m at 5% strain) a 5.25 m height reinforced embankment should be stable with safety factor of 1.3. However, from the finite element modeling, the 5.25 m height reinforced embankment resulted in 6% long-term reinforcement strain, which implied that the limit equilibrium analysis overestimated the stability of the reinforced embankment. For this particular soil and a reinforcement stiffness of 2000 kN/m, only a 5 m high reinforced embankment could be constructed if the long-term reinforcement strain was to be limited to an allowable design strain of 5%.

Figure 8 shows the effect construction rate and reinforcement stiffness on the long-term mobilized reinforcement strain. For a rate-sensitive soil, a faster construction rate led to higher short-term shear strength of the soil and hence smaller end of construction (EOC) strains in the reinforcement. In Case I (faster construction rate, 10 m/month), the foundation soil carried most of the load at the end of construction and so the EOC reinforcement strain was only 1.6%. In contrast, at a lower construction rate (Case II, 2 m/month) the maximum EOC reinforcement strain was 2.6%. However the amount of overstress in the soil was reduced at this slower rate and this reduced the subsequent creep deformation as well as the delayed excess pore water pressure (as discussed later) and ultimately resulted in smaller long-term reinforcement stains (6.9% for Case II) than were obtained for the higher construction rate (8.3 % for Case I).

The results from Case I and III demonstrated the effect of reinforcement stiffness for a given construction rate (10 m/month) and as expected, the stiffer reinforcement (Case III) gave smaller strain both at the end of construction and long-term condition (1.3% and 4.9%, respectively) compare to those from Case I (1.6% and 8.3%).

PVDs greatly enhanced the rate of excess pore pressure dissipation and reduced the amount of overstress in the soil. Consequently, the effects of viscoplastic response such as long-term creep deformation, stress

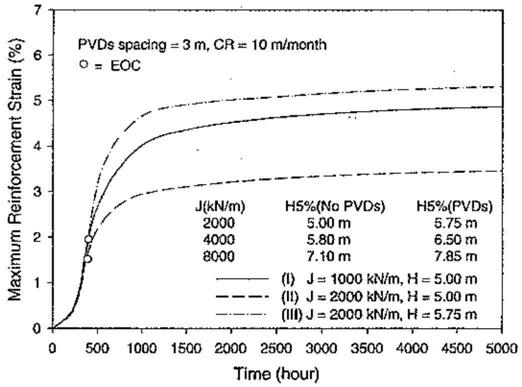

Figure 9. The effect of PVDs on long-term reinforcement strains (Soil RS1).

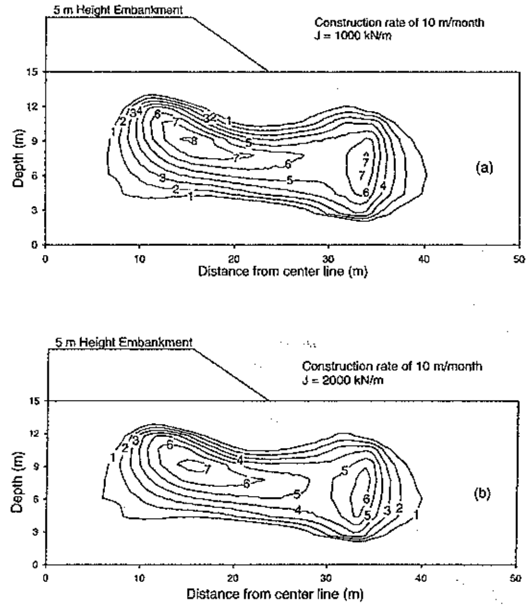

Figure 10. Increase in excess pore water pressure between end of construction and 1 month after the end of construction (Soil RS1).

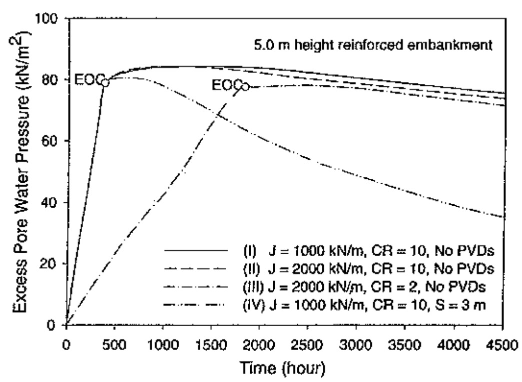

Figure 11. The effect of reinforcement stiffness and PVDs on the excess pore pressure dissipation (Soil RS1).

relaxation and delayed creep induced pore pressure are minimized. For the case of 5 m a high embankment with the reinforcement stiffness of 1000 kN/m constructed as slow as 2 m/month, the reinforcement strain still reached 6.9% for long-term conditions and this exceeds the typical design limit of 5% strain (Case II in Figure 8). In contrast when PVDs were installed with 3 m spacing (Case I in Figure 9), the faster construction rate of 10 m/month could be employed and the long-term reinforcement was limited to 4.8%. With stiffer reinforcement ($J = 2000$ kN/m), the PVDs reduced the long-term reinforcement strain from 5.0% to only 3.8% (Case III in Figure 8 versus Case II in Figure 9). If the reinforcement stiffness of 2000 kN/m was combined with PVDs, the reinforced embankment height could be increased up to 5.75 m without the long-term reinforcement strain exceeding 5.0% (Case III in Figure 9). For a 5% long-term strain and a PVD spacing of 3 m, reinforced embankments could be constructed up to 6.5 m and 7.85 m for reinforcement stiffness of 4000 and 8000 kN/m, respectively (see insert to Figure 9).

4.4 *Effects on the excess pore water pressure*

During and following embankment construction on a rate-sensitive soil, two processes occur simultaneously: (1) the generation of creep induced pore water pressures, and (2) the dissipation of excess pore pressures due to consolidation. Figures 10a and 10b show contours of the increase in excess pore pressure between the end of construction and 1 month after the construction of a 5 m high reinforced embankment with reinforcement stiffness of 1000 and 2000 kN/m, respectively. The shear induced generation of pore pressure was evident since the contours of increase in pore pressure formed along the potential failure zone even though there was pore pressure dissipation. The effective stress and shear strength was reduced after the end of construction thus the critical period with respect to the embankment stability (minimum factor of safety) occurred following the end of construction. Reinforcement provided a stabilizing force, as discussed earlier, and the stiffer reinforcement gave more restraint and hence a slight decrease in the creep induced pore pressure (compare Figures 10a and 10b).

The excess pore pressures at 6 m beneath the embankment crest, where the maximum increase in pore pressure occurred, are shown in Figure 11 for four cases. The excess pore pressures at the end of construction were approximately 80 kPa for all cases examined and kept increasing even when no more

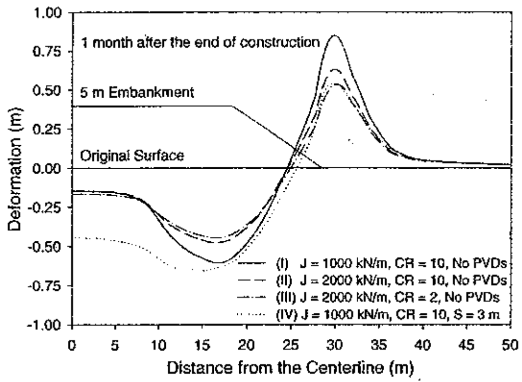

Figure 12. Effect of reinforcement stiffness and PVDs on the differential settlement and heave of the foundation (Soil RS1).

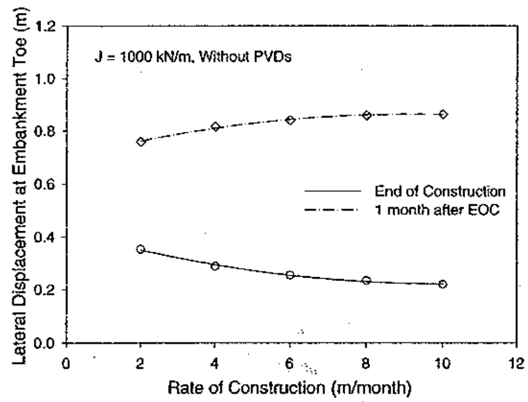

Figure 13. Effect of construction rate on the lateral displacement at the embankment toe (Soil RS1).

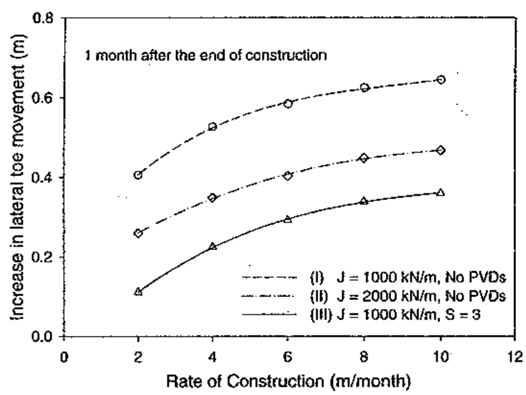

Figure 14. Effect of rate of construction on the increase in lateral displacement of the embankment toe between the end of construction and 1 month after the end of construction (Soil RS1).

fill was added. The stiffer the reinforcement, resulted in slightly smaller creep induced pore pressures with time and hence slightly faster pore pressure dissipation (compare Cases I and II in Figure 11). Construction rate also affected the rate of excess pore pressure dissipation. A slower construction rate allowed higher degree of partial consolidation during the construction and this reduced the amount of overstress and long-term creep in the foundation.

The installation of PVDs significantly reduced the effect of delayed excess pore pressure build up in this rate-sensitive soil. Figure 11 also demonstrates that with the use of PVDs, the excess pore pressure promptly reduced following the end of construction (Cases IV).

4.5 *Effects on differential settlement and lateral deformation of the foundation*

Reinforcement can significantly reduce the differential settlement and toe heave for embankments constructed on rate-sensitive soil. For a fill thickness of 5 m on soil RS1, Figure 12 shows the ground surface profile at 1 month following the construction for several reinforced embankment scenarios.

The results in Case I and II illustrated the effect of reinforcement stiffness. The stiffer (J = 2000 kN/m) reinforcement (Case II) limited heave to 0.63 m and reduced the differential settlement between centerline and embankment crest to 0.34 m compared to 0.85 m and 0.47 m, respectively, for the less stiff reinforcement (J = 1000 kN/m, Case I). The effect of the construction rate on the differential settlement and heave of the foundation was insignificant.

The presence of PVDs reduced the differential settlement of the foundation for reinforcement stiffens of 1000 kN/m from 0.47 m (Case I) to 0.15 m (Case IV). The PVDs reduced the heave from 0.85 m (Case I) to 0.54 m (Case IV) and hence gave a very similar toe heave to that obtained with much stiffer (J = 2000 kN/m) reinforcement (Case II).

Figure 13 shows the effect of construction rate on the lateral movement at the embankment toe. As previously discussed, faster construction rate led to higher short-term shear strength of the foundation and that resulted in smaller end of construction lateral movement at the toe (solid line in Figure 13. However, faster construction also led to higher overstress in the soil and this generated larger creep deformation that resulted in higher lateral movements 1 month after construction than was observed for slower construction rates (dashed line in Figure 13).

The increase in lateral toe movement, between the end of construction and 1 month later is shown in Figure 14. The higher construction rate resulted in a substantially greater post construction increase in lateral movement for the reasons described above. Stiffer reinforcement had a beneficial effect on reducing the lateral deformation of the embankment (compare Cases I and II). The combined use of geosynthetic reinforcement ($J = 1000$ kN/m) and PVDs (Case III) gave the lowest increase in post construction lateral movement of the three cases examined.

5 CONCLUSIONS

This paper has examined the time-dependent behaviour of reinforced embankments constructed over rate-sensitive soil. The results indicated that the viscoplastic characteristic of rate-sensitive soil had a significant effect on the performances of reinforced embankments, especially after the end of construction. The behaviour of reinforced embankments on rate-sensitive soil, within the range of cases and parameters considered in this paper, can be summarized as follow.

For the rate-sensitive soil, the faster the rate of construction, the higher is the short-term stability of the embankment. However, the consequent large overstress generated in the soil led to larger viscoplastic deformations and could result in failure after the end of construction. Geosynthetics reinforcement reduced the outward shear force on the foundation and, in so doing, minimized the long-term creep deformation and improved the short-term stability of the embankment. The short-term reinforcement strain at the critical height decreased with increasing reinforcement stiffness. PVDs greatly improved the short-term stability of the reinforced embankments by accelerating the consolidation process and hence increased the rate of strength gain in the foundation.

Faster construction rates resulted in higher short-term strength therefore less force was transferred to the reinforcement and only small strains were developed at the end of construction. However, the larger overstress associated with faster construction resulted in larger long-term viscoplastic deformation and larger long-term reinforcement strain. PVDs substantially improved the beneficial effects of reinforcement. With the combined used of reinforcement and PVDs, less stiff reinforcement could be used with the faster construction rate while limiting the long-term reinforcement strain to acceptable values.

Delayed creep induced pore pressures were generated in the rate-sensitive soil. Reinforcement could reduce creep deformation and the associated pore pressures, thus the rate of pore pressure dissipation was accelerated. Without PVDs, the excess pore pressures kept increasing until they reached a peak sometime after the end of construction; therefore, the critical period with respected to the stability of the embankment could occur after the construction was completed. PVDs resulted in rapidly reduced excess pore pressures after the end of construction.

The post-construction deformations (differential settlement, heave and lateral toe movement) of the reinforced embankments were minimized by using geosynthetic reinforcement. The stiffer the reinforcement, the smaller the post-construction deformations. Other things being equal, a higher construction rate led to higher overstress in the foundation and hence larger long-term creep and lateral toe movements. The effect of reinforcement on the long-term deformation of the embankments could be enhanced with the use of PVDs.

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