The Interaction Between Reinforcement and Vertical Drains and Effect on the Performance of Embankments on Soft Ground

R.K. Rowe, C. Taechakumthorn

Abstract. This paper reviews the behaviour of reinforced embankments on soft ground. Case of the Almere test embankment is used as an example to demonstrate the key function of reinforcement in improving the performance of embankments on soft foundation. The effects of partial drainage are summarized for reinforced embankments and contrasted the results from undrained analyses to highlight the effect of partial consolidation during construction. Effects of the interaction between reinforcement and prefabricated vertical drains (PVDs) are presented. It is concluded that the combined effects of partial consolidation provided by PVDs and the tension mobilized in reinforcement can substantially increase the stability of an embankment on a given soft soil. This paper also provides brief explanation of a recent design approach for embankments on soft soil, considering the combined effect of reinforcement and PVDs. Effects of the creep/relaxation characteristics of geosynthetic reinforcement and rate-sensitive nature of soft cohesive foundation soil are discussed. It is shown that time-dependent nature of geosynthetics and foundation can decrease the failure height of a reinforced embankment. Also, the long-term performances of a reinforced embankment can vary significantly depending on the soil and reinforcement characteristics. The results suggest the need for care when the foundation soil is rate-sensitive.

Keywords: reinforced embankment, geosynthetics, PVDs, creep/relaxation, soft ground, design methods.

1. Introduction

Geosynthetic reinforcement and prefabricated vertical drains (PVDs) have revolutionized many aspects of the design and construction of embankments on soft ground. They have been shown to provide a cost effective alternative to more traditional techniques, when appropriately designed and installed. The behaviour of reinforced embankments on typical soft deposits is now well understood and many design procedures have been proposed (*e.g.* Fowler & Koerner, 1987; Humphrey & Holtz, 1987; Jewell, 1987; Rowe & Soderman, 1987; Rowe & Li, 1999; Bergado *et al.*, 2002; Varuso *et al.*, 2005; Kelln *et al.*, 2007; Bergado & Teerawattanasuk, 2008; Abusharar *et al.*, 2009; Tolooiyan *et al.*, 2009; and Huang & Han, 2009). However, while these design methods are conservative for conventional (rate-insensitive) soils, they may be quite unconservative for less conventional (rate-sensitive) soils (Rowe & Li, 2005; Li & Rowe, 2008 and Rowe & Taechakumthorn, 2008a). There has been limited research into the behaviour of embankments on rate-sensitive soils. One key case study was reported by Rowe *et al.* (1995).

The beneficial effects of PVDs for accelerating the gain in soil strength are well recognized (*e.g.* Li & Rowe, 1999; Indraratna & Redana, 2000; Bergado *et al.*, 2002; Bo, 2004; Zhu & Yin, 2004; Chai *et al.*, 2006; Taechakumthorn & Rowe, 2008; Sinha *et al.*, 2009; Saowapakpiboon *et al.*, 2010; Saowapakpiboon *et al.*, 2011; Karunaratne, 2011 and Indraratna *et al.*, 2011). For example, when PVDs are used in conjunction with basal reinforcement, the presence of PVDs can substantially reduce the long-term creep deformation while allowing more rapid construction than could be safely considered without the use of PVDs (Li & Rowe, 2001 and Rowe & Taechakumthorn, 2008a).

The objective of this paper is to summarize research on the effect of basal reinforcement and PVDs on the design and construction of embankments over soft ground. The short-term and long-term performances of reinforced embankments are discussed. The effect of partial drainage during the construction, stage construction, and the presence of PVDs is illustrated. This paper also summarizes a design approach (Li & Rowe, 2001) which considers the effects of the interaction between reinforcement and PVDs for embankments constructed on typical (rate-insensitive) soft clay deposits. The effect of creep/relaxation of geosynthetic reinforcement and foundation soil on the behaviour of reinforced embankments is demonstrated. Finally, a number of parametric studies are used to highlight some design considerations and potential problems that might be anticipated during construction. This paper is an extended version of the keynote lecture presented at the symposium of new techniques for design and construction in soft clays (Rowe & Taechakumthorn, 2010).

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2. Reinforced Embankment on Soft Ground

When embankments are constructed on soft cohesive foundations, the lateral earth pressure within the embankment fill imposes shear stresses on the foundation soil, reducing the bearing capacity of the foundation and hence embankment stability (Jewell, 1987). The role of the basal reinforcement is to provide confining stress to counteract some or all of the earth pressure within the embankment and to resist the lateral deformation of the foundation, thereby increasing the bearing capacity and embankment stability. Typically, reinforced embankments are designed based on consideration of (a) bearing capacity, (b) global stability, (c) pullout/anchorage and (d) deformations (Rowe & Soderman, 1987; Leroueil & Rowe, 2001). Before going into the detailed design procedures it is, however, useful to understand when and how reinforcement contributes to the embankment stability. The role of reinforcement can be illustrated with respect to the Almere test embankments (Rowe & Soderman, 1984).

The Almere test embankment allows the comparison of the observed and calculated behaviour of both an unreinforced embankment and an embankment reinforced using a multi-filament woven geotextile (with tensile stiffness J = 2000 kN/m) constructed on a soft soil deposit. The deposit was comprised of approximately 3.3 m of very soft organic clay, with an undrained strength of 8 kPa, underlain by dense sand. A trench was excavated (see insert to Fig. 1) at the edge of the proposed embankment and the clayey soil was placed over the reinforcement to form a retaining bank (see insert to Fig. 1). The hydraulic fill was then placed until failure occurred. The reinforced section experienced a relatively ductile failure at a height of 2.75 m, after 25 h of sand filling. This was in contrast to the rapid failure of the unreinforced section at a height of 1.75 m. It seems likely that the geosynthetic reinforcement was the major reason for the differences in the observed behaviour. Figure 1 shows that for fill heights less than 1 m, the clay was largely elastic and the strains in the reinforcement remained essentially constant. As the fill thickness was increased from 1 m to 2 m, there was extensive plastic failure within the clay.

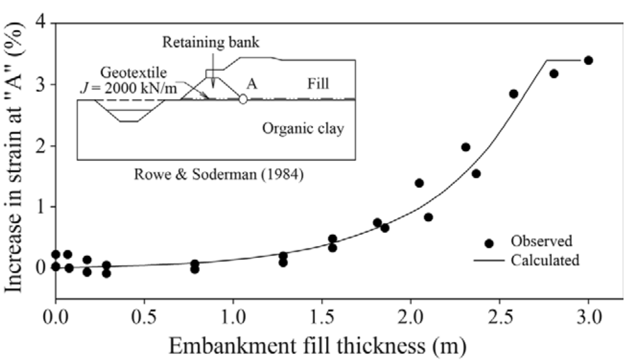

Figure 1 - Comparison of predicted and observed reinforcement strains at A (modifiedfrom Rowe & Jones 2000).

At a given embankment height, the reinforcement reduced the growth of the plastic region within the soil. For example, in the unreinforced case the analysis predicted failure at a fill height of 1.8 m (Rowe & Soderman, 1984). In contrast, at the same height in the reinforced embankment, the displacements were smaller and the plastic region was not contiguous. The analysis indicated that a contiguous plastic region had developed in the soil at a fill height of 2.05 m (approximately 15% higher than the corresponding height for the unreinforced embankment; Rowe & Soderman, 1984).

The development of a contiguous plastic region (at about 2.0 m in this case) represented the first stage of collapse for a reinforced embankment since, after that, the embankment was completely dependent upon the reinforcement for the support of any additional fill. As a result, while geosynthetic reinforcement was trying to maintain the integrity of the system, placing additional fill caused reinforcement strains to increase rapidly until either loading ceases or failure occurs (in this case at a predicted height of 2.66 m due to failure at the geosynthetic-soil interface).

3. Undrained Behaviour of Reinforced Embankments

In an undrained analysis of an unreinforced embankment, the collapse height of the embankment simply corresponds to the height at which the soil shear strength is fully mobilized along a potential failure surface (Rowe & Soderman, 1985 and Rowe & Mylleville, 1990). However, for most reinforced embankments, collapse also involves failure of soil-reinforcement system which may include (a) failure of reinforcement, (b) failure of the soil-reinforcement interfaces, or (c) failure because the reinforcement is not stiff enough to control deformations to an acceptable level. The concepts of net embankment height (defined as fill thickness minus maximum settlement) and allowable compatible reinforcement strain were introduced to account for failure due to excessive displacements before the reinforcement reaches its pullout capacity or its ultimate tensile strain (Rowe & Soderman, 1985).

For example, Fig. 2 shows net embankment height and the maximum reinforcement strain plotted against the fill thickness for an embankment constructed quickly on a soft clayey foundation. The failure of this reinforced embankment due to excessive subsidence occurred at a fill thickness equal to 2.4 m and a reinforcement strain of 5.2%, which is well below the tensile failure strain for most of geosynthetic products (Shinoda & Bathurst, 2004). Therefore, it is important to define an allowable ‘compatible’ reinforcement strain corresponding to the failure thickness of a reinforced embankment. A second allowable strain will be related to the reinforcement strength. The lower of these two strains would be used together with reinforcement stiffness to get the allowable reinforcement force used in a limit equilibrium calculation. Figure 3 shows the variation of allowable compatible strain, ε_a, (for the case of reinforced embankment on soft foundations having uniform undrained shear strength with depth) with the dimensionless parameter, Ω, (Rowe & Soderman, 1985) defined as:

$$\Omega = \left(\frac{\gamma_f H_c}{s_u}\right)\left(\frac{s_u}{E_u}\right)\left(\frac{D}{B}\right)_e^2 \tag{1}$$

where; γ_f is a bulk unit weight of the embankment fill; H_c is the collapse height of the unreinforced embankment; s_u and E_u are undrained shear strength and modulus of the soft foundation, respectively; $(D/B)_e$ is the ratio of the effective depth of the deposit to the crest width, as defined in Fig. 3. It should be noted that when using Eq. (1) it is not conservative to underestimate the undrained modulus of the soft foundation, since a lower value of E_u corresponds to a high value of ε_a, which in turn gives a high reinforcement force, T_{rein}.

For the cases when embankments are constructed on a foundation whose strength increases with depth, the inclusion of reinforcement changes the collapse mechanism by forcing the failure surface to pass through stronger and stiffer soil. This case is not addressed by design chart proposed by Rowe & Soderman (1985) for the soil having constant shear strength with depth and so Hinchberger & Rowe (2003) developed a design chart for estimating the reinforcement strain at failure for the reinforced embankment on foundations having increasing shear strength with depth (Fig. 4). The strain presented in Fig. 4 represents an upper limit; the allowable strain may in some cases be controlled by the strain at rupture of the reinforcement (which in turn may be reduced by some appropriate partial factor). Also, for soft brittle soils which are susceptible to strain-softening, the limiting reinforcement strain may be as low as 0.5%-2.0% in order to reduce the maximum shear strain developed in foundation soils to an acceptable level (Rowe & Mylleville, 1990 and Mylleville & Rowe, 1991).

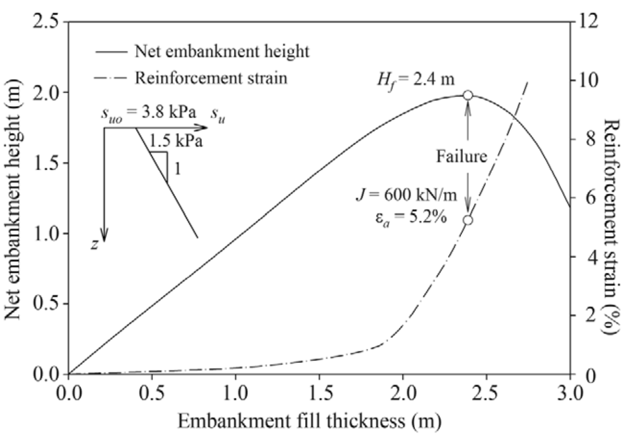

Figure 2 - Maximum net embankment height and allowable reinforcement strain (modified from Hinchberger & Rowe 2003).

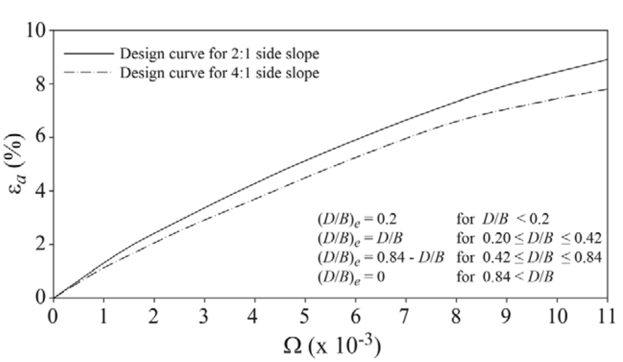

Figure 3 - Variation of allowable compatible strain ε_a with dimensionless parameter Ω (modified from Rowe & Soderman 1985).

4. Partially Drained Behaviour of Reinforced Embankments

The observed construction-induced excess pore water pressures from a large number of field cases suggest that significant partial consolidation of the foundation may occur during embankment construction at typical construction rates (Crooks *et al.*, 1984; Leroueil & Rowe, 2001). This applies to natural soft cohesive deposits that are typically slightly overconsolidated. It also has been reported that often there may be a significant strength gain due to partial consolidation during embankment construction (*e.g.* Bergado *et al.*, 2002; Bo, 2004; Chai *et al.*, 2006 and Saowapakpiboon *et al.*, 2010).

Although field cases suggest the importance of considering partial consolidation, they do not allow a direct comparison of cases where it is, or is not, considered. Finite element analyses, however, do provide a powerful tool for comparing the behaviour of reinforced embankments constructed under undrained and partially drained conditions (Rowe & Li, 1999). For example, Fig. 5 shows the variation in calculated embankment failure height with reinforce-

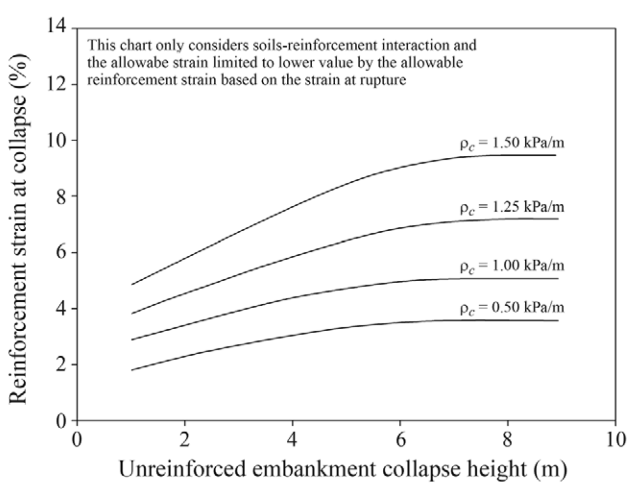

Figure 4 - Chart for estimating reinforcement strains at embankment failure for foundation soils with strength increase with depth (modified from Hinchberger & Rowe 2003).

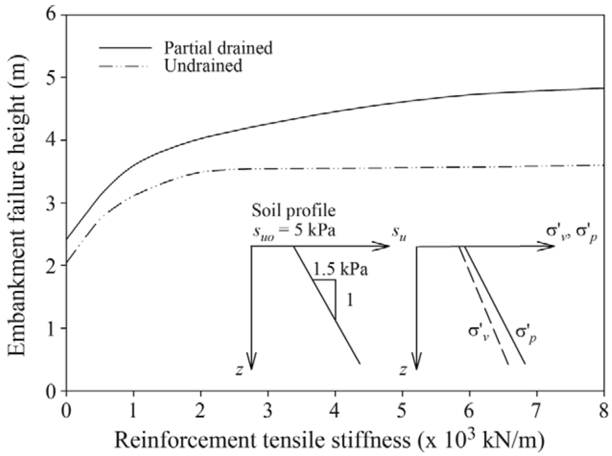

Figure 5 - Embankment failure height against reinforcement tensile stiffness (modified from Li & Rowe 2001).

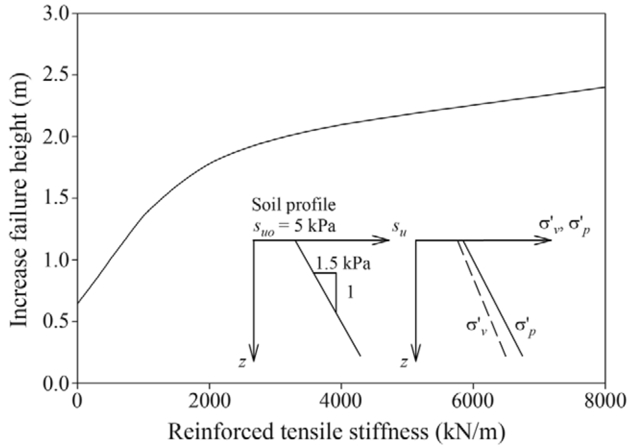

Figure 6 - Increase of failure height after 95% consolidation at end of first stage construction (modified from Li & Rowe 2001).

ment stiffness for undrained and partially drained conditions. The construction rate employed in the analysis was 1 m/month to allow partial dissipation of the excess pore water pressure during construction. The fully coupled analyses gave an increase in the unreinforced embankment failure height from 2.1 m (for undrained analysis) to 2.4 m. A change of reinforcement stiffness from 500 kN/m to 8000 kN/m also resulted in an increase in failure height by between 1.4 m and 3.8 m, compared with between 0.7 m to 1.4 m for the undrained analysis. This implies that the reinforcement had a greater effect for the partially drained cases than for undrained cases. However, for this particular soil profile (see insert to Fig. 5: s_u = undrained shear strength, σ'_v = vertical effective stress and σ'_p = maximum preconsolidation pressure), the increase in reinforcement stiffness had the most significant effect on the embankment failure height for stiffness values up to only J = 2000 kN/m and the benefit of increasing reinforcement stiffness diminishes for very stiff reinforcement.

When a soft foundation soil does not initially have the strength to safely support a given embankment, stage construction may be employed to allow sufficient consolidation and strength gain to occur to support the final embankment load. Li & Rowe (1999) showed that geosynthetic reinforcement may eliminate the need for stage construction or, in cases where staging was still needed; it reduced the number of stages required. The effect of reinforcement stiffness on multi-stage construction is illustrated in Fig. 6. To obtain this figure, embankments were first numerically constructed to the maximum height permitted with a factor of safety of 1.3 at the end of stage one and allowed to consolidate to 95% average degree of consolidation. Then additional fill was placed until failure. It can be seen that the stiffer the reinforcement, the greater the increase in embankment failure height due to foundation soil strength gain. These results are encouraging but the time to 95% consolidation was too long for most practical cases. This does, however, imply that there may be significant benefit arising from combining reinforcement with methods of accelerating consolidation, such as PVDs, as discussed in the following section.

5. Interaction Between Reinforcements and PVDs

Since the first prototype of a prefabricated drain made of cardboard (Kjellman, 1948), prefabricated vertical drains have been widely used in embankment construction projects, due to their advantages in terms of cost and ease of construction (*e.g.* Hansbo, 1981; Nicholson & Jardine, 1981; Jamiolkowski *et al.*, 1983; Holtz, 1987; Lockett & Mattox, 1987; Holtz *et al.*, 2001; Bergado *et al.*, 2002; Bo, 2004; Zhu & Yin, 2004; Chai *et al.*, 2006; Sinha *et al.*, 2007; Sinha *et al.* 2009 and Saowapakpiboon *et al.*, 2010). PVDs accelerate soil consolidation by shortening the drainage path and taking advantage of a naturally higher horizontal hydraulic conductivity of the foundation soil. This technique improves embankment stability by allowing strength gain in the foundation soil associated with the increase in effective stress due to consolidation.

The combined effects of reinforcement and PVDs have been investigated by Li & Rowe (1999, 2001) and Rowe & Taechakumthorn (2008a). It has been shown that the use of PVDs in conjunction with typical construction rates results in relatively rapid dissipation of excess pore pressures and when combined with geosynthetic reinforcement it enhances the stability of the embankment. Figure 7 shows the variation of net embankment height with fill thickness from finite element simulations, where S is the spacing of PVDs in a square pattern. For this particular foundation soil A (see insert in Fig. 7) and PVDs at a spacing of 2 m, the unreinforced embankment can be constructed to a height of 2.85 m. If reinforcement with tensile stiffness J = 250 kN/m is used, the failure height increases to 3.38 m. It is noted that, for these assumed soil properties

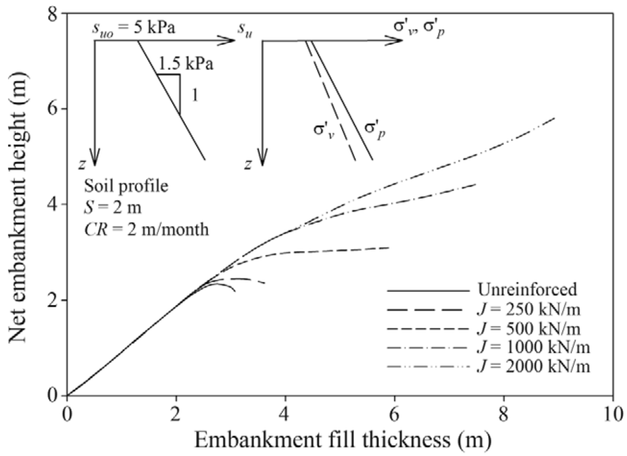

Figure 7 - The combined effect of reinforcement and PVDs on the short-term stability of the embankment (modified from Rowe & Li 2005).

and a construction rate of 2 m/month, the embankment will not fail due to bearing capacity failure of the foundation soil if the reinforcement stiffness is greater than 500 kN/m.

Reinforcement also reduces the shear stress and consequent shear deformations in the foundation soil. When the use of PVDs is combined with reinforcement, it can enhance the beneficial effect of the reinforcement in reducing horizontal deformations of the foundation soil below the embankment as illustrated in Fig. 8. With the use of PVDs, less stiff reinforcement can be employed while still providing about the same control on lateral deformation as the use of stiffer reinforcement without PVDs

6. Consolidation of the PVDs-Improved Soils Under Linear Loading Condition

Even though, the significant increase in degree of soil consolidation during embankment construction, owing to the presence of PVDs, has been reported (*e.g.* Lockett & Mattox, 1987; Fritzinger, 1990; Schimelfenyg *et al.*, 1990; Volk *et al.*, 1994; Holtz *et al.*, 2001; Bergado *et al.*, 2002; Bo, 2004; Zhu & Yin, 2004; Chai *et al.*, 2006; Sinha *et al.*, 2009 and Saowapakpiboon *et al.*, 2010), the magnitude and distribution of strength gain have received relatively little attention. Based on finite element analyses, Li & Rowe (2001) have shown that there is significant increase in undrained shear strength of foundation soils improved with PVDs. Figure 9 shows the contours of the increase in undrained shear strength of the foundation soil during construction for a reinforced (J = 2000 kN/m) embankment having height H = 4.4. For the sake of clarity, Fig. 9 does not include the increase in undrained shear strength near the top and bottom layers, where the gradient of shear strength increase is high because of the drainage boundary effects. Owing to the presence of the PVDs, the average increase in undrained shear strength was rather uniform throughout most of the thickness of the deposit (with some drainage boundary effects at the top and bottom of the foundation).

To analyze the consolidation of PVDs-improved soils during embankment construction, consideration should be given to vertical and radial drainage, construction rate, as well as the difference between consolidation coefficients of soils in the overconsolidated and normally consolidated stress ranges. Generally, a numerical analysis is required to consider these factors. Li & Rowe (2001) proposed an approximate method to calculate the consolidation of foundation soils allowing for the aforementioned factors. The proposed method can be performed by hand, or by using a spreadsheet calculation, without rigorous numerical analysis as outlined below.

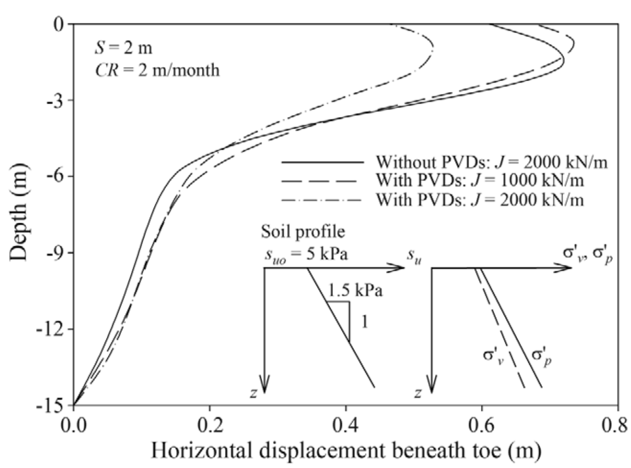

Figure 8 - The combined effect of reinforcement and PVDs on lateral deformation beneath the toe of the embankment (modified from Rowe & Li 2005).

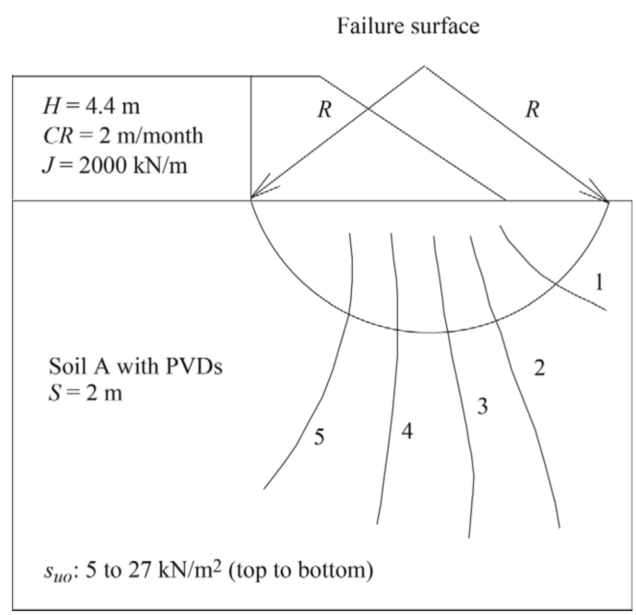

Figure 9 - Contours showing the increase in undrained shear strength, Δs_u in kPa, at end of construction, as calculated from FEM analyses (modified from Li & Rowe 2001).

The analysis is greatly simplified due to the fact that by including PVDs, the dissipation of pore pressure is essentially uniform with depth (except at the top and bottom boundaries) as implied by the strength gain contours shown in Fig. 9. The procedure, as described by Li & Rowe (2001) considers an embankment expected to apply a vertical stress of $\Delta\sigma$ over a period of time t_c as shown in Fig. 10. It is assumed that soil becomes normally consolidated when the average degree of consolidation at a particular time, $t_{O/C,}$ is such that the average vertical effective stress of the soil is equal to the preconsolidation pressure. At this time, the compressibility of the soil changes from the recompression index ($C_{O/C}$) to compression index ($C_{N/C}$). For a deposit with two-way drainage, the average degree of consolidation at any time is defined as:

$$\overline{U} = \frac{D\Delta\sigma(t) - \int_o^D u dz}{D\Delta\sigma} \tag{2}$$

where D is the thickness of the deposit; $\Delta\sigma(t)$ is the applied stress at time t; and u is excess pore pressure at time t. At time $t_{O/C}$ the average degree of consolidation is $\overline{U}_{O/C}$ (*i.e.* calculated using the coefficient of consolidation of soil in overconsolidated state, $c_{vO/C}$) for a total stress of $\Delta\sigma$, and the average change in effective stress at this time is $\Delta\sigma\overline{U}_{O/C}$. After the application of full stress $\Delta\sigma$, the average excess pore pressure that needs to dissipate is equal to $\Delta\sigma(1 - \overline{U}_{O/C})$. The remaining excess pore water pressure is assumed to be developed over a period of time due to a change in stress of $\Delta\sigma(1 - \overline{U}_{O/C})$. After $t_{O/C}$, the average degree of consolidation, $\overline{U}_{N/C}$, is calculated using coefficient of consolidation of soil in normally consolidated state ($c_{vN/C}$). Figure 10 shows that the linear load function, O-A, is replaced by two linear load functions: O-B and O'-A for soil in overconsolidated and normally consolidated states, respectively. It's assumed that average degree of consolidation under load O-A after time $t_{O/C}$ is equivalent to the average degree of consolidation under the load O'-A plus the average degree of consolidation under load O-B that occurred at time $t_{O/C}$. Therefore, the total average degree of consolidation at time $t \geq t_{O/C}$ is described as:

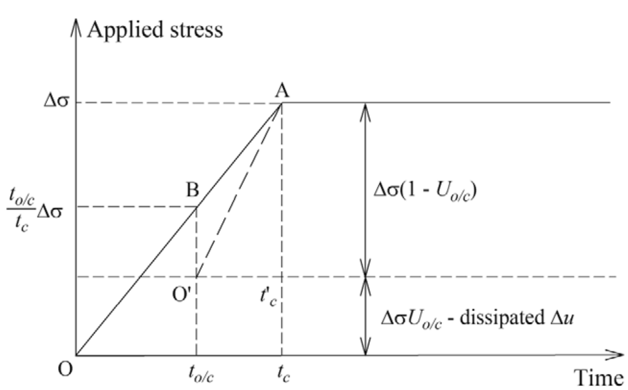

Figure 10 - Breakdown of linear ramp load function for consolidation analysis considering the soil in its overconsolidated and normally consolidated states (modified from Li & Rowe 2001).

$$\overline{U} = \overline{U}_{O/C} + (1 - \overline{U}_{O/C})\overline{U}_{N/C} \tag{3}$$

To consider the consolidation of soil under time-dependent loading, Olson (1977) derived relatively simple solutions considering both vertical and radial drainage for linear ramp loading problem. $\overline{U}_{O/C}$ and $\overline{U}_{N/C}$ can be calculated separately using Olson's (1977) solution as follows.

For vertical consolidation:

$$T \leq T_c: \overline{U}_v = \frac{T}{T_c}\left\{1 - \frac{2}{T}\sum\frac{1}{M^4}\left[1 - \exp(-M^2 T)\right]\right\} \tag{4}$$

$$T > T_c: \overline{U}_v = 1 - \frac{2}{T_c}\sum\frac{1}{M^4}\left[\exp(-M^2 T_c) - 1\right] \times \exp(-M^2 T) \tag{5}$$

where T is the time factor for vertical consolidation; T_c is the time factor at the end of construction and $M = \pi\,(2m + 1)/2$, $m = 0, 1, 2, 3$, until the sum of all remaining term is insignificant.

For horizontal (radial) consolidation:

$$T_h \leq T_{hc}: \overline{U}_h = \frac{1}{T_{hc}}\left\{T_h - \frac{1}{A}\left[1 - \exp(-AT_h)\right]\right\} \tag{6}$$

$$T_h > T_{hc}: \overline{U}_h = 1 - \frac{1}{AT_{hc}}\left[\exp(AT_{hc} - 1)\right]\exp(-AT_h) \tag{7}$$

where T_h is the time factor for horizontal consolidation; T_{hc} is the time factor at the end of construction and $A = 8/\mu$, μ is defined as:

$$\mu = \ln\left(\frac{n}{s}\right) + \left(\frac{k}{k_s}\right)\ln(s) - \frac{3}{4} + \pi z(2l - z)\frac{k}{q_w} \tag{8}$$

$$n = \frac{R}{r_w},\ s = \frac{r_s}{r_w} \quad \text{and} \quad q_w = \pi k_w r_w^2 \tag{9}$$

where: k, k_s and k_w are the hydraulic conductivity of soil in the horizontal direction, soil in the smear zone (the hydraulic conductivity of soil in smear zone was assumed to be isotropic and same as vertical hydraulic conductivity) and the vertical drain, respectively; q_w is the equivalent discharge capacity for the axisymmetric unit cell; r_w, r_s and R are the radius of the vertical drain, smear zone and influence zone, respectively. For the combined vertical and radial consolidation the method proposed by Carrillo (1942) can be employed as:

$$\overline{U} = 1 - (1 - \overline{U}_h)(1 - \overline{U}_v) \tag{10}$$

7. Design of Embankment of Soft Ground: Considering The Interaction Between Reinforcements and PVDs

Design of the reinforced embankment and PVDs are usually treated separately in current design methods even if both reinforcement and PVDs are used together. The design of reinforced embankments is usually based on undrained stability analyses without considering the effect of PVDs (*e.g.* Jewell, 1982; Mylleville & Rowe, 1988; Holtz *et al.*, 1997). Li & Rowe (2001) proposed a design method for reinforced embankment allowing the effect strength gain due to consolidation of the foundation soil. This design method is based on a limit state design philosophy and concepts proposed by Ladd (1991). The design procedure consists of four main steps, (a) select design criteria and parameters for both embankment fill and foundation soil, (b) establish the pattern and spacing of PVDs according to the required average degree of consolidation at the time to be considered, (c) estimate the average strength gain along the potential failure surface due to consolidation, and (d) select the required tensile stiffness of the reinforcement associated with the allowable compatible reinforcement strain (Rowe & Soderman, 1985 and Hinchberger & Rowe, 2003) using an undrained stability analysis (*i.e.* limit equilibrium method). The detailed design procedures, based on Li & Rowe (2001), are summarized as follows:

a) Select the design criteria and soil parameters including:

1. Embankment geometry: height (H), width (B), and side slope (n)

2. Required average degree of consolidation ($\overline{U}$) and available time (t) to achieve $\overline{U}$

3. Anticipated construction rate (CR)

4. Soil profile: undrained shear strength (s_u), preconsolidation pressure (σ'_p), vertical effective stress (σ'_v), coefficient of lateral earth pressure at rest (K'_o), coefficient of consolidation of soil in overconsolidated ($c_{vO/C}$) and normally consolidated ($c_{vN/C}$) state, vertical and horizontal hydraulic conductivity of the undisturbed soil (k_v and k_h), and hydraulic conductivity of the disturbed soil (k_s)

5. The longest vertical drainage path (H_d)

6. Embankment fill parameters: friction angle (ø) and bulk unit weight (γ_{fill})

b) Design of prefabricated vertical drains system:

1. Select the configuration of the PVDs system: installation pattern (*i.e.* triangular or square pattern), spacing of PVDs (S), and length of a single drain (L)

2. Estimate parameters used in radial consolidation analysis: effective diameter of drain influence zone (D_e), diameter of smear zone caused by installation (d_s), equivalent diameter (d_w) and equivalent discharge capacity (q_w)

3. Calculate the average degree of consolidation at available time, t, using Eqs. (2) and (3). If the calculated average degree of consolidation is less than the required $\overline{U}$, select the new PVDs configuration (*i.e.* spacing, S, and length, L) until $\overline{U}$ is met.

c) Estimate the average strength gain along the potential failure surface:

1. Estimate the average influence factor (I_q) for the increase in total stress along the potential failure surface using:

$$I_q = \frac{\Delta\sigma_m}{\Delta\sigma} \text{ where } \Delta\sigma = \gamma_{fill} H$$
$$\text{and } \Delta\sigma_m = \frac{1}{3}(\Delta\sigma_x + \Delta\sigma_y + \Delta\sigma_z) \tag{11}$$

where $\Delta\sigma_x$, $\Delta\sigma_y$ and $\Delta\sigma_z$ can be estimated using elastic solutions (*e.g.* Poulos & Davis, 1974).

2. Calculate the average degree of consolidation along the potential failure surface at the end of construction ($\overline{U}_f$).

3. Estimate the average strength increase (Δs_{uf}) of soil along the potential failure surface at the end of construction using the method proposed by Li & Rowe (2001) as:

$$\Delta s_{uf} = \left[\beta(\sigma'_{mi} + \gamma_{fill} H I_q \overline{U}_f)\right] - s_{uo}$$
$$\text{where } \beta = \frac{3}{1+2K'_o}\frac{s_u}{\sigma'_p} \tag{12}$$

where σ'_{mi} is the initial effective mean stress.

d) Selecting the required tensile stiffness of the reinforcement:

1. Apply partial factor to both load and resistance of the system as appropriate: f_c for the undrained shear strength of the foundation soil ($s^*_{uf} = s_{uf} / f_c$; $s_{uf} = s_{uo} + \Delta s_{uf}$), $f_ø$ for friction angle of fill material ($\tan^* \phi' = (\tan\phi') / f_\phi$), and f_γ for the unit weight of the fill material ($\gamma^*_{fill} = \gamma_{fill} f_\gamma$)

2. Use limit equilibrium method to calculate the equilibrium ratio (ERAT) of the restoring moment to overturning moment of the embankment without reinforcement using the factored soil parameters. If ERAT ≥ 1, the reinforcement is not needed. However, if ERAT < 1, reinforcement is required.

3. Use limit equilibrium program designed for the analysis of the reinforced embankment (*e.g.* REAP: Mylleville & Rowe, 1988) to calculate the required reinforcement tensile force, T_{req}, using the factored soil parameters (*i.e.* the tensile force that required to give ERAT = 1).

4. Choose an allowable reinforcement strain, ε_{all}, and then the required reinforcement stiffness can be selected as:

$$J \geq \frac{T_{req}}{\varepsilon_{all}} \tag{13}$$

This approach can be easily applied for a stage construction sequence by adding the consolidation during the stoppage between stages when calculating the average degree of consolidation, while keep the other steps the same. In order to ensure embankment stability during construction, it is important to monitor the development of reinforcement strains, excess pore water pressure, settlement, and horizontal deformation to confirm that the observed behaviour is consistent with the design assumptions (Rowe & Li, 2005).

8. Reinforced Embankment on Rate-Sensitive Soil

It has been recognized by many researchers (Lo & Morin, 1972; Vaid & Campanella, 1977; Vaid *et al.*, 1979; Graham *et al.*, 1983; Kabbaj *et al.*, 1988 and Leroueil, 1988) that natural soft deposits exhibit significant time-dependent behaviour and their undrained shear strength is strain-rate dependent (rate-sensitive). The performance of the reinforced embankment constructed on the rate-sensitive soil also has been investigated by both field studies and numerical analyses (Rowe *et al.*, 1996; Hinchberger & Rowe, 1998; Rowe & Hinchberger, 1998; Rowe & Li, 2002; and Rowe & Taechakumthorn, 2008a,b). For example, Rowe *et al.* (1996) showed that in order to accurately predict the responses of the Sackville embankment on a rate-sensitive soil, it is essential to consider the effect of soil viscosity. Rowe & Hinchberger (1998) proposed an elasto-viscoplastic constitutive model and demonstrated that the model could adequately describe the behaviour of the Sackville test embankment. The proposed model was also verified with another well documented field study, the Gloucester test embankment (Bozozuk & Leonards, 1972), and showed good prediction compared with the observed field data (Hinchberger & Rowe, 1998). Following subsections summarize the key finding from sensitivity analyses on the effect of soil viscosity using the aforementioned elasto-viscoplastic model (Rowe & Hinchberger, 1998).

8.1. Short-term stability of reinforced embankment

By definition, the undrained shear strength of rate-sensitive soils depends on the rate of loading (*i.e.* rate of embankment construction); the faster is the loading rate, the stronger the soil appears. For that reason, the loading rate is an important factor when conducting an analysis of embankment performance on a rate-sensitive soil. The effect of construction rate and geosynthetic reinforcement on the short-term stability of reinforced embankments is illustrated in Fig. 11. Series of reinforced embankments with axial stiffness of 0 (unreinforced), 500 and 1000 kN/m were constructed numerically at different construction rates until failure. It is evident (Fig. 11) that faster construction rate results in a higher short-term embankment failure height for all cases. The reinforcement also improved embankment stability. The stiffer the reinforcement, the higher the short-term failure height. However, this short-term benefit hides a long-term problem as will be discussed later.

8.2. Long-term mobilized reinforcement strains

To investigate the effect of the various parameters such as reinforcement stiffness, construction rate and the effect of PVDs on the long-term behaviour of a reinforced embankment on the rate-sensitive soil, a series of 5 m high reinforced embankments were numerically constructed on rate-sensitive foundation soil. The results from Case I and Case II (Fig. 12) show the effect of construction rate. The reinforcement strains at the end of the construction were 1.6% and 2.6% for Cases I and II, respectively. The reinforcement strain for the slower construction rate (Case II) was higher because the soil exhibited lower short-term strength and transferred more load to the reinforcement. However, this slower construction rate 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. This resulted in smaller long-term reinforcement strains. The results from Case I and III (Fig. 12) show the effect of reinforcement stiffness and as expected the stiffer reinforcement (Case III) gave smaller strains at both the end of construction and also in the long-term. Designers usually aim to limit reinforcement strains to about 5%-6% (Rowe & Li, 2005). The results for Cases I and II correspond to

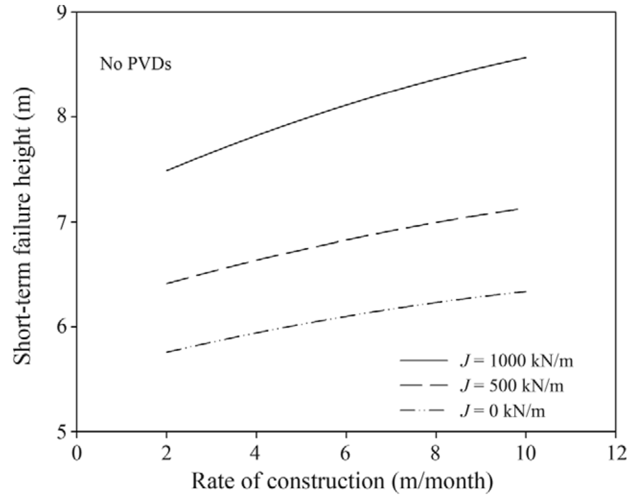

Figure 11 - The effect of construction rate and reinforcement stiffness on short-term stability of the embankment (modified from Rowe & Taechakumthorn 2008b).

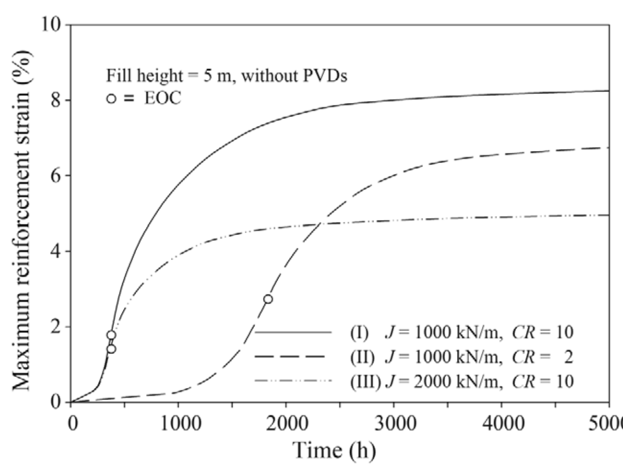

Figure 12 - The effect of construction rate and reinforcement stiffness on mobilized reinforcement strains (modified from Rowe & Taechakumthorn 2008a).

long-term reinforcement strains of 8.3% and 6.9%, respectively and hence exceed typical desirable limits. Stiffer geosynthetic reinforcement would be required to control the long-term reinforcement strain to within the allowable limit. For example with the stiffer reinforcement (Case III), the long-term reinforcement strain can be limited to 4.9%.

The rate of excess pore water dissipation and the consequent rate of shear strength gain in the soil can be increased using PVDs. Results given in Fig. 13 show that with the use of PVDs, the long-term mobilized reinforcement strain can be significantly reduced. For example the 5 m high reinforced embankment with the reinforcement stiffness J = 1000 kN/m, even a construction rate as low as 2 m/month, gave rise to a long-term reinforcement strains of 6.9% which exceeds the typical allowable limit of 5% (Fig. 12). In contrast, with PVDs at 3 m spacing and an even faster construction rate at 10 m/month, the construction still only gave a maximum long-term reinforcement strain of 4.6% (Case I, Fig. 13). With stiffer (J = 2000 kN/m) reinforcement, PVDs reduced the long-term reinforcement strain from 4.9% to 3.3% (Case III in Fig. 12 and Case II, Fig. 13). In fact, for reinforcement with a stiffness of 2000 kN/m, a reinforced embankment could be constructed up to 5.75 m without the long-term reinforcement strain exceeding about 5% (Case III, Fig. 13). For this same 5% long-term limit strain and PVDs at 3 m spacing, embankments could be constructed to 6.50 and 7.85 for J = 4000 and 8000 kN/m respectively (see insert to Fig. 13).

8.3. Excess pore water dissipation

In contrast to a rate-insensitive soft soil, for a rate-sensitive foundation there are two processes happen simultaneously during and following embankment construction: (a) excess pore water pressure dissipation due to consolidation, and (b) generation of excess pore water pressure due to the creep of the foundation soil. Figure 14 shows the contours of the change in excess pore water pressure between immediately after and 1 month after the end of construction for a 5 m high reinforced embankment (J = 2000 kN/m; no PVDs). The foundation soil has same basic soil properties as those of the rate-insensitive soil discussed earlier (*i.e.* insert drawing in Figs. 5 to 8) and the rate-sensitive characteristics similar to Sackville soil described by Rowe & Hinchberger (1998). The shear induced generation of pore pressures is evident in the areas of higher shear stress along the potential slip surface (Fig. 14). Thus, for rate-sensitive soil the maximum excess pore water pressure and hence the minimum factor of safety with respect to embankment stability, often occur after the end of construction.

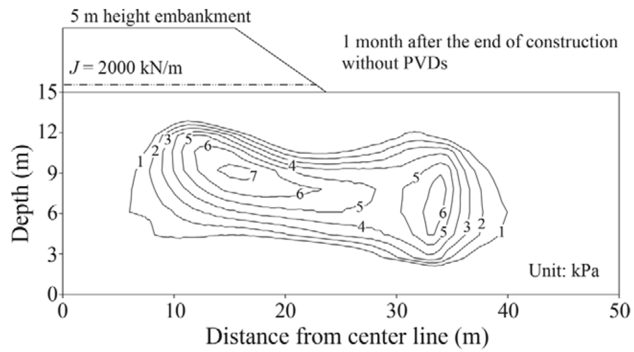

Figure 14 - Contours of the change in excess pore water pressure between immediately after and 1 month after the end of construction (modified from Taechakumthorn & Rowe 2008).

The effect of reinforcement stiffness and PVDs on the excess pore water pressure is presented in Fig. 15. The excess pore water pressures were monitored at a point 6 m beneath the crest of the embankment where the maximum increase in excess pore water pressure was indicated (Fig. 14). The excess pore water pressures at the end of construction were approximately 80 kPa for all cases at the construction rate of 10 m/month and kept increasing post construction for all reinforcement stiffnesses considered until a peak was reached. This phenomenon is similar to

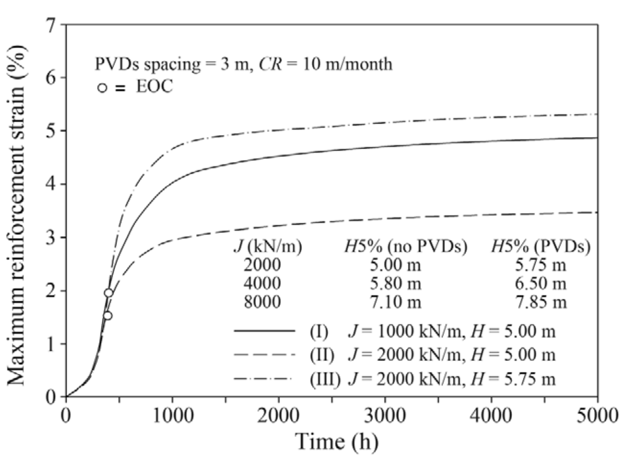

Figure 13 - The effect of PVDs and reinforcement stiffness on mobilized reinforcement strains (modified from Taechakumthorn & Rowe 2008).

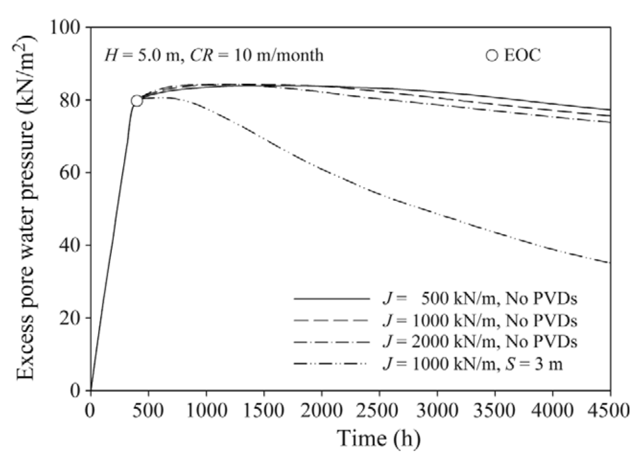

Figure 15 - The effect of reinforcement stiffness and PVDs on dissipation of excess pore pressures (modified from Taechakumthorn & Rowe 2008).

that observed at the Sackville test embankment (Rowe & Hinchberger, 1998). As noted above, the excess pore pressures decreased due to consolidation but also increased due to creep of the foundation soil. By providing greater confinement to the soil, the stiffer reinforcement reduced the effects of creep induced pore water pressure and resulted in faster dissipation of pore pressure as shown in Fig. 15. The installation of PVDs significantly minimized the effect of delayed excess pore water pressure on the rate-sensitive soil. As demonstrated in Fig. 15, with PVDs, the excess pore water pressure rapidly decreases following the end of construction.

8.4. Differential settlement and lateral deformation

Reinforcement has the potential to reduce differential settlement and heave of the foundation for embankments on rate-sensitive soil. Figures 16 and 17 show profiles of ground surface and lateral deformation beneath the toe for embankments with different reinforcement stiffnesses at 1 month 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 1.1 m but for the reinforced embankment, this was reduced to 0.5 and 0.3 m for reinforcement stiffness of 1000 and 2000 kN/m (Fig. 16). The maximum calculated heaves were 1.8, 0.8, and 0.6 m for the unreinforced embankment and for the reinforcement stiffnesses of 1000 and 2000 kN/m, respectively. The presence of PVDs considerably reduced the differential settlement of the foundation. The results from Case IV in Fig. 16 show that with the use of PVDs, even with the less stiff reinforcement (J = kN/m), the differential settlement was reduced to 0.2 m and the maximum heave was 0.5 m.

Reinforcement also had a beneficial effect on lateral deformation as demonstrated in Fig. 17. The maximum lateral deformation below the embankment toe was reduced from 2.4 m, for the unreinforced case, to 1.0 and 0.8 m for

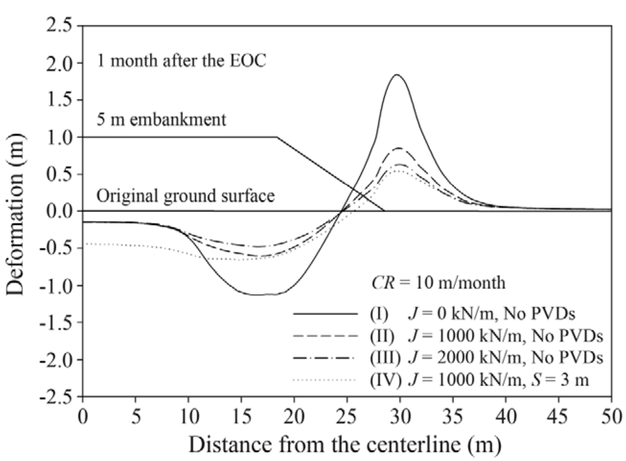

Figure 16 - The effect of reinforcement stiffness and PVDs on the differential settlement and heave of the foundation (modified from Taechakumthorn & Rowe 2008).

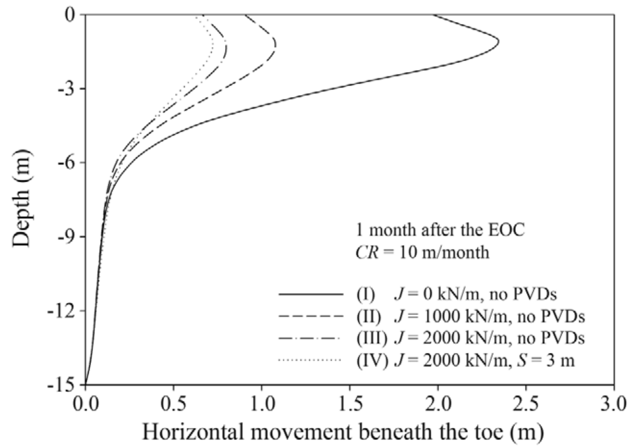

Figure 17 - The effect of reinforcement stiffness and PVDs on the differential and lateral deformation (modified from Taechakumthorn & Rowe 2008).

the reinforcement stiffness of 1000 and 2000 kN/m, respectively. With the use of lower reinforcement stiffness (J = 1000 kN/m) combined with PVDs, the maximum lateral deformation was reduced to only 0.7 m. This was smaller than that obtained from Case III using a reinforcement stiffness of 2000 kN/m, as a result of higher degree of partial consolidation and consequently higher soil strength increase as well as less overstress in the foundation occurs when the PVDs were employed.

9. Effects of Creep/Relaxation of Geosynthetics Reinforcements

Experimental studies have shown that geosynthetics typically made of polyester (PET), polypropylene (PP) and polyethylene (PE) are susceptible to creep to some extent (Allen *et al.*, 1982; McGown *et al.*, 1982; Christopher *et al.*, 1986; Greenwood & Myles, 1986; Jewell & Greenwood, 1988; Bathurst & Cai, 1994; Leshchinsky *et al.*, 1997; Shinoda & Bathurst, 2004; Jones & Clarke, 2007; Kongkitkul & Tatsuoka, 2007 and Yeo & Hsuan, 2010). The importance of considering creep/relaxation of geosynthetics reinforcement, to understand the time-dependent behaviour of the reinforced embankment on soft ground has been highlighted in the literature (Li & Rowe, 2001; Li & Rowe 2008 and Rowe & Taechakumthorn, 2008b).

For creep-sensitive reinforcement, the reinforcement strain may significantly increase with time owing to creep of the reinforcement after embankment construction (Li & Rowe, 2001). Figure 18 shows (solid lines) the development of reinforcement strain with time up to 98% consolidation for embankments reinforced (on rate-insensitive soil) using HDPE (upper figure) and PET (lower figure) geosynthetics. Also shown (dashed lines) are the strains that would be developed if the reinforcement was assumed elastic with stiffness selected such that, at the end of construction, the reinforcement strain is the same as that developed in the viscous reinforcement. Thus, the difference

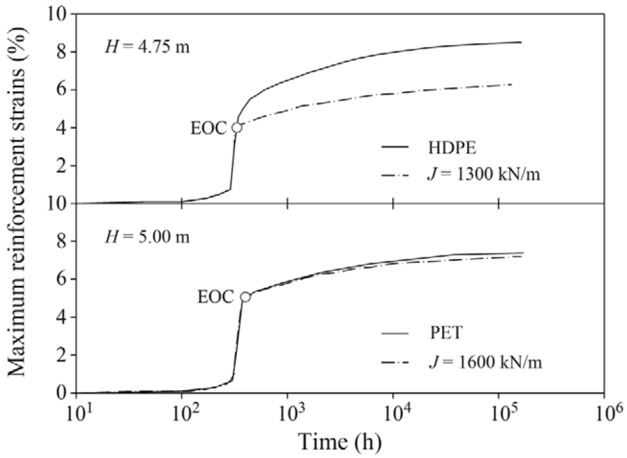

Figure 18 - Variation of reinforcement strain with time during and following embankment construction (modified from Rowe & Li 2005).

between the solid and dashed lines represents the creep strain due to the viscous nature of the reinforcement. For the PET reinforcement, creep is insignificant and the long-term reinforcement strains for both viscous and elastic reinforcement are practically the same. For the HDPE geogrid reinforcement, there is about 2% creep strain between the end of construction and the time of 98% consolidation.

Li & Rowe (2001) demonstrated that the isochronous stiffness deduced from standard creep test can reasonably represent the stiffness of geosynthetics reinforcement at the critical stage, for rate-insensitive foundation soils. The study also recommended that the isochronous stiffness should be used in design to estimate the mobilized reinforcing force at the end of embankment construction. Figure 19 compares the mobilized reinforcement stiffness with isochronous stiffness deduced from in-isolation creep test data during and after the construction of the HDPE geogrid and PET geosynthetic-reinforced embankments. It can be seen that the mobilized stiffness decreases with time and very

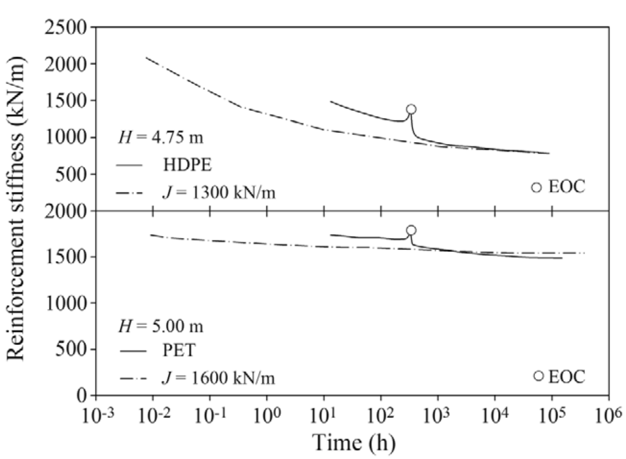

Figure 19 - Variation of reinforcement tensile stiffness with time during and following embankment construction (modified from Rowe & Li 2005).

closely approaches the isochronous stiffness in the long term. This also agrees with the finding of Li & Rowe (2008) and Rowe & Taechakumthorn (2008b) for the case of rate-sensitive foundation.

Time-dependence of the mobilized reinforcement stiffness shown in Fig. 19 also implies that the force in the reinforcement following the end of embankment construction may be significantly lower than expected in design owing to the viscous behaviour of geosynthetic reinforcement during embankment construction. This highlights the need for care when applying tensile stiffness from standard load-strain tests to deduce the design tensile force. In addition to creep effects, consideration should be given to potential construction damage of reinforcement (Allen & Bathurst, 1994, 1996).

10. Conclusions

The behaviour of reinforced embankments and the current design approaches have been examined for a number of different situations. The field case study of the Almere embankment shows that the use of geosynthetic reinforcement can substantially increase the failure height of the embankment over soft ground. The results demonstrated that the performance of the reinforced embankment can change significantly depending on the type of geosynthetic used and/or the nature of the foundation soil. Therefore, careful consideration must be given when selecting the type of constitutive relationship used to model each component of a reinforced embankment. Basal reinforcement can improve the stability of an embankment on both conventional (rate-insensitive) as well as rate-sensitive soil. Furthermore, the effect of partial consolidation during embankment construction can enhance the effect of reinforcement which encourages the combining of reinforcement with methods of accelerating consolidation, such as PVDs. When stage construction is required, the use of reinforcement can reduce the number of stages needed by increasing the height that can be safely attained in each stage. With the presence of PVDs, the assumption of total stress analysis is too conservative and the design method proposed by Li & Rowe (2001) can be employed to address the effect of strength gain, associated with the partial consolidation, during the construction.

For the reinforced embankment constructed over rate-sensitive soil, although the viscoplastic nature of the foundation can increase the short-term stability of the embankment, it significantly degrades the long-term embankment stability following the end of construction. The use of reinforcement provides a confining stress to the system and limits creep in the foundation. PVDs can provide a significant enhancement to the performance of reinforced embankments. For example, PVDs allow a higher degree of consolidation during and following the construction, which minimizes overstress and creep in the soil, and results in less differential settlement and lateral movement as well as long-term reinforcement strain.

Due to the time-dependent nature of the geosynthetic reinforcement, reinforcement stiffness at the end of construction is less than that provided by the standard tensile test. This implies that the reinforcement force used in the design may not represent what has been mobilized in the field. The isochronous stiffness measured from a standard creep tests appears reasonably, and conservatively, to represent the reinforcement stiffness in the field at the end of construction. The results also suggest that reinforcement creep and stress-relaxation allow an increase in the shear deformations of the foundation soil which will degrade the long-term performance of the reinforced embankment and may even lead to long-term failure, if the foundation soil exhibits strain-softening behaviour. Care must be taken in the design when dealing with creep-susceptible reinforcement and/or when the foundation soil is rate-sensitive.

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List of Symbols

J: tensile stiffness of reinforcement
ε_a: allowable compatible strain
ε_{all}: allowable reinforcement strain
T_{rein}: reinforcement tensile force
T_{req}: required reinforcement tensile force
Ω: dimensionless parameter
γ_f: bulk unit weight of the embankment fill
H_c: the collapse height of the unreinforced embankment
s_u: undrained shear strength of the soft foundation
E_u: modulus of the soft foundation
$(D/B)_e$: ratio of the effective depth of the deposit to the crest width
σ'_v: vertical effective stress
σ'_p: maximum preconsolidation pressure
S: spacing of PVDs
$\Delta\sigma$: apply vertical stress
$C_{N/C}$: compression index
$C_{O/C}$: recompression index
$\Delta\sigma(t)$: applied stress at time t
u: excess pore pressure at time t
$\overline{U}_{O/C}$: average degree of consolidation for the overconsolidated soil
$\overline{U}_{N/C}$: average degree of consolidation for the normally consolidated soil
T: time factor for vertical consolidation
T_c: time factor for vertical consolidation at the end of construction
T_h: time factor for horizontal consolidation
T_{hc}: time factor for horizontal consolidation at the end of construction
k_v: hydraulic conductivity of soil in the vertical direction
k_h: hydraulic conductivity of soil in the horizontal direction
k_s: hydraulic conductivity of soil in the smear zone
k_w: hydraulic conductivity of the vertical drain
q_w: equivalent discharge capacity for the axisymmetric unit cell
r_w: equivalent radius of the vertical drain
r_s: equivalent radius of the smear zone
R: equivalent radius of the influence zone
I_q: influence factor
fc: partial factor for the undrained shear strength of the foundation soil
$f\phi$: partial factor for friction angle of fill material
$f\gamma$: partial factor for the unit weight of the fill material