Reinforced Embankments on Soft Deposits: Behaviour, Analysis and Design

C. Taechakumthorn[1] and R.K. Rowe[2]
[1]*SEAFCO Public Company Limited, Bangkok, Thailand*
[2]*Department of Civil Engineering, Queen's University, Kingston, Ontario, Canada*
[2]*E-mail*: kerry@civil.queesu.ca

ABSTRACT: This paper reviews the key mechanisms on how basal reinforcement improves embankment behaviour through an examination of cases where embankments were constructed on soft organic clay and peat. The embankment responses from undrained and partially-drained simulations are compared to highlight the effect of consolidation during the construction. The benefits from the combined use of basal reinforcement and PVDs are presented. This paper also provides an overview of a design approach for embankments on soft ground taking account the interaction between basal reinforcement and PVDs as well as explores the effect the time-dependent behaviour of geosynthetics and rate-sensitive soils on the long-term performances of reinforced embankment under working condition. The limitations of the current design method are discussed. Finally a case study involving a reinforced embankment constructed over soft sensitive clay with a weathered crust is presented to illustrate the effect a stiff crust and soil structure can have on the effectiveness of the basal reinforcement used for this particular case.

KEYWORDS: Reinforced embankment, soft soil, basal reinforcement, geotextiles, geogrids

1. INTRODUCTION

Potential instability and excessive deformation (especially differential deformations) are the primary concerns in the design of embankments founded on soft deposit. When appropriately designed and installed, basal reinforcement has been shown to effectively increase embankment stability and decrease shear deformation (Rowe and Li 2005, Indraratna et al. 2005, Bergado and Teerawattanasuk 2008, Rowe and Taechakumthorn 2011, Taechakumthorn and Rowe 2012a, b). In parallel, prefabricated vertical drains (PVDs) have been widely used to accelerate consolidation of deep soft deposits. This technique provides a short horizontal drainage path and takes advantage of the normally higher horizontal (than vertical) permeability of natural soils. Thus, the rate of gain in strength and stiffness of the soil - as a result of consolidation - is increased (Bergado et al. 2002, Chai et al. 2006, Rowe and Taechakumthorn 2008, Indraratna et al. 2012). The combined use of geosynthetic reinforcement and PVDs has been shown to be extremely effective in reducing the post construction deformations of the embankment while allowing faster construction than could be safety considered with the use of either method alone (Li and Rowe 2001, Rowe and Li 2005, Rowe and Taechakumthorn 2008). In particular the combined use of PVD and basal reinforcement can allow effective surcharging of the embankment which can, on removal of the surcharge, reduce long term (creep) deformations.

This invited paper, which is to be included in a special issue honoring the significant contribution to soft soil engineering of Professor Dennis T. Bergado, reviews the key mechanisms by which basal reinforcement improves embankment behaviour through an examination of cases where embankments were constructed on soft organic clay and fibrous peat. The short-term and long-term performances of reinforced embankments are examined. The effect of partial drainage during construction and the presence of PVDs is illustrated. In the current design method, the analysis of basal reinforcement and PVDs are treated separately in the design, however this is not the most effective approach. This paper summarizes a recently published design approach for embankments on soft ground which considers the interaction between the basal reinforcement and PVDs. The role that the creep/relaxation of geosynthetic reinforcement and that of the viscosity of rate-sensitive soil reinforced embankments stability is discussed. The effect of such time-dependent characteristics on the performances of reinforced embankments under working stress conditions is discussed. A number of parametric studies are employed to highlight some concerns and potential problems that might be anticipated during design and construction. Recent research on the post-peak strength reduction of some soft natural deposits is discussed. To illustrate the limitation of basal reinforcement, a case study involving a reinforced embankment constructed over soft sensitive clay with a weathered crust is presented.

2. REINFORCED EMBANKMENT ON SOFT CLAY

When embankments are constructed on a soft foundation, the outward lateral thrust generated by the horizontal stress in the embankment fill results in an outward shear stress at the base of the embankment that reduce the bearing capacity of the foundation and hence the stability of the embankment (Jewell 1987). Basal reinforcement can support some (or, if stiff and strong enough, all) of the embankment outward shear stress; as a result, it increases the bearing capacity of the foundation and minimizes the lateral deformations of the embankment. If stiff and strong enough, the basal reinforcement can also resist the outward movement of the soft soil below the reinforcement thereby inducing an inward shear and further increasing bearing capacity (Jewell 1987; Rowe and Soderman 1987). In general, the design of reinforced embankment on soft ground focuses on the (i) bearing capacity of the soil, (ii) global stability of the embankment, (iii) pullout/anchorage capacity of the reinforcement, and (iv) embankment deformations (Leroueil and Rowe 2001). However before discussing design procedures, it is useful to understand when and how reinforcement contributes to the improvement of embankment stability.

The role of basal reinforcement can be demonstrated with respect to the Almere test embankments (SCW 1981, Rowe and Soderman 1984). A reinforced and an unreinforced test embankment were constructed on an approximate of 3.3 m thick soft organic clay deposit underlain by a dense sand layer. The undrained shear strength of the organic clay was about 8 kPa. A multi-filament woven geotextile, with the tensile stiffness (J) of 2000 kN/m, was used as a basal reinforcement. In both cases, a trench was excavated near the embankment toe and the soil used to form a retaining bank at the edge of the trench. In the case of the reinforced embankment, the excavated material to form a retaining bank was placed over a geotextile (insert to Figure 1a). In both cases, hydraulic fill was then placed behind the retaining bank until failure occurred. The reinforced section experienced a relatively ductile failure at a height of 2.75 m, after 25 hours of sand filling. In contrast, the unreinforced section failed rapidly at a height of 1.75 m. This case study shows clear evidence (i.e., a 60% increase in failure height) of the benefits arising from the inclusion of basal reinforcement. The measured increase in strains at location "A" (see insert to Figure 1a)

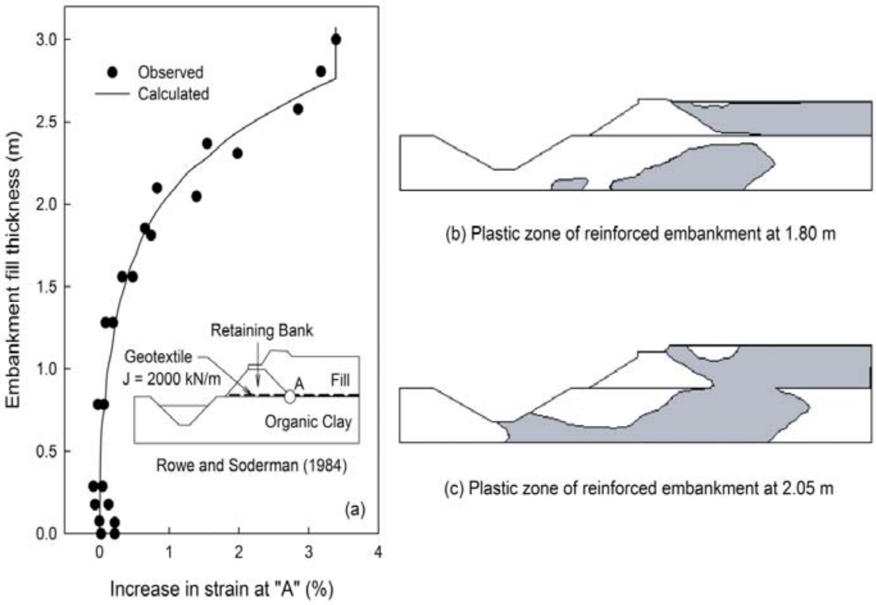

Figure 1 Comparison of predicted versus observed reinforcement strains at location "A" (see insert) and the development of plastic zone for reinforced and unreinforced embankments (modified from Rowe and Soderman 1984)

due to the placement of the hydraulic fill (Figure 1a) show that the reinforcement strains remained essentially constant at the fill heights less than 1 m because the clay foundation responded elastically and carried all the load induced by the fill up to this height. As the filling progressed from 1 m to 2 m, there was a gradual increase in the reinforcement strain as a zone of plasticity developed within the organic clay.

A finite element analysis indicated that, at a given embankment height, the reinforcement reduced the extension of the plastic region in the soil. For the unreinforced section, the predicted failure height was 1.8 m but for the reinforced section, at the same height, the displacements were smaller and the plastic region was contained (Figure 1b). The analysis indicated that a contiguous plastic region developed in the foundation at a fill height of 2.05 m (Figure 1c); approximately 15% higher than the corresponding height for the unreinforced embankment. For the reinforced embankment, the development of contiguous plastic region represented the point at which the foundation soil could not carry any additional load. Subsequently, embankment stability was entirely dependent on the basal reinforcement to support the additional stresses caused by further hydraulic filling. While geosynthetic reinforcement was maintaining the integrity of the system, the placing additional fill caused reinforcement strains to increase rapidly until failure occurred at a predicted height of 2.7 m (due to pullout of geotextile from below the retaining bank thereby limiting the force that could be developed in the reinforcement). In this case, the reinforcement increased the failure height by almost 60%

3. REINFORCED EMBANKMENT ON PEAT

Peat deposits are the partly decomposed and fragmented remains of vegetations that have accumulated under water and been largely preserved (Mesri and Ajlouni 2007). Typically these deposits possess a high percentage of fiber and a very high water content. Consequently, embankments constructed over peat deposits often experience excessive deformation and, sometimes, failure. Rowe (1984) discussed the difficulties in the analysis of embankments founded over peat deposits which include: (i) its behaviour cannot be categorized as either truly drained or undrained, (ii) the use of shear strength from the field vane shear test for peat is of doubtful validity, and (iii) the assumption of small strain implicit in the conventional limit equilibrium analyses is not applicable because of the large deformations. The behavior of reinforced and unreinforced embankments constructed on peat was investigated by Rowe and co-workers (Rowe et al. 1984, Rowe and Soderman 1985a, b, 1986, Rowe and Mylleville 1996). Some findings are presented herein to demonstrate the function of basal reinforcement and to emphasize key factors requiring consideration in the design and construction of embankments on peat.

At typical construction rates, the excess pore water pressures developed during the construction of embankments on peat deposits are less than would be expected for undrained conditions but may still have a significant effect on embankment performance (Rowe and Soderman1985a). For the analyses reported herein, the excess pore water pressure immediately after the end of construction was calculated using equation:

$$\Delta u = B\Delta\sigma_1 \quad (1)$$

where: Δu is the excess pore water pressure at a point, $\Delta\sigma_1$ is the increase in total major principle stress at that point, and B is an empirical pore water pressure parameter, assumed to vary with depth, and is given by:

$$B = (u/u_{max})B_{max} \quad (2)$$

where: the variation of (u/u_{max}) can be estimated using the isochrom of the excess pore water pressure variation with depth obtained from field piezometers, as shown in Figure 2 (Rowe and Soderman 1984). Clearly the maximum excess pore water pressure will depend on the rate of construction and the drainage conditions. The values of B_{max} which can be deduced from published field cases is typically in the range of 0.1 – 0.35 (Rowe 1984), although higher values have also been reported (Lupien et al. 1983). In general, it is impractical to construct the embankment so slowly no excess pore water pressures are developed. Rowe and Soderman (1985a) demonstrated that problems can be anticipated if the rate of embankment construction is too fast and results in B_{max} greater than 0.34. They recommended that B_{max} should be treated as a control parameter during the construction of embankments on peat. Thus the subsequent discussion will be only for cases where $B_{max} \leq 0.34$ since this value represents an upper bound for most of documented cases where embankments have been successfully constructed on peat (Rowe 1984).

Finite element simulations of embankments constructed on peat were conducted by Rowe and Soderman (1984) using a nonlinear elasto-plastic constitutive model with appropriate modification for the large strain analysis. They demonstrated the benefit of a basal reinforcement as shown in Figure 3. An inspection of the plastic zone, at the end of construction, shows that the unreinforced embankment (Figure 3a) could not be safely constructed to a height of 1.5 m above the original ground level (OGL) for the assumed condition (i.e., B_{max} = 0.34). In Figure 3a, the contiguous plastic zone that has developed to the ground surface corresponds to a rotation shear failure. This is confirmed by the settlement profile of where the vertical settlement beneath the crest is larger than that at the centerline of the embankment as a result of the rotational shear deformation.

However with the use of basal reinforcement (with a tensile stiffness J = 2000 kN/m), the growth of the plastic zone was limited (Figure 3b), resulting in a far dish-shaped settlement profile at the end of construction. Figure 3c shows the surface settlement profile and the plastic zone after the excess pore pressures have dissipated and the embankment in Figure 3b is brought back to grade (1.5 m above the original ground level). In this case the use of basal reinforcement with J = 2000 kN/m would allow a 1.5 m high embankment to be safely constructed on peat, even though the settlement (2.5 m) was large and a total fill thickness of 4 m would be required. Additional guidance regarding the construction of nts over fibrous peat is given by Rowe and Soderman (1985a, 1986)

4. PARTIALLY DRAINED ANALYSIS OF REINFORCED EMBANKMENTS

Field observations of excess pore water pressure beneath embankments on soft ground suggest that, at a typical construction rates, significant consolidation can occur during the construction (Crooks et al. 1984, Leroueil and Rowe 2001). This partial consolidation during embankment construction has been reported to give a corresponding significant increase in foundation shear strength (e.g., Bergado et al. 2002, Bo 2004, Chai et al. 2006, Saowapakpiboon et al. 2010) for natural soft clay deposits that are usually slightly overconsolidated prior to embankment construction.

While field cases highlight the importance of partial consolidation, they do not provide a direct comparison between cases where effect of consolidation was, or was not, considered. Finite element analyses, however, do provide this insight regarding the expected performance of the reinforced embankments constructed under undrained and partially drained conditions. In finite element analysis, the performance of embankments can be evaluated using the concept of net embankment height, defined as the fill thickness minus the maximum settlement (Rowe and Soderman 1985b). The failure height (i.e., maximum net embankment height) is controlled by (i) when the reinforcement reaches its ultimate tensile strength, or (ii) its pullout capacity (e.g. in the Almere case – Rowe and Soderman 1984), or (iii) when there is excessive deformation (e.g. embankment founded on peat – Rowe and Soderman 1985a, 1986) such that the addition of more fill during construction does not result in any increase in the net embankment height.

Figure 4 shows the variation in calculated embankment failure height with reinforcement stiffness for undrained and partially drained conditions. With a simulated construction rate of 1 m/month and a particular soil profile (insert to Figure 4), considering the consolidation that occurred during construction gave an increase in failure height of the unreinforced embankment from 2.1 m (for undrained analysis) to 2.4 m. An increase of reinforcement stiffness from 500 kN/m to 8000 kN/m also increased failure height by between 0.8 m and 2.5 m when drainage is permitted, compared with between 0.7 m to 1.4 m for the undrained condition. Thus the reinforcement had a greater effect for the partially drained cases than for undrained cases. However, for this particular soil profile, the increase in reinforcement stiffness had the most significant effect on the embankment failure height for stiffness values up to only 2000 kN/m because the maximum failure height of the embankments was partly governed by the ultimate bearing capacity of the foundation soil. Since most designs without PVDs assume undrained conditions the improvement in performance that arises with partial drainage during construction is a reason why the observed strains in the reinforcement are less than expected based on the design.

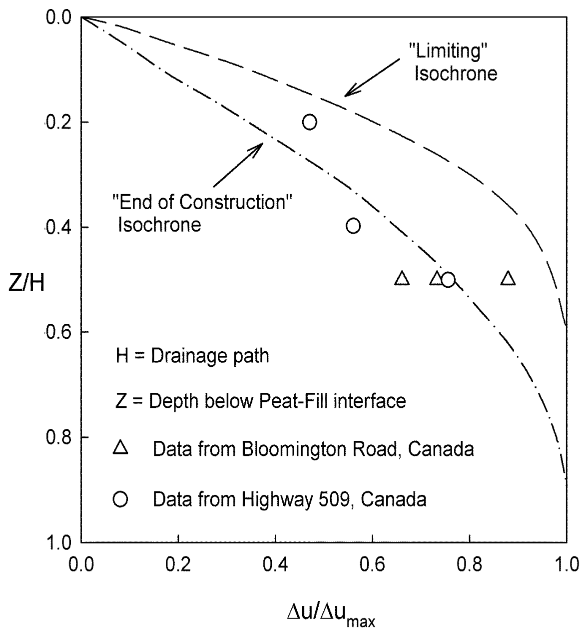

Figure 2 Variation in excess pore water pressure with depth for embankments constructed on peat (modified from Rowe and Soderman 1985a)

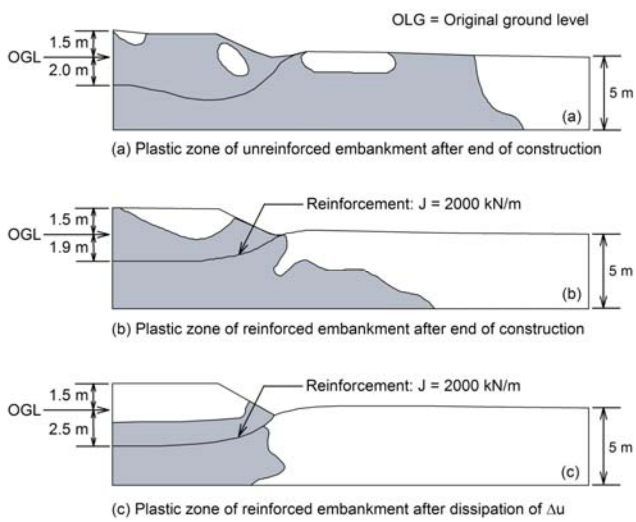

Figure 3 Deformed profile and calculated plastic zone for unreinforced and reinforced embankments founded on fibrous peat deposits (modified from Rowe and Soderman 1985a)

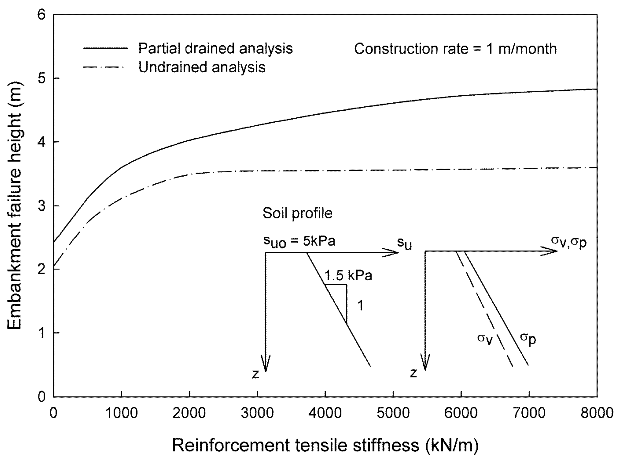

Figure 4 Effect of partial consolidation and reinforcement stiffness on the failure height of embankments (modified from Rowe and Li 1999)

The effects of partial consolidation are usually not relied upon unless action is taken to ensure that to ensure a given level of pore pressure of dissipation can be achieved. The "stage construction" technique is one means of doing so. However, in many cases the waiting period for consolidation during stage construction is impractically long. A convenient alternative is the combined use of reinforcement with the use of prefabricated vertical drains (PVDs) to accelerating consolidation.

5. INTERACTION BETWEEN REINFORCEMENT AND PVDs

Due to the advantages in terms of cost and ease of construction, prefabricated vertical drains (PVDs) have been widely used to accelerate consolidation in the construction of embankments over soft soil (Bergado et al. 2002, Bo 2004, Rowe and Taechakumthorn 2008, Indraratna et al. 2012). The combined effects of reinforcement and PVDs have been investigated (Rowe and Li 1999, Li and Rowe 2001, Rowe and Taechakumthorn 2008). It has shown that the use of PVDs with typical construction rates allows relatively rapid dissipation of excess pore water pressures and when combined with basal reinforcement it can greatly enhance the stability of the embankment. For example, the variation of net embankment height with fill thickness from finite element analyses is presented in Figure 5, where S is the spacing of PVDs in a square pattern. For this particular case (i.e., PVDs with a spacing of 2 m and soil profile as shown in the insert to Figure 5), the unreinforced embankment can be constructed to a height of 2.8 m. If reinforcement with tensile stiffness J = 250 kN/m is used, the failure height increases to 3.4 m. For these assumed soil properties and a construction rate of 2m/month, the embankment will not fail due to bearing capacity failure of the foundation if the reinforcement stiffness is greater than 500 kN/m.

As discussed earlier, basal reinforcement reduces the outward shear stress and if stiff enough induces inward shear stresses giving a consequent decrease in the shear deformations in the foundation. When the use of PVDs is combined with reinforcement, it can enhance the beneficial effect of the reinforcement by further reducing horizontal deformations of the soil below the embankment (Figure 6). 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 (e.g., Figure 6). A technique for taking account of both the role of PVDs and basal reinforcement is outlined in the next section.

6. DESIGN OF REINFORCED EMBANKMENT ON SOFT GROUND

Li and Rowe (2001) proposed a design method for reinforced embankments incorporating the effect of strength gain caused by consolidation of the foundation soils. This approach is based on a limit state design philosophy and the SHANSEP (stress histories and normalised soil engineering properties) concept proposed by Ladd and Foott (1974). The proposed design procedure consists of four main steps: (i) selecting design criteria and parameters for both fill material and foundation soil, (ii) establishing the pattern and spacing of PVDs according to the required average degree of consolidation at the time being considered, (iii) estimating the average shear strength gain due to consolidation along the potential failure surface, and (iv) selecting the required tensile stiffness of the reinforcement associated with the allowable compatible strain (Rowe and Soderman 1985b, Hinchberger and Rowe 2003), using undrained stability analysis (i.e., limit equilibrium method).

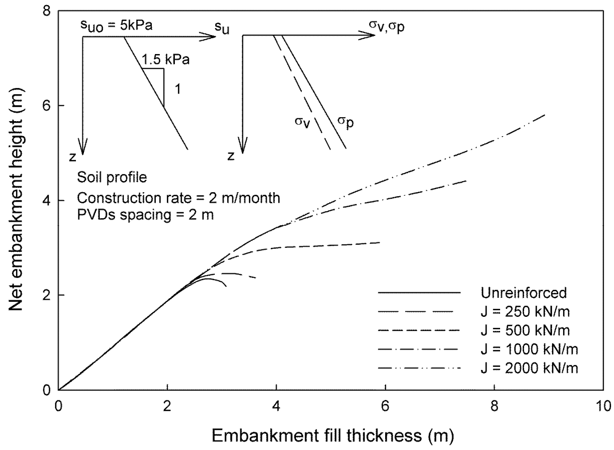

Figure 5 The combined effect of reinforcement and PVDs on the short-term stability embankments (modified from Rowe and Li 2005)

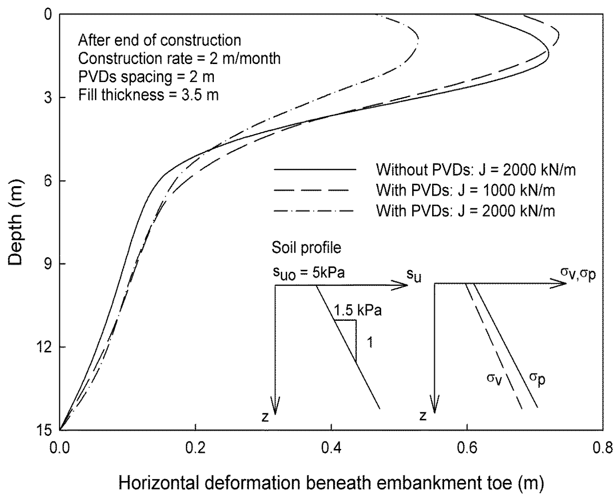

Figure 6 The combined effect of reinforcement and PVDs on horizontal deformation beneath the toe of three reinforced embankment (modified from Rowe and Li 2005)

Firstly, the design criteria and representative soil parameters are selected/estimated including embankment geometry, required average degree of soil consolidation and the available time to achieve the requirement, anticipated average construction rate, soil profile (i.e., undrained shear strength, preconsolidation pressure, vertical effective stress, both coefficient of consolidation for soil in normally consolidated and overconsolidated state, as well as vertical and horizontal hydraulic conductivity), longest vertical drainage path, and embankment fill properties. Second, the designer selects the configuration of the PVDs including pattern (i.e., triangular or square pattern), spacing and length of PVDs. Then the method proposed by Li and Rowe (2001) can be utilized to calculate the average degree of consolidation at a specific time, required in the design criteria. If the calculated average degree of consolidation is less than the required average degree of consolidation, the designer must select a new PVDs configuration (e.g, spacing, S) until the required average degree of consolidation is met.

Once a PVDs system has been selected to give the specified average degree of consolidation at the required time, the strength gain of soils under the embankment centre can be estimate using SHANSEP method (Ladd and Foott 1974). For locations along the potential failure surface, the average increase in undrained shear strength can be estimated using the method presented by Li and Rowe (2001). Using this undrained shear strength, the required reinforcement forces are calculated using a limit equilibrium analysis. Finally, knowing the required reinforcement forces associated with the required factor of safety and the allowable reinforcement strain, the design reinforcement stiffness can be established. Details and example calculations associated with the design approach summarized in this paper are provided in Li and Rowe (2001).

This approach can be easily used for a stage construction by adding the consolidation during the stoppage between stages when calculating the average degree of consolidation, while keeping the other steps the same. To ensure embankment stability during construction, it is important to monitor the development of reinforcement strains, excess pore water pressures, settlement, and horizontal deformation to confirm that the observed behavior is consistence with the design assumptions (Rowe and Li 2005).

7. EFFECT OF SOIL AND REINFORCEMNET VISCOSITY

Many natural soft deposits exhibit significant time-dependent behaviour such that their undrained shear strength and stiffness are strain-rate dependent (Lo and Morin 1972, Vaid and Campanella 1977, Vaid et al. 1979, Graham et al. 1983, Leroueil 1988). Embankments constructed on these soils are often accompanied by the development of creep induced excess pore pressures causing a reduction in effective stress and shear strength after the end of construction. Figure 7 shows the contours of the increase in excess pore water pressure, deduced from a finite element simulation, 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 in Figures 4 to 6) and the rate-sensitive characteristics similar to Sackville soil described by Rowe and Hinchberger (1998). The generation of shear induced pore pressures is evident in the areas of higher shear stress along the potential slip surface. 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. In parallel, experimental studies have shown that geosynthetics made of polyester (PET), and especially those made of polypropylene (PP) and polyethylene (PE), are susceptible to creep/relaxation (Leshchinsky et al. 1997, Shinoda and Bathurst 2004, Kongkitkul and Tatsuoka 2007, Yeo and Hsuan 2010, Bathurst et al. 2012). This time-dependant response of the geosynthetic reinforcement can be particularly important for rate-sensitive foundation.

Taechakumthorn and Rowe (2012a, b) demonstrated that even if the allowable long-term reinforcement strain is limited to about 5% as used in a common design practice (FHWA 1995), the combined effect of reinforcement and soil viscosity could result in embankment deformations too large for some engineering application, such as embankment supporting a major highway and railway. Examining the effect of an allowable long-term reinforcement strain on the long-term net embankment height of an embankment on a rate-sensitive soil R1 (i.e., with properties similar to Sackville soil - Rowe and Hinchberger 1998) showed that, at a construction rate of 10 m/month and with HDPE geogrid (GR1) reinforcement, the net embankment height only increased by 0.14 m when the allowable strain was increased from 3% to 5% (Figure 8). Although the net embankment can be increased, it has a significant effect on embankment deformations. Figure 9 shows the relationships between the net embankment height and the horizontal toe displacement. The 5% increase in the net embankment height from 2.90 m to 3.04 m (i.e., increasing the allowable reinforcement strain from 3% and 5%), caused the horizontal toe displacement to increase by about 0.3 m (i.e., a 67% increase). The rapid increase in horizontal toe displacement is indicative of significant the plastic shear failure in foundation soil when the allowable long-term strain exceeds about 3%. This suggests that to control embankment deformations on these rate sensitive soils such as the Sackville soil (Soil R1), the allowable long-term reinforcement strain should probably not exceed about 3%. Similar parametric studies performed for a soil (Soil R2) which captures the average behavior of 26 soft cohesive clays reported by Kulhawy and Mayne (1990) indicated that for this soil, the optimum allowable reinforcement strain was about 4%.

Based on a series of sensitivity analyses, the design approach for reinforced embankments on soft ground that considers both the effect of soil and reinforcement viscosity has been proposed (Rowe and Taechakumthorn 2011). This approach is based on the limit state design concept and incorporates concepts of the in-situ soil strength at what is termed the “critical stage”, the operating reinforcement stiffness (i.e., stiffness selected from creep tests at the corresponding strain rate in the field at critical stage) as well as the optimum allowable long-term strains discussed above.

8. EMBANKMENTS OVER HIGHLY SENSITIVE CLAYS

Highly sensitive clays are found in locations such as eastern Canada and Scandinavia. For this problematic soil, plastic straining that occurs during deformation progressively breaks down the inter-particle bonding and results in a very substantial post peak strength reduction (Vaid et al. 1979, Quigley 1980, Leroueil and Vaughan 1990, Torrance 1999, Lo and Hinchberger 2006, Hinchberger and Qu 2009). Taechakumthorn and Rowe (2012c) modified an existing elastoviscoplastic constitutive model (Rowe and Hinchberger 1998) by incorporating the concept of state-dependent soil fluidity parameters and the damage strain (Hinchberger and Qu 2009) to account for the strain softening nature of the highly sensitive clays. The modified model (i.e., structured soil model) was employed to simulate the performance of a well documented case study of a reinforced test embankment constructed on sensitive Champlain clay deposit in Saint Alban, Quebec (Busbridge et al. 1985). To examine the benefit of basal reinforcement for this specific case, analyses were performed assuming: (i) elastic reinforcement with an axial tensile stiffness of 300 kN/m (the long-term stiffness of the HDPE reinforcement), (ii) the viscoelastic properties of the HDPE reinforcement actually used, and (iii) no reinforcement. Significantly, the soil profile comprised a 2 m thick weathered clay crust underlain by a 13.7 m thick deposit of soft grey-blue marine clay. Beneath the clay there is a layer of dense fine to medium coarse sand underlain by bedrock (Busbridge et al. 1985).

The test embankment failed at a height of 6.1 m about 10 days after the start of the construction. Assuming elastic reinforcement with J = 300 kN/m, the calculated failure height of the reinforced embankment is 6.0 m. Numerical simulations using the viscoelastic HDPE reinforcement and no reinforcement gave slightly smaller failure heights of 5.9 m. Thus, for this particular soil profile, basal reinforcement had very little effect on the stability of the embankment. Figures 10 and 11 compare the observed field data and the calculated results of vertical settlement at the centerline (SP-9) and excess pore water pressure at 3.0 m below the crest (PN-15) of the reinforced embankment, respectively. The almost identical calculated results for all cases (i.e., without reinforcement and with elastic and viscoelastic HDPE reinforcement) imply that there is very little, if any benefit, to be realized from the inclusion of

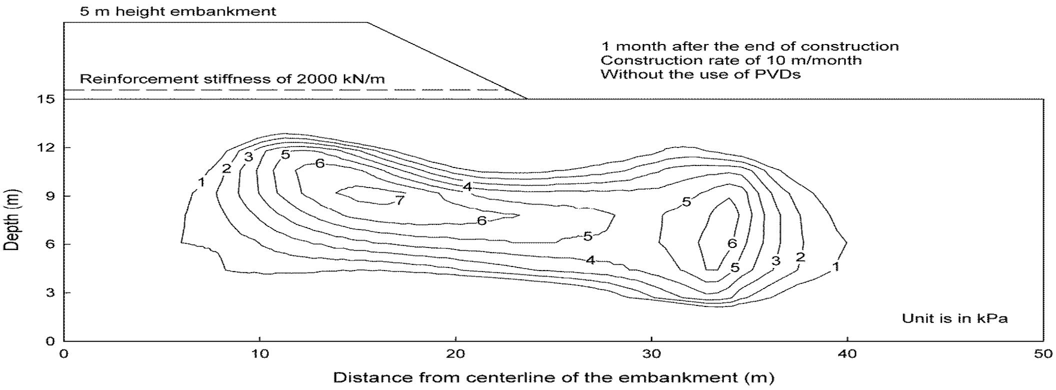

Figure 7 Contours of the increase in excess pore water pressure between immediately after and 1 month after the end of construction on a rate sensitive soil (modified from Rowe and Taechakumthorn 2008)

this basal reinforcement, in this particular case. This is probably because the reinforcement was not sufficiently stiff relative to the overconsolidated crust to improve the performance of the embankment prior to failure and was not strong enough to prevent the failure once strain softening of this sensitive clay was initiated (Taechakumthorn and Rowe 2012c). However, these comparisons show that the structured soil model can reasonably predict the performance of the test embankments on highly sensitive clay.

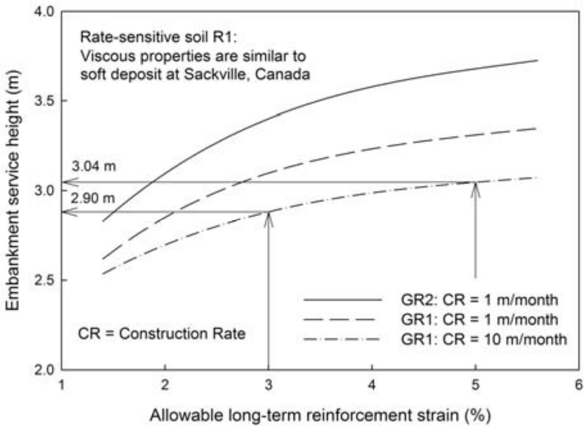

Figure 8 Effect of allowable long-term reinforcement strain on embankment service height for three reinforced embankments constructed on Soil R1 (modified from Taechakumthorn and Rowe 2012a)

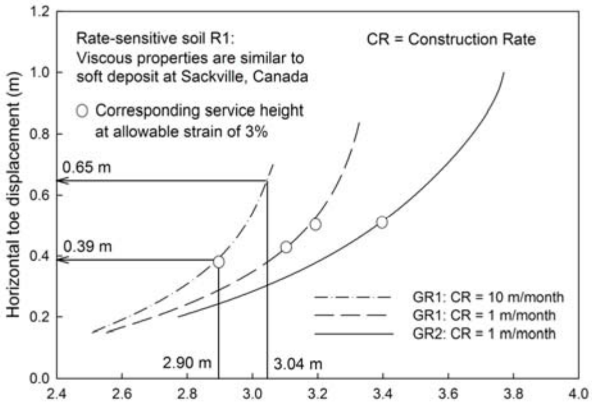

Figure 9 Effect of construction rate and reinforcement type on horizontal toe movement of three reinforced embankments constructed on Soil R1 (modified from Taechakumthorn and Rowe 2012a)

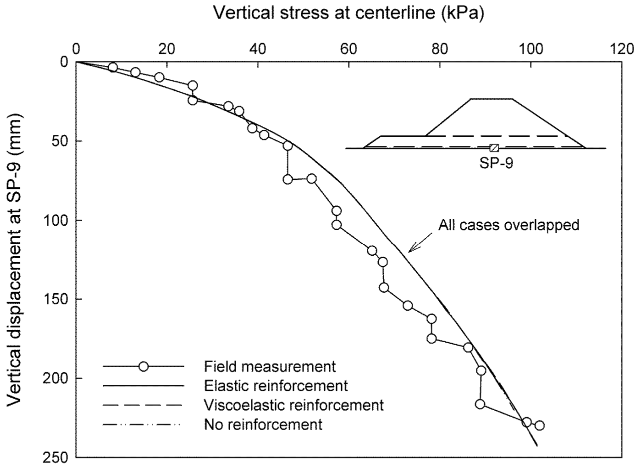

Figure 10 Vertical settlement at the centerline of the reinforced embankment at location SP-9 below the centerline of the embankment (modified from Taechakumthorn and Rowe 2012c)

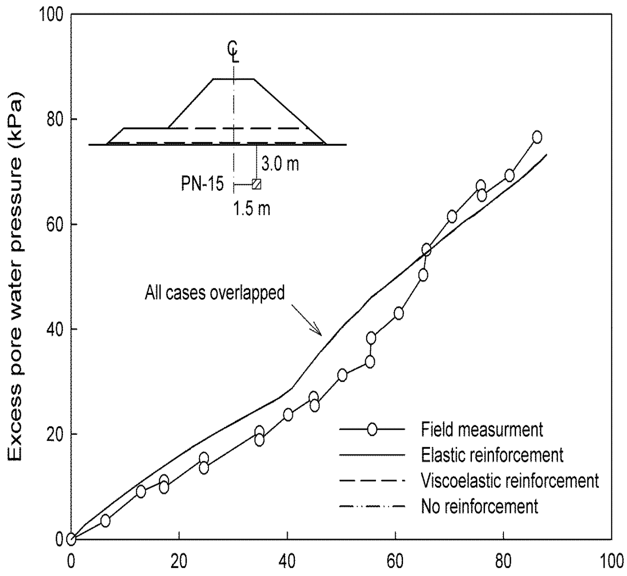

Figure11 Relationship between increase in total vertical stress and excess pore water pressure at piezometer PN-15 (modified from Taechakumthorn and Rowe 2012c)

9. CONCLUSIONS

The behavior of unreinforced and reinforced embankments have been examined for a number of different situations. The field case study of the Almere embankment shows that the inclusion of basal reinforcement provides the additional confining stress to the reinforced system and the foundation. This improves the bearing capacity and minimizes the growth of plastic failure zone in soft foundation soils, and hence substantially increases the failure height of the embankment. The numerical simulation of embankment construction on fibrous peat suggests that there will usually be significant dissipation of pore water pressures during construction but that stability will be controlled by the remaining excess pore water pressures. Basal reinforcement can be very useful for allowing the construction of higher embankments and on peat, especially when used to allow surcharging since, following removal of the surcharge, the long-term deformations of the peat can be greatly reduced. However even with the use of basal reinforcement, it is recommended that the rate of construction should be controlled such that the maximum excess pore pressure does not exceed about 34% of the increase in total major principle stress.

The effect of increase in soil strength and stiffness due to consolidation during embankment construction can enhance the beneficial effect of basal reinforcement. This encourages the combining of reinforcement with methods of accelerating consolidation, such as PVDs. When PVDs are used together with basal reinforcement, the combination allows the cost-effective construction of significantly higher embankments on soft clay in a substantially shorter time than could be achieved using either technique alone. The design method proposed by Li and Rowe (2001) can be used to consider the effect of strength gain with the partial consolidation during the construction (e.g., with the use of PVDs) when combined with the use of basal reinforcement.

For rate-sensitive soils, the most critical situation with respect to the embankment stability may occur following the end of construction due to the generation of creep induced excess pore water pressures. Because of the time-dependent nature of rate-sensitive soils and geosynthetic reinforcement, the use of the traditional 5% allowable strain in design may lead to excessive deformations and violate serviceability limits for important structures. Based on parametric studies for a range of rate-sensitive soils and viscoelastic characteristics of commonly used geosynthetic reinforcement, it is suggested that for these soils the maximum allowable long-term reinforcement strains should be limited to about 3% to prevent excessive deformation while optimizing the service height the reinforced embankments.

Finally, although basal reinforcement can significantly improve embankments stability in many practical situations involving soft soil, it is not suitable for all soft soils. For examples on highly sensitive soils (especially those with an over consolidated crust) traditional HDPE geogrid reinforcement is neither stiff enough to play any significant role prior to the onset of foundation failure nor strong enough to control the failure once strain softening is initiated in quick clay.

10. ACKNOWLEDGEMENTS

The research reported in this paper was funded by a research grant from the Natural Sciences and Engineering Research Council of Canada (NSERC).

11. REFERENCES

Bathurst, R.J., Huang, B.Q., and Allen, T.M. (2012). “Interpretation of laboratory creep testing for reliability-based analysis and load resistance factor design (LFRD) calibration”. Geosynthetics International, 19(1): 39-53.

Bergado, D.T., and Teerawattanasuk, C. (2008). “2D and 3D numerical simulations of reinforced embankments on soft ground”. Geotextiles and Geomembranes, 26(1): 39-55.

Bergado, D.T., Balasubramaniam, A.S., Fannin, R.J., and Holtz, R.D. (2002). “Prefabricated vertical drains (PVDs) on soft Bangkok clay: a case study of the new Bangkok International Airport project”. Canadian Geotechnical Journal, 39(4): 304-315.

Bo, M.W. (2004). “Discharge capacity of prefabricated vertical drains and their field measurements”. Geotextiles and Geomembranes, 22(1-2): 37-48.

Busbridge, J.R., Chan, P., Milligan, V., La Rochelle, P.R., and Lefebvre, L.D. (1985). The effect of geogrid reinforcement in the stability of embankments on a soft sensitive Champlain clay deposit. Summary report prepared for Transportation development centre, Transport Canada.

Chai, J.C., Carter, J.P., and Hayashi, S. (2006). “Vacuum consolidation and its combination with embankment loading”. Canadian Geotechnical Journal, 43(10): 985-996.

Crooks, J.H., Becker, D.E., Jeffries, M.G., and McKenzis, K. (1984). “Yield behaviour and consolidation I: Pore pressure response”. In Proceedings of ASCE Symposium on Sedimentation Consolidation Models, Prediction and Validation, San Francisco: 356-381.

Federal Highway Administration (1995). Geosynthetics Design and Construction Guidelines. National Highway Institute, McLean, Virginia, USA.

Graham, J., Crooks, H.A., and Bell, A.L. (1983). “Time effects on the stress-strain behavior of natural soft clays”. Geotechnique, 33(3): 327-340.

Hinchberger, S.D., and Qu, G. (2009). “A constitutive approach for rate-sensitive structured clay”. Canadian Geotechnical Journal, 46(6): 609-626.

Hinchberger, S.D., and Rowe, R.K. (2003). “Geosynthetic reinforced embankments on soft clay foundations: predicting reinforcement strain at failure”. Geotextiles and Geomembranes, 21(3): 151-175.

Indraratna, B., Rujikiatkamjorn, C., Balasubramaniam A.S., and McIntosh, G. (2012). “Soft ground improvement via vertical drains and vacuum assisted preloading”. Geotextiles and Geomombranes, 1(1): 16-23.

Indraratna, B., Sathananthan, I., Bamunawita, C., and Balasubramaniam, A.S. (2005). Chapter 2 - Theoretical and numerical perspectives and field observations for the design and performance evaluation of embankments constructed on soft marine clay. Ground Improvement – Case Histories, Indraratna and Chu (ed.): 51-90.

Jewell, R.A. (1987). Reinforced soil walls analysis and behavior. The Application of Polymeric Reinforcement in Soil Retaining Structures, Jarret P.M., and McGown, A. (ed.) Kluwer Academic Publishers: 365-408.

Kongkitkul, W., and Tatsuoka, F. (2007). “A theoretical framework to analyze the behavior of polymer geosynthetic reinforcement in temperature-accelerated creep test”. Geosynthetics International, 14(1): 23-38.

Kulhawy, F.H., and Mayne, P.W. (1990). Manual on Estimate Soil Properties for Foundation Design. Report No. EL-6800, Electric Power Research Institute, Palo Alto, CA.

Ladd, C.C., and Foott, R. (1974). “New design procedure for stability of soft clay”. Journal of the Geotechnical Engineering, 100(GT7): 763-786.

Leroueil, S. (1988). “Tenth Canadian Geotechnical Colloquium: Recent developments in consolidation of natural clays”. Canadian Geotechnical Journal, 25(1): 85-107.

Leroueil, S., and Rowe, R.K. (2001). Embankments over soft soil and peat. Geotechnical and Geoenvironmental Engineering Handbook, R.K. Rowe (ed.), Kluwer Norwell, MA: Kluwer Academic: 463-499.

Leroueil, S., and Vaughan, P.R. (1990). “The general and congruent effects of structure in natural soils and weak rocks”. Geotechnique, 40(3): 467-488.

Leshchinsky, D., Dechasakulsom, M., Kaliakin, V.N., and Ling, H.I. (1997). Creep and stress relaxation of geogrids. Geosynthetics International, 4(5): 463-479.

Li, A.L., and Rowe, R.K. (2001). "Combined effects of reinforcement and prefabricated vertical drains on embankment performance". Canadian Geotechnical Journal, 38(6): 1266-1282.

Lo, K.Y., and Morin, J.P. (1972). "Strength anisotropy and time effects of two sensitive clays". Canadian Geotechnical Journal, 9(3): 261-277.

Lo, K.Y., and Hinchberger, S.D. (2006). "Stability analysis accounting for macroscopic and microscopic structures in clays". In Proceeding of 4th International Conference on Soft Soil Engineering. Vancouver, Canada: 3-34.

Lupien, C., Lefebvre, G., Rosenberg, P., Pare, J.J., and Lavalle, J.G. (1983). "The use of fabrics for improving the placement of till on peat foundation". Paper presented in the 62nd Annual Meeting of Transportation Research Board, Washington.

Mesri, G., And Ajlouni, A. (2007). "Engineering properties of fabrous peats". Journal of Geotechnical and Geoenvironmental Engineering, 133(7): 850-866.

Quigley, R.M. (1980). "Geology, mineralogy and geochemistry of Canadian soft soils: a geotechnical perspective". Canadian Geotechnical Journal, 17(2): 261-285.

Rowe, R.K. (1984). Recommendations for the Use of Geotextile Reinforcement in the Design of Low Embankments on Very Soft/Weak Soils. Research Report GEOT-1-84, Faculty of Engineering Science, The University of Western Ontario.

Rowe, R.K., and Hinchberger, S.D. (1998). "The significance of rate effects in modeling the Sackville test embankment". Canadian Geotechnical Journal, 35(3): 500-516.

Rowe, R.K., and Li, A.L. (1999). "Reinforced embankments over soft foundations under undrained and partially drained conditions". Geotextiles and Geomembranes, 17(3): 129-146.

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

Rowe, R.K., and Mylleville, B.L.J. (1996). "A geogrid reinforced embankment on peat over organic silt: a case history". Canadian Geotechnical Journal, 33(1): 106-122.

Rowe, R.K., and Soderman, K.L. (1984). "Comparison of predicted and observed behavior of two test embankments". Geotextiles and Geomembranes, 1(2): 143-160.

Rowe, R.K., and Soderman, K.L. (1985a). "Geotextile reinforcement of embankments on peat". Geotextiles and Geomembranes, 2(4): 277-298.

Rowe, R.K., and Soderman, K.L. (1985b). "An approximate method for estimating the stability of geotextile reinforced embankments". Canadian Geotechnical Journal, 22(3): 392-398.

Rowe, R.K., and Soderman, K.L. (1986). "Reinforced embankments on very poor foundations". Geotextiles and Geomembranes, 4(1): 65-81.

Rowe, R.K. and Soderman, K.L. (1987). "Very soft soil stabilization using high strength geotextiles: The role of finite element analysis". Geotextiles and Geomembranes, 6(1):53-81.

Rowe, R.K., and Taechakumthorn, C. (2008). "Combined effect of PVDs and reinforcement on embankments over rate-sensitive soils". Geotextiles and Geomembranes, 26(3): 239-249.

Rowe, R.K., and Taechakumthorn, C. (2011). "Design of reinforced embankments on soft clay deposits considering the viscosity of both foundation and reinforcement". Geotextiles and Geomembranes, 29(5): 448-461.

Rowe, R.K., MacLean, M.D., and Soderman, K.L. (1984). "Analysis of a geotextile reinforced embankment constructed on peat". Canadian Geotechnical Journal, 21(3): 563-576.

Saowapakpiboon, J., Bergado, D.T., Youwai, S., Chai, J.C., Wanthong, P., and Voottipruex, P. (2010). "Measured and predicted performance of prefabricated vertical drains (PVDs) with and without vacuum preloading". Geotextiles and Geomembranes, 28(1): 1-11.

Study Centre for Road Construction (SCW), (1981) Stability of Slopes Constructed with Polyester Reinforcing Fabric, Arnhem, The Netherlands.

Shinoda M., and Bathurst, R.J. (2004). "Lateral and axial deformation of PP, HDPE and PET geogrids under tensile load". Geotextiles and Geomembranes, 22(4): 205-222.

Taechakumthorn, C., and Rowe, R.K. (2012a). "Choice of allowable long-term strains for reinforced embankments on a rate-sensitive foundation". Geosynthetics International, 19(1): 1-10.

Taechakumthorn, C., and Rowe, R.K. (2012b). Performance of reinforced embankments on rate-sensitive soils under working conditions considering effect of reinforcement viscosity. ASCE International Journal of Geomechanics, 12(4): 381-390.

Taechakumthorn, C., and Rowe, R.K. (2012c). "Performance of a reinforced embankment on a sensitive Champlain clay deposit". Canadian Geotechnical Journal, 49(8): 917-927.

Torrance, J.K. (1999). "Physical, chemical and mineralogical influences on the rheology of remolded low-activity sensitive marine clay". Applied Clay Science, 14(4): 199-223.

Vaid, Y.P., and Campanella, G. (1977). "Time-dependent behaviour of undisturbed clay". Journal of the Geotechnical Engineering, 103(GT7): 693-709.

Vaid, Y.P., Robertson, P.K., and Campanella, R.G. (1979). "Strain rate behaviour of Saint-Jean-Vianney clay". Canadian Geotechnical Journal, 16(1): 34-42.

Yeo, S.S., and Hsuan, Y.G. (2010). "Evaluation of creep behavior of high density polyethylene and polyethylene-terephthalate geogrids". Geotextiles and Geomembranes, 28(6):409-421.