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Geosynthetic-reinforced embankments over soft foundations

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ABSTRACT: The behaviour of reinforced embankments over conventional soft cohesive soil, rate sensitive soil and peat deposits is reviewed and recent design and analyses methods are summarized. The findings from both field observations and finite element analyses are presented. Both undrained and partially drained behaviour of reinforced embankments are considered. The use of reinforcement in combination with prefabricated vertical drains is addressed. The effects of both the viscous and inviscous characteristics of reinforcement and foundation soils on embankment behaviour are discussed. It is concluded that the partial consolidation provided by PVDs and the tension mobilized in reinforcement can substantially increase embankment stability. However, creep of geosynthetics can decrease the embankment failure height. The mobilization of reinforcement during and after embankment construction can vary significantly depending on the soil and reinforcement characteristics. Particular care must be taken for designs where the foundation soil is rate sensitive.

INTRODUCTION

The behaviour and design of geosynthetic reinforced embankments over soft soil has attracted considerable attention in both practice and in the literature. The behaviour of basal reinforced embankments over typical soft soils is now well understood and a number of papers have addressed these issues (e.g. Humphrey and Holtz 1987; Jewell 1988; Rowe 1997; Leroueil and Rowe 2001; Rowe and Li 2001) and books such as Jewell (1996) have summarized some common design methodologies. However it has also been found that these design methods may be overly conservative for conventional soils and may be unconservative for less conventional soils. The objectives of this paper are six fold. First, to summarize typical methods of analysis and design for reinforced embankments. Second, to discuss the performance of reinforced embankments as observed in field cases. Third, to discuss the issues surrounding the selection of "compatible strain" as a criterion for the design of reinforced embankments. Fourth, to examine the effect of partial drainage and the use of prefabricated vertical drains (PVDs) on the design and performance of reinforced embankments on soft clay and to present a new design methodology. Fifth, to summarize design considerations and methods for embankments on fibrous peat. Finally, to provide new insights regarding the performance and design of reinforced embankments on rate sensitive soils and to examine the potential effect of creep in the reinforcement itself.

METHODS OF ANALYSIS

Jewell (1988) provided a rational understanding of the role of the reinforcement in embankments on soft soil. Key to these considerations was the recognition that the lateral earth pressure within an embankment over a soft cohesive foundation imposes shear stresses on the foundation soil which reduces the bearing capacity of the foundation and hence embankment stability. The basal reinforcement can serve to resist some or all of the earth pressure within the embankment and to resist the lateral deformations of the foundation thereby increasing bearing capacity and stability. A number of idealized failure mechanisms can be identified for reinforced embankments: lateral sliding of embankments over the base reinforcement layer, foundation extrusion (bearing capacity failure), rotational slope failure involving breakage or pull-out of reinforcement, and excessive displacement.

If the interface shear strength between the reinforcement (e.g. geosynthetic) and fill is inadequate, active earth pressure within the embankment may cause the embankment to slide laterally on top of the reinforcement although, in practice, this is rarely a critical case. Alternatively, with shallow deposits of low strength soil, the foundation material can be laterally extruded from beneath the reinforced embankment. If the reinforcement is placed directly on the foundation material then this mechanism may involve horizontal movement of the foundation soil relative to the reinforcement and the overlying embankment. The key parameters controlling these two mechanisms are the shear strength of the foundation soil and the reinforcement-soil interface strength in direct shear.

The factor of safety against a rotational slip failure may be increased by the inclusion of geosynthetic reinforcement. The tensile force required to maintain stability must be developed in the reinforcement by means of shear stresses between the reinforcement and the soil above and below it. Once the interface shear strength is reached, the reinforcement will pull-out of the soil and rotational failure will occur. Alternatively, if the tensile strength of the reinforcement is reached, the breakage of the reinforcement will result in a rotational failure. These two possibilities are fairly obvious. There is, however, a third and somewhat less obvious potential failure mechanism. The embankment may fail at a reinforcing force lower than that expected based on stability considerations due to the stress-strain-time characteristics of the reinforcement. If the reinforcement has a low mobilized tensile stiffness, J, then large deformations of the foundation may occur prior to reinforcement failure. Under these circumstances, it may not be possible to construct the embankment to the desired height even though "collapse" has not occurred. Furthermore, for some soils, significant movement along the potential failure surface may result in strain softening of the soil. Additional load will then be transferred to the reinforcement leading to even larger strains until eventually the reinforcement will break accompanied by

failure of the embankment. In order to prevent this failure mechanism, consideration must be given to: (a) the reinforcement-soil interface shear strength under conditions where the reinforcement is pulled out from between the soil above and below it; (b) the tensile strength of the reinforcement; and (c) the stress-strain characteristics of the reinforcement relative to those of the foundation soil.

Embankments on highly compressible foundations may fail due to excessive displacements. For a particular geometry and soil profile, there is a threshold reinforcement tensile stiffness below which the reinforcement has no effect upon settlement. For reinforcement with tensile stiffness (modulus) in excess of this threshold value, the reinforcement will reduce lateral spreading and local yield. This effect will be greatest for shallow deposits or for deposits where the soil strength and modulus increase with depth. However, the reinforcement cannot eliminate settlement. For each geometry there is also an upper threshold tensile stiffness above which any further increase in reinforcement stiffness does not alter the settlement. Thus, under some circumstances, excessive deformations may occur even if a high tensile stiffness reinforcement is used. This possibility must be recognized at the design stage and, if necessary, consideration should be given to the use of a lightweight fill material (e.g. see Rowe and Soderman 1985b, 1986).

Many embankments are constructed on deposits with a relatively stiff crust or root mat overlying a weaker and more compressible main deposit. This crust/root mat is a natural reinforcement and will contribute significantly to embankment stability while reducing settlements. However, in doing so, tensions may develop within the crust/root mat. If the limited tensile strength of the crust is exceeded, tensile failure will occur followed by the embankment sinking into the soft underlying soil. In these cases, the major role of the reinforcement is to limit the tension developed within the root mat, and this can only be achieved provided the reinforcement is relatively stiff compared to the root mat/crust and provided that the tensile strength of the reinforcement is not reached.

Thus, in addition to the shear strength of the foundation soils, the important properties for the design of reinforced embankments are the soil-reinforcement interface shear strength under direct shear and pullout conditions, the stress-strain characteristics of the reinforcement, and the tensile strength of the reinforcement. The objective of this section is to review existing methods of analyzing the behaviour of reinforced embankments over soft clayey soil.

Bearing capacity

The first step in the design of an embankment over soft soil is to evaluate whether or not reinforcement or other soil improvement (e.g. PVDs) is required to achieve the design embankment height with the desired factor of safety against collapse under undrained conditions. If the desired factor of safety can be achieved without reinforcement or other soil improvement technique then the analysis stops. However, if the desired height cannot be safely achieved with an unreinforced design, then one should check whether it is possible to attain the desired height using reinforcement. This can be done by means of a bearing capacity calculation, however, as discussed by Humphrey and Holtz (1987), traditional bearing capacity theory assumes a constant strength with depth and can lead to overconservative estimates of collapse load where the soft foundation has a strength profile that increases with depth and/or is of limited depth.

Plasticity solutions published by Davis and Booker (1973) consider the effect of strength increase with depth on the bearing capacity of rigid footings. To allow convenient design of reinforced embankments, Rowe and Soderman (1987a) synthesized the bearing capacity factors of Davis and Booker (1973) and Matar and Salencon (1977) for rough footings (Fig. 1) and proposed a simple method of estimating the stability of a highly reinforced embankment. This approach, which will be briefly outlined here, considers the effect of increasing undrained shear strength with depth as well as the effect of the relative thickness of the underlying cohesive soil deposit. A reinforced embankment can never be reinforced beyond the point of being rigid, hence these solutions place an upper limit on the improvement in stability that can be achieved using high strength/tensile stiffness reinforcement.

Since an embankment will generally be trapezoidal in shape and the plasticity solutions are for a rigid footing of width b, an approximation must be made to obtain the equivalent width of the embankment. From plasticity considerations, the pressure at the edge of a rigid footing is $(2+\pi)s_{uo}$, where s_{uo} is the undrained shear strength directly beneath the footing. It is assumed here that the effective width of the footing b will extend between the points on either side of the embankment when the applied pressure γh^* is equal to $(2+\pi)s_{uo}$. Thus

$$h^* = (2+\pi)s_{uo}/\gamma \qquad (1)$$

and hence (from Fig. 2)

$$b = B + 2n(H-h^*) \qquad (2)$$

where s_{uo} = the undrained shear strength at the top of the deposit, γ = weight of the fill, B = the crest width, H = the embankment height, and n = the cotangent of the slope angle (see Fig. 2).

The bearing capacity q_u of the equivalent rigid footing of width b is given by

$$q_u = N_c\, s_{uo} + q_s \qquad (3)$$

where q_s is a uniform surcharge pressure applied to the foundation soil surface outside of the footing width. The bearing capacity factor N_c is obtained from Figure 1. Inspection of Figure 2 shows that the triangular edge of the embankment is providing a surcharge which would increase stability and hence an estimate of q_s in terms of the pressure applied by this triangular distribution is required. Figure 3 shows the depth, d, to which the failure mechanism is expected to extend. The lateral extent of the plastic region involved in the collapse of a rigid footing extends a distance x from the footing where x is approximately equal to the minimum of d, as obtained from Figure 3, and the actual thickness of the deposit D, i.e.

$$x = \min(d,D) \qquad (4)$$

Thus distributing the applied pressure due to the triangular distribution over a distance x gives

$$q_s = 0.5n\,\gamma(h^*)^2/x \qquad \text{for } x > nh^* \qquad (5)$$

and

$$q_s = 0.5\gamma\,(2nh^*-x)\,/n \qquad \text{for } x < nh^* \qquad (6)$$

The value of q_u deduced from Equ 3, 5 and 6 may then be compared with the average applied pressure q_a due to the embankment over the width b, viz.

$$q_a = \gamma[BH + n(H^2-(h^*)^2)]/b \qquad (7)$$

The maximum possible factor of safety (defined here as $FS=q_u/q_a$) that could be achieved for a given embankment geometry and soil profile under undrained conditions, can be calculated directly from Eqs.1-7. If the desired factor of safety

for the proposed embankment is less than FS calculated in this manner, then reinforcement can potentially allow construction of the embankment to the desired height. If not, then either a lower factor of safety would have to be accepted (together with the use of reinforcement) or alternative measures (e.g. PVDs, stage loading, berms, or the use of lightweight fill) would be needed in addition to the use of reinforcement.

The maximum possible factor of safety (FS) calculated above implicitly assumes that the reinforcement has sufficient strength and stiffness to develop the required reinforcement forces without breaking or excessive deformation. Having established that it is possible to build the embankment to the desired factor of safety, the next step (to be discussed in the following section) is to select reinforcement which has sufficient strength and stiffness to achieve the desired factor of safety.

Limit equilibrium methods

Limit equilibrium methods have been used extensively to assess the short-term (undrained) stability of reinforced embankments constructed on soft foundation soils (Haliburton 1981; Jewell 1982; Ingold 1982; Milligan and LaRochelle 1984; Rowe and Soderman 1985a; Mylleville and Rowe 1988; Low et al. 1990; Holtz et al. 1997; Koerner 1997; Li and Rowe 2001a; and others). These methods have been used to examine the equilibrium of the following mechanisms: 1) bearing capacity failure of the foundation which involves the entire embankment; 2) lateral sliding of a block along the embankment fill-reinforcement interface, foundation-reinforcement interface or along a weak layer in the foundation soil; and 3) a slip circle-type failure mechanism passing through the embankment fill and foundation soil. The various methods are similar in that limiting equilibrium is established for the system of external forces acting on an assumed failure mass.

It is generally agreed that the reinforcement can be represented by an external restoring force acting on the failure mass. Historically, there has been debate regarding the inclination (orientation) and magnitude of this force. However there is now strong evidence that the reinforcement force should be taken to act in its original horizontal orientation.

Mylleville and Rowe (1988) proposed a limit equilibrium method which is a modified version of a method by Jewell (1982). In this method, the failure surface in the foundation is approximated by a circular arc and the embankment is modelled by a means of an equivalent surcharge pressure on the foundation and a horizontal thrust (due to earth pressure within the embankment) as illustrated in Figure 4.

The restoring force due to the reinforcement is assumed to act along the line of its original (horizontal) orientation, at the intersection of the failure surface and the reinforcement. In the following, it is assumed that the reinforcement is located within the fill material and not directly on top of the foundation. The procedure can be modified to consider the case where the reinforcement is directly on the foundation soil.

A commonly used limit equilibrium method considers moment equilibrium about the circle centre under consideration. The overturning moments are made up of two components, one being that due to the embankment fill weight contained within the slip circle and the other due to a thrust force within the embankment fill itself. The thrust force tends to push the embankment fill outwards. The restoring moments are derived from the reinforcement and shear strength of the clay foundation along the assumed failure surface. A closed form expression is used to compute the resisting moment due to the clay foundation and allows one to consider either a homogeneous deposit or a deposit where the shear strength varies with depth. Figure 4 shows the general arrangement of the limit equilibrium problem (taking the embankment toe as the origin of the co-ordinate system).

$$\frac{\text{Restoring Moments}}{\text{Overturning Moments}} \quad (8a)$$

$$\text{i.e. ERAT} = \frac{\text{MRR} + \text{MRSOIL}}{\text{MOFILL} + \text{MOPT}} \quad (8b)$$

where: for limit equilibrium we require ERAT = 1;
MRR = Z_R T is the restoring moment due to limiting force developed in the reinforcement; and
T = min (T_1, T_2, T_3, T_4) and T_1- T_4 are as defined below:

1) T_1 = sum of thrust force in fill and clay-fill interface shear:

$$T_1 = 1/2\ K_A\gamma h^2 + \alpha s_{uo}(x_c + R\sin(\theta/2)) \quad (9)$$

α = clay-fill interface adhesion factor; K_A= coefficient of active earth pressure

2) T_2 = the pullout capacity of the reinforcement:

$$T_2 = 2\int_0^{x_c + R\sin(\theta/2)} \sigma_N dx \quad (10)$$

σ_N = normal stress acting on the reinforcement

3) T_3 = the allowable reinforcement force governed by strength.

4) T_4 = the allowable reinforcement force governed by allowable strain ε_a,

$$T_4 = J\ \varepsilon_a \quad (11)$$

J is the secant tensile stiffness of the reinforcement over the strain range (0 - ε_a)

MRSOIL = restoring moment due to the mobilized shear strength along the circular failure surface in the clay foundation

$$= \int_{-\theta/2}^{\theta/2} s_u(z) R^2 d\delta \quad (12a)$$

$$s_u(z) = s_{uo} + (R\cos\delta - z_c)\rho_c \quad (12b)$$

$$\therefore \text{MRSOIL} = s_{uo}R^2\theta - \rho_c Z_c R^2\theta + 2R^3\rho_c\sin(\theta/2) \quad (12c)$$

MOFIILL = sum of overturning moments due to embankment fill self-weight applied to the clay foundation. (The embankment fill is subdivided into a number of regions to simplify computations).

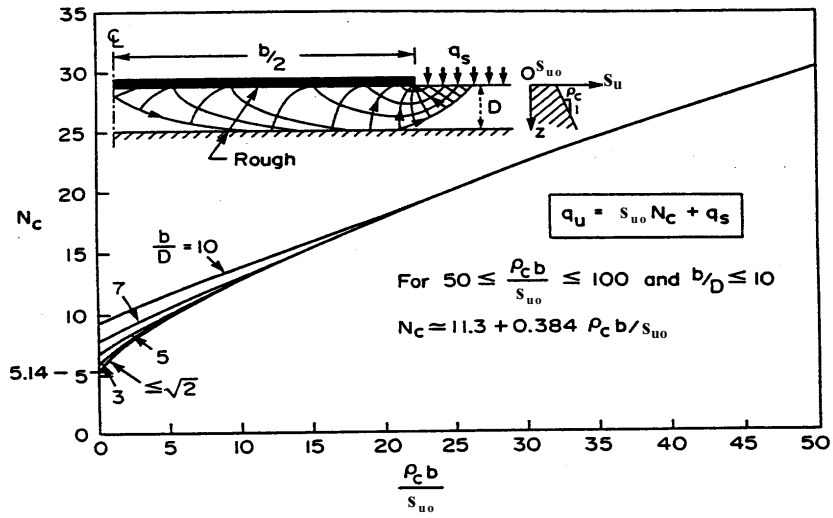


Figure 1. Bearing capacity factor for nonhomogenous soil (modified from Rowe and Soderman 1987a).

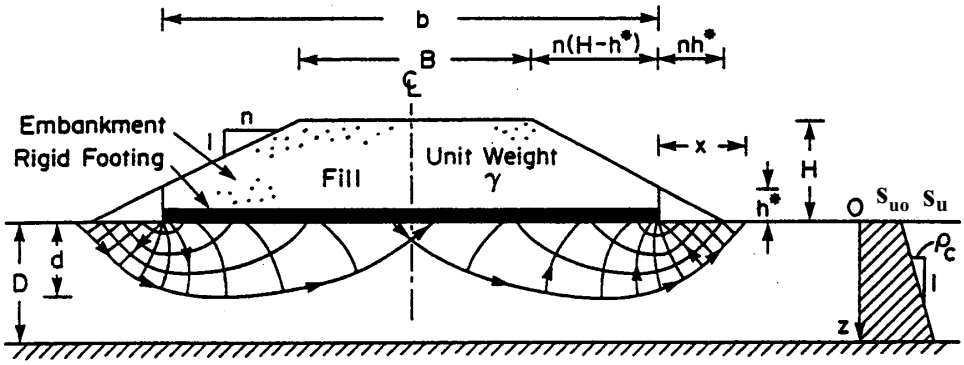


Figure 2. Definition of variables used to estimate collapse height for a perfectly reinforced embankment (modified from Rowe and Soderman 1987a).

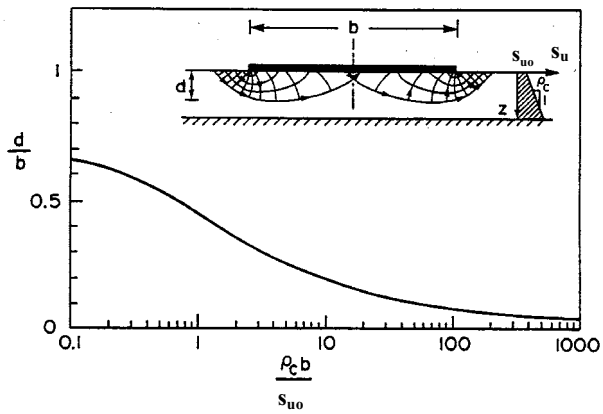


Figure 3. Effect of nonhomogeneity on depth of the failure zone beneath a rough rigid footing (modified from Matar and Salencon 1977).

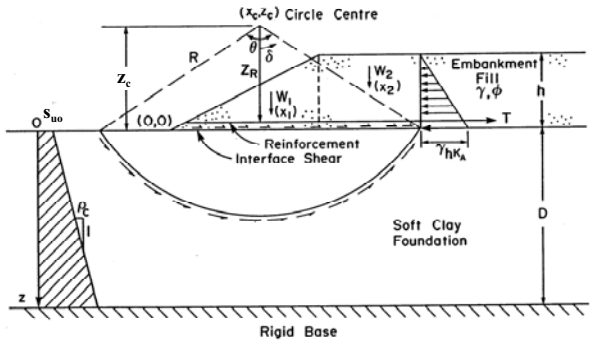


Figure 4. General arrangement of the limit equilibrium method by Mylleville and Rowe (1988).

$\therefore$ MOFILL = $W_1 (x_1 - x_c) + W_2(x_2 - x_c) + \ldots$

$$= \sum_{i=1}^{nr} W_i (x_i - x_c) \quad (13)$$

where: W_i = weight due to embankment fill of region i;
x_i = centroid x coordinate of region i;
$(x_c\ z_c)$ = are the coordinates of circle centre; and
nr = number of regions

MOPT = overturning moment due to horizontal thrust pressure (force) in the embankment fill

$$\therefore \text{MOPT} = 1/2\ K_A \gamma h^2 (z_c - h/3) \quad (14)$$

The limit equilibrium method described above can be easily implemented in the form of a computer program, which can search for a slip circle giving the lowest Equilibrium Ratio, ERAT, for a given embankment. The maximum height to which an embankment can be constructed may be calculated by an iterative approach. If the soil strength is used and the lowest ERAT value is equal to unity, the corresponding height is the collapse height. Partial factors may be applied to the interface strength, pullout capacity and reinforcement strength as appropriate. After the appropriate partial factors have been applied, one seeks reinforcement such that ERAT $\geq$ 1.

Finite element methods

Methods of analysis such as limit equilibrium and plasticity solutions (previously described) provide no information about deformations or strains which develop in the reinforcement for a given reinforced embankment. Reinforced embankments are a composite system consisting of three components: namely, the foundation soil, the reinforcement, and the embankment fill. Their performance is highly dependent on deformations and the interaction between these components.

The cost of constructing and monitoring full scale field test embankments in order to assess the performance of various reinforcing schemes is sufficiently large that it is generally impractical. An alternative is to perform numerical simulations of "embankment construction" using appropriate numerical models (Rowe and Soderman 1987b; Chai and Bergado 1993a; Hird et al. 1996; Li and Rowe 2001a) verified against what limited field cases are available.

The finite element method has proven to be powerful technique for use in the evaluation of slope and embankment behaviour since its first use for this application by Clough and Woodward (1967). Numerous researchers have employed finite element techniques to interpret the field behaviour of reinforced embankments and to conduct sensitivity studies (e.g. Andrawes et al. 1980; Rowe 1982, 1984; Boutrop and Holtz 1983; Monnet et al. 1986; Duncan et al. 1987; Schaefer and Duncan 1988; Humphrey and Holtz 1989; Hird and Kwok 1990; Mylleville and Rowe 1991; Chai and Bergado 1993b; Litwinowicz et al. 1994; Rowe et al. 1996; Varadarajan et al. 1997; Rowe and Hinchberger 1998). The finite element codes used by these investigators have incorporated soil models with different levels of sophistication including the hyperbolic elastic model (e.g. Andrawes et al. 1980), Mohr-Coulomb model (e.g. Rowe and Soderman 1985), Cam-Clay model (e.g. Schaefer and Duncan 1988); Modified Cam-Clay model (e.g. Chai and Bergado 1993b; Rowe et al. 1996), the viscoplastic Elliptical Cap model (Rowe and Hinchberger 1998). A linear, bilinear or nonlinear bar model is commonly used to model reinforcement sheet (e.g. Rowe and Soderman 1985; Hird and Kwok 1989; Bathurst et al. 1992). With newly developed fast scalar, parallel and vector computers, it has become feasible to use finite element models that are sufficiently sophisticated to model the essential characteristics of reinforced embankments on soft foundations, and to investigate time dependent behaviour of reinforced embankments under a wide range of conditions (Li 2000).

The numerical models and techniques used should provide adequate predictive accuracy to capture the essential behavioral characteristics of the soil, the reinforcement, and the soil-reinforcement interactions. The actual construction process can be simulated by turning on the gravity load of each layer (or lift) of embankment elements at a rate corresponding to the construction rate.

As noted earlier, the constitutive models for the soil used in analysing reinforced embankments may be subdivided into four categories: namely, nonlinear elastic (e.g hyperbolic), elasto-perfectly-plastic, strain hardening (e.g. cap model) and elasto-viscoplastic models. Nonlinear elastic models may provide acceptable results at low stress levels but cannot correctly model plastic failure and plastic strains within the soil mass. Thus these models will give poor predictions of the behaviour of the foundation soil prior to embankment failure. Nonlinear elastoplastic models with a Mohr-Coulomb failure surface and non-associated flow rule are suitable to model the behaviour of typical granular soils used for embankment fill and for foundations that do not experience significant strain hardening or viscous behaviour over the time period of interest. However, accurate prediction often requires consideration of the yielding behaviour of cohesive soils under embankment loading and a critical state model (e.g. Modified Cam-Clay model) or Elliptical Cap model may be needed. Although some soil can be adequately modelled using Cam-Clay or Modified Cam-Clay, the Elliptical Cap model has greater flexibility for modeling embankment behaviour since it allows the shape of the yield surface to be adjusted for the particular soil being considered.

Nonlinear elastoplastic models fully coupled with Biot consolidation theory (Biot 1941) have been widely used for consolidation analyses. The variation in the hydraulic conductivity of cohesive soils during loading and consolidation (Tavenas and Leroueil 1980) can be considered by using Taylor's Equation (Taylor 1948) which correlates the hydraulic conductivity to the void ratio of soils.

Viscoplastic soil models are needed to model the time dependent behaviour of reinforced embankments on rate sensitive soft foundation soils (e.g. Rowe and Hinchberger 1998). Any such viscoplastic model should be able to capture the creep-induced excess pore pressure and strain rate dependence of undrained shear strength.

If an embankment is reinforced using creep insensitive reinforcement (e.g. some high tensile stiffness PET geogrids), the reinforcement can be modelled using a one-dimensional bar element with a linear or nonlinear elastic constitutive relation. For creep susceptible geosynthetic reinforcement (e.g. polyolefin geogrids), a nonlinear viscoelastic or viscoplastic model should be used to describe the creep and stress-relaxation behaviour of geosynthetic materials (e.g. Li and Rowe 2001b).

The interaction between the soil mass and reinforcement can be modelled by using interface elements which can have linear elastic deformations prior to slip and allow slippage between soil and reinforcement after the interface strength is reached. In general, a Mohr-Coulomb failure criterion is adequate to predict the failure of a soil-reinforcement interface although cases have been reported where non-linear (hyperbolic) models were considered necessary (Bathurst et al. 1992). Based on the authors' experience, a Mohr-Coulomb model has proven adequate for embankments with granular fills.

Reinforced embankments on foundation soils in which PVDs have been installed require special consideration. Strictly speaking, the analysis of a system involving discrete vertical drains should be conducted with a fully 3-D analysis, whereas, most embankments are modelled for plane strain conditions. To

avoid the need for a full 3-D analysis, some approximation is required to allow vertical drains to be reasonably modelled in a plane strain analysis. Both Li (2000) and Sharma and Bolton (2001) have shown that the techniques proposed by Hird et al. (1992, 1995) are suitable for matching a plane strain vertical drain system with an axisymmetric vertical drain system.

CASE HISTORIES

Numerous case studies have shown that the use of geosynthetics typically reduces the cost of construction, increases the feasibility of construction, and increases the stability of embankments on poor foundation soils. However, it has also been reported that the field behaviour of reinforced embankments is significantly different from that expected based on the simplifications made in current design methods (e.g. Bassett and Yeo 1988; Duarte and Satterlee 1989; Fritzinger 1990; Litwinowicz et al. 1994). This section highlights the key findings based on the field behaviour reported in the literature and represents a summary of a more detailed discussion by Rowe and Li (2001). The key issues are summarized in Table 1.

Benefits due to the use of reinforcement

The benefits of reinforcement in increasing embankment stability were clearly demonstrated by the Almere test embankment where unreinforced and reinforced (tensile stiffness, J = 2000 kN/m) granular fill sections were constructed on a 3.3m thick organic clay foundation with an undrained shear strength of 8 kPa. The reinforced embankment experienced a relatively ductile failure at a height of 2.75m which is in remarkable contrast to the rapid failure of the unreinforced section at 1.75m thickness. Rowe and Soderman (1984) demonstrated that both limit equilibrium and finite element analysis could capture the effect of the reinforcement on embankment stability. Bergado et al. (1994) and Loke et al. (1994) reported the field behaviour of two reinforced embankments constructed over soft Bangkok clay using two types of geosynthetic reinforcement. It has been shown that the tensile force mobilized in reinforcement increased the embankment stability after the foundation soil became plastic and reduced the lateral deformation of the foundation soil. The stiffer the reinforcement, the higher the embankment could be constructed.

It is evident from the field cases cited in Table 1 that the use of basal geosynthetic reinforcement can increase embankment stability and reduce deformations. The benefits arising from the use of geosynthetic reinforcement include the improvement of the embankment behaviour, cost savings, an increase in the feasibility of embankment construction, and the elimination of stage construction in some cases.

Field reinforcement force and strain values

Current design methods for reinforced embankments are usually based on limit equilibrium analyses (Jewell 1982; Fowler and Koerner 1987; Leshchinsky 1987; Holtz et al. 1997; and others). For embankments on soft cohesive deposits, the foundation soils are commonly assumed to respond in an undrained manner during embankment construction and the critical time with respect to stability is typically considered to be at the end of construction.

It has been reported that the field behaviour of reinforced embankments can be significantly different from that anticipated in the design (Fowler and Edris Jr 1987; Hadj-Hamou and Bakeer 1991; Hashizume et al. 2000). The mobilized reinforcement force (or strain) is often only a small fraction of that predicted in design (Table 1). This suggests that either the current design methods may be too conservative or it raises the question as to why the behaviour was not consistent with design expectations. These issues will be addressed in later sections of the paper.

Use of reinforcement and prefabricated vertical drains

The synergistic effects of the use of geosynthetic reinforcement and prefabricated vertical drains can improve the performance of embankments over soft foundations (Li and Rowe 1999b, 2001a). Lau and Cowland (2000) reported a case where neither reinforcement nor PVDs alone would have been sufficient to allow safe embankment construction to the design height. The combined use of both reinforcement and PVDs increased the short-term stability and made it feasible to construct this 4 m-high embankment. The use of PVDs and control of the construction rate effectively reduced the excess pore pressure during construction and accelerated the dissipation of pore pressure after construction. The rate of strength gain of the soft clays due to partial consolidation arising from the presence of the prefabricated vertical drains was rapid and significant.

It has been also reported (Schimelfenyg et al. 1990) that the use of PVDs decreased the foundation heave and horizontal shear deformations during construction. In this case, the construction of a relatively high embankment over a foundation soil having extremely low undrained shear strength was achieved by the combined use of reinforcement and PVDs. These cases illustrate the importance of strength gain in the foundation due to partial consolidation during embankment construction where PVDs are used and highlight the benefit that can arise from considering partial consolidation in design (as discussed in Section 6).

Factors contributing to low reinforcement strain

It is often reported that the mobilized reinforcement strain and force are significantly lower than expected in design. Varuso et al. (1999) showed that a reinforced levee designed with a factor of safety of one was stable and performed well. There are two factors contributing to this observation. Firstly, the design was based on undrained shear strength measured using unconsolidated-undrained tests which typically underestimate the actual undrained shear strength (due to disturbance). Secondly, there was a substantial increase in shear strength of the foundation soil due to partial consolidation during construction, which increased the levee stability relative to that assumed in design.

Partial consolidation, occurring during construction over foundations improved by the installation of vertical drains, can result in mobilized reinforcement strains well below the design value (Fritzinger 1990). Chai and Bergado (1993b) reported the case of an 8.5 m-high reinforced embankment on a soft Malaysian Muar clay where the reinforcement was not fully mobilized due to the strength gain of the foundation resulting from the installation of PVDs.

The cases examined herein have shown that the observed reinforcement strain and force are usually less than the design values for a required factor of safety or the values are less than predicted for equilibrium assuming the undrained strength of the soil has been fully mobilized. This can be attributed to three primary factors. Firstly, current design methods conservatively assume undrained conditions for the foundation soils during embankment construction. However, in reality, significant partial consolidation can occur when the soil is overconsolidated during early stages of loading (Leroueil et al. 1978; Li and Rowe 1999a; Leroueil and Rowe 2001) and the consequent beneficial effect of the partial consolidation on stability is enhanced by the presence of embankment reinforcement (Li and Rowe 1999a).

Table 1 Summary of case histories of reinforced embankments

	H (m)	Foundation soils	Reinforcement	Performance	References
Benefits from the use of geosynthetic reinforcement	2.75	3.3m thick organic clay with s_u = 8 kPa	1 layer of PET W-GT with J = 2000 kN/m	The use of reinforcement increased the failure height by 57%. Reinforcement reduced the plastic deformations of the foundation soil.	Rowe and Soderman (1984)
	6.6	2.74-5.18m thick soft organic silty clay overlying sand layers s_u=8-11 kPa	6 layers of HDPE uniaxial GG with T_{ult}=80 kN/m	Significant savings; elimination of stage construction; excellent performance due to the combined use of reinforcement and PVDs.	Lockett & Mattox (1987)*
	Emb. A: 4.2 Emb. B: 6	8m thick soft Bangkok clay with s_u=15-30 kPa underlying a 2m crust with s_u=20-40 kPa	Emb. A: 4 layers of NW -GT with T_{ult}=8-18 kN/m Emb. B: 1 layer of W-GT with T_{ult}=200 kN/m	The use of reinforcement increased the embankment failure height; the higher strength and stiffness gave better performance.	Bergado et al. (1994) and Loke et al. (1994)
	5.5	4m thick organic clay with LL=56% and PL=32% overlying 13.5m of high plasticity organic clay	3 layers of uniaxial GG with T_{ult} = 80 kN/m and $J_{5\%}$ = 1080 kN/m	Both geogrid reinforcement and stage construction increased the factor of safety. Measured pore pressures were less than predicted.	Mattox and Fugua (1995)+
Observed behaviour vs. design expectation	3.8	Low plasticity organic silt overlying highly plastic clay with s_u=7-22 kPa	1 layer of W-GT with T_{ult}=664 kN/m and $J_{5\%}$=5950 kN/m	The geotextile strains and loads measured in field were significantly less than the values predicted in design.	Fowler & Edris Jr (1987)
	3.1	Extremely soft to soft thick clay deposits with an average s_u=7 kPa for the first 6m of soil	2 layers of HPDE uniaxial GG with T_{ult} = 80 kN/m and $J_{5\%}$ = 1080 kN/m	The mobilized reinforcement force was only 73% of the design strength.	Hadj-Hamou and Bakeer (1991)
	13.2	15m thick soft ground	1 layer of GG with T_{req} =700 kN/m	Observed reinforcement strain and force were less than the design values.	Hashizume et al. (2000)
Combined use of reinforcement and PVDs	4.0	6-12m thick river mud and alluvial clay deposits with s_u=7.5-16 kPa	1 layer of W-GT with T_{req}=200 kN/m	Consolidation during construction and strength gain were significant.	Lau and Cowland (2000)*
	5.0	1.2-5.2m thick organic clay with s_u=1-12 kPa and w_n=105%	1 layer of PET W-GT with T_{ult}=880 kN/m and $J_{5\%}$=8800 kN/m	The use of PVDs decreased both the foundation shear deformations and reinforcement strains.	Schimelfenyg et al. (1990)*+
Factors contributing to low mobilization of reinforcement strains and forces	3.7	25m organic silty clay, very soft to medium consistency with high water content and s_u=7-30 kPa.	Three sections: A: 1 layer of GT B: 1 layer of GG C: 2 layers of GG. Each section had total reinforcing force of $T_{5\%}$=85 kN/m	Strength gain during and shortly after construction was 50-135%. This resulted in low mobilized reinforcement strain.	Varuso et al. (1999)
	6.0	7.5-30.5m thick soft and highly compressible silts and clays s_u=4.8-9.4 kPa	1 layer of PET W-GT with T_{ult}=260 kN/m and J=3300 kN/m	Consolidation during construction resulted in observed reinforcement strains below design strain of 5%.	Fritzinger (1990)*+
	8.5	2m crust overlying very soft silty clay with w_n=80-105% and s_u=10-30 kPa	2 layers of GG with T_{ult}=110 kN/m	Stage construction and PVDs increased embankment stability and decreased reinforcement strain.	Chai & Bergado (1993)*+

Table 1. Summary of case histories of reinforced embankments (cont'd).

	H (m)	Foundation soils	Reinforcement	Performance	References
Importance of controlled construction rates	1.3-1.7	1.8-1.9m of fibrous peat with w_n=250-700% overlying 2.6-3m very soft organic silt with w_n=250-480% and s_u=4-12 kPa	1 layer of PP biaxial GG with T_{ult}=19 kN/m and $J_{5\%}$=280 kN/m	The control of construction rate was vital to maintain embankment stability.	Rowe & Mylleville (1993, 1996)+
	2.9	6-7m very soft organic marine clay with w_n=90-400% and s_u=1.4-9.6 kPa	2 layers of PET W-GT one with T_{ult}=730 kN/m and J=3500 kN/m; one with and T_{ult}=438 kN/m and J=5940 kN/m	The strength gain of soils due to stage construction permitted construction to the design height without exceeding the design reinforcement strain.	Volk et al. (1994)*+
	4.3	9-15m very soft to soft lacustrine clayey silt with s_u=10-19 kPa	2 layers of PET W-GT with T_{ult}=200 kN/m	The use of high strength GT and stage construction made it feasible to construct the embankment within the project schedule.	Shimel & Gertje (1997)
Reinforced embankment constructed over rate sensitive soils	8.2	Rate sensitive soft organic clayey silt with w_n=40-110%, LL=42-76%, LI >1 and s_{uvane}=22-40 kN/m	1 layer of PET W-GT with T_{ult}=216 kN/m and $J_{5\%}$=1466 kN/m	The soil deformations and excess pore pressures increased after EOC due to creep of the foundation soils. The vane shear strength over estimated the strength of this rate sensitive soil.	Rowe et al. (1995) and Rowe & Hinchberger (1998)
Increase of reinforcement strain after construction	2.8	4-10m very soft/soft organic marine silty clay with w_n=40-120%, LI=1.5-2.5 and s_u=5-12 kPa.	Two sections: A: 1 layer of HPDE GG with $J_{3\%}$=3500 kN/m B: 1 layer of PET-GG with $J_{3\%}$=2100 kN/m	During the postconstruction periods, creep of HPDE reinforcement was significant and creep of PET reinforcement was insignificant.	Litwinowicz et al. (1994)
	7m	Soft clay overlying peat overlying soft clay with total thickness of 4.5m	1 layer of HPDE GG with T_{ult}=79 kN/m and $J_{2\%}$=1094 kN/m	Both reinforcement strain and force increased significantly during 13 months after EOC	Bassett & Yeo (1988)*
Reinforced embankments over peat	Emb. A: 3.9 Emb. B: 5.9	Peat with w_n=445% to 785%	Two sections: A: 1 layer of W-GT with T_{ult}=41 kN/m and J=230-850 kN/m B: 1 layer of W-GT with T_{ult}=178 kN/m and J=150 kN/m	Reinforcement reduced the shear deformations and had no effect on consolidation settlements. Effective stress analyses resulted in good agreement with field data.	Rowe et al. (1984a, b)
	5.8	1.5-3m peat with w_n=150-319% overlying 1-2m clay w_n=44-86% and s_u=40-60 kPa	2 layers of W-GT with T_{ult}=120 kN/m and J=600-1200 kN/m	The use of reinforcement made it feasible to construct this embankment. The mobilized reinforcement force was about one third of the design value	Matichard (1994)
	6.0	1.2-11.3m thick peat with w_n=260-400%	5 layers of GG one layer with T_{ult}=17.7 kN/m and four layers with T_{ult}=108 kN/m	The reinforcement layers made construction possible and resulted in the rigid-footing-like behaviour of the embankment	Oikawa (1996)+
	5.0	2.5m of peat with w_n=92-581% overlying 2.5-5.5m of soft clay with w_n=10-60% and s_u=5-10 kPa	1 layer of HPDE GG with design strength of 61 kN/m	The use of reinforcement eliminated the need for removing peat and reduced the construction time by half	Kerr et al. (2001)

* with PVDs; + with stage construction;
H = embankment fill thickness; W-GT = woven geotextile; NW-GT = nonwoven geotextile; GG = geogrid; PET = polyester; PE = polyethylene; HDPE = high density polyethylene; s_u = undrained shear strength; w_n = natural water content; LL= liquid limit; LI = liquidity index; PL= plastic limit; T_{ult} = ultimate tensile strength; J = tensile stiffness.

The second source of conservatism arises from the selection of undrained shear strength values for the foundation soils. Due to the uncertainty associated with the in-situ operational shear strength of foundation soils, the design strength is often conservatively selected as a lowerbound fit to the data from in-situ and laboratory tests rather than the expected value based on this same data. The third factor contributing to the mobilization of reinforcement strains lower than the design value is that the embankment is usually designed with a required global factor of safety greater than unity in conventional working stress design or using factored foundation strength in limit states design. Thus it is to be expected that under working conditions, the mobilized reinforcement strain and force should be lower than the design values unless the shear strength of the foundation has been overestimated in the design.

Finite element analyses (Rowe and Li 1999; Li and Rowe 2001b) have demonstrated that the magnitude of reinforcement strain at working conditions typically ranges between 1% and 3%, which is consistent with many field observations and is substantially lower than the typical design strain of 5%.

The effect of construction rate and stage construction

Due to the low undrained shear strength of very soft soils, it is often essential to allow the dissipation of excess pore pressure during embankment construction. For example, control of the rate of construction is important in maintaining the short-term stability of embankments over peat. Rowe and Soderman (1985b) recommended that the construction rate should be slow enough to ensure that the pore pressure parameter, B_{max}, remain below 0.34 (where $B_{max} = \Delta u/\Delta\sigma_v$, and Δu is the maximum excess pore pressure generated by a change in vertical total stress $\Delta\sigma_v$). In the Hubrey Road embankment case, this recommendation was not followed at one section and a failure occurred when the excess pore pressure parameter B_{max} was 0.7 (Rowe and Mylleville 1996).

Stage construction is often used to achieve sufficient strength gain of the foundation soil to allow final construction of embankments to the design height. The combined use of embankment reinforcement and stage construction can be very efficient since the beneficial effect of consolidation is enhanced by the use of reinforcement (Rowe and Li 1999; Li and Rowe 2001a, see Section 5). Field examples include the use of two layers of reinforcement combined with stage construction to allow the construction of a 2.9 m-high embankment on a very soft organic soil as reported by Volk et al. (1994). Shimel and Gertje (1997) also reported that the use of high strength reinforcement in combination with stage construction allowed timely construction of a 4.3m-high embankment over a soft foundation within the project schedule. The rate of embankment construction was controlled to maintain stability by monitoring pore pressures and horizontal displacements during fill placement.

The field cases cited herein show that the control of the construction rate to allow the dissipation of excess pore pressures during embankment construction can be a useful approach for ensuring embankment stability over soft foundations and that partial consolidation should be considered in design, especially when both reinforcement and PVDs are used.

Reinforced embankments on rate sensitive soils

A fully instrumented embankment with both reinforced and unreinforced sections was constructed over a soft compressible clayey silt deposit in Sackville, New Brunswick (Rowe et al. 1995, 2001, Rowe and Hinchberger 1998). The natural water content ranged from 40 % to 70%, the liquid limit from 42% to 76% and the plastic limit from 15% to 23%. The liquidity index exceeded unity at depths from 1 to 6 m.

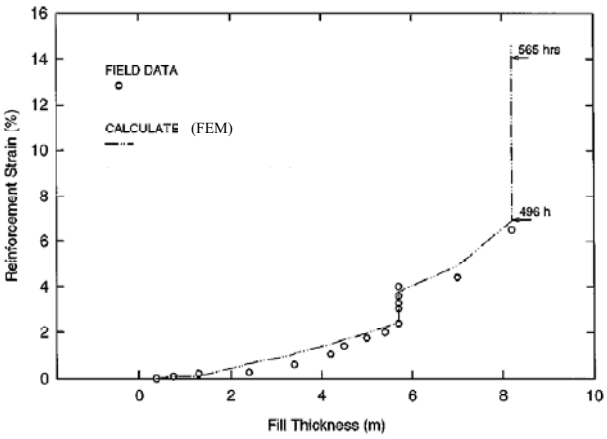


Figure 5. Calculated and measured reinforcement strains versus fill thickness at a point 8.8m from embankment toe (after Rowe and Hinchberger 1998).

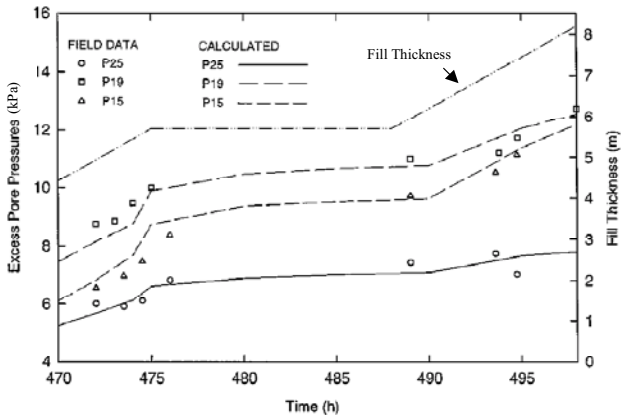
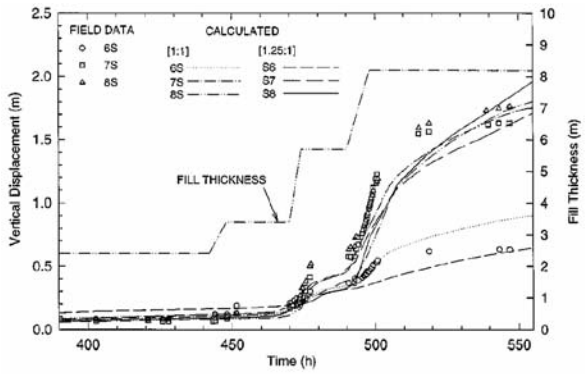


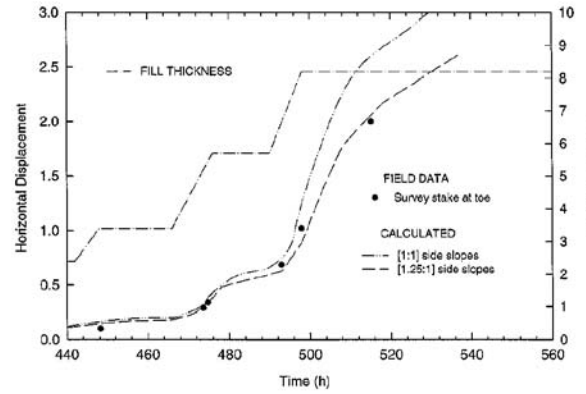
Figure 6. Calculated and measured excess pore pressures at piezometers 15, 19 and 25 for fill thicknesses between 4 and 8.2m (after Rowe and Hinchberger 1998).

The field monitoring indicated that the embankment behaved elastically up to a fill thickness of about 2.4m and that there were significant plastic deformations in the soil during the construction of the embankment from 5m to 5.7m. The unreinforced embankment failure height was about 6.1m (Rowe et al. 2001) and the reinforced embankment failure height was 8.2m. Rowe et al. (1995) described the failure of the Sackville test embankment as a viscous type of failure. For fill thicknesses greater than 2.4 m, the reinforcement strain and embankment deformations increased significantly with time after each embankment lift was placed. The reported behaviour (Rowe et al. 1995) has provided field data that can be used to validate a viscoplastic soil model for rate sensitive foundation soils under embankment loadings (Rowe and Hinchberger 1998). The calculated reinforcement strains were shown to agree with the measured strains during embankment construction and the numerical model closely predicted the time-dependent increase in reinforcement strain (Fig. 5).

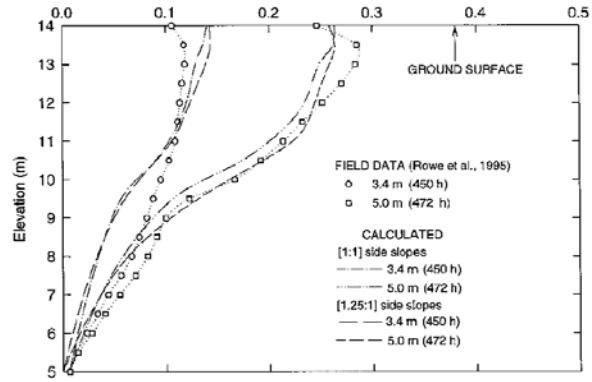
The viscoplastic model successfully predicted the creep-induced excess pore pressure during the stoppage at a fill thickness of 5.7m (Fig. 6). The deformations of Sackville soils were influenced by the viscoplastic characteristics of the rate sensitive soil and time-dependent deformations were also generally well predicted by the model (Fig. 7).



(a) Vertical displacement (m) with time (hours) at settlement plates.



(b) Horizontal displacements (m) at the embankment toe



(c) Relative horizontal displacement profile (m) at inclinometer 23I

Figure 7. Comparison of calculated and measured vertical and horizontal deformations (modified from Rowe and Hinchberger 1998).

The Sackville reinforced embankment case (Rowe et al. 1995) has also shown that the field vane shear strength may overestimate the operational strength rate of sensitive soils even after application of the Bjerrum correction. Subsequently it has been shown that the undrained shear strength correction factor proposed by Li and Rowe (2002) can be used to estimate the operational strength of rate sensitive soils under embankment loadings (Section 8).

The increase of reinforcement strain after construction

Litwinowicz et al. (1994) reported on a 2.8m-high embankment constructed on 4 - 10m deep very soft to soft marine clay. One section was reinforced with an HDPE geogrid and the other was reinforced using a polyester geogrid. The maximum strain in the HDPE geogrid increased by 100% after the end of construction while the strain in the PET geogrid increased slightly. The increase of HPDE geogrid strain at constant embankment load was mainly attributed to creep of the geogrid. However the magnitude of increase (i.e. 0.5%) was significantly lower than the value of 2.5% predicted from isochronous creep curves. This may be due to the fact that creep of the reinforcement can be limited by the soil under working conditions (Li and Rowe 2001b).

Bassett and Yeo (1988) also reported a significant increase of strains in reinforcement after embankment construction. In their case, a 7m-high trial embankment reinforced with a uniaxial HDPE geogrid was constructed across a 4.5m thickness of soft clay/peat/soft clay installed with vertical drains. Following construction, the strain increased by 50% (i.e. from 2% to 3%) and the geogrid force increased by 10%. The increase in reinforcement strain and force after the end of construction was contrary to design expectations. These cases demonstrate that creep of both the reinforcement and foundation soil can increase the reinforcement strain; this will be discussed further in Sections 8 and 9.

Embankments over peat

Geosynthetic reinforcement has been successfully used in the construction of embankments over peat foundations. Rowe et al. (1984a, b) reported that the Bloomington road embankment was constructed over a highly compressible peat deposit using geotextile reinforcement. The thickness of the peat in this deposit varied between 5m and 7.6 m. The average water content of the peat was 445% and 785% at the two investigated Sections A and B. Section A (3.9m fill thickness) was reinforced by a woven geotextile with an ultimate wide-width tensile strength T_{ult} = 41 kN/m and Section B (5.9m fill thickness) was reinforced with another woven geotextile with T_{ult} = 178 kN/m. The high tensile stiffness geotextile used in Section B reduced lateral movements. In the analysis of embankment performance, the use of effective deformation and strength parameters (combined with pore pressures) provided the best agreement between calculated and observed behaviour.

In the case reported by Matichard et al. (1994), the increase of effective stress in the peat due to the relative rapid dissipation of the excess pore pressures during embankment construction resulted in reinforcement strains significantly lower than the design value.

Oikawa et al. (1996) reported that a relative high embankment was successfully constructed over a peat foundation using multiple-layers of geosynthetic reinforcement. The reinforcement layers resulted in a rigid-footing-like behaviour of the embankment. The relatively small shear deformations of the peat foundation during and after the fill placement was attributed to the use of reinforcement and the rapid consolidation of the peat.

True peats (as opposed to organic clays) typically have a high natural water content (50 to 2000%), high void ratio (5 to 15 but may be up to 25), and high compressibility. The hydraulic conductivity of peat is usually initially high but reduces significantly as the peat compresses. Since the porous nature of peat usually allows partial dissipation of excess pore pressure during construction for typical rates of construction, undrained bearing capacity failures are rare for peat deposits underlain by a firm foundation. Failures of embankments over peats are usually caused by excessive shear deformations of the embankment rather than a definite sliding surface. Effective stress analyses, rather than total stress analyses based on the "undrained shear strength", are applicable to embankments over peat foundations and give good predictions (Rowe et al. 1984b).

Undrained shear strengths reported for fibrous peat should be viewed with suspicion (Landva 1980).

Summary

Numerous embankments have been successfully constructed on soft foundations using geosynthetics. Reinforcement and/or PVDs allowed construction on these difficult foundation soils within prescribed construction schedules and performance criteria.

Reinforcement increased the factor of safety against rotational failure and served to maintain the structural integrity of the embankments. The bearing capacity of the foundation soil was also increased due to the use of reinforcement. The reinforced embankment failure height is usually greater than that calculated based on classical bearing capacity theory using the Prandtl solution (Prandtl 1920) for a strip footing on a deep homogeneous clay layer (i.e. $H_f = 5.14\ s_u$) due to either a significant increase in undrained strength with depth in foundation soils or/and a firm stratum at relatively shallow depth beneath the soft layer. Field data has shown that the use of high tensile strength, high tensile stiffness geosynthetics effectively reduced the movements at ground level, minimized lateral spreading, and reduced differential consolidation settlements.

To reach a design grade it is sometimes necessary to combine reinforcement with stage construction so that the foundation soil can have sufficient strength gain for the final embankment load. It has been shown that the effect of strength gain of foundation soils due to partial consolidation is enhanced by the use of geosynthetic reinforcement. At typical construction rates, partial consolidation can be significant when the soil is initially over consolidated, PVDs are installed, or when the foundation includes a fibrous peat layer. The consequent strength gain in the foundation can effectively reduce the reinforcement strain developed at the end of construction. The magnitude of the mobilized reinforcement strain and force have been found to be significantly less than the values used or expected with current design methods for numerous cases where the embankments were constructed over conventional soft soils. This is mainly attributed to the conservative assumption of undrained conditions (used in most current design methods) combined with conservative assumptions associated with the selection of soil strength parameters. The behaviour of a number of reinforced embankments indicates that the assumption of undrained conditions may often be too conservative and that a design method allowing for the effects of partial consolidation could be used for the reinforced embankments.

Reinforced embankments constructed on rate sensitive soils require special care. For such foundation soils, the most critical time with respect to embankment stability may be at some time after the end of construction rather than during construction. Thus, the reinforcement strain and force can increase substantially with time due to the creep of the foundation soils. Another factor contributing to the time dependent behaviour of reinforced embankments is the viscous behaviour of geosynthetics, especially those made of high density polyethylene and polypropylene. In these cases, the reinforcement strains increase with time due to creep of geosynthetics under constant embankment load. There is a paucity of data showing the stress relaxation of reinforcement after construction. This is considered to be because the post construction horizontal deformations of foundation soils resulting from creep and consolidation deformations of foundation soils give rise to an increase in reinforcement force that can offset the stress relaxation in reinforcement.

UNDRAINED BEHAVIOUR OF REINFORCED EMBANKMENTS

Previous studies (e.g. Rowe and Soderman 1985a, 1987b; Rowe and Mylleville 1989; Hird and Kwok 1990) provide significant insights to the undrained behaviour of reinforced embankments over soft plastic foundations. For example, Rowe and Soderman (1985a, 1987b) have shown that the embankment failure height substantially increases with reinforcement tensile stiffness when the undrained shear strength of foundation soils increases with depth. They also showed that the reinforcement strain developed at embankment failure is a function of shear strength and tensile stiffness of the foundation soils and the geometry of the embankment-foundation system. Based on finite element parametric studies, Hird and Kwok (1990) have shown that reinforcement with sufficient stiffness and strength can significantly reduce the undrained displacement within the foundation (Fig. 8) and that the benefit of increasing reinforcement stiffness diminishes for very stiff reinforcement. This is consistent with the findings reported by Rowe and Li (1999) based on analyses of partially drained conditions.

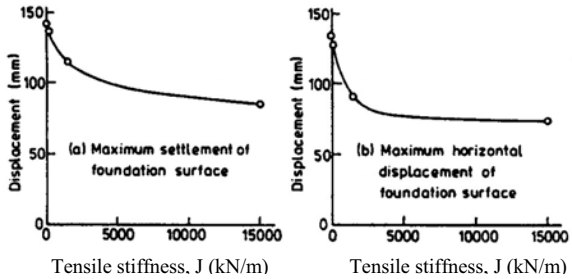


Figure 8. The effect of reinforcement tensile stiffness, J, on the foundation deformations (modified from Hird and Kwok 1990).

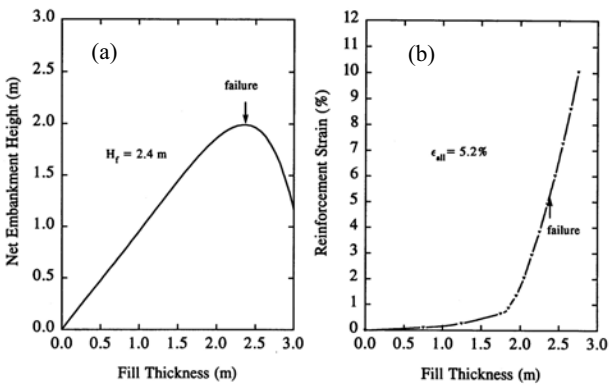


Figure 9. Net embankment height and reinforcement strain for a reinforced (J = 600 kn/m) embankment on soil with s_{uo} = 3.8 kPa and Δ_c = 1.5kPa/m (after Hinchberger 1996).

The collapse mechanism for a reinforced embankment may be different from that of an unreinforced embankment. For an unreinforced embankment, the collapse height of the embankment corresponds to the height at which the soil shear strength is fully mobilized along a potential rupture surface. For reinforced embankments, collapse involves failure of both the soil and the reinforcement or soil-reinforcement interfaces, but practical failure can occur prior to collapse. Prior to actual failure of the geosynthetic reinforcement or the geosynthetic-soil interface, the contribution of the reinforcement to the embankment stability is governed by strain compatibility and the relative moduli/stiffness of the soil and reinforcement. Thus

reinforced embankments constructed on soft foundations may fail or have excessive displacements long before the reinforcement reaches its ultimate tensile strain or pullout occurs. To account for this scenario, the concepts of net embankment height (defined as fill thickness minus maximum vertical deflection) and allowable compatible reinforcement strain were introduced by Rowe and Soderman (1985a, 1987b). For example, Figure 9 shows both 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 of 2.4m at a reinforcement strain of 5.2%. Most geosynthetic products can sustain strains well above 5.2% before they reach tensile failure (GFR Specifier's Guide 2002). For this embankment, placement of fill beyond a thickness of 2.4m only degraded embankment performance without increasing the net embankment height (Fig. 9a). Thus it is important to define an allowable "compatible" reinforcement strain corresponding to the maximum net embankment height for a reinforced embankment. A second allowable strain will be related to the reinforcement properties and will be selected to eliminate the likelihood of reinforcement rupture. The lower of these two strains will govern the design.

The allowable "compatible" reinforcement strain will be discussed in the following subsections. If an embankment has a reinforcement with a tensile stiffness so small that negligible load is developed in the geosynthetic when a failure mechanism develops one would expect that the strains just prior to failure and the failure height of the reinforced embankment to be the same as those corresponding to failure of a similar unreinforced embankment. The maximum strain that would occur in this geosynthetic just prior to this failure is referred to the allowable compatible strain (Rowe and Soderman 1985a).

For reinforced embankments, the construction sequence may have a significant effect on the deformations of the foundation soil based on the analyses reported by Rowe (1982) and Borges and Cardoso (1998). It has been found that for each embankment fill lift, a fill placement sequence from the embankment toe to the embankment centre is favorable to reducing foundation horizontal deformations. However, the magnitude and distribution of reinforcement strains is independent of construction sequence based on the analyses of Hinchberger (1996).

Soils with uniform undrained shear strength with depth

Rowe and Soderman (1985a) conducted an extensive finite element study to establish the allowable compatible strain for reinforced embankments on soils with uniform undrained shear strength. It was found that the compatible reinforcement strain developed prior to embankment failure was function of the ratio of the thickness of the soft deposit D to the embankment width B, the ratio of the undrained shear strength s_u to the undrained modulus, E_u, of the soft foundation soil, and the ratio of $\gamma_f H_c/s_u$ (where, γ_f = the bulk unit weight of the embankment fill; and H_c = the height of an unreinforced embankment at collapse). Figure 10 shows the variation of allowable compatible strain with the dimensionless parameter, Ω, defined as:

$$\Omega = (\gamma_f H_c/s_u)(s_u/E_u)(D/B)^2_e \qquad (15)$$

where $(D/B)_e$ is the ratio of the effective depth of the deposit to the crest width and

$(D/B)_e = 0.2$, when $D/B < 0.2$ (16a)

$(D/B)_e = D/B$, $0.2 \le D/B \le 0.42$ (16b)

$(D/B)_e = 0.84 - D/B$, $0.42 \le D/B \le 0.84$ (16c)

$(D/B)_e = 0$, $0.84 \le D/B$ (16d)

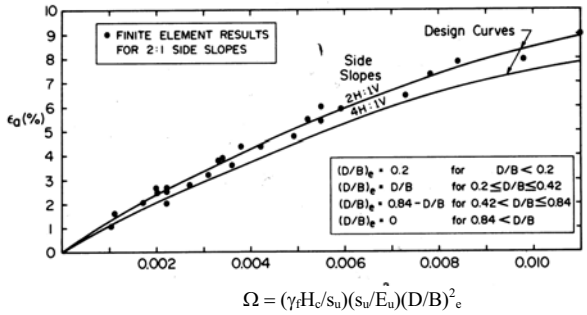


Figure 10. The variation of the allowable compatible strain ε_a with dimensionless parameter Ω (modified from Rowe and Soderman 1985).

Thus in a limit equilibrium analysis of a reinforced embankment, the tensile force in the reinforcement, T, contributing to the restoring moment should not exceed the force that can be developed at the allowable compatible strain (i.e. $T = J\varepsilon_a$, where here J is the secant tensile stiffness of the reinforcement over the strain range $0\text{-}\varepsilon_a$). For reinforced embankments on a deep uniform foundation soil ($D/B > 0.84$), the value of ε_a is equal to zero and hence $T = 0$ (Fig. 10). This implies that the reinforcement has no effect on deep-seated stability and is consistent with the fact that the bearing capacity of a rough rigid footing (~ a reinforced embankment) on a deep uniform cohesive layer is identical to that of a smooth flexible footing (~ an unreinforced embankment) at $q_u = 5.14s_u$ for both cases. It should be noted that when using Equation 15 it is not conservative to underestimate the undrained modulus of the soft foundation soil since a lower value of E_u corresponds to a high value of ε_a, which in turn gives a higher reinforcement force T.

Soils whose strength increase with depth

As is shown in the previous subsection, when the strength of the foundation is uniform with depth the effectiveness of reinforcement is highly dependent on the geometry of the embankment relative to the layer depth. For layers with depths in excess of 0.5 times the embankment crest width, the contribution of even high tensile stiffness reinforcement to stability may be quite small. However, the effect of reinforcement is highly dependent on the reinforcement stiffness and the rate of the increase in undrained shear strength of the foundation with depth.

Finite element results show that the use of reinforcement changes the failure mechanism. In an example from Rowe and Soderman (1987b), the slip surface for the unreinforced embankment was relatively shallow (2m below the ground surface) due to the low strength in the upper layer foundation soils. However, the inclusion of basal reinforcement forced the collapse mechanism (i.e. the slip surface) down into the stronger soil at depth. In fact, the use of reinforcement with J = 4000 kN/m moved the edge of the slip surface from near the shoulder to near the centreline of the embankment and forces it from a depth of 2m to a depth of 8.5m (Fig. 11). Here, the mobilization of the higher soil strength at depth and the tensile resistance in the reinforcement resulted in a significant increase of embankment failure height.

For reinforced embankments over soft foundations, it is important to consider deformations in any assessment of embankment failure, and this is a function of reinforcement stiffness. For example, Figure 12 shows the variation in net fill height above original ground level with the fill thickness for

reinforcement with different tensile stiffnesses. For the unreinforced embankment (J = 0), the maximum net fill height is about 3m and corresponds to the onset of contiguous plasticity in the foundation soil. In this case the failure and collapse height are the same and the net fill height at the onset of collapse is only slightly less than the fill thickness. However, for the reinforced embankments, the maximum net fill height depends on the reinforcement tensile stiffness. Any attempt to place additional fill after attaining this maximum height will result in the loss of net height. The maximum height is considered to be the failure height of the embankment and is directly related to the reinforcement tensile stiffness for a given soil profile. The addition of fill will eventually cause collapse of the reinforced embankment. The collapse load is independent of reinforcement stiffness (i.e. 120 kPa for this case).

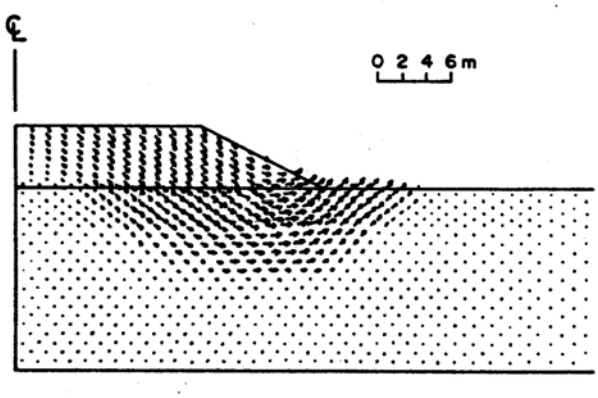


Figure 11. Velocity field at collapse for reinforced embankment (J = 4000 kN/m) on foundation soil with s_{u0} = 7.69 kPa and ρ_c = 1.54 kPa (after Rowe and Soderman 1987b).

The fill thickness at failure depends on three factors: (1) the increase in geosynthetic tensile stiffness, (2) the rate of increase in soil strength with depth, and (3) the strength of the foundation-reinforcement interface. Generally, the greater the increase in strength with depth, the greater the effect of increasing the reinforcement stiffness (Fig. 13, Rowe and Soderman 1987b). For a strength increase with depth of 2 kPa/m, increases in the allowable pressure in excess of a factor of 2 may be achieved with high stiffness reinforcement. The potential for achieving these increases is also controlled by the strength of soil-reinforcement interface (the undrained shear strength of the foundation soil at the ground level). For example, a comparison of the results given in Figs. 13a and c for ρ_c = 2 kPa/m, indicates that for an interface strength of 15 kPa, the increase in allowable pressure with reinforcement stiffness is much greater than that for the case with an interface strength of 5 kPa. The effectiveness of reinforcement is also influenced by the stiffness of the foundation crust. For embankments on soft brittle soils with a high strength crust, the effect of the crust dominates, even if a very high tensile stiffness geosynthetic is used (Mylleville and Rowe 1991).

The approach proposed by Rowe and Soderman (1985a) for embankments on soil whose strength is constant with depth limits the force in the geosynthetic corresponding to an allowable compatible strain which was deduced from the analysis of unreinforced embankments. This approach worked well since for these cases the reinforced collapse mechanism was similar to the unreinforced collapse mechanism. However, this is not the case when embankments are constructed on a foundation whose strength increases with depth. For these cases, the inclusion of reinforcement changes the strain pattern giving rise to a much deeper collapse mechanism passing through stronger and stiffer soil (Fig. 11). Thus the allowable "compatible strain" which is deduced for the unreinforced case is not representative of the reinforcement strains developed prior to failure for reinforced embankments. The reinforcement strain mobilized at the maximum net embankment height (i.e. onset of embankment failure) can be estimated from field instrumentation or finite element analysis.

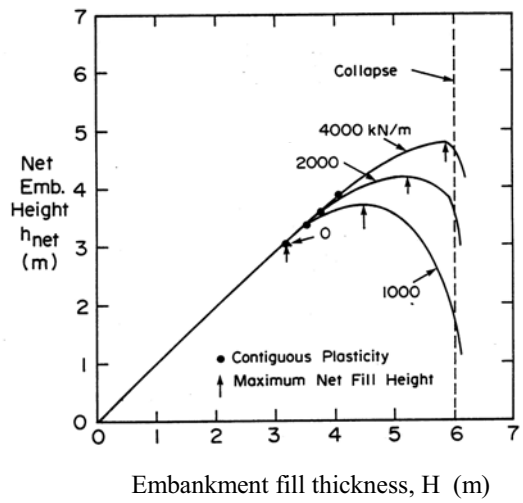


Figure 12. Net embankment height versus fill thickness for reinforcement tensile stiffness, J, values of: 0 (no reinforcement), 1000, 2000 and 4000 kN/m (after Rowe and Soderman 1987b).

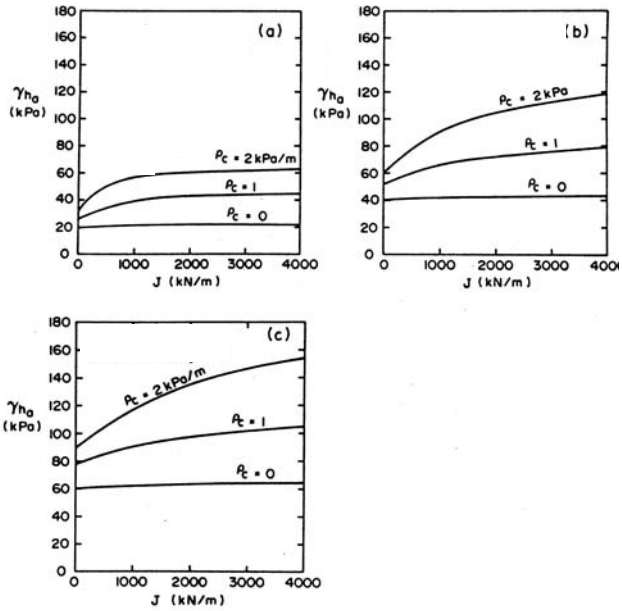


Figure 13. Allowable pressures, (h_a (based on FS = 1.3) versus tensile stiffness, J, for different foundation soils where ρ_c is the rate of increase in undrained shear strength with depth (kPa/m) (a) s_{u0} = 5 kPa, (b) s_{u0} = 10 kPa , (c) s_{u0} = 15 kPa (modified from Rowe and Soderman 1987b).

Hinchberger (1996) found that strain at embankment failure was strongly influence by the kinematics of the collapse mechanism developed within the foundation soil, the undrained shear strength at ground surface, and the rate of increase in strength with depth. The reinforcement strain at failure can be estimated from Figure 14. However the strain may be limited by the strain at rupture of the reinforcement. Also for soft brittle

foundation soils which are susceptible to strain softening, the limiting reinforcement strain may be as low as 0.5 % - 2% in order to reduce the maximum shear strain developed in the foundation soil to acceptable levels (Rowe and Mylleville 1990; Mylleville and Rowe 1991).

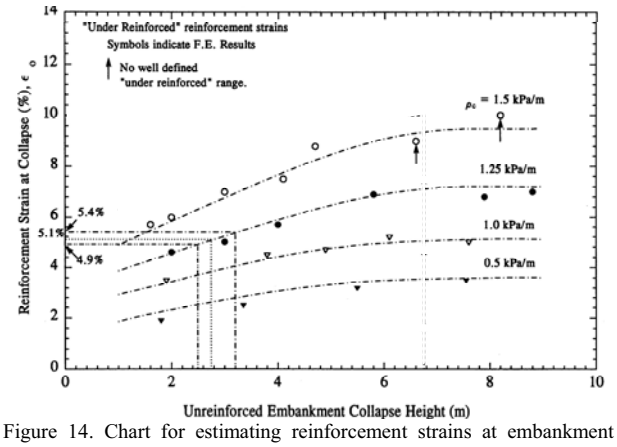


Figure 14. Chart for estimating reinforcement strains at embankment failure for foundation soils with strength increasing with depth (modified from Hinchberger 1996). Note: This chart only considers soil-reinforcement interaction and the allowable strain may be limited to a lower value by the allowable reinforcement strain based on considerations of reinforcement rupture.

Multiple layers of reinforcement

Rowe and Mylleville (1990) found that the failure height of a reinforced embankment was essentially the same for two closely-spaced layers of geosynthetic with a given tensile stiffness and a single layer with twice the stiffness. They also found that the maximum geosynthetic strains at failure were essentially the same for two layers as for one layer of reinforcement when the sum of stiffnesses for the two layer system was the same as the stiffness of the single layer system.

Foundations with surface crust

Humphrey and Holtz (1989) showed that the effect of reinforcement on the embankment stability and deformations was dependent on the strength and compressibility of the surface crust of the soft foundation. The increase in embankment failure height due to the use of a given type of basal reinforcement increases as the crust strength decreases and compressibility increases. This is consistent with the findings by Rowe and Mylleville (1990) that for a deposit with a crust, very high stiffness reinforcement may be required to mobilize significant force prior to embankment failure. For a surface crust with weak zones, reinforcement can be an effective means of increasing embankment stability (Humphrey and Holtz 1989).

PARTIALLY DRAINED BEHAVIOUR OF REINFORCED EMBANKMENTS

An examination of the construction induced excess pore pressures observed in a large number of field cases suggests that significant partial consolidation of the foundation may occur during embankment construction at typical construction rates (Leroueil et al. 1978; Crooks et al. 1984; Leroueil and Rowe 2001). This applies to natural soft cohesive deposits that are typically slightly overconsolidated and, in a number of cases cited in Section 3, it has been reported that there was significant strength gain due to partial consolidation during embankment construction (e.g. Volk et al. 1994; Varuso et al. 1999; Lau and Cowland 2000). This issue requires more consideration than it has been given in the past and is discussed in the following subsections.

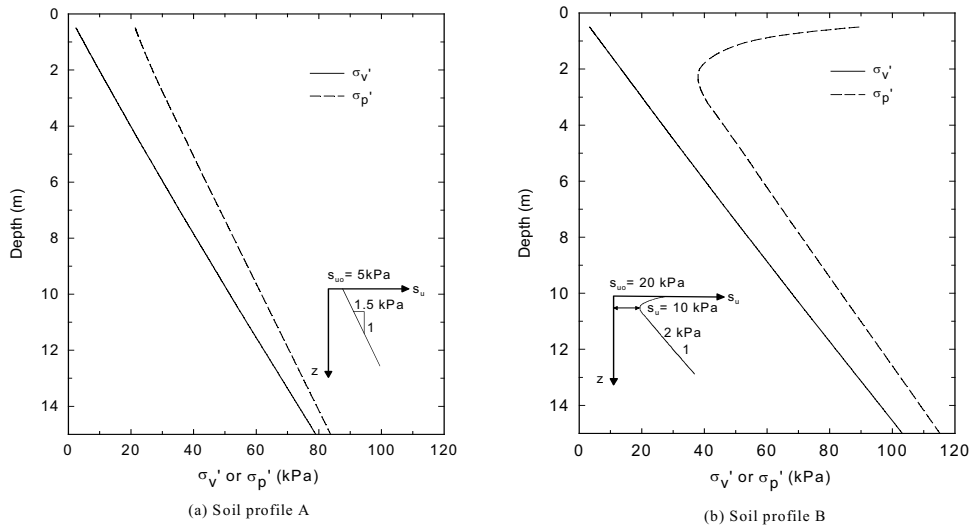


Figure 15. Preconsolidation and strength profiles of foundation soils A and B (modified from Rowe and Li 1999).

Single stage construction of reinforced embankments

Although field cases suggest the importance of considering partial drainage, they do not allow a direct comparison of cases where it is, or is not, considered. However, finite element analyses do provide a powerful tool for comparing the behaviour of reinforced embankments constructed under undrained and partially drained conditions (Rowe and Li 1999). For example, foundation soil A (Fig.15) gave rise to the calculated variation in embankment failure height with reinforcement stiffness given in Figure 16 for undrained and partially drained conditions. For the undrained cases, the unreinforced embankment failure height was 2.1 m. A change of reinforcement stiffness from 500 kN/m to 8000 kN/m resulted in an increase in failure height by between 0.68m to 1.44m relative to the unreinforced case. The effect of reinforcement on the embankment failure height was most significant when J was increased from 500 kN/m to 2000 kN/m.

Fully coupled analyses which allowed for the dissipation of excess pore pressure during construction at a rate of about 1 m/month gave an increase in the unreinforced embankment failure height of 2.44m (Fig. 16). A change of reinforcement stiffness from 500 kN/m to 8000 kN/m resulted in an additional increase in failure height by between 0.71m to 2.36m (relative to the unreinforced case). The reinforcement had a greater effect for the partially drained cases than for undrained cases. An increase in reinforcement stiffness had the most significant effect on the embankment failure height for stiffness values up to J = 2000 kN/m.

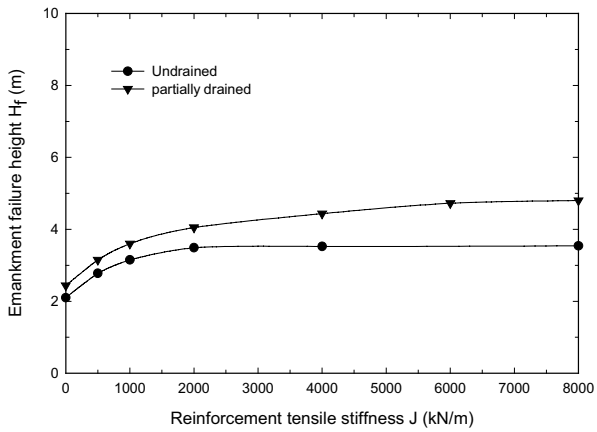


Figure 16. Embankment failure height vs reinforcement tensile stiffness (Soil profile A – Fig. 15).

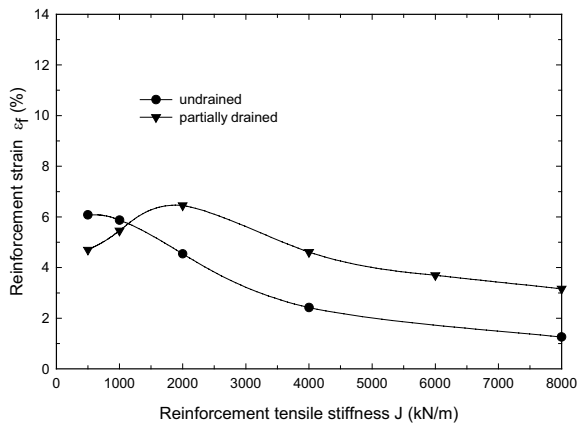


Figure 17. Reinforcement strain at failure (Soil profile A – Fig. 15).

Figure 17 shows the maximum reinforcement strain developed at embankment failure for both undrained and partially drained conditions assuming that the reinforcement can sustain these strains without reinforcement failure. For the reinforced embankments, the failure strains ε_f = 1.3% - 6.1% for undrained conditions can be compared with ε_f = 3.2% - 6.5% for partially drained conditions. This arises because the stronger foundation soil results in some change in the failure mechanism which, in turn, results in a higher embankment and higher reinforcement strain at failure. It is also worth noting that the maximum value for reinforcement strain at failure occurs at very different reinforcement stiffness values (i.e. 500 kN/m for the undrained case and 2000 kN/m for the partially drained case).

Multi-stage construction of reinforced embankments

When a foundation soil does not initially have the strength to safely support a given embankment, stage construction may be used to allow sufficient consolidation and strength gain to occur to support the final embankment load. Li and Rowe (1999) showed the geosynthetic reinforcement may eliminate the need for stage construction or, in cases where it was still needed, it could reduce the number of stages required. For example (Fig. 18) the failure height achieved with 1-stage construction and reinforcement J = 4000 kN/m was similar to that obtained using reinforcement with J = 2000 kN/m and four stage construction (with a 9 month consolidation period between stages and each embankment stage constructed to a height such that a factor of safety of 1.3 was maintained at the end of each stage except the last stage which was constructed until the failure height was reached). The reason for the limited gain in failure height due to multi-stage construction is that while there may be a significant dissipation of excess pore pressure during the early stages of loading (when the soil is initially overconsolidated), there is very little dissipation of excess pore pressure during the 9 months between construction stages once the soil becomes normally consolidated due to the low hydraulic conductivity of clay. In order to gain a greater improvement due to consolidation, one would either need a longer waiting period between construction stages or, alternatively, vertical drains could be used to speed up the dissipation of excess pore pressures.

In some cases the use of high stiffness reinforcement may eliminate the need for stage construction and therefore shorten construction time.

The use of reinforcement enhances the benefit of partial consolidation of the foundation soil during single stage construction (Fig. 16). Reinforcement can also enhance the strength gain made in multi-stage construction (Fig. 19). For example, the increase in failure height (relative to undrained conditions) achieved for different levels of reinforcement stiffness for embankments that were first constructed to the maximum height permitted with a factor of safety of 1.3 and then constructed to failure after an average of 95% consolidation of the foundation soil had been achieved at the end of stage one is shown in Figure 19. It can be seen that the stiffer the reinforcement, the greater the increase in embankment failure height due to foundation soil strength gain. For example, the net increase in failure height was 0.6m (from H_f = 2.5m to H_f = 3.1 m) and 2.4m (from H_f = 4.8m to H_f = 7.2 m) for the unreinforced and heavily reinforced (J = 8000 kN/m) embankments respectively. This improvement occurs because the reinforcement helps to fully mobilize the shear strength of the foundation soil after consolidation. These results also imply that there may be benefit arising from combining reinforcement with methods of accelerating consolidation, such as PVDs, as discussed in the following section.

REINFORCED EMBANKMENTS WITH PVDS

Since the first prototype of a prefabricated drain made of cardboard (Kjellman 1948), prefabricated vertical drains (also called wick drains) have been widely used in embankment construction projects (e.g. Hansbo et al. 1981; Nicholson and Jardine 1981; Jamiolkowski et al. 1983; Lockett and Mattox 1987; Holtz 1987; Holtz et al. 2001). Due to the advantages of prefabricated vertical drains in terms of cost and ease of construction, PVDs have almost entirely replaced conventional sand drains as vertical drains. PVD's accelerate soil consolidation by shortening the drainage path and taking advantage of any naturally higher horizontal hydraulic conductivity of the foundation soil. This improves embankment

stability due to the strength gain in the foundation soil associated with the increase in effective stress due to consolidation.

The use of geosynthetic reinforcement in combination with prefabricated vertical drains has the potential to allow the cost-effective construction of substantially higher embankments in considerably shorter time periods than conventional construction methods (e.g. Lockett and Mattox 1987; Bassett and Yeo 1988; Schimelfenyg et al. 1990).

The synergistic effect of the combined use of reinforcement and prefabricated vertical drains has been investigated by Li and Rowe (1999b, 2001a). It has been shown that the use of PVDs in conjunction with typical construction rates results in relatively rapid dissipation of excess pore pressures. This can be enhanced by the use of geosynthetic reinforcement as shown in the following sections.

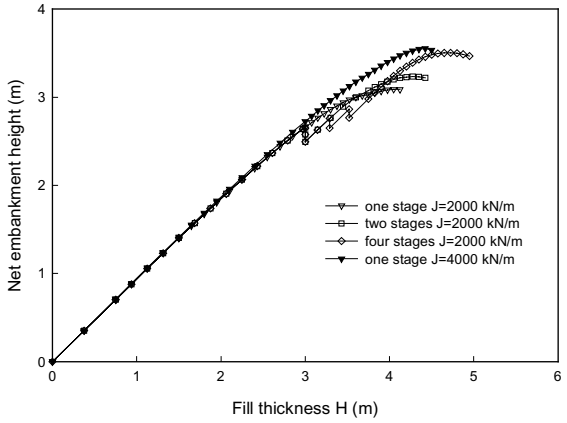


Figure 18. Effect of multi-stage construction on embankment height (Soil profile A- Fig. 15).

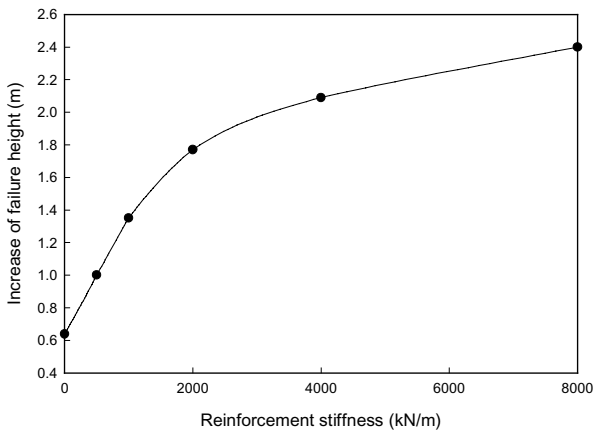


Figure 19. Increase in failure height after 95% consolidation at the end of first stage of construction (Soil profile A- Fig. 15).

The combined effect of reinforcement and PVDs

Figure 20 shows the variation of net embankment height with fill thickness from finite element simulations of reinforced embankment construction, where S is the PVD spacing in a square pattern. For foundation soil profile A (Fig. 15) 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 the assumed soil properties 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. The findings from this finite element analysis are consistent with the findings of Sharma and Bolton (2001) who conducted centrifuge tests on reinforced embankments over foundations with PVDs and found that the PVDs in combination with basal reinforcement gave rise to better mobilization of tension in the reinforcement.

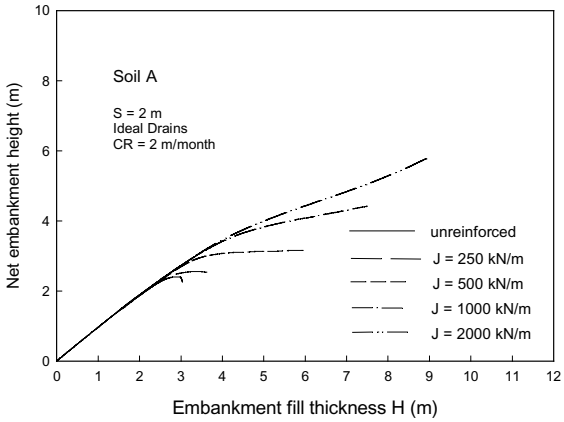


Figure 20. Variation of net embankment height with fill thickness for different reinforcement stiffnesses, J, PVD spacing S = 2 m (Soil profile A – Fig. 15).

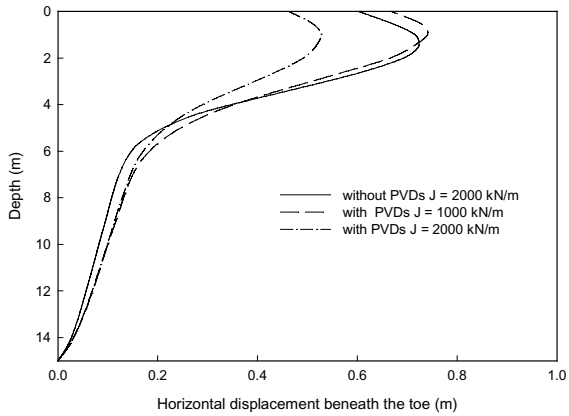


Figure 21. Comparison of horizontal displacements below the toe at the end of construction, H = 3.5m (Soil profile A – Fig. 15).

It is known that reinforcement can reduce the shear stress and consequent shear deformations in foundation soils. In fact both reinforcement and PVDs can decrease horizontal deformations in the foundation below the embankment toe (Fig. 21). For a 3.5m-high embankment over foundation Soil profile A, the combination of PVDs and reinforcement with stiffness J = 2000 kN/m gave the least horizontal foundation displacement while the case without PVDs and J = 2000 kN/m had the about the same horizontal displacements as the case with PVDs and J = 1000 kN/m. These findings are also consistent with the observations by Sharma and Bolton (2001) that the installation of vertical drains reduced the lateral spreading of the soft foundation based on centrifuge test results.

The effect of construction rate

Due to the drainage enhancement provided by PVDs, significant consolidation may occur during embankment construction. The partial consolidation that occurs during construction is a function of construction rate. Therefore, construction rates may be an important factor influencing the stability of embankments.

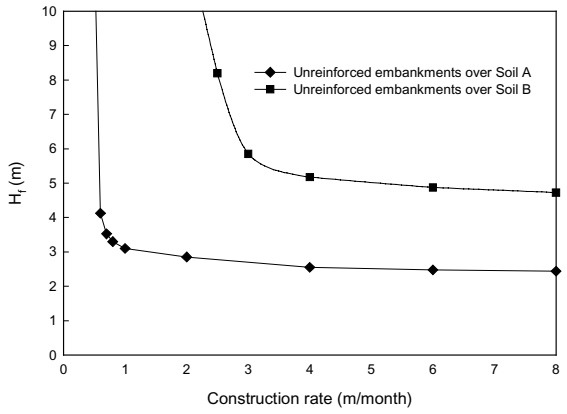


Figure 22. The effect of construction rate on an unreinforced embankment failure height, H_f (see Fig. 15 for soil profiles A & B).

The fact that embankment stability is sensitive to construction rate is evident from the failure heights shown in Figure 22 for an unreinforced embankment. It was found that there was a threshold construction rate below which the embankment would not fail due to bearing capacity failure of the foundation soil. For the configuration of PVDs and two foundation soils examined, the threshold rate was 0.5 m/month and 2 m/month for Soil profiles A and B respectively. This implies that if construction is controlled to a rate slower than this threshold rate, reinforcement is not needed to maintain embankment stability. Conversely, the use of reinforcement could efficiently increase embankment stability when the construction rate is greater than the threshold rate.

Consolidation and strength gain due to PVDs

The use of PVDs to gain significant consolidation of foundation soils during construction has been reported in a number of cases cited in Section 3 (e.g. Lockett and Mattox 1987; Schimelfenyg et al. 1990; Fritzinger 1990; Volk et al. 1994). However, the magnitude and distribution of strength gain has received relatively little attention in the literature.

Based on finite element analyses, Li and Rowe (2001a) have shown that there is significant consolidation of foundation soils with PVDs. For example, Figure 23 shows the contours of the increase in undrained shear strength, Δs_u, of the foundation soil during construction for two reinforced (J = 2000 kN/m) embankments having heights H = 4.4m and 6.5m over Soil profiles A and B (Fig. 15) respectively. For the sake of clarity, Figure 23 does not include the increase in undrained shear strength near the top and bottom layers where the gradient of shear strength increase is high due to the drainage boundary effects. Due 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). The calculated increase in shear strength of soil under the embankment centre was 5 kPa (about 40% of the initial strength) for soil A and 11 kPa (about 50% of the initial strength) for soil B. The increase under the embankment slope was about 3 kPa (about 25% of the initial strength) for soil A and 6.5 kPa (about 30% of the initial strength) for soil B. It is evident that that the increase in shear strength during embankment construction is significant below the embankment but it is also clear that the distribution of the change in undrained shear strength with position needs to be considered in assessing stability since the geometry of the critical failure mechanism may change as a result of the change in magnitude and distribution of shear strength.

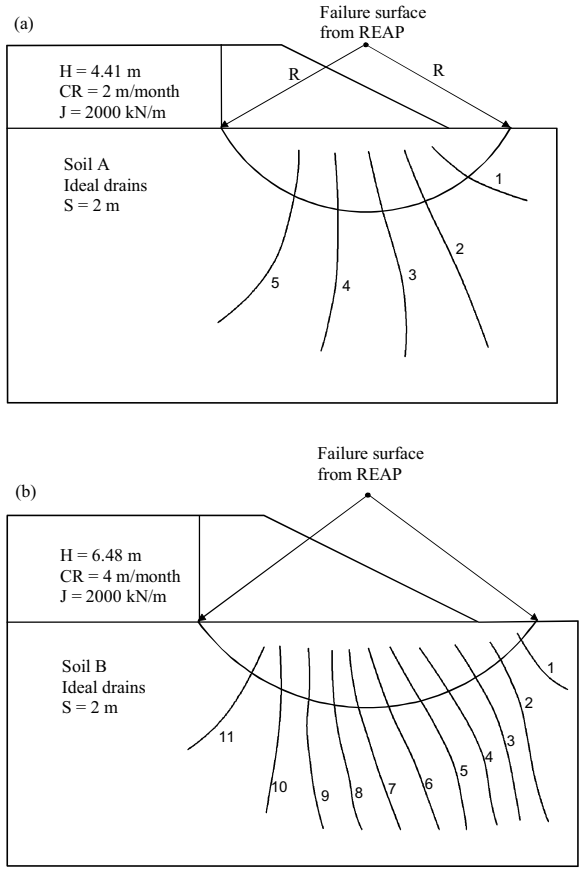


Figure 23. Contours showing the increase in undrained shear strength,)s_u (in kPa) at the end of construction from FEM analyses (after Li and Rowe 2001a).

In analyzing the consolidation of PVD-enhanced foundation soils during embankment construction, consideration should be given to radial and vertical drainage, construction rate, and the difference between consolidation coefficients of soils in the overconsolidated and normally consolidated stress ranges. Generally, a numerical analysis is needed to consider these factors. Li and Rowe (2001) proposed an approximate method for calculating the consolidation of foundation soils allowing for the aforementioned factors. This method can be performed by hand or using a spreadsheet calculation without numerical analysis as outlined below.

The analysis is greatly simplified by the fact that with PVDs the dissipation of pore pressures is essentially uniform with depth (except at the top and bottom boundaries) as implied by the strength gain contours shown in Figure 23. All assumptions of Terzaghi's consolidation theory are preserved except for the change in compressibility as a soil moves from the overconsolidated to normally consolidated state, the time dependent loading and the presence of radial drainage paths. It is assumed that the soil becomes normally consolidated when the average degree of consolidation at a particular time is such that the average vertical effective stress of soils is equal to the preconsolidation pressure. At this time, the change of soil compressibility is a step change (i.e. from recompression index C_r to compression index C_c). The time dependent loading is taken to be a linear ramp loading 0-A as shown in Figure 24. An embankment is constructed to apply a vertical stress of $\Delta\sigma$ over a period of time, $t_{C.}$ During the period to $t_{O/C,}$ when the

soil is overconsolidated, the consolidation is governed by the consolidation coefficient $c_{O/C}$, and after $t_{O/C}$ when the soil becomes normally consolidated, the consolidation is governed by consolidation coefficient $c_{N/C}$. For a deposit with two-way drainage, the average degree of consolidation at any time is defined as

$$\overline{U} = \frac{D\Delta\sigma(t) - \int_0^D u dz}{D\Delta\sigma} \tag{17}$$

where D = the thickness of the deposit; $\Delta\sigma(t)$ = applied stress at time, t; u = excess pore pressure at time, t. At time, $t_{O/C}$, the applied stress $\Delta\sigma(t)$ is $\Delta\sigma\, t_{O/C}/t_C$, the average degree of consolidation is $\overline{U}_{O/C}$, and the average change in effective stress at this time is $\Delta\sigma\overline{U}_{O/C}$. The average excess pore pressure that needs to dissipate after application of the full stress $\Delta\sigma$ is $\Delta\sigma(1-\overline{U}_{O/C})$ and it is assumed that this excess pore pressure is developed over a period of time, $t_C' = t_C - t_{O/C}$. After $t_{O/C}$ the average degree of consolidation, $\overline{U}_{N/C}$, is calculated using $c_{N/C}$ for the ramp load of $\Delta\sigma(1-\overline{U}_{O/C})$. Figure 24 shows 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 is assumed that the average degree of consolidation under the load O-A after $t_{O/C}$ is equivalent to the average degree of consolidation under the load O′-A plus the average degree of consolidation under the load O-B that has occurred at time, $t_{O/C}$. Thus, the total average degree of consolidation at time, $t \geq t_{O/C}$ is:

$$\overline{U} = \overline{U}_{O/C} + \left(1 - \overline{U}_{O/C}\right)\overline{U}_{N/C} \tag{18}$$

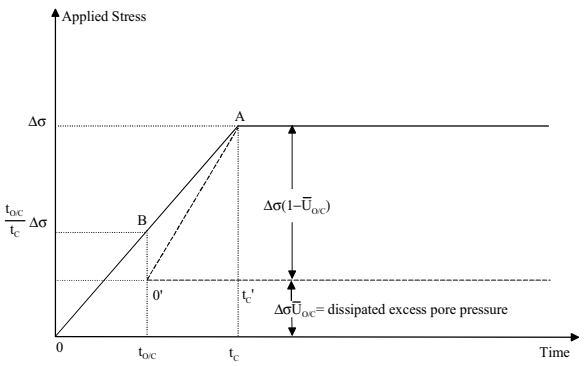


Figure 24. The breakdown of the linear ramp load function for consolidation analysis considering the soil in its overconsolidated and normally consolidated states (modified from Li and Rowe 2001a).

To consider the consolidation of soil under a time dependent loading, Olson (1977) derived relatively simple solutions considering both vertical and radial drainage for a linear ramp loading problem based on the assumptions of the classic consolidation theories except for the time dependent loading. $\overline{U}_{O/C}$ and $\overline{U}_{N/C}$ can be calculated separately using Olson's solution (1977) as will be shown in Section 6.4. Li and Rowe (2001a) verified this approximate method by comparing results using this approach to calculate $\overline{U}$ with values obtained from finite element analyses and showed that this method (Fig. 24) significantly improved the accuracy compared to traditional methods.

As shown in Figure 23, the variation of the strength gain with depth is relatively uniform for a relatively uniform foundation soil and the magnitude of strength gain for soil along the failure surface gradually decreases from a maximum below the embankment crest to a minimum in front of the embankment toe. Based on the average degree of consolidation of the foundation soil (i.e. Eq. 17), the strength gain of soils beneath the embankment centre can be estimated as follows using the SHANSEP method (Ladd and Foott 1974) assuming that the embankment loading is one-dimensional:

$$\Delta s_{uc} = [\alpha(\sigma_{vo}' + H\gamma_{fill}\overline{U})] - s_{uo} \tag{19}$$

where the ratio $\alpha = s_u/\sigma_p'$ is constant for a given soil; σ_{vo}' and s_{uo} are initial vertical effective stress and undrained shear strength prior to embankment construction respectively.

For the locations along the potential slip surface below the embankment slope, the strength gain can be estimated using the method proposed by Li and Rowe (2001a) as following:

$$\Delta s_{uf} = [\beta(\sigma_{mi}' + \gamma_{fill} H I_q \overline{U}_f)] - s_{uo} \tag{20}$$

where

$$\beta = \frac{3}{1 + 2K_0'}\alpha \tag{21}$$

and σ_{mi}' is the initial effective mean stress; I_q is the an influence factor for total mean stress σ_m based on elastic solutions (e.g. Poulos and Davis 1974); $\overline{U}_f$ is the average degree of consolidation of soils along the potential failure surface (i.e. with respect to applied stress of $\Delta\sigma_m = \gamma_{fill} H I_q$); K_o' is the coefficient of lateral earth pressure at rest for soil in its normally consolidated state. The methods for estimating strength gain described herein can be used in the design of the combined use of reinforcement and PVDs as shown in the following section.

Design of reinforcement 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 consideration given to the effects of PVDs (e.g. Jewell 1982; Mylleville and Rowe 1988; Holtz et al. 1997). On the other hand, the design of PVDs is based on consolidation analyses and the drain spacing is selected to achieve a required degree of consolidation within an allowable time period (e.g. Holtz et al. 1991, 2001).

Li and Rowe (2001a) proposed a design method allowing for the combined effects of reinforcement and prefabricated vertical drains. Within a limit state design philosophy, this method uses an undrained strength analysis (USA) method suggested by Ladd (1991) with a total stress analysis allowing for strength gain of a foundation soil at a given time to be considered. The approach is summarized as follows:

1. Select design criteria:

H, B, n	Height, width and slope of embankment;
$\overline{U}$	Average degree of consolidation required;
t	Available time to achieve $\overline{U}$;
CR	Construction rate.

2. Select soil parameters for the embankment fill and foundation:

s_u	Undrained shear strength profile;

σ_p', σ_v',	Preconsolidation pressure and current vertical effective stress with depth;
K_o'	Coefficient of lateral earth pressure at rest profile;
s_u/σ_m'	Normalized shear strength for soil in its normally consolidated state, where σ_m' = effective mean stress = $(1+2K_0)\ \sigma_v'/3$ for soil in its normally consolidated state;
$c_{O/C}$, $c_{N/C}$	Coefficient of consolidation of soil in overconsolidated and normally consolidated states;
k_h, k_v	Horizontal and vertical hydraulic conductivity for undisturbed soil;
k_s	Hydraulic conductivity of disturbed soil;
H_d	Length of longest drainage path in vertical direction;
ϕ, γ_{fill},	Friction angle and unit weight of embankment fill.

3. Select a prefabricated vertical drain system

S	Spacing of PVDs (established during design iterations);
D_e	Effective diameter of drain influence zone; D_e = 1.05 S for triangular pattern; D_e = 1.13 S for square pattern
L	Length of single drain (equal to the thickness of clayey deposits in most cases);
d_s	Diameter of smear zone caused by drain installation;
d_w, q_w	Equivalent diameter and discharge capacity of single drain.

4. Calculate the average degree of consolidation at the available time, t, using Eqs. 17-18. If the calculated average degree of consolidation is less than required $\overline{U}$, repeat Step 3 to select a new PVD configuration (e.g. spacing, S) until $\overline{U}$ is met.

5. Estimate the average influence factor I_q ($\Delta\sigma_m = \gamma_{fill}\ H\ I_q$) for the increase in total mean stress of the foundation soil along the potential slip surface using elastic solutions (e.g. Poulos and Davis 1974; Li and Rowe 2001a).

$$\Delta\sigma_m = \frac{1}{3}\left(\Delta\sigma_x + \Delta\sigma_y + \Delta\sigma_z\right) \tag{22}$$

$$I_q = \frac{\Delta\sigma_m}{\Delta\sigma} \tag{23}$$

6. Calculated the average degree of consolidation along the potential slip surface at the end of the embankment construction, $\overline{U}_f$.
7. Estimate the average strength increase, Δs_{uf}, of soil along the potential failure surface at the end of construction using Eq. 20.

8. Factor strength of soils using partial factor f_c for undrained shear strength of foundation soil ($s_u^* = s_u/f_c$) and f_ϕ for fill material ($\tan\phi'^* = (\tan\phi')/f_\phi$), and f_γ for unit weight of fill γ_{fill} ($\gamma_{fill}^* = \gamma_{fill}\ f_\gamma$) as appropriate.

9. Factor Δs_{uf} using partial factor, f_c ($\Delta s^*_{uf} = \Delta s_{uf} / f_c$).

10. Using a limit equilibrium method calculate the equilibrium ratio (ERAT) of restoring moment to overturning moment for the embankment without reinforcement using the factored soil parameters of embankment fill and factored undrained shear strength profile including the strength gain during construction, i.e. ($s_u^* + \Delta s^*_{uf}$). If ERAT ≥ 1.0, reinforcement is not needed. If ERAT < 1.0, reinforcement is needed, continue with the following steps.

11. Use a limit equilibrium program designed for the analysis of reinforced embankments (e.g. REAP - Mylleville and Rowe 1988) to calculate the required reinforcement tensile force, T_{req}, using new factored undrained shear strength profile obtained in step 10 (T_{req} is the force required to give overturning moment = restoring moment based on factored soil properties, i.e. ERAT = 1).

12. Choose an allowable strain, ε_{all}, for the reinforcement. The required reinforcement tensile stiffness is given by:

$$J_{reg} = T_{req} / \varepsilon_{all} \tag{24}$$

In this procedure, the limit states examined involve failure of the embankment, the foundation, and failure of reinforcement. All calculations except the slope stability analysis can be done by hand or using a spread sheet program. This approach can be made equally applicable to a stage construction sequence by adding the consolidation during stoppage in step 4 and 6 while keeping the other steps the same. Design parameters (e.g. S, L, T_{req} and J_{req}) can be obtained iteratively from steps 1 to 12. In the design iteration, the construction rate and stage sequences can be varied such that the design grade can be achieved in an optimum time schedule. In order to ensure embankment stability during construction, it is essential to monitor the development of reinforcement strains, excess pore pressures, settlement and horizontal deformations and to confirm that observed the behaviour is consistent with the design assumptions. A worked example showing the use of this design method has been given by Li and Rowe (2001a).

REINFORCED EMBANKMENTS OVER PEAT

As discussed in Section 3.8, fibrous peats are characterized by high water content, void ratio, hydraulic conductivity and compressibility. However, organic soils can vary substantially in terms of their engineering properties and response to embankment loads. There are many different classification systems (see Landva et al. 1983) for peat which may have a highly variable inorganic component (ranging from less than 20% to 80% ash content). Thus the term "peat" is often used to include a vast range of organic soils which may range from jelly-like organic silts and very soft organic clay, to extremely course meshes of wood remains and fibres. Peat with 80% ash content and a significant clay component will respond to embankment loads in a manner similar to a clay whereas a fibrous peat with less than 20% ash content may behave more like a frictional material than a cohesive material. For the purpose of this paper, the term "peat" is reserved for soils with less than 20% ash content.

Due to the high void ratio and compressibility of peats, it is usually not practical to obtain realistic strength parameters from conventional triaxial tests due to excessive deformation rather than shear failure under compression (see Adams 1961; Edil and Dhowian 1981; Rowe et al. 1984b). Also, it may be difficult to obtain reliable strength parameters from direct shear tests for some peats due to their fibrosity. Landva (1980) has had success using the ring shear apparatus. Rowe (Rowe et al. 1984b; Rowe and Mylleville 1996 and various unpublished reports) has had considerable success with the Norwegian simple shear apparatus which gave measured friction angles typically between 26^0 and 29^0 for a number of peats.

The failures of peat foundations do not usually involve the formation of a definite sliding surface or bearing capacity failure (e.g. Ripley and Leonoff 1961; Lupien et al. 1983). This is due to the fact that peats can experience significant dissipation of pore pressures during embankment construction at typical construction rates. Therefore, in the analysis of embankment performance, the use of effective deformation and strength parameters (combined with pore pressures) provide the best agreement between calculated and observed behaviour (Rowe et al. 1984 a,b).

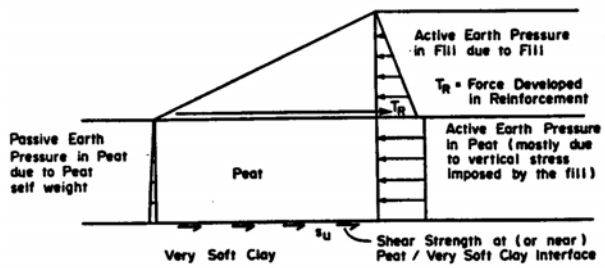


Figure 25. Potential failure mechanism due to lateral thrust and sliding on a deeper low strength layer (e.g. very soft clay beneath a peat layer).

The basal reinforcement can contribute to increasing the embankment stability as shown in Figure 25, where a reinforced embankment on a peat layer is underlain by very soft clay. The active earth pressure in the fill creates a lateral thrust. The applied pressure due to the embankment resting on the peat also induces a lateral thrust. Generally, due to the low effective stress in the peat outside the embankment, the passive resistance is negligible. Thus, the reinforcement and the shear strength at the peat/very soft clay interface must resist the lateral force.

Rate of loading

Since the initial effective stress both within the peat and in any very soft clay or silt layer below the peat is very low, even with very high strength reinforcement there is a limit to the embankment height that can be achieved in single stage construction. Therefore, stage construction and/or a controlled construction rate is recommended for embankments over peat foundations. In either case, it is critical to monitor excess pore pressures and control the construction rate to ensure sufficient increase the effective stress and shear strength of the peat and underlying soft soils

Weber (1969) reported that in his experience with peat deposits in the San Francisco Bay area, fills in excess of 1.5m above original ground level were subject to collapse. The exact height at which instability occurred depended upon the foundation conditions and rate of loading. At a construction rate of 0.9m of fill per week he reported a collapse of a fill at a height of 2.4 m. At an even slower construction rate (0.15-0.3m per week) fills to a height of about 3m could be constructed without failure.

Both field and laboratory observations have indicated that embankments constructed on peat settle rapidly during construction and then continue to settle at a reduced rate after construction (e.g. Adams 1965; Weber 1969; Rowe et al. 1984a). An examination of the time-settlement curves indicates that primary consolidation occurs relatively quickly (typically 5-200 days after construction even though "creep induced pore pressures" may remain for considerable periods during secondary compression.

Based on data in the literature, it appears that where drainage of the peat is provided and a slow rate of construction is used, there will be little difficulty in constructing low (2m or less) embankments on peat, although settlements may be large. However, when an impervious fill is used or where the peat is underlain by soft marl or clay, it may not be possible to construct even low embankments on the peat unless special measures are taken to ensure stability (e.g. provide drainage, berms, geosynthetic reinforcement, or use of lightweight fill etc.).

Embankments on peat underlain by a firm base

Embankments constructed on peat underlain by a firm foundation are less prone to collapse although some cases have been reported. For example, Flaate and Rygg (1964), Ripley and Leonoff (1961) and Lupien (1983) reported "shear failures" which, on the basis the published data, do not appear to involve failure in the soil underlying the peat. These failures do not usually involve the formation of a definite sliding surface. Rather, the collapse involves rapid and excessive shear deformations which give rise to large embankment settlement, lateral movement and mud waves.

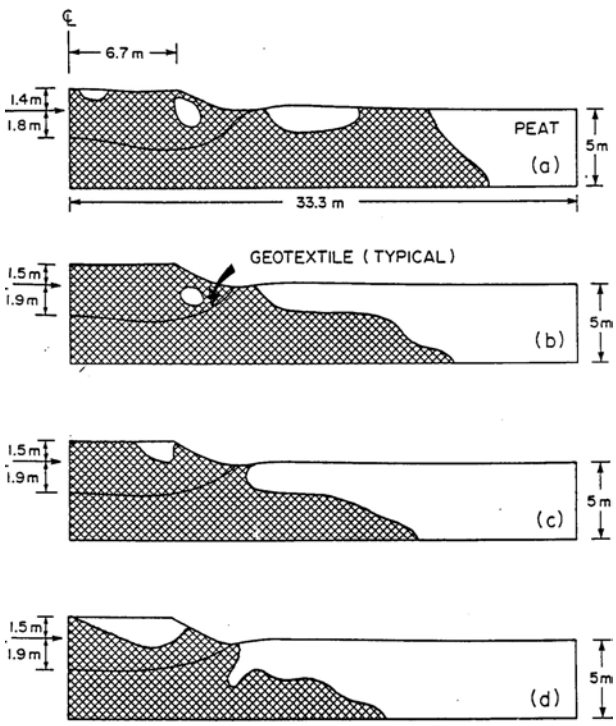


Figure 26. Deformed profile and plastic regions for an embankment approximately 1.5m above original ground level at the end of construction, B_{max} = 0.34; (a) No reinforcement; (b) J = 150 kN/m; (c) J = 500 kN/m, and (d) J = 2000 kN/m (after Rowe and Soderman 1985b).

Rowe and Soderman (1985b) have reported the results of a series of analyses that were performed to examine the effect of geosynthetic reinforcement on the stability and deformations of embankments on peat. The effect of geosynthetic reinforcement is illustrated in Figure 26. The cases examined involved granular fill ($c' = 0$, $\phi' = 32^0$ and $\gamma = 21$ kN/m) and peat (with typical parameters: $c' = 1.8$ kPa, $\phi' = 27^0$, $K'_o = 0.19$ and $e_o = 9$) with a construction rate limited to ensure that the excess pore pressure at the end of construction gave $B_{max} = 0.34$. Figure 26a shows the plastic region obtained in attempting to construct the embankment to 1.5m without reinforcement. In this case excessive shear deformations occurred when the fill was 1.4m above original ground level and the target height of 1.5m above grade could not be achieved. The improvement in the performance of the embankment due to the reinforcement is

evidenced by the fact that the design height of 1.5m could be achieved in each case. The effect of tensile stiffness can be appreciated by comparing the size of the plastic regions and the deformations of the original ground surface line beneath the shoulder of the embankment in Figure 26b,c,d with that in Figure 26a. The unreinforced embankment has the most settlement beneath the shoulder (caused by the rotational movement of the embankment slope). A geosynthetic with a tensile stiffness, J, of 2000 kN/m gives a far more satisfactory dish shaped settlement profile as shown in Figure 26d.

Embankments on peat underlain by soft sediments

Peat deposits are typically encountered in regions that have been subjected to recent glaciation. Frequently, the depositional history of these deposits involves the sedimentation of clay, silts or lake marl (which may have some organic content) followed by the formation of a peat deposit. Because the peat has low unit weight, the clay, silt or marl in a recent normally consolidated deposit will be very weak just below the peat. Under embankment loadings, the underlying soft sediments will behave in an undrained manner. For the cases reported in the literature, the vane shear strength in a very soft stratum below the peat is typically in the range from 5 to 15 kPa although shear strengths as low as 2.5-3 kPa have been reported (e.g. Lea and Brawner 1963). It has been reported that construction and maintenance problems and shear failure are far more likely to occur when the peat is underlain by a weak soil than when it rests on a firm stratum (e.g. Lea and Brawner 1963; Raymond 1969).

Design considerations based on finite element results

Rowe and Soderman (1985b, 1986) have conducted extensive finite element analyses of reinforced embankments on peat underlain by firm or soft deposits. The results have provided a means for design of reinforced embankments on such foundations. For the case of a firm base, the expected variation in required tensile stiffness with design height is shown in Figure 27. The geosynthetic to be used should have a secant tensile stiffness (over the strain range from zero to the expected strain) greater than or equal to that taken from the chart. Table 2 summarizes the cases where the analyses indicated that an embankment could be constructed to a particular height, h, of 1, 1.5, 2 or 2.5m above the original ground level assuming a value of B_{max} in the peat equal to 0.34 and other parameters as given by Rowe and Soderman (1985b, 1986). Note that the fill thickness, H, will be considerably greater than the height, h, above original grade due to settlement that occurs during construction (i.e H= h + EOC settlement). The height, h, should include any surcharge that may be applied prior to pore pressure dissipation.

The tensile stiffness of the geosynthetic required to provide stability under the assumed conditions together with the expected maximum geosynthetic strain under these conditions is given in Table 2. For a number of cases, a range of tensile stiffness values is given. The analysis would indicate that the embankment could be constructed using a geosynthetic with a tensile stiffness within the range specified. However, if the conditions are as bad as assumed, the embankment would be very close to failure using geosynthetics at the low end of the stiffness range and for these cases it would be far better to adopt a geosynthetic with a tensile stiffness in the upper end of the specified range. If there is a competent root mat and/or the anticipated value of B_{max} is less than 0.25, then a geosynthetic with a tensile stiffness in the lower end of the range may be adequate.

The numbers given in brackets in Table 2 represent the expected maximum strain in the given geosynthetic for the worst assumed conditions. The expected force in the geosynthetic can be deduced from the strain and the tensile stiffness. Since the strains are sensitive to construction sequence, strains greater than or less than those indicated in Table 2 may be anticipated under some circumstances and any geosynthetic selected should have an adequate factor of safety against failure of the geosynthetic itself. Thus a geosynthetic should be selected which (a) has an appropriate tensile stiffness, and (b) has an adequate factor of safety against failure of the geosynthetic.

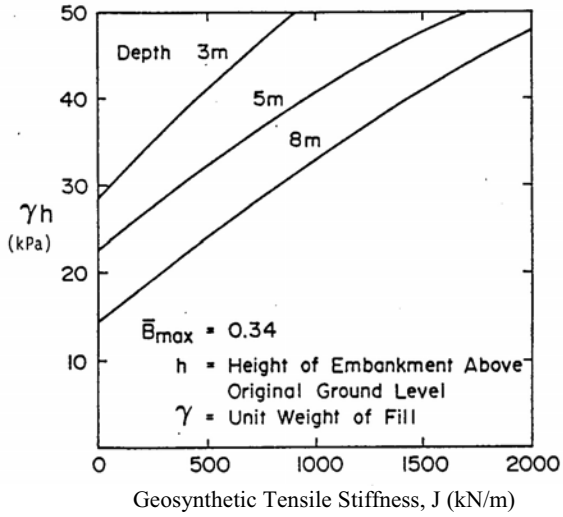


Figure 27. Design chart for peat underlain by a firm base. "Depth" corresponds to the thickness of the peat deposit (modified from Rowe and Soderman 1985b).

REINFORCED EMBANKMENTS OVER RATE SENSITIVE SOILS

Natural soft cohesive deposits often exhibit viscous behaviour and hence the undrained shear strength is strain rate dependent (Casagrande and Wilson 1951; Graham et al. 1983; Leroueil and Marques 1996). Graham et al. (1983) observed that for a number of cohesive soils, an order of magnitude increase in the strain rate during laboratory shear typically resulted in increase of the measure undrained shear strength by between 10 and 20%. Kulhawy and Mayne (1990) compiled data obtained from 26 different rate sensitive clays and showed that an average increase in undrained shear strength typically equaled 10% per logarithm cycle of strain rate (Fig. 28).

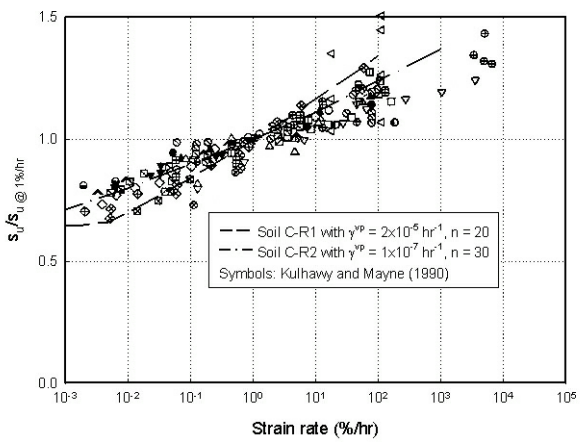


Figure 28. The effect of strain rate on the undrained shear strength, s_u, of Soils C-R1 and C-R2 (after Li and Rowe 2002).

Table 2. Reinforced geosynthetic tensile stiffness values for embankments on fibrous peat (for limitations, see Rowe and Soderman 1986).

Peat Thickness (m)	Underlying strata	Strength of underlying strata (kPa)	Maximum height, h, of fill above original ground level (m)			
			1	1.5	2	2.5
3	Firm	-	NRR[1]	NRR	500 (5%)	1000 (6%)
	2m Clay	15	NRR	NRR	500 (5%)	1000 (6%)
		10	NRR	NRR	500 (6%)	1000 (6.5%)
		7.5	NRR	150(10%)	500 (8%)	1000 (7%)
		5	NRR	150(14%) – 500(8%)	2000 (4%)	PF
		2.5-5	150[2] (10%)[3]	2000 (5.5%)	PF[4]	PF
5	Firm	-	NRR	150(14%) – 500(6.5%)	500(9.5%) – 1000(5.5%)	1000(6.5%) – 2000(4%)
	3m Clay	15	NRR	150(14%) – 500(6.5%)	500(9.5%) – 1000(5.5%)	1000(6.5%) – 2000(4%)
		10	150 (5%)	500 (8.5%) - 1000 (7.5%)	2000 (5.5%)	PF
		7.5	500 (5%)	1000 (8.5%) - 2000 (5%)	PF	PF
8	Firm	-	350 (15%) - 1000 (7%)	500 (14%) – 1000(8%)	1000(9%) - 2000(6%)	2000 (6%)

1 NRR = No reinforcement required; 2 Minimum tensile stiffness value
3 Expected reinforcement strain for geosynthetic with the tensile stiffness indicated
4 PF = Reinforcement alone is not sufficient to provide stability and failure is expected for these conditions unless additional measures are taken to ensure stability.

Critical stage and critical strain rate

Due to the viscoplastic behaviour of foundation soils, embankments may experience significant post construction creep deformations or even failure some time after construction when the excess pore pressures increase or remain at a nearly constant level following the completion of construction (Crooks et al. 1984; Keenan et al. 1986; Kabbaj et al. 1988; Rowe et al. 1996). Under embankment loading, the excess pore pressure and deformation response of rate sensitive foundation soft soils is often reported to be anomalous compared to the response described or predicted by classical consolidation theory. For example, Rowe et al. (1995, 1996) showed that at the Sackville test site substantial vertical and horizontal displacements were recorded in the absence of pore pressure dissipation during periods of a constant embankment load. Kabbaj et al. (1988) summarized a number of embankment cases where the excess pore pressure increased to a maximum value at times after the end of construction ranging from a few days to as much as 150 days after the construction. Crooks et al. (1984) reported that for 11 of the 31 cases they examined, the excess pore pressure in the foundation soils continued to increase significantly following completion of loading and at 6 sites very slow or insignificant porewater pressure dissipation occurred for long periods following construction. These field observations have indicated that the immediate end of construction is not necessarily the most critical stage for embankments on rate sensitive foundation soils.

Duncan and Schaefer (1988) found that at a constant embankment height the force in the reinforcement increased with time due to undrained creep in the foundation soil.

This section presents some results from finite element analyses of a reinforced embankment over a rate sensitive foundation, denoted as Soil C-R1, which was modeled using an Elliptical Cap viscoplastic constitutive relation (Rowe and Hinchberger 1998). This rate sensitive soil has properties similar to Sackville soil described by Rowe and Hinchberger (1998). The liquid limit, plasticity index and natural water content of Soil C-R1 are approximately 50%, 18% and 53% respectively and the undrained shear strength increase by about 16% per logarithm cycle of strain rate (Fig. 28). The limit equilibrium analysis based on the undrained strength at a strain rate of 4.8% per hour (recommended by Bishop and Henkel 1962 for triaxial compression tests) indicates that the design height of 6.75m can be achieved using a reinforcement with design stiffness J = 2000 and ultimate strength of 200 kN/m at 10% strain.

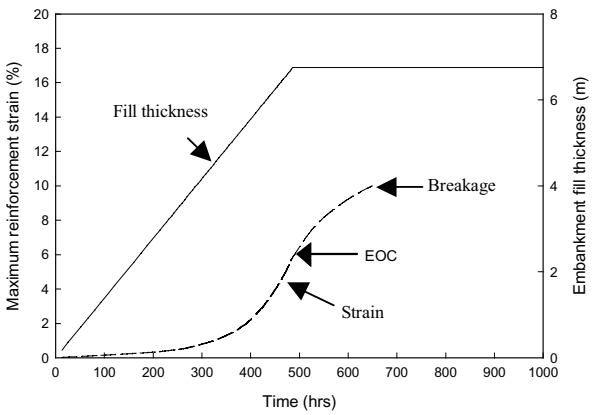


Figure 29. Reinforcement strain developed during short-term for embankments with CR = 10 m/month, J = 2000 kN/m over Soil C-R1.

Figure 29 shows the development of maximum reinforcement strains for a 6.75m-high reinforced embankment on this rate sensitive soil. The maximum reinforcement strain of 5.8% at the end of construction (EOC) exceeded the design limit of 5% based on the limit equilibrium analysis. Furthermore, it kept increasing after the end of construction and reached the

failure strain of 10% at 163 hours (6.8 days) after the end of construction. The breakage of reinforcement can result in the failure of the embankment as was the case in the test embankment reported by Rowe et al. (1995) and Rowe and Hinchberger (1998). Figure 30 shows the horizontal and vertical displacements of the embankment toe before and after the breakage of reinforcement at 10% strain. After the end of construction and before the breakage of reinforcement, the velocity of the toe movement diminished with time. However, at the onset of breakage of reinforcement, the toe movement accelerated rapidly (i.e. velocities became very large) and the analysis indicated that the embankment collapsed upon failure of the reinforcement at 10% strain.

The results shown in Figs. 29 and 30 suggest that the undrained shear strength profile of the foundation soil that would be deduced at a strain rate of 4.8% per hour (i.e. in the range of strain rates recommended by Bishop and Henkel (1962) for triaxial compression tests) might exceed that which could be mobilized in the field. This implies that an arbitrary choice of the strain rate for use in the determination of undrained shear strength for rate sensitive soils may lead to potential post construction failure. Conventional undrained analyses usually make the assumption that embankment stability is critical at the end of construction. This assumption could lead to significant errors for rate sensitive soils as shown in Figs. 29 and 30, and the embankment may fail sometime after the end of construction due to the viscous behaviour of the rate sensitive soil.

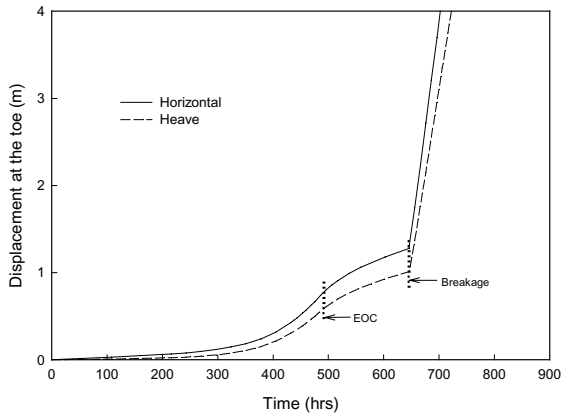


Figure 30. Displacements at the toe for the case with J = 2000 kN/m, H = 6.75m and reinforcement breaking at ε = 10%.

The "critical stage" of an embankment on a rate sensitive foundation soil, as defined by Li and Rowe (2001c), can be illustrated in terms of the calculated effective stress path and strain rate at the mid point along the potential slip surface as shown in Figure 31. During the initial elastic loading there was some limited pore pressure dissipation and the stress path was nearly vertical while the strain rate increased slightly. After the soil became viscoplastic, the stress path moved towards the failure envelope in an undrained manner and the strain rate increased rapidly. Subsequently the stress path moved above the long-term strength envelope due to rate effects and the strain rate reached its maximum at the end of construction. After the end of construction the stress path moved downwards during creep and stress-relaxation and the mobilized strength of soil decreased with time. The critical stage was reached when the stress path moved to the lowest location corresponding to the lowest undrained shear strength. At this point, the embankment was least stable and the corresponding point in time is called the "critical stage". After the critical stage, the stress-relaxation was overcome by the increase in effective stresses due to consolidation, and the stress path eventually moved away from the failure envelope. The strain rate decreased rapidly with time during consolidation.

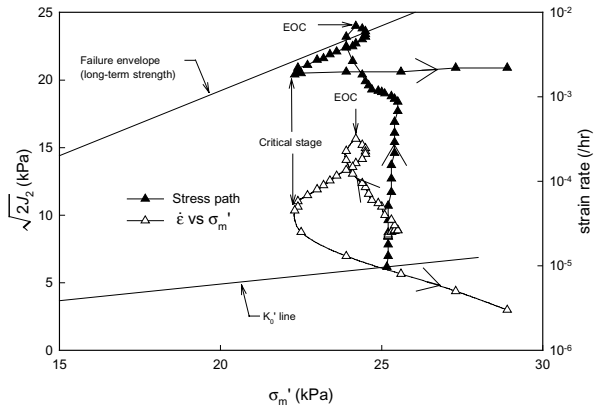


Figure 31 Stress path and strain rate at mid point along the potential slip surface for a 5m-high reinforced (J = 2000 kN/m) embankment over Soil C-R1 (CR = 10 m/month; J_2 = second stress invariant)

The strain rate corresponding to the critical stage is defined as the "critical strain rate". At the critical stage, the mobilized shear strength is the operational shear strength that governs the stability of the embankment. Li and Rowe (2002) have shown that the operational undrained shear strength of foundation soil beneath an embankment is the strength that will be mobilized when the soil is sheared at the critical strain rate.

Operational shear strength of rate sensitive soils

The importance of strain rate effects on embankment stability is discussed by Bjerrum (1973), Tavenas and Leroueil (1980) and Leroueil and Marques (1996). For design, the undrained shear strength may be estimated using field tests, such as the vane test (Bjerrum 1972), and laboratory tests (Ladd and Foott 1974). Field vane tests usually involve high strain rates and hence may be expected to overestimate the undrained shear strength of soils. It has been shown that the undrained shear strength based on field vane tests may significantly overestimate the operational undrained strength of rate sensitive soils (Rowe and Hinchberger 1998; Rowe and Li 2002). The strain rate for triaxial undrained compression tests is typically recommended to be between 2.4 and 4.8 %/hr (Bishop and Henkel 1962) or 0.5 and 1.0 %/hr (Germaine and Ladd 1988). These rates may be much faster than the critical strain rate of soil under an embankment (Li 2000). A design based on the measured strength without appropriate correction may lead to post construction failure of embankments (Rowe and Li 2002). It is convenient to introduce a correction factor, : , to the measured shear strength to allow for strain rate effects (i.e. s_{ou}= : s_u). However, the correction factor for strain rate effect has been difficult to choose due to the uncertainty regarding the strain rate that governs the operational shear strength in foundation soils.

Based on the critical stage concept, Li and Rowe (2002) have proposed an undrained shear strength correction factor that can be calculated using critical strain rate. It has been shown that the embankment reaches the critical stage some time after construction during the period of significant creep and stress-relaxation before significant consolidation occurs. A power function is used to correlate the undrained shear strength at different strain rates as follows:

$$\frac{s_{ou}}{s_u} = \left(\frac{\dot{\varepsilon}_c}{\dot{\varepsilon}} \right)^{1/m} \tag{25}$$

where m = the strain rate parameter (similar to the viscoplastic model parameter n); $\dot{\varepsilon}_c$ = the critical strain rate; $\dot{\varepsilon}$ = the strain rate used in tests; and, $s_{ou:}$ = the field operational undrained strength.

The parameter m in Eq. 25 represents the gradient of the undrained shear strength and logarithmic strain rate relationship. Based on a summary of a number of clays reported in literature, Hinchberger (1996) has shown that the parameter m ranges from approximately 42 for remolded Boston Blue clay to 11 for highly sensitive Mastemyr clay. Soga and Mitchell (1996) have shown that m typically ranges between 15 and 29 for a number of clays. Li (2000) has calculated critical strain rates that are in the range of 5×10^{-6} /hr and 5×10^{-5} /hr with a typical value of 1×10^{-5} /hr for a relatively wide range of rate sensitive foundation soils. As a first approximation it appears that for a typical rate sensitive soil, the average critical strain rate of 1×10^{-5} /hr may be used to estimate the operational undrained shear strength. From Equation 25 the correction factor, μ, that is applied to the measured shear strength, obtained at a known strain rate $\dot{\varepsilon}$ (hr^{-1}), can be expressed as follows:

$$\mu = \left(\frac{1 \times 10^{-5} / hr}{\dot{\varepsilon}} \right)^{1/m} \tag{26}$$

Equation 26 is generally expected to be conservative since partial drainage during construction can significantly reduce the creep deformations of foundation soils and improve the stability as shown by Rowe and Li (2002).

Effect of foundation soils on reinforcement strains

The deformations of creep and excess pore pressures may increase after the end of construction due to undrained creep of a rate sensitive foundation soil. The deterioration foundation strength after the end of construction and before the critical stage can result in mobilization of increased reinforcing forces (and strains) as needed to maintain stability. In the Sackville reinforced embankment case, the reinforcement strain increased significantly with time at constant fill thickness after the foundation soil became viscoplastic (Figs 5 and 6 and Rowe and Hinchberger 1998).

Li (2000) has shown that for reinforced embankments over rate sensitive soils the time dependent reinforcement strain is significant until the critical stage is reached. For example, Figure 32 shows the development of maximum reinforcement strain from beginning of construction to 95% consolidation for a 5m-high reinforced (J = 2000 kN/m) embankment constructed over foundation Soil C-R1 at rates of CR = 2 and 10 m/month. It can be seen that the maximum reinforcement strains are relatively small at the end of construction and increase significantly between the end of construction (EOC) and the critical stage. For the CR = 10 m/month case, the maximum reinforcement strains at the end of construction, the critical stage and 95% consolidation are 1.4%, 4.6% and 5.5% respectively. There is a greater than three fold increase in reinforcement strain due to foundation creep occurring between the end of construction and the critical stage. During consolidation, following the critical stage, the reinforcement strain only increases slightly.

The maximum reinforcement strain developed both during and after the construction was affected by the construction rate and the consequent amount of partial drainage(Figure 32). For this case, a five-fold decrease in construction rate results in a reduction of reinforcement strain from 4.5% to 3.7% at the critical stage, and from 5.5% to 4.8% at 95% consolidation. Thus it is evident that the partial consolidation of the foundation soil during construction reduced the post construction creep deformations of the foundation soils and the strain in the reinforcement.

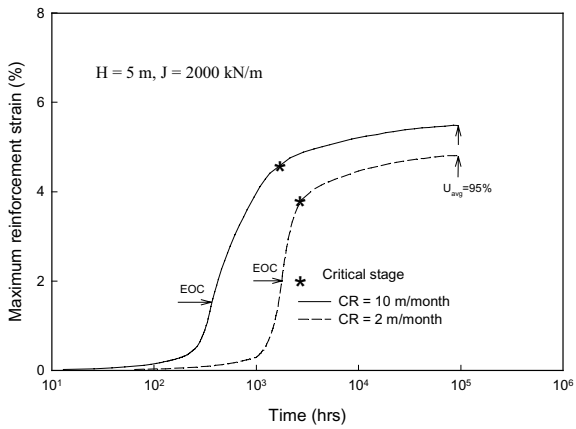


Figure 32. Short-term and long-term reinforcement strains developed for embankments over Soil C-R1.

CREEP OF REINFORCEMENT IN EMBANKMENTS

Experimental studies have shown that geosynthetic materials, especially those made from polyethylene (PE) and polypropylene (PP), are all susceptible to creep (e.g. McGown et al. 1984; Christopher et al. 1986; Greenwood and Myles 1986; Leshchinsky et al. 1997). Generally, the creep rate of polyethylene (PE) is greater than that of polypropylene (PP), which is greater than that of polyester (PET) (den Hoedt 1986; Jewell and Greenwood 1988; Greenwood 1990). The stress-strain behaviour of a geosynthetic is also the function of strain rates (Shrestha and Bell 1982; Rowe and Ho 1986; Bathurst and Cai 1994; Nothdurft and Janardhanam 1994; Boyle et al. 1996) and ambient temperature (Wrigley 1987; Jewell and Greenwood 1988; Bush 1990; Koerner et al. 1993; Rimoldi and Montanelli 1993; Thornton et al. 1997).

The time dependent behaviour of a reinforced embankment may result from the time dependent response of both the geosynthetic reinforcement and soft cohesive foundation soils (Li and Rowe 2001d). The viscoelastic nature of geosynthetics can significantly influence the performance of reinforced embankments over soft soils.

Viscous behaviour of geosynthetics during construction and after construction

The construction of embankments over soft foundations usually takes some days or months due to construction conditions associated with poor foundations. The construction time can influence the response of geosynthetic reinforcement to the embankment loading due to the viscous behaviour of the geosynthetic (Li and Rowe 2001b). Figure 33 shows the calculated net embankment height versus fill thickness for two embankments constructed over foundation Soil B (Fig. 15). The first embankment is reinforced using a uniaxial HDPE geogrid reinforcement (G2) with wide-width tensile stiffness $J_{5\%}$ = 1940 kN/m at 5% strain, the ultimate strength T_{ult} = 166 kN/m, and a

creep strain of 5% over a 20-month period at 40% of the ultimate strength. The second embankment is reinforced using an elastic reinforcement with a stiffness J = 1940 kN/m, which is equal to the secant stiffness at 5% strain of the geogrid reinforcement measured from the wide-width tensile test at a strain rate of 10%/min. Both embankments are constructed at a construction rate of 10 m/month. For the first embankment, the failure height H_f (i.e. the fill thickness at failure) is 4.88m and the mobilized reinforcement strain at embankment failure was 5.3%. At this strain, the mobilized reinforcement tensile force at embankment failure is 67 kN/m. It is evident that the mobilized force for embankment construction is significantly less than the ultimate strength, T_{ult} = 166 kN/m. This implies that in the design of a reinforced embankment the ultimate strength of the reinforcement should not be directly used to estimate the factor of safety.

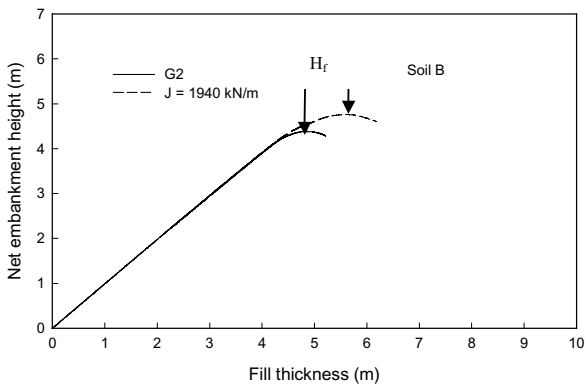


Figure 33. Variation of net embankment height with fill thickness.

For the embankment reinforced using a perfectly elastic reinforcement, the embankment failure height H_f is 5.7m and the failure strain was 9.4%. The mobilized tension in reinforcement was 182 kN/m. It is evident that the viscous behaviour of the geogrid reinforcement can affect the embankment failure height, and the maximum mobilized reinforcement tensile strain and force. If follows from Figure 33 that the creep sensitive reinforcement behaves in a less stiff manner than it would in a tensile test at a relatively fast strain rate (i.e. 10% /min) due to creep and stress-relaxation of reinforcement during construction. Consequently, the viscoelastic behaviour of geosynthetic reinforcement decreases the embankment stability (Figure 33) in terms of the embankment failure height. For the case shown in Figure 33 the viscous behaviour of the reinforcement during embankment construction decreased the embankment failure height by 14%.

For creep sensitive geosynthetic reinforcement, the reinforcement strain may significantly increase with time due to creep of the geosynthetic after embankment construction (Li and Rowe 2001b). Figure 34 shows the time dependent reinforcement strain for two stable embankments using two different geosynthetics (i.e. HDPE geogrid vs. PET geosynthetic) with allowable strain between 4 and 5%. To identify the effect of viscoelastic properties of geosynthetics, the dashed lines show strains for the same embankment reinforced using an inviscid reinforcement with stiffness having the same value developed by the viscous reinforcement at the end of construction. Since the end of construction strains of both viscous and inviscous reinforcement are the same, the difference in the long-term reinforcement strains is mostly attributed to creep of the viscous reinforcement. For the PET geosynthetic reinforced embankment, the creep of geosynthetic is insignificant since the long-term reinforcement strains for both viscous and inviscid reinforcement are practically the same. For the HDPE geogrid reinforced embankment, the reinforcement strain is 4% and 8.4% at the end of construction and 98% consolidation respectively. Even though the long-term reinforcement strain has exceeded the allowable strain the embankment is still stable due to the strength gain of the foundation soil. The creep strain that occurred between the end of construction and 98% consolidation for the HDPE geogrid is about 2%. The predicted increase of reinforcement strain after construction shown in Figure 34 is consistent with field observations as discussed in Section 3.7.

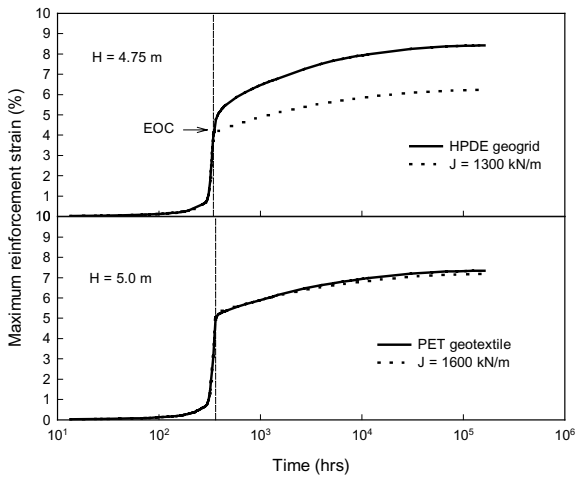


Figure 34. Variation of reinforcement strain with time for embankments (Soil profile B- Fig. 15).

Mobilized and isochronous reinforcement stiffness

Based on the reinforcement force and strain developed, the mobilized reinforcement stiffness (i.e. secant stiffness) can be calculated at different times. For the HDPE geogrid reinforced embankment shown in Figure 33, the reinforcement stiffness developed at embankment failure is 1264 kN/m, which is only 65% of the tensile stiffness at 5% strain determined by an ASTM D4595 test. This implies that for creep sensitive reinforcement the tensile stiffness from the standard test can significantly overestimate the reinforcement stiffness that can be developed in the field.

Li and Rowe (2001b) showed that the time dependent stiffness can reasonably and conservatively represent the operational stiffness at the end of construction. It is recommended that the isochronous stiffness should be used in design to estimate the mobilized reinforcing force at the end of embankment construction. Figure 35 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 closely approaches the isochronous stiffness in the long-term. The time dependency of the mobilized reinforcement stiffness is affected by the creep sensitivity of the geosynthetics. The mobilized stiffness at end of construction is 60% and 95% of the secant stiffness at 5% strain measured in the standard test at a tensile strain rate of 10%/min for HDPE and PET reinforcement respectively. Thus the force in the reinforcement at the end of embankment construction may be significantly lower than expected in design due 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 (e.g. ASTM D4595) to deduce the design tensile force. In

addition to creep effects, consideration should be given to potential construction damage of reinforcement (Allen and Bathurst 1994, 1996).

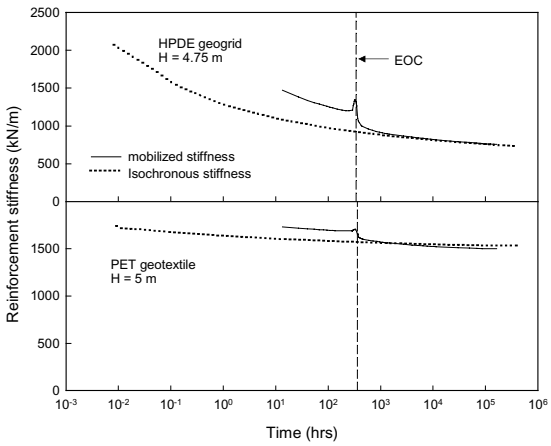


Figure 35. Variation of reinforcement tensile stiffness with time (Soil profile B - Fig. 15).

Effect of reinforcement creep on deformations

Creep and stress-relaxation of geosynthetic reinforcement can potentially allow an increase in foundation deformation that could be excessive in some cases. Li and Rowe (2001b) have shown that creep of the basal embankment reinforcement during the post construction period can result in embankment slope movements along the potential slip surface.

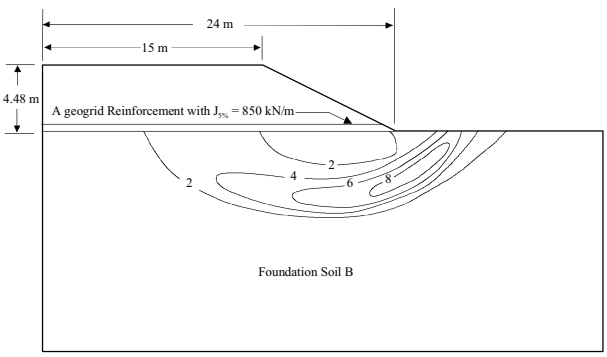


Figure 36. Contours of maximum shear strain (%) in foundation Soil profile B (Fig. 15) for a 4.5 m-high reinforced embankment due to the increase in reinforcement strain of 2.4% between the end of construction and 98% consolidation.

Figure 36 shows the contours of maximum shear strain in the foundation Soil B (Fig 15) that corresponds to a 2.4% creep induced increase in reinforcement tensile strain at a constant embankment fill thickness between the end of construction and 98% consolidation for a particular case. It is evident that reinforcement creep and stress-relaxation allows an increase in the shear deformations of the foundation soil. The maximum increase of foundation shear strain is over 8%, which is about 4 times the creep strain of the reinforcement.

CONCLUSIONS

The current state-of-the-art with respect to the behaviour and design of geosynthetic-reinforced embankments over soft foundations has been examined. The conclusions that can be drawn from this study are summarized below.

- Field case histories have shown that the use of geosynthetic reinforcement provides a cost-effective alternative to conventional construction methods.
- The maximum reinforcement strains observed in the field under working conditions are usually lower than the design values. This can be attributed to a combination of the low shear strength adopted in design, partial consolidation of foundation soils during construction, and working stress conditions.
- The finite element method provides an effective tool to analyze reinforced embankments constructed over soft deposits. However, careful consideration must be given to the types of constitutive relationships used to model the components of the reinforced embankment system.
- Geosynthetic reinforcement can substantially increase failure heights for embankments constructed over both rate sensitive and rate insensitive soft cohesive soils.
- The reinforcement has the greatest beneficial effect for embankments on deposits where the soil strength increases with depth.
- The increase in stability due to the use of stiff reinforcement is greater under partially drained conditions than under undrained conditions.
- The beneficial effect of partial consolidation during embankment construction is enhanced by the use of reinforcement.
- The use of reinforcement can reduce the number of construction stages and consequently shorten the construction time for embankments on soft soil.
- Prefabricated vertical drains may allow significant partial consolidation of the foundation soil during embankment construction. In this case, the assumption of undrained conditions is too conservative and the strength gain of the foundation soil due to partial consolidation can be considered in design using the method proposed by Li and Rowe (2001a).
- The combination of reinforcement and PVDs efficiently increases embankment stability and reduces deformations. The design procedure proposed by Li and Rowe (2001a) integrates consideration of reinforcement and PVDs in design.
- Embankments can be safely constructed over peat soils using reinforcement in combination with appropriate construction rates. The major effect of reinforcement is to reduce lateral spreading and increase stability.
- For reinforced embankments constructed over rate sensitive foundation soils, the viscoplastic behaviour of foundation soils can significantly reduce embankment stability after the end of construction. With respect to stability, the critical stage occurs at the end of essentially undrained creep and stress-relaxation of foundation soils and before significant consolidation occurs.
- The critical strain rate at which foundation soils will deform controls the operational strength of foundation soils.
- The operational undrained shear strength of rate sensitive foundation soils can be estimated by applying the correction factor proposed by Li and Rowe (2002). Nevertheless, more field verification of this approach is desirable.
- The use of reinforcement can significantly reduce the creep deformations of the foundation soils. The stiffer

the reinforcement the less creep deformation is developed.

- The strain rate characteristics of viscoplastic foundation soils may result in a small reinforcement strain being mobilized at the end of construction. However, due to the creep of the foundation soils after construction, reinforcement strains may increase significantly with time before the critical stage is reached.
- Due to strain rate effects during embankment construction, viscoelastic geosynthetic reinforcement behaves in a less stiff manner than implied by wide-width tensile tests conducted at a standard rate of 10%/min.
- The isochronous stiffness measured from in-isolation creep tests appears to reasonably and conservatively represent the end of construction stiffness of reinforcement.
- Creep and stress-relaxation of reinforcement can magnify the foundation shear deformations.
- Consolidation, the viscous characteristics of rate sensitive clays, and the viscous nature of geosynthetics all contribute to the time dependent behaviour of reinforced embankments.

The use of geosynthetic reinforcement to increase the stability of embankments on conventional soft clay and peat foundations is now well established as discussed herein. This paper has highlighted some of the recent advances in (a) considering the effect of partial consolidation during construction and in particular combining the effect of reinforcement and PVDs; (b) understanding the behaviour of reinforced embankment on rate sensitive soils and allowing for their rate sensitive nature in design; and, (c) understanding the effect of reinforcement creep on embankment performance.

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