Performance of a reinforced embankment on a sensitive Champlain clay deposit

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Abstract
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Keywords
sensitive, clay, embankment, reinforced, performance, champlain, deposit

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Performance of a reinforced embankment on a sensitive Champlain clay deposit

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Notation

$\phi'$ friction angle

$\psi$ dilation angle

$\dot{\varepsilon}_{ij}$ strain rate tensor

$\dot{\varepsilon}_a$ axial strain rate

$\dot{\varepsilon}_{sv}$ threshold strain rate

$S_{ij}$ deviatoric stress

$E$ Young’s modulus

$G$ shear modulus

$\sigma_{ii}$ summation of the principal stresses

$K$ bulk modulus

$\nu$ Poisson’s ratio

$e_o$ initial void ratio

$k_o$ initial hydraulic conductivity

$C_k$ hydraulic conductivity change index

$P_o$ atmospheric pressure

$\Phi(F)$ flow function

$\sigma_{os}^{(d)}$ overstress

$n$ strain rate exponent

$f$ plastic potential function and yield surface

$l$ mean effective stress corresponding to the center of the ellipse

$R$ the ratio between major and minor axis of the ellipse

$\sigma'_m$ mean effective stress

$\sigma'_{my}$ intercept of the ellipse with the $\sigma'_m$ axis

$\sigma'_p$ preconsolidation pressure

$M_{NC}$ slope of the failure surface in normally consolidated soil

$M_{OC}$ slope of the failure surface in overconsolidated soil

$\lambda$ critical parameter - compression index
Notation (continued)

\( \kappa \)  
critical parameter - recompression index

\( J \)  
reinforcement axial tensile stiffness

\( \gamma_{s}^{p} \)  
fluidity of the undisturbed clay fabric

\( \gamma_{i}^{p} \)  
fluidity of the destructed clay fabric

\( \gamma^{p}(\varepsilon_{d}) \)  
state-dependent fluidity of the clay fabric

\( \omega_{o} \)  
parameter define soil structure level

\( \omega(\varepsilon_{d}) \)  
state-dependent soil structure level

\( \varepsilon_{d} \)  
damage strain

\( d\varepsilon_{d} \)  
external damage strain

\( d\varepsilon_{v}^{p} \)  
external plastic volumetric strain

\( d\varepsilon_{s}^{p} \)  
external plastic shear strain

\( \alpha \)  
material parameter governing the rate of destructuration

\( A \)  
weighting parameter

\( K_{s} \) and \( m \)  
material constants for nonlinear elastoplastic fill material

\( \sigma_{T} \)  
tensile stress of the independent spring

\( a_{o}, a_{s}, \delta, \beta \)  
material constants for nonlinear viscoelastic reinforcement

\( n_{K} \)  
number of Kelvin elements

\( \tau_{i} \)  
retardation time

\( E_{i} \)  
spring modulus

\( \eta_{i} \)  
dashpot viscosity
Performance of a reinforced embankment on a sensitive Champlain clay deposit

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ABSTRACT
An existing elasto-viscoplastic constitutive model is modified using concepts of the state-dependent fluidity parameters and the damage law, to incorporate the effect of soil structure and its destructuration. The model is employed to simulate the performance of a well documented case study of the reinforced test embankment constructed over sensitive Champlain clay deposit at Saint Alban, Quebec. The finite element calculations, using both original (non-structured) and modified (structured) elasto-viscoplastic soil model, are compared with the observed field data from a test embankment brought to failure. The results from the structured elasto-viscoplastic soil model show better agreement with the field data than those obtained using a non-structured elasto-viscoplastic soil model. The modified model captures many features of the reinforced embankments observed behaviour such as vertical settlement, excess pore water pressure response and reinforcement force. However, the horizontal deformations in the clay deposit were not modeled satisfactorily. The role of geosynthetic reinforcement and its viscosity on the short-term response of the reinforced embankment is also discussed

KEYWORDS: elasto-viscoplastic; strain softening; embankment; reinforced soil; numerical modelling; time-dependent behaviour, rate-sensitive clay.

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Introduction

In many parts of eastern Canada and Scandinavia, soft clay deposits are highly sensitive. For these sensitive/structured soils, deformation and plastic strain will break down inter-particle bonding and results in a post peak strength reduction which may significantly affect the behaviour of soft clay deposits (Vaid et al. 1979; Quigley 1980; Leroueil and Vaughan 1990; Burland 1990; Torrance 1999; Malandraki and Toll 2000; Lo and Hinchberger 2006; and Hinchberger and Qu 2009). A number of constitutive models have been proposed for structured and rate-sensitive clays (e.g., Kim and Leroueil 2001; Rocchi et al. 2003; and Hinchberger and Qu 2009).

Rowe and Hinchberger (1998) proposed an elasto-viscoplastic constitutive model based on the concept of over stress viscoplasticity (Perzyna 1963). The model has been extensively verified and provides good prediction for the behaviour of field test embankments constructed on non-structured and rate-sensitive foundation soils (Hinchberger 1996; Hinchberger and Rowe 1998; and Rowe and Hinchberger 1998). Hinchberger and Qu (2009) extended the Rowe and Hinchberger (1998) model using the concept of state-dependent fluidity parameters and a damage law to describe the destructuration of rate-sensitive structured clay. Hinchberger and Qu (2009) also demonstrated that the proposed model can adequately describe many features of soil behaviour such as accelerated creep rupture, post-peak strength reduction as well as the strain rate dependency of the undrained shear strength and the apparent preconsolidation pressure of the Saint Jean Vianney clay in the laboratory. Despite good predictions of the laboratory results, the model has never been tested against the full scale test data.

In this study, the Hinchberger and Qu (2009) structured elasto-viscoplastic soil model is implemented into the finite element program; AFENA (Carter and Balaam 1990) for two-
dimensional (2D) plane strain analysis. The model is used to simulate the performance of a well
documented case study of the reinforced test embankment constructed on sensitive Champlain
clay deposit in Saint Alban, Quebec (Busbridge et al. 1985). The calculated results are compared
with the observed field data and those predicted using the non-structured elasto-viscoplastic soil
model [i.e. Rowe and Hinchberger (1998)]. The structured elasto-viscoplastic soil model is then
employed to investigate the effect of reinforcement and its viscosity on the short-term behaviour
of the reinforced embankment examined in this study. The practical implications as well as the
effectiveness and limitations of the model are also discussed.

**Ground condition at Saint Alban**

The subsurface conditions across the site area where the embankment under consideration was
constructed are inferred from a detailed geotechnical investigation reported by Trak et al. (1980),
Leroueil et al. (1983), Tavenas et al. (1983), Lefebvre et al. (1988) and Lefebvre and Pfendler
(1996). The soil profile consists of a 2.0 m thick weathered clay crust underlain by a 13.7 m
thick deposit of soft grey blue marine clay. Beneath the clay there is a layer of dense fine to
medium coarse sand underlain by bedrock. The groundwater table is at 0.7 m below the ground
surface. Studies of this soft clay indicated that the deposit has low to medium plasticity, with
measured water contents appreciably higher than the liquid limit. The clay in the crust is lightly
overconsolidated, with an overconsolidation ratio (OCR) of about 2.2. The bulk unit weight of the
soil in the crust is estimated to be 19 kN/m³, whereas that of the soil below the crust is 16 kN/m³.
Figure 1 presents a typical profile of the geotechnical properties of the soft deposit – modified
from Trak et al. (1980).
The permeability (hydraulic conductivity) of Champlain clays ranges between $10^{-10}$ m/s and $10^{-8}$ m/s depending on the void ratio (Tavenas et al. 1983). However, at Saint Alban, the deposit exhibits a reducing clay fraction and decreasing plasticity with depth as the soil progressively changes from a clay into a silty material; as a result, the effect of reducing void ratio is compensated by changes in void shape and tortuosity so that the in-situ permeability is nearly constant with both depth and void ratio with a value of about $4 \times 10^{-9}$ m/s (Tavenas et al. 1983). Based on Tavenas et al. (1983) the ratio of horizontal to vertical hydraulic conductivity is very close to unity.

**Geometry, construction and modelling of the test embankment**

A plan view and typical cross section of the reinforced embankment are shown in Figs. 2a and 2b, respectively. According to Busbridge et al. (1985), after the subsurface instrumentation (including piezometers, settlement plates, vertical extensometers, and inclinometers) was installed and allowed to stabilize; embankment fill material was placed with the construction rate of 0.6 m/day. The embankment fill material was a uniform medium to coarse sand with a friction angle of about 34°. The unit weight of the fill material was measured by means of a portable nuclear density test apparatus with the average of 16.9 kN/m$^3$. The embankment side slopes were maintained at a gradient of 1.5:1 (horizontal:vertical). Three side slopes of the embankment were stabilized with the extra 1.5 m high berm so that any failure was forced to the side where the instrumentation was concentrated. A high density polyethylene geogrid, Tensar SR-2, was selected for the basal reinforcement. The tensile strength of the geogrid at high strain rates is 79 kN/m-width. Two horizontal layers of the geogrid were incorporated at the base of the embankment. The lower layer was placed directly at the ground surface and the upper layer was
placed inside the fill material at elevation of 1.5 m above the ground surface. Figure 2a and 2b only show the location of instrumentation used to obtain the data that will be compared with the results from numerical analyses—full details of instrumentation are given by Busbridge et al. (1985). The reinforced embankment failed at a height of about 6.1 m, 10 days after the start of the construction.

The finite element mesh used to model the embankment and foundation soil is shown in Fig. 3. The far field boundaries were assumed to be smooth/rigid boundaries. The bottom boundary (the sand) was assumed to be rough/rigid with free drainage. The finite element mesh consisted of 3386 of six-noded triangle elements (6121 nodes) to model layers of soft clay deposits and embankment fill materials. The geogrid reinforcement was modeled using two-noded bar elements. Two-noded rigid-perfectly plastic interface elements proposed by Rowe and Soderman (1985) were used to model the fill/reinforcement and fill/foundation interfaces. A small strain finite element analysis was performed.

**Constitutive model for rate-sensitive structured clay and material parameters**

The following provides a brief summary of the model used in this study. Full details regarding the derivation of the constitutive model, the state dependent fluidity parameters concept and the damage law are given by Hinchberger and Qu (2009) and Qu (2008).

**Overstress elasto-viscoplasticity**

The Hinchberger and Qu (2009) model is fully coupled with Biot’s (1941) consolidation theory and incorporates Perzyna’s (1963) theory of overstress viscoplasticity. The derivation of the
model is based on an elliptical yield cap model (Chen and Mizuno 1990), a Drucker-Prager failure envelope and concepts drawn from critical state soil mechanics. According to Perzyna’s (1963) overstress theory of viscoplasticity, the governing equation can be expressed in terms of strain-rate tensor:

$$\dot{\epsilon}_{ij} = \frac{\dot{S}_{ij}}{2G} + \frac{1}{3K} \sigma_{ij} + \gamma^{vp}(\varepsilon_{ij}) \left(\phi(F)\right) \frac{\partial f}{\partial \sigma_{ij}}$$

(1)

where $S_{ij}$ is deviatoric stress; $G$ is shear modulus; $\sigma_{ii}$ is summation of the principal stresses; $K$ is bulk modulus; $\gamma^{vp}(\varepsilon_{ij})$ is the state-dependent viscoplastic fluidity parameter (to be discussed in the following subsection) and $\phi(F)$ is a flow function that can be expressed in term of overstress as:

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

(2)

where $\sigma_{my}^{(d)}$ is overstress, defined as the distance between dynamic and static yield surface at the current stress state (Rowe and Hinchberger 1998); $n$ is strain rate exponent.

In the normally consolidated clay, the general equation of elliptical yield surface in $\sigma'_m - \sqrt{2} J_2$ space can be expressed as:

$$f = (\sigma'_m - l)^2 + 2J_2R^2 - (\sigma'_{my} - l)^2 = 0$$

(3)

where $l$ is a mean effective stress corresponding to the center of the ellipse; $R$ is the ratio between major and minor axis of the ellipse; and $\sigma'_{my}$ is the intercept of the ellipse with the $\sigma'_m$ axis. Drucker-Prager failure criterion having a slope of $M_{N/C}$ and $M_{O/C}$ governs the failure of the model for the normally and overly consolidated clay, respectively.

**State-dependent fluidity concept and the damage law**
The state-dependent fluidity concept introduces a new parameter, $\omega_o$, to mathematically define the structure of the soil (Hinchberger and Qu 2009).

$$\omega_o = \left( \frac{\gamma_{i}^{vp}}{\gamma_{v}^{vp}} \right)^{1/n}$$  \hspace{1cm} (4)

where; $\gamma_{v}^{vp}$ is fluidity of the undisturbed clay fabric, $\gamma_{i}^{vp}$ is fluidity of the destructed clay fabric and $n$ is strain rate exponent. Next, the concept of the damage strain, $\varepsilon_d$, (Rouainia and Wood 2000) is employed to define the transition from an initially highly viscous state (structured state) to a more fluid destructured state. The damage strain is expressed as:

$$d\varepsilon_d = \sqrt{(1-A)(d\varepsilon_{vol}^{vp})^2 + A(d\varepsilon_{s}^{vp})^2}$$  \hspace{1cm} (5)

where; $d\varepsilon_d$ is the incremental damage strain, $d\varepsilon_{vol}^{vp}$ and $d\varepsilon_{s}^{vp}$ are the plastic volumetric and plastic shear strain, respectively. $A$ is a weighting parameter, which is assumed to be 0.5 similar to Baudet and Stellebras (2004). Finally, the exponential damage law is introduced to describe rate of soil structure degradation (Hinchberger and Qu 2009), expressed as:

$$\omega(\varepsilon_d) = \left[ 1 + (\omega_o^{*} - 1) \exp(-\alpha \cdot \varepsilon_d) \right]^{1/n}$$  \hspace{1cm} (6)

where; $\alpha$ is a material parameter governing the rate of destructuration, $\varepsilon_d$ is a damage strain, $\omega_o^{*}$ defines the initial structure and $\omega(\varepsilon_d)$ describes the state-dependent structure level.

Accordingly, the fluidity parameter is a function of damage strains as given by:

$$\gamma_{i}^{vp}(\varepsilon_d) = \frac{\gamma_{v}^{vp}}{\omega^{*}(\varepsilon_d)}$$  \hspace{1cm} (7)

where; $\gamma_{i}^{vp}(\varepsilon_d)$ defines state-dependent fluidity of the clay fabric. Thus the viscoplastic strain-rate tensors can be expressed as:


For a structured soil, the initial high viscosity of the soil structure restrains the plastic strain that can be developed and allows overstress to be built up relative to the static yield surface (destructured/remolded state). However, with increasing damage plastic strain, the viscosity of the soil decreases (increasing the structural fluidity) to simulate the breaking down of the bond between soil particles. As a result, soil strength decreases and eventually reaches a completely destructured state strength (the critical state).

The change in hydraulic conductivity of soft clay during loading is taken to be a function of current void ratio (Taylor 1942) as:

\[
\kappa_v = \kappa_{vo} \exp\left(\frac{e - e_o}{C_k}\right) \tag{9}
\]

where; \(\kappa_{vo}\) is the initial in-situ hydraulic conductivity assumed as \(4 \times 10^{-9}\) m/s (Tavenas et al. 1983); \(e_o\) is the initial void ratio as estimated from the soil profile and \(C_k\) is hydraulic conductivity change index. The value of \(C_k = 0.22e_o\) and a ratio of horizontal to vertical hydraulic conductivity of unity: (i.e., \(k_h/k_v = 1\)) for Saint Alban clay were selected based on the literature (Tavenas et al. 1983).

**Embankment fill, reinforcement and interfaces parameters**

The granular soil used for the embankment fill had a unit weight \(\gamma = 16.9\) kN/m\(^3\) (Busbridge et al. 1985). The nonlinear elastic behaviour of the fill was modelled using Janbu’s (1963) equation:

\[
\frac{E}{P_a} = K_s \left(\frac{\sigma_3}{P_a}\right)^m \tag{10}
\]
where $E$ is the Young’s modulus; $P_a$ is the atmospheric pressure; $\sigma_3$ is the minor principal stress and $K_s$ and $m$ are material constants selected to be 300 and 0.5, respectively (Rowe and Hinchberger 1998). The yielding of the sand fill was model using the Mohr-Coulomb failure criterion with a friction angle $\phi' = 34^\circ$, and the plastic flow was governed by non-associated flow rule with dilatancy angle $\psi = 6^\circ$.

Two types of constitutive model (i) the elastic bar elements and (ii) the nonlinear viscoelastic bar element (Zhang and Moore 1997) were used to model the short-term performances of reinforcement in this study. The axial tensile stiffness, $J = 300$ kN/m, of the elastic bar element was determined from the isochronous load strain curves as shown in Fig. 4 (Busbridge et al. 1985).

The governing equation for the nonlinear viscoelastic bar element (i.e. multi-Kelvin elements model) used in this study can be expressed in terms of strain rate as:

$$
\dot{\varepsilon} = \frac{\sigma_T}{a_o \exp\left(-a_i \sigma_T^3\right)} + \sum_{i}^{n_k} \left( \frac{\sigma_T}{E_i \tau_i} - \frac{\varepsilon_i^v}{\tau_i} \right)
$$

where; $\sigma_T$ is tensile stress of the independent spring; $a_o$ and $a_i$ are material constants; $n_k$ is the number of Kelvin elements; $\tau_i = \eta_i / E_i$ is the retardation time; $E_i$ and $\eta_i$ are the spring modulus (stiffness of geosynthetic reinforcement) and the dashpot viscosity of the $i^{th}$ Kelvin element, respectively. The following equations are proposed to reduce the number of material constants (Zhang and Moore 1997):

$$
E_i = \delta^{i+1} E_i \quad \text{and} \quad \tau_i = \beta^{i+1} \tau_i
$$
where; $E_i$ and $\tau_i$ are the material constants.

The required seven material constants are $a_o$, $a_i$, $\delta$, $\beta$, $E_i$, $\tau_i$, and $n_k$. The constitutive parameters for the nonlinear viscoelastic reinforcement used examined in this study (i.e. HDPE and PET geogrids) were selected based on values given in the literature for the same products (Li and Rowe 2001). All parameters are also presented in Table 1.

The rigid-plastic joint elements (Rowe and Soderman 1985) used to model the fill/reinforcement interfaces were assumed to be frictional with $\phi' = 34^\circ$.

### Selection of foundation soil parameters

The basic soil parameters such as initial void ratio, water content, unit weight and current states of stress were obtained from the soil profile presented in Fig. 1. Poisson’s ratio was assumed to be constant for the clay. A value of 0.3 was used based on Tavenas et al. (1974). The critical state parameters (e.g., $\lambda$ and $\kappa$) – used to define the hardening rule in model – were selected based on the recommendation of Zdravkovic et al. (2002). The estimated coefficient of earth pressure at rest, $K_o$, for a normally consolidated material was taken to be 0.49 based on the established limit-state curves for undisturbed samples (Tavenas et al. 1978; and Zdravkovic et al. 2002). For the overconsolidated crust, the corresponding $K_o$ profile was calculated from the Mayne and Kulhawy (1982) formula ($K_{oO/C} = K_{oN/C} \times OCR^{n_p}$).

Specific parameters such as fluidity parameters of the soil, degree of soil structures and rate of soil structure degradation with respect to accumulated plastic strain were calibrated using experimental results. Figure 5 shows the effect of strain rate on the apparent preconsolidation pressure for Saint Alban clay (Leroueil et al. 1988). The strain rate exponent ($n$) was established from the reciprocal of slope of log ($\sigma'_p$) – log ($\dot{\varepsilon}_p$) relationship (Qu et al. 2010). In addition, the
relationship between apparent preconsolidation pressure and strain rate can also be used to estimate the fluidity of the undisturbed clay fabric ($\gamma_s^{vp}$). Qu et al. (2010) derived equations based on strain rate controlled testing and showed that $\gamma_s^{vp} = \sqrt{3/5} \dot{\varepsilon}_m^t$; where $\dot{\varepsilon}_m^t$ is a threshold strain rate which divides soil behaviour between rate-insensitive and rate-sensitive. If soil is subjected to any strain rates faster than the threshold limit, the strain rate effect will be mobilized. However, for Saint Alban clay, the foundation soil still exhibits the effect of strain rate-sensitivity even at strain rates as low as $6 \times 10^{-9}$/min. Qu et al. (2010) suggested that, in these cases, the threshold strain rate of $6 \times 10^{-9}$/min would be adequate to account for the effect of strain rate-sensitivity characteristic over first 25-30 years. Therefore, a threshold strain rate of $6 \times 10^{-9}$/min was assumed in this study and the corresponding fluidity of the undisturbed clay fabric ($\gamma_s^{vp}$) was estimated to be $\gamma_s^{vp} = 1.3 \times 10^{-9}$/min.

The structure parameter ($\omega_s$) can be estimated from either (i) peak versus remolded undrained shear strength or (ii) from intrinsic versus structured preconsolidation pressure (Qu 2008). In this study, the former approach was employed. Figure 6 shows the stress-strain curves of the unconsolidated undrained test (La Rochelle et al. 1974). From Fig. 6, the structure parameter was estimated to be $\omega_s = \frac{s_u{\text{peak}}}{s_u{\text{remolded}}} = 1.35$. The fluidity of the destructed clay fabric ($\gamma_i^{vp}$) was calculated using Equation 4. Equation 5 was then used to calculate the magnitude of strain at which the intrinsic state is reached (i.e., refer to Fig. 6). Lastly, from Equation 6, the constitutive parameter $\alpha$ governing the rate of destructuration was estimated. All the constitutive soil parameters are summarized in Table 2. The stress-strain behaviour predicted by axisymmetric finite element analysis using state-dependent viscoplastic parameter model
Comparison of calculated and measured responses

The results from the 2D plane strain finite element analyses are compared with field measurements for the geogrid reinforced test embankment at Saint Alban (Busbridge et al. 1985), to allow an evaluation of the extended elasto-viscoplastic constitutive model. Due to the highly anisotropic nature of Saint Alban clay, the shape of yield surface used in the analyses was selected to match the soil yield surface and the model yield surface for the applied stress path associated with vertical loading. Figure 7 shows a comparison of the soil yield surface and the modeled yield surface together with the calculated stress path that the soil experienced at location A during construction (see insert in Fig. 7). As demonstrated in Fig. 7, the implemented yield surface matched the in-situ yield surface well for the stress range that the foundation soil experienced (i.e., above the $K_o$ line) beneath the crest of the embankment.

To examine the effect of incorporating the soil structure into the model for this particular case study, analyses also were performed using the original elasto-viscoplastic model (Rowe and Hinchberger 1998) for comparison with those obtained from the extended elasto-viscoplastic model (Hinchberger and Qu, 2009). To evaluate the beneficial effect of geosynthetic reinforcement used in this study, analyses were performed using the structured elasto-viscoplastic soil model assuming (i) elastic reinforcement (with an axial tensile stiffness, $J = 300$ kN/m, determined from the isochronous load strain curves as shown in Fig. 4), (ii) considering the viscoelastic properties of the high density polyethylene (HDPE) reinforcement actually used to illustrate the effect of reinforcement...
viscosity on short-term stability, (iii) without the use of reinforcement and (iv) with stiffer and less creep susceptible polyester (PET) reinforcement with viscoelastic constitutive parameters, given in Table 1 (based on Li and Rowe 2001).

Structured versus Non-structured elasto-viscoplastic soil model

The benefit of incorporating effect of soil structure and its destructuration into the model is illustrated in Figs. 8 and 9. The original elasto-viscoplastic soil model (Rowe and Hinchberger 1998) underestimates vertical settlement at the centerline (Fig. 8) and excess pore water beneath the embankment shoulder (Fig. 9). Moreover, there is no failure of the simulated embankment, even after constructed up to 6.3 m. This is because Rowe and Hinchberger (1998) model cannot capture the de-structuring of the Saint Alban soil. Therefore, soil strength and stiffness remain almost unchanged after yielding which results in smaller deformations.

Because the non-structured elasto-viscoplastic soil model underestimates the overall deformation of the soils; in Fig. 10a and 10b, the prediction of horizontal deformations near the embankment toe (IN-1 and IN-2) seems better than those predicted using structured elasto-viscoplastic soil model. Since the non-structured elasto-viscoplastic soil model could not capture the vertical response or pore pressure development for the St Alban soil as well as the structured model, the discussion of the effect of reinforcement below will focus on the results from the structured elasto-viscoplastic soil model (Hinchberger and Qu, 2009).

Failure height and vertical settlement

Assuming the properties of either the elastic reinforcement or the stiffer viscoelastic PET reinforcement, the calculated failure height of the test embankment was 6.0 m, which is close to
the observed failure height of about 6.1 m. The numerical analyses using viscoelastic HDPE
reinforcement as well as those with no reinforcement both gave slightly smaller failure heights of
5.9 m. This suggests that for this particular soil the geogrid reinforcement had very little effect
on embankment performance. Although there was a slight difference in the calculated failure
height due to different types of reinforcement, the calculated settlements at the center line were
almost identical. The calculated and observed settlements at the centerline (SP-9) showed good
agreement (Fig. 8) with the in-situ measurement – for all types of reinforcement modeled – until
the fill thickness reached about 2.4 m (vertical stress of about 40 kPa) where there was a rapid
change in the load-settlement curve indicating yielding of the foundation soil. From this point,
the finite element analyses tend to underestimate the centerline settlement. This is due to the fact
that the high initial viscosity of the soil modeled prevents deformations at the beginning stage of
loading. However, the differences are modest and the overall trend was well captured. The final
settlement at the centerline was also well predicted. The final settlements measured just before
failure were 0.23 m and 0.24 m for the field measurement and finite element analyses,
respectively.

Excess pore water pressure

The data from the piezometer which showed the maximum response, PN-15 (Fig. 2) was used to
illustrate the buildup of pore water pressure under the reinforced embankment. The relationship
between the excess pore water pressure and the increase in vertical total stress (Fig. 9) showed
that the model tended to overestimate the excess pore water pressure at the beginning of loading
up to the fill thickness of about 3.9 m (i.e. the increase of vertical stress of about 65 kPa). This
might be due to the fact that the high initial viscosity of the model (i.e. soil structure) restrains
soil movement as discussed earlier. Consequently, it delayed the soil consolidation and hence reduced the rate of excess pore water dissipation at early time compared with the observed field behaviour.

In the field, as the total vertical stress increased beyond about 60 kPa, the slope of the observed pore pressure response significantly increased compared with at the earlier stage of loading. The slope of applied stress against excess pore pressure relationship exceeded unity suggesting that destructuring/collapsing of the clay fabric. This caused the rapid increase in excess pore water pressure at a rate which exceeded the rate of consolidation during this period up to failure. The model captured some, but not all, of this change because the viscosity of the model still prevented the rapid deformation (collapse) of the soil. As a result, the field excess pore pressure response rose above the calculated values at the later stage of loading. Despite some limitations, the agreement between the field measurement and the predicted excess pore water pressure is still considered reasonable.

Figure 9 shows that the predicted excess pore water pressures were essentially the same for all four analyses (i.e. no reinforcement, elastic reinforcement, viscoelastic HDPE reinforcement, and the stiffer viscoelastic PET reinforcement) implying that neither the particular geogrid reinforcement that was used nor other commonly used reinforcement would significantly contribute to improving the stability of the embankment examined in this study.

**Horizontal deformation**

The shape of the deformed inclinometer casings at IN-1 and IN-2 (Fig. 2), corresponding with the centerline surcharge about 77 kPa (Busbridge et al. 1985), are presented together with the calculated results from finite element analyses in Fig. 10a and 10b, respectively. In all cases, the
calculated horizontal deformation profiles are significantly greater than the observed values. This is consistent with previous published experience using small strain analysis where numerical methods have tended to overestimate horizontal deformations (Poulos 1972; Tavenas et al. 1979; Rowe et al. 1996; and Hinchberger and Rowe 1998). The difference between calculated and measured horizontal deformation may be caused by the combined effect of significant rotation of principal stress under the embankment slope and the highly anisotropic characteristics of the foundation soil.

The calculated rate of increase in horizontal deformation (Fig. 11) accelerated when the centerline surcharge pressure exceeded about 77 kPa, which agrees well with what was observed (Busbridge et al. 1985). The results presented in Fig. 10a, 10b and 11 show some slight differences in the prediction of horizontal deformation for all cases. However, the differences are practically insignificant.

Performance of geogrid reinforcement

According to the summary report (Busbridge et al. 1985), the load and strain mobilized in the reinforcement were relatively small during construction and up to failure. The maximum loads measured in the geogrid under the crest of the reinforced embankment were 9.1 and 6.4 kN/m at the lower and upper level of reinforcement, respectively. The maximum reinforcement loads calculated at the same location using finite element analysis were 9.8 and 8.3 kN/m at the lower and the upper level of reinforcement, respectively. Figure 12 shows the development of the reinforcement loads and strain with the vertical stress at the centerline of the embankment. During the construction of the embankment, only low reinforcement loads were observed in the field for both the lower and upper layer of geogrid which is consistent with the calculations from
the finite element analyses. However the field report indicated that there was a rapid straining
just before failure and the back-calculated reinforcement rupture load exceeded 45 kN/m and
probably approached 60 kN/m (Busbridge et al. 1985). The rapid increase in load was likely
associated with major destructuring, and consequent strength loss, of this highly sensitive clay at
failure. The reinforcement was not sufficient to sustain the loads when the foundation soil failed
and likely tore. The constitutive model examined here indicated the onset of failure but was not
able to capture the behaviour during failure.

Conclusions

The results from finite element analyses conducted using elasto-viscoplastic constitutive
models with/without incorporating the state-dependent fluidity parameter concept and damage
law were compared with field observations for the reinforced test embankment constructed on
sensitive Champlain clay deposit at Saint Alban, Quebec. The structured elasto-viscoplastic
model (Hinchberger and Qu 2009) was shown to better capture many aspects of the embankment
performance compared to the original elasto-viscoplastic model. This was because the original
elasto-viscoplastic soil model could not capture the effect of soil structure and the destructuration
process. As a result, the model could not simulate the strain softening behaviour of the Saint
Alban clay. Consequently, it tended to underestimates deformations of the Saint Alban clay,
examined in this study. The following conclusions relate to the predictions of reinforced
embankment behavior obtained using the structured elasto-viscoplastic soil model.

The vertical settlement at the centerline of the embankment was well predicted during the
early stages of loading (up to 2.4 m of fill thickness). As the load increased, the model tended to
somewhat underestimate the vertical settlement at the centerline. However, the trend and the
final vertical settlement just before failure were in good agreement.

The calculated excess pore water pressure response overestimated the field measurement
at the early stages of construction. As failure was approached, the field response showed a
significant increase in the rate of excess pore water pressure development. This phenomenon and
its consequences were not well captured because the initial modeled soil viscosity used to
simulate the effect of soil structure prevented the rapid decrease in void ratio and hence
consolidation of the soil. This is a fundamental limitation of this particular model. Even though
this use of soil viscosity to model de-structuring had some limitations, the proposed model was
still able to provide reasonable estimates of the excess pore water pressure response.

The finite element analyses over-predicted the horizontal deformation profiles. The
difference between calculated and measured horizontal deformation might cause by the
combined effect of significant rotation of principal stress under the embankment slope combined
with the anisotropic characteristics with respect to strength and stiffness—of the foundation soil.
The analyses, however, did indicate the point at which the rate of horizontal toe movement
started to accelerate. To provide adequate predictions of the horizontal deformation where there
is significant rotation of principal stress, the effects of strength and stiffness anisotropy need to
be addressed in the constitutive model. With respect to the response of reinforcement, the
structured elasto-viscoplastic soil model provided good agreement with the observed field data in
terms of reinforcement load and strain prior to failure.

The effect of reinforcement was explored numerically and it was found that the
calculated response with and without the geogrid reinforcement and with stiffer and less creep
susceptible PET reinforcement were almost identical giving only minor differences. It is
concluded that for this particular embankment being studied (i.e. constructed on highly sensitive clay with a heavily overconsolidated crust at a very fast construction rate) the geogrid reinforcement use had no significant beneficial effect in terms of redistributing shear stress in foundation soil. This is likely because the reinforcement was not sufficiently stiff relative to the overconsolidated crust to play any significant role prior to the onset of foundation failure and not strong enough to control the failure once failure of the foundation soil was initiated.

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References


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### Table 1. Nonlinear viscoelastic model parameter for reinforcement (Li and Rowe 2001)

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<thead>
<tr>
<th>Reinforcement material</th>
<th>Material constants</th>
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<tr>
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<td>$a_o$ (kN/m)</td>
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<td>HDPE</td>
<td>1050</td>
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<td>PET</td>
<td>1800</td>
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### Table 2. Elliptical cap soil model parameters

<table>
<thead>
<tr>
<th>Soil Parameter</th>
<th>Value</th>
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<tbody>
<tr>
<td>Failure envelope $M_{N/C}$</td>
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</tr>
<tr>
<td>Failure envelope $M_{O/C}$</td>
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<tr>
<td>Aspect ratio $R$</td>
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<tr>
<td>Compression index $\lambda$</td>
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<tr>
<td>Recompression index $K$</td>
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<tr>
<td>Coefficient of earth pressure at rest $K_o$</td>
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<tr>
<td>Poisson’s ratio $v$</td>
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<tr>
<td>Hydraulic conductivity $k_v_o (m/s)$</td>
<td>$4 \times 10^{-9}$</td>
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<tr>
<td>Intrinsic viscoplastic fluidity $i_{vp} (hr^{-1})$</td>
<td>$7 \times 10^{-4}$</td>
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<tr>
<td>Strain rate exponent $n$</td>
<td>24.4</td>
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<tr>
<td>Structure parameter $\omega_o$</td>
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<tr>
<td>Parameter controlling rate of destructuration $\alpha$</td>
<td>174</td>
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Figure 1. Typical soil profiles of the test site at Saint Alban, Quebec (modified from Trak et al. 1980)
Figure 2. Plan view and cross sectional profile of reinforced test embankment (modified from Busbridge et al. 1985)
Figure 3. Finite element mesh for the reinforced test embankment examined
Figure 4. Isochronous load strain curves of Tensar SR2 geogrids (modified from Busbridge et al. 1985)
Figure 5. Effect of strain rate on the preconsolidation pressure of the Saint Alban clay (modified from Leroueil et al. 1988)
Figure 6. Results from the unconsolidated undrained compression test on Saint Alban clay and calculated value using the parameters adopted in this paper (experimental data from La Rochelle et al. 1974)
Figure 7. Comparison between experimental and implemented yield surface and stress path of the soil under centerline of the reinforced embankment (experimental data from Tavenas et al. 1974)
Figure 8. Vertical settlement of the reinforced embankment at settlement plate SP-9 (field data from Busbridge et al. 1985)
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Figure 11. Calculated horizontal toe movement
Figure 12. Calculated reinforcement load under embankment crest versus applied pressure at embankment centerline.