

A viscoplastic SANICLAY model for natural soft soils

Mohammad Rezaia^a, Mahdi Taiebat^{b,c,*}, and Elisa Poletti^d

^aDepartment of Civil Engineering, University of Nottingham, Nottingham, UK

^bDepartment of Civil Engineering, University of British Columbia, Vancouver, BC, Canada

^cDepartment of Civil & Environmental Engineering, Massachusetts Institute of Technology,
Cambridge, MA, USA

^dDepartment of Civil Engineering, University of Minho, Azurém, Guimarães, Portugal

Abstract

This paper focuses on constitutive and numerical modeling of strain-rate dependency in natural clays while also accounting for anisotropy and destructuration. For this purpose the SANICLAY model that accounts for the fabric anisotropy with the additional destructuration feature that accounts for sensitivity of natural clays, is considered as the reference model. An associated flow rule is adopted for simplicity. The model formulation is refined to also account for the important feature of strain-rate dependency using the Perzyna's overstress theory. The model is then implicitly integrated in finite element program PLAXIS. Performance of the developed and implemented model is explored by comparing the simulation results of several element tests and a boundary value problem to the available experimental data. The element tests include the constant strain-rate under one-dimensional and triaxial conditions on different clays. The boundary value problem includes a test embankment, namely embankment D constructed at Saint Alban, Quebec. For comparison, the test embankment is also analysed using the Modified Cam-Clay (MCC) model, the SANICLAY model, and the viscoplastic model but without destructuration. Results demonstrate the success of the developed and implemented viscoplastic SANICLAY in reproducing the strain-rate dependent behavior of natural soft soils.

Keywords: viscoplasticity; strain-rate dependency; anisotropy; destructuration; clay

*Corresponding author. Tel.: +1 604 822 3279.

E-mail addresses: mohammad.rezania@nottingham.ac.uk (M. Rezaia), mtaiebat@civil.ubc.ca (M. Taiebat), elisapoletti@civil.uminho.pt (E. Poletti).

25 1 Introduction

26 Modeling the stress-strain response of natural soft soils constitutes a challenge in practical
27 geotechnical engineering; it is governed by a series of factors that are not always included in conventional
28 constitutive models. In particular, the three main inherent features that influence their response are a)
29 anisotropy, b) destructuration (degradation of the inter-particle bonds), and c) strain-rate dependency.

30 Since modeling the full anisotropy of natural clay behavior is not practical due to the number of
31 parameters involved, efforts have been mainly focused on development of models with reduced number of
32 parameters while maintaining the capacity of the model [1]. Historically, for practical model development
33 purposes, the initial orientation of soil fabric is considered to be of cross-anisotropic nature, which is a
34 realistic assumption as natural soils have been generally deposited only one-dimensionally in a vertical
35 direction. It is also a well-established fact that the yield surfaces obtained from experimental tests on
36 undisturbed samples of natural clays are inclined in the stress space due to the inherent fabric anisotropy
37 in the clay structure (e.g., [2-4]). Based on the above, a particular line of thought has become popular in
38 capturing the effects of anisotropy on clayey soil behavior, by development of elasto-plastic constitutive
39 models involving an inclined yield surface that is either fixed (e.g., [2]), or can changed its inclination by
40 adopting a rotational hardening (RH) law in order to simulate the development or erasure of anisotropy
41 during plastic straining (e.g., [5-6]). For obvious reasons a model accounting for both inherent and
42 evolving anisotropy would be more representative of the true nature of response in clays; hence, since the
43 first proposal of such model by Dafalias [5-6] similar framework has been adopted by a number of other
44 researchers for development of anisotropic elasto-plastic constitutive models (e.g., [7-11]). Based on the
45 original model, Dafalias et al. [12] proposed what they called SANICLAY model, altering the original RH
46 law and introducing a non-associated flow rule. A destructuration theory was later applied to the
47 SANICLAY model [13] to account for both isotropic and frictional destructuration processes. In these
48 works, the SANICLAY has been shown to provide successful simulation of both undrained and drained
49 rate-independent behaviour of normally consolidated sensitive clays, and to a satisfactory degree of
50 accuracy of overconsolidated clays.

51 Past experimental studies have also shown that soft soils exhibit time-dependent response (e.g., [14-
52 17]). Time-dependency is usually related to the soil viscosity that could lead to particular effects such as
53 creep, stress relaxation, and strain-rate dependency of response. Time-dependency of soil response can be
54 observed experimentally by means of creep tests, stress relaxation tests, or constant rate of strain (CRS)
55 tests [18]. Rate-sensitivity is a particular aspect of time effect that has been investigated extensively; it

56 influences both strength and stiffness of soils. Various studies using CRS tests have shown how faster
57 strain rates for a certain strain level lead to higher effective stresses; also, the general observation,
58 particularly in soft soils, is that higher undrained strengths can be achieved by increasing the loading rate
59 (e.g., [16-17,19-20]). The reported observations from laboratory studies all imply that consideration of soil
60 viscosity effects could be key for correct prediction of long term deformations in field conditions; although,
61 neglecting soil viscosity seemingly provide sufficiently correct predictions in short-term [21]. Landslides or
62 long-term deformations of tunnels and embankments on soft soils are examples of common practical
63 problems where a sustainable remediation and/or design solution can only be achieved if time-dependent
64 behavior of soil is taken into consideration.

65 In order to account for the time-dependency of soft clays' behavior, various frameworks can be found
66 in the literature. Among a number of popular frameworks such as the isotache theory of Šuklje [22] or the
67 non-stationary surface theory of Naghdi and Murch [23], the overstress theory of Perzyna [24-25] is a
68 common framework often used in geomechanics for this purpose due to its relative simplicity. The first
69 overstress-type viscoplastic models were based on isotropic Cam-Clay or modified Cam-Clay models (e.g.,
70 [26-32]). More recently, several models accounting for either only the fabric anisotropy (e.g., [33]), or both
71 anisotropy and destructuration [34] have also been introduced. A shortcoming of these models is the
72 absence of bounds for the evolution of rotational hardening variables which could eventually lead to an
73 excessive rotation of the yield surface for loading at very high values of stress-ratio [35-36]. Furthermore,
74 destructuration theories have so far only addressed isotropic destructuration (usually constituting a
75 mechanism of isotropic softening of the yield surface with destructuration), neglecting frictional
76 destructuration.

77 In this paper, a new Elasto-ViscoPlastic Simple ANisotropic CLAY plasticity (EVP-SANICLAY)
78 model is proposed. The model is a new member of the SANICLAY family of models, which are based on
79 the classical modified Cam-Clay model and include rotational hardening and destructuration features for
80 simulation of anisotropy and sensitivity, respectively. Perzyna's overstress theory [24-25] is employed to
81 account for soil viscosity effects. Being based on the SANICALY model, the new viscoplastic model
82 restricts the rotation to within bounds necessary to guarantee the existence of real-valued solutions for
83 the analytical expression of the yield surface [12]. In the following sections, the theoretical formulation of
84 the model will be discussed, followed by the details of its numerical implementation based on an
85 algorithm proposed by Katona [28]. The validation of the new model is done by comparing the model
86 simulation results against several experimental data at the element level and also field measurements for a

87 boundary value problem. In particular, at element level the measured behavior observed from CRS and
 88 undrained triaxial tests over a number of different soft clays are used. Within these examples,
 89 determination of model parameter values is also discussed. For the boundary value problem, a well-
 90 studied test embankment, namely St. Alban embankment, is modeled and the predicted deformations
 91 using the EVP-SANICLAY model are compared with the recorded *in-situ* values. In order to better
 92 highlight the merits of the newly proposed constitutive model, the simulation results are also compared
 93 with those obtained using the MCC model, the SANCILAY model, and also the EVP-SANICLAY model
 94 but without the destructureation feature. Note that in this paper all stress components are effective
 95 stresses and as usual in geomechanics, both stress and strain quantities are assumed positive in
 96 compression.

97

98 **2 EVP-SANICLAY**

99 **2.1 Model formulation**

100 According to Perzyna's theory, the total strain increment, $\Delta\boldsymbol{\varepsilon}$, associated with a change in effective
 101 stress, $\Delta\boldsymbol{\sigma}$, during a time increment of Δt , is additively decomposed to elastic and viscoplastic parts

$$\Delta\boldsymbol{\varepsilon} = \Delta\boldsymbol{\varepsilon}^e + \Delta\boldsymbol{\varepsilon}^{vp} \quad (1)$$

102 where the superscripts *e* and *vp* represent the elastic and the viscoplastic components, respectively. The
 103 elastic strain increment, $\Delta\boldsymbol{\varepsilon}^e$, is time-independent; whereas, the viscoplastic strain increment, $\Delta\boldsymbol{\varepsilon}^{vp}$, is
 104 irreversible and time-dependent. **Adopting the isotropic hypoelastic relations for simplicity [12]**, the elastic
 105 part of the total strain can be shown as

$$\Delta\boldsymbol{\varepsilon}^e = \mathbf{D}^{-1} : \Delta\boldsymbol{\sigma} \quad (2)$$

106 where \mathbf{D} is the elastic stiffness matrix **with more details presented in the Appendix**, and symbol $:$ in
 107 implies the trace of the product of two tensors.

108 The time-dependent viscoplastic strain increment is evaluated as

$$\Delta\boldsymbol{\varepsilon}^{vp} = \dot{\boldsymbol{\varepsilon}}^{vp} \cdot \Delta t \quad (3)$$

109 where $\dot{\boldsymbol{\varepsilon}}^{vp}$ is the viscoplastic strain rate tensor (a superposed dot denotes the time derivative), and
 110 following the original proposal by Perzyna [24-25], it can be defined as

$$\dot{\boldsymbol{\varepsilon}}^{vp} = \mu \cdot \langle \Phi(F) \rangle \cdot \frac{\partial g}{\partial \boldsymbol{\sigma}} \quad (4)$$

111 where μ is referred to as the fluidity parameter, g is the viscoplastic potential function represented by the
 112 dynamic loading surface (DLS - explained in the sequel), and $\Phi(F)$ is the so-called overstress that is the
 113 normalised distance between the current static yield surface (SYS) and the DLS (see Figure 1). The
 114 application of Macauley brackets in Equation (4) ensures that

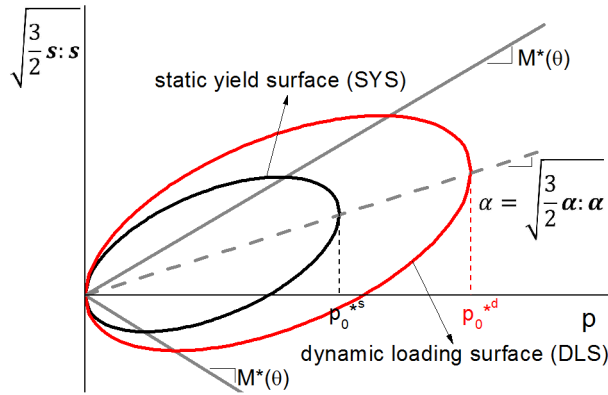
$$\langle \Phi(F) \rangle = \begin{cases} 0 & \text{for } \Phi(F) \leq 0 \\ \Phi(F) & \text{for } \Phi(F) > 0 \end{cases} \quad (5)$$

115 Several different relationships for $\Phi(F)$ have been proposed in the literature (e.g. [26,37]). In this
 116 work the following exponential function proposed by Fodil et al. [38] is employed

$$\Phi(F) = \exp(F) - 1 = \exp \left[N \left(\frac{p_0^d}{p_0^s} - 1 \right) \right] - 1 \quad (6)$$

117 where p_0^s and p_0^d are the size of the SYS and the DLS, respectively (see Figure 1), N is the strain-rate
 118 coefficient that together with μ are the two viscous parameters of this model.

119



120

121 Figure 1. Graphical representation of the EVP-SANICLAY model in the stress space

122

123 This specific choice of $\Phi(F)$ ensures that its value is always greater or equal to zero. Thus, from
 124 Equation (7) it is evident that if the stress state lies on or inside the SYS, the soil response would be
 125 purely elastic. If the stress state lies outside the SYS, viscoplastic strain will be developed proportional to
 126 its distance from the current SYS.

127 In this work the elliptical surface of the SANICLAY model [12] is adopted as the SYS. The
 128 SANICLAY model was originally proposed with a non-associated flow rule; however, for simplicity
 129 purposes an associated flow rule is adopted here for its elastic-viscoplastic extension. In the general stress
 130 space, the SYS function can be expressed as

$$f^s = \frac{3}{2} (\mathbf{s} - p\boldsymbol{\alpha}) : (\mathbf{s} - p\boldsymbol{\alpha}) - \left(M^*(\theta)^2 - \frac{3}{2} \boldsymbol{\alpha} : \boldsymbol{\alpha} \right) (p_0^{*s} - p)p = 0 \quad (7)$$

131 In the above expression, $\mathbf{s} = \boldsymbol{\sigma} - p\mathbf{I}$ is the deviatoric component of stress tensor $\boldsymbol{\sigma}$ (\mathbf{I} being the fourth
132 order identity tensor). $\boldsymbol{\alpha}$ is the deviatoric fabric tensor that accounts for anisotropy by coupling the
133 deviatoric and volumetric plastic strain rates. $p_0^{*s} = S_i p_0^s$ defines the size of the structured SYS where
134 $S_i \geq 1$ is an isotropic destructure factor and p_0^s is the size of the intrinsic SYS. $M^*(\theta) = S_f M(\theta)$ where
135 $S_f \geq 1$ is a frictional destructure factor and $M(\theta)$ is the critical stress-ratio that in the general stress
136 space its value is interpolated between M_c and M_e by means of a Lode angle θ . In the stress space
137 illustrated in [Figure 1](#) the scalar $\alpha = \sqrt{(3/2) \boldsymbol{\alpha} : \boldsymbol{\alpha}}$ defines the rotation of the SYS and DLS. As shown in
138 [Figure 1](#), the DLS has the same shape and orientation as the smaller SYS, and following the adoption of
139 associate flow rule it coincides the viscoplastic potential surface too. **A summary of the hardening
140 equations and the Lode angle formulation are presented in the Appendix for the sake of completeness.**

141 The model constants of EVP-SANICLAY can be divided into 4 categories: (1) the elasticity
142 constants κ and ν and the critical state constants M_c , M_e and λ which are the same as those in the MCC
143 model (with the exception that in MCC we have $M_e = M_c$); (2) the rotational hardening (RH) constants C
144 and x , which are specific to the SANICLAY model; (3) the destructure constants k_i , k_f and A ; and
145 (4) the viscosity parameters N and μ , which constitute the two new additional parameters of the EVP-
146 SANICLAY and they can be determined as discussed in [Yin and Hicher \[31\]](#). Furthermore, similar to the
147 SANICLAY, $\boldsymbol{\alpha}$ and p_0^s constitute the hardening internal variables in the EVP-SANICLAY model. It
148 should be noted that despite the large number of model parameters, they have clear physical meaning and
149 can be determined following straightforward processes. The detailed procedure for evaluating the initial
150 values of the model state variables, and hardening and destructure parameters can be found in
151 [Taiebat et al. \[13\]](#).

152

153 2.2 Numerical Integration

154 The numerical solution algorithm for the elasto-viscoplastic model can be developed by using a step-
155 by-step time integration algorithm with a Newton-Raphson iteration procedure [\[28\]](#). In this scheme it is
156 assumed at the beginning of a certain defined time interval and strain increment, the values of stresses,
157 strains, and state variables are known. The objective is to determine the subsequent elastic and
158 viscoplastic strain components, which in turn allow finding the subsequent stresses and internal variables.
159 From [Equations \(1,2\)](#) the incremental constitutive relationship for a time step can be expressed as

$$\Delta\boldsymbol{\sigma} = D(\Delta\boldsymbol{\varepsilon} - \Delta\boldsymbol{\varepsilon}^{vp}) \tag{8}$$

160 For approximation of $\Delta\boldsymbol{\varepsilon}^{vp}$, a finite difference scheme is employed as:

$$\Delta\boldsymbol{\varepsilon}^{vp} = \Delta t[(1 - \beta)\dot{\boldsymbol{\varepsilon}}_t^{vp} + \beta\dot{\boldsymbol{\varepsilon}}_{t+\Delta t}^{vp}] \quad (9)$$

161 where $\dot{\boldsymbol{\varepsilon}}_t^{vp}$ is the value of viscoplastic strain rate at time t , and β is a time interpolation parameter
 162 ($0 \leq \beta \leq 1$); $\beta = 0$ represents an explicit forward (Euler) interpolation, $\beta = 0.5$ represents central (Crank-
 163 Nicolson) interpolation, and $\beta = 1$ implies an implicit backward interpolation. Lewis and Schrefler [39]
 164 showed that in this scheme the solution is conditionally stable for $0 \leq \beta < 0.5$ and $\beta = 1$, and
 165 unconditionally stable for $0.5 \leq \beta < 1$. Substituting Equation (9) into Equation (8) and rearranging the
 166 terms give:

$$D^{-1}\boldsymbol{\sigma}_{t+\Delta t} + \Delta t \cdot \beta \cdot \dot{\boldsymbol{\varepsilon}}_{t+\Delta t}^{vp} = \Delta\boldsymbol{\varepsilon} - \Delta t \cdot (1 - \beta)\dot{\boldsymbol{\varepsilon}}_t^{vp} + D^{-1}\boldsymbol{\sigma}_t \quad (10)$$

167 where the terms on the right hand side are known (at time t), while the left hand side terms are
 168 unknowns (at time $t + \Delta t$) and they are to be solved in an iterative procedure. A Modified Newton-
 169 Raphson approach is used for the iterative solution of Equation (10). To do this, a limited Taylor series is
 170 applied to the unknown quantities $\boldsymbol{\sigma}_{t+\Delta t}$ and $\dot{\boldsymbol{\varepsilon}}_{t+\Delta t}^{vp}$:

$$\begin{cases} \boldsymbol{\sigma}_{t+\Delta t} = \boldsymbol{\sigma}_i + d\boldsymbol{\sigma}_i & (11a) \\ \dot{\boldsymbol{\varepsilon}}_{t+\Delta t}^{vp} = \dot{\boldsymbol{\varepsilon}}_i^{vp} + \frac{\partial \dot{\boldsymbol{\varepsilon}}_i^{vp}}{\partial \boldsymbol{\sigma}} d\boldsymbol{\sigma}_i & (11b) \end{cases}$$

171
 172 Note that subscript i refers to the i -th iteration at the current time step. Substituting Equations
 173 (11a) and (11b) into Equation (10) and successive rearrangements result in the following form for
 174 computation of stress increment:

$$d\boldsymbol{\sigma}_i = \left[D^{-1} + \Delta t \cdot \beta \cdot \frac{\partial \dot{\boldsymbol{\varepsilon}}_i^{vp}}{\partial \boldsymbol{\sigma}} \right]^{-1} : [(\Delta\boldsymbol{\varepsilon} - \Delta t \cdot (1 - \beta)\dot{\boldsymbol{\varepsilon}}_t^{vp} + D^{-1} \cdot \boldsymbol{\sigma}_t) - (D^{-1}\boldsymbol{\sigma}_i + \Delta t \cdot \beta \cdot \dot{\boldsymbol{\varepsilon}}_i^{vp})] \quad (12)$$

175 If it is assumed that function K represents the term $(\Delta\boldsymbol{\varepsilon} - \Delta t \cdot (1 - \beta)\dot{\boldsymbol{\varepsilon}}_t^{vp} + D^{-1}\boldsymbol{\sigma}_t)$ with known
 176 quantities remaining constant during the iteration, and that function U represents the iterative term
 177 $(D^{-1}\boldsymbol{\sigma}_i + \Delta t \cdot \beta \cdot \dot{\boldsymbol{\varepsilon}}_i^{vp})$, then Equation (12) can be presented in a short form as:

$$d\boldsymbol{\sigma}_i = \left[\frac{\partial U}{\partial \boldsymbol{\sigma}} \right]^{-1} \cdot [K_t - U_i] \quad (13)$$

178 The most efficient solution scheme for continuum problems using overstress-type elasto-viscoplastic
 179 constitutive equations can be obtained with $\beta = 0.5$ [40]; hence, this value is adopted for the time
 180 interpolation parameter in the present work. For the solution algorithm, at every time step Equation (13)
 181 is iteratively solved. At each iteration $d\boldsymbol{\sigma}_i$ is calculated and subsequently $\boldsymbol{\sigma}_i$ is updated as $\boldsymbol{\sigma}_i = \boldsymbol{\sigma}_{i-1} +$
 182 $d\boldsymbol{\sigma}_i$. When convergence is achieved (i.e. when $d\boldsymbol{\sigma}_i < \text{tolerance} \sim 10^{-7}$), the iterative procedure stops and

183 the incrementally accumulated stress values will become the stresses at the corresponding time step (i.e.,
184 $\sigma_{t+\Delta t}$); subsequently, viscoplastic strain tensor can be calculated as $\epsilon_{t+\Delta t}^{vp} = \epsilon_{t+\Delta t} - \mathbf{D}^{-1}\sigma_{t+\Delta t}$. The
185 implementation makes it possible to apply the whole strain increment through a number of sub-
186 increments, not all at once. After the completion of the integration process at a time increment the
187 procedure advances to the next time step.

188 The EVP-SANICLAY model has been implemented into PLAXIS finite element program as a user-
189 defined soil model in order to be used for both element level and boundary value problem simulations. In
190 the following, first the performance of the model is validated by simulation of a number of element test
191 data on various clays. The model is then used for settlement study of a real instrumented test
192 embankment and the simulation results are discussed in detail. The embankment simulation also aims to
193 compare details of the predicted response using the proposed model and also using an isotropic and rate-
194 independent model that is often used in practice.

195

196 **3 Model validation based on element level tests**

197 For the element test simulations the implemented user-defined model has been employed through the
198 PLAXIS Soil Test application [41] to simulate several undrained triaxial shear and CRS test data on four
199 different soft soils reported in the literature, namely Kawasaki clay, Haney clay, St. Herblain clay, and
200 Batiscan clay [14,16,42-43]. The values of model constants and state variables used for the four soil types
201 analysed in this paper are summarised in Tables 1 and 2. In accordance with the natural or reconstituted
202 state of the clay sample being simulated the destructuration feature of the model has been switched on or
203 off by setting respective values to the structuration parameters.

204

205

Table 1. Constants of the EVP-SANICLAY model adopted for four types of clays

Model constant		Kawasaki	Haney	St. Herblain	Batiscan
Elasticity	κ	0.021	0.05	0.022	0.037
	ν	0.2	0.25	0.3	0.3
Critical state	$M_c (M_e)$	1.65 (1.24)	1.28	1.25	0.98
	λ	0.16	0.32	0.41	0.41
Rotational hardening	C	12	12	10	12
	x	2.6	2.4	1.5	1.7
Destructuration	k_i	0	1.5	0	1.4
	k_f	0	1.4	0	1.3
	A	0	0.3	0	0.5
Viscosity	N	12	17	9	12
	$\mu [s^{-1}]$	7×10^{-6}	5×10^{-11}	5×10^{-9}	2×10^{-9}

Table 2. Initial values of state variables adopted for four types of clays

Model state variable		Kawasaki	Haney	St. Herblain	Batiscan
Initial void ratio	e	1.07	2	2.26	1.92
Initial size of the SYS	p_0 [kPa]	250	340	30	50
Initial rotation of the SYS	α	0.60	0.43	0.46	0.36
Initial isotropic structuration factor	S_i	1	6	1	3
Initial frictional structuration factor	S_f	1	1.3	1	1.5

210 3.1 Kawasaki clay

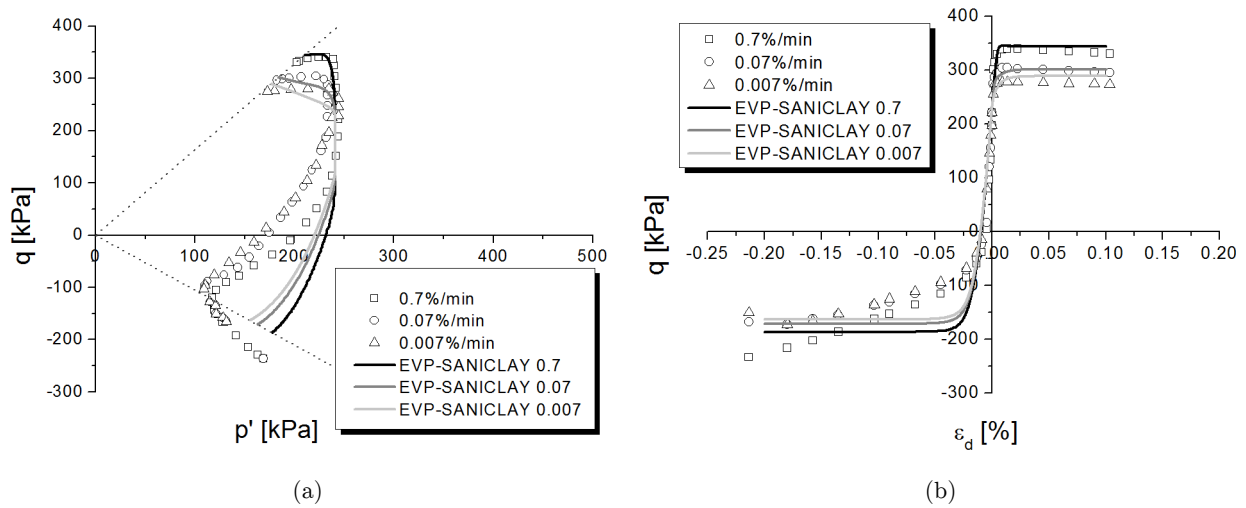
211 To evaluate the strain-rate dependency, Nakase and Kamei [42] performed undrained triaxial
 212 compression and extension tests with various shearing rates on anisotropically consolidated reconstituted
 213 Kawasaki clay specimens. The index properties of Kawasaki clay samples were reported as plasticity index
 214 $I_p = 29.4$, specific gravity $G_s = 2.69$, liquid limit $w_L = 55.3\%$, plastic limit $w_p = 25.9\%$, and clay content
 215 22.3% . All tests were conducted under a vertical effective consolidation pressure of 392 kPa with a back-
 216 pressure of 196 kPa in the consolidation stage. The samples were consolidated under a K_0 value of 0.42,
 217 and then the samples were sheared in both compression and extension with axial strain rates of 0.7, 0.07,
 218 and 0.007%/min.

219 Kamei and Sakajo [44] reported the values of conventional soil parameters, such as κ , λ , M and
 220 initial void ratio, for the samples of Kawasaki clay. Based on the test data, the critical stress ratio in
 221 triaxial compression and extension were measured as 1.65 and 1.24, respectively. Rotational hardening
 222 parameters were determined according to Dafalias et al. [12]. For the simulations, the destructuration

223 feature of the model was switched off by setting $S_i = S_f = 1$, as the soil specimens were reconstituted.
 224 Viscosity parameters were determined through calibration based on data from tests at only two strain-
 225 rates.

226 Figure 2 shows the comparison between experimental and numerical results obtained using the EVP-
 227 SANICLAY model. The tests were simulated considering the consolidation stage. As it is seen in the
 228 figure, the response during triaxial compression has been captured very well by the model, while for the
 229 extension part the results are less accurate, even though the Lode angle dependency was considered in
 230 order to better reproduce the clay behavior in extension. As illustrated in Figure 3 of [Taiebat et al. \[13\]](#)
 231 this could be in part due to adoption of an associated flow rule in the EVP model.

232



233 Figure 2. Undrained triaxial test: (a) effective stress path; (b) deviator stress versus axial strain

234
 235 It can be noted that in triaxial compression, a better comparison between experimental and
 236 numerical results is achieved for higher strain rates. As the strain rate decreases, the numerical stress
 237 paths tend to be more lenient towards the critical state. As no destructuration was considered for this
 238 simulation and also associated flow rule was employed, the modeling results did not reproduce a
 239 noticeable softening behavior (Figure 2b). This is observed in both compression and extension. The initial
 240 stiffness of the curve was also well represented. The Lode angle dependency of the model allows capturing
 241 the anisotropy in strength as it was observed by [Nakase and Kamei \[42\]](#).

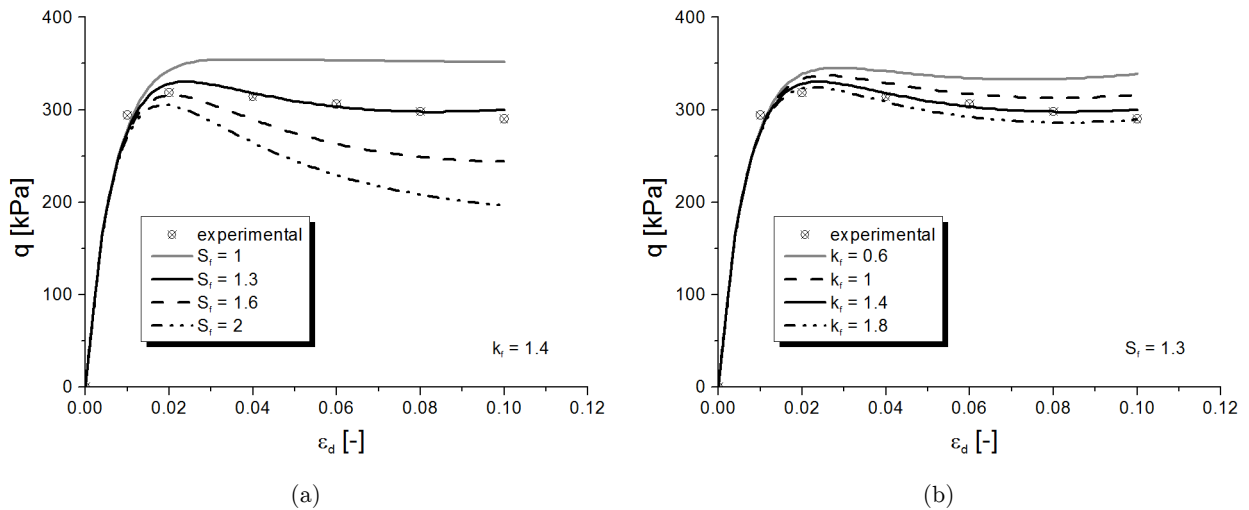
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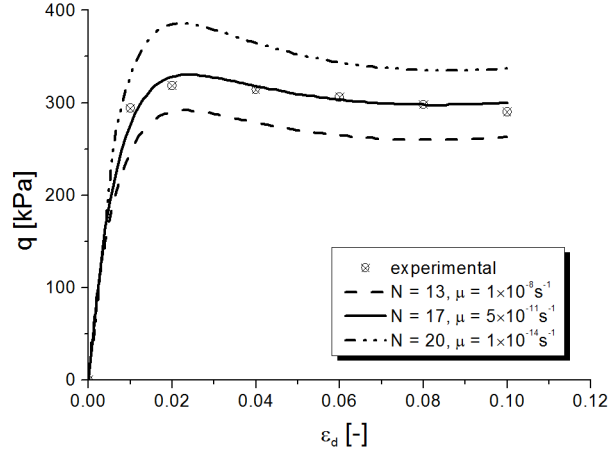
243 **3.2 Haney clay**

244 Vaid and Campanella [14] carried out undrained triaxial tests on undisturbed saturated sensitive
 245 marine clay known as Haney clay. It is a silty clay with $w_L = 44\%$, $w_p = 26\%$ and a sensitivity within the
 246 range of 6 to 10. All test samples were normally consolidated, with an all-around confining pressure of
 247 515 kPa. Consolidation was allowed for a period of 36 hr after which the samples were left undrained for
 248 12 hr under the consolidation stresses prior to shearing. In order to study the rate dependency of
 249 undisturbed clay response, the undrained shearing stage of the tests was performed at different constant
 250 strain rates, varying from 10^{-3} to 1.1% /min.

251 Values of conventional soil parameters listed in Tables 1 and 2 were reported by Vermeer and Neher
 252 [45]. After determination of the initial value of α , the values of anisotropy constants, C and χ , were
 253 obtained via curve fitting. Destructuration parameter values were also calibrated via trial runs.
 254 Structuration factor and destructuration constants influence the softening behavior after peak strength,
 255 and to a lesser degree, the shear strength achieved. Figure 3a and 3b show the influence of frictional
 256 destructuration in soft clay behavior. An increase of the frictional structuration factor leads to a larger
 257 softening behavior and a noticeable decrease in shear strength (Figure 3a).

258





(c)

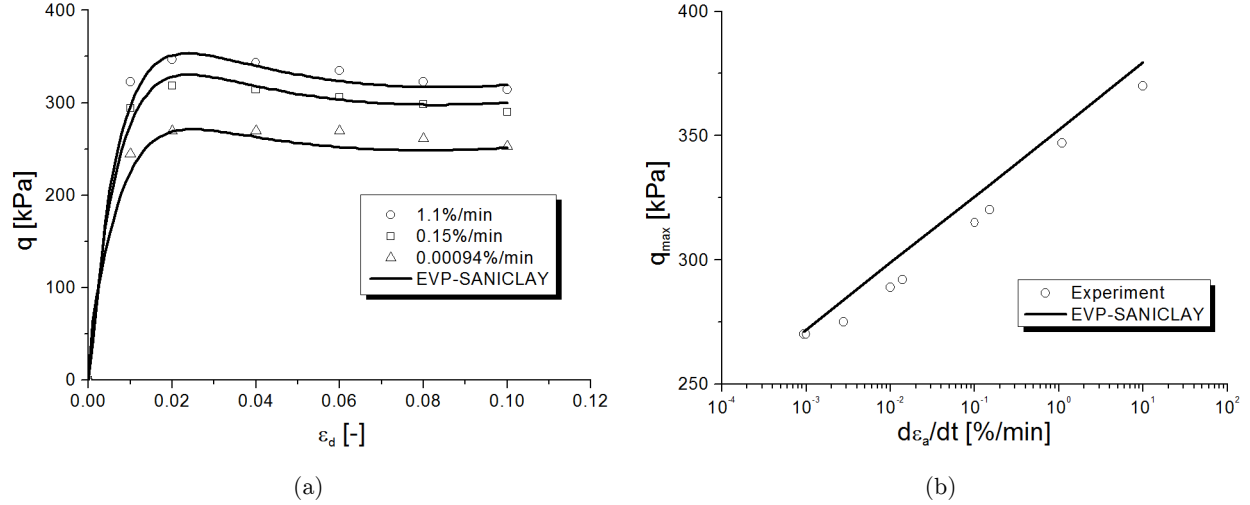
259 Figure 3. Calibration of structuration and viscosity parameters for Haney clay: (a) influence of frictional
 260 structuration parameter S_f for a constant rate $k_f = 1.4$; (b) influence of rate of frictional destructuration
 261 k_f for a constant value of $S_f = 1.3$; (c) combined influence of viscosity parameters N and μ

262

263 A similar, if less marked, behavior is seen in relation to the rate of frictional destructuration (Figure
 264 3b), with larger softening observed for higher destructuration rates. Viscosity parameter values were
 265 calibrated based on the results of two tests (i.e. at two strain rates) only. As it is shown in Figure 3c,
 266 viscosity parameters play an important role in the overall calibration of the model, particularly with
 267 regards to the shear strengths achieved. In order to obtain an improved match with the experimental
 268 results, instead of the default value of 0.5, a value of 0.3 was adopted for the destructuration parameter
 269 A .

270 For model simulations using EVP-SANICLAY, three specific strain rates, at 0.00094%/min,
 271 0.15%/min and 1.1%/min, have been taken into account to reproduce the observed shear stress-shear
 272 strain curves. Also the peak strengths achieved at different strain rates were considered to evaluate the
 273 model performance. The experimental versus numerical results are shown in Figure 4. It can be seen from
 274 the figure that the model simulations compare very well with the observed behavior. The model, with its
 275 destructuration function on, is able to simulate the softening behavior of natural clay response after peak
 276 (Figure 4a). Also Figure 4b indicates that the model provides a reasonably good representation for the
 277 variations of maximum shear strength with loading rate.

278



279 Figure 4. Undrained triaxial compression tests: (a) deviator stress versus axial strain; (b) evolution of
 280 maximum deviator stress with strain rate

281

282 3.3 St. Herblain clay

283 A particular CSR oedometer test was performed by Rangeard [43] on St. Herblain clay, a clayey
 284 river alluvial deposit. Two different strain rates were considered during the test. The test was started
 285 with a strain rate of $3.3 \times 10^{-6} \text{ s}^{-1}$ until an axial strain of 12%, at this strain the loading rate was lowered to
 286 a strain rate of $6.6 \times 10^{-7} \text{ s}^{-1}$ and was kept at that until a vertical strain of 15.5%, then again the rate was
 287 switched back to the initial strain rate and was kept constant until the end of the test.

288 Soil parameter values, obtained from oedometer and triaxial tests, were also reported by Rangeard
 289 [43]. The clay sample used for the experiments was taken from a depth of 6.5–7.5 m, it had a bulk unit
 290 weight $\gamma = 14.85 \text{ kN/m}^3$ and a water content of 87%. A vertical pre-consolidation pressure of 52 kPa was
 291 determined from the oedometer tests. The model parameters adopted are summarised in Tables 1 and 2.

292 Given that the clay was slightly structured, for the simulations the destructuration feature of the
 293 model was switched off. Figure 5 shows the experimental data versus simulation results. It can be seen
 294 that the model predictions are in good agreement with the data, particularly with regards to vertical
 295 stresses. The model also captures the indentation due to the change in strain-rate during the test.

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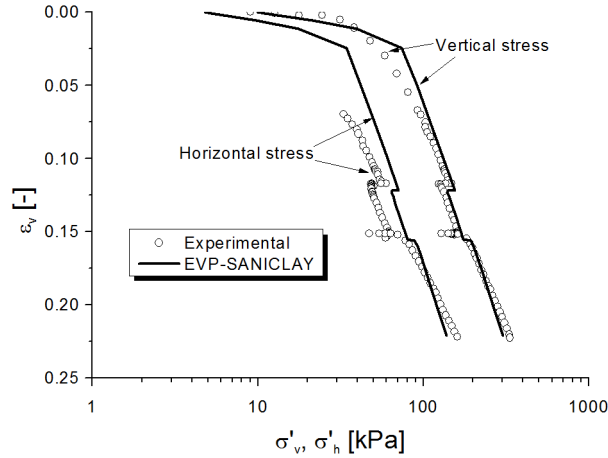


Figure 5. Simulations of CRS oedometer test results over St. Herblain clay

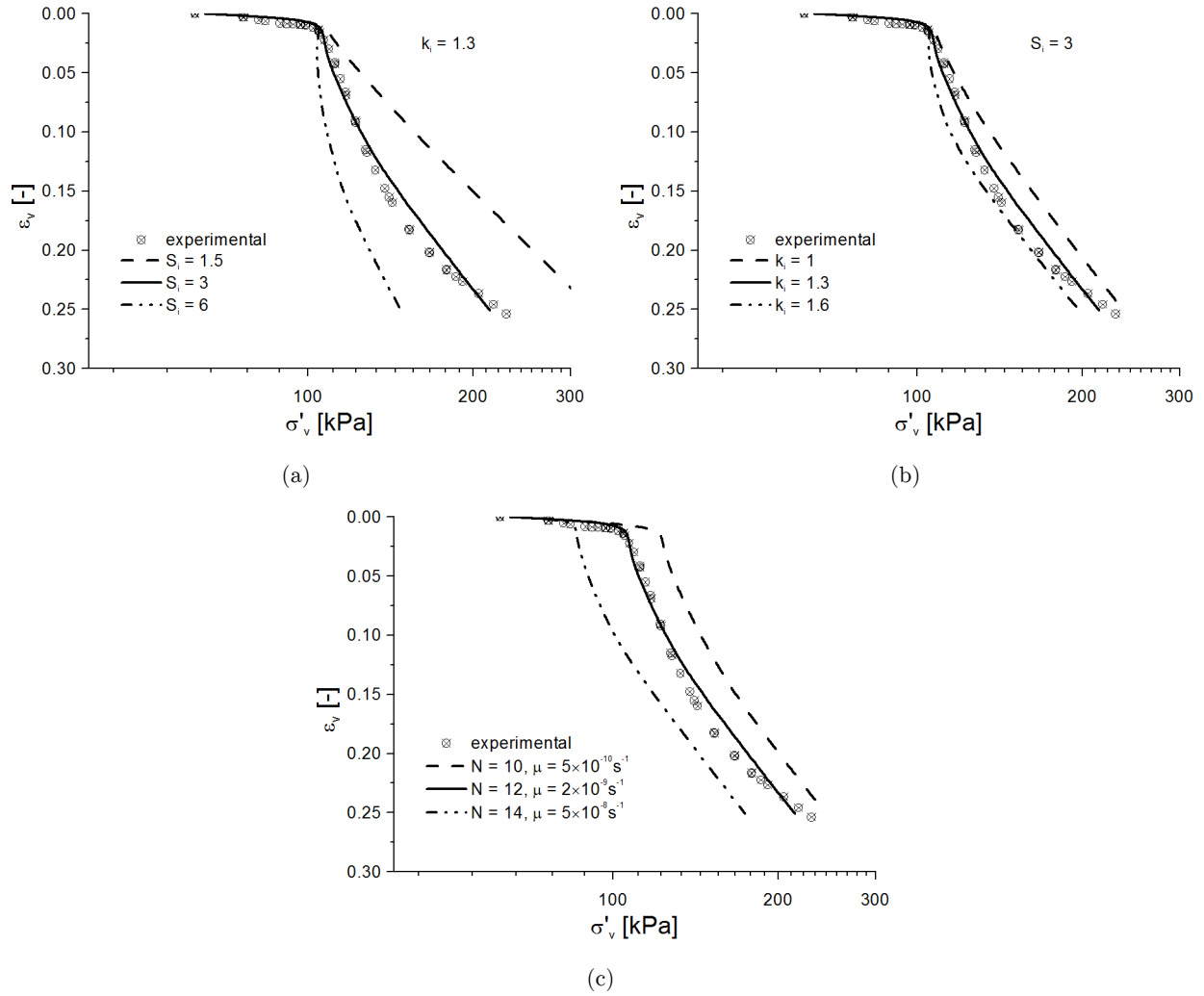
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299 3.4 Batiscan clay

300 CSR oedometer tests were performed on Batiscan clay by [Leroueil et al. \[16\]](#). The clay samples were
 301 taken from a depth of 7.25 - 7.46 m; the samples reportedly had a water content of 80%, liquidity index I_L
 302 = 2.7, $I_p = 21$, and $\gamma = 17.5 \text{ kN/m}^3$. The pre-consolidation pressure, determined from conventional
 303 oedometer tests, was evaluated as 88 kPa. The strain-rates for the CRS tests varied between $1.7 \times 10^{-8} \text{ s}^{-1}$
 304 and $4 \times 10^{-5} \text{ s}^{-1}$. The initial vertical effective stress was taken equal to 65 kPa, corresponding to a size of
 305 the initial yield surface of 50 kPa. Conventional soil parameter values reported in Tables 1 and 2 were
 306 obtained from [Leroueil et al. \[16\]](#) and [Rocchi et al. \[46\]](#).

307 Combinations of initial degree of structuration and rate of destructuration have been studied and the
 308 best coupled values were chosen for the numerical simulations. As it is shown in Figure 6a, larger values
 309 of initial structuration S_i result in a larger reduction of final vertical stress due to the higher softening
 310 occurring. For a constant value of S_i , the value of the rate of destructuration does not appear to have as
 311 much influence, but it follows the same trend (Figure 6b), with higher rates leading to a higher vertical
 312 stress reduction. Viscosity parameters are typically obtained from long-term oedometer tests via curve
 313 fitting. The calibration of the coupled values is showed in Figure 6c. Note that viscosity parameters
 314 greatly change the stress value at the end of the initial stiff elastic regime. The calibrated model
 315 parameter values are summarised in Tables 1 and 2.



316 Figure 6. Calibration of structuration and viscosity parameters for Batiscan clay: (a) influence of isotropic
 317 structuration factor S_i for a constant rate $k_i = 1.3$; (b) influence of rate of isotropic destructuration k_i for
 318 a constant structuration value of $S_i = 3$; (c) combined influence of viscosity parameters N and μ

319

320 Model simulations using EVP-SANICLAY are shown in Figure 7. It is seen that a good correlation is
 321 obtained between the numerical results and experimental data. Also clearly the strain-rate effects are well
 322 captured; the exponential trend of the curves indicates the progress of destructuration at large strains.

323

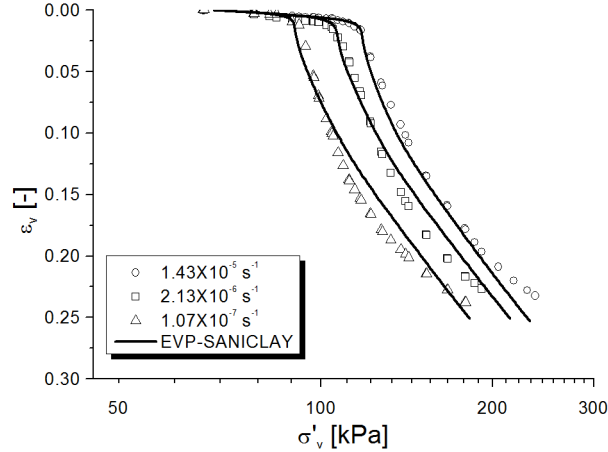


Figure 7. Oedometer test results: vertical strain versus vertical stress

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Considering that all above element test simulations performed using EVP-SANICLAY, it appears that in addition to the anisotropy and destructuration effects, the model is able to reasonably capture the strain-rate dependency in behavior of natural clays. Also for the simulations performed above, the model implementation proved to be sufficiently robust.

331 4 Boundary value level modeling

332 In order to study the performance of the proposed elasto-viscoplastic constitutive model at the
333 boundary value level, the simulation of a test embankment was carried out. In particular, embankment D
334 of a set of four test embankments built on a soft, sensitive and cemented clay in Saint-Alban, Quebec,
335 Canada was selected [47-48]. This is a well-known and well-instrumented embankment for which soil
336 parameters are readily available in the literature.

337 4.1 Model description

338 Embankment D has a height of 3.28m, a uniform crest width of 7.6m and slope angles of 13.75°. The
339 embankment material consists of uniform medium sand compacted to a unit weight of 18.56 kN/m³. It
340 was constructed on 13.7 m deep natural clay deposit known as Champlain clay, underlain by a dense fine
341 to medium sand layer down to a depth of 24.4m [49]. The soft deposit is overlain by approximately 1.5 m
342 thick weathered crust. In order not to disturb the very soft and sensitive clay deposit at the site, the
343 embankment was built directly on the existing natural ground, without excavating the thin dry crust
344 layer at the top. In this work a two-dimensional plane strain finite element model of the embankment was

345 created using PLAXIS AE [41], and taking advantage of the symmetry, only half of the embankment was
346 modeled. A finite element mesh with 1723 15-noded triangular elements (Figure 8.a) was used for the
347 analyses. Each element has pore water pressure (PWP) degrees of freedom at corner nodes. Mesh
348 sensitivity studies were carried out to ensure that the mesh was dense enough to produce accurate results.
349 The geometry of the finite element model is shown in Figure 8a. The far right boundary of the model was
350 assumed at distance of 40m from the embankment centerline. The bottom boundary of the clay deposit
351 was assumed to be completely fixed in both horizontal and vertical directions, whereas, the left and right
352 vertical boundaries were only restrained horizontally. Drainage was allowed at the ground level, while the
353 bottom boundary was considered impermeable. Impermeable drainage boundaries were also assigned to
354 the lateral boundaries. Based on ground data, the water table was assumed at 0.7m depth.

355 The embankment was built in stages, with an initial layer of 0.6m and after 6 days the normal
356 construction began (Figure 8.b), with an average rate of 0.24m/day [48]. The same construction pace was
357 adopted in the numerical model. For the calculation phases, plastic analyses were carried out
358 corresponding to the construction process of the embankment, after which the consolidation analysis was
359 performed.

360 For the numerical analysis, the embankment itself was modeled with the simple linear elastic-
361 perfectly plastic Mohr-Coulomb model using the following reported values for the embankment material:
362 Young's modulus $E = 40,000$ kPa, Poisson's ratio $\nu = 0.3$, friction angle $\phi = 44^\circ$, and cohesion $c = 0$ kPa.
363 The dry crust layer above the water table was also modeled with the Mohr-Coulomb model using shear
364 modulus $G = 880$ kPa, Poisson's ratio $\nu = 0.3$, $\phi = 27^\circ$, $c = 1$ kPa. Unit weight $\gamma = 19$ kN/m³ is used for
365 both [47,50]. The sensitive Champlain clay deposit below the water table was modeled using the
366 implemented user-defined EVP-SANICLAY model, with a unit weight $\gamma = 16$ kN/m³ [47].

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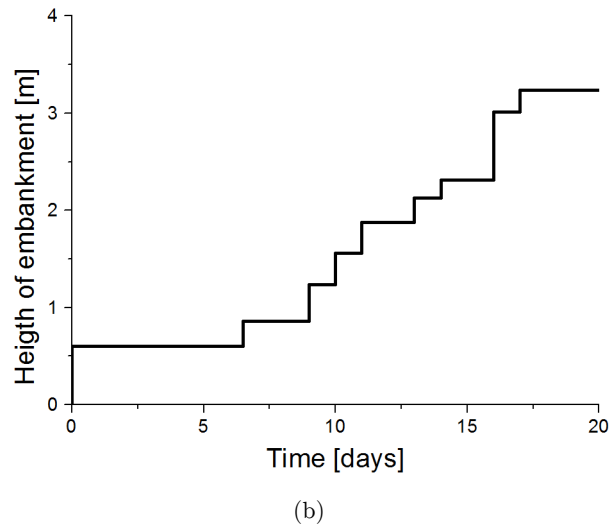
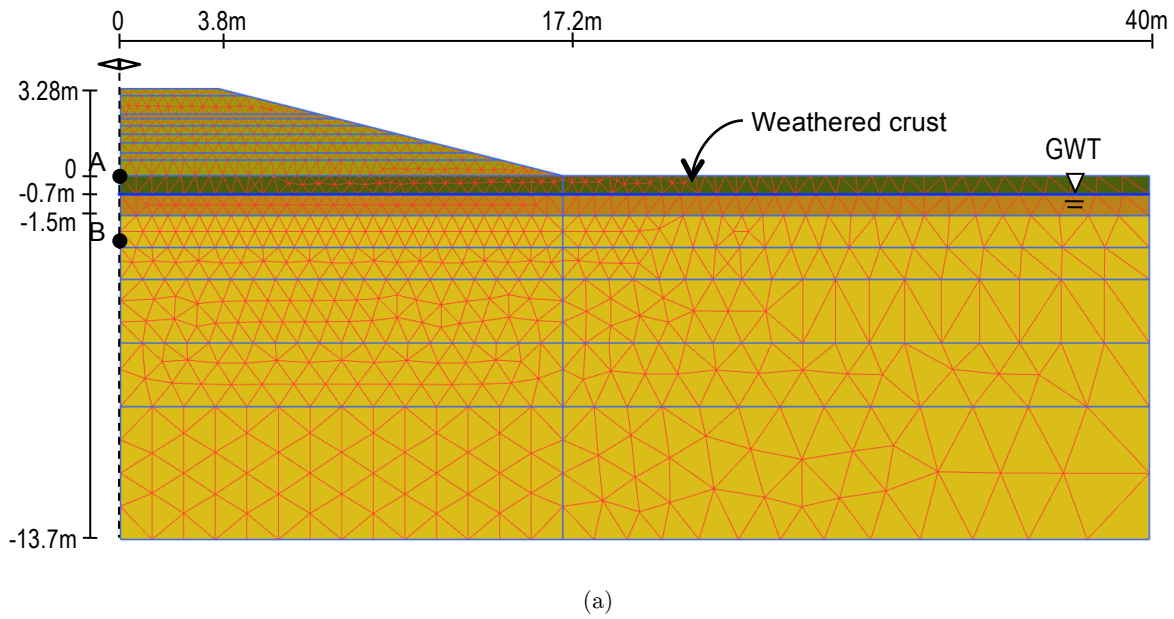
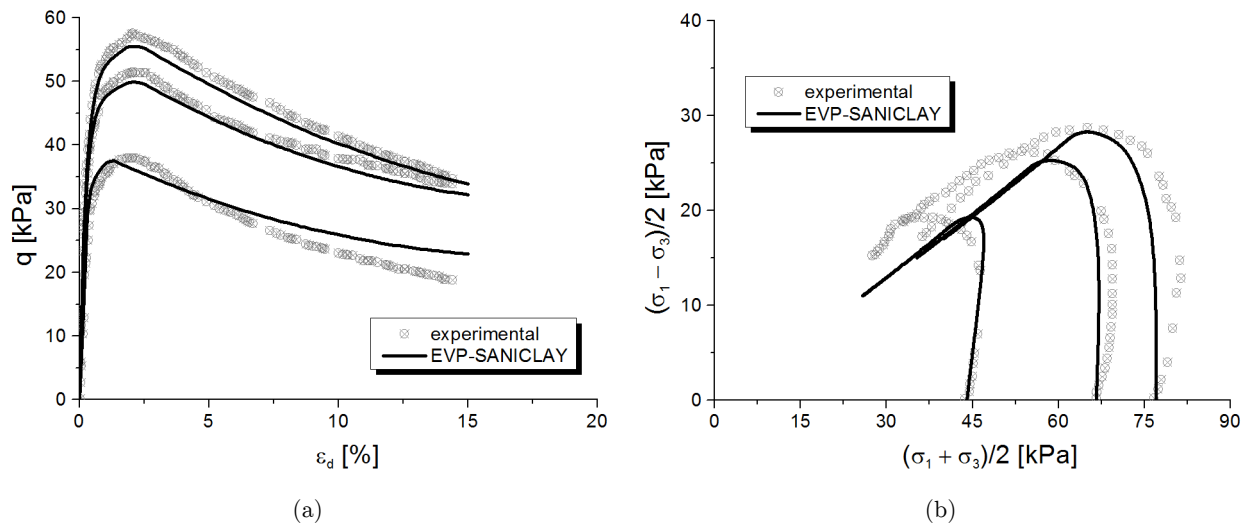


Figure 8. (a) Geometry of the model embankment and the finite element mesh adopted; (b) construction history of the St. Alban embankment D

370 The material parameter values for the Champlain clay layers were determined using the available
 371 data obtained from testing of samples taken at a depth of 6m below the ground surface [15]. Conventional
 372 parameter values were derived from existing studies based on soil element test results [47-48,50]. Similar
 373 to the section on element level simulations, the anisotropy parameter values were determined following

374 the approach proposed by Dafalias et al. [12]. The destructuration parameters were calibrated against
 375 experimental data available for undrained triaxial compression tests over samples of Champlain clay
 376 taken from a depth of approximately 3m [51]. For three tests presented in Figure 9 the samples were first
 377 isotropically consolidated up to three different pre-consolidation pressures of 44, 66.6 and 77 kPa, and
 378 subsequently sheared. Figure 9 shows a good agreement between the experimental data and the numerical
 379 simulations both in terms of stress-strain response and of stress paths. The destructuration trend after
 380 peak strengths was also well captured.

381



382 Figure 9. Validation of numerical simulations versus experimental results for undrained triaxial
 383 compression tests: (a) deviator stress versus axial strain; (b) effective stress paths

384

385 In the absence of appropriate soil test data, such as long-term oedometer tests with at least two
 386 different strain rates, viscosity parameters were calibrated using trial runs.

387

388 Table 3 summarises the soil and state parameters adopted for the simulation of St. Alban test
 389 embankment, and Table 4 lists the calibrated anisotropy and destructuration parameter values. The
 390 permeability, k , of the clay, assumed to be isotropic, was reported to be equal to 3.46×10^{-4} m/day. It
 391 should also be added that the initial state of stress was generated by adopting K_0 -procedure [41] where
 392 the reported K_0 value of 0.8 was employed [52]. Results from oedometer tests performed on Champlain
 393 clay reported that over-consolidation ratio (OCR) varied between 1.8 and 2.2 [47]; a mean value of 2.0
 394 was assumed for the analyses performed here.

395

396

Table 3 – Constants of the EVP-SANICLAY model adopted for St. Alban test embankment D

Model constant		Top Champlain clay layer (0.7-1.5 m)	Bottom Champlain clay layer (1.5-13.7 m)
Elasticity	κ	0.012	0.013
	ν	0.3	0.3
Critical state	M_c (M_e)	1.07	1.07
	λ	0.36	0.25
Rotational hardening	C	10	10
	x	1.7	1.7
Destructuration	k_i	1.5	1.5
	k_f	1.4	1.4
	A	0.5	0.5
Viscosity	N	13	13
	μ [s ⁻¹]	5×10^{-9}	5×10^{-9}

397

398

Table 4 – Initial values of state variables adopted for St. Alban test embankment D

Model state variable		Top Champlain clay layer (0.7-1.5 m)	Bottom Champlain clay layer (1.5-13.7 m)
Initial void ratio	e	1.7	1.8
Overconsolidation ratio	OCR	2.0	2.0
Initial rotation of the SYS	α	0.41	0.41
Initial isotropic structuration factor	S_i	4.5	4.5
Initial frictional structuration factor	S_f	1.2	1.2

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In order to assess the performance of EVP-SANICLAY model, the finite element analysis of the embankment was repeated twice where instead of the EVP-SANICLAY model the MCC model and the EVP-SANICLAY model without destructuration (i.e., with $S_i = S_f = 1$) were used.

4.2 Simulations results

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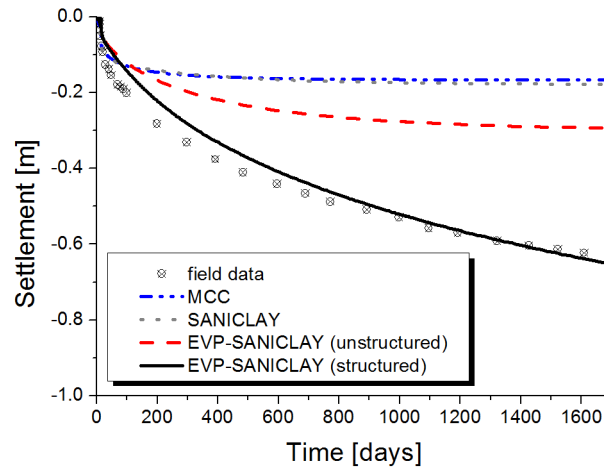
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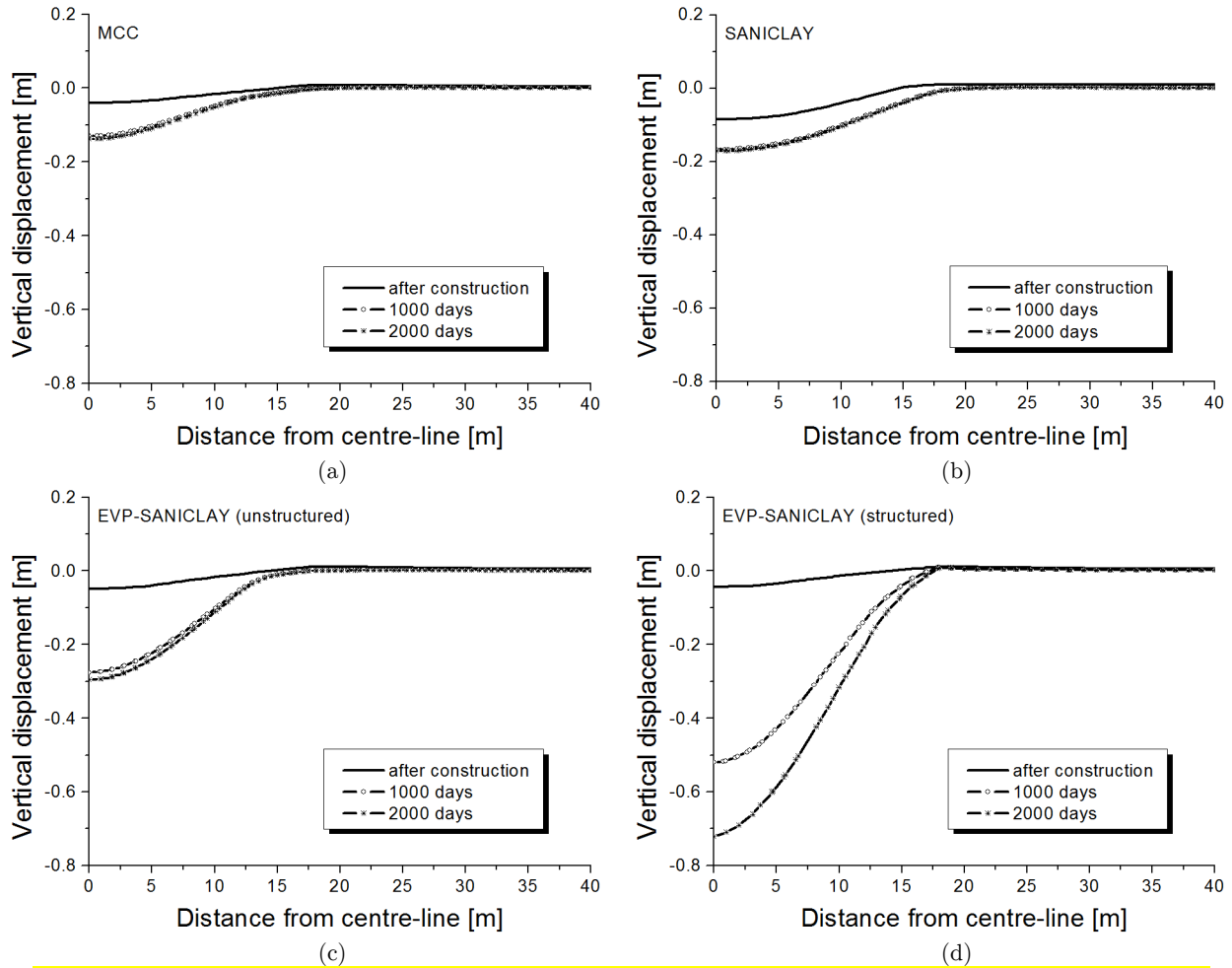
The results from numerical analyses were compared with the available field measurement data for the time period following the construction [47,50,53]. Figure 10 shows settlement predictions versus time at the node on the centerline at the base of the embankment (point A in Figure 8a), using different models. From the figure it is clear that the proposed EVP-SANICLAY model gives a rather good prediction when compared to the *in-situ* measurements. When destructuration is switched off, the model

410 significantly underestimates the settlement over time. The underestimation of settlement is even more
 411 pronounced with the MCC and SANICLAY models; in this case the predicted settlement reaches an
 412 approximately constant value after 400 days, pointing out that the model is clearly time-independent.
 413



414 Figure 10. Time-settlement predictions versus field measurements at point A in Figure 8a

415
 416 No additional field data is available for surface settlements recorded at different times, but a
 417 comparison between the numerical results adopting different soil constitutive models can be made. Such
 418 numerical simulation results are shown in Figure 11. Generally they all show a typical behavior, with the
 419 main vertical settlements at the centerline of the embankment and diminishing values at larger distances
 420 from the centerline. However, as consideration of soil viscosity during plastic deformation delays the
 421 consolidation process, settlements through using EVP-SANICLAY (Figure 11d) represent more realistic
 422 deformation pattern with time. The simulation performed using the MCC and SANICLAY (Figure 11a,b)
 423 clearly shows that with the time-independent models the consolidation process completes rapidly after
 424 which the vertical deformation stops. When the effect of soil structure is ignored (Figure 11c) a behavior
 425 similar to the complete EVP-SANICLAY model is obtained, but with significantly lower values for the
 426 vertical settlement. This is expected, given that Champlain clay is highly structured clay with a
 427 sensitivity value of about of 22 [15].

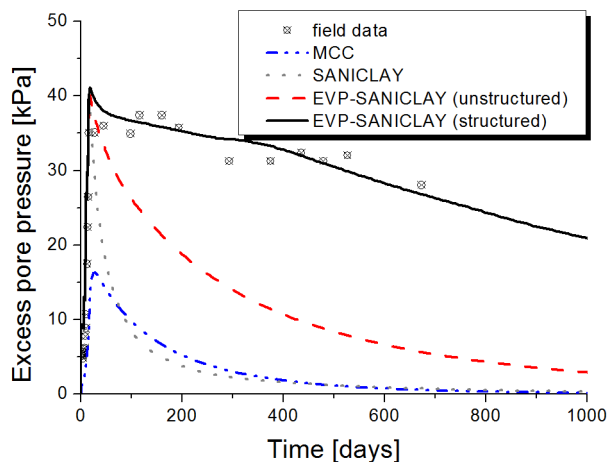


428 Figure 11. Numerical simulation results for surface settlement using: (a) MCC model; (b) SANICLAY
 429 model; (c) EVP-SANICLAY model without structure; (d) EVP-SANICLAY model

430

431 Pneumatic piezometers were installed at different depths underneath the embankment to monitor the
 432 excess pore water pressure variations with time [48,50]. Figure 12 shows the *in-situ* measurements related
 433 to a piezometer located on the centerline at a depth of 2.6m under the base of the embankment (point B
 434 in Figure 8a). The excess PWP initially increased during the embankment construction and then
 435 gradually dissipated with time. The figure also shows the results of numerical simulations with the
 436 models. As it can be seen, a better approximation of the excess PWP variation is obtained with the EVP-
 437 SANICLAY model, in comparison with the MCC, SANICLAY, and the anisotropic EVP model without
 438 structure. Interestingly, for the SANICLAY and both of the EVP-SANICLAY model simulations, with
 439 and without structure, the maximum PWP value is reasonably close to the field measurement, but when
 440 the initial structure and degradation of bonds are not taken into consideration, a faster dissipation of

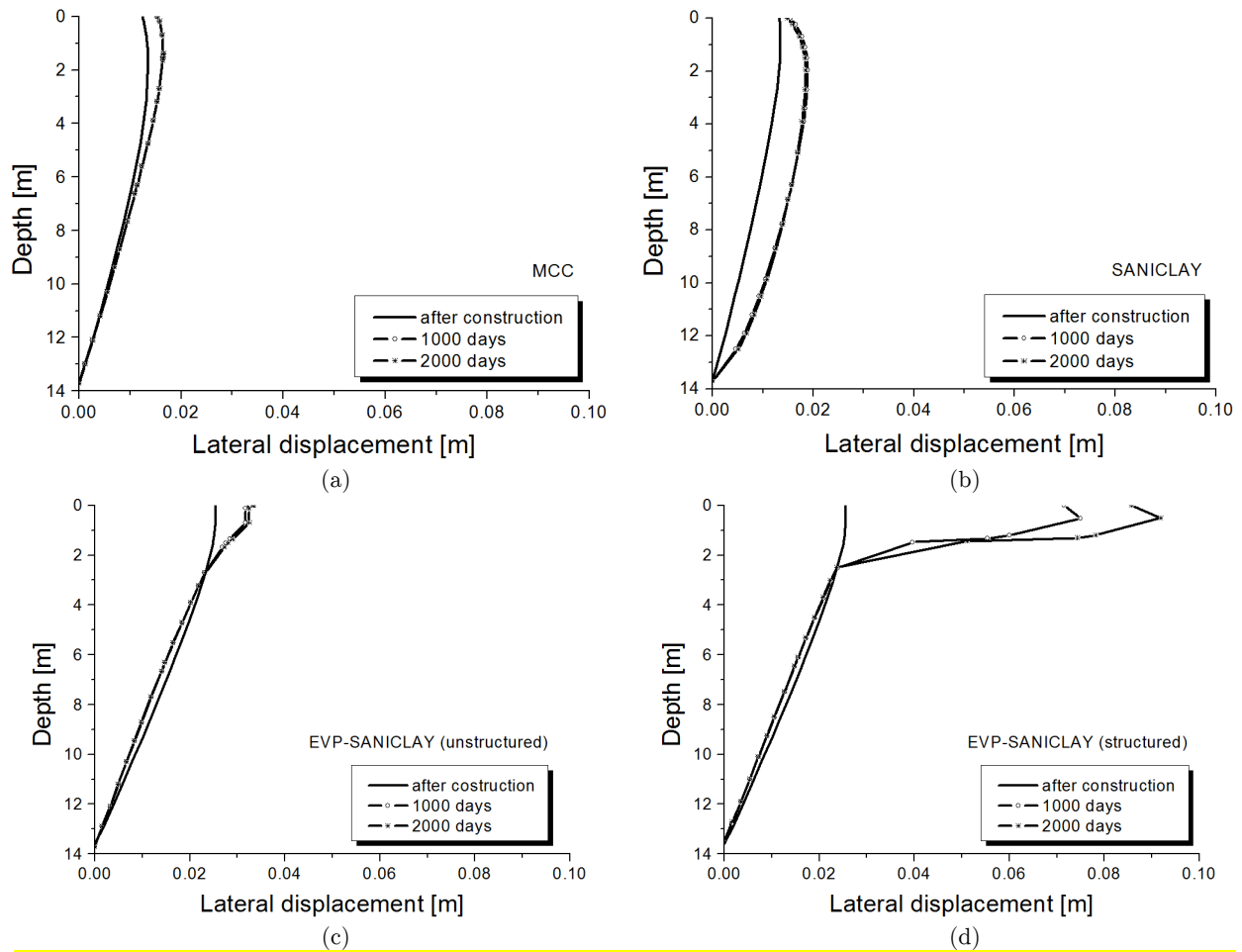
441 excess PWP is observed. MCC model underestimates the maximum excess PWP immediately after the
 442 construction; additionally, after the construction excess PWP is dissipated very quickly, contrary to the
 443 observed *in-situ* scenario. The observed delayed pore pressure dissipation can be captured only when the
 444 viscosity of soil behavior is taken into consideration.
 445



446
 447 Figure 12. Excess PWP predictions at point B in Figure 8a
 448

449 Field data for lateral displacements at depth are not available for the embankment [54]; therefore,
 450 simulation results presented in Figure 13, for the lateral deformation profiles at the toe of the
 451 embankment, could not be compared with the actual measurements. From Figure 13d, EVP-SANICLAY
 452 model simulations show deformation profiles similar in shape to what was reported for other embankment
 453 sites. For example in case of St. Alban embankment B, the maximum lateral displacement was reported
 454 to have more than doubled during the initial 4.5 years of consolidation [54], and the maximum value was
 455 at a depth of about 1m. The MCC and SANCILAY models led to smaller lateral displacement near the
 456 surface (Figure 13a,b). For the EVP simulations in Figure 13c,d, the lateral displacements increased near
 457 the surface, and delayed deformation became more pronounced. When structure effects were ignored in
 458 the EVP model (Figure 13c), the general shape of the lateral deformation profiles did not change much
 459 compared to Figure 13d but the predicted values became smaller, without noticeable difference between
 460 the profiles at 1000 and 2000 days. Clearly consideration of the soil initial structure and its degradation
 461 result in a greater pace of viscoplastic strain developments. For example, monitoring the development of
 462 viscoplastic strains at point B under the embankment, i.e. the position of the piezometer, it can be seen in
 463 Figure 14 that after an initial elastic response, the viscoplastic strains begin to develop while still in the

464 construction phase, and then continue to evolve with consolidation progress. It is particularly apparent
 465 how ignoring soil structure effects lead to significantly lower viscoplastic strain accumulation, a trend also
 466 observed in previous figures.



467 Figure 13. Numerical simulation results for lateral displacement under the toe using: (a) MCC model; (b)
 468 SANICLAY model; (c) EVP-SANICLAY model without structure; (d) EVP-SANICLAY model

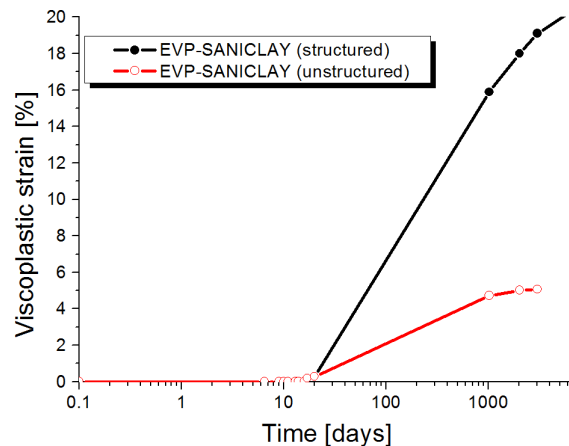


Figure 14. Development of viscoplastic strains at point B in Figure 8a

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4.3 Non-uniqueness of viscosity parameters

As already mentioned, calibration of viscosity parameters N and μ has been done directly on the embankment model as no appropriate test data has been available for the foundation soil. It should be pointed out that the Perzyna-type viscosity parameters for a particular clay are not necessarily a unique set and more than one combination of the two viscosity parameters can be found for a clay, depending on how one wants to fit the experimental data [55]. Figure 15 shows an example of how for three different sets of viscosity parameter values it is still possible to obtain a good approximation of the field observation for settlements at point A under the embankment. For these particular sets, a maximum difference of only 3% was found among the vertical settlement results, and similar minor variations were observed among the corresponding lateral displacement and excess PWP predictions.

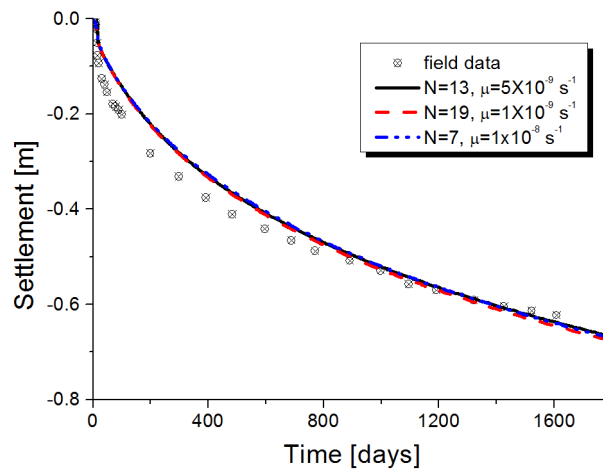


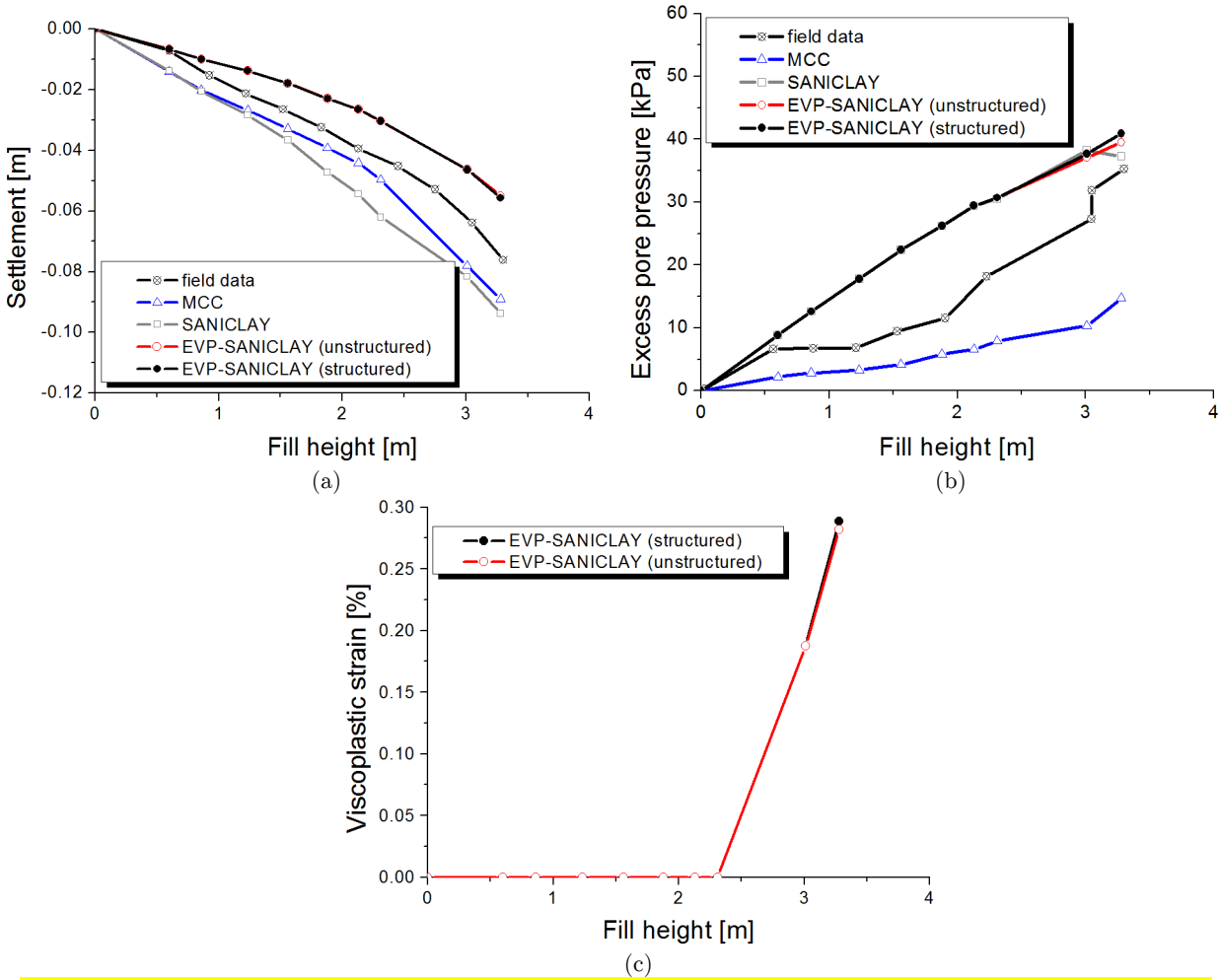
Figure 15. Illustrating the non-uniqueness of viscosity parameters for prediction of time-settlement at point A in Figure 8a

4.4 Discussion on behavior during construction

Additional field measurement data on settlement and excess pore pressure generation during embankment construction process is also available [48]. The data could be used to assess the performance of the developed model in reproducing the short-term response of the embankment. Simulation results during the construction are shown in Figure 16.

Figure 16a shows that at point A in Figure 8a, EVP-SANICLAY model somewhat underestimates the results; although, as it was observed in Figure 7a, it is then able to gain accuracy during consolidation. MCC and SANCILAY, on the contrary, overestimates short-term settlements during the

493 construction. In terms of excess PWP at point B in Figure 8a, EVP-SANICLAY is able to give a good
 494 prediction of the pore pressure generation during the embankment construction (Figure 16b). Based on
 495 EVP-SANICLAY predictions, PWP develops rapidly during the construction until embankment reaches a
 496 height of approximately 2.31 m (corresponding to 16 days after the start of construction) when the excess
 497 PWP generation slightly decelerates. From the figure, it is clear that the MCC model underestimates
 498 excess PWP generation during the construction. Compared to the full EVP model, EVP-SANICLAY
 499 without structure provides lower predictions of excess PWP generation after the stage at which the
 500 embankment reaches a height of 2.31 m.
 501



502 Figure 16. Field measurements versus numerical simulation results for the duration of construction: (a)
 503 settlements at point A in Figure 8a; (b) excess pore water pressure at point B in Figure 8a; (c)
 504 development of viscoplastic strains at point B in Figure 8a
 505

506 **Figure 16c** shows the development of viscoplastic strains at point B under the embankment (i.e. at
507 the position of the pneumatic piezometer considered). The figure shows that the viscoplastic strains start
508 to develop when the embankment reaches a height of about 2.31m, which approximately corresponds to
509 the time when excess pore pressure generation changes its pace.

510

511 **5 Conclusions**

512 The response of natural soft soil is governed by anisotropy, structure and time-dependency. In this
513 work, in order to concurrently account for these advanced features of soil behavior a time-dependent
514 elasto-viscoplastic extension of a well-established anisotropic clay model, namely SANICLAY, has been
515 proposed. The model is numerically implemented in finite element program PLAXIS using an implicit
516 integration scheme. The performance of the model at the element-level has been validated against
517 experimental data obtained from testing four different clays at both structured and un-structured states.
518 Furthermore the time-dependent behavior of St. Alban embankment D on the well-structured Champlain
519 clay was analysed using the proposed EVP-SANICLAY model. The paper presented the results for
520 settlements, lateral deformations, and excess PWP variations during the construction and the subsequent
521 consolidation, comparing model predictions with the field measurements where available. It was observed
522 that the developed model considers the delayed excess pore pressure dissipation following the completion
523 of the embankment construction reasonably well; hence it is able to yield more realistic predictions of the
524 long-term vertical and horizontal deformations. The boundary value problem simulation results also
525 illustrated that considering clay initial structure and subsequent destructuration effects significantly
526 improve the accuracy of predictions, particularly when dealing with a highly sensitive soft clay such as
527 Champlain clay. Furthermore, the model also predicted the immediate displacements as well as the
528 development of excess pore pressures during early stages of construction with reasonable accuracy.

529 In general, EVP-SANICLAY proved to be able to much better predict both short- and long-term
530 behavior of natural clay behavior when compared with a commonly used critical state based model such
531 as MCC, and also the SANCILAY model.

532

533 Acknowledgements

534 Support to conduct this study is provided by the University of Nottingham's Dean of Engineering
535 award, and the Natural Sciences and Engineering Research Council of Canada (NSERC).

536

537 Appendix

538 For the sake of completeness of presentation, some of the key components of the SANICLAY model
539 that are not presented in the main body of this paper are summarized here. Both stress and strain
540 quantities are assumed positive in compression (as is common in geomechanics), and the effect of this sign
541 convention has been considered on the model equations. All stress components in this paper should be
542 considered as effective stress. Finally, in terms of notation, tensor quantities are denoted by bold-faced
543 symbols and operations explained accordingly.

544 The hypoelastic formulation, considered for simplicity, constitutes of a shear modulus G , for
545 calculating increments of elastic deviatoric strains, and a bulk modulus K , for calculating increments of
546 elastic volumetric strains, where

$$547 \quad G = \frac{3K(1 - 2\nu)}{2(1 + \nu)}; \quad K = \frac{p(1 + e)}{\kappa} \quad (A.1)$$

548 where ν is the Poisson's ratio, e is the void ratio, $p = (\text{tr}\boldsymbol{\sigma})/3$ is the mean effective stress (where tr stands
549 for the trace), and κ is the slope of elastic swelling lines in the $e - \ln p$ space.

550 The isotropic hardening law of the model describing the evolution of the size of structured SYS, i.e.
551 p_0^{*s} , is defined as

$$552 \quad \dot{p}_0^{*s} = \dot{S}_i p_0^s + S_i \dot{p}_0^s \quad (A.2)$$

553 where \dot{S}_i is the evolution rate of the isotropic destructure factor (explained in the sequel), and
554 $\dot{p}_0^s = [(1 + e)/(\lambda - \kappa)] p_0 \dot{\epsilon}_v^{vp}$ is the evolution of the size of SYS, that is a proportional to viscoplastic
555 volumetric strain rate, with λ indicating the slope of normal compression line.

556 The rotational hardening law describing the evolution of fabric anisotropy with viscoplastic staining
557 can be expressed in the general stress space as:

$$558 \quad \dot{\boldsymbol{\alpha}} = \left(\frac{1 + e}{\lambda - \kappa} \right) C \left(\frac{p}{p_0} \right)^2 |\dot{\epsilon}_v^{vp}| \left[\frac{3}{2} (\mathbf{r} - \boldsymbol{\alpha}) : (\mathbf{r} - \boldsymbol{\alpha}) \right]^{1/2} (\boldsymbol{\alpha}^b - \boldsymbol{\alpha}) + \dot{\boldsymbol{\alpha}}_f \quad (A.3)$$

559 In the above equation, $\dot{\boldsymbol{\alpha}}_f = (\dot{S}_f/S_f)\boldsymbol{\alpha}$ controls the contribution of destructure over the change of
560 orientation of the yield surface (\dot{S}_f explained in the sequel); $\mathbf{r} = \mathbf{s}/p$ is the shear stress ratio; $\boldsymbol{\alpha}^b =$

558 $\sqrt{2/3} M \mathbf{n}_x$ is the bounding ‘image’ stress-ratio tensor, where \mathbf{n}_x is an auxiliary unit tensor defined as
 559 $\mathbf{n}_x = \|(\mathbf{r}/x) - \boldsymbol{\alpha}\|$ and $\| \cdot \|$ denoting the norm operator; and $|\dot{\varepsilon}_v^{vp}|$ is the absolute value of the
 560 viscoplastic volumetric strain rate.

561 In order to express the isotropic and frictional destructurements, an auxiliary internal variable called
 562 the destructurement viscoplastic strain rate, $\dot{\varepsilon}_d^{vp}$, is defined by

$$\dot{\varepsilon}_d^{vp} = \sqrt{(1 - A)(\dot{\varepsilon}_v^{vp})^2 + A(\dot{\varepsilon}_q^{vp})^2} \quad (\text{A.4})$$

563 where $\dot{\varepsilon}_v^{vp}$ and $\dot{\varepsilon}_q^{vp}$ are the volumetric and deviatoric viscoplastic strain rates, respectively, and A is a
 564 model parameter could be set to 0.5 as a default value. The evolution equations for the S_i and S_f read

$$\dot{S}_i = -k_i \left(\frac{1 + e}{\lambda - \kappa} \right) (S_i - 1) \dot{\varepsilon}_d^{vp} \quad (\text{A.5})$$

$$\dot{S}_f = -k_f \left(\frac{1 + e}{\lambda - \kappa} \right) (S_f - 1) \dot{\varepsilon}_d^{vp} \quad (\text{A.6})$$

565 where k_i and k_f are model parameters.

566 As indicated in model formulation section, the critical stress-ratio is defined as a function of the Lode
 567 angle θ . To regulate the variation of $M(\theta)$ between its values M_c for compression and M_e for extension,
 568 the expression proposed by Sheng et al. [56] has been adopted here which reads as

$$M(\theta) = M_c \left(\frac{2m^4}{1 + m^4 + (1 - m^4) \sin 3\theta} \right)^{1/4} \quad (\text{A.7})$$

569 where $m = M_e/M_c$, $-\pi/6 \leq \theta = (1/3)\sin^{-1}[-3\sqrt{3}J_3/(2J_2^{3/2})] \leq \pi/6$, with J_2 and J_3 being the second and
 570 third invariants of the modified stress deviator $\mathbf{s} - p\boldsymbol{\alpha}$.

572

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691

692

A viscoplastic SANICLAY model for natural soft soils

Mohammad Rezania^a, Mahdi Taiebat^{b,c,*}, and Elisa Poletti^d

^aDepartment of Civil Engineering, University of Nottingham, Nottingham, UK

^bDepartment of Civil Engineering, University of British Columbia, Vancouver, BC, Canada

^cDepartment of Civil & Environmental Engineering, Massachusetts Institute of Technology, Cambridge,
MA, USA

^dDepartment of Civil Engineering, University of Minho, Azurém, Guimarães, Portugal

Abstract

This paper focuses on constitutive and numerical modeling of strain-rate dependency in natural clays while also accounting for anisotropy and destructuration. For this purpose the SANICLAY model that accounts for the fabric anisotropy with the additional destructuration feature that accounts for sensitivity of natural clays, is considered as the reference model. An associated flow rule is adopted for simplicity. The model formulation is refined to also account for the important feature of strain-rate dependency using the Perzyna's overstress theory. The model is then implicitly integrated in finite element program PLAXIS. Performance of the developed and implemented model is explored by comparing the simulation results of several element tests and a boundary value problem to the available experimental data. The element tests include the constant strain-rate under one-dimensional and triaxial conditions on different clays. The boundary value problem includes a test embankment, namely embankment D constructed at Saint Alban, Quebec. For comparison, the test embankment is also analysed using the Modified Cam-Clay (MCC) model, the SANICLAY model, and the viscoplastic model but without destructuration. Results demonstrate the success of the developed and implemented viscoplastic SANICLAY in reproducing the strain-rate dependent behavior of natural soft soils.

Keywords: viscoplasticity; strain-rate dependency; anisotropy; destructuration; clay

1 Introduction

Modeling the stress-strain response of natural soft soils constitutes a challenge in practical geotechnical engineering; it is governed by a series of factors that are not always included in conventional constitutive models. In particular, the three main inherent features that influence their response are a) anisotropy, b) destructuration (degradation of the inter-particle bonds), and c) strain-rate dependency.

*Corresponding author. Tel.: +1 604 822 3279.

E-mail addresses: mohammad.rezania@nottingham.ac.uk (M. Rezania), mtaiebat@civil.ubc.ca (M. Taiebat), elisapoletti@civil.uminho.pt (E. Poletti).

30 Since modeling the full anisotropy of natural clay behavior is not practical due to the number of
31 parameters involved, efforts have been mainly focused on development of models with reduced number of
32 parameters while maintaining the capacity of the model [1]. Historically, for practical model development
33 purposes, the initial orientation of soil fabric is considered to be of cross-anisotropic nature, which is a
34 realistic assumption as natural soils have been generally deposited only one-dimensionally in a vertical
35 direction. It is also a well-established fact that the yield surfaces obtained from experimental tests on
36 undisturbed samples of natural clays are inclined in the stress space due to the inherent fabric anisotropy in
37 the clay structure (e.g., [2-4]). Based on the above, a particular line of thought has become popular in
38 capturing the effects of anisotropy on clayey soil behavior, by development of elasto-plastic constitutive
39 models involving an inclined yield surface that is either fixed (e.g., [2]), or can be changed its inclination by
40 adopting a rotational hardening (RH) law in order to simulate the development or erasure of anisotropy
41 during plastic straining (e.g., [5-6]). For obvious reasons a model accounting for both inherent and evolving
42 anisotropy would be more representative of the true nature of response in clays; hence, since the first
43 proposal of such model by Dafalias [5-6] similar framework has been adopted by a number of other
44 researchers for development of anisotropic elasto-plastic constitutive models (e.g., [7-11]). Based on the
45 original model, Dafalias et al. [12] proposed what they called SANICLAY model, altering the original RH law
46 and introducing a non-associated flow rule. A destructuration theory was later applied to the SANICLAY
47 model [13] to account for both isotropic and frictional destructuration processes. In these works, the
48 SANICLAY has been shown to provide successful simulation of both undrained and drained rate-independent
49 behaviour of normally consolidated sensitive clays, and to a satisfactory degree of accuracy of
50 overconsolidated clays.

51 Past experimental studies have also shown that soft soils exhibit time-dependent response (e.g., [14-
52 17]). Time-dependency is usually related to the soil viscosity that could lead to particular effects such as
53 creep, stress relaxation, and strain-rate dependency of response. Time-dependency of soil response can be
54 observed experimentally by means of creep tests, stress relaxation tests, or constant rate of strain (CRS) tests
55 [18]. Rate-sensitivity is a particular aspect of time effect that has been investigated extensively; it influences
56 both strength and stiffness of soils. Various studies using CRS tests have shown how faster strain rates for a
57 certain strain level lead to higher effective stresses; also, the general observation, particularly in soft soils, is
58 that higher undrained strengths can be achieved by increasing the loading rate (e.g., [16-17,19-20]). The
59 reported observations from laboratory studies all imply that consideration of soil viscosity effects could be
60 key for correct prediction of long term deformations in field conditions; although, neglecting soil viscosity
61 seemingly provide sufficiently correct predictions in short-term [21]. Landslides or long-term deformations
62 of tunnels and embankments on soft soils are examples of common practical problems where a sustainable
63 remediation and/or design solution can only be achieved if time-dependent behavior of soil is taken into
64 consideration.

65 In order to account for the time-dependency of soft clays' behavior, various frameworks can be found in
66 the literature. Among a number of popular frameworks such as the isotache theory of Šuklje [22] or the non-

67 stationary surface theory of Naghdi and Murch [23], the overstress theory of Perzyna [24-25] is a common
 68 framework often used in geomechanics for this purpose due to its relative simplicity. The first overstress-
 69 type viscoplastic models were based on isotropic Cam-Clay or modified Cam-Clay models (e.g., [26-32]). More
 70 recently, several models accounting for either only the fabric anisotropy (e.g., [33]), or both anisotropy and
 71 destructuration [34] have also been introduced. A shortcoming of these models is the absence of bounds for
 72 the evolution of rotational hardening variables which could eventually lead to an excessive rotation of the
 73 yield surface for loading at very high values of stress-ratio [35-36]. Furthermore, destructuration theories
 74 have so far only addressed isotropic destructuration (usually constituting a mechanism of isotropic softening
 75 of the yield surface with destructuration), neglecting frictional destructuration.

76 In this paper, a new Elasto-ViscoPlastic Simple ANIsotropic CLAY plasticity (EVP-SANICLAY) model is
 77 proposed. The model is a new member of the SANICLAY family of models, which are based on the classical
 78 modified Cam-Clay model and include rotational hardening and destructuration features for simulation of
 79 anisotropy and sensitivity, respectively. Perzyna's overstress theory [24-25] is employed to account for soil
 80 viscosity effects. Being based on the SANICALY model, the new viscoplastic model restricts the rotation to
 81 within bounds necessary to guarantee the existence of real-valued solutions for the analytical expression of
 82 the yield surface [12]. In the following sections, the theoretical formulation of the model will be discussed,
 83 followed by the details of its numerical implementation based on an algorithm proposed by Katona [28]. The
 84 validation of the new model is done by comparing the model simulation results against several experimental
 85 data at the element level and also field measurements for a boundary value problem. In particular, at element
 86 level the measured behavior observed from CRS and undrained triaxial tests over a number of different soft
 87 clays are used. Within these examples, determination of model parameter values is also discussed. For the
 88 boundary value problem, a well-studied test embankment, namely St. Alban embankment, is modeled and the
 89 predicted deformations using the EVP-SANICLAY model are compared with the recorded *in-situ* values. In
 90 order to better highlight the merits of the newly proposed constitutive model, the simulation results are also
 91 compared with those obtained using the MCC model, the SANCILAY model, and also the EVP-SANICLAY model
 92 but without the destructuration feature. Note that in this paper all stress components are effective stresses
 93 and as usual in geomechanics, both stress and strain quantities are assumed positive in compression.

94

95 2 EVP-SANICLAY

96 2.1 Model formulation

97 According to Perzyna's theory, the total strain increment, $\Delta\boldsymbol{\varepsilon}$, associated with a change in effective stress,
 98 $\Delta\boldsymbol{\sigma}$, during a time increment of Δt , is additively decomposed to elastic and viscoplastic parts

$$\Delta\boldsymbol{\varepsilon} = \Delta\boldsymbol{\varepsilon}^e + \Delta\boldsymbol{\varepsilon}^{vp} \quad (1)$$

99 where the superscripts *e* and *vp* represent the elastic and the viscoplastic components, respectively. The
 100 elastic strain increment, $\Delta\boldsymbol{\varepsilon}^e$, is time-independent; whereas, the viscoplastic strain increment, $\Delta\boldsymbol{\varepsilon}^{vp}$, is

101 irreversible and time-dependent. Adopting the isotropic hypoelastic relations for simplicity [12], the elastic
 102 part of the total strain can be shown as

$$\Delta \boldsymbol{\varepsilon}^e = \mathbf{D}^{-1} : \Delta \boldsymbol{\sigma} \quad (2)$$

103 where \mathbf{D} is the elastic stiffness matrix with more details presented in the Appendix, and symbol $:$ implies
 104 the trace of the product of two tensors.

105 The time-dependent viscoplastic strain increment is evaluated as

$$\Delta \boldsymbol{\varepsilon}^{vp} = \dot{\boldsymbol{\varepsilon}}^{vp} \cdot \Delta t \quad (3)$$

106 where $\dot{\boldsymbol{\varepsilon}}^{vp}$ is the viscoplastic strain rate tensor (a superposed dot denotes the time derivative), and following
 107 the original proposal by Perzyna [24-25], it can be defined as

$$\dot{\boldsymbol{\varepsilon}}^{vp} = \mu \cdot \langle \Phi(F) \rangle \cdot \frac{\partial g}{\partial \boldsymbol{\sigma}} \quad (4)$$

108 where μ is referred to as the fluidity parameter, g is the viscoplastic potential function represented by the
 109 dynamic loading surface (DLS - explained in the sequel), and $\Phi(F)$ is the so-called overstress that is the
 110 normalised distance between the current static yield surface (SYS) and the DLS (see Figure 1). The
 111 application of Macauley brackets in Equation (4) ensures that

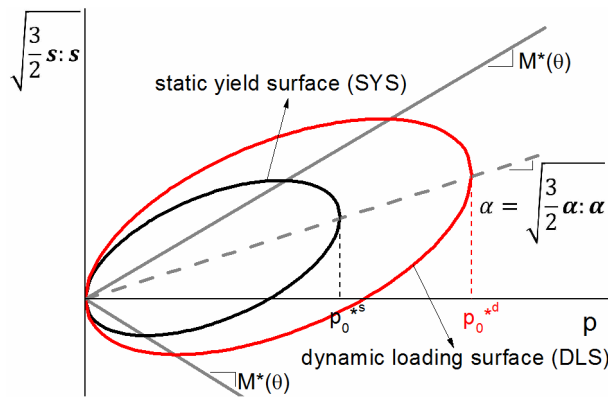
$$\langle \Phi(F) \rangle = \begin{cases} 0 & \text{for } \Phi(F) \leq 0 \\ \Phi(F) & \text{for } \Phi(F) > 0 \end{cases} \quad (5)$$

112 Several different relationships for $\Phi(F)$ have been proposed in the literature (e.g. [26,37]). In this work
 113 the following exponential function proposed by Fodil et al. [38] is employed

$$\Phi(F) = \exp(F) - 1 = \exp \left[N \left(\frac{p_0^d}{p_0^s} - 1 \right) \right] - 1 \quad (6)$$

114 where p_0^s and p_0^d are the size of the SYS and the DLS, respectively (see Figure 1), N is the strain-rate
 115 coefficient that together with μ are the two viscous parameters of this model.

116



117

118 Figure 1. Graphical representation of the EVP-SANICLAY model in the stress space

119

120 This specific choice of $\Phi(F)$ ensures that its value is always greater or equal to zero. Thus, from Equation
 121 (7) it is evident that if the stress state lies on or inside the SYS, the soil response would be purely elastic. If the

122 stress state lies outside the SYS, viscoplastic strain will be developed proportional to its distance from the
 123 current SYS.

124 In this work the elliptical surface of the SANICLAY model [12] is adopted as the SYS. The SANICLAY
 125 model was originally proposed with a non-associated flow rule; however, for simplicity purposes an
 126 associated flow rule is adopted here for its elastic-viscoplastic extension. In the general stress space, the SYS
 127 function can be expressed as

$$f^s = \frac{3}{2}(\mathbf{s} - p\boldsymbol{\alpha}) : (\mathbf{s} - p\boldsymbol{\alpha}) - \left(M^*(\theta)^2 - \frac{3}{2} \boldsymbol{\alpha} : \boldsymbol{\alpha} \right) (p_0^{*s} - p) p = 0 \quad (7)$$

128 In the above expression, $\mathbf{s} = \boldsymbol{\sigma} - p\mathbf{I}$ is the deviatoric component of stress tensor $\boldsymbol{\sigma}$ (\mathbf{I} being the fourth order
 129 identity tensor). $\boldsymbol{\alpha}$ is the deviatoric fabric tensor that accounts for anisotropy by coupling the deviatoric and
 130 volumetric plastic strain rates. $p_0^{*s} = S_i p_0^s$ defines the size of the structured SYS where $S_i \geq 1$ is an isotropic
 131 destructuration factor and p_0^s is the size of the intrinsic SYS. $M^*(\theta) = S_f M(\theta)$ where $S_f \geq 1$ is a frictional
 132 destructuration factor and $M(\theta)$ is the critical stress-ratio that in the general stress space its value is
 133 interpolated between M_c and M_e by means of a Lode angle θ . In the stress space illustrated in Figure 1 the
 134 scalar $\alpha = \sqrt{(3/2) \boldsymbol{\alpha} : \boldsymbol{\alpha}}$ defines the rotation of the SYS and DLS. As shown in Figure 1, the DLS has the same
 135 shape and orientation as the smaller SYS, and following the adoption of associate flow rule it coincides the
 136 viscoplastic potential surface too. A summary of the hardening equations and the Lode angle formulation are
 137 presented in the Appendix for the sake of completeness.

138 The model constants of EVP-SANICLAY can be divided into 4 categories: (1) the elasticity constants κ
 139 and ν and the critical state constants M_c , M_e and λ which are the same as those in the MCC model (with the
 140 exception that in MCC we have $M_e = M_c$); (2) the rotational hardening (RH) constants C and x , which are
 141 specific to the SANICLAY model; (3) the destructuration constants k_i , k_f and A ; and (4) the viscosity
 142 parameters N and μ , which constitute the two new additional parameters of the EVP-SANICLAY and they can
 143 be determined as discussed in Yin and Hicher [31]. Furthermore, similar to the SANICLAY, α and p_0^s constitute
 144 the hardening internal variables in the EVP-SANICLAY model. It should be noted that despite the large
 145 number of model parameters, they have clear physical meaning and can be determined following
 146 straightforward processes. The detailed procedure for evaluating the initial values of the model state
 147 variables, and hardening and destructuration parameters can be found in Taiebat et al. [13].

148

149 2.2 Numerical Integration

150 The numerical solution algorithm for the elasto-viscoplastic model can be developed by using a step-by-
 151 step time integration algorithm with a Newton-Raphson iteration procedure [28]. In this scheme it is
 152 assumed at the beginning of a certain defined time interval and strain increment, the values of stresses,
 153 strains, and state variables are known. The objective is to determine the subsequent elastic and viscoplastic
 154 strain components, which in turn allow finding the subsequent stresses and internal variables. From
 155 Equations (1,2) the incremental constitutive relationship for a time step can be expressed as

$$\Delta\boldsymbol{\sigma} = \mathbf{D}(\Delta\boldsymbol{\varepsilon} - \Delta\boldsymbol{\varepsilon}^{vp}) \quad (8)$$

156 For approximation of $\Delta\boldsymbol{\varepsilon}^{vp}$, a finite difference scheme is employed as:

$$\Delta\boldsymbol{\varepsilon}^{vp} = \Delta t[(1 - \beta)\dot{\boldsymbol{\varepsilon}}_t^{vp} + \beta\dot{\boldsymbol{\varepsilon}}_{t+\Delta t}^{vp}] \quad (9)$$

157 where $\dot{\boldsymbol{\varepsilon}}_t^{vp}$ is the value of viscoplastic strain rate at time t , and β is a time interpolation parameter ($0 \leq \beta \leq$
 158 1); $\beta = 0$ represents an explicit forward (Euler) interpolation, $\beta = 0.5$ represents central (Crank-Nicolson)
 159 interpolation, and $\beta = 1$ implies an implicit backward interpolation. Lewis and Schrefler [39] showed that in
 160 this scheme the solution is conditionally stable for $0 \leq \beta < 0.5$ and $\beta = 1$, and unconditionally stable for
 161 $0.5 \leq \beta < 1$. Substituting Equation (9) into Equation (8) and rearranging the terms give:

$$\mathbf{D}^{-1}\boldsymbol{\sigma}_{t+\Delta t} + \Delta t \cdot \beta \cdot \dot{\boldsymbol{\varepsilon}}_{t+\Delta t}^{vp} = \Delta\boldsymbol{\varepsilon} - \Delta t \cdot (1 - \beta)\dot{\boldsymbol{\varepsilon}}_t^{vp} + \mathbf{D}^{-1}\boldsymbol{\sigma}_t \quad (10)$$

162 where the terms on the right hand side are known (at time t), while the left hand side terms are unknowns
 163 (at time $t + \Delta t$) and they are to be solved in an iterative procedure. A Modified Newton-Raphson approach is
 164 used for the iterative solution of Equation (10). To do this, a limited Taylor series is applied to the unknown
 165 quantities $\boldsymbol{\sigma}_{t+\Delta t}$ and $\dot{\boldsymbol{\varepsilon}}_{t+\Delta t}^{vp}$:

$$\begin{cases} \boldsymbol{\sigma}_{t+\Delta t} = \boldsymbol{\sigma}_i + d\boldsymbol{\sigma}_i & (11a) \\ \dot{\boldsymbol{\varepsilon}}_{t+\Delta t}^{vp} = \dot{\boldsymbol{\varepsilon}}_i^{vp} + \frac{\partial \dot{\boldsymbol{\varepsilon}}_i^{vp}}{\partial \boldsymbol{\sigma}} d\boldsymbol{\sigma}_i & (11b) \end{cases}$$

166

167 Note that subscript i refers to the i -th iteration at the current time step. Substituting Equations (11a)
 168 and (11b) into Equation (10) and successive rearrangements result in the following form for computation of
 169 stress increment:

$$d\boldsymbol{\sigma}_i = \left[\mathbf{D}^{-1} + \Delta t \cdot \beta \cdot \frac{\partial \dot{\boldsymbol{\varepsilon}}_i^{vp}}{\partial \boldsymbol{\sigma}} \right]^{-1} : [(\Delta\boldsymbol{\varepsilon} - \Delta t \cdot (1 - \beta)\dot{\boldsymbol{\varepsilon}}_t^{vp} + \mathbf{D}^{-1} \cdot \boldsymbol{\sigma}_t) - (\mathbf{D}^{-1}\boldsymbol{\sigma}_i + \Delta t \cdot \beta \cdot \dot{\boldsymbol{\varepsilon}}_i^{vp})] \quad (12)$$

170 If it is assumed that function K represents the term $(\Delta\boldsymbol{\varepsilon} - \Delta t \cdot (1 - \beta)\dot{\boldsymbol{\varepsilon}}_t^{vp} + \mathbf{D}^{-1}\boldsymbol{\sigma}_t)$ with known
 171 quantities remaining constant during the iteration, and that function U represents the iterative term
 172 $(\mathbf{D}^{-1}\boldsymbol{\sigma}_i + \Delta t \cdot \beta \cdot \dot{\boldsymbol{\varepsilon}}_i^{vp})$, then Equation (12) can be presented in a short form as:

$$d\boldsymbol{\sigma}_i = \left[\frac{\partial U}{\partial \boldsymbol{\sigma}} \right]^{-1} \cdot [K_t - U_i] \quad (13)$$

173 The most efficient solution scheme for continuum problems using overstress-type elasto-viscoplastic
 174 constitutive equations can be obtained with $\beta = 0.5$ [40]; hence, this value is adopted for the time
 175 interpolation parameter in the present work. For the solution algorithm, at every time step Equation (13) is
 176 iteratively solved. At each iteration $d\boldsymbol{\sigma}_i$ is calculated and subsequently $\boldsymbol{\sigma}_i$ is updated as $\boldsymbol{\sigma}_i = \boldsymbol{\sigma}_{i-1} + d\boldsymbol{\sigma}_i$.
 177 When convergence is achieved (i.e. when $d\boldsymbol{\sigma}_i < \text{tolerance} \sim 10^{-7}$), the iterative procedure stops and the
 178 incrementally accumulated stress values will become the stresses at the corresponding time step (i.e., $\boldsymbol{\sigma}_{t+\Delta t}$);
 179 subsequently, viscoplastic strain tensor can be calculated as $\boldsymbol{\varepsilon}_{t+\Delta t}^{vp} = \boldsymbol{\varepsilon}_{t+\Delta t} - \mathbf{D}^{-1}\boldsymbol{\sigma}_{t+\Delta t}$. The implementation
 180 makes it possible to apply the whole strain increment through a number of sub-increments, not all at once.
 181 After the completion of the integration process at a time increment the procedure advances to the next time
 182 step.

183 The EVP-SANICLAY model has been implemented into PLAXIS finite element program as a user-defined
 184 soil model in order to be used for both element level and boundary value problem simulations. In the
 185 following, first the performance of the model is validated by simulation of a number of element test data on
 186 various clays. The model is then used for settlement study of a real instrumented test embankment and the
 187 simulation results are discussed in detail. The embankment simulation also aims to compare details of the
 188 predicted response using the proposed model and also using an isotropic and rate-independent model that is
 189 often used in practice.

191 3 Model validation based on element level tests

192 For the element test simulations the implemented user-defined model has been employed through the
 193 PLAXIS Soil Test application [41] to simulate several undrained triaxial shear and CRS test data on four
 194 different soft soils reported in the literature, namely Kawasaki clay, Haney clay, St. Herblain clay, and Batiscan
 195 clay [14,16,42-43]. The values of model constants and state variables used for the four soil types analysed in
 196 this paper are summarised in Tables 1 and 2. In accordance with the natural or reconstituted state of the clay
 197 sample being simulated the destructuration feature of the model has been switched on or off by setting
 198 respective values to the structuration parameters.

200 Table 1. Constants of the EVP-SANICLAY model adopted for four types of clays

Model constant		Kawasaki	Haney	St. Herblain	Batiscan
Elasticity	κ	0.021	0.05	0.022	0.037
	ν	0.2	0.25	0.3	0.3
Critical state	$M_c (M_e)$	1.65 (1.24)	1.28	1.25	0.98
	λ	0.16	0.32	0.41	0.41
Rotational hardening	C	12	12	10	12
	x	2.6	2.4	1.5	1.7
Destructuration	k_i	0	1.5	0	1.4
	k_f	0	1.4	0	1.3
	A	0	0.3	0	0.5
Viscosity	N	12	17	9	12
	$\mu [s^{-1}]$	7×10^{-6}	5×10^{-11}	5×10^{-9}	2×10^{-9}

202 Table 2. Initial values of state variables adopted for four types of clays

Model state variable		Kawasaki	Haney	St. Herblain	Batiscan
Initial void ratio	e	1.07	2	2.26	1.92
Initial size of the SYS	p_0 [kPa]	250	340	30	50
Initial rotation of the SYS	α	0.60	0.43	0.46	0.36
Initial isotropic structuration factor	S_i	1	6	1	3
Initial frictional structuration factor	S_f	1	1.3	1	1.5

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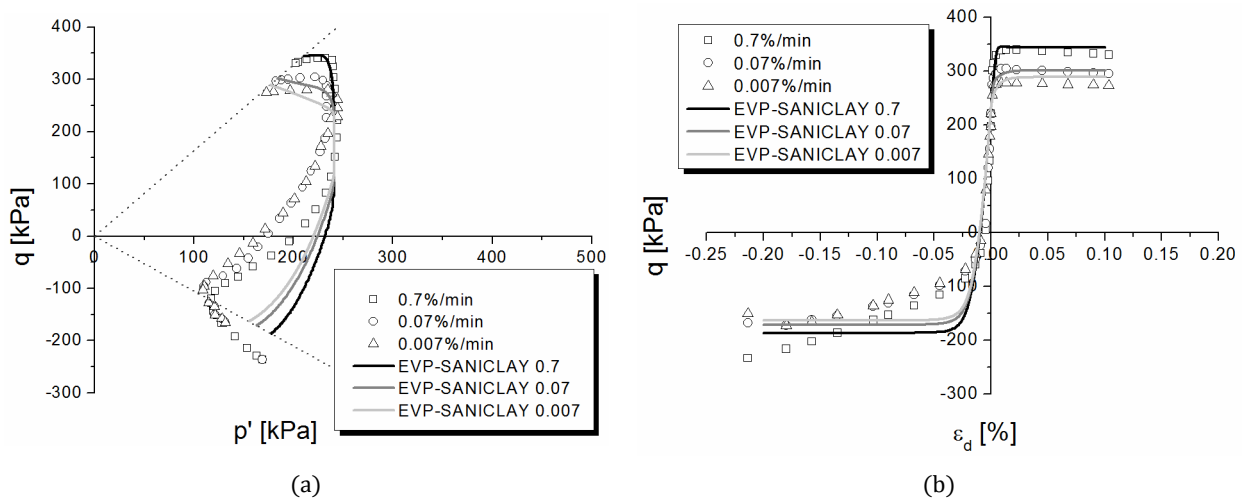
204 **3.1 Kawasaki clay**

205 To evaluate the strain-rate dependency, Nakase and Kamei [42] performed undrained triaxial
 206 compression and extension tests with various shearing rates on anisotropically consolidated reconstituted
 207 Kawasaki clay specimens. The index properties of Kawasaki clay samples were reported as plasticity index
 208 $I_p = 29.4$, specific gravity $G_s = 2.69$, liquid limit $w_L = 55.3\%$, plastic limit $w_p = 25.9\%$, and clay content
 209 22.3%. All tests were conducted under a vertical effective consolidation pressure of 392 kPa with a back-
 210 pressure of 196 kPa in the consolidation stage. The samples were consolidated under a K_0 value of 0.42, and
 211 then the samples were sheared in both compression and extension with axial strain rates of 0.7, 0.07, and
 212 0.007%/min.

213 Kamei and Sakajo [44] reported the values of conventional soil parameters, such as κ , λ , M and initial
 214 void ratio, for the samples of Kawasaki clay. Based on the test data, the critical stress ratio in triaxial
 215 compression and extension were measured as 1.65 and 1.24, respectively. Rotational hardening parameters
 216 were determined according to Dafalias et al. [12]. For the simulations, the destructuration feature of the
 217 model was switched off by setting $S_i = S_f = 1$, as the soil specimens were reconstituted. Viscosity parameters
 218 were determined through calibration based on data from tests at only two strain-rates.

219 Figure 2 shows the comparison between experimental and numerical results obtained using the EVP-
 220 SANICLAY model. The tests were simulated considering the consolidation stage. As it is seen in the figure, the
 221 response during triaxial compression has been captured very well by the model, while for the extension part
 222 the results are less accurate, even though the Lode angle dependency was considered in order to better
 223 reproduce the clay behavior in extension. As illustrated in Figure 3 of Taiebat et al. [13] this could be in part
 224 due to adoption of an associated flow rule in the EVP model.

225



226 Figure 2. Undrained triaxial test: (a) effective stress path; (b) deviator stress versus axial strain

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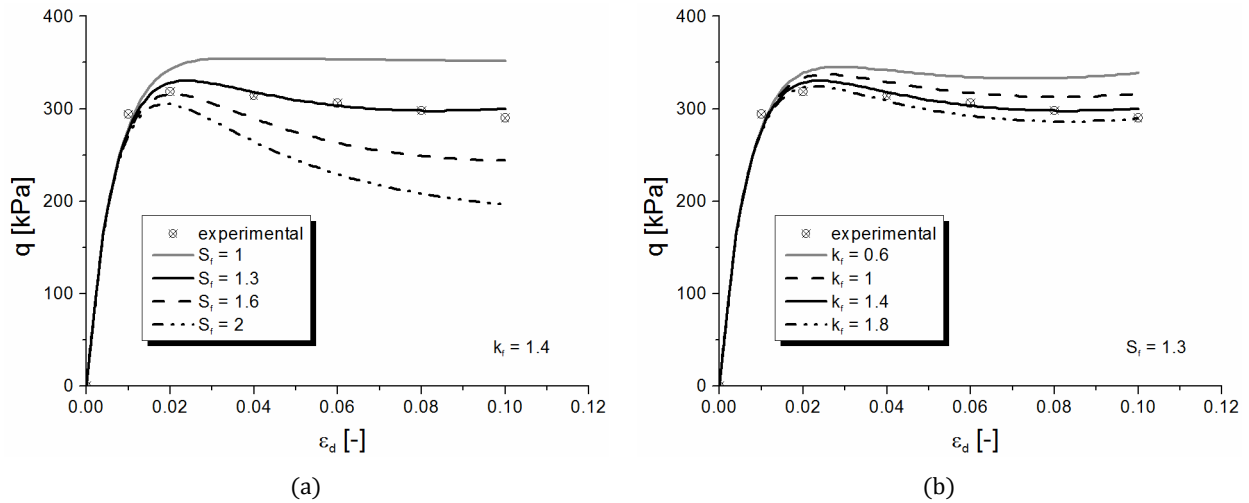
228 It can be noted that in triaxial compression, a better comparison between experimental and numerical
 229 results is achieved for higher strain rates. As the strain rate decreases, the numerical stress paths tend to be

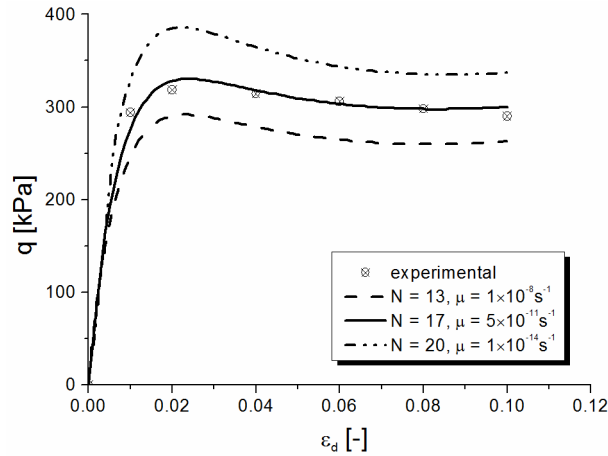
230 more lenient towards the critical state. As no destructuration was considered for this simulation and also
 231 associated flow rule was employed, the modeling results did not reproduce a noticeable softening behavior
 232 (Figure 2b). This is observed in both compression and extension. The initial stiffness of the curve was also
 233 well represented. The Lode angle dependency of the model allows capturing the anisotropy in strength as it
 234 was observed by Nakase and Kamei [42].
 235

236 3.2 Haney clay

237 Vaid and Campanella [14] carried out undrained triaxial tests on undisturbed saturated sensitive marine
 238 clay known as Haney clay. It is a silty clay with $w_L = 44\%$, $w_p = 26\%$ and a sensitivity within the range of 6 to
 239 10. All test samples were normally consolidated, with an all-around confining pressure of 515 kPa.
 240 Consolidation was allowed for a period of 36 hr after which the samples were left undrained for 12 hr under
 241 the consolidation stresses prior to shearing. In order to study the rate dependency of undisturbed clay
 242 response, the undrained shearing stage of the tests was performed at different constant strain rates, varying
 243 from 10^{-3} to 1.1% /min.

244 Values of conventional soil parameters listed in Tables 1 and 2 were reported by Vermeer and Neher
 245 [45]. After determination of the initial value of α , the values of anisotropy constants, C and x , were obtained
 246 via curve fitting. Destructuration parameter values were also calibrated via trial runs. Structuration factor
 247 and destructuration constants influence the softening behavior after peak strength, and to a lesser degree, the
 248 shear strength achieved. Figure 3a and 3b show the influence of frictional destructuration in soft clay
 249 behavior. An increase of the frictional structuration factor leads to a larger softening behavior and a
 250 noticeable decrease in shear strength (Figure 3a).
 251





(c)

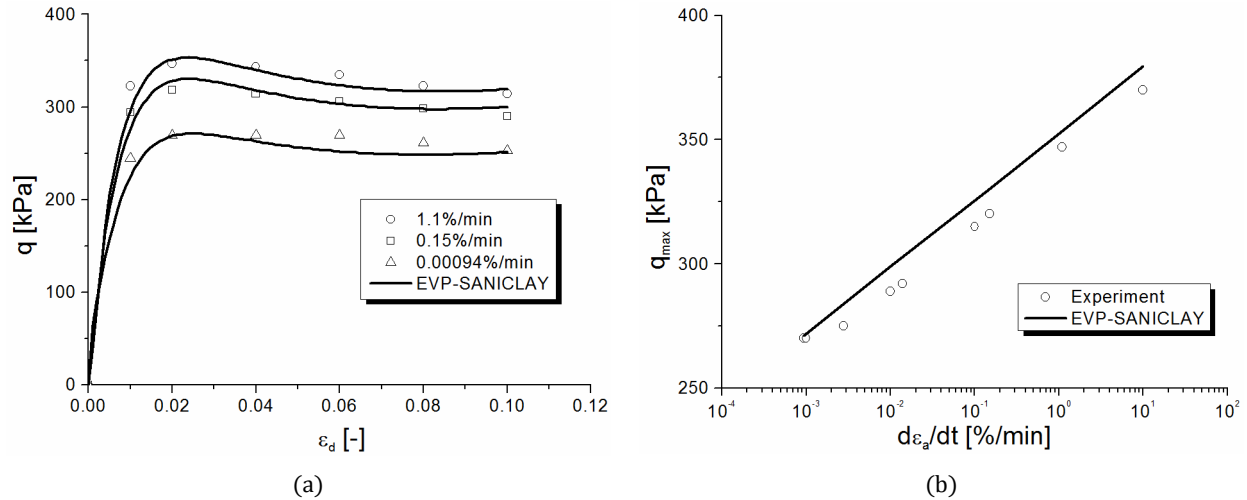
252 Figure 3. Calibration of structuration and viscosity parameters for Haney clay: (a) influence of frictional
 253 structuration parameter S_f for a constant rate $k_f = 1.4$; (b) influence of rate of frictional destructuration k_f
 254 for a constant value of $S_f = 1.3$; (c) combined influence of viscosity parameters N and μ

255

256 A similar, if less marked, behavior is seen in relation to the rate of frictional destructuration (Figure 3b),
 257 with larger softening observed for higher destructuration rates. Viscosity parameter values were calibrated
 258 based on the results of two tests (i.e. at two strain rates) only. As it is shown in Figure 3c, viscosity
 259 parameters play an important role in the overall calibration of the model, particularly with regards to the
 260 shear strengths achieved. In order to obtain an improved match with the experimental results, instead of the
 261 default value of 0.5, a value of 0.3 was adopted for the destructuration parameter A .

262 For model simulations using EVP-SANICLAY, three specific strain rates, at 0.00094%/min, 0.15%/min
 263 and 1.1%/min, have been taken into account to reproduce the observed shear stress-shear strain curves. Also
 264 the peak strengths achieved at different strain rates were considered to evaluate the model performance. The
 265 experimental versus numerical results are shown in Figure 4. It can be seen from the figure that the model
 266 simulations compare very well with the observed behavior. The model, with its destructuration function on,
 267 is able to simulate the softening behavior of natural clay response after peak (Figure 4a). Also Figure 4b
 268 indicates that the model provides a reasonably good representation for the variations of maximum shear
 269 strength with loading rate.

270



271 Figure 4. Undrained triaxial compression tests: (a) deviator stress versus axial strain; (b) evolution of
 272 maximum deviator stress with strain rate

273

274 3.3 St. Herblain clay

275 A particular CSR oedometer test was performed by [Rangeard \[43\]](#) on St. Herblain clay, a clayey river
 276 alluvial deposit. Two different strain rates were considered during the test. The test was started with a strain
 277 rate of $3.3 \times 10^{-6} \text{s}^{-1}$ until an axial strain of 12%, at this strain the loading rate was lowered to a strain rate of
 278 $6.6 \times 10^{-7} \text{s}^{-1}$ and was kept at that until a vertical strain of 15.5%, then again the rate was switched back to the
 279 initial strain rate and was kept constant until the end of the test.

280 Soil parameter values, obtained from oedometer and triaxial tests, were also reported by [Rangeard \[43\]](#).
 281 The clay sample used for the experiments was taken from a depth of 6.5–7.5 m, it had a bulk unit weight
 282 $\gamma = 14.85 \text{ kN/m}^3$ and a water content of 87%. A vertical pre-consolidation pressure of 52 kPa was
 283 determined from the oedometer tests. The model parameters adopted are summarised in Tables 1 and 2.

284 Given that the clay was slightly structured, for the simulations the destructuration feature of the model
 285 was switched off. Figure 5 shows the experimental data versus simulation results. It can be seen that the
 286 model predictions are in good agreement with the data, particularly with regards to vertical stresses. The
 287 model also captures the indentation due to the change in strain-rate during the test.

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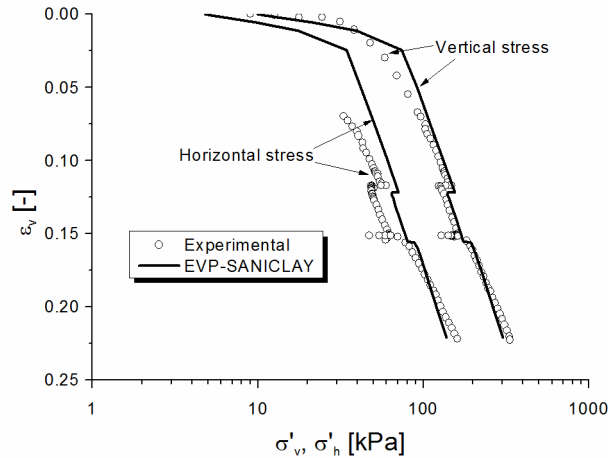


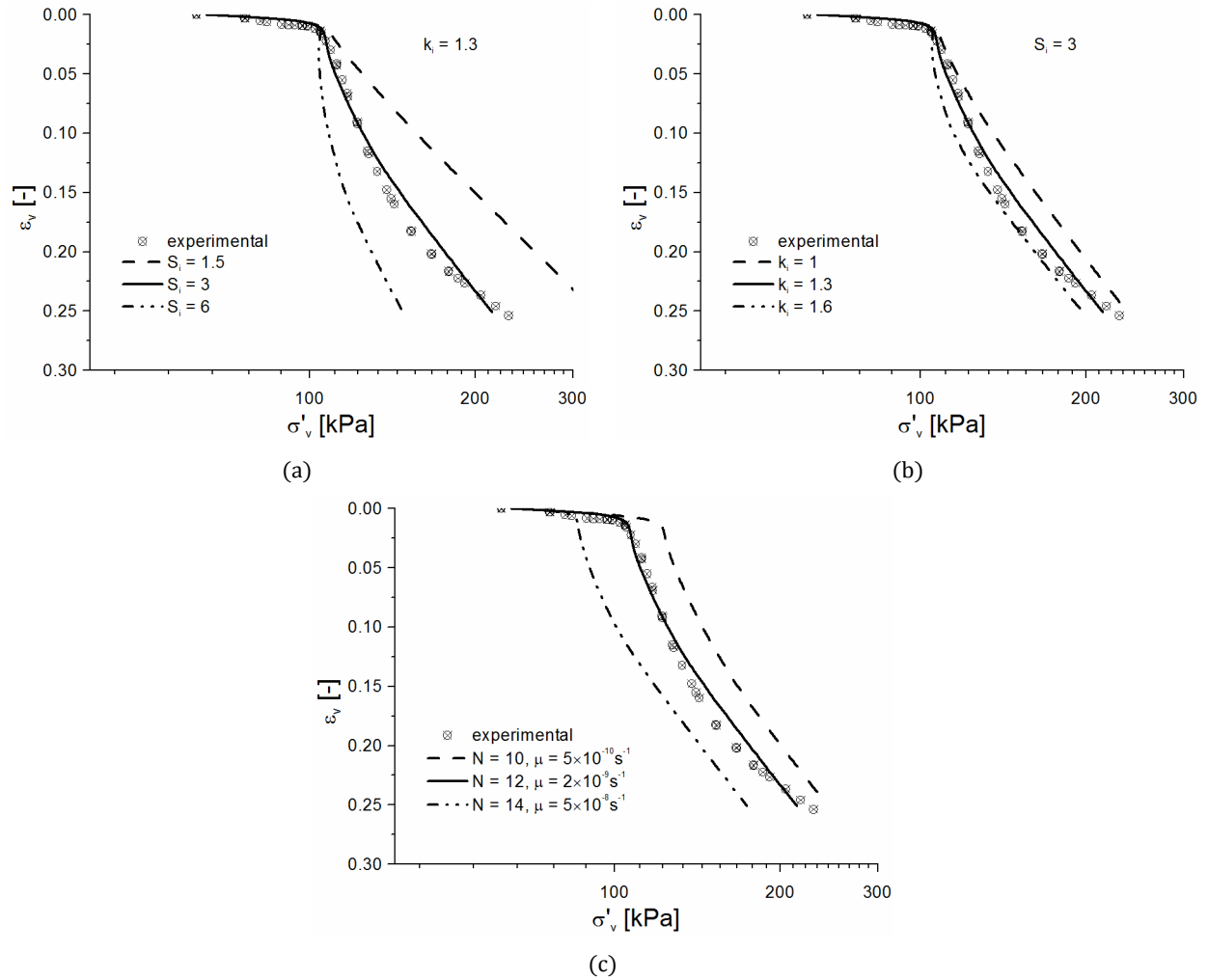
Figure 5. Simulations of CRS oedometer test results over St. Herblain clay

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291 3.4 Batiscan clay

292 CSR oedometer tests were performed on Batiscan clay by [Leroueil et al. \[16\]](#). The clay samples were
 293 taken from a depth of 7.25 - 7.46 m; the samples reportedly had a water content of 80%, liquidity index $I_L =$
 294 2.7, $I_p = 21$, and $\gamma = 17.5 \text{ kN/m}^3$. The pre-consolidation pressure, determined from conventional oedometer
 295 tests, was evaluated as 88 kPa. The strain-rates for the CRS tests varied between $1.7 \times 10^{-8} \text{ s}^{-1}$ and $4 \times 10^{-5} \text{ s}^{-1}$.
 296 The initial vertical effective stress was taken equal to 65 kPa, corresponding to a size of the initial yield
 297 surface of 50 kPa. Conventional soil parameter values reported in Tables 1 and 2 were obtained from [Leroueil](#)
 298 [et al. \[16\]](#) and [Rocchi et al. \[46\]](#).

299 Combinations of initial degree of structuration and rate of destructuration have been studied and the
 300 best coupled values were chosen for the numerical simulations. As it is shown in Figure 6a, larger values of
 301 initial structuration S_i result in a larger reduction of final vertical stress due to the higher softening occurring.
 302 For a constant value of S_i , the value of the rate of destructuration does not appear to have as much influence,
 303 but it follows the same trend (Figure 6b), with higher rates leading to a higher vertical stress reduction.
 304 Viscosity parameters are typically obtained from long-term oedometer tests via curve fitting. The calibration
 305 of the coupled values is showed in Figure 6c. Note that viscosity parameters greatly change the stress value at
 306 the end of the initial stiff elastic regime. The calibrated model parameter values are summarised in Tables 1
 307 and 2.



308 Figure 6. Calibration of structuration and viscosity parameters for Batiscan clay: (a) influence of isotropic
 309 structuration factor S_i for a constant rate $k_i = 1.3$; (b) influence of rate of isotropic destructuration k_i for a
 310 constant structuration value of $S_i = 3$; (c) combined influence of viscosity parameters N and μ
 311

312 Model simulations using EVP-SANICLAY are shown in Figure 7. It is seen that a good correlation is obtained
 313 between the numerical results and experimental data. Also clearly the strain-rate effects are well captured;
 314 the exponential trend of the curves indicates the progress of destructuration at large strains.
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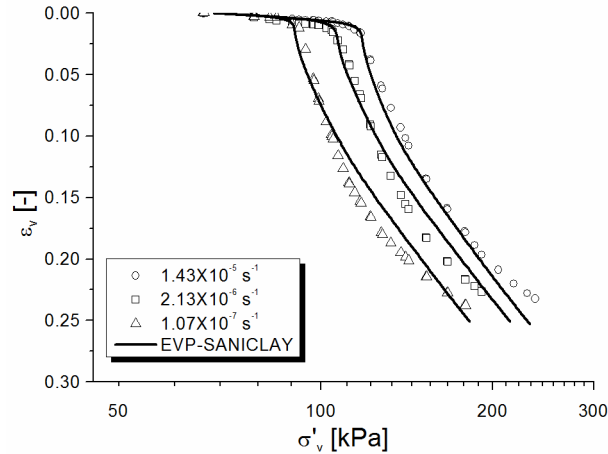


Figure 7. Oedometer test results: vertical strain versus vertical stress

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Considering that all above element test simulations performed using EVP-SANICLAY, it appears that in addition to the anisotropy and destructuration effects, the model is able to reasonably capture the strain-rate dependency in behavior of natural clays. Also for the simulations performed above, the model implementation proved to be sufficiently robust.

323 4 Boundary value level modeling

324 In order to study the performance of the proposed elasto-viscoplastic constitutive model at the
325 boundary value level, the simulation of a test embankment was carried out. In particular, embankment D of a
326 set of four test embankments built on a soft, sensitive and cemented clay in Saint-Alban, Quebec, Canada was
327 selected [47-48]. This is a well-known and well-instrumented embankment for which soil parameters are
328 readily available in the literature.

329 4.1 Model description

330 Embankment D has a height of 3.28m, a uniform crest width of 7.6m and slope angles of 13.75°. The
331 embankment material consists of uniform medium sand compacted to a unit weight of 18.56 kN/m³. It was
332 constructed on 13.7 m deep natural clay deposit known as Champlain clay, underlain by a dense fine to
333 medium sand layer down to a depth of 24.4m [49]. The soft deposit is overlain by approximately 1.5 m thick
334 weathered crust. In order not to disturb the very soft and sensitive clay deposit at the site, the embankment
335 was built directly on the existing natural ground, without excavating the thin dry crust layer at the top. In this
336 work a two-dimensional plane strain finite element model of the embankment was created using PLAXIS AE
337 [41], and taking advantage of the symmetry, only half of the embankment was modeled. A finite element mesh
338 with 1723 15-noded triangular elements (Figure 8.a) was used for the analyses. Each element has pore water
339 pressure (PWP) degrees of freedom at corner nodes. Mesh sensitivity studies were carried out to ensure that
340 the mesh was dense enough to produce accurate results. The geometry of the finite element model is shown

341 in [Figure 8a](#). The far right boundary of the model was assumed at distance of 40m from the embankment
342 centerline. The bottom boundary of the clay deposit was assumed to be completely fixed in both horizontal
343 and vertical directions, whereas, the left and right vertical boundaries were only restrained horizontally.
344 Drainage was allowed at the ground level, while the bottom boundary was considered impermeable.
345 Impermeable drainage boundaries were also assigned to the lateral boundaries. Based on ground data, the
346 water table was assumed at 0.7m depth.

347 The embankment was built in stages, with an initial layer of 0.6m and after 6 days the normal
348 construction began ([Figure 8.b](#)), with an average rate of 0.24m/day [\[48\]](#). The same construction pace was
349 adopted in the numerical model. For the calculation phases, plastic analyses were carried out corresponding
350 to the construction process of the embankment, after which the consolidation analysis was performed.

351 For the numerical analysis, the embankment itself was modeled with the simple linear elastic-perfectly
352 plastic Mohr-Coulomb model using the following reported values for the embankment material: Young's
353 modulus $E = 40,000$ kPa, Poisson's ratio $\nu = 0.3$, friction angle $\phi = 44^\circ$, and cohesion $c = 0$ kPa. The dry crust
354 layer above the water table was also modeled with the Mohr-Coulomb model using shear modulus $G = 880$
355 kPa, Poisson's ratio $\nu = 0.3$, $\phi = 27^\circ$, $c = 1$ kPa. Unit weight $\gamma = 19$ kN/m³ is used for both [\[47,50\]](#). The
356 sensitive Champlain clay deposit below the water table was modeled using the implemented user-defined
357 EVP-SANICLAY model, with a unit weight $\gamma = 16$ kN/m³ [\[47\]](#).

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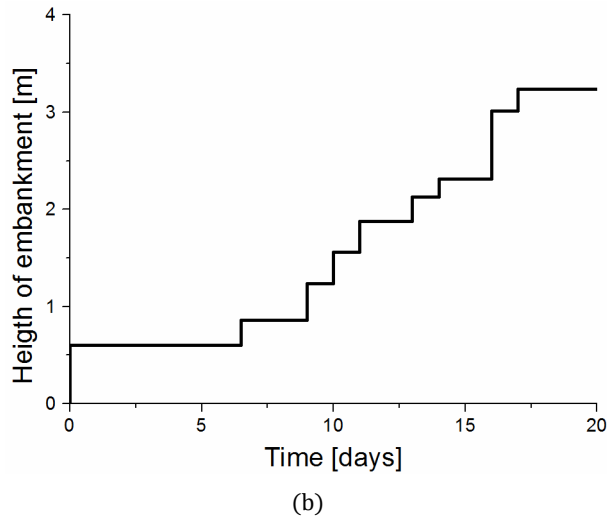
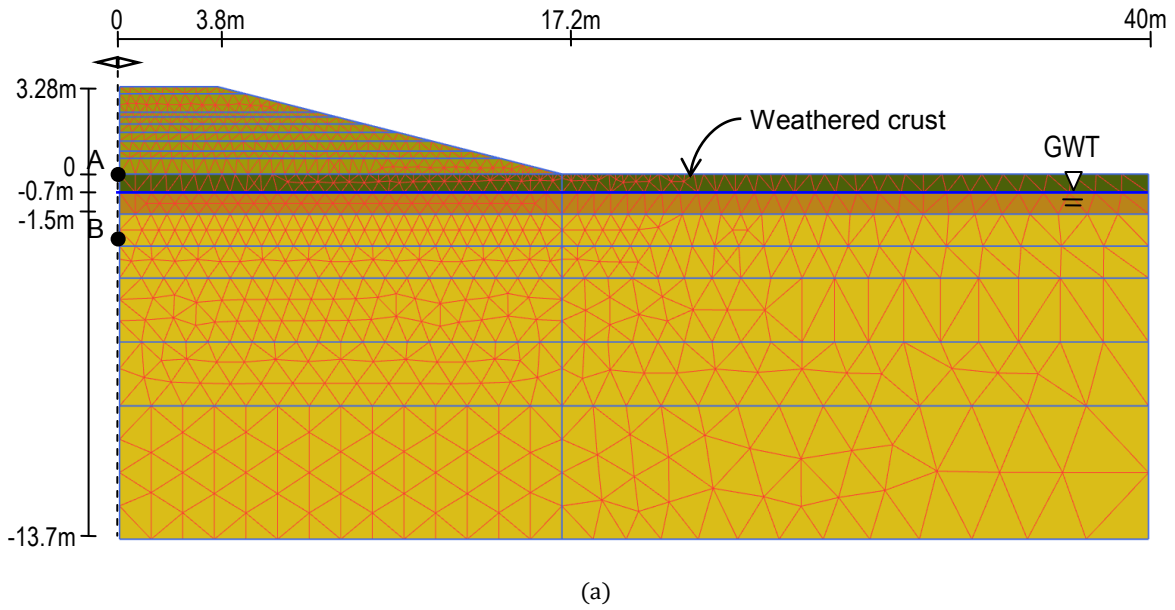


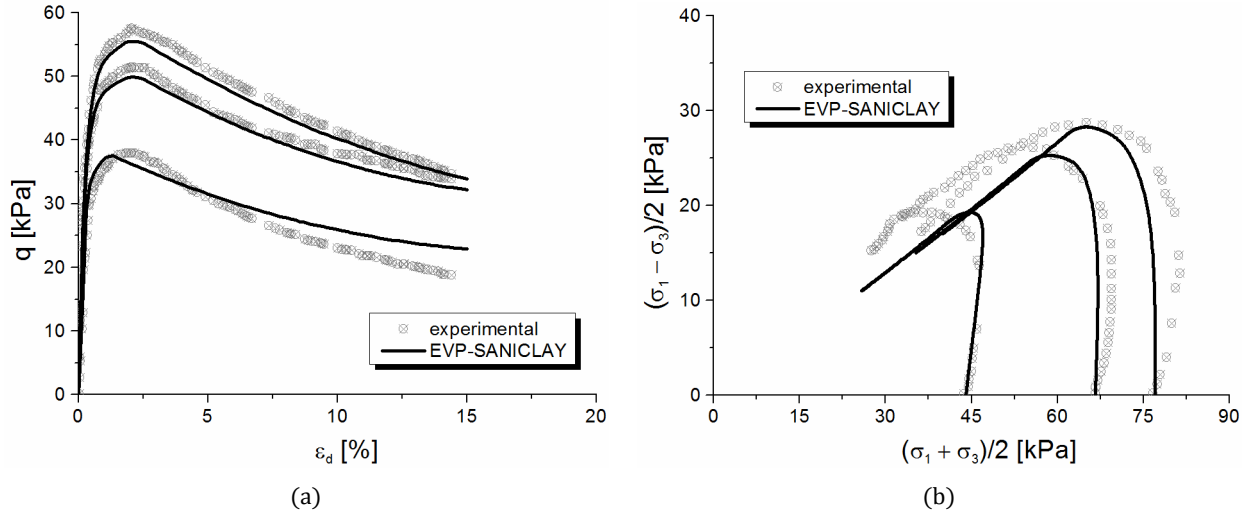
Figure 8. (a) Geometry of the model embankment and the finite element mesh adopted; (b) construction history of the St. Alban embankment D

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361 The material parameter values for the Champlain clay layers were determined using the available data
362 obtained from testing of samples taken at a depth of 6m below the ground surface [15]. Conventional
363 parameter values were derived from existing studies based on soil element test results [47-48,50]. Similar to
364 the section on element level simulations, the anisotropy parameter values were determined following the
365 approach proposed by Dafalias et al. [12]. The destructuration parameters were calibrated against
366 experimental data available for undrained triaxial compression tests over samples of Champlain clay taken

367 from a depth of approximately 3m [51]. For three tests presented in Figure 9 the samples were first
 368 isotropically consolidated up to three different pre-consolidation pressures of 44, 66.6 and 77 kPa, and
 369 subsequently sheared. Figure 9 shows a good agreement between the experimental data and the numerical
 370 simulations both in terms of stress-strain response and of stress paths. The destructuration trend after peak
 371 strengths was also well captured.

372



373 Figure 9. Validation of numerical simulations versus experimental results for undrained triaxial compression
 374 tests: (a) deviator stress versus axial strain; (b) effective stress paths

375

376 In the absence of appropriate soil test data, such as long-term oedometer tests with at least two different
 377 strain rates, viscosity parameters were calibrated using trial runs.

378

379

380 Table 3 summarises the soil and state parameters adopted for the simulation of St. Alban test
381 embankment, and Table 4 lists the calibrated anisotropy and destructuration parameter values. The
382 permeability, k , of the clay, assumed to be isotropic, was reported to be equal to 3.46×10^{-4} m/day. It should
383 also be added that the initial state of stress was generated by adopting K_0 -procedure [41] where the reported
384 K_0 value of 0.8 was employed [52]. Results from oedometer tests performed on Champlain clay reported that
385 over-consolidation ratio (OCR) varied between 1.8 and 2.2 [47]; a mean value of 2.0 was assumed for the
386 analyses performed here.

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Table 3 – Constants of the EVP-SANICLAY model adopted for St. Alban test embankment D

Model constant		Top Champlain clay layer (0.7-1.5 m)	Bottom Champlain clay layer (1.5-13.7 m)
Elasticity	κ	0.012	0.013
	ν	0.3	0.3
Critical state	M_c	1.07	1.07
	(M_e)		
Rotational hardening	λ	0.36	0.25
	C	10	10
Destructuration	x	1.7	1.7
	k_i	1.5	1.5
	k_f	1.4	1.4
Viscosity	A	0.5	0.5
	N	13	13
	μ [s^{-1}]	5×10^{-9}	5×10^{-9}

390

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Table 4 – Initial values of state variables adopted for St. Alban test embankment D

Model state variable		Top Champlain clay layer (0.7-1.5 m)	Bottom Champlain clay layer (1.5-13.7 m)
Initial void ratio	e	1.7	1.8
Overconsolidation ratio	OCR	2.0	2.0
Initial rotation of the SYS	α	0.41	0.41
Initial isotropic structuration factor	S_i	4.5	4.5
Initial frictional structuration factor	S_f	1.2	1.2

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In order to assess the performance of EVP-SANICLAY model, the finite element analysis of the embankment was repeated twice where instead of the EVP-SANICLAY model the MCC model and the EVP-SANICLAY model without destructuration (i.e., with $S_i = S_f = 1$) were used.

397 4.2 Simulations results

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The results from numerical analyses were compared with the available field measurement data for the time period following the construction [47,50,53]. Figure 10 shows settlement predictions versus time at the node on the centerline at the base of the embankment (point A in Figure 8a), using different models. From the figure it is clear that the proposed EVP-SANICLAY model gives a rather good prediction when compared to the *in-situ* measurements. When destructuration is switched off, the model significantly underestimates the settlement over time. The underestimation of settlement is even more pronounced with the MCC and SANICLAY models; in this case the predicted settlement reaches an approximately constant value after 400 days, pointing out that the model is clearly time-independent.

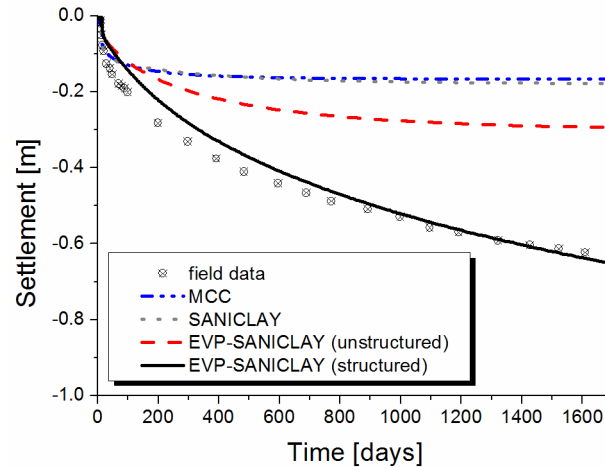


Figure 10. Time-settlement predictions versus field measurements at point A in Figure 8a

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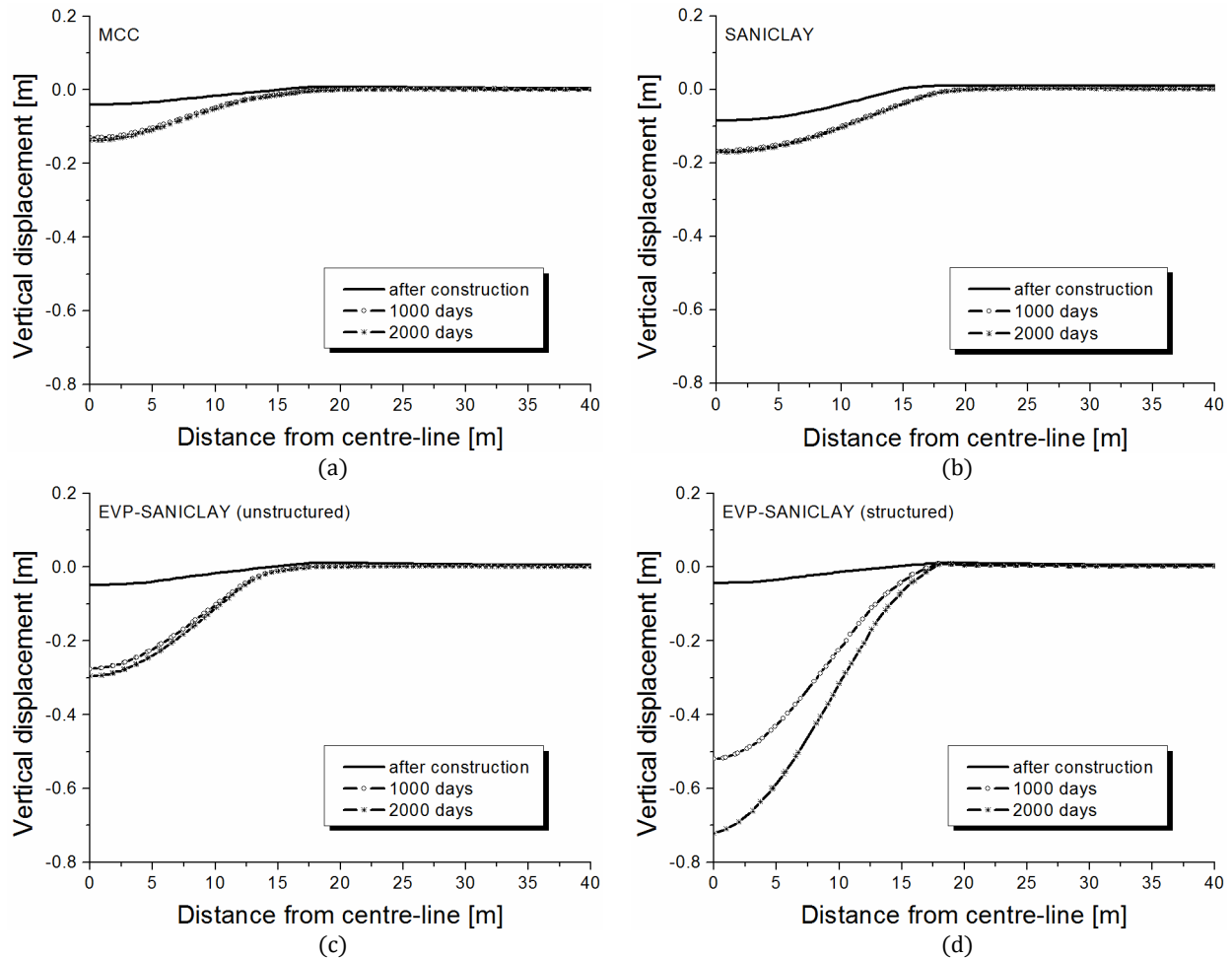
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No additional field data is available for surface settlements recorded at different times, but a comparison between the numerical results adopting different soil constitutive models can be made. Such numerical simulation results are shown in Figure 11. Generally they all show a typical behavior, with the main vertical settlements at the centerline of the embankment and diminishing values at larger distances from the centerline. However, as consideration of soil viscosity during plastic deformation delays the consolidation process, settlements through using EVP-SANICLAY (Figure 11d) represent more realistic deformation pattern with time. The simulation performed using the MCC and SANICLAY (Figure 11a,b) clearly shows that with the time-independent models the consolidation process completes rapidly after which the vertical deformation stops. When the effect of soil structure is ignored (Figure 11c) a behavior similar to the complete EVP-SANICLAY model is obtained, but with significantly lower values for the vertical settlement. This is expected, given that Champlain clay is highly structured clay with a sensitivity value of about of 22 [15].

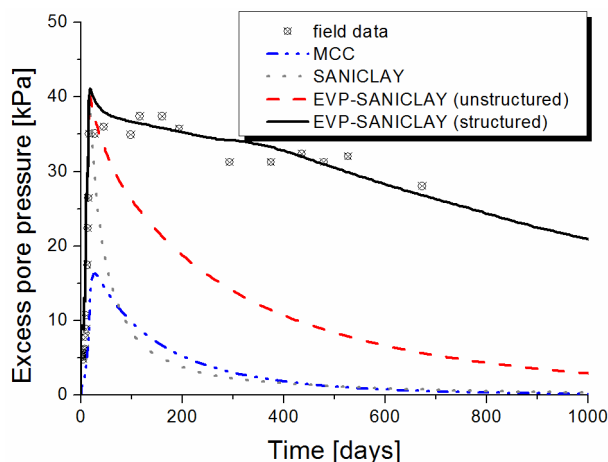


422 Figure 11. Numerical simulation results for surface settlement using: (a) MCC model; (b) SANICLAY model;
 423 (c) EVP-SANICLAY model without structure; (d) EVP-SANICLAY model

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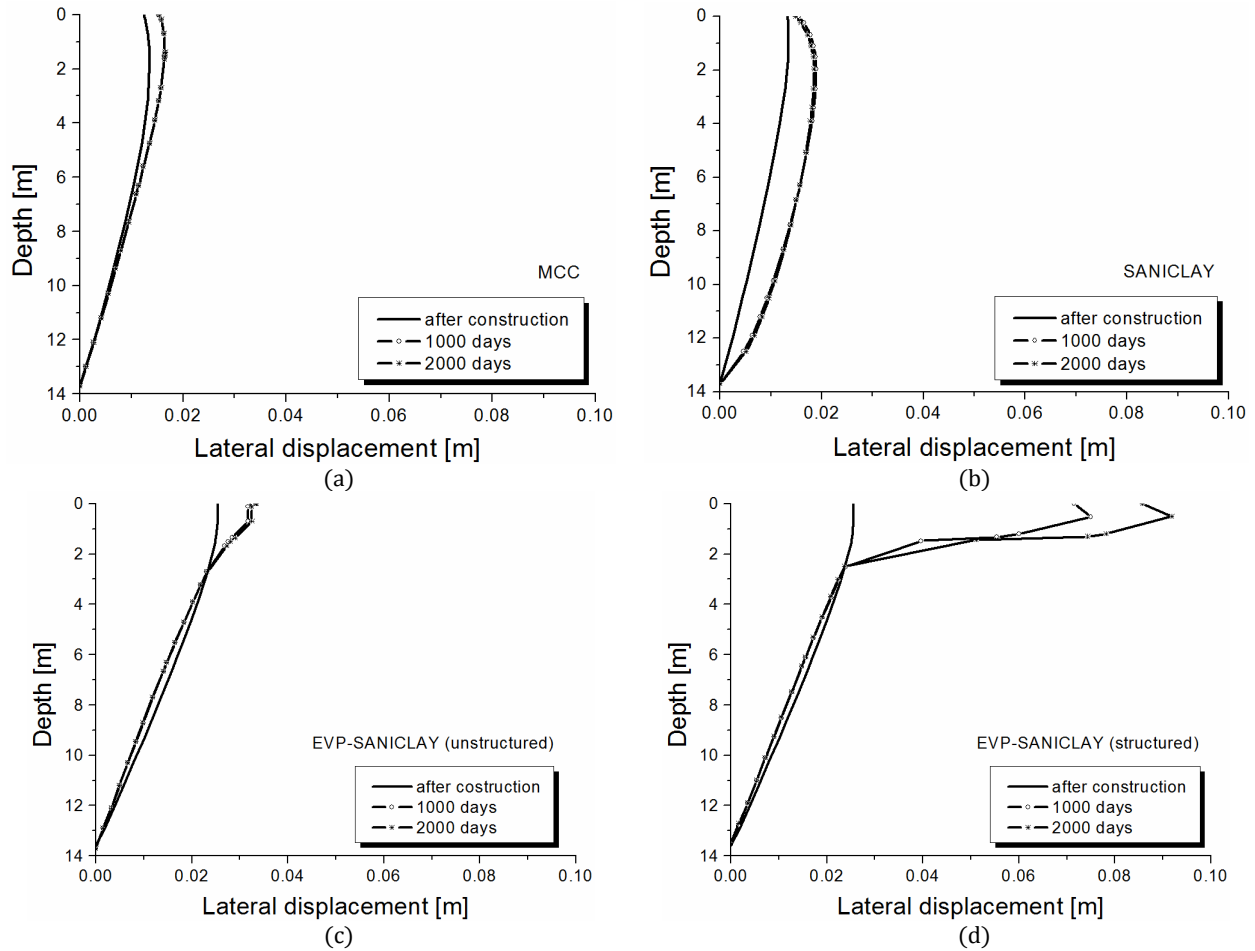
425 Pneumatic piezometers were installed at different depths underneath the embankment to monitor the
 426 excess pore water pressure variations with time [48,50]. Figure 12 shows the *in-situ* measurements related
 427 to a piezometer located on the centerline at a depth of 2.6m under the base of the embankment (point B in
 428 Figure 8a). The excess PWP initially increased during the embankment construction and then gradually
 429 dissipated with time. The figure also shows the results of numerical simulations with the models. As it can be
 430 seen, a better approximation of the excess PWP variation is obtained with the EVP-SANICLAY model, in
 431 comparison with the MCC, SANICLAY, and the anisotropic EVP model without structure. Interestingly, for the
 432 SANICLAY and both of the EVP-SANICLAY model simulations, with and without structure, the maximum PWP
 433 value is reasonably close to the field measurement, but when the initial structure and degradation of bonds
 434 are not taken into consideration, a faster dissipation of excess PWP is observed. MCC model underestimates
 435 the maximum excess PWP immediately after the construction; additionally, after the construction excess PWP

436 is dissipated very quickly, contrary to the observed *in-situ* scenario. The observed delayed pore pressure
 437 dissipation can be captured only when the viscosity of soil behavior is taken into consideration.
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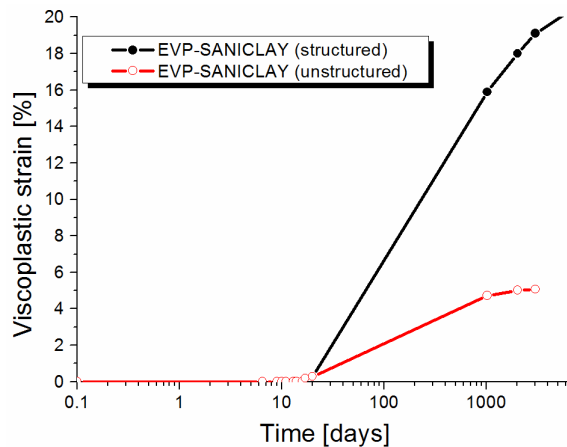


439
 440 Figure 12. Excess PWP predictions at point B in Figure 8a
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442 Field data for lateral displacements at depth are not available for the embankment [54]; therefore,
 443 simulation results presented in Figure 13, for the lateral deformation profiles at the toe of the embankment,
 444 could not be compared with the actual measurements. From Figure 13d, EVP-SANICLAY model simulations
 445 show deformation profiles similar in shape to what was reported for other embankment sites. For example in
 446 case of St. Alban embankment B, the maximum lateral displacement was reported to have more than doubled
 447 during the initial 4.5 years of consolidation [54], and the maximum value was at a depth of about 1m. The
 448 MCC and SANCILAY models led to smaller lateral displacement near the surface (Figure 13a,b). For the EVP
 449 simulations in Figure 13c,d, the lateral displacements increased near the surface, and delayed deformation
 450 became more pronounced. When structure effects were ignored in the EVP model (Figure 13c), the general
 451 shape of the lateral deformation profiles did not change much compared to Figure 13d but the predicted
 452 values became smaller, without noticeable difference between the profiles at 1000 and 2000 days. Clearly
 453 consideration of the soil initial structure and its degradation result in a greater pace of viscoplastic strain
 454 developments. For example, monitoring the development of viscoplastic strains at point B under the
 455 embankment, i.e. the position of the piezometer, it can be seen in Figure 14 that after an initial elastic
 456 response, the viscoplastic strains begin to develop while still in the construction phase, and then continue to
 457 evolve with consolidation progress. It is particularly apparent how ignoring soil structure effects lead to
 458 significantly lower viscoplastic strain accumulation, a trend also observed in previous figures.



459 Figure 13. Numerical simulation results for lateral displacement under the toe using: (a) MCC model; (b)
 460 SANICLAY model; (c) EVP-SANICLAY model without structure; (d) EVP-SANICLAY model

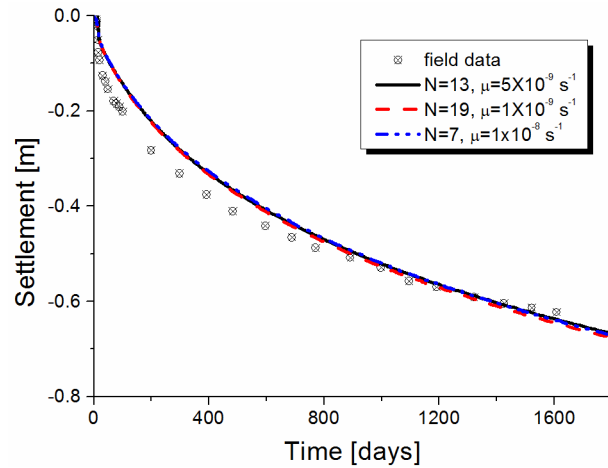


461
 462 Figure 14. Development of viscoplastic strains at point B in Figure 8a

463 4.3 Non-uniqueness of viscosity parameters

464 As already mentioned, calibration of viscosity parameters N and μ has been done directly on the
 465 embankment model as no appropriate test data has been available for the foundation soil. It should be

466 pointed out that the Perzyna-type viscosity parameters for a particular clay are not necessarily a unique set
 467 and more than one combination of the two viscosity parameters can be found for a clay, depending on how
 468 one wants to fit the experimental data [55]. Figure 15 shows an example of how for three different sets of
 469 viscosity parameter values it is still possible to obtain a good approximation of the field observation for
 470 settlements at point A under the embankment. For these particular sets, a maximum difference of only 3%
 471 was found among the vertical settlement results, and similar minor variations were observed among the
 472 corresponding lateral displacement and excess PWP predictions.

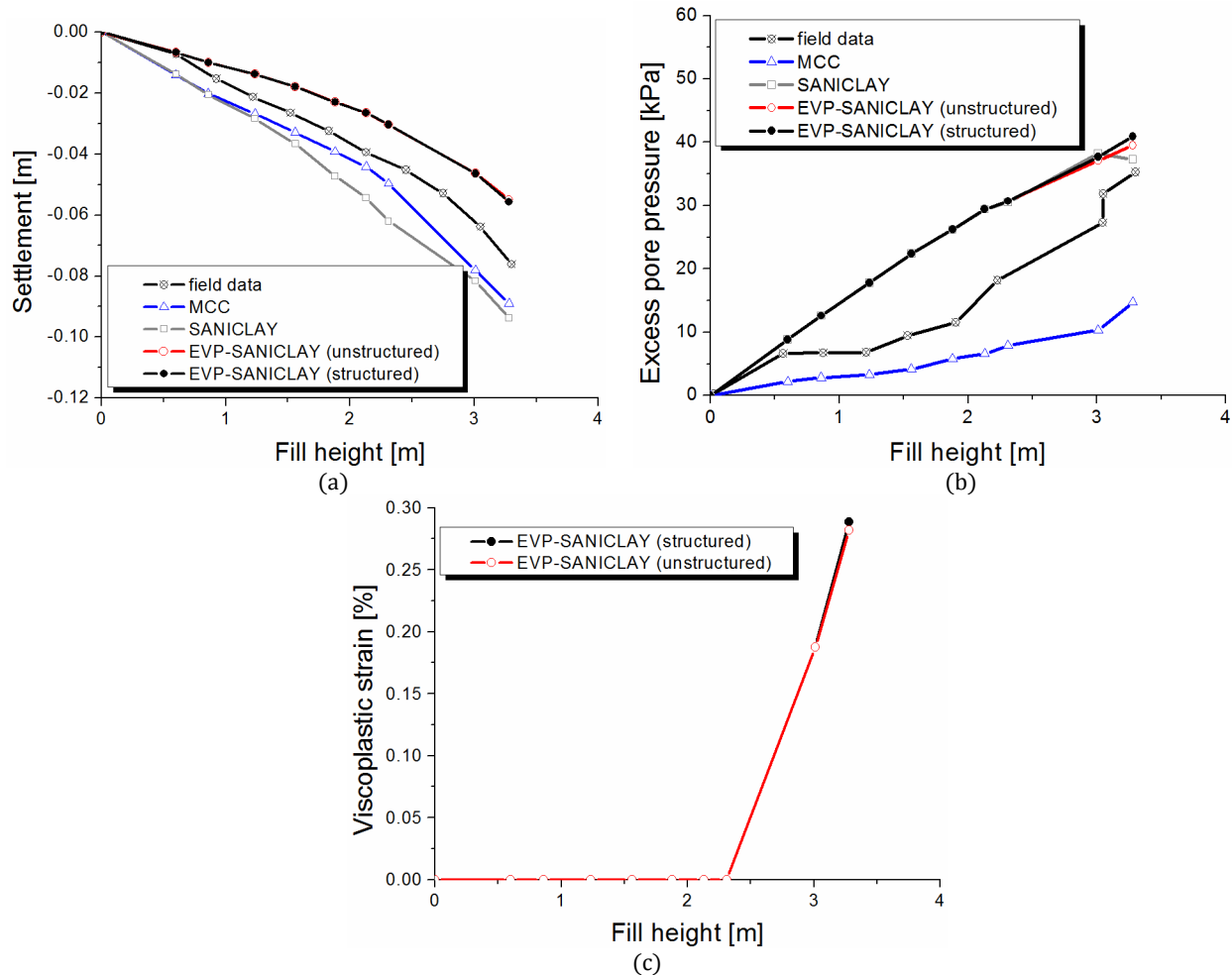


473
 474 Figure 15. Illustrating the non-uniqueness of viscosity parameters for prediction of time-settlement at point A
 475 in Figure 8a
 476

477 4.4 Discussion on behavior during construction

478 Additional field measurement data on settlement and excess pore pressure generation during
 479 embankment construction process is also available [48]. The data could be used to assess the performance of
 480 the developed model in reproducing the short-term response of the embankment. Simulation results during
 481 the construction are shown in Figure 16.

482 Figure 16a shows that at point A in Figure 8a, EVP-SANICLAY model somewhat underestimates the
 483 results; although, as it was observed in Figure 7a, it is then able to gain accuracy during consolidation. MCC
 484 and SANCILAY, on the contrary, overestimates short-term settlements during the construction. In terms of
 485 excess PWP at point B in Figure 8a, EVP-SANICLAY is able to give a good prediction of the pore pressure
 486 generation during the embankment construction (Figure 16b). Based on EVP-SANICLAY predictions, PWP
 487 develops rapidly during the construction until embankment reaches a height of approximately 2.31 m
 488 (corresponding to 16 days after the start of construction) when the excess PWP generation slightly
 489 decelerates. From the figure, it is clear that the MCC model underestimates excess PWP generation during the
 490 construction. Compared to the full EVP model, EVP-SANICLAY without structure provides lower predictions
 491 of excess PWP generation after the stage at which the embankment reaches a height of 2.31 m.
 492



493 Figure 16. Field measurements versus numerical simulation results for the duration of construction: (a)
 494 settlements at point A in Figure 8a; (b) excess pore water pressure at point B in Figure 8a; (c) development of
 495 viscoplastic strains at point B in Figure 8a
 496

497 Figure 16c shows the development of viscoplastic strains at point B under the embankment (i.e. at the
 498 position of the pneumatic piezometer considered). The figure shows that the viscoplastic strains start to
 499 develop when the embankment reaches a height of about 2.31m, which approximately corresponds to the
 500 time when excess pore pressure generation changes its pace.
 501

502 5 Conclusions

503 The response of natural soft soil is governed by anisotropy, structure and time-dependency. In this work,
 504 in order to concurrently account for these advanced features of soil behavior a time-dependent elasto-
 505 viscoplastic extension of a well-established anisotropic clay model, namely SANICLAY, has been proposed.
 506 The model is numerically implemented in finite element program PLAXIS using an implicit integration
 507 scheme. The performance of the model at the element-level has been validated against experimental data

508 obtained from testing four different clays at both structured and un-structured states. Furthermore the time-
509 dependent behavior of St. Alban embankment D on the well-structured Champlain clay was analysed using
510 the proposed EVP-SANICLAY model. The paper presented the results for settlements, lateral deformations,
511 and excess PWP variations during the construction and the subsequent consolidation, comparing model
512 predictions with the field measurements where available. It was observed that the developed model
513 considers the delayed excess pore pressure dissipation following the completion of the embankment
514 construction reasonably well; hence it is able to yield more realistic predictions of the long-term vertical and
515 horizontal deformations. The boundary value problem simulation results also illustrated that considering
516 clay initial structure and subsequent destructuration effects significantly improve the accuracy of predictions,
517 particularly when dealing with a highly sensitive soft clay such as Champlain clay. Furthermore, the model
518 also predicted the immediate displacements as well as the development of excess pore pressures during early
519 stages of construction with reasonable accuracy.

520 In general, EVP-SANICLAY proved to be able to much better predict both short- and long-term behavior
521 of natural clay behavior when compared with a commonly used critical state based model such as MCC, and
522 also the SANCILAY model.

523

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527

528 **Appendix**

529 For the sake of completeness of presentation, some of the key components of the SANICLAY model that
530 are not presented in the main body of this paper are summarized here. Both stress and strain quantities are
531 assumed positive in compression (as is common in geomechanics), and the effect of this sign convention has
532 been considered on the model equations. All stress components in this paper should be considered as
533 effective stress. Finally, in terms of notation, tensor quantities are denoted by bold-faced symbols and
534 operations explained accordingly.

535 The hypoelastic formulation, considered for simplicity, constitutes of a shear modulus G , for calculating
536 increments of elastic deviatoric strains, and a bulk modulus K , for calculating increments of elastic volumetric
537 strains, where

$$538 \quad G = \frac{3K(1 - 2\nu)}{2(1 + \nu)}; \quad K = \frac{p(1 + e)}{\kappa} \quad (A.1)$$

539 where ν is the Poisson's ratio, e is the void ratio, $p = (\text{tr}\boldsymbol{\sigma})/3$ is the mean effective stress (where tr stands for
the trace), and κ is the slope of elastic swelling lines in the $e - \ln p$ space.

540 The isotropic hardening law of the model describing the evolution of the size of structured SYS, i.e. p_0^{*s} , is
 541 defined as

$$\dot{p}_0^{*s} = \dot{S}_i p_0^s + S_i \dot{p}_0^s \quad (\text{A.2})$$

542 where \dot{S}_i is the evolution rate of the isotropic destructuration factor (explained in the sequel), and
 543 $\dot{p}_0^s = [(1+e)/(\lambda-\kappa)]p_0 \dot{\varepsilon}_v^{vp}$ is the evolution of the size of SYS, that is a proportional to viscoplastic
 544 volumetric strain rate, with λ indicating the slope of normal compression line.

545 The rotational hardening law describing the evolution of fabric anisotropy with viscoplastic staining can
 546 be expressed in the general stress space as:

$$\dot{\alpha} = \left(\frac{1+e}{\lambda-\kappa}\right) C \left(\frac{p}{p_0}\right)^2 |\dot{\varepsilon}_v^{vp}| \left[\frac{3}{2}(\mathbf{r} - x\alpha) : (\mathbf{r} - x\alpha)\right]^{1/2} (\alpha^b - \alpha) + \dot{\alpha}_f \quad (\text{A.3})$$

547 In the above equation, $\dot{\alpha}_f = (\dot{S}_f/S_f)\alpha$ controls the contribution of destructuration over the change of
 548 orientation of the yield surface (\dot{S}_f explained in the sequel); $\mathbf{r} = \mathbf{s}/p$ is the shear stress ratio; $\alpha^b = \sqrt{2/3} M \mathbf{n}_x$
 549 is the bounding ‘image’ stress-ratio tensor, where \mathbf{n}_x is an auxiliary unit tensor defined as $\mathbf{n}_x = \|(\mathbf{r}/x) - \alpha\|$
 550 and $\| \cdot \|$ denoting the norm operator; and $|\dot{\varepsilon}_v^{vp}|$ is the absolute value of the viscoplastic volumetric strain
 551 rate.

552 In order to express the isotropic and frictional destructurations, an axillary internal variable called the
 553 destructuration viscoplastic strain rate, $\dot{\varepsilon}_d^{vp}$, is defined by

$$\dot{\varepsilon}_d^{vp} = \sqrt{(1-A)(\dot{\varepsilon}_v^{vp})^2 + A(\dot{\varepsilon}_q^{vp})^2} \quad (\text{A.4})$$

554 where $\dot{\varepsilon}_v^{vp}$ and $\dot{\varepsilon}_q^{vp}$ are the volumetric and deviatoric viscoplastic strain rates, respectively, and A is a model
 555 parameter could be set to 0.5 as a default value. The evolution equations for the S_i and S_f read

$$\dot{S}_i = -k_i \left(\frac{1+e}{\lambda-\kappa}\right) (S_i - 1) \dot{\varepsilon}_d^{vp} \quad (\text{A.5})$$

$$\dot{S}_f = -k_f \left(\frac{1+e}{\lambda-\kappa}\right) (S_f - 1) \dot{\varepsilon}_d^{vp} \quad (\text{A.6})$$

556 where k_i and k_f are model parameters.

557 As indicated in model formulation section, the critical stress-ratio is defined as a function of the Lode
 558 angle θ . To regulate the variation of $M(\theta)$ between its values M_c for compression and M_e for extension, the
 559 expression proposed by Sheng et al. [56] has been adopted here which reads as

$$M(\theta) = M_c \left(\frac{2m^4}{1+m^4+(1-m^4)\sin 3\theta} \right)^{1/4} \quad (\text{A.7})$$

560 where $m = M_e/M_c$, $-\pi/6 \leq \theta = (1/3)\sin^{-1}[-3\sqrt{3}J_3/(2J_2^{3/2})] \leq \pi/6$, with J_2 and J_3 being the second and
 562 third invariants of the modified stress deviator $\mathbf{s} - p\alpha$.

563

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