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Calibration of the Creep-SCLAY1S Constitutive Model Parameters for Champlain Sea Clay in Quebec

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ABSTRACT

The structured behaviour of Champlain Sea clay makes it extremely sensitive to strains that may destroy the bonds between soil particles. This soil behaviour can have significant effects on foundation design, but it cannot be easily captured using standard finite element models such as Mohr-Coulomb. The PLAXIS Creep-SClay1S constitutive model was developed to simulate the anisotropic, rate-dependent behaviour of soft structured soils including creep. This constitutive model is considered appropriate for the analysis of Champlain Sea clay due to its ability to simulate the bonding and destructuration process of natural clays.

The Creep-SClay1S model was applied to predict settlement due to an embankment construction over Champlain Sea clay at a site in the region of Montérégie in Quebec. Design parameters required for the Creep-SClay1S models were calibrated against laboratory tests and field-testing measurements using the PLAXIS "Soil Test" module. The PLAXIS "Soil Test" module outputs (Deviator vs. Axial Strain, PWP vs. Axial Strain) were able to replicate the stress paths of CIUC triaxial tests with some reliability. The calibration showed that the depositional history of Champlain Sea Clay must be taken into consideration in the PLAXIS modeling. A summary of the calibrated input parameters for the Creep-SClay1S model is presented here.

A comparison of settlement prediction using conventional settlement theory for embankment construction to settlement predicted using Mohr-Coulomb and the Creep-SClay1S model was undertaken. Results indicate that the settlement predicted by conventional settlement theory and Mohr-Coulomb may greatly underestimate the magnitude of settlement in Champlain Sea clays compared to the Creep-SClay1S model.

RÉSUMÉ
La structure de l'argile de la mer de Champlain la rend extrêmement sensible aux contraintes qui peuvent détruire les liaisons entre les particules du sol. Ce comportement du sol peut avoir des effets importants sur la conception des fondations, mais il ne peut pas être facilement saisi à l'aide d'un modèle d'éléments finis standard tel que Mohr-Coulomb. Le modèle constitutif PLAXIS Creep-SClay1S a été développé pour simuler le comportement anisotrope dépendant du taux des sols structurés mous, y compris le fluage. Ce modèle constitutif est jugé approprié pour l'analyse de l'argile de la mer de Champlain en raison de sa capacité à simuler le processus de liaison et de déstructuration des argiles naturelles.

Le modèle Creep-SClay1S a été appliqué pour prédire le tassement dû à la construction d'un remblai au-dessus de l'argile de mer Champlain à un site dans la région de la Montérégie au Québec. Les paramètres de conception requis pour les modèles Creep-SClay1S ont été calibrés par rapport aux tests de laboratoire et aux mesures de test sur le terrain à l'aide du module PLAXIS 'Soil Test'. Les sorties du module PLAXIS 'Soil Test' (stress déviatorique vs déformation axiale, PWP vs déformation axiale) ont pu reproduire les chemins de contrainte des tests triaxiaux CIUC avec une certaine fiabilité. L'étalonnage a montré que l'historique des dépôts d'argile marine de Champlain doit être pris en compte dans la modélisation PLAXIS. Un résumé des paramètres d'entrée calibrés pour le modèle Creep-SClay1S est présenté ici.

Une comparaison de la prédiction de tassement en utilisant la théorie conventionnelle de tassement pour la construction de remblais au tassement prévu en utilisant Mohr-Coulomb et le modèle Creep-SClay1S a été entreprise. Les résultats indiquent que les tassements prédits par la théorie des tassements conventionnels et Mohr-Coulomb peuvent sous-estimer considérablement l'ampleur des tassements dans les argiles de la mer Champlain par rapport au modèle Creep-SClay1S.

1 INTRODUCTION

The impact of soil destructuring can be catastrophic for projects in certain soils. However, the effects of soil debonding and destructuration cannot be easily understood using conventional design methods.

For strain sensitive soils such as the Champlain Sea clay, the use of conventional design methods and standard constitutive models such as Mohr-Coulomb (MC) are inadequate. These methods only consider linear stiffness and strength-based parameters, which do not adequately reflect the debonding and destructuration of this sensitive clay.

The PLAXIS Creep-SClay1S constitutive soil model is capable of simulating anisotropic and rate dependent behaviours of soft clays, including creep. Its ability to simulate the bonding and destructuration of natural clays makes it a more realistic constitutive model than MC for Champlain Sea clay.

1.1 Champlain Sea Clay

Champlain Sea clay was deposited under the marine environment of the Champlain Sea, which invaded the St. Lawrence valley upon the retreat of the Laurentide Ice Sheet (Crawford 1968). The initially saline nature of the sea has been attributed to the development of bonding between clay particles in Champlain Sea clay.

The structure of the clay was also developed through the process of flocculation over a relatively slow depositional period. This slow depositional period (between 12,000 to 8,000 years B.P.) allowed the clay to maintain a relatively open structure under subsequent depositional stresses. The structure of undisturbed Champlain Sea clay is often described as an open "card-house" structure. The open structure and cemented nature make Champlain Sea clay sensitive to strains that may destroy the bonds between particles, making it susceptible to rapid transformations from a relatively brittle material to a liquid mass when disturbed (Crawford, 1968; La Rochelle and Lefebvre, 1971).

1.2 Project Site

An embankment construction was proposed at a site in the region of Montérégie, Quebec. The site stratigraphy comprises a layer of fill over Champlain Sea clay which is underlain by Till, and Shale at depth. The site's groundwater is located at an elevation of approximately 6m.

A geotechnical investigation was completed in 2016 to inform the geotechnical design. The investigation consisted of soil index testing (Atterberg limits, grain-size distribution, etc.), laboratory testing (CIDC/CIUC triaxial tests, oedometer tests) and field-testing measurements including cone penetration testing (CPT), and peak and residual field vane measurements. A total of 10 triaxial tests and 12 oedometer tests were undertaken.

Results of the ground investigation indicated the potential for two distinct Champlain Sea clay deposits present at the site. These clay deposits will be referred to as the Upper and Lower Clay deposits.

2 AVAILABLE GEOTECHNICAL DATA

2.1 Soil Index Testing

The Upper Clay is reported to have a moisture content between 52% and 63%, a plastic limit between 23% and 30%, and liquid limit between 55% and 70%. Generally, the moisture content in the Upper Clay is near the liquid limit, indicating a highly plastic clay. The Atterberg limits of the Upper Clay are generally consistent with depth.

The Lower Clay is reported to have a moisture content between 25% and 40%, plastic limit between 15% and 23% and liquid limit between 30% and 46%. The Lower Clay is also observed to contain more silt than the Upper Clay. The Lower Clay displays both lower moisture content and plasticity index than the Upper Clay, along with a lower void ratio. Figure 1 displays the relationship between soil index properties with elevation.

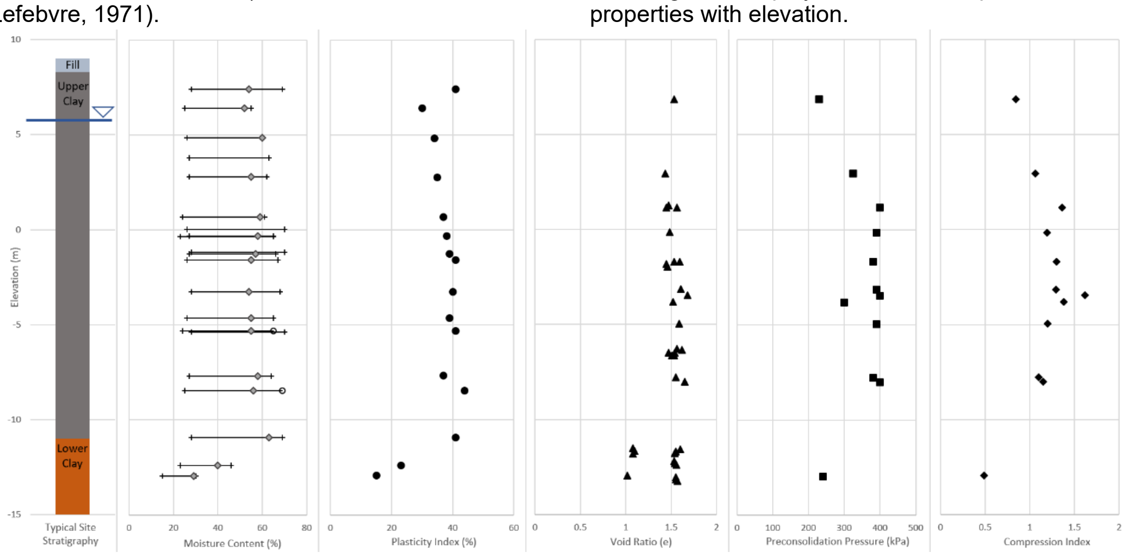

Figure 1: Soil index testing results (moisture content, Atterberg limits, void ratio (e_o), preconsolidation pressure (σ'_p) and compression index (C_c)

2.2 Undrained Shear Strength and Sensitivity

Undrained shear strength (S_u) measurements were primarily completed using a Nilcon field vane, results of the measurements are plotted with elevation in Figure 2.

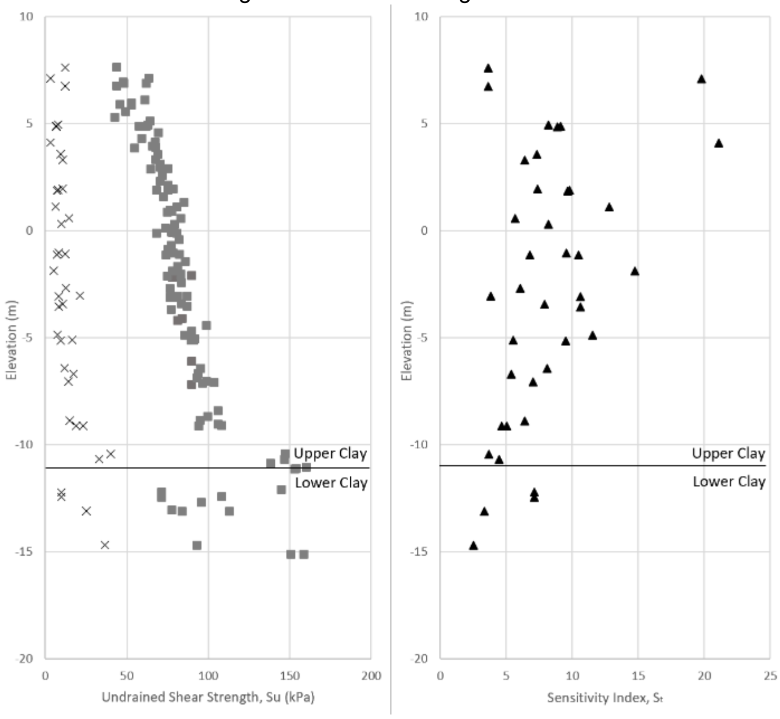

Figure 2: Undrained shear strength and sensitivity profile of project site.

Shear vane readings indicate a general increase in shear strength with depth. The Lower Clay below El -10m is observed to have a larger spread of undrained shear strength results. However, the general trend continues to indicate an increase in S_u with depth.

Both peak and remoulded shear strengths were measured to determine the sensitivity of the clay. The sensitivity index is defined as the ratio of peak shear strength to remoulded shear strength. Plots of sensitivity with elevation are presented in Figure 2. The sensitivity was observed to decrease with depth.

2.3 Compression Testing and Cementation

A total of 12 oedometer tests were completed over the course of the geotechnical investigation. The results of the tests highlighted the apparent over-consolidated nature of the Champlain Sea clay at the site. A secondary interpretation was completed to determine whether the apparent over-consolidation was a result of erosion or cementation and bonding of the natural clay.

Perret et al (1995), presented a set of idealized charts based on Over Consolidation Difference, OCD ($\sigma'_p - \sigma'_{vo}$), Over Consolidation Ratio, OCR (σ'_p / σ'_{vo}) and Over Consolidation Gradient, OCG ($\Delta\sigma'_p$ / $\Delta\sigma'_{vo}$) which provide an overview of the expected trendlines associated with the processes that may lead to over-consolidation. Figure 3 outlines the trendlines expected with depth for the processes of secondary consolidation, erosion, and cementation. Results of the oedometer tests from the Montérégie site were plotted with elevation (Figure 4) and displayed a strong correlation to the cementation trendlines in Figure 3.

Erosion has typically been considered as the cause of over-consolidation in clays and is typically assumed in foundation designs. However, the apparent over-consolidation from the oedometer tests at the Montérégie site is a result of cementation rather than erosion.

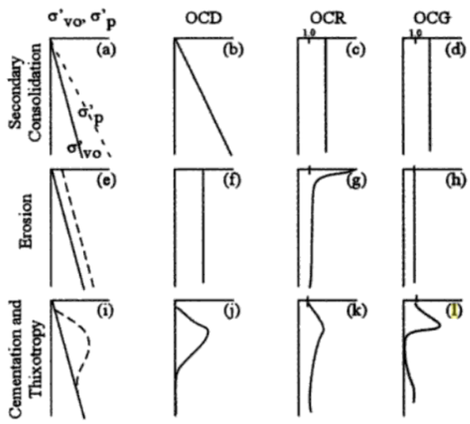

Figure 3. Idealized OCD, OCR and OCG Charts (Perret et al, 1985).

The presence of cementation highlights the importance of capturing the potential debonding and destructuring of the Champlain Sea clay during the design by utilizing the Creep-SClay1S model.

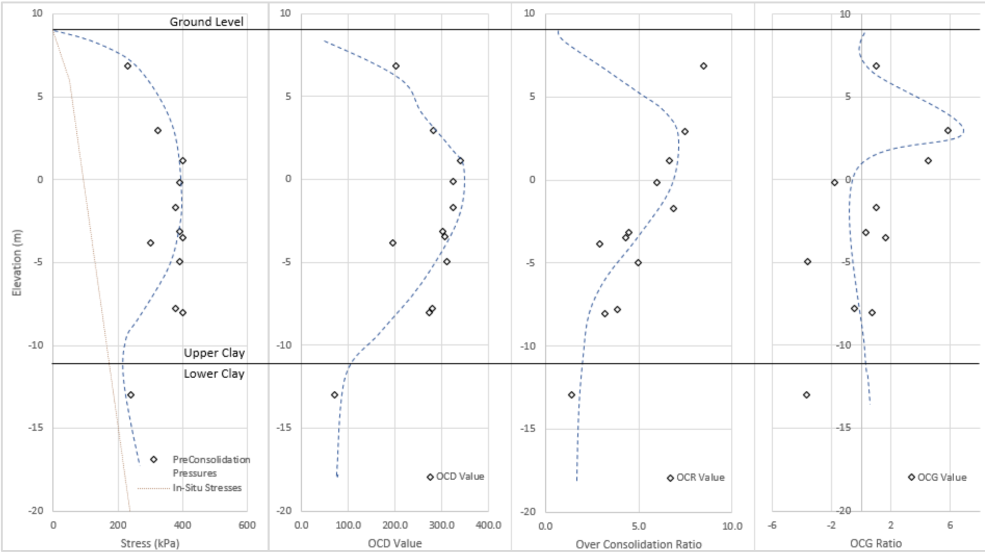

Figure 4: Oedometer data plotted vs. idealized cementation profiles for OCD, OCR, OCG.

2.4 Triaxial Testing (CIUC Results)

CIUC and CIDC triaxial tests were completed during the geotechnical investigation. Tests were completed over a range of effective stresses from 50 to 300 kPa. All tests were progressed down the critical state line (CSL) to inform of post peak behaviour. The critical state line (M_c, i.e. q/p') was observed to be fairly consistent between 1.3 and 1.4. A value of 1.3 was adopted, corresponding to a critical state friction angle (φ'_{cv}) of 32 degrees. The majority of tests were observed to display strain softening behaviours post-peak. Dilative pore water pressures were observed in some tests at lower effective stresses (50 kPa). The stress path and testing behaviours are displayed in Figure 5.

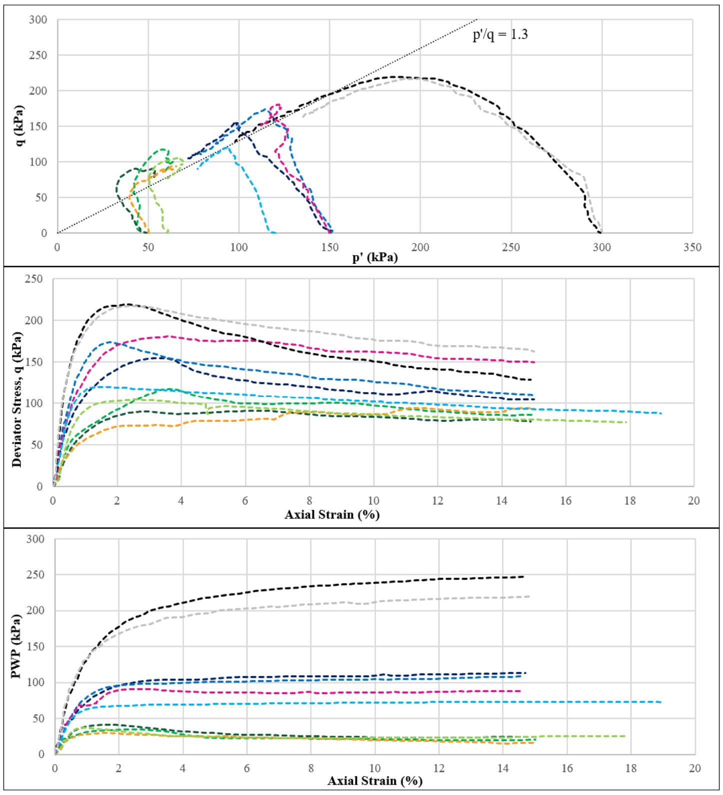

Figure 5: CIUC Triaxial test results (p vs q, q vs. axial strain, PWP vs axial strain).

3 CREEP-SCLAY1S DESIGN PARAMETERS

The Creep-SClay1S constitutive model was calibrated with available test results from the ground investigation. This model was then used to predict the embankment settlement proposed at the site. The following parameters are required for the Creep-SClay1S constitutive model.

Table 1. Creep-SClay1S model parameters

Parameter Type	Symbol	Description
Critical state Parameters	M_c	Slope of critical state line in compression
	M_e	Slope of critical state line in extension
Isotropic stiffness parameters	λ_i^*	Modified intrinsic compression index
	κ'	Modified swelling index
	ν'	Poisson's ratio
Bonding parameters	ε	Absolute rate of destruction
	ε_d	Relative deviatoric rate of destruction
Anisotropy parameters	ω	Absolute effectiveness of rotational hardening
	ω_d	Relative deviatoric effectiveness of rotational hardening
Creep parameters	μ_i^*	Modified intrinsic creep index
	τ	Reference time (usually 1 day)
Initial state variables	e_0	Initial Void Ratio
	POP	Pre-Overburden Pressure
	OCR	Over-Consolidation Ratio
	α_0	Initial anisotropy
	X_0	Initial amount of bonding
Other parameters	K_o^{NC}	Coefficient of lateral stress in normal consolidation

4 CREEP-SCLAY1S MODEL CALIBRATION

4.1 Strength Parameters

The slope of the critical state line in compression and extension were determined from CIUC triaxial tests. As observed from Figure 5, the critical state line was relatively consistent for all tests and corresponded to an M_c ratio of 1.3 and a friction angle (φ'_{cv}) of approximately 32 degrees (see Equation 1, PLAXIS, 2017).

$$M_c = 6\sin\varphi'_{cv} / (3-\sin\varphi'_{cv}) \quad [1]$$

No triaxial extension tests were completed during the geotechnical investigation. A M_e of 0.96 was therefore calculated based on a φ'_{cv} = 32 degrees as outlined in Equation 2 (PLAXIS, 2017).

$$M_e = 6\sin\varphi'_{cv} / (3+\sin\varphi'_{cv}) \quad [2]$$

4.2 Stiffness Parameters

The modified compression index, λ^*, is obtained from the slope of the compression line in the ε_v – ln(p') plane. Similarly, the modified swelling index κ^*, is the slope of the unload/reload curve in the ε_v – ln(p') plane.

The intrinsic value of the modified compression index, λ_i^*, is obtained from oedometer test results at very high stresses, or by using reconstituted samples. It is assumed that at very high stresses the bonds within the sample have been completely destroyed. Figure 6 displays the relationship between λ^*, κ^* and λ_i^*.

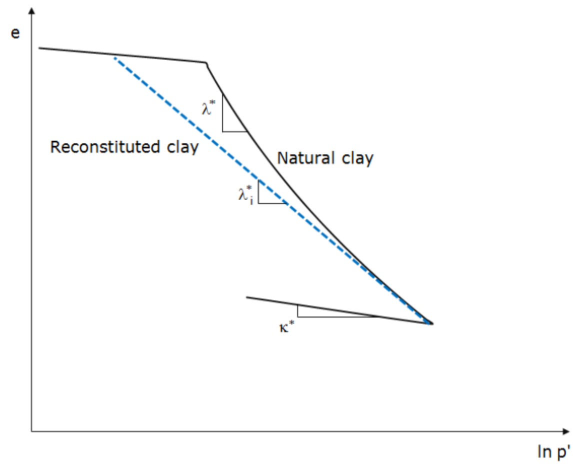

Figure 6. Compression parameter determination for the Creep-SClay1S model (PLAXIS, 2017)

All the oedometer tests undertaken in the Upper Clay were progressed to very high stresses and the clay samples were likely destructured. The modified compression index, intrinsic compression index and modified swelling index were plotted from oedometer test results for the Upper Clay. A value of 0.15 and 0.015 was selected for λ_i^* and κ^*, respectively, for the Upper Clay.

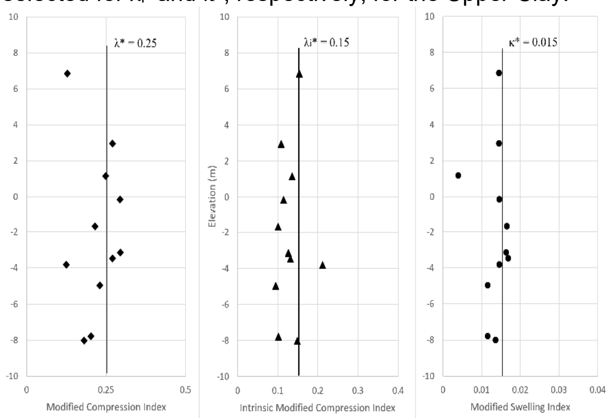

Figure 7. Compression parameters for Upper Clay from oedometer tests.

The oedometer tests performed on the Lower Clay did not progress to sufficiently high stresses to result in the complete destructuration of the clay. Therefore, a measurement of the intrinsic compression index could not be made based on the laboratory test results. The modified compression index of the Lower Clay was observed to be lower than the Upper Clay, and therefore a lower bound λ_i^* of 0.1 was selected for the Lower Clay.

4.3 Bonding Parameters

The initial degree of bonding, X_0, can be approximated based on the sensitivity index of the clay (S_t) following Equation 3 (PLAXIS, 2017).

$$X_0 = S_t - 1 \quad [3]$$

As observed in Figure 2, the sensitivity ranged from approximately 15 to 5 within the Upper Clay, and from 5 to 2 in the Lower Clay.

The absolute rates of destructuration (ξ) and relative deviatoric rate of destructuration (ξ_d) were estimated within the PLAXIS "Soil Test" Module. The destructuring parameters were observed to influence the post-peak behaviour of the clays, influencing both the rate of strain softening and the total loss in strength. The ξ and ξ_d were estimated based on trial and error to mimic the stress paths observed in the CIUC triaxial tests.

4.4 Anisotropy Parameters

An initial anisotropy/inclination of the yield surface was calculated using Equation 4 (PLAXIS, 2017). A value of 0.48 was adopted for the Upper and Lower Clay.

$$\alpha_0 = \eta^2_{Ko}{}^{NC} + 3\,\eta_{Ko}{}^{NC} - M_c{}^2 / 3 \quad [4]$$

Where,

$$\eta_{Ko}{}^{NC} = 3(1 - K_o{}^{NC}) / (1+2K_o{}^{NC}) \quad [5]$$

And,

$$K_o{}^{NC} = 1 - \sin\varphi'_{cv} \quad [6]$$

A value of 0.97 was calculated for the relative effectiveness of creep strains in rotational hardening, ω_d, for the Upper Clay and Lower Clay using Equation 7 (PLAXIS, 2017).

$$\omega_d = 3(4M_c^2 - 4\eta^2_{Ko}{}^{NC} - 3\eta_{Ko}{}^{NC})/8(\eta^2_{Ko}{}^{NC} - M_c^2 + 2\eta_{Ko}{}^{NC}) \quad [7]$$

The absolute effectiveness of rotational hardening, ω was calculated for the Upper and Lower Clay, and were estimated following Equation 8 (PLAXIS, 2017).

$$1.5 / (\lambda^* - \kappa^*) \leq \omega \leq 4.2 / (\lambda^* - \kappa^*) \quad [8]$$

An initial value of 11.36 and 7.36 were selected for the Upper and Lower Clay, respectively.

4.5 Creep Parameters

The creep parameters μ^* and τ are used to determine the viscoplastic multiplier that defines creep strain rates for the clay. This parameter can be estimated from oedometer tests at very high stresses, where the specimen is subjected to constant loading for an extended time to enable creep behaviour to be observed. However, no creep testing was completed during the geotechnical investigation, and the value for μ^* was estimated based on published literature.

Published values reported for other marine clays (Yin et al., 2014) ranged between 0.004 and 0.01. The μ^* values assumed for the Upper and Lower Clay are 0.006 and 0.008, respectively.

The reference time τ, is representative of the duration of the load step in the oedometer test used to obtain preconsolidation pressure. It is expressed in days within the PLAXIS constitutive model. A value of 1 is representative of a 24h oedometer test.

4.6 OCR and POP Implications

The parameters relating to over-consolidation ratio, OCR, and pre-overburden pressure, POP, in the PLAXIS Creep-SClay1S model are more significant for sedimentary clays than for cemented clays. The OCR and POP were calibrated in the PLAXIS "Soil Test" module based on triaxial data. Sensitivity analyses indicated that M_c, POP and OCR had the greatest influence on the peak strength and stress path during a simulated triaxial test.

The calibration indicates that the OCR value is to be kept constant at 1.2 for both the Upper and Lower Clay. A POP of 80 was adopted for the Upper Clay and 60 for the Lower Clay.

5 PLAXIS "SOIL MODULE" TEST RESULTS

The design parameters presented above were verified against site specific triaxial tests using the PLAXIS "Soil Test" module. The "Soil Test" module mimicked the stress path and stress-strain behaviour of a triaxial test.

Figure 8 presents the stress path, deviatoric strain, and excess pore water from the PLAXIS "Soil Test" Module for both the Upper and Lower Clay. The PLAXIS output was plotted with the laboratory triaxial data. Results indicate that the calibrated PLAXIS Creep-SClay1S soil constitutive model was well calibrated and mimicked the behaviour of the clay on the project site with a reasonable degree of confidence.

values. An initial void ratio of 2 was adopted for both the Upper and Lower Clay. An initial X_0 of 15 within the Upper Clay and 10 within the Lower Clay were adopted for the depositional stage 12,000 years ago. Following the simulation of 12,000 years of consolidation/erosion and

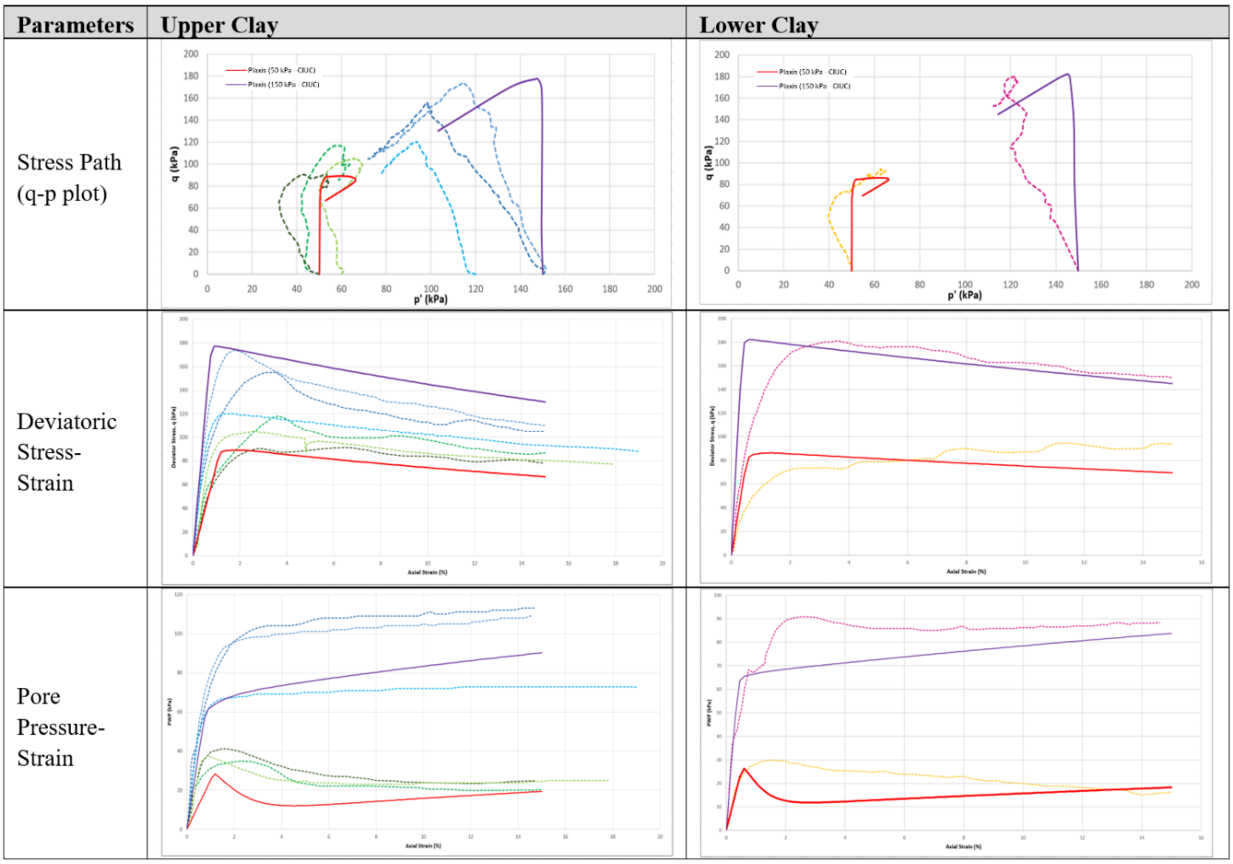

Figure 8: PLAXIS “Soil Test” Triaxial results plotted versus laboratory CIUC tests.

6 CREEP-SCLAY1S MODEL ADDITIONAL CONSIDERATIONS

Due to the creep component of the constitutive model, the depositional history and age of the soil deposit are important considerations during model set up. The PLAXIS model should be staged to account for clay deposition, erosion, and aging to ensure that the present in-situ conditions are reflected. This ensures a more accurate representation of the current in-situ stress and strain levels.

All the designed parameters outlined above are based on test data which represent the current site conditions, not the conditions present when the clay was first deposited. The two main design parameters affected by time and depositional history were observed to be e_0 and X_0. For this project, the depositional history was estimated to have begun approximately 12,000 years ago. When the PLAXIS model depositional history was modelled with the present day e_0 and X_0, these values were observed to decrease during the depositional phases, and therefore, in-situ conditions within the model did not match present day levels. As a result, modifications were made to these aging, the e_0 and X_0 values were observed to be more representative of present day in-situ conditions.

7 CREEP-SCLAY1S MODEL INPUT SUMMARY

The calibrated input parameters for the Creep-SClay1S constitutive soil model for the site in the region of Montérégie are summarized in Table 2.

Table 2. Creep-SClay1S input parameters

Parameter	Upper Clay	Lower Clay
γ	16	16
κ'	0.015	0.01
ν'	0.12	0.12
λ_i^*	0.15	0.1
M_c	1.3	1.3
M_e	0.96	0.96

ω	11.36	7.36
ω_d	0.97	0.97
ε	3	3.5
ε_d	0.8	0.5
OCR	1.2	1.2
POP	80	60
e_0	2.0*	2.0*
α	0.48	0.48
X_0	15*	10*
τ	1	1
μ	6x10-3	8x10-3
K_o^{NC}	0.47	0.47

Notes:
1) * Denotes design parameters affected by time and depositional history.

8 EMPIRICAL SETTLEMENT CALCULATION VS. CREEP-SCLAY1S

A 6m high embankment was proposed at the site. A comparison of the estimated settlements of the proposed embankment using conventional methods and Creep-SClay1S was undertaken. A conventional settlement calculation using one-dimensional primary consolidation was carried out based on a surcharge loading of 120 kPa. Settlement was calculated using Equation 9. A C_c value of 1.3 (See Figure 1) and an e_0 value of 1.5 for the Upper and 1.0 for the Lower Clay were adopted.

$$S_c = C_c(H/(1+e_o)) \log((\sigma'_{vo} + \Delta\sigma_v)/ \sigma'_{vo}) \quad [9]$$

Results indicated a predicted settlement of 207mm under the embankment load during primary consolidation. A settlement calculation was also performed in PLAXIS using a Mohr-Coulomb constitutive model, adopting a Young's Modulus, E, of 15 MPa for the Upper and Lower Champlain Sea clay (based on E_{50} results from Triaxial testing). Results of the Mohr-Coulomb settlement calculation indicated a predicted settlement of approximately 139mm.

The predicted settlements within the Champlain Sea clay from the Creep-SClay1S were observed to be greater than 1m.

The conventional settlement calculations do not account for the potential bonding of soil particles, and are primarily based around preconsolidation pressure, OCR, C_c and E, and the assumption that soil stiffness is linear. When debonding and destructuration are considered in settlement analyses, such as with the Creep-SClay1S constitutive model, it is observed that large settlements may occur due to the breaking of bonds under increased stresses.

These results highlight the potential underestimations of settlements which may occur if debonding and destructuration are not considered during design. Table 3 provides a summary of the predicted settlement values and the associated percentage error when compared to the Creep-SClay1S model.

Table 3. Settlement results and percent error.

Settlement Calculation Method	Predicted Settlement (mm)	% Error Compared to Creep-SClay1S Model (%)
Conventional	207	87.2
Mohr Coulomb	139	91.5
Creep-SClay1S	1620	N/A

9 CONCLUSION

The calibrated Creep-SClay1S constitutive model is shown to be able to mimic the stress paths for triaxial tests in Champlain Sea clay with some degree of confidence. This constitutive model is more appropriate than conventional methods in simulating the potential destructuring of Champlain Sea clay.

Finite element modelling using the Creep-SClay1S constitutive model must take into account the depositional history of Champlain Sea clay. The current in-situ conditions on site should also be calibrated within the PLAXIS model after the constitutive model calibration. The depositional history timeline and creep components are therefore an integral consideration during model set up. The two main design parameters affected by time and depositional history are e_0 and X_0.

The current study highlights the potential for the underestimation of settlements in Champlain Sea clay if debonding and destructuration are not considered in the calculation. Settlements from the conventional analysis and the Mohr-Coulomb analysis were observed to be an order of magnitude less than the Creep-SClay1S analysis.

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