Effect of Temperature on the 1-D Behaviour of Plastic Clay

Hamid Batenipour, David Kurz, Marolo Alfaro and Jim Graham
Department of Civil Engineering, University of Manitoba, Winnipeg R3T 5V6, Manitoba, Canada

ABSTRACT
The paper reports results of oedometer tests at temperatures of 3°C and 21°C on plastic clay from a test site in Northern Manitoba. Clay in the discontinuous permafrost region of Northern Manitoba is often at temperatures in the range 0ºC to 5ºC, yet laboratory testing is usually done in the range 20ºC to 25ºC. Differences in temperature can be expected to affect the viscoplastic behaviour of clays in compression and shear. This is especially true when the clay exhibits diffuse double layer (DDL) effects. Changes in DDL affect the position (but not the slope C_c) of the Normal Consolidated Line (NCL). It also affects the coefficient of secondary compression $C_{\alpha e}$. The results will be used in elastic thermoplastic modeling of settlements of a highway embankment on degraded permafrost near Thompson, Manitoba.

RÉSUMÉ
Le rapport démontre les résultats des tests d'oedomètre à des températures de 3°C et 21° C sur de l'argile d'un site au nord du Manitoba. L'argile dans la région discontinuée du pergélisol au nord du Manitoba se retrouve souvent entre 0ºC à 5ºC, pourtant les essais en laboratoire sont habituellement effectués de 20ºC à 25ºC. C'est attendu que ces différences en température vont affecter le comportement viscoplastique des argiles en compression et en cisaillement. Ceci est particulièrement vrai lorsque l'argile montre des effets de DDC (double couche diffuse). Des changements dans DDC affectent la position (mais pas la pente C_c) de la ligne consolidée normale (LCN). Ceci affecte aussi le coefficient de compression secondaire $C_{\alpha e}$. Les résultats vont être utilisés dans la modélisation thermoplastique élastique des tassements de terrain d'un remblai d'autoroute sur un pergélisol discontinu près de Thompson au Manitoba.

1 INTRODUCTION

The influence of temperature on the mechanical properties of soils is a major consideration for construction and infrastructure projects in Northern regions where permafrost exists. Climate changes provide an added cause for concern. Changes in meteorological conditions, precipitation, solar radiation, wind speed, and other factors induce temperature changes at ground level and at greater depths. The changes in temperature affect civil engineering infrastructure, particularly in areas with mean annual temperatures close to 0°C where permafrost is locally discontinuous. In these regions, thawing may produce large settlements and non-recoverable shear deformations.

Frozen soil is stronger and less compressible than unfrozen soil. Frozen silty sands, silts, and silty clays frequently contain layers of ice that form as a result of water migration to negative water potentials at the freezing front (Konrad 2008). If thawing occurs, either as a result of warming climate or changes in heat transfer due to engineering activity, ice lenses melt and water moves towards the ground surface. The resulting decreases in effective stresses in the soil cause weakening and deformations in foundations for buildings and pipelines, airport runways, rail beds, and highway sub-bases, cuts and fills. Out-migration of water often produces irregular settlements that lead to serviceability issues. Roads and runways remove vegetation cover, affect snow cover, increase heat transfer, and affect drainage patterns. They therefore contribute significantly to disturbance of discontinuous permafrost.

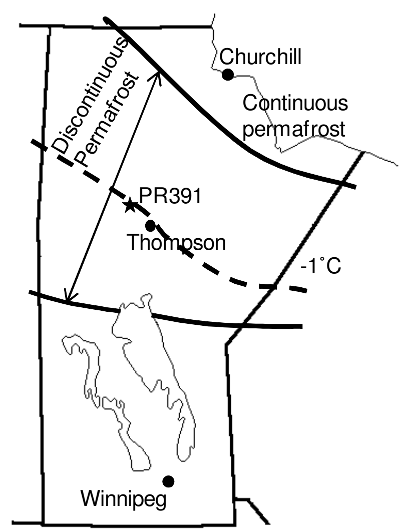

Figure 1. Location of test site and permafrost in Manitoba, Canada

Northern Canada is home to many First Nations communities and rich in mineral, petrocarbon and hydroelectric resources that will require future infrastructure projects. In Manitoba (MB), discontinuous permafrost is encountered (Figure 1) north of an isotherm that corresponds approximately to a mean annual air temperature of 0ºC (about 2500ºC-days of frost). It becomes continuous further north near the Hudson Bay coast. Road, rail, and air communications are essential in the North and are becoming increasingly important with further resource development. Thawing of summer ice in the Arctic Ocean may lead to increased shipping into the port of Churchill. Additional roads and railways will have to be constructed over soils with engineering properties that may further degrade with climate change and land-use.

Construction of highway fills in Northern Manitoba generally follows similar practices to those used in warmer regions. The fill materials typically have high thermal conductivity, leading to heat transfer into underlying layers and thawing of previously frozen foundation soil. Asphalt surfacing absorbs heat from the sun and transfers it to the embankment. Generally, degradation of permafrost begins at the toe of embankments. When the hydraulic conductivity of the foundation soil is low, melting of ice increases pore water pressures, reduces strengths, and increases compressibilities. This leads to differential settlements, lateral spreading, and instability.

The instrumented test site that forms the basis for this study is about 18 km north west of Thompson on Provincial Road PR391 (Figure 1). This is the only road connecting Thompson to northern mining towns, hydroelectric generating stations, and first Nations communities in north-western Manitoba. The road bed was constructed on discontinuous permafrost. Since construction, changes in heat transfer have caused permafrost degradation at locations where possible permafrost was detected, particularly in the foundations of embankments. The thawing has led to large ongoing irregular deformations and dangerous trafficability issues.

This paper presents the results of a laboratory investigation performed on plastic clay extracted from the test site, which is a low highway embankment on degraded permafrost. The effects of temperature on mechanical properties of the clay were investigated by comparing results of one-dimensional compression tests performed at 21°C, (room temperature), with results of similar tests at 3°C. The results will later be used in elastic thermoplastic modeling of irregular settlements of the embankment.

2 MATERIAL AND TEST PROCEDURES

2.1 Material

A site investigation program at PR391 in 2008 involved two cross-sections, one of which was designated as ‘stable’ and the other as ‘unstable’. The ‘stable’ section has not deformed significantly, while the ‘unstable’ section has settled considerably. (‘Stable’ and ‘unstable’ are here used in the sense of a serviceability limit state and not an ultimate limit state.) Boreholes were drilled at the toe of each section to collect samples for laboratory testing. Additional holes were drilled at the mid-height and the crest of the embankments.

2.2 Test Procedures

Standard lever-arm consolidation frames (oedometers) with cells fitted with 64-mm-diameter by 19-mm-high consolidation rings were used for the one-dimensional compression tests carried out in this program. Tests were conducted to examine the compression behaviour of the clay with changes of temperature. Given this, a total of six ASTM standard consolidation tests were carried out at 3°C and 21°C. For the low temperature tests, the oedometer frames were placed in an environmental chamber which could control temperatures within ±1°C.

To inhibit swelling and reduce consequent destructuring, the first applied pressure was 23 kPa. Loads on the samples were increased using a load increment ratio of about 1 (that is, doubling of the applied load with each increment). Each of the early loads was applied for 24 hours. Following loading to the maximum applied stress (about 1300 kPa), the specimens were unloaded in stages and then reloaded again to the maximum applied stress. This vertical stress was then maintained for five to six days in order to measure the secondary compression behaviour of the clay. Changes in height of the specimen were measured throughout the consolidation process (ASTM D2435-96). Natural water contents and Atterberg limits of the specimens were determined using ASTM D2216-98 and ASTM D4318-98.

Table 1 shows the initial properties of the specimens and the temperatures at which tests were carried out.

Table 1. Material properties and test temperatures.

Test	Section	Depth (m)	w (%)	I_p (%)	T (°C)
HBO09	Stable	1.5 to 2.3	35.9	37.4	21
HBO10	Stable	1.5 to 2.3	37.0	35.0	3
HBO11	Stable	1.5 to 2.3	37.7	40.1	21
HBO12	Stable	1.5 to 2.3	39.7	40.4	3
HBO13	Unstable	6.0 to 6.8	35.6	31.9	21
HBO14	Unstable	6.0 to 6.8	32.1	30.1	3

3 RESULTS AND ANALYSIS

When a soil is loaded one-dimensionally under a constant normal stress, its compression has two components. One, primary consolidation, is associated with the hydrodynamic expulsion of water, increasing effective stress, and consequent reorganization of the soil structure. Two, the clay continues to settle even after excess pore water pressures have dissipated and effective stresses are constant. Secondary compression (creep) is the change in volume of soil caused by the adjustment of the soil fabric after primary consolidation has been completed. It is associated with viscoplastic reorganization of diffuse double layers (DDLs) around clay particles.

3.1 Primary Consolidation Behaviour

Primary consolidation is the change in volume of soil caused by the expulsion of water from the voids and transfer of loading from the excess pore water pressures to the soil particles. The primary consolidation stage is characterized by the compression index C_c (in the normally consolidated range). The values of C_c are obtained from plots of void ratio (e) versus log of normal stress (σ'_z) (Budhu 2007):

$$C_c = \frac{e_1 - e_2}{\log(\sigma'_{z2}/\sigma'_{z1})} = \frac{|\Delta e|}{\log(\sigma'_{z2}/\sigma'_{z1})} \qquad [1]$$

The unloading/reloading index (or recompression index) (C_r) and coefficient of consolidation (c_v) values were also calculated. The C_r is the average slope of unloading/reloading curves in plots of void ratio (e) versus log of normal stress (σ'_z). For c_v, the t_{50} values (times to 50% consolidation) were used for calculating the c_v for the load of 650 kPa using the log time method (Budhu 2007). Table 2 summarizes the values of C_r, C_c and c_v.

Table 2. Test Results.

Test	Section	T (°C)	C_r	C_c	c_v (m^2/yr)	$C_{\alpha e}$
HBO09	Stable	21	0.0653	0.229	0.465	0.0039
HBO10	Stable	3	0.0753	0.242	0.346	0.0038
HBO11	Stable	21	0.0630	0.246	0.631	0.0050
HBO12	Stable	3	0.0690	0.255	0.329	0.0050
HBO13	Unstable	21	0.0570	0.261	0.832	0.0066
HBO14	Unstable	3	0.0490	0.229	1.032	0.0059

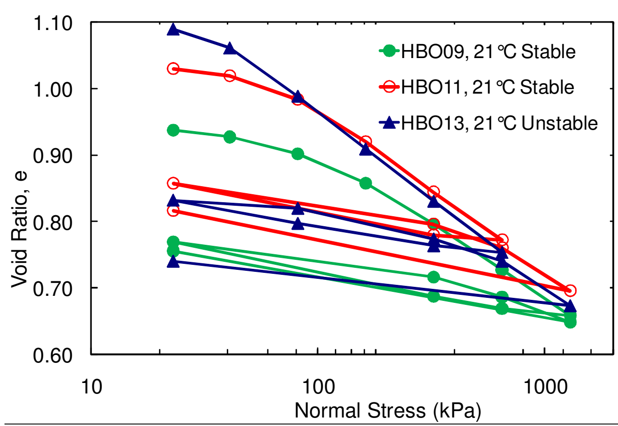

Figure 2. Void ratio versus log normal stress plots for the 21°C tests

Figure 2 shows plots of void ratio versus log(normal stress) for the tests carried out at 21°C. Specimens HBO09 and HBO11 were taken from the stable section. Specimen HBO13 was taken from the unstable section. Figure 2 shows that the normal consolidation curve for HBO13 has a slightly steeper slope (that is, higher C_c, see Table 2) than HBO09 and HBO11. That is, the specimen from the unstable section is more compressible than the specimens from the stable section. The consolidation curves for HBO09 and HBO11 are essentially parallel. This shows a high level of repeatability of response. The unload-reload sections also showed good consistency, with only relatively small hysteresis. Despite initially loading the specimens to 23 kPa to inhibit swelling, the preconsolidation pressures are not well-defined.

Figure 3 compares plots of void ratio versus log(normal stress) for samples from the stable section and unstable section at different temperatures. Figures 3(a) and 3(b) show that tests from the stable section have a slightly steeper slope on the loading path at the lower temperature (3°C) than at room temperature (21°C). Conversely, Figure 3(c) shows that the higher the temperature, the steeper is the slope of the loading path and therefore the higher is the value of C_c (See values of C_c on Table 2). As illustrated in Figure 2, the specimen from the unstable section is more compressible, presumably as a result of a weaker soil structure. Figure 3(c) shows that it tends to compress more at the higher temperature.

3.2 Secondary Compression Behaviour

Secondary compression is usually characterized by the index $C_{\alpha e}$, defined as the slope of the curve obtained by plotting void ratio (e) versus log (time) in the normally consolidated range of loading (Yin et al. 2002, Budhu 2007, Kelln et al. 2008):

$$C_{\alpha e} = -\frac{e_1 - e_2}{\log(t/t_p)} = \frac{|\Delta e|}{\log(t/t_p)}\ ;\ t > t_p \qquad [2]$$

Table 2 shows values of $C_{\alpha e}$ calculated from the specimens in this program.

Figure 4 shows plots of void ratio versus log(time) for the tests carried out at 21°C. As seen in this figure, the HBO13 creep curve has a relatively steeper slope (that is, higher $C_{\alpha e}$, see Table 2) compared to related values from HBO09 and HBO11.

Figure 5, which shows plots of void ratio versus log(time), illustrates distinctly different creep responses of clay specimens from the unstable section and specimens from the stable section. Values of $C_{\alpha e}$ are shown in Table 2. From the stable section, Figures 5(a) and 5(b) show that tests at 3°C and 21°C produce plots with similar slopes. Changes in temperature do not seem to affect the creep behaviour of the clay in the stable section. Figure 5(c) for the unstable section, shows a distinctly different behaviour. This figure shows that the results from the 21°C test were steeper than corresponding values in the 3°C test. This illustrates that the creep behaviour of these specimens is affected by changes in temperature. This would be expected from consideration of the effects of temperature on diffuse double layers, including its separate effects on the dielectric constant, (Mitchell and Soga 2005). Specimens from the unstable section tested at higher temperature, have a higher secondary compression index

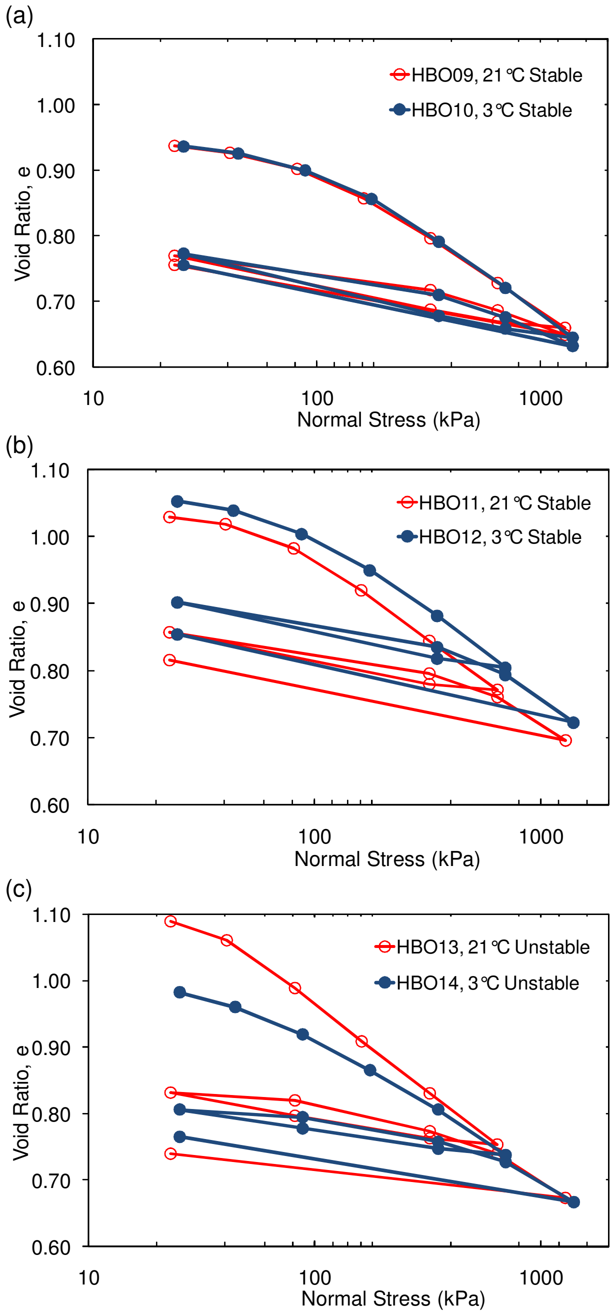

Figure 3. Void ratio versus log normal stress plots for (a) 21°C test (HBO09) and 3°C test (HBO10) at the stable section (b) 21°C test (HBO11) and 3°C test (HBO12) at the stable section and (c) 21°C test (HBO13) and 3°C test (HBO14) at the unstable section

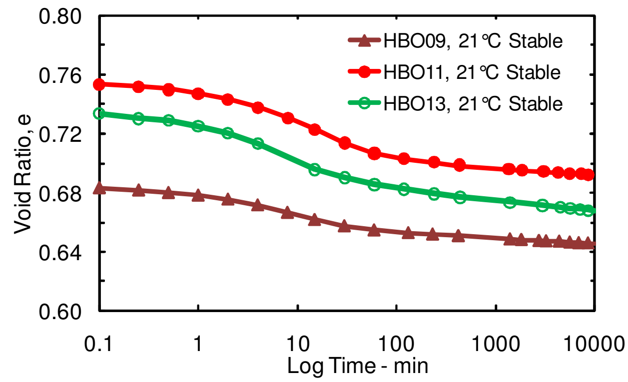

Figure 4. Void ratio versus log time plots for the 21°C tests

3.3 Ratio of Secondary to Primary Consolidation Indices

The secondary compression and primary compression indices of clays depend on the mineralogy of the clay particles and the chemistry of the pore fluid. They have been related empirically through the ratio $C_{\alpha e}/C_c$, which can frequently be considered constant in the normally consolidated range for a given soil (Mesri and Godlewski 1977). Mesri et al. 1995 showed that the ratio of $C_{\alpha e}/C_c$ defines the compression behaviour of clays and generally remains within a narrow range of perhaps 0.04 to 0.08.

Table 3 shows values of $C_{\alpha e}/C_c$ obtained from the present series of tests. The results are inconclusive. Comparisons of values of $C_{\alpha e}/C_c$ for HBO09, HBO10, HBO11, and HBO12, (samples taken from the stable section), show that the ratio of $C_{\alpha e}/C_c$ slightly increases with increases of temperature. On the other hand, results from HBO13 and HBO14 show a different trend. The values of $C_{\alpha e}/C_c$ reduce when temperature increases. The differences are small in each case.

Table 3. $C_{\alpha e}/C_c$ ratio values.

Test	Section	T (°C)	$C_{\alpha e}$ / C_c
HBO09	Stable	21	0.0170
HBO10	Stable	3	0.0157
HBO11	Stable	21	0.0203
HBO12	Stable	3	0.0196
HBO13	Unstable	21	0.0253
HBO14	Unstable	3	0.0258

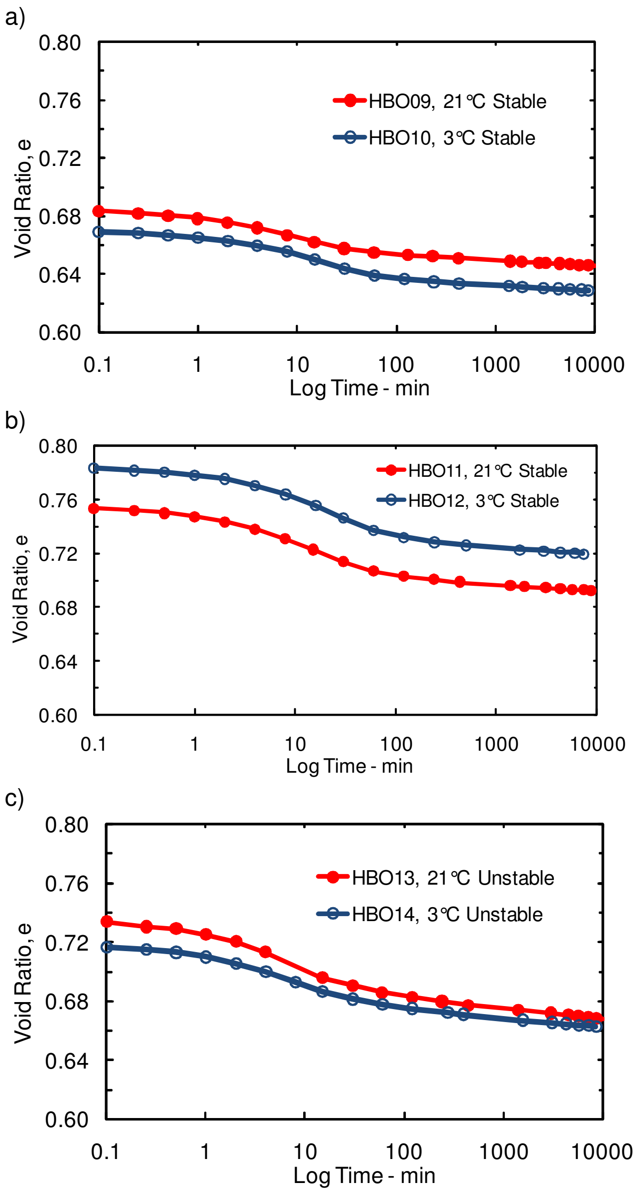

Figure 5. Void ratio versus log time plots for (a) 21°C test (HBO09) and 3°C test (HBO10) at the stable section, (b) 21°C test (HBO11) and 3°C test (HBO12) at the stable section, and (c) 21°C test (HBO13) and 3°C test (HBO14) at the unstable section

4 SUMMARY

Creep is an inherent feature of the behaviour of clays. It arises from the viscoplastic rearrangement of interparticle contact forces over time. Viscoplastic behaviour is seated in the interaction of diffuse double layers (DDLs) of adsorbed water that surround the electrically charged surfaces of clay particles. Under constant effective stress, the separation of particles depends on potential distributions in the DDLs. It therefore depends on particle mineralogy, pore fluid chemistry and temperature, as expressed for example by Gouy-Chapman theory (Mitchell and Soga, 2005). The thickness of the DDL varies with temperature and with the dielectric constant, which itself varies with temperature.

It could be expected therefore that plastic clays, which have high surface activities, could be expected to exhibit high viscoplastic effects that would depend on temperature. Evidence for the importance of viscoplasticity, which is important not only in compression but also in shear, has been recently identified by Kelln et al. (2009), who also commented that primary consolidation and creep must be considered concurrent mechanisms.

With this background, it appears, that $C_{\alpha e}$ / C_c for the Thompson clay (Table 3) is lower than one would expect for clay with moderate plasticity (I_P of 30 – 40%). Similarly, the small and non-systematic variations of $C_{\alpha e}$ with temperature are unexpected. It should be noted however that the specimens from the stable and unstable sections came from different depths and had therefore experienced different levels of weathering disturbance. Further testing is being done on specimens from similar depths.

The longer term objective of this project is to develop the elastic viscoplastic models outlined by Yin et al. (2002) and Kelln et al. (2008) to take account of variations of $C_{\alpha e}$ with temperature. With anticipated climate changes and increased development in Northern Canada, it will become more important to be able to simulate the performance of civil engineering infrastructure as temperatures change with time.

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