Results from an Instrumented Highway Embankment on Degraded Permafrost

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ABSTRACT
Roads and highways in Northern Manitoba are negatively affected by settlement of embankments in areas of degraded or degrading permafrost, particularly where the permafrost is locally discontinuous with mean annual temperatures close to 0ºC. Changes in temperature lead to thawing, settlements of road surfaces and shoulders, and lateral spreading. These can cause potentially dangerous trafficability issues. The highway embankment in this research project is 18 km northwest of Thompson, Manitoba. Research involves field instrumentation, data collection, laboratory testing, and numerical modeling. This paper reports data collected from the field instruments and numerical modeling during the first 18 months of operation.

RÉSUMÉ
Les routes dans le Nord du Manitoba sont négativement affectées par le tassement des digues dans les régions dégradées par le permagel, explicitement où le permagel est localement discontinu avec une température moyenne annuelle près de 0°C. Le changement en température résulte dans la décongélation, le tassement des surfaces et des accotements et le déplacement latéral. Ces derniers peuvent créer des problèmes de traficabilité dangereux. La digue discutée se trouve à 18 km au nord-ouest de Thompson, Manitoba. La recherche comprend des instruments d'observation, la collecte de données, des épreuves de laboratoire et de la modélisation numérique. Les résultats de ce papier sont accumulés par les instruments d'observation et de la modélisation numérique durant les premiers dix-huit mois d'activité.

1 INTRODUCTION

Permafrost is defined as ground, whether soil or rock, that remains at or below a temperature of 0°C for a minimum period of two years (Williams 1986). The characteristics of permafrost vary by climatic, topographic, geographic, hydrologic and geological factors. The thicknesses of the permafrost and the associated active layer are controlled by local climate (including effects of long-term warming), the insulating cover of vegetation and snow, drainage, the thermal properties of soil and rock, and the complex boundary layer between soil and air (Osterkamp and Lachenbruch 1990). 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, warming may produce thawing of ground ice, thickening of the active layer, large settlements, and non-recoverable shear deformations. Observations have shown that mean annual air temperatures near 0ºC may be increasing more rapidly than further north or further south. Increases of perhaps 2.7°C to 2.9°C have been suggested by 2040-2070 (CCCSN 2009). Unless mitigated by engineering interventions, these temperature changes will lead to degrading of permafrost, and corresponding impacts on civil engineering infrastructure.

Frozen soil is stronger and less compressible than unfrozen soil. Frozen silty sands, silts, and silty clays frequently contain layers of ice, often in distinct lenses 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, these ice lenses

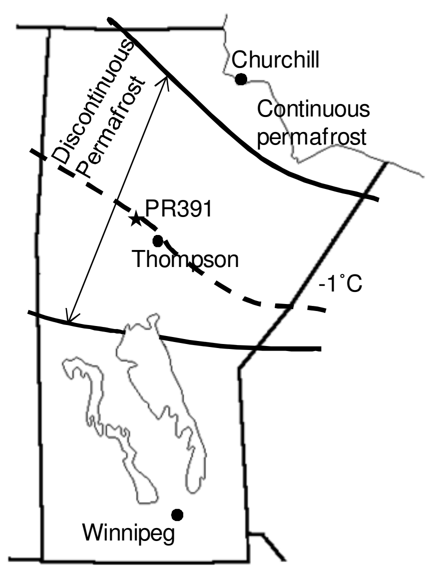

Figure 1. Location of test site and permafrost in Manitoba, Canada.

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 alter drainage patterns. They therefore contribute significantly to changes in the engineering behaviour of soils in areas of discontinuous permafrost.

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 the isotherm that corresponds approximately to a current mean annual air temperature of 0ºC (about 2500ºC-days of frost). The permafrost becomes continuous further north near the Hudson Bay coast.

Road, rail, and air communications are essential in the North and are becoming increasingly important as projects are developed to extract natural resources. Thawing of summer ice in the Arctic Ocean can be expected to lead to increased shipping into the port of Churchill. Additional roads and railways will have to be constructed into northern Manitoba and Nunavut over soils with engineering properties that may further degrade with climate change and land-use. Design and

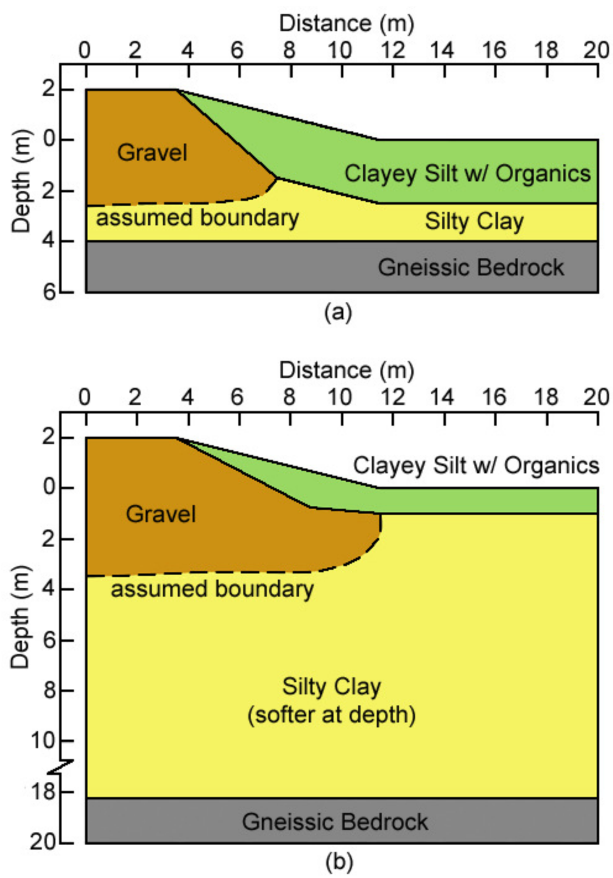

Figure 2. a) 'stable' section; b) 'unstable' section.

maintenance of this new infrastructure will have to take account of future warming over the anticipated life of these projects.

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. Manitoba Infrastructure and Transportation is facing increasing maintenance costs, reconstruction of damaged structures, and the replacement of an extensive system of winter roads in eastern Manitoba.

In order to improve design and maintenance procedures, Manitoba Infrastructure and Transportation (MIT) and the University of Manitoba (UM) are collaborating on several projects that involve field instrumentation, laboratory testing and numerical modeling. This paper reports work on a project on Provincial Road PR391 in a region of discontinuous permafrost about 18 km northwest of Thompson, Manitoba (Figure 1). This is the only road connecting Thompson to northern mining towns, hydroelectric generating stations, and first Nations communities in north-western Manitoba.

2 GEOTECHNICAL SITE INVESTIGATIONS AND FIELD INSTRUMENTATIONS

2.1 Site Investigation

The PR391 was initially constructed as a compacted earthen road on discontinuous permafrost in the mid-1960s and then converted to a gravel road in the early 1970s. (No construction logs have yet been found.) In the early 1980s, it was upgraded with a bituminous pavement surface. 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.

Responses by MIT included construction of stabilizing berms and insulating peat berms beside the project embankment in the early 1990s. Over a period of years, the berm settled into its foundation soil and essentially disappeared. It currently provides no additional support to the original embankment. Regular maintenance has added two to three metres of gravel fill since initial construction. What had been an asphalt pavement since the early 1980s has not been replaced. In places that require extra maintenance, the wearing surface has been returned to gravel from asphalt.

In 1991, drilling encountered frozen soil at depths from 1.9m to 10.5m below the toe of the embankment. Later drilling in 2005, detected frozen soil at depths from 4.6m to 10.7m. Perhaps surprisingly, no frozen soil was identified in the recent drilling program in late 2008 (continuous flight, solid stem augers). A later section will show no sub-zero temperatures below the active surface layer. Figure 2 is the authors' interpretation and simplification of the results of the site investigation at PR391 in 2008. Considerable maintenance was required at the site but records outlining the construction techniques and annual application of gravel are unavailable. The investigation in 2008 involved two cross-sections, one of which was designated as 'stable' and the other as 'unstable'. The 'stable' section is only about 2m high and has not deformed significantly. The 'unstable' section is also about 2m high above the surrounding natural ground. However, it settled considerably and is assumed to now contain about 5m - 6m of gravel (Figure 2). The gravel is partly from the original construction and partly from ongoing re-grading. 'Zero depth' in Figure 2 and in subsequent figures is referenced to the level of the original ground surface and of the surrounding undisturbed land.

The terms 'stable' and 'unstable' are here used in the sense of a serviceability limit state and not an ultimate limit state. There are no indications of deep-seated rotational movements. Boreholes were drilled at the mid-slope and toe of each section to examine the stratigraphy, collect samples for laboratory testing, and install instruments to record the behaviour of the foundation soils over several years. Batenipour et al. (2009a,b) provided additional information about site characterization, instrumentation and material properties for this project.

Figure 2 shows that soil conditions below the original ground level (at depth '0' in the figure) at the two sections are considerably different. The 'stable' section consists of approximately 2.0m of soft-to-firm clayey silt followed by nearly 2.0m of silty clay. At the mid-slope, the soil has been classified as clayey silt but contains peat and gravel intrusions varying from thin stratifications to pockets. The 'unstable' section consists of approximately 1.0m of clayey peat-silt, underlain by a layer of highly plastic clay. This clay is firm, brown, silty clay at upper levels and becomes very soft and grey to a depth approaching 18.0m. Both sections are underlain by gneissic bedrock. The exact quantity of gravel is unknown but has been estimated from borehole logs. Preliminary results of laboratory tests have been reported by Batenipour et al. 2009a. The surrounding area is relatively poorly drained - there is free-standing water within approximately 20.0m of the embankment toe during most of the year.

2.2 Instrumentation

Instrument clusters were installed at the mid-slope and toe of both the 'stable' and 'unstable' sections (Figure 3). The instrumentation includes thermistor strings at 1m intervals, vibrating wire piezometers and standpipes, surface settlement plates, slope inclinometers, and lateral displacement extensometers at the toe of the embankment. The piezometers and standpipes have been used to help identify possible upwards or downwards hydraulic gradients. Temperatures have now

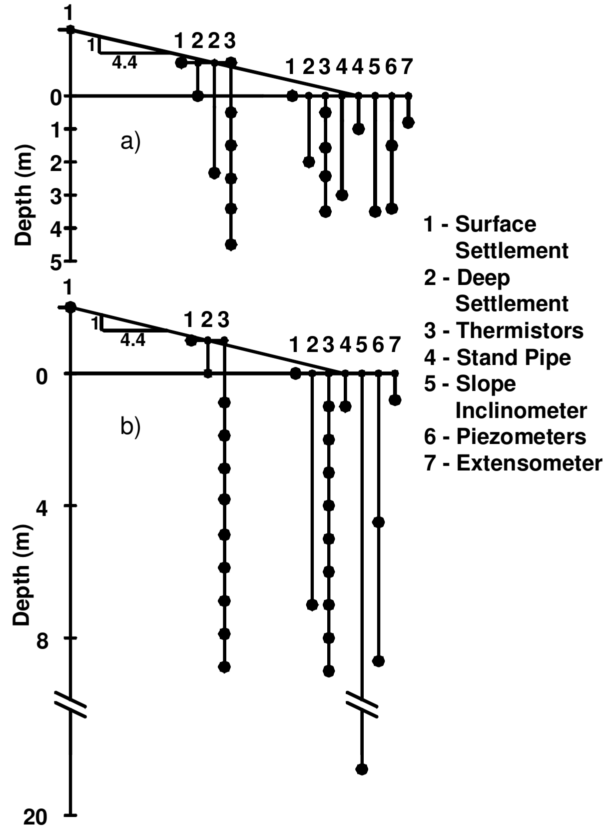

Figure 3. Instrumentation at a) 'stable' section; b) 'unstable' section.

been collected during two winter cycles. Readings are planned for another one to two years. This will allow calibration of thermal properties over several freeze-thaw cycles. Future modeling will simulate temperature changes resulting from anticipated climate warming.

Readings are taken monthly on a data acquisition system and downloaded manually. Telephone access was not available at reasonable cost at this relatively remote site.

3 DATA FROM FIELD INSTRUMENTATION

This section of paper reports reading of monthly temperatures, pore water pressures, and lateral displacement extensometer data.

3.1 Ground Temperature Data

Soil temperatures vary from month to month as a function of solar radiation, precipitation, seasonal changes in overlying air temperature, vegetation cover, type of soil, and depth in the ground. Figures 4 and 5 show the monthly temperature profiles for one cycle of cooling and heating between November 2008 and October 2009. The two figures show data from instrument clusters at a) mid-slope and b) toe for the 'unstable' and 'stable' sections respectively. Please note the different scales used for temperatures and depths in the figures, particularly in Figure 4b.

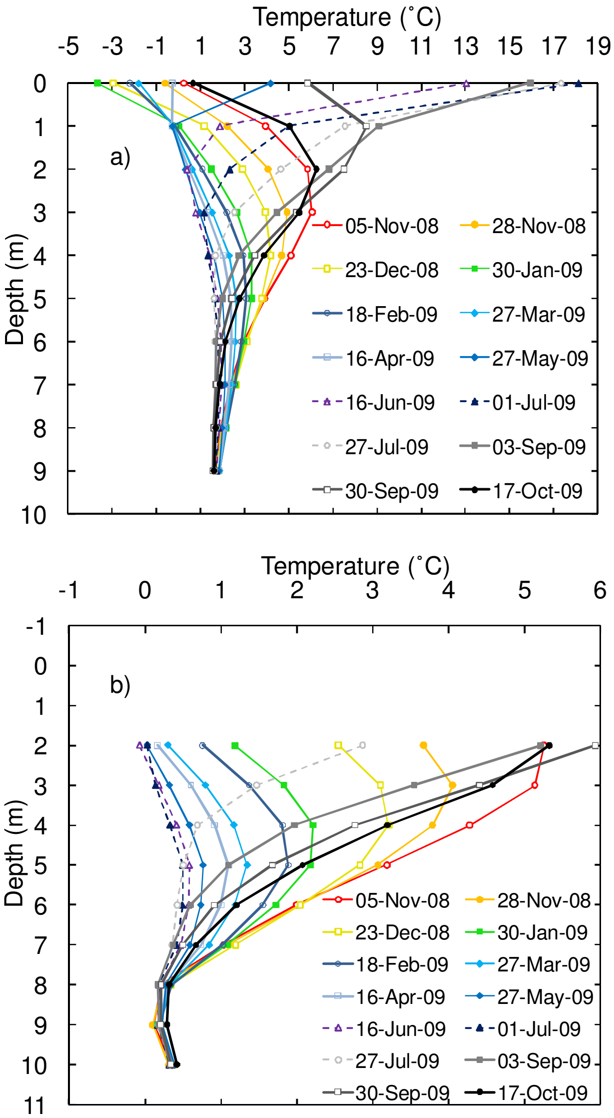

Figure 4. Monthly temperature profiles between November 2008 and October 2009 for the unstable section at the a) toe; b) mid-slope.

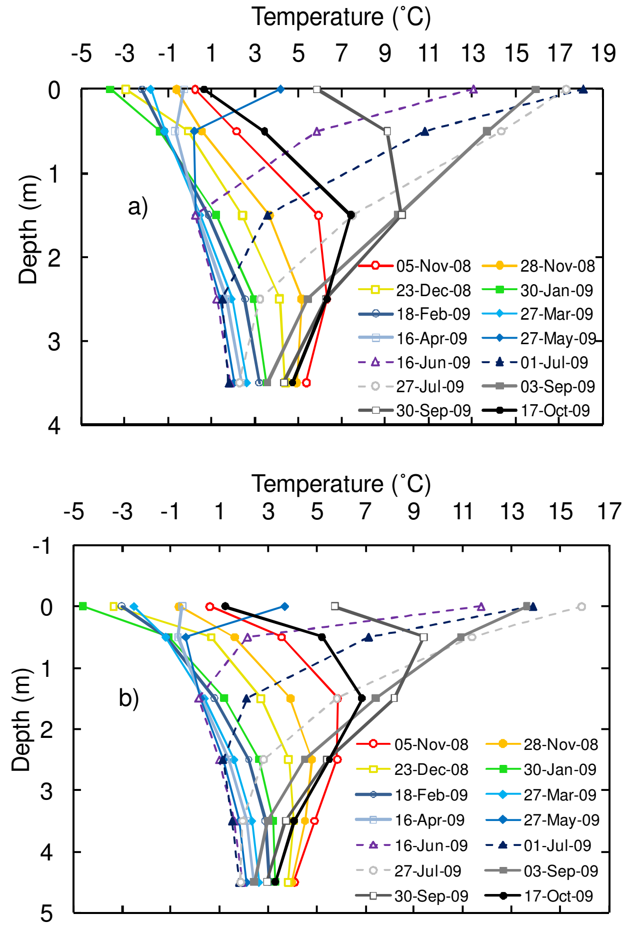

Figure 5. Monthly temperature profiles between November 2008 and October 2009 for the stable section at the a) toe; b) mid-slope.

With increasing depth in the ground, the seasonal difference in temperature decreases. The depth of zero annual temperature amplitude is the depth at which the seasonal variations of temperatures are essentially zero (Andersland and Ladanyi, 2004). Figures 4a and 4b show that the depth of zero annual amplitude at the 'unstable' section is about 8m – 9m below the ground level. Although the respective temperatures profiles for the mid-slope and toe converge at about the same depth, there is approximately a 1.6℃ difference. Figures 5a and 5b do not show the level of zero annual amplitude at the stable section. The bedrock was encountered at 4m depth. It was not drilled to install thermistors.

In Figure 4a, for example, the results start in early November 2008, when temperatures at shallow depths were already beginning to decrease with the onset of winter. Gradually, the low temperatures move deeper into the soil profile. Heating near the surface begins in mid-April and again moves slowly down the profile during summer and fall months until cooling begins again in late September.

Figures 6 and 7 show minimum, average, and maximum temperature profiles with depth for the 'unstable' and 'stable' sections respectively between November 2008 and October 2009. As in Figures 4 and 5, results are shown for the thermistor clusters at both the toe and mid-slope of the two sections. The geothermal gradient is the rate of change of temperature with depth in the ground. The average temperature profile indicates approximately whether there is a net heat flow into, or out of, the ground surface over a period of one year. Figures 6 and 7 show the average temperature profiles over one year of observation at the four instrumented locations decrease with increasing depth. This indicates warming of the ground surface and heat flux into the soil. It is important to note, however, that this observation is based on only one year of temperature readings. It also supports the observations reported in an earlier paragraph that initially frozen soil has thawed during the lifetime of the embankment and that no sub-zero temperatures were observed during 2008-09 (Figures 4 and 5). Future

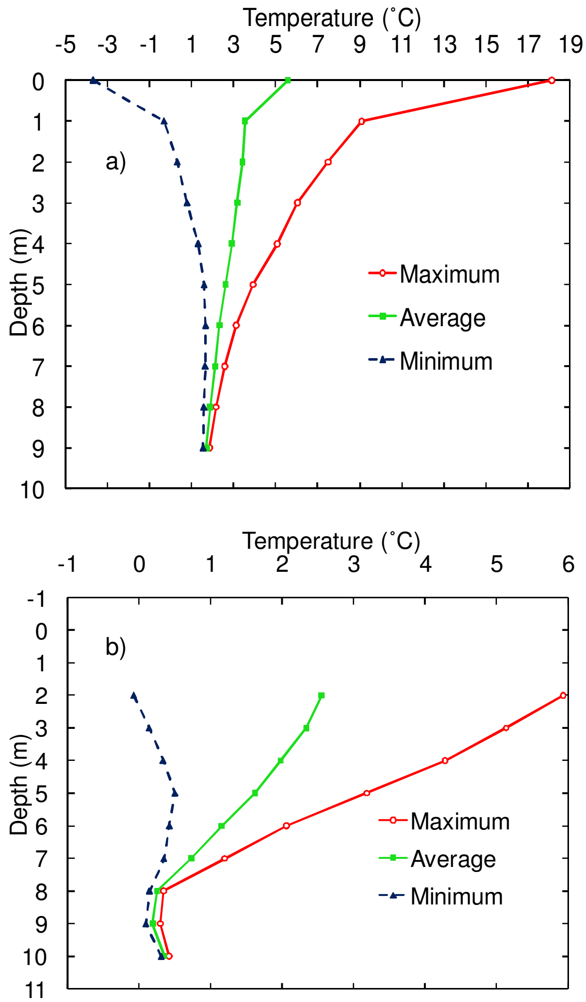

Figure 6. Annual temperature envelopes between November 2008 and October 2009 for the unstable section at the a) toe; b) mid-slope.

collection and analysis of temperature data will provide a better understanding of the annual temperature changes in the ground.

An earlier section noted that successive site investigations showed loss of ice or frozen soil during the life of the fill. This is now confirmed by the temperature profiles in Figures 4 and 5. Loss of ice may explain the major settlements experienced at the unstable section.

Figures 8 and 9 plot temperature versus time (here shown as date) at different depths for the 'unstable' and 'stable' sections respectively. As expected, the figures show, perhaps more clearly than Figures 4 and 5, that the seasonal variations in temperature decrease with depth. The dates of the maximum and minimum values shift towards later dates with increasing depth. The thermistors in the active layer near the ground surface stay below 0℃ for about five months from November to May.

3.2 Ground Water Data

Figure 10 shows the variation of pore water pressure versus time (date) at different depths in the 'unstable' and

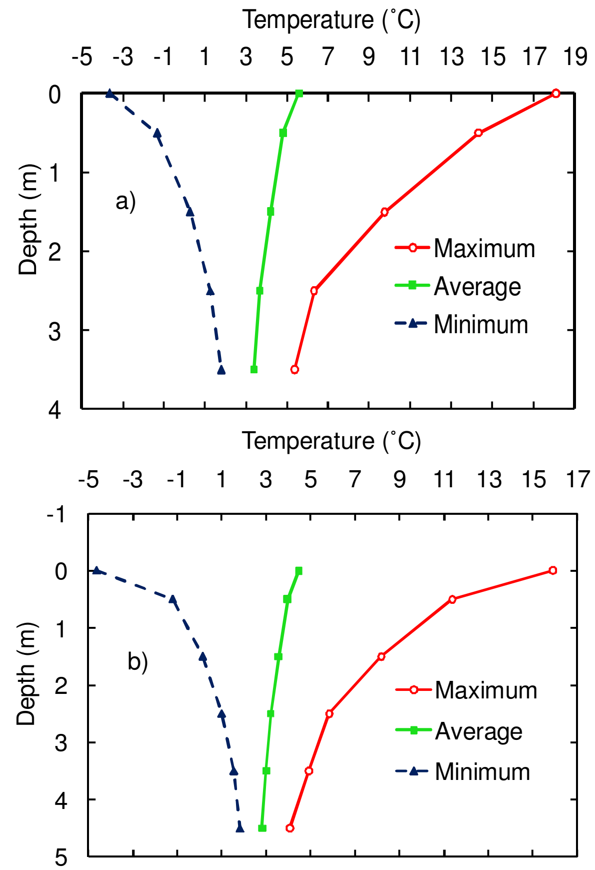

Figure 7. Annual temperature envelopes between November 2008 and October 2009 for the stable section at the a) toe; b) mid-slope.

'stable' sections. Figure 10a shows that pore water pressure increases from May when thawing occurs and decreases from October when freezing season begins at the 'unstable' section. Figure 10b shows the same trend at the 'stable' section, except the pore water pressure increases at an earlier date, in March. This can be explained by the proximity of the piezometers to the ground surface at the 'stable' section.

Figure 10a illustrates a relatively limited development of seasonal pore water pressures. The maximum and minimum differences of pore water pressures at depths are about 50kPa and 40kPa, which occur in November and July respectively. Considering 4.2m difference in depths and having the water table close to the ground level, 10kPa pore water pressure difference was developed during the thawing season. This can establish the presence of upward flow in the thawing season, reduction of shear strengths, and higher compressibilities. As noted before, future collection and analysis of data will help develop a more complete understanding of the changes of pore water pressure in clay, flow in the ground, and subsequently the effective stress and the shear strength of the embankment clay.

3.3 Lateral displacements

Figure 11 shows horizontal strains versus time for an 18 month period in 2008 - 2010 at the toe of the 'unstable'

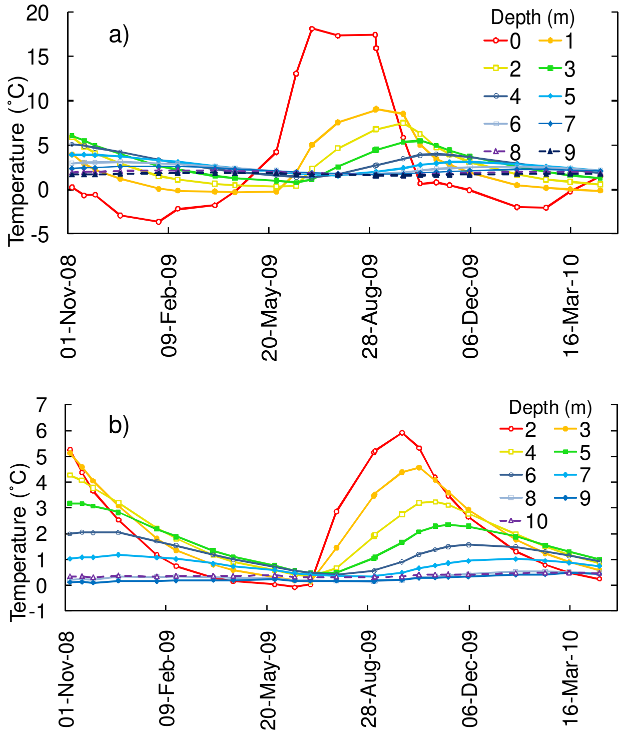

Figure 8. Temperature vs. time at different depths for the unstable section at the a) toe; b) mid-slope.

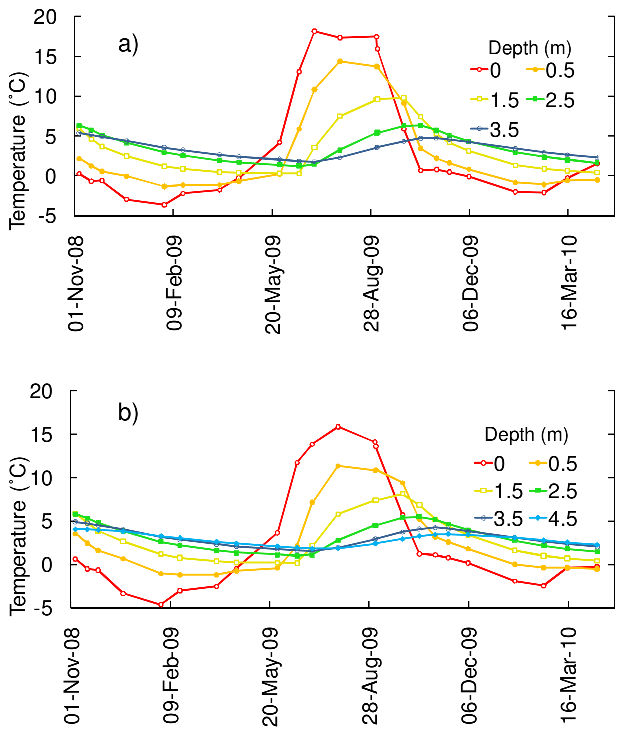

Figure 9. Temperature vs. time at different depths for the stable section at the a) toe; b) mid-slope.

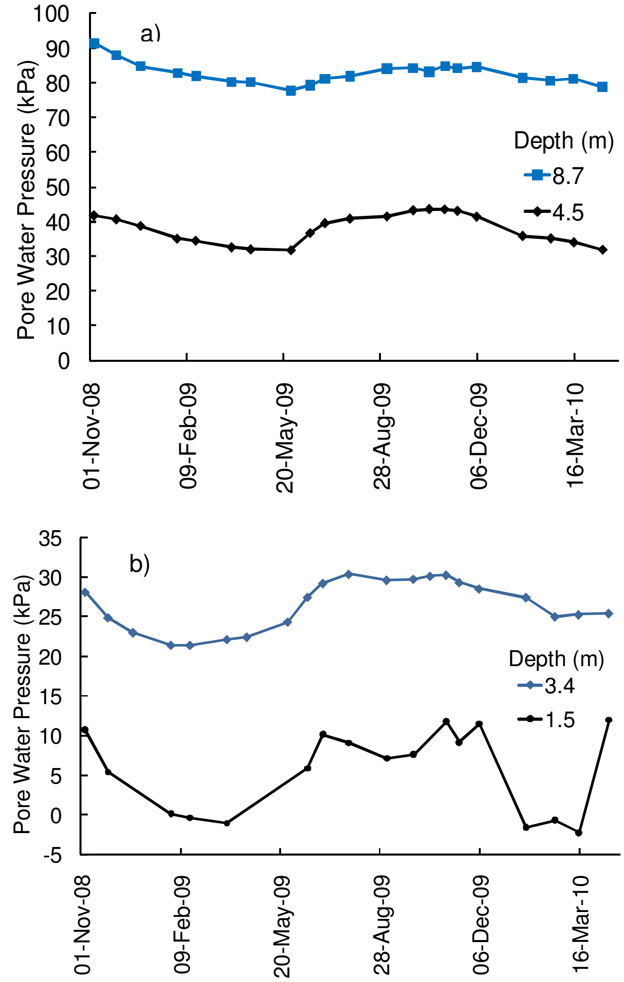

Figure 10. Pore water pressure vs. time at different depths at the toe for the a) unstable section; b) stable section.

section. Originally, soil extensometers were installed at the toe of embankment at a depth of 0.8m in both the 'unstable' and 'stable' sections. The intention of the instruments was to monitor lateral deformations of soil, and hopefully at a later date, to be able to relate lateral spreading and longitudinal cracking of the road surface with movements at the toe. Unfortunately, the extensometer at the 'stable' section failed shortly after installation. Figure 11 shows displacements corresponding to about 0.55% horizontal strain over a period of 18 months at the 'unstable' section. The figure indicates that the rate of deformation increased rapidly during the first freezing season (November 2008 to June 2009). It then decreased slightly from July to October 2009 and then increased again in the period November 2009 to May 2010. Some of the early movement may result from natural re-densification of soil around the extensometer. Close attention will be given to future readings.

Currently the vertical and lateral deformation of embankment is being monitored by the extensometer at

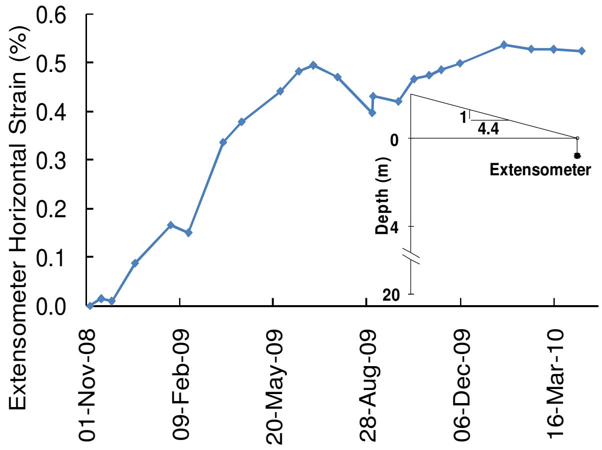

Figure 11. Extensometer horizontal strain vs. time at 0.8m depth at the toe of the unstable section.

the toe of the 'unstable' section. Measurements are also being taken from slope inclinometers, and settlement gauges installed at different depths below the shoulder of the road, at mid-slope, and at the toe of the embankment at both the 'stable' and 'unstable' sections. Readings and surveying are being taken monthly. The movement of PR391 embankment will be thoroughly analyzed for at least two completed freeze-thaw cycle seasons, once enough data are available, to determine whether the first year's movements were representative. This information will be used later in numerical modeling of the embankment.

4 NUMERICAL MODELING PROGRAM

Thermal numerical modeling of the PR391 embankment is being developed using the cross sections indicated in Figure 2. The modeling is being performed in TEMP/W[1]. Calibration of the modeling uses historic climate data from Environment Canada's monitoring station at Thompson Airport and the measured ground temperatures shown in Figures 4 and 5.

At this preliminary stage, the model has been developed in two stages; a steady state condition to establish the initial isotherms in the model and two subsequent transient analyses using separate sets of climate data. The first set of data starts on 01 October 2008 and the second on 01 October 2006. Both sets end on 31 March 2010. This gives two analyses, one over a period of 1.5 years and a second over 3.5 years. The date of 01 October was chosen to give the model one month of climate data before the field measurements began, while the additional two years of data allow potential effects of the initial steady state analysis to be dissipated within the model.

In the absence of measured thermal properties for the soils on the site, thermal properties have been assumed using empirical methods and typical values found in the

[1] GeoStudio 2007, GeoSlope International, Calgary, Alberta.

Table 1: Thermal Material Properties

Material	Vol. Heat Capacity (kJ/m³/°C)		Thermal Conductivity (kJ/d/m/°C)		Insitu VWC* (m³/m³)
	C_{vu}	C_{vf}	k_u	k_f	
Gravel	2400	2160	220-260	340-410	0.12 - 0.18
Clayey Silt w/ Organics	4020	3090	125	220	0.51
Silty Clay	3850	2720	180	310	0.44

* Volumetric Water Content

literature for similar materials (Andersland and Ladanyi, 2004, Côté and Konrad, 2005). Efforts are currently underway to establish actual thermal properties as part of this ongoing research. Table 1 indicates the volumetric heat capacity, thermal conductivity and insitu volumetric water contents that were assume in the modeling.

Boundary conditions on the model include a zero flux boundary along the centreline of the embankment, constant temperatures at a depth of approximately 9m as indicated in Figures 4 and 5, and a constant temperature and climate data at the surface in the steady state and transient models, respectively. Temperatures for the surface boundary in the steady state analyses were taken from the first day of the respective sets of climate data.

Initial results for the modeling are promising. Figure 12 shows the 'best' results obtained from the modeling thus far. These results compare the measured and modeled temperature profiles with depth at the mid-slope of the 'unstable' section. The temperature profiles generated using the 1.5 year and 3.5 year climate data are compared to data measured at three dates in the field program. The dates chosen for Figure 12 correspond approximately to the maximum, minimum and average temperature profiles along the mid-slope of the 'unstable' section. Days indicated in the figure are days elapsed from the start of the two transient models.

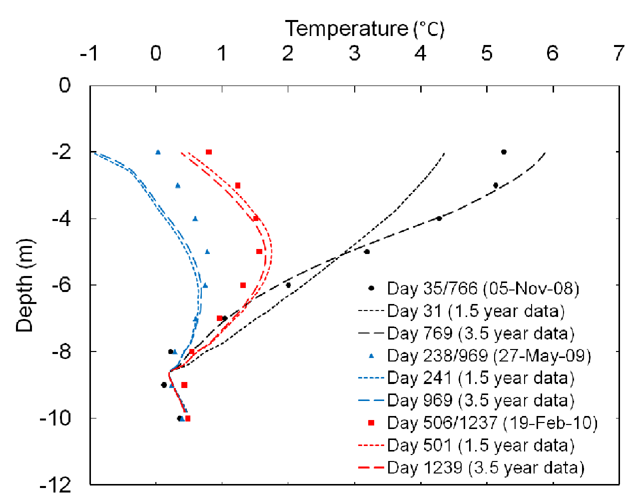

Figure 12: Unstable section, mid-slope, temperature vs. depth modeling results for three dates.

Overall, the modeling results show a good fit between the modeled and measured temperatures. Comparisons of simulated temperatures for day 35, day 769, and the respective measured temperatures illustrate potential effects of the starting date and the isotherms established in the steady state analysis. The results indicate that using the longer (3.5 year) climate data diminishes the effect of the start date on the subsequent isotherms.

Recent work by Kelln et al. (2008, 2009) on embankment settlements has shown the importance of including creep behaviour in numerical simulations of embankment behaviour. Their work assumes that viscous behaviour is present through all stages of a soil's response to loading and that creep behaviour can be described by a single creep parameter C_α, which depends on plasticity index (and temperature) but is otherwise constant in the same way that the compression index C_c and the unload-reload index C_r are constant.

Creep behaviour depends on the viscosity of bipolar water molecules adsorbed on the charged surfaces of the clay particles (Mitchell and Soga 2005). It therefore varies with temperature. Batenipour et al (2009a) provided preliminary results for the variation of C_α with temperature at the Thompson site. At a later stage, the authors intend to add temperature-dependent creep behaviour to the elastic-viscoplastic model developed by Kelln and his colleagues (2008).

5 SUMMARY

The paper reports 18 months of measurements of temperatures, pore water pressures, and toe displacements from a low highway embankment near Thompson in northern Manitoba. It also reports preliminary results from numerical modeling of temperature distributions with time. The project is also supported by a laboratory testing program on samples from the site.

The results show the expected variations of temperature with seasonal changes. These have been modelled with some success, bearing in mind that this is a 'history-matching' exercise based on assumed thermal properties of the embankment and the foundation soils. As is customary in this work, using data from a longer period for inputting transient weather information improves the quality of the simulations. It is interesting to note that since the road was first constructed in 1991, frozen soil found at depth in the initial site investigation is no longer seen in the recent measurements.

Future work on the project involves field measurements for at least one more winter season and further development of thermal and elastic thermo-viscoplastic models.

ACKNOWLEDGEMENTS

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REFERENCES

Andersland, O. B., Ladanyi, B., (2004). *Frozen Ground Engineering* (2nd Edition). John Wiley & Sons.

Batenipour, H., Kurz, D., Alfaro, M., and Graham, J. 2009a. Effect of Temperature on the 1-D Behaviour of Plastic Clay. In Proceedings of 62nd Canadian Geotechnical Conference, Halifax NS, September 2009, pp. 47-52.

Batenipour H., Kurz D.R., Alfaro M.C., Graham J. and Ng T.N.S., 2009b. Highway Embankment on Degrading Permafrost. In proceedings of the 14th Conference on Cold Regions Engineering in Duluth, Minnesota, USA, pp. 512-521.

CCCSN (Canadian Climate Change Scenarios Network) 2009. Arctic Ensemble Scenarios.

Côté, J. and Konrad, J.-M. 2005. Thermal conductivity of base-course materials. Canadian Geotechnical Journal **42**, 61-78.

Kelln, K., Sharma, J., Hughes, D, and Graham, J. 2008. An Improved Framework For An Elastic-Viscoplastic Soil Model. Canadian Geotechnical Journal, **45**, 1356-1376.

Kelln, C., Sharma, J., Hughes, D. and Graham, J. 2009. Finite element analysis of an embankment on soft estuarine deposit using an elastic-viscoplastic soil model. Canadian Geotechnical Journal **46**, 357-368.

Konrad, J.-M. 2008. Freezing-induced water migration in compacted base-course materials. Canadian Geotechnical Journal **45**, 895-909.

Mitchell J. K. and Soga K., 2005. Fundamentals of Soil Behavior, John Wiley & Sons.

Osterkamp, T.E. and Lachenbruch, A.H. 1990. Thermal regime of permafrost in Alaska and predicted global warming. ASCE Journal of Cold Regions Engineering **4**, 38-42.

Williams, P. J. 1986, *Pipelines & Permafrost: Science in a Cold Climate*, Carleton University Press, Don Mills, Ontario, 129 pp.