Deformations of a highway embankment on degraded permafrost

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ABSTRACT
Changes in temperature affect civil engineering infrastructure, particularly in areas with mean annual temperatures close to 0°C where permafrost is locally discontinuous. Climate warming and human activities can lead to increases in the temperature of permafrost and to thawing. In regions of discontinuous permafrost, thawing may produce thickening of the active layer, large settlements and non-recoverable shear deformations.
The paper examines the behaviour of a highway embankment about 18 km northwest of Thompson, Manitoba. Research involves site investigation, instrumentation, laboratory testing, and numerical modeling. The paper reports data from the field instruments during the first two years of operation, that is, over two freeze-thaw cycles. Results provide insight into the nature and cause of the embankment deformations and mechanisms producing functional failure (serviceability limit state) of highways on degraded permafrost.

RESUMEN
Los cambios en la temperatura afectan las obras de Ingeniería Civil, particularmente en áreas con temperatura promedio cercana a 0°C donde el permafrost es localmente discontinuo. El calentamiento global y las acitividades humanas pueden llevar a incrementar la temperatura del permafrost y generar descongelamiento. En regiones de permafrost discontinuo, el descongelamiento puede ocasionar el incremento de la capa activa del permafrost, aumentar los asentamientos del suelo y crear deformaciones cortantes irrecuperables.
Este artículo examina el comportamiento de un terraplén para una carretera localizado aproximadamente 18 km al noroeste de Thompson, Manitoba. La investigación involucra pruebas de campo, instrumentación, pruebas de laboratorio y modelamiento númerico. El artículo presenta resultados de monitoreo durante los dos primeros años de operación, es decir, durante dos ciclos de congelamiento/descongelamiento. Los resultados proveen una evaluación de las causas de la deformación de terraplenes y los mecanismos que producen la falla funcional (estados limites de servicio) de las carreteras construidas en permafrost degradado.

1 INTRODUCTION

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 discontinuous permafrost may be locally present. 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 with climatic, topographic, geographic, hydrologic and geological factors.

Climate warming and human activities can lead to increases in ground temperature and thickening of the active layer - the top layer of soil that freezes during winter months and thaws during summer. Seasonal cycles of freezing and thawing can cause large settlements and non-recoverable shear deformations in fine-grained soils (Brown 1967).

Observations show that mean annual air temperatures in Canada are increasing more rapidly close to the 0°C isotherm than further north or further south (CCCSN 2009, IPCC 2007). Increases of 2.7 °C to 2.9 °C by 2040-2070 have been suggested for the site of the authors' research in northern Manitoba. Unless mitigated by engineering interventions, these temperature changes will degrade existing permafrost and impact many forms of 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 migration of water to negative potentials at the freezing front (Konrad 2008). Heaving is produced in seasonally frozen soils as a result of increasing the volume of water in the void spaces as it freezes, and more importantly as a result of ice segregation during freezing, which produces alternating bands of frozen soil and ice.

When thawing occurs, whether as a result of seasonal warming, climatic warming over a longer period, or changes in heat transfer due to human or engineering activities, water from the melting ice moves towards the ground surface. Resulting decreases in effective stresses produce deformations and weakening in foundations for buildings and pipelines, airport runways, rail beds, and highway sub-bases, cuts and fills.

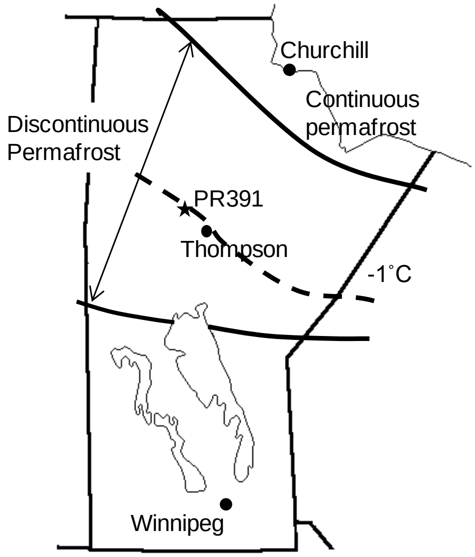

Figure 1. Location of test site and permafrost in Manitoba Canada

Because the ice lenses are typically irregular in thickness and extent, out-migration of water often produces differential settlements, lateral spreading, and reduced stability that lead to serviceability issues. . Typically, this *thaw settlement* is irregular (Hinkel et al. 2003).

Although the permafrost regions in northern Canada are not heavily populated, their economic importance has increased substantially in recent decades owing to the presence of mineral, petrocarbon and hydroelectric resources. Thawing of summer ice in the Arctic Ocean can be expected to lead to additional shipping into the port of Churchill. New roads and railways will have to be constructed into northern Manitoba and Nunavut over soils with engineering properties that may degrade with climate change and land-use. Design, construction, and maintenance of this new infrastructure will have to take account of future warming (Hinkel et al. 2003).

Figure 1 shows that Manitoba has discontinuous permafrost north of the isotherm for a current mean annual air temperature of about 0ºC (about 2500ºC-days of frost). The permafrost becomes continuous further north near the Hudson Bay coast.

Construction in permafrost regions requires special techniques where soil profiles contain ice lenses. Disturbance of existing ground cover can change the energy regime (Nelson and Outcalt, 1982). This often leads to thawing of underlying ice-rich permafrost and subsidence of the road bed. Deformations can be sufficiently large that the road becomes inoperable. Several methods are currently used to prevent or minimize thawing of permafrost and development of thermokarst. These include above-ground construction, use of thermosyphons, and buildings on piles or gravel pads above original ground level.

Highway construction in northern Canada follows broadly similar practices to those in warmer regions. Fills generally have high thermal conductivity, leading to heat transfer into underlying layers and thawing of previously frozen foundation soil (Batenipour et al. 2009b). Asphalt surfacing absorbs heat from the sun and transfers it to the embankment. Generally, degradation of permafrost begins at the toe of embankments and extends later beneath the centre. Melting reduces strength and increases pore water pressures when the hydraulic conductivity of the foundation soil is low. This leads to differential settlements, lateral spreading, and instability.

In order to improve local 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, 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. The site is in a region of discontinuous permafrost.

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. In the early 1980s, it was upgraded with a bituminous pavement surface. Since construction, changes in heat transfer have melted permafrost that had been detected earlier, particularly under embankments. Thawing led to large ongoing irregular deformations and dangerous traffic issues.

Responses by MIT included construction of stabilizing berms and insulating peat berms beside the project embankment in the early 1990s. Over some years, the berms settled into the foundation soil and essentially disappeared. They currently provide no additional support to the original embankment. Regular maintenance has added several metres of gravel fill since initial construction. Where extra maintenance is required, the wearing surface has been returned to gravel from asphalt: the asphalt pavement from the early 1980s has not been replaced.

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 a recent drilling program in late 2008 using continuous flight, solid stem augers. A later section will show no sub-zero temperatures below the active surface layer. Considerable maintenance was required at the site. Records outlining maintenance procedures and annual application of gravel are unavailable.

Figure 2 is the authors' interpretation of the results of the site investigation at PR391 in 2008. The investigation involved two cross-sections, one of which was designated as 'stable' and the other as 'unstable'. The stable section is only about 2m high (Figure 2a) and has not deformed significantly. The unstable section is also about 2m high above the surrounding natural ground. It has settled considerably and now contains about 5m - 6m of gravel (Figure 2b). The gravel is partly from the original construction and partly from ongoing re-grading. The '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.

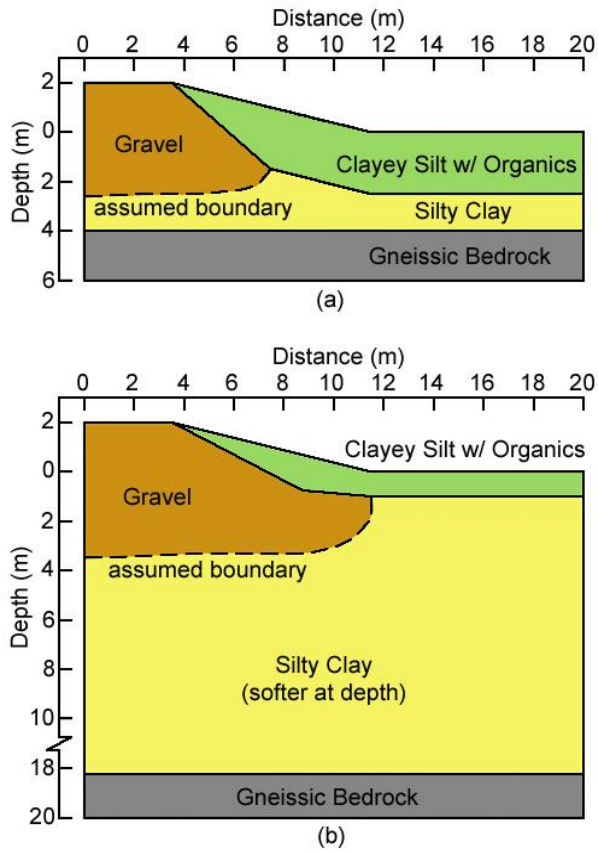

Figure 2. a) stable section; b) unstable section

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 and 2010) provided additional information about site characterization, instrumentation and material properties. This paper provides information on two full years of field measurements.

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 4.0m of soft-to-firm clayey silt/silty clay with peat intrusions that vary from thin stratifications to pockets near the toe. At the mid-slope, the soil is primarily silt to clayey silt. The toe of the unstable section consists of 1.0m of clayey peat-silt, 1.0m of fine gravel, followed 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 18m. The mid-slope of the unstable section consists of almost 2.0m of clayey peat-silt, over 2.0m of loose fine gravel, followed by firm brown silty clay that transitions to grey and soft clay. Both sections are underlain by gneissic bedrock. The surrounding area is poorly drained - there is free-standing water within approximately 20m of the embankment toe when snow is not present.

2.2 Instrumentation

Clusters of instruments were installed at the shoulder, mid-slope and toe of both the stable and unstable sections (Figure 3). They include 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 were used to identify possible upwards or downwards hydraulic gradients.

Temperatures have been collected during two winter cycles. Readings were taken monthly on a data acquisition system and downloaded manually. Telephone access was not available at reasonable cost at the relatively remote site.

3 DATA FROM FIELD INSTRUMENTATION

Following sections analyse the results of the field observations. The data include surveyed results from the surface settlement plates, displacements measured by inclinometers and horizontal extensometers, pore water pressures, and monthly temperatures from the thermistor strings. Inserts in following figures show the positions of the instruments for which results are being reported.

3.1 Movements measured by surface settlement plates

Figures 4 and 5 show displacements of surface settlement plates in metres in both the stable and unstable sections at the shoulder of the road, at mid-slope, and at the toe of the embankments. Vertical movements are shown as elevation changes in Figure 4, and horizontal (lateral) movements perpendicular to the centreline of the road in Figure 5.

Figures 4a, 4b, and 4c show the vertical movements at the shoulder, mid-slope and toe of the two sections. Both show seasonal movements that are largely, but not completely recoverable. Because of the gravel in the embankment, the seasonal movements at the shoulder are small. There appears to be little cumulative movement at the shoulder of the stable section. In contrast, there has been about 0.15m of settlement at the shoulder of the unstable section. Seasonal heaves are larger at mid-slope, and larger again at the toe. Again the stable section shows little cumulative movement, while there is about 0.08m of cumulative settlement at mid-slope and 0.04m at the toe.

The lateral movements at the ground surface (again from the settlement plates) in Figures 5a, 5b, and 5c show somewhat similar trends, though seasonal effects are perhaps only apparent at the toes of the embankments. Again, the stable section did not move much in the two

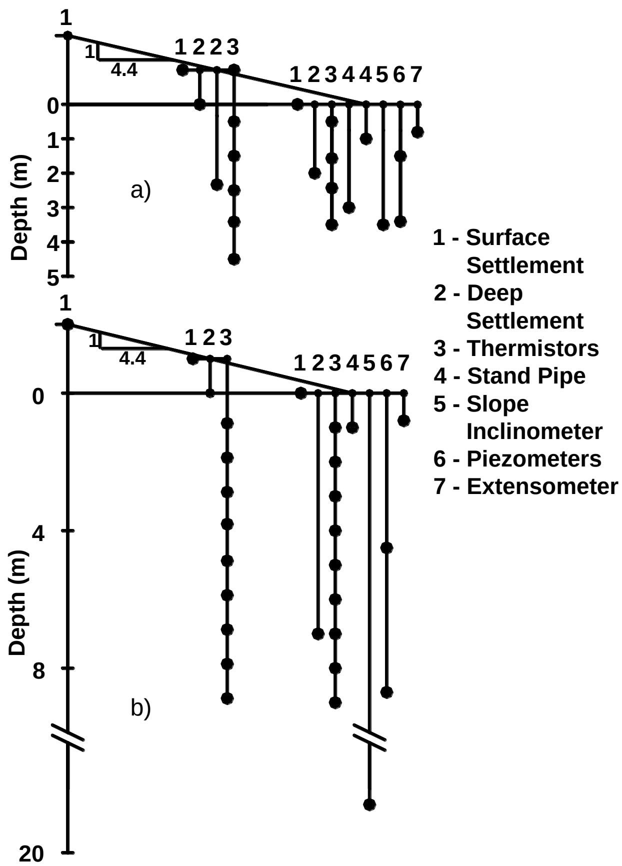

Figure 3. Instrumentation at a) stable section; b) unstable section

years of observation – the scatter in the data is probably about equal to the precision of the observations. All three plates at the unstable section show cumulative movements, about 0.07m at the shoulder and mid-slope, and 0.03m at the toe.

'Frost heave' of the road embankment is associated with the formation of ice lenses in seasonally frozen ground. Heave results when the water freezes, attracts additional water by capillary action, and forms ice lenses. Lenses form in all soil types by the addition of water during stationary or slow movement of the freezing front into the soil but are most common in silts and fine sand. The supply of water and ease of water movement (hydraulic conductivity) determines the extent and thickness of ice lenses. Frost heave occurs at the frost line, with T ≤ 0°C (Andersland and Ladanyi, 2004).

The seasonal heaves seen in Figure 4 for the unstable section are believed to be due to frost heave. The toe of the embankment shows the highest amount of frost-heave during the freezing season while the shoulder shows almost no frost heave, probably due to the gravel fill on the shoulder. Gravel can be considered to be a non-frost susceptible material (Koyama and Sasaki 1967). It drains freely by gravity and does not lead to upward movement of capillary moisture.

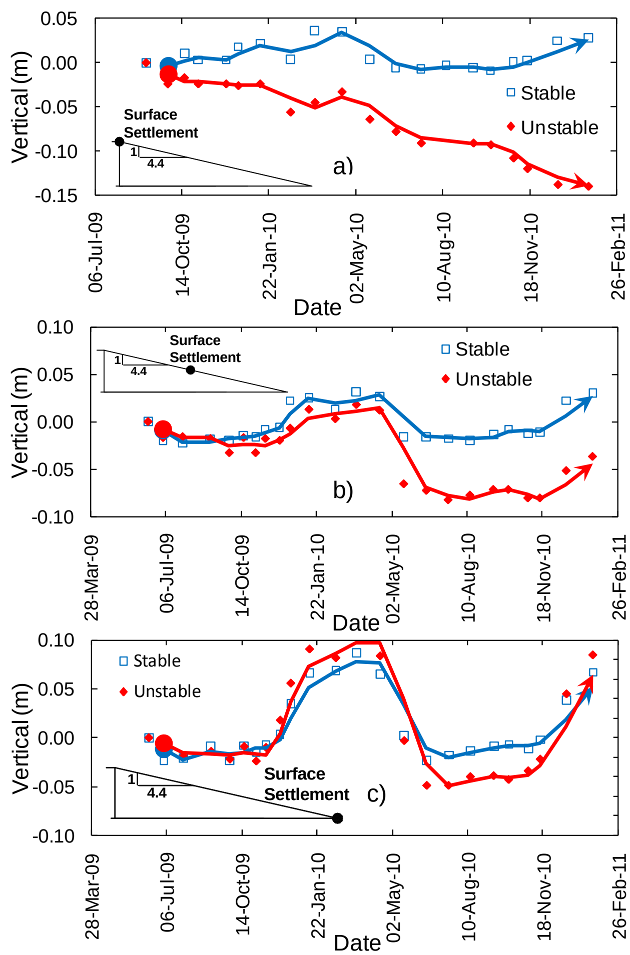

Figure 4. Vertical movements (settlements) at shoulder; mid-slope; and toe of the embankment

3.2 Lateral Displacements from Shallow Extensometers

Vibrating wire extensometers with a gauge-length of 1.0m were installed at the toes of both test sections at a depth of 0.8m. The instruments were intended to monitor lateral deformations at shallow depths and hopefully to relate movements at the toe to lateral spreading and longitudinal cracking of the road surface. Unfortunately, the extensometer at the stable section failed shortly after installation. Figure 6 shows horizontal strains at the toe of the unstable section plotted against time over a 28 month period in 2008-2011.

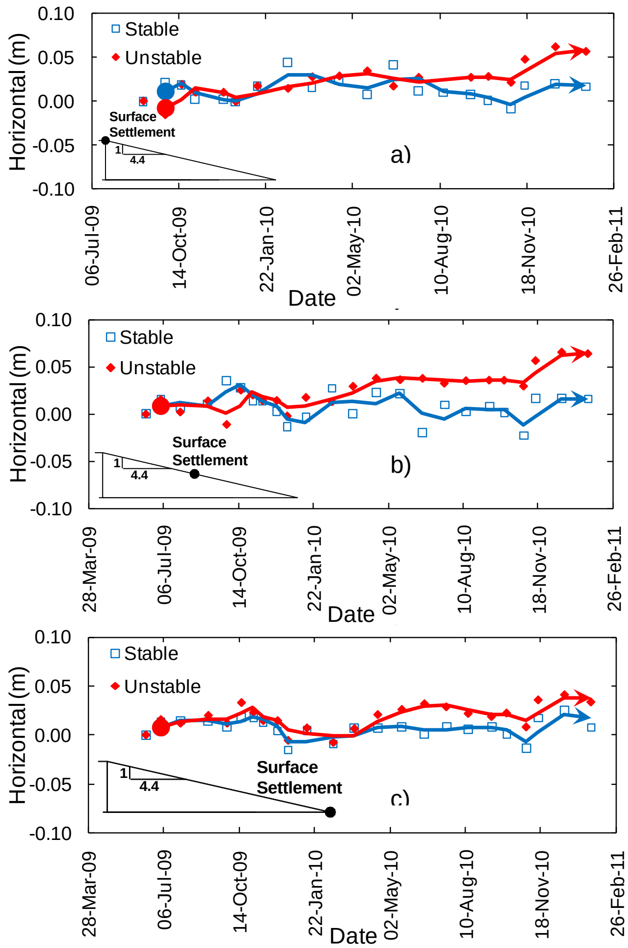

Figure 5. Horizontal movements in down-slope direction at shoulder; mid-slope; and toe of the road

The results in Figure 6 show ongoing outward displacements, currently with about 0.75% horizontal strains at the toe of the unstable section over a period of 28 months. The figure indicates fairly rapid deformation rates during the first freezing season (November 2008 to June 2009). This may be due to natural re-densification of soil around the extensometer following installation. After the first year, from July 2009 to present, the rate of horizontal straining decreased.

3.3 Lateral Displacements from Slope Inclinometers at Depth

Slope inclinometers were installed beneath the toes of the embankments at both sections. The lateral movements beneath the toe of the stable section are small and will not be discussed here. Figure 7 shows cumulative displacements measured by the slope inclinometer at the toe of the unstable section. The figure shows that the inclinometer experienced lateral displacements away from the centre of the embankment at depths greater than about 3m. Maximum displacement of about 0.09m occurs at a depth of 8m. At shallower depths, the slope inclinometer moved back towards the centre of the

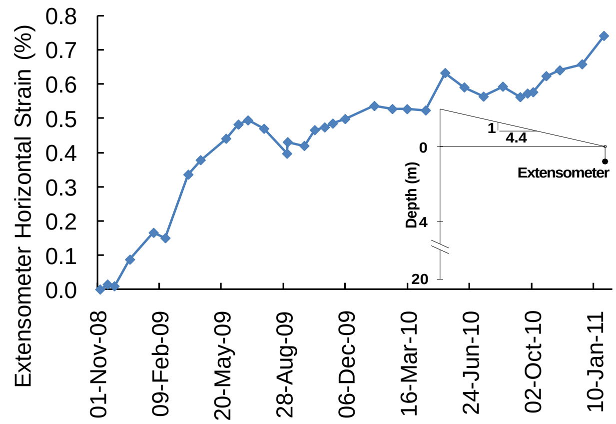

Figure 6. Extensometer horizontal strain vs. time at 0.8m depth at the toe of the unstable section

embankment. This is believed to be due to creation of ice at the frost front during the freezing season and rearwards bending of the slope indicator casing.

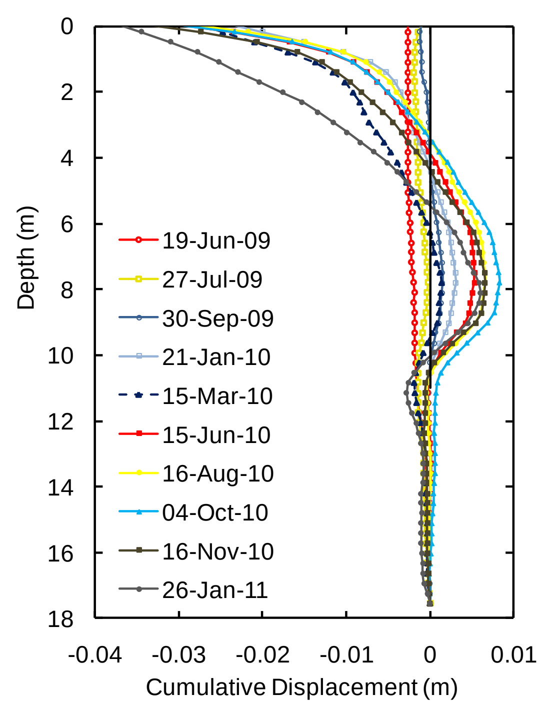

Figure 7. Slope Inclinometer at the toe of the unstable section

In general, the combined surface and deep measurements of displacements show that the cumulative movements of the embankment are largely downward at the shoulder and smaller at the toe. The resulting shear strains push the lower levels of clay horizontally away from the centre of the embankment at greater depths.

3.4 Ground Water Data

Figures 8a and 8b show total heads plotted against time (indicated here as dates) at two different depths in the stable and unstable sections respectively. (Remember that the silty clay is much thicker beneath the unstable section [Figure 2].) The figures imply seasonal changes in the gradients of total head at both sections. Ground water conditions appear to be broadly hydrostatic during summer months, with the ground water level approximately at the ground surface. During freezing, upwards gradients develop. These are believed to be caused by upward flow of water towards suction pressures (negative potentials) at the freezing front as it moves downwards during the winter.

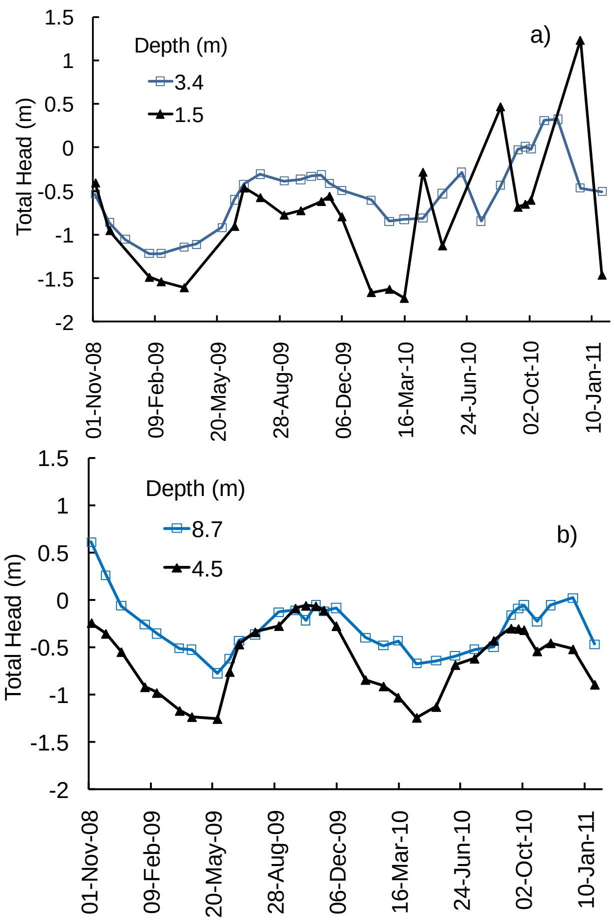

Figure 8. Total head vs. time at different depths at the toe of the a) stable section; b) unstable section

The effects of frost action involve a combination of frost heave during a downward advance of the freezing front, with the accompanying formation of ice lenses; and a subsequent reduction of shear strengths and higher compressibilities when the ice melts during the spring thaw (Andersland and Ladanyi, 2004).

3.5 Ground Temperatures

Figures 9 and 10 show minimum, average, and maximum temperature profiles with depth for the unstable and stable sections respectively, for years 2008-09 and 2009-10. Please note the different scales used for depths in these figures. They arise from the different thicknesses of the foundation soils under the two sections, (Figure 2). Again, the original ground level is at depth ‘0’.

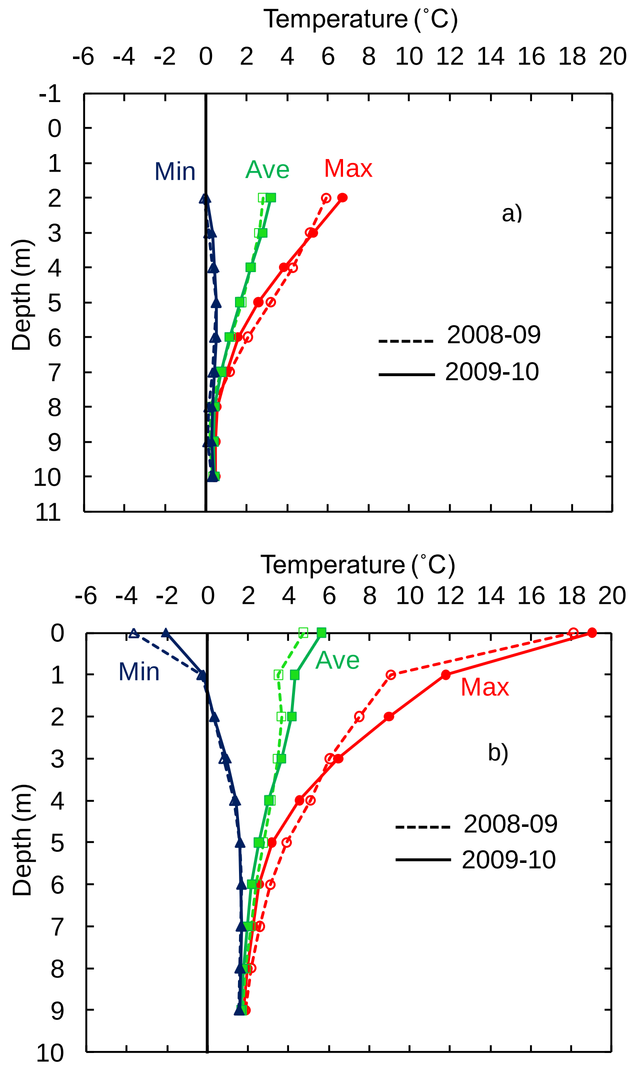

Figure 9. Annual temperature envelopes between November 2008 and October 2010 for the unstable section at the a) mid-slope; b) toe

The figures show temperatures measured by the thermistor clusters at the toes and mid-slopes of both sections. ‘Geothermal gradient’ is the rate of change of temperature with depth in the ground. When average temperatures are examined in this way, the slope of the average temperature profile can be taken as an approximate indicator of net heat flow into, or out from, the ground surface over a period of one year.

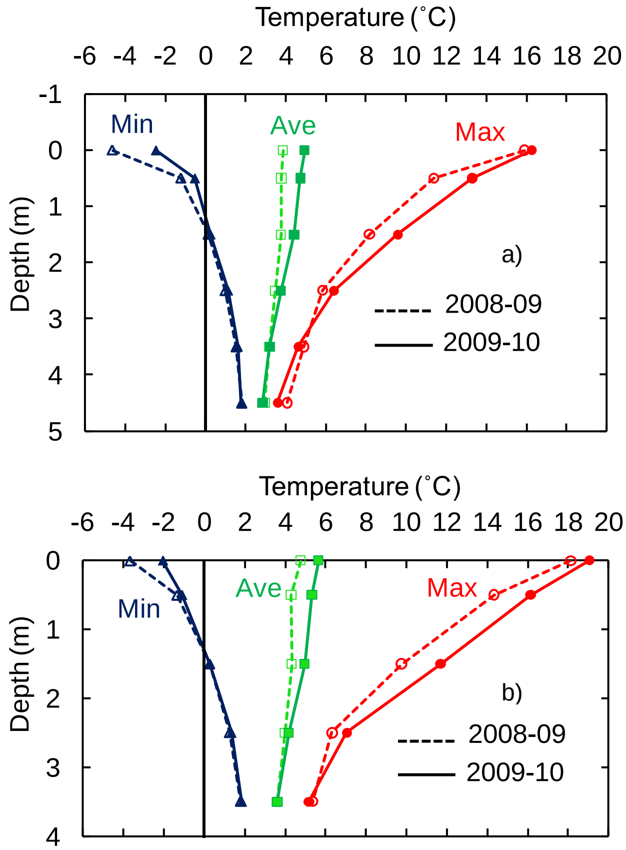

Figure 10. Annual temperature envelopes between November 2008 and October 2010 for the stable section at the a) mid-slope; b) toe

Figures 9 and 10 show average temperature profiles over two years at the four instrumented locations, that is at the toes and mid-slopes of the two test sections. No sub-zero temperatures were observed during 2008-09 and 2009-10. The average temperatures at all four locations decrease with increasing depth. This indicates a net heat flux into the foundation soils. It also supports the observations in an earlier section that initially frozen soil has thawed during the lifetime of the soil - there has been general warming of the embankments and their foundations.

Figure 11 shows the monthly temperature profiles for one cycle of cooling and heating between November 2009 and October 2010. The two graphs in Figure 11 contain data from instrument clusters at a) mid-slope and b) toe for the unstable section. (For reasons of space and clarity, we are not including observations from 2008-2009 and 2010-2011.)

With increasing depths, seasonal differences in temperature become less. 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 11a and 11b show that the depth of zero annual amplitude at the unstable section is about 8 – 9m below the original ground

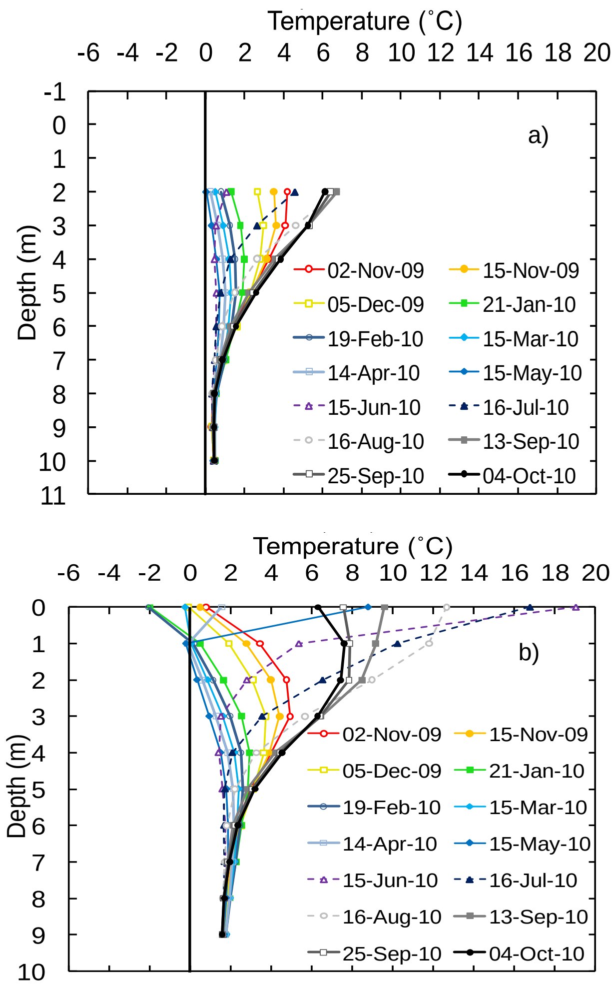

Figure 11. Monthly temperature profiles between November 2009 and October 2010 for the unstable section at the a) mid-slope; b) toe

level.

The respective temperatures profiles for the mid-slope and toe converge at about the same depth, although there is approximately a 1.6°C difference between the two values. The soil underneath the embankment is colder than soil at the same depth underneath the toe. It is believed that this can be explained by the previous presence and thawing of a frost bulb underneath the embankment or alternatively by differences in thermal conductivity between gravel in the embankment and clay in the foundation soils. This observation could have been verified if thermistor strings had been installed under the centre of the embankment. Perhaps unfortunately, safety, snow clearing, and grading made this impossible. Cooler temperatures beneath the embankment may also be due to an insulating cover of snow beneath the toe, combined with the effects of show clearing on the roadway.

4 SUMMARY

The paper reports and analyzes measurements of surface settlements, toe displacements, pore water pressures and temperatures from a low highway embankment near Thompson in northern Manitoba.

The deformation results indicate a combination of seasonal and cumulative displacements, with larger displacements at the toe than at the shoulder. The results show a combination of seasonal heaving and lateral spreading. The pore water pressure results suggest the development of cyclic seasonal gradients of total head. These indicate hydrostatic water pressure conditions during summer months and upwards flow during the freezing season. Measured temperatures also showed seasonal changes. Net heat flow into the ground is possibly a confirmation of general warming in the area, although it we note that only two years of readings have been collected. The data confirm field observations of the thawing of ice that was present in the first drilling of the site in 1991 but absent in recent drilling in 2008.

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