A STUDY OF ADAPTATION STRATEGIES FOR ROAD EMBANKMENTS BUILT ON PERMAFROST AFFECTED BY CLIMATE CHANGE

BY

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A Thesis
Submitted to the Faculty of Graduate Studies
in Partial Fulfilment of the Requirements for the Degree of

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THE UNIVERSITY OF MANITOBA

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ABSTRACT

Road embankments in Northern Canada experience lateral spreading and settlements that result in longitudinal cracking of the road surface. Highways and road embankments can degrade permafrost because their construction and use increase the thermal regime beneath the embankments. In addition, the climate warming trend could potentially magnify this problem. When thawed, the discontinuous permafrost beneath the roads results in differential settlements lateral spreading and cracks, which contribute to a concern of public safety.

A study was carried out to asses the impact of climate change on road embankments over degrading permafrost. The model was calibrated using field measurements of ground temperatures for a three-year monitoring period. With a realistic prediction of ground thermal regime, the model was used as the basis for further assessing the impacts of climate change to the ground thermal regime in the permafrost beneath road embankments for a fifty-year period. Based on the simulated impact of climate change to the permafrost, an idealized stress-deformation scenario was determined to evaluate the deformation and stability of embankments with and without mitigation measures. The Thesis will discuss the results of the evaluation of adaptation strategies using selected soil/ground improvement techniques to mitigate the impacts of climate warming to road embankments on permafrost.

ACKNOWLEDGEMENTS

I would like to deeply thank the various people who, during the several months that this endeavor lasted, provided me with advise, friendship, and financial and sentimental support. Without their care and consideration, the outcome of this project would likely not have maturated.

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Most important, I would like to express my sincere thanks and love to family and friends for their support, encouragement and understanding during the long months of this marvelous experience. In particular to Sara and my family back home that always will be inspiration to overcome difficulties.

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GLOSSARY OF TERMS

This glossary of terms has been prepared to help with the understanding of the terminology used along the thesis. Basic definitions are provided about the thermal, hydraulic and mechanical properties of soils. Most of the definitions were textually taken from the following references: National Research Council of Canada, 1988; Budhu, 2000; Johnston, 1981; Dingman, 2002; and Fetter, 2001.

Active Layer: The top layer of ground subject to annual thawing and freezing in areas underlain by permafrost.

Atterberg Limits: Denomination given to the liquid and plastic limits named after the Swedish soil scientist A. Casagrande.

Coefficient of Consolidation: It is a measure of the rate of change of volume during primary consolidation.

Compression Index: The slope of the normal compression line (NCL) and critical state line (CSL).

Conduction: It is the flow of heat by the passage of energy from one soil particle to another or through soil pore fluids.

Consolidation: It is the time-dependent settlement of soils resulting from the expulsion of water from the soil pores.

Continuous Permafrost: Permafrost occurring everywhere beneath the exposed land surface throughout a geographic region.

Convection: It is the transmission of energy (heat /sound) from one place to another by movement of a fluid such as air or water.

Degree of Saturation: It is the ratio, often expressed as a percentage, of the volume of water to the volume of voids.

Density of Frozen Soil: The mass of a unit volume of frozen soil or rock.

Depth of Thaw: The minimum distance between the ground surface and frozen ground at any time during the thawing season.

Discontinuous Permafrost: Permafrost occurring in some areas beneath the exposed land surface throughout a geographic region where other areas are free of permafrost.

Dry Density: It is the mass of soil grains (ignoring water) contained in a unit volume of soil.

Elasticity Modulus: It is the slope of the stress-strain line for linear isotropic material.

Elevation head: It is the height of a point above a given datum fro seepage problems.

Effective Stresses: They are the stresses carried by the soil particles.

Fine Grained Soil: Soils containing more than 35% of particles smaller than 0.06 mm in size.

Freezing Front: The advancing boundary between frozen (or partially frozen) ground and unfrozen ground.

Frost Heave: It is the upward or outward movement of the ground surface (or objects on, or in, the ground) caused by the formation of ice in the soil.

Frost Penetration: It is the movement of the freezing front into the ground during freezing.

Frozen Ground: Soil or rock in which part or all of the pore water consists of ice.

Green House Gases: Gases in the atmosphere that trap the sun's energy and thereby contribute to rising surface temperatures.

Ground Thermal Regime: It is a general term encompassing the temperature distribution and heat flows in the ground and their time-dependence.

Heat Capacity: It is the amount of heat required to raise the temperature of a unit mass of a substance by one degree

Hydraulic Gradient: It is the difference in total head or hydraulic head between tow points divided by the length of the flow path.

Hydraulic Head: It is the total mechanical energy per unit weight of water. It is also equal to the sum of the elevation head and the pressure head.

Ice Content: It is the amount of ice contained in frozen or partially frozen soil or rock.

Internal Friction Angle (ø): It is a measure of the shear strength of soils.

Latent Heat: It is the amount of heat required to melt the ice (or freeze the water) in a unit volume of soil.

Liquid Limit: It is the water content at which a soil changes from a plastic state to a liquid state.

Long Wave Radiation: It is the energy in the infrared range between wavelengths of 4 and 50 μm.

Modulus of Volumetric Compressibility: It is the slope of the curve between two stress points in a plot of vertical effective stress versus vertical strain.

Peat: It is a soil with high organic content, high moisture content and holding capacity resulting in high buoyancy and high pore volume leading to a low bulk density and low bearing capacity.

Permafrost: Ground (soil or rock) that remains at or below 0°C for at least two years.

Permafrost Base: The lower boundary surface of permafrost, above which temperatures are perennially below 0°C and below which temperatures are perennially above 0°C.

Permafrost Boundary: The geographical boundaries between zones of continuous and discontinuous permafrost.

Permafrost Degradation: A naturally or artificially caused decrease in the thickness and/or areal extent of permafrost.

Permafrost Table: The upper boundary of permafrost.

Permafrost Thickness: The vertical distance between the permafrost table and the permafrost base.

Permeability (Hydraulic Conductivity): It is the ability of a rock or soil to transmit water that, together with its ability to hold water, constitute the most significant hydrologic properties.

Plastic Limit: It is the water content at which a soil changes from a semisolid state to a plastic state.

Poisson's Ratio: it is the ratio of the radial (or lateral) strain to the vertical strain.

Pore Water Pressures: It is the pressure of the water held in the soil pores.

Porosity: It is the ratio of the volume of void to the total volume of soil.

Pre-consolidation Pressure: It is the maximum vertical effective stress that a soil was subjected in the past.

Pressure head: It is the height of a column of water required to develop a given pressure at a given point.

Recompression Index: It is the average slope of the unloading/reloading curves in a plot of void ratio versus the natural logarithm of mean effective stress.

Resistivity: It is a measure of how well a soil passes electric current. The higher the resistivity of a given soil, the less electric current passes through.

Sensible Heat: It is the heat energy that can be directly sensed via measurement of the temperature.

Shear Strength: It is the maximum shear stress a soil can sustain under a given set of conditions.

Shear Stress: It is the force per unit area acting tangentially to a given plane or surface.

Solar Radiation: It is virtually all the sun's energy arriving at the surface at wavelengths less than 4 μm.

Specific Gravity: It is the ratio of the mass of a body or a substance to the mass of an equal volume of water.

Specific Surface Area: It is the total surface area of all particles in a unit mass of soil.

Soil Stresses: It is the intensity of force per unit area; normal stress is applied perpendicularly to a surface or plane; shear stress is applied tangentially to a surface or plane.

Strain: It is the ratio of the change in a dimension to the original dimension or the ratio of change in length to the original length.

Coupled-Consolidation Analysis: It is the process of solving both the flow and equilibrium conditions simultaneously.

Stress Path: It is a graphical representation of the locus of stresses on a body.

Talik: A layer or body of unfrozen ground in a permafrost area.

Thaw: Melting of the ice in frozen ground, usually as a result of a rise in temperature.

Thaw Consolidation: Time-dependent compression resulting form thawing of frozen ground and subsequent drainage of pore water.

Thaw Consolidation Ratio: It is a dimensionless ratio of the rate of thaw to the rate of consolidation of the thawing soil, which is considered to be a measure of the relative rates of generation and expulsion of excess pore fluids during thaw.

Thawing Front: The advancing boundary between thawed ground and frozen ground.

Thaw Settlement: Compression of the ground due to thaw consolidation.

Thermal Conductivity: It is the measure of the quantity of heat that will flow through a unit area of substance in unit time under a unit time gradient

Thermal Cracking: It is a tensile fracture resulting from thermal stresses in frozen ground.

Thermistor: It is a type of resistor used to measure temperature changes.

Thermokarst: It is the process by which characteristics landforms result from the thawing of ice-rich permafrost.

Thermosyphon: A passive heat transfer device installed to remove heat from the ground.

Total Stress: It is the stress carried by the soil particles and the liquids and gases in the voids.

Total Water Content: The total amount of water (unfrozen water and ice) contained in soil or rock.

Unfrozen Ground: Soil or rock that does not contain any ice.

Unfrozen Water Content: The amount of unfrozen (liquid) water contained in frozen soil or rock.

Void Ratio: It is the ratio of the volume of the voids to the volume occupied by the soil grains

Volumetric Heat Capacity: It is the amount of heat required to raise the temperature of a unit volume of a substance by one degree.

Volumetric Water Content: It is the ratio of the volume of water and ice in a sample to the volume of the whole sample, expressed as a fraction.

CHAPTER 1

INTRODUCTION

1.1 THE RESEARCH

The research consists of numerical analysis to evaluate various ground improvement techniques like lightweight fill materials, rockfill columns, sheet-piles, and geosynthetics as adaptation strategies for road embankments on permafrost affected by climate change. The ground thermal regime beneath road embankments was first simulated using numerical modelling to delineate frozen and unfrozen ground. The numerical model was calibrated with data from a three-year monitoring program using thermistor records. Once calibrated it was used to determine the thermal behavior of foundation soils and their thermal response to simulated global warming trend. A coupled-consolidation analysis was then performed to determine stresses and deformations when thawing of permafrost is in progress incorporating various ground improvement techniques as adaptation strategies for road embankments tolerant to climate change. These have been evaluated through their stability and serviceability performance.

1.2 RESEARCH NEEDS AND BACKGROUND INFORMATION

Most of the northern regions of Canada are underlain by either continuous or discontinuous permafrost. The principal characteristic of these layers of soil

the high content of ice, which makes them susceptible to consolidation during thawing. If consolidation occurs beneath embankments it can lead to differential settlements, which result in instability and failure.

Embankments have a large influence on the thermal regime of the ground. They modify the energy balance of the ground surface. Disturbance of the surface due to the construction of embankments increases the mean annual surface temperature (MAST) resulting in warmer conditions that enhance permafrost thawing. The effects of any postulated global warming trend are likely to magnify the problem if the effect of increasing the air temperature leads to an increase in the ground temperature within 1 to 3°C. This is the range of temperatures where the discontinuous permafrost is susceptible (Esch and Osterkamp, 1990; Ladanyi, 1996).

Thawing is a major problem to consider when dealing with the design of road embankments in northern regions that are dominated with fine-grained soil deposits. When thawing occurs, it results in the generation of excess pore water pressures, differential settlements, lateral spreading of the slopes, decrease in bearing capacity and many deformities in the pavement that can pose risks and danger to motorists.

Significant settlements in embankments and loss of shoulder support have been reported north of the Sasagiu Rapids Bridge located approximately 65 Km south of Thompson, Manitoba, in areas of discontinuous permafrost. This

situation has become a public concern due to public safety and cost of maintenance of this highway over the last 40 years. The historical summary and sequence of events have been supplied by Manitoba Transportation and Government Services (MTGS) and are described in the following paragraphs.

Between 1964 and 1965 the highway from Ponton to Thompson was originally constructed as a mud (clay) surfaced Provincial Road. In the late 1960's the road was upgraded to granular and by the early 80's was converted to a bituminous pavement traffic surface. Between 1978 and 1980 permafrost was sporadically identified ranging in thickness from 2.4 to 6.0 m.

Reports by Mollard (1979) and Underwood (1979) suggest that permafrost was likely to be found in areas with significant peat cover overlying deep clay deposits. Permafrost is identified to be in a very advanced degradation phase particularly below slope embankments and ditches, which is manifested as shoulder settlements and softness. Ground temperatures were collected in 1980 to evaluate the performance of the permafrost. Results from that study indicated that temperatures were constant over most part of the year and at depths greater than 2.0 m where peat berms of more than 3.0 m thick were used.

AMEC (2000) was commissioned by MTGS to perform a geotechnical investigation and assessment of the most suitable remedial measures to be used in mitigating potential damages to road embankments due to permafrost degradation. Slope stability analysis and modelling of vertical drains to accelerate

consolidation were performed. The report concludes that vertical drains are the preferred technical solution to increase the rate of settlement to improve highway performance over the long-term. However, this technique seems not to be suitable for existing embankments because settlements are achieved in a shorter period and transportation might be compromised. Other recommendations include: (1) reducing the highway elevation or flattening embankments shoulders and improving the drainage in the ditch to increase bearing capacity of the foundation; (2) evaluate techniques to accelerate permafrost degradation by removing the compressible peat providing drainage and including techniques to improve the stability of foundation soils.

The research is important to determine and better understand the mechanisms controlling the failure of embankments constructed over degrading permafrost. Once this is defined and understood, it is possible to select suitable remedial measures allowing construction and rehabilitation of new and existing road embankments in the Northern Regions that can be tolerant to potential damage caused by climatic warming.

1.3 OBJECTIVES AND SCOPE

Specific objectives of this research are:

1) To estimate the average thermal properties of the foundation soils for the thermal numerical analysis from the information provided by Manitoba

Transportation and Government Services, AMEC's report, and complemented with those found in the literature.

2) To calibrate the numerical model by using thermistor data provided by Manitoba Transportation and Government Services during the period of 1996 to 1998.

3) To determine a proper climate model scenario based on climate models available at the study site and investigate how climatic warming can affect the ground thermal regime of the foundation beneath the road embankments for a period of 50 years from now to fully understand the mechanism of permafrost degradation.

4) To carry out stability and deformation analysis through numerical analysis after incorporating the simulated thermal regime of the ground produced by the climatic warming trend.

5) To propose adaptation strategies to mitigate the adverse effects of the permafrost degradation to improve the stability and deformation performance of road embankments on degrading permafrost.

The numerical analysis is done with the following assumptions regarding the material properties:

a) The thermal properties of the soils are determined from the information provided by Manitoba Transportation and Government Services as well as values found in the literature. This is done due to the lack of thermal properties from laboratory and field tests.

b) Mechanical properties are estimated from empirical relationships found in the literature because no data has been provided for the research.

c) Rigorous thermal-mechanical-hydraulic coupling is not available in the computer software used in this study. Therefore, thermal analysis is not coupled with the stress-deformation analysis.

1.4 RESEARCH APPROACH

The following research approach is undertaken to accomplish the proposed objectives:

A literature review investigating the performance of road embankments in the northern regions was performed to gain an understanding of the thermal behavior and consequences associated with building road embankments in zones of discontinuous permafrost.

Thermal modelling was carried out to determine the thermal behavior of foundation soils under a climate warming trend. The thermal properties of the soils are determined based on the data provided by Manitoba Transportation and

Government Services, AMEC's report, and available values found in the literature. Air temperature conditions are determined by reviewing the air stations available at the study site.

The model was calibrated by using thermistor data provided by Manitoba Transportation and Government Services. Once reasonable calibration has been established, a 50 year model was run to simulate permafrost degradation under a warming trend scenario. The warming trend scenario was determined by comparing available climate models and typical air temperatures at the study site. This allowed the modelling of the ground thermal regime beneath the road embankments.

A stress-deformation model was set up to account for thawing consolidation in the ground during permafrost degradation. This was possible by dividing the model in sequential periods that allow updating of the unfrozen and frozen areas.

Based on the results of the stress-deformation analysis, various ground improvement techniques, namely: columnar inclusions, lightweight fill materials, geosynthetics as basal reinforcements and sheet-piles were evaluated as potential adaptation strategies to improve the stability and reduce deformations in the ground and embankment.

1.5 THESIS ORGANIZATION

The thesis is organized as follows:

Chapter 2 comprises the results of the literature review investigating the following topics: performance of road embankments in the northern regions; climate change; consolidation of thawing soils; and adaptation strategies.

Chapter 3 thoroughly describes the thermal modelling, which includes: the location of the problem; the thermal properties of the soils; the model cross section; the calibration of the model; and the model results for a 50 year simulation period based on a climate model scenario.

Chapter 4 corresponds to the stress-deformation analysis. It contains the model set up, the mechanical properties used in the model, the model simulation procedures, and the modelling results.

The investigation of the various ground improvement techniques as adaptation strategies for climate change damage tolerant road embankments is described in Chapter 5.

Chapter 6 presents conclusions and discussions of the research.

CHAPTER 2

LITERATURE REVIEW

2.1 INTRODUCTION

This chapter presents the literature review on the performance of road embankment on discontinuous permafrost, the influence of climate change and the potential adaptation strategies to mitigate and prevent failures of foundation soils in Northern Regions where the presence of discontinuous permafrost is appreciable.

2.2 DESCRIPTION OF LITERATURE SEARCH

A total of twenty nine documents were reviewed as they are closely related to this study. The selected documents are listed in the Bibliography. Documents support, those that were reviewed but not selected are also listed in the Bibliography and as citations along the document. The literature search included: (1) engineering periodicals and journals; (2) ready-available research papers and texts; (3) conference proceedings; and (4) documents on the World Wide Web. The search included agencies and library catalogues shown below.

Special Library Catalogues

- University of Manitoba Bison Catalogue

Research Centers

- Canadian Centre for Climate Modelling and Analysis
- Intergovernmental Panel in Climate Change
- Geological Survey of Canada
- Centre for Transportation Research and Education Iowa State

Professional Associations

- Canadian Geotechnical Society
- American Society of Civil Engineers

Scientific and Engineering Journals

- Canadian Geotechnical Journal
- Technical Council on Cold Regions Engineering
- Journal of Cold Regions Engineering
- Cold Regions Science and Technology
- Géotechnique
- Journal of Geotechnical and Geoenvironmental Engineering
- International Journal of Geomechanics

Government Agencies

- Manitoba Transportation and Government Services
- Natural Resources Canada
- Environment Canada
- U.S. Artic Research Commission
- U.S. National Academy of Sciences
- Iowa Department of Transportation

The selected documents were classified according to the topics to be discussed in the thesis. A brief explanation of what is found in every paper is

provided. A chronological explanation of events is given to better describe the evolution of the problem and what has been done so far to improve the performance of road embankments.

2.3 CLIMATE CHANGE

About one-fifth of the land area of the earth is underlain by permafrost. Its characteristics are controlled by climatic, topographic, geographic, hydrologic and geological factors. The permafrost thickness is variable depending on the seasonal variation of the active layer, the insulating cover vegetation and snow, drainage, thermal properties of soil and rock and a complex upper boundary layer in the air. Therefore, changes in meteorological conditions, rain and snow precipitation, solar radiation, wind speed among others factors induce changes in the surface temperature of the permafrost that are difficult to predict (Osterkamp and Lachenbruch, 1990).

Nevertheless the effects of air warming are not easily accounted for; most of the discontinuous permafrost would be destabilized and eventually disappear with a small rise in temperature. This is because most of the discontinuous permafrost is within 1 – 3°C (Esch and Osterkamp, 1990; Ladanyi, 1996). In addition the problem may be expected to be even more severe as the temperature of the discontinuous permafrost approaches the melting point because of the presence of unfrozen water content in the soil particles. Figure 2.1 shows the permafrost distribution for Canada.

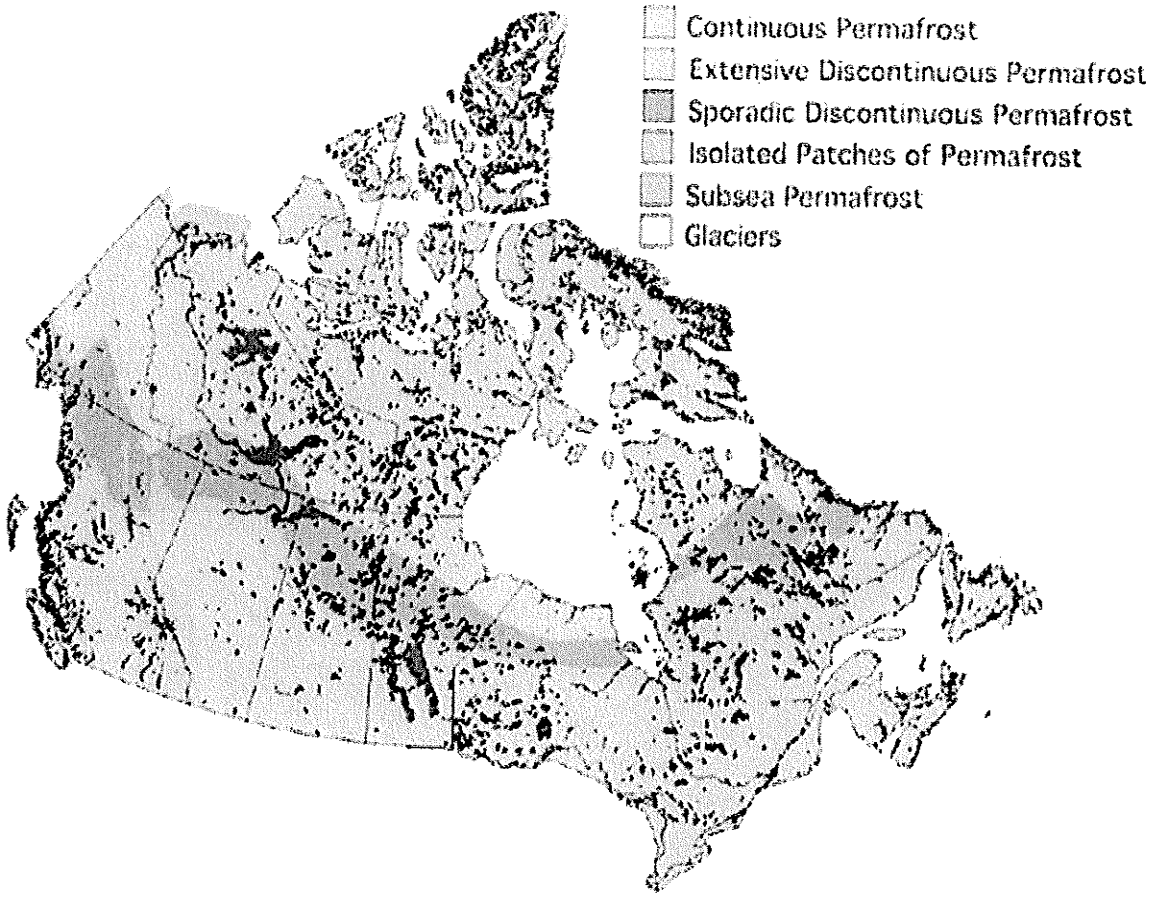

Figure 2.1: Permafrost distribution for Canada (after NRC, 2005b)

A global warming trend is in progress with more significant effects in the higher latitudes. For Canada, the data from Environment Canada indicates a trend of 1.9°C, 1.2°C, 0.8°C and 0.3°C for winter, spring, summer and fall respectively over the last 57 years). If the projected climate warming scenario shown in Figure 2.2 occurs, not only the discontinuous permafrost will be affected but also will modify the distribution even in the continuous permafrost. Thickening of the active layer may disturb the duration and magnitude of discharge and amplify river and seepage icing problems. Two solutions related to the problems

associated to the thawing of permafrost are either to preserve it by using heat removal methods or eliminate it by different pre-thawing techniques (Esch, 1985; Esch, 2004).

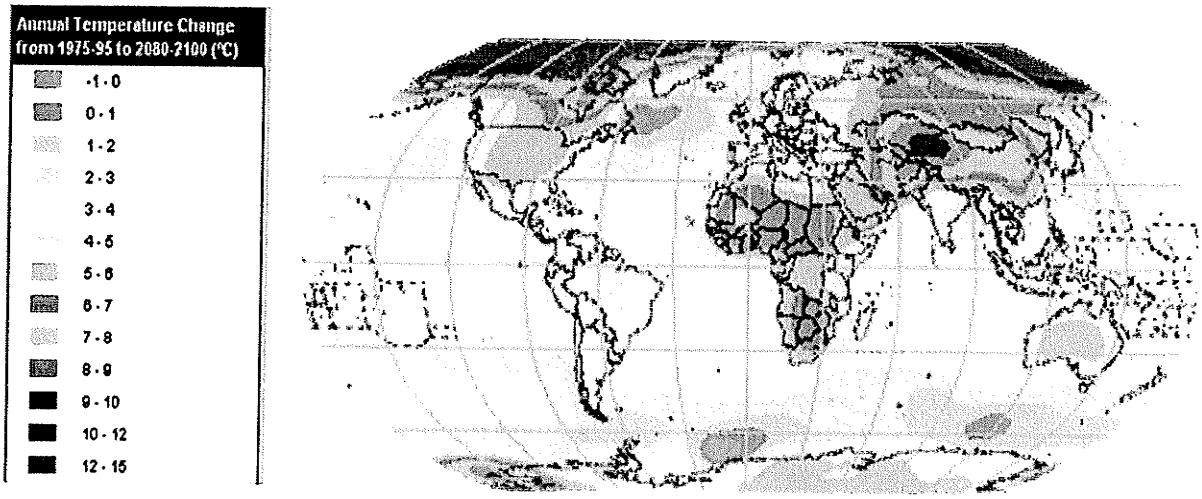

Figure 2.2: Projected global climate warming trend (after NRC, 2005c)

The rise in temperature is provoked by accumulation of carbon dioxide and other green house gases that would induce an increment in rainfall, and changes in snow cover. Precipitation could be a significant factor in accelerating permafrost degradation, while increased snow depths would affect the response of the ground to higher winter air temperatures (Smith 1990).

Changes in seasonal precipitation levels, mean cloud cover, and wind speed, would all have a significant impact on the heat flows between the air and the permafrost surface. However, permafrost temperatures and active layer

thicknesses under engineering structures may be expected to correlate more directly with air temperatures since the effects of precipitation and evaporation are minimized. If the climate warms significantly, thawing will begin with thickening of the active layer; this would generate consolidation and differential settlements of the structures (Esch and Osterkamp 1990).

The results of warming in the continuous and discontinuous permafrost are going to be different. The effects in the continuous permafrost would be to warm it up and possibly change the depth of the active layer. Thawing at the base of the permafrost would start several centuries later and would proceed at a rate of about a centimetre or more per year. However, the consequences in the discontinuous zone would be extremely serious, since most of this permafrost is within few degrees of thawing. This thawing process, even though may last many centuries to melt all the permafrost can begin immediately. Thawing would produce settlements that can range up to 5 metres or more in ice-rich soils (Osterkamp and Lachenbruch 1990).

Thawing of the ice rich permafrost may induce settlements of the ground surface, which often has adverse consequences for infrastructure and natural ecosystems. The potential for severe thawing disruptions to engineered works is imminent and problems are likely to intensify under conditions of global warming. Networks of primary and secondary roads, railways and airfields are going to be

affected in most of the Northern Regions (U.S. Artic Research Commission, 2003; Tucker et. al., 2004).

Permafrost plays three very important roles in the context of climatic change: as a record keeper by functioning as a temperature climate archive; as a translator of climatic change by translating to the humankind the consequences associated with the thawing of the permafrost, and as a climatic facilitator since large quantities of carbon are held in the upper layers of permafrost and an increase in the thawed layer could lead to the release of large quantities of gases to the atmosphere (U.S. Artic Research Commission, 2003).

The accumulation of carbon dioxide and other radioactively active (green house) gases (RAGS) in the atmosphere are and will continue inducing the progressive global warming trend. Consideration of the rates of rise of these gases, which include CO_2, NO_2, Chloro-fluoro Carbon (CFC's), Ozone, and Methane, generally lead to forecast an effective atmospheric RAGS doubling effect between the years 2030 and 2070 (Esch and Osterkamp, 1990). Global "green house" warming predictions indicate that the higher latitudes of the earth may warm by about 3 - 12°C by the middle of the next century as the result of the doubling content of these gases in the atmosphere (Weller, 2004).

2.4 PERFORMANCE OF ROAD EMBANKMENTS IN NORTHERN REGIONS

Road embankments have a large influence on the thermal conditions of the foundation soil. The construction of embankments modifies the pre-existing surface conditions and thermal regime of the ground, which may produce thawing of the permafrost. The modification of the ground thermal regime produces a Mean Annual Surface Temperature (MAST) that differs from the Mean Annual Air Temperature (MAAT). These differences in thermal conditions enhance thawing of the permafrost, which leads to consolidation and thaw settlement. To avoid thawing degradation the MAST must be maintained below 0°C (Goering, 1996; Goering and Kumar, 1996; Goering, 2004).

Modification of the MAST, consolidation and thaw settlements are not uniform leading to the most common embankments failure modes: thermal balance of soil surfaces; differential settlements; spreading and longitudinal cracks; creep movements; and thermal cracking as it is shown in Figure 2.3. In the case of road embankments these differential settlements start at the toe (Figure 2.4) since the thin soil cover at that point increases the transmission of heat from the surface.

Rooney and Vinson (1996) have identified three main failure mechanisms in paved roads. First, distortion and failure related to thaw weakening in the active layer and thaw degradation of permafrost. Second, cracking caused by both traffic loading and cold environment. Third, disintegration and wear due to

ravelling and moisture susceptibility of the pavement. Longitudinal asphalt cracking in Northern Regions might extend deep into the embankment. Crack movement rates of up to 25 mm per month and 100 mm per year have been recorded (Scher, 1996).

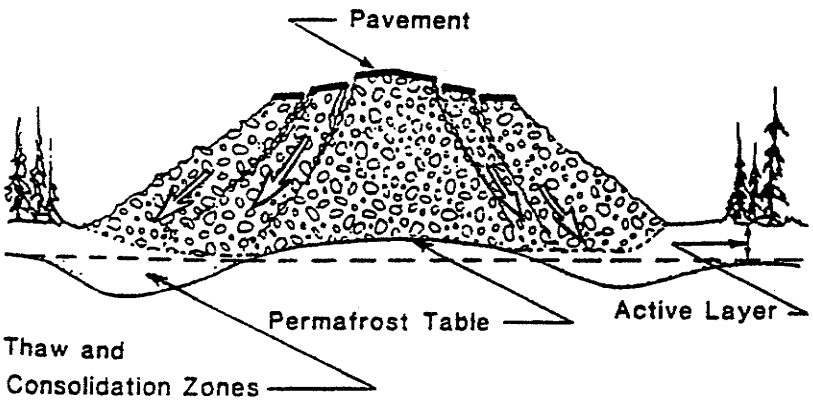

Figure 2.3: Process of permafrost degradation (after Esch, 1996)

Not only road embankments cause degradation of the permafrost but also building structures that progressively thaw the underlying permafrost. They also elevate the ground surface temperature and the active layer of the permafrost may thaw in summer. Differential movements may occur that can cause serious structural damage (Vangool, 1996).

Undesirable differential settlements and consequently structural damage may be reduced if the interaction between the structure and the soil is accurately assessed. The performance of buildings, bridges, embankments, and transmission towers may be improved if the mechanism of heat transfer in soils

and rocks are clearly understood. Heat transfer occurs by conduction through solid and solid-liquid interfaces, by convection through pore fluids, and by vapour transfer of heat. The thermal properties that control the mechanism of heat transfer are not constant; rather they vary with soil conditions (Steinmanis et. al., 1996).

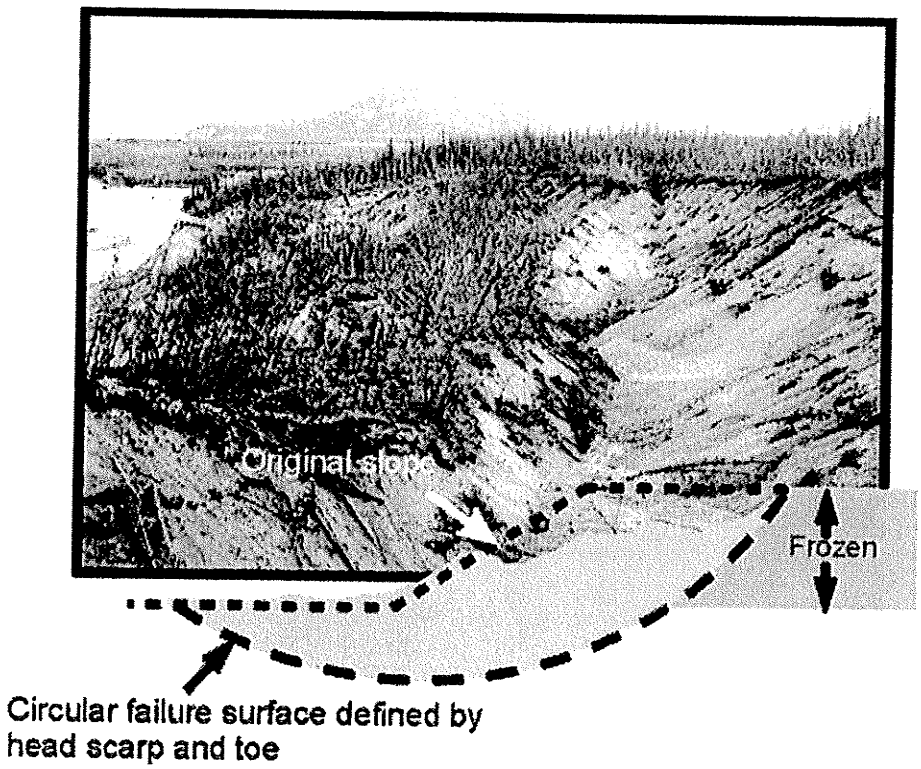

Figure 2.4: Failure processes of road embankments (after NRC, 2005a)

Thermal models become a powerful tool to asses the thermal change of the ground temperature due to a combination of factors like construction of geotechnical structures and climate warming trends. The results of thermal models might lead to address issues related to the limitation of the seasonal thaw

penetration, techniques of soil improvement, and provision of drainage to improve the bearing capacity of the soils (Instanes et. al., 1998). However, one of the most important things to know for the geotechnical engineer is the depth of thaw or depth of frost and their rate of change with time. The boundary between these two regions is the controlling factor in the consolidation of thawing soils that produce differential settlements and failure of geotechnical structures (Nixon and McRoberts, 1973; Nixon, 2004).

2.5 THAW CONSOLIDATION IN SOILS

Thaw settlement in soils that lead to failure of road embankments can be classified into two types: sudden thaw settlement and gradual thaw settlement. The sudden settlement occurs when roads are built in a region where the upper portion of the permafrost contains a thick clayey and ice-rich soil. When the permafrost melts the soil achieves a supersaturated state, loses its bearing capacity, and leads to large amounts of subsidence. Gradual thaw settlement follows after the sudden thaw settlement. This type of settlement is a function of density of the frozen soil, height of the fill materials, loading, and traffic volume (Ningyuan and Haas, 1996).

To be able to account for the consolidation in thawing soils, Morgenstern and Nixon (1971) developed an analytical solution. The solution in thawing soils follows the same theory of consolidation of fine grained soils. It differs in the fact that there exists a lower moving boundary delimited by the permafrost table.

Basically, the theory is a combination of the theories of heat conduction and the linear consolidation of a compressible soil. It has been used to determine the rate of consolidation of this type of soils.

The theory establishes that the linear consolidation is a function of a moving boundary. This moving boundary is the limit between the frozen and unfrozen zones, which is called the freeze-thaw interface. The theory states that only the unfrozen zone in considered in the analysis since permafrost does not transmit any deformations or pore water pressures. A detailed explanation of the theory is given in appendix B of this document.

2.6 ADAPTATION STRATEGIES

Many efforts have been taken to mitigate the effects of thawing of the permafrost produced by the construction of geotechnical structures. Different techniques to prevent the degradation of the permafrost were firstly used without much success. Ground improvement techniques can be implemented to prevent or mitigate the effects of permafrost thawing in geotechnical structure foundations, and thus can help adapt road embankments to unstable degrading permafrost caused by climatic warming. Some techniques are not intended for thawing soils; however, they can be adapted to comply with the purpose of improving foundation soil properties.

Esch (1996) summarized the chronological evolution of the design and construction techniques of embankments in the Northern Regions. These include the following techniques: 1) reflective surfaces and paint coatings (1963); 2) plastic foam insulations (1969); 3) peat underlays (1973); 4) embankment lateral berms (1974); 5) air cooling ducts (1974); 6) prethawing before construction (1980); 7) construction timing (1983); 8) surface coverings (1984); 9) inclined thermosyphons (1985); 10) geosynthetic reinforcement (1985); 11) lightweight fill materials (1987); 12) foundation bridge construction (1992); 13) air convection embankments (1995); 14) open road embankments (1996).

Most of the efforts have been concentrated on warmer permafrost sites. The recommendation for design engineers is to be aware of all prior experimental research work to know what kind of technique is applicable for every particular site. In addition, special attention must be paid to account for the effects of climate warming (Esch, 1996).

Maintenance also plays an important role in achieving colder ground surface temperatures. Two of those techniques are: 1) Removal of snow from side slopes throughout the winter season with heavy equipment; 2) the use of bulk rock placed on side slopes. The first technique will reduce the ground temperatures preserving the permafrost. However, leaving a blanket of snow during spring would delay the surface warming. The second technique acts as a

solar shade during the summer moths and promotes natural convective cooling during the winter months (Zarling and Rajesh, 1996)

Snow removal in combination with painting has been used to strengthen railroad and roadbed bases constructed on icy permafrost. Other maintenance techniques include: construction of sheds to protect against sun and precipitation; and cooling pipes to remove heat. These techniques are expected to preserve the permafrost and avoid degradation due to the construction of embankments (Kondratjev, 1996).

The use of bulk rock has been incorporated in many reconstruction projects in the Northern Regions. One example is the Shakwak project that included the rebuilding of 500 km of highways in North-western Yukon. This project involved the implementation of a number of techniques to achieve a stable road. Toe berms were found to be very effective to minimize embankment instabilities. Granular backslope blankets were used to stabilize ice rich cut slopes, and the use of insulated bedding helped stabilize drainage structures as culverts (Walsh et al., 1996).

Recently most of the efforts have been concentrated in incorporating classical ground improvement techniques for soft soils in degrading permafrost. Vertical drains, rock columns, geosynthetics, lightweight fill materials are some potential techniques to improve the bearing capacity of thawing soils.

Since thawing soils achieve a saturated state that leads to failure, vertical drains are a good option to drain water resulting from the melting of permafrost. They also facilitate thawing consolidation by increasing the rate of dissipation of excess pore water pressures resulting from external or embankment loads. This rapid consolidation leads to a much faster recovery of the strength of the soil. Vertical drains can be built with natural or synthetic materials that offer high hydraulic conductivity; the most popular of these are sands and geosynthetic wick drains. Figure 2.5 provides a good illustration of the consolidation process using vertical drains. Geosytnthetics have been tested in laboratory studies to enhance thawing consolidation and to reduce frost heave of soils (D'Andrea and Sage, 1989; Henry and Ellis, 1996).

The exact rate of consolidation cannot be totally understood if the conditions at the site are not accurately reproduced in computer simulations. One of the challenges is to be able to reproduce three dimensions in plane strain or axisymetric conditions. Hird et al. (1992) present a matching technique to model the behaviour of vertical drains. The matching technique can be achieved by adjusting the geometry (drain spacing) and/or the permeability of the soil. They validated the results by comparing the modelling results with theoretical results.

In addition to the comparison between the modelling and theoretical results, Hird et al. (1995) applied the matching technique for plane strain and axisymetric conditions to three case histories. In all the three cases a good match

was obtained between the average degrees of consolidation in plane strain and axisymetric conditions. This implies that the technique is a powerful tool to model vertical drains in two-dimensional analysis

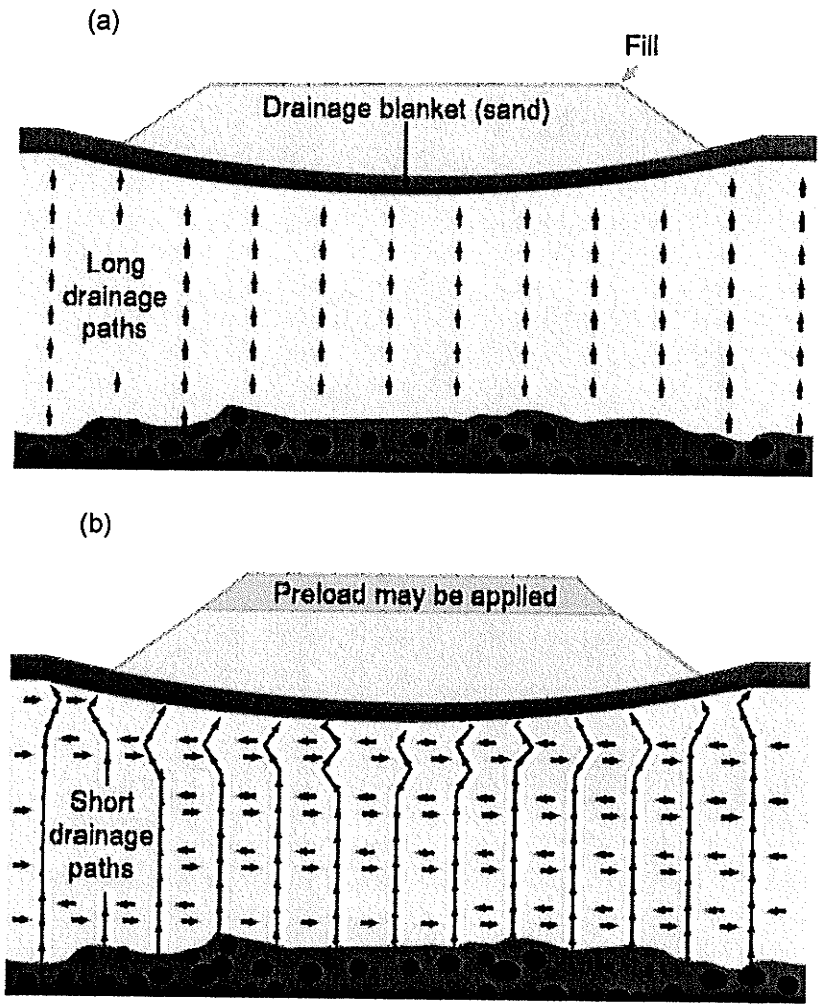

Figure 2.5: Effect of vertical drains in consolidation process: (a) without vertical drain; (b) with vertical drain (after American Drainage Systems, 2005)

Geosynthetics are being widely used to improve the bearing capacity and slope stability of soils as well as preventing differential settlements of foundations. They can be used alone (Figure 2.6) or together with other

techniques to provide a better performance of soft soils when used as foundation soils (Figures 2.7 and 2.8). One common use is to reinforce unpaved roadways over thermokarst zones that include voids. They act as bridges between voids to prevent differential settlements (Kinney and Connor, 1990). Geostynthetics can also reduce frost heave potential; increase the slopes of embankments and retaining walls; separate base from subgrade materials; drain excess water; and also act as elements controlling erosion (Kinney, 1996).

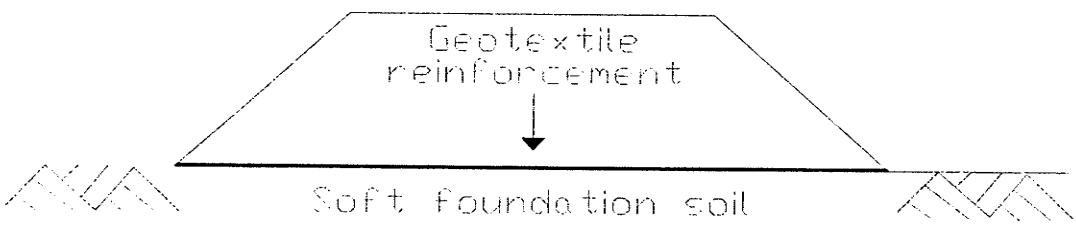

Figure 2.6: Geosynthetics used as reinforcements in road embankments

If geosynthetics are used in combination with other techniques they can further increase the stability of a foundation soil. Geosynthetics combined with prefabricated vertical drains can be a useful technique for the construction of new embankments in Northern Regions (Figure 2.7). Even though the technique is not specific for permafrost zones, it can be applicable since the shared effect of geosynthetics and PVD leads to a shear strength gain of the foundation soil. Gesosynthetics prevent differential settlements and lateral spreading of embankments, while PVD's accelerates the dissipation of excess pore water pressures that can be generated due to thawing permafrost. Construction rate

and the spacing of these vertical drains affect the degree of consolidation and their effect must be included in the design of this technique (Li and Rowe, 2001).

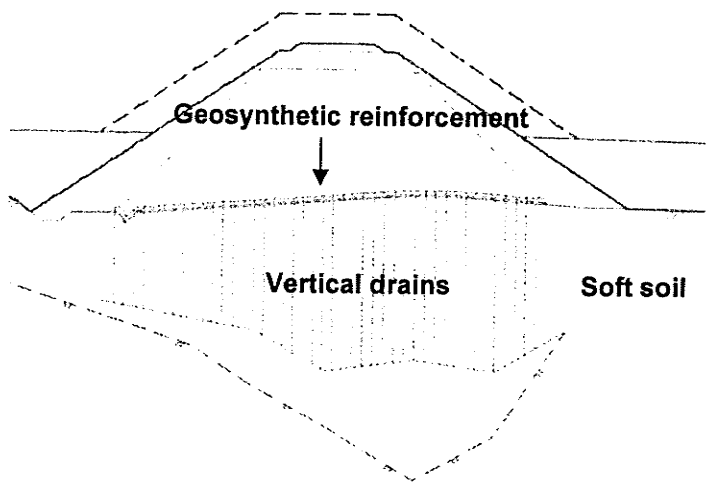

Figure 2.7: Combined effect of vertical drains and geosynthetics

Pile-supported earth platforms are also a good alternative to increase the bearing capacity and to reduce the settlement of soft soils. They can be used for different purposes: (1) to build superstructures in a single stage without prolonged waiting times; (2) to significantly reduce total settlements; (3) to reduce earth pressures; and (4) to avoid excavation and refill employed in typical situations. Piles can be of concrete, timber, grouting or stone columns (Han and Gabr, 2002).

Differential settlements can occur that may increase depending on the spacing between columns. To improve the performance, pile-supported earth

platforms can be used along with geosynthetics (Figure 2.8). The piles increase the bearing capacity of the soil, while geosynthetics can be used to minimize differential settlements between pile caps (Han and Gabr, 2002).

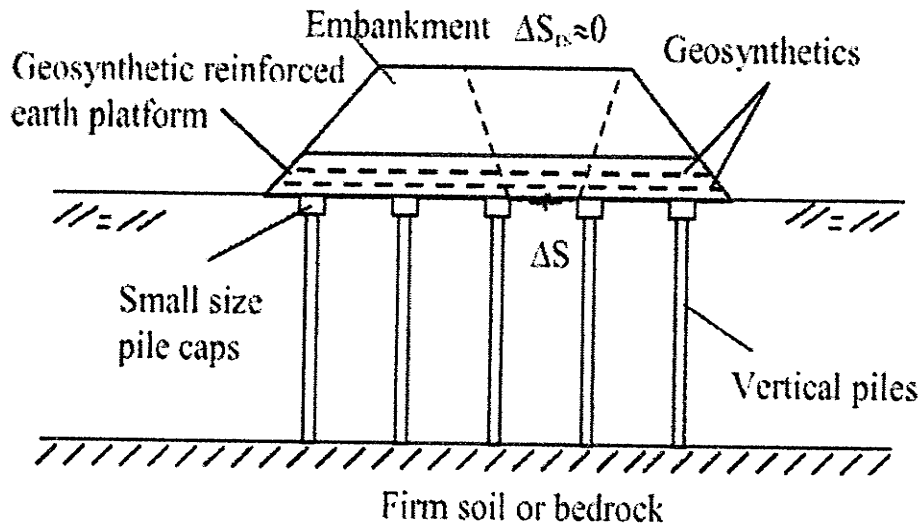

Figure 2.8: Combined effect of rock columns and geosynthetics

The use of pile supported structures is increasing and many materials are currently tested and reported in the literature to have worked properly in combination with geosynthetics. Han and Akins (2002) presented three case studies of use of geogrid-reinforced and pile-supported earth structures. Three different foundations were used: vibro-concrete columns; jet-grouting columns; and cast-in-place concrete piles. Satisfactory performance of these projects proved effectiveness in reducing total and differential settlements. In another application, Tweedie et al. (2004) reported results of a geotechnical investigation, slide remediation with stone columns, stability evaluation and performance

observations since completion of the repair of a highway embankment fill over an arch culvert.

Rockfill columns (also known as stone columns) can be designed to have drainage capabilities such that the rockfill columns provide increase in bearing capacity and the drainage capabilities increase the rate of consolidation or dissipation of excess pore water pressure. However, the rate of consolidation might be influenced by the construction process. Smear and well resistance zones due to aggregates contaminated with fine-grained soil particles might be generated around the rock columns that reduce their effectiveness in dissipating the excess pore water pressures, which delays the consolidation process. These problems have been analysed and a theoretical solution for consolidation rates of stone column-reinforced foundations accounting for smear and well resistance effects has been developed (Han and Ye, 2002).

CHAPTER 3

GROUND THERMAL MODELLING

3.1 INTRODUCTION

Embankments are one of the geotechnical structures that contribute to the disturbance of the permafrost. They have a large influence in the ground thermal regime since they modify the balance of the ground surface The disturbance of the surface due to the construction of embankments increases the mean annual surface temperature (MAST), this increment is also associated with the effect of the mean annual air temperature (MAAT) over the soil surface. The difference between the MAST and the MAAT can be as high as 8°C (Goering, 1996; Goering and Kumar, 1996; Goering, 2004). These differences depend on many factors such as snow and vegetation cover, slope and surface orientation, soil thermal properties, meteorological conditions, surface and subsurface drainage (Zarling and Rajesh, 1996).

Densification plays an important role. When soils are compacted the grain to grain contacts are increased and resistivity is lowered. This happens more notably at low moisture contents and in well graded soils where small particles can occupy spaces between larger particles. One of the factors that most influences thermal resistivity is the water content in the soil structure. When the

soil is dry, spaces between particles are filled with air increasing resistivity, but when these spaces are replaced by water soil thermal resistivity decreases (Steinmanis et al., 1996).

In order to avoid extreme degradation of discontinuous permafrost, it is necessary to asses the subsurface thermal regime and its interaction with geotechnical structures. Heat transfer through the soils takes place by conduction through the solid and solid-liquid interface, by convection through pore fluids and by vapour transfer of heat. Most of the heat is conducted through the soil particles and their contacts; therefore, resistivity is minimized for soil particles that maximize those contacts (Steinmanis et al., 1996).

This study investigates various strategies for road embankments to adapt the projected degradation of the discontinuous permafrost. This requires proper understanding of ground thermal regime and its interaction with the overlying road embankment. Thermal numerical analyses constitute a key factor in the prediction and performance of embankments over discontinuous permafrost. The following section describes the thermal model that has been calibrated with field measurements of ground temperatures for a three-year monitoring period. (1996-1998). The data was provided by Manitoba Transportation and Government Services. With a realistic modelling of the ground thermal regime, the thermal model was used to investigate the impacts of climate change on the permafrost. A climate model and scenario that was used in this study reproduced the air

temperature observed in the site. It was used to investigate the impacts of climate change on the permafrost beneath the road embankment for a fifty-year period. This allows a reasonable description of the ground thermal regime beneath road embankments and provides temporal estimation of freezing and thawing cycles of the ground.

3.2 BACKGROUND INFORMATION

The Northern Regions are an important economic and social resource for the country. Canada depends on these territories for energy, mining and natural reserves that are considered a large proportion of the provincial and national economy. To access these lands, roads and railways have had to be constructed over soils with very poor engineering properties. The construction of embankments is necessary to comply with a good road design. However, their construction contributes to the degradation of the permafrost since they modify the thermal regime of the soil.

Private and government agencies are interested in determining the conditions that cause most of the permafrost degradation and what can be done to mitigate the progressive deterioration of the road embankments in the Northern Regions.

The construction of embankments in Northern Manitoba has followed the same construction techniques as those used in warmer regions. The filling

material that is used in the embankment construction has high thermal conductivity, which transmits heat to the underlying layers leading to thawing of the previously stable foundation soil. The asphalt surface worsens this condition as it absorbs the heat for the sun and transfers it directly to the embankment. More favourable conditions occur when a layer of peat is present as part of the foundation soil since this soil is considered to be a natural insulator due to its low thermal conductivity.

It is generally observed that the permafrost degradation begins at the toe of the road embankment. This is because of the higher transmission of heat from the surface to the soil foundation due to the low fill thickness at the toe and accumulation of snow in side slopes; and ponding water that can increase the rate of degradation.

3.3 SITE LOCATION AND STRATIGRAPHY

Manitoba Transportation and Government Services (MTGS) has spent significant amount of money in the last 40 years to maintain transportation access with the Northern Regions. To improve the understanding of the performance of road embankments over discontinuous permafrost, MTGS installed thermistor strings at various depths beneath road embankments. The study site is located along PR #290, northeast of Gillam. Soil properties were determined based on boreholes information from this site as well from PTH #6,

approximately 65 km south of Thompson. Figure 3.1 shows the locations of these two sites.

These sites exhibit characteristics typical of road embankments constructed over degrading permafrost and include features as poor drainage, settlements, and loss of shoulder support. Soil data is complemented with information reported by AMEC Earth and Environmental Ltd (2002).

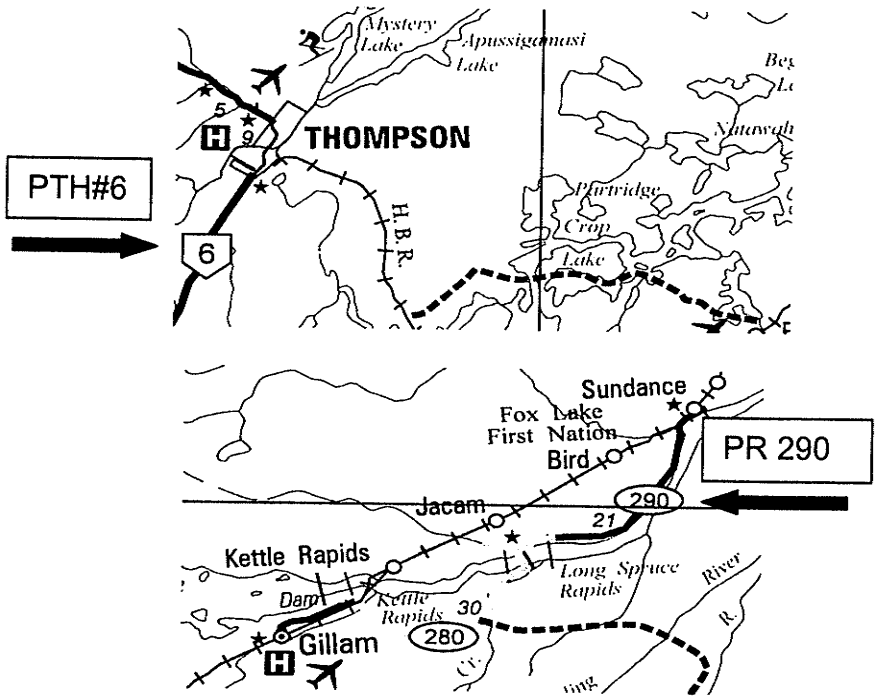

Figure 3-1: Site location

The stratigraphy at the site consists of 1.5 to 4 metres of coarse-grained soil embankment fill over 0 to 1 metre of organic soil (peat) on top of 6 to 8 metres of soft highly plastic clay. A complete sketch with the cross section is

provided in section 3.5 of this Chapter. The boreholes do not reach bedrock in any location and they were terminated between 6 and 12 metres depth. Permafrost was sporadically found in some locations as was expected. The groundwater table was found almost at the ground surface. This condition indicates complete saturation of the soils below the water table. The water content of the coarse-grained soil in the road embankment varies, with some indications that in some places it was not saturated; however, full saturation was assumed.

Peat is considered to be a natural insulator. Its low thermal conductivity prevents the degradation of the permafrost and therefore must be considered as an engineered material. However, both the lack of information about its thermal and mechanical properties and also the variability in thickness, water content and density from borehole to borehole provide additional concerns for the condition of permafrost.

The borehole logs as well as AMEC's report provided information about the moisture content, Atterberg limits, soil descriptions and classification. Soil samples were not available for determination of strength, deformation and flow characteristics and so these are estimated from index soil properties using empirical correlations found in the literature.

3.4 NUMERICAL MODELLING

Numerical modelling is often used in the performance prediction of geotechnical structures over discontinuous permafrost. It can provide information for developing innovative design and construction techniques appropriate for road embankments in the Northern Regions.

A commercially-available computer program TEMP/W (Geo-Slope International, 2004c) was selected as a platform for thermal modelling. It is a finite element code that can simulate thermal changes in the ground caused by climatic changes or by the construction of facilities such as road embankments, both of which can modify the thermal regime in the ground.

3.4.1 HEAT FLOW CHARACTERISTICS

Conduction is the most important mechanism for heat flow through soils in most of the engineering problems involving freezing and thawing. Conduction is the flow of heat by the passage of energy from one soil particle to another or through soil pore fluids (Farouki, 1985). The heat flux (q) is proportional to the thermal conductivity (k) and the thermal gradient ($\partial T / \partial x$).

[3-1] $$q = -k\frac{\partial T}{\partial x}$$

This formulation is similar to the formulation of fluid flow through porous medium following Darcy's law. The negative sign, as in Darcy's law, indicates that heat flows in the direction from high temperature to low temperature.

3.4.2 GOVERNING EQUATION

The governing differential equation used is (Geo-Slope International, 2004c):

[3-2]
$$\frac{\partial}{\partial x}\left(k_x \frac{\partial T}{\partial x}\right)+\frac{\partial}{\partial y}\left(k_y \frac{\partial T}{\partial y}\right)+Q=\lambda \frac{\partial T}{\partial t}$$

Where:

T = Temperature

k_x = thermal conductivity in the x-direction

k_y = thermal conductivity in the y-direction

Q = applied boundary flux

λ = capacity for heat storage or heat capacity

t = time

This equation states that the difference between the heat flux entering and leaving an elemental volume of soil at a point in time is equal to the change in the stored heat energy. Under steady-state conditions, the flux entering and leaving

an elemental volume is the same at all times. The right side of the equation vanishes and reduces to:

[3-3]
$$\frac{\partial}{\partial_x}\left(k_x \frac{\partial T}{\partial x}\right)+\frac{\partial}{\partial y}\left(k_y \frac{\partial T}{\partial y}\right)+Q=0$$

There are two parts in the heat capacity term (λ). The first part is the volumetric heat capacity of the material frozen or unfrozen. The second part is the latent heat associated with the phase change due to heat released or absorbed.

[3-4]
$$\lambda = c + L\Theta \frac{\partial \Theta_u}{\partial T}$$

Where:

c = volumetric heat capacity of the material

Θ = Volumetric water content at the initiation of freezing

L = Latent heat of water

$\partial \Theta_u / \partial T$ = change in unfrozen water content of the soil with temperature

Substituting λ in Equation [2] leads to the complete differential equation:

[3-5]
$$\frac{\partial}{\partial x}\left(k_x \frac{\partial T}{\partial x}\right)+\frac{\partial}{\partial y}\left(k_y \frac{\partial T}{\partial y}\right)+Q=\left(c+L\Theta \frac{\partial \Theta_u}{\partial T}\right)\frac{\partial T}{\partial t}$$

The term $L\Theta\partial\Theta_u / \partial T$ represents the amount of heat released or absorbed as the temperature of the soil changes by ∂T. Equation number [3-5] is applied to every element in the model domain to account for the change in temperature in a transient analysis. Equation [3-5] reduces to Equation [3-6] for the cases where freezing or thawing are not occurring.

[3-6]
$$\frac{\partial}{\partial x}\left(k_x \frac{\partial T}{\partial x}\right) + \frac{\partial}{\partial y}\left(k_y \frac{\partial T}{\partial y}\right) + Q = c\frac{\partial T}{\partial t}$$

3.5 MODEL GEOMETRY AND BOUNDARY CONDITIONS

3.5.1 MODEL GEOMETRY

The model cross-section used for this analysis is the same used by Thiessen (2003), which was based on the cross-section used in the preliminary numerical analysis performed by Manitoba Hydro in conjunction with the studies done by Artic Foundations (Artic Foundations of Canada, 1995). Some changes have been made to optimize the finite element mesh. Figure 3-2 represents the cross section, materials and finite element mesh. Symmetry was assumed at the centerline of the embankment to maximize computing efficiency.

The model cross section was compared to the cross sections reported by AMEC in 2002 on PTH #6 at a site south of Thompson, Manitoba, where basic soil properties were also obtained. These materials were similar to those found in

Gillam on PR #290. Soil data from both Gillam and Thompson sites were therefore used in the modelling including the determination of thermal and mechanical soil properties from empirical correlations.

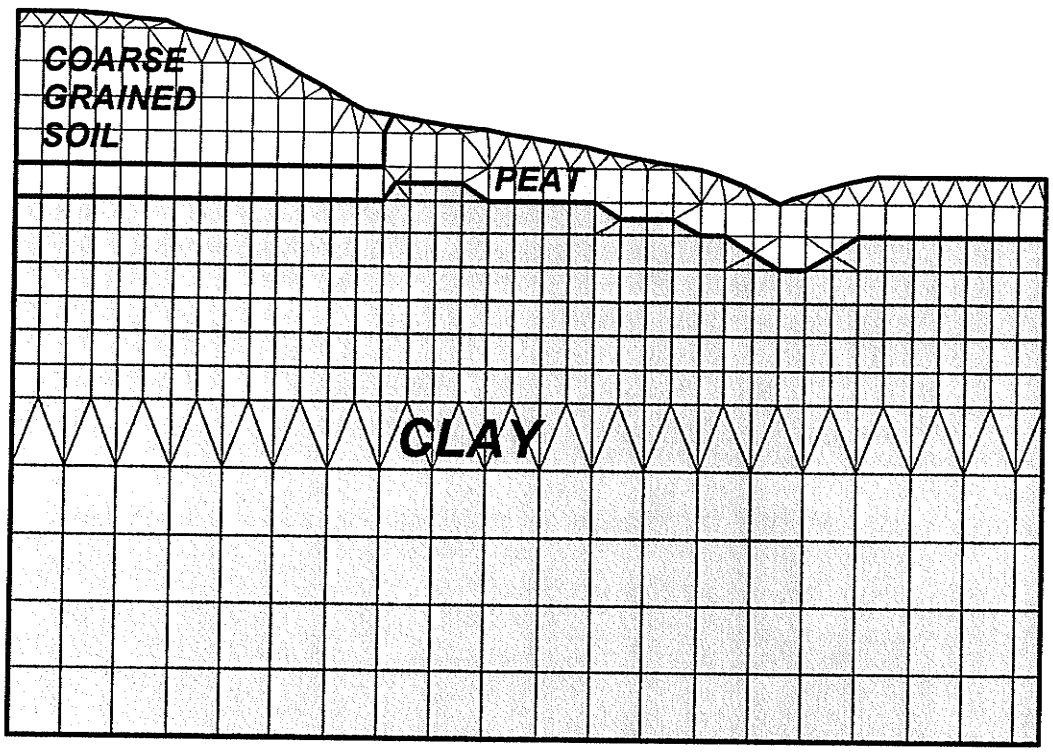

Figure 3-2: Cross section, materials and finite element mesh for thermal modelling.

As shown in Figure 3-2, the model has three materials: 1) coarse-grained soil used as fill material, 2) peat, which is located at the upper layer of the ground, and 3) clay. The model is 29.25 metres wide and 10.40 metres in depth from the road elevation. The fill material is 2.12 metres deep at the centerline with a slope of 3:1. The peat layer is 0.5 metres deep beneath the embankment, and the clay layer extends to the bottom of the model domain.

The finite element mesh is primarily made up of quadrilateral elements with some triangular elements in zones of transition between materials. The mesh is more detailed in the upper part of the model domain to account for the greatest variations of temperature near the surface.

3.5.2 BOUNDARY CONDITIONS

The model has two vertical zero flux boundaries, a bottom constant temperature boundary of 0.5 °C and an upper boundary condition that is dependent on empirical factors, corresponding to the type of material, and temperature. Figure 3-3 is the cross section with the description of the boundary conditions and the surface materials: asphalt, coarse-grained soil and peat.

The zero flux boundary condition on the left hand side is applicable because symmetry of the model forces the energy balance at the centerline to be zero (heat flow coming in and going out). On The right hand side it is assumed that the zero-flux applied boundary condition at this extent will not affect significantly the outcome of the model in areas of interest, which is at the vicinity of the road embankment.

Thiessen (2003) analysed the model reported by Artic Foundations, which presents the bottom boundary condition as zero-flux boundary. With a flux boundary equal to zero, the model becomes very sensitive to the season at which the modelling had started. If the model is started in fall or winter the

permafrost table reaches the bottom of the model domain because the temperatures are very low. If the model is started in spring, the high summer temperatures followed by the winter season would unrealistically trap heat near the bottom of the model domain.

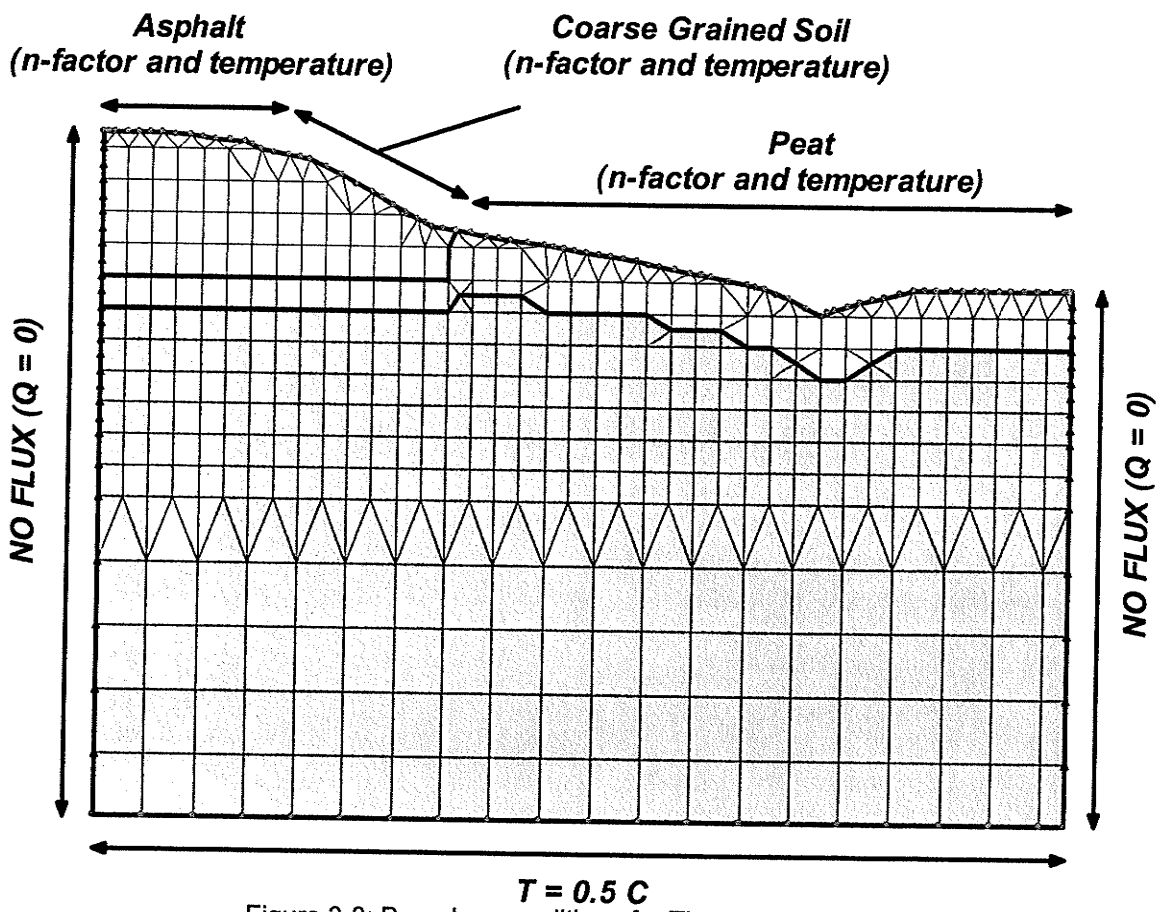

Figure 3-3: Boundary conditions for Thermal Modelling

To represent more accurately the actual conditions at the bottom boundary, two different types of approaches might be used: 1) a constant temperature boundary if its influence in the model is negligible; for instance, the bottom boundary is deep enough or 2) infinite elements at the bottom boundary

might be applied. The infinite boundary condition can be useful for analyzing unbounded problems where the boundary conditions are unknown or are only known at some distance.

Andersland and Layandi (2004) indicated that there is very little seasonal variation in temperatures at about 9 to 15 metres below the ground surface as it is shown in Figure 3-4. Therefore, the bottom boundary at about 9 metres below the ground surface was set to 0.5°C based on the average of thermistor data provided by MTGS from PR #290.

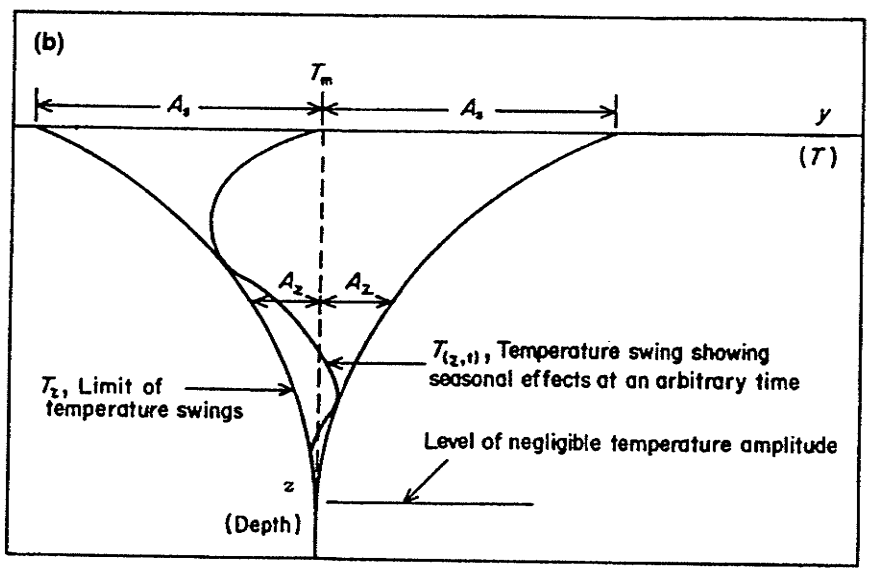

Figure 3-4: Temperature attenuation with depth (After Andersland and Ladanyi, 2004).

The use of infinite elements at the bottom boundary was also investigated leading to similar results when using the 0.5°C constant temperature at the bottom boundary (Thiessen, 2003). This approach, even though more realistic, was not selected due to the amount of computing time required.

For the upper boundary, it is necessary to determine the heat transfer at the ground surface. The varying climate conditions (air temperature), make the ground surface temperature to vary from year to year as shown in Figure 3-5. This sinusoidal variation is transmitted to the ground with amplitude that decreases with increasing depth until, at about 9 to 15 metres, where the temperature remains approximately constant throughout the year (Andersland and Ladanyi, 2004) as represented in Figure 3-4.

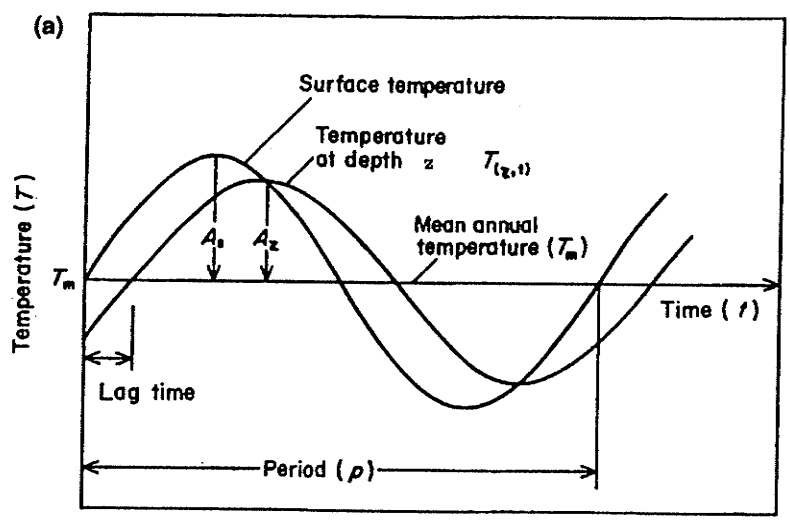

Figure 3-5: Surface and ground temperature, sinusoidal fluctuations (After Andersland and Ladanyi, 2004).

TEMP/W uses two different approaches to calculate the ground surface temperatures: 1) an analytical approach based on climate data and 2) an empirical approach based on empirical factors. The analytical approach requires values of maximum and minimum temperature and relative humidity, wind speed, amount of precipitation, starting and ending period of the precipitation event, longitude, latitude, maximum snow temperature, and minimum rain temperature.

With this information TEMP/W computes the factors to solve the energy balance at the surface, which is given by Equation [3-7] to calculate the ground surface temperatures.

[3-7]
$$Q_G = -k\frac{\partial T}{\partial n} + Q_{SW} + Q_{LW} + Q_H + Q_E$$

Where:

Q_G = ground surface heat flux

k = thermal conductivity at the ground surface

$\partial T / \partial n$ = heat gradient normal to and evaluated at the ground surface

Q_{SW} = net flux of solar radiation

Q_{LW} = net flux of long wave radiation

Q_H = net flux of sensible heat

Q_E = net flux of latent heat associated with evaporation of moisture from the surface

Although the analytical approach seems to be applicable for a very detailed analysis, it was not considered for the following reasons. 1) It is uncertain to estimate with reasonable precision the individual heat flux components of Equation [3-7] from the climate data (maximum and minimum temperature and relative humidity, wind speed, amount of precipitation, starting

and ending period of the precipitation event, maximum snow temperature and minimum rain temperature), which can introduce large errors in the model. 2) TEMP/W does not consider the infiltration of water into the soil from melting snow pack that can influence the energy balance. 3) It is challenging to represent the thermal behaviour of asphalt, gravel and peat when they are exposed to sunlight or covered by snow.

The empirical approach, which is the most commonly used, correlates boundary modifiers and average air temperature to calculate the ground surface temperatures. The following paragraphs give a thorough explanation about this approach.

Boundary modifiers, "n-factors", for the upper boundary

The most common practice to determine the ground surface temperature is to use boundary modifiers (empirical factors) that along with the air temperature permit the estimation of the ground surface temperature of any surface material. The "n-factors" for freezing, n_f, and thawing, n_t, conditions are defined as the ratio of the surface freezing or thawing index (I_{sf} and I_{st}) to the air freezing or thawing index (I_{af} and I_{at}) as describe in the following equations:

[3-8]
$$n_f = \frac{I_{sf}}{I_{af}}$$

[3-9]
$$n_t = \frac{I_{st}}{I_{at}}$$

The air freezing index is the number of negative (T < 0°C) degree-days between the highest and the lowest points on a curve of cumulative degree-days versus time (Figure 3-6). The air thawing index is the number of degree-days between the minimum in spring and the maximum next autumn (Andersland and Ladanyi, 2004).

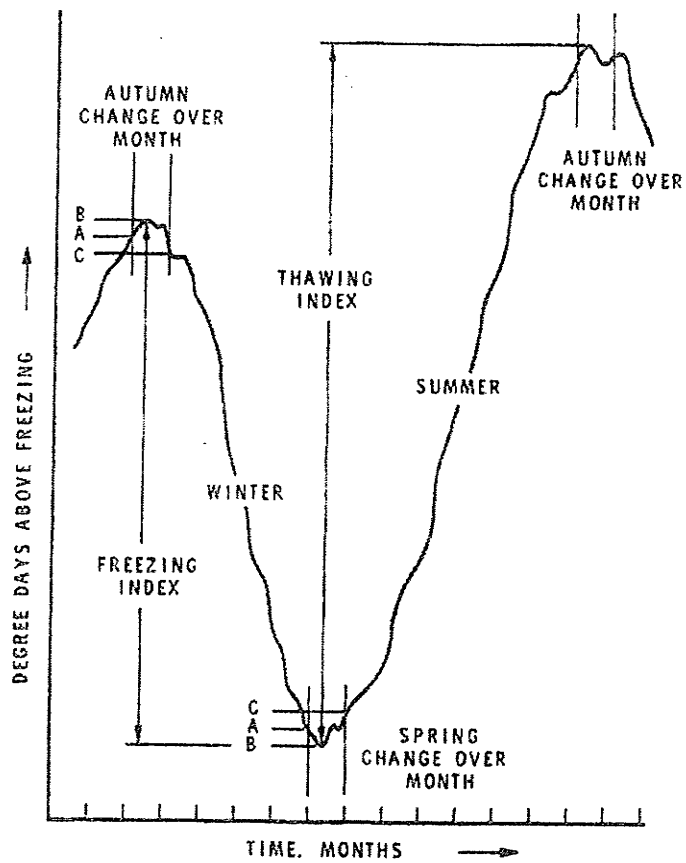

Figure 3-6: Definition of thawing and freezing indices (After Andersland and Ladanyi, 2004).

According to the definition of both air freezing and thawing indices and n-factors, Equations [3-8] and [3-9] lead to:

[3-10]
$$n_f = \frac{\sum_{i=1}^{n} T_{sf}}{\sum_{i=1}^{n} T_{af}} = \frac{I_{sf}}{I_{af}}$$

Where:

T_{af} = average daily temperature for the freezing season

T_{sf} = average daily surface temperature for the freezing season

The same procedure can be used to calculate n_t. If the values of air average daily temperature for freezing and thawing as well as the values of n-factor for freezing and thawing conditions are known, the values of ground temperature can be calculated by using Equation [3-10]. Ground surface temperatures are the temperatures that TEMP/W uses to run the simulation.

The magnitude of these "n-factors" depends on the climatic conditions as well as the type of surface. Many references provide values for a combination of different types of surfaces (asphalt, concrete, gravel, bush, etc) and climatic and hydrological conditions (snow cover, net radiation, surface relief, subsurface drainage, etc). Table 3-1 summarizes the values of n-factors used in the model.

Freezing n-factors also include the effect of snow cover in the winter season (Andersland and Ladanyi, 2004; Lunardini, 2004).

Table 3-1: N-factors for the surface materials in the model

MATERIAL	n-factor freezing	n-factor thawing
Asphalt	0.9	1.9
Coarse-Grained Soil	0.3	1.5
Peat	0.3	0.73

Air temperatures

Since the calculation of the ground surface temperatures involves a combination of boundary modifiers and air temperatures it was necessary to determine the average air temperatures for the site. The station Thompson A was selected because it is the closest station with the largest historic database. The information of this station is available in Environment Canada Weather Office web page (http://www.climate.weatheroffice.ec.gc.ca).

Manitoba Transportation and Government Services (MTGS) provided thermistor data information taken for a period of three years from 1996 to 1998. To have a comparison between the modelled data and the observed data the values of daily average air temperature from 1996 to 2004 are used to run the simulation and to calibrate the model.

Figure 3-7 shows the corresponding data for the time period between 1996 and 2004. This information along with the boundary modifiers is used by TEMP/W to determine the ground surface temperature.

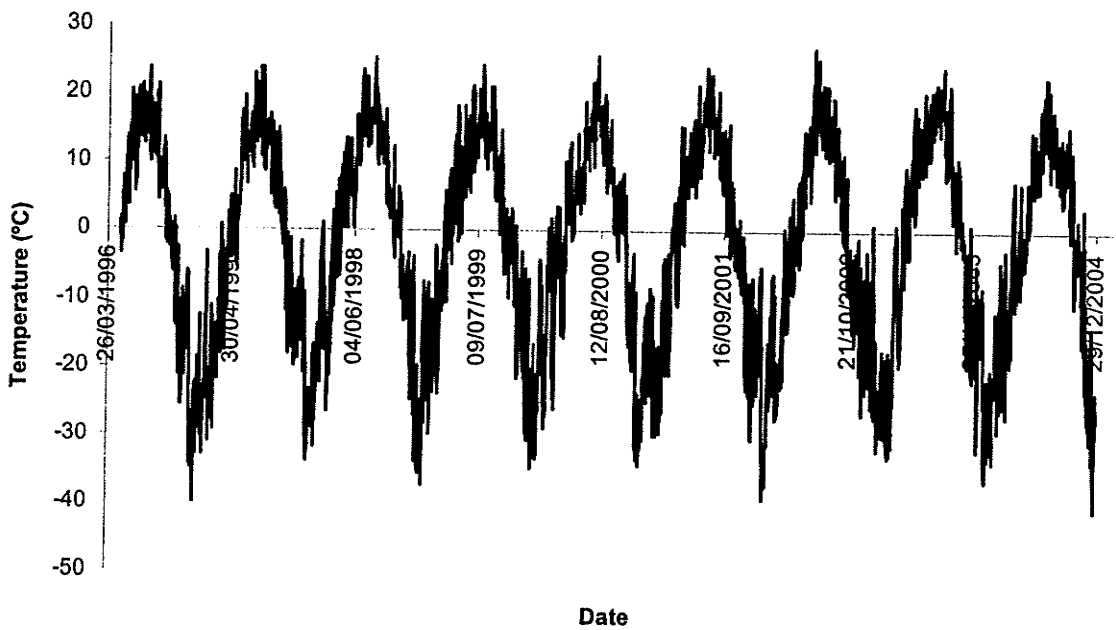

Figure 3-7: Temperature data: Station Thompson A, time period from May 1996 to December 2004

3.6 INITIAL CONDITIONS

To be able to run the simulation, TEMP/W needs to have specified the initial values of temperature in every node. This can be done by manually specifying the temperature in every node of the mesh or by running a steady-state simulation in TEMP/W and use that file as the initial conditions. Figure 3-8 is the cross section with the initial conditions used to calculate the initial temperatures.

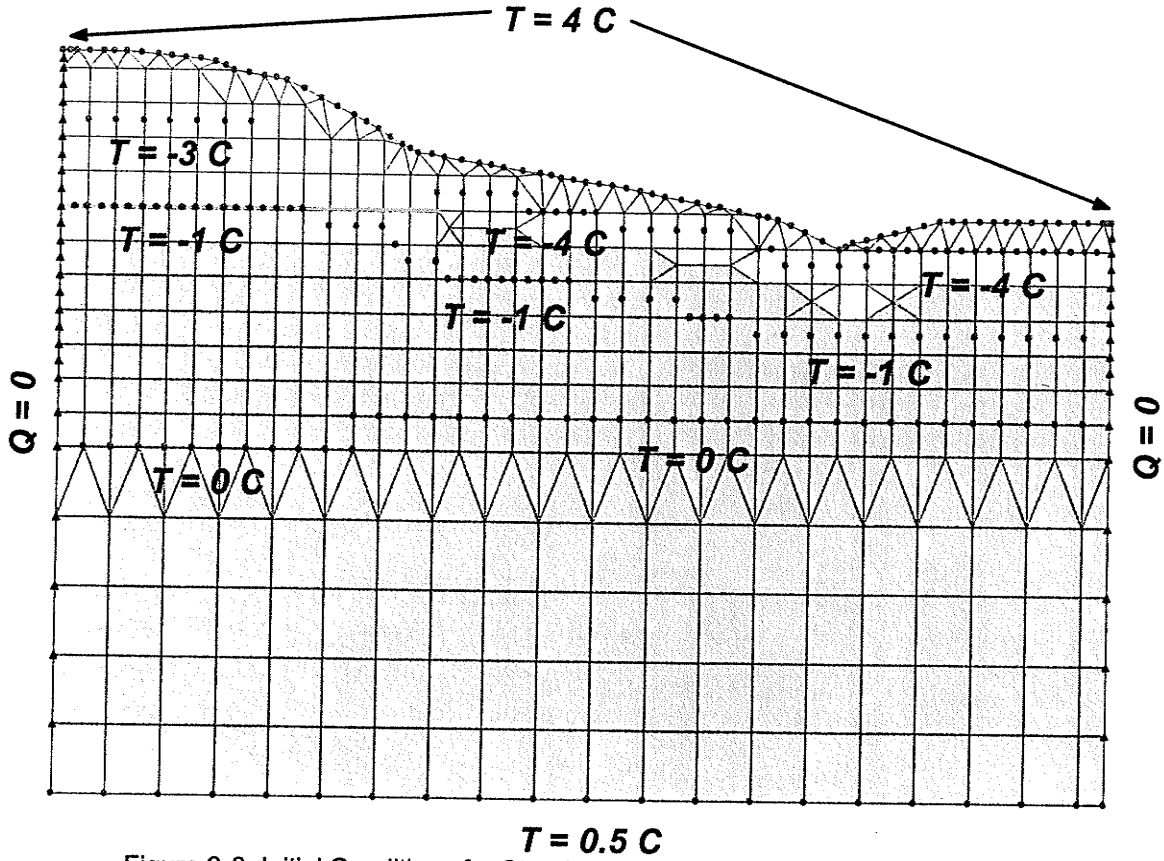

Figure 3-8: Initial Conditions for Steady-state analysis for Thermal Modelling.

The thermistor data provided by MTGS was used as input information to calculate the initial conditions in every node by running a steady-state analysis. The time period begins on May 4, 1996, the date in which ground temperature data collection started.

3.7 MATERIAL PROPERTIES

Thiessen (2003) estimated soil parameters based on the information provided by MTGS. These values were recalculated to prove reliability and in

some occasions were changed when it was considered appropriate and ensuring that they were within normal ranges reported in the literature.

3.7.1 VOLUMETRIC WATER CONTENT

The volumetric water content represents the fraction of the total volume of soil that is occupied by water. The volumetric water content can be defined by:

[3-11]
$$\Theta = \frac{V_l}{V_t} = \frac{V_l}{V_s + V_p}$$

Where:

V_l = Volume of liquid phase in the soil sample

V_s = Volume of solid phase in the soil sample

V_p = Volume of pore space in the soil sample

V_t = Volume of total pore space

The volumetric water content can be also expressed in terms of either the water content or the degree of saturation according to the following formulas:

[3-12]
$$\Theta = w\left(\frac{\rho_b}{\rho_w}\right)$$

[3-13] $$\Theta = nS$$

Where:

ρ_b = bulk density of the soil

ρ_w = water density

n = porosity

S = degree of saturation

w = water content

MTGS data and AMEC's report are used in this case to determine the water contents of the soil layers. Since most of the data comes from different locations close to the site it was decided to take the average water contents from all the boreholes to represent the typical characteristics instead of the conditions at a specific site. Most of the data is in terms of water content (not volumetric); thus, it is necessary to convert these values to volumetric water contents by using Equations 3-11, 3-12 and 3-13 and also the following phase relationships:

[3-14] $$e = \frac{wG_s}{S}$$

[3-15] $$n = \frac{e}{1+e}$$

[3-16] $$\rho_d = G_s(1-\Theta)$$

Where:

e = void ratio

G_s = Specific gravity

ρ_d = dry density

Table 3-2: Phase relationships for soil layers

MATERIAL	Water content (w)	Porosity (n)	Void ratio (e)	Dry density (ρ_d)	Volumetric water content (Θ)
Coarse-Grained soil	0.06	0.14	0.16	275	0.14
Peat	3.25	0.89	8.45	2200	0.89
Clay	0.18	0.33	0.49	1810	0.33

Table 3-2 compiles the information data calculated during the process of converting the water contents to volumetric water contents. Some assumptions had to be made to calculate the volumetric water contents. Values of specific gravity of 2.65, 2.7 and 2.6 for coarse-grained soil, clay and peat respectively were taken from the literature (Andersland and Ladanyi, 2004). Also, it was assumed that the soils are 100% saturated. This assumption seems reasonable given that the water table was found to be at the surface based on field site investigation conducted by MTGS field engineers.

3.7.2 UNFROZEN WATER CONTENT

For frozen fine-grained soils, some water remains unfrozen over a considerable range of negative temperatures. This water is adsorbed around the clay particles and can increase in amount with increasing specific surface area of the clay particles. The unfrozen water content includes water that is both mobile and immobile.

Unfrozen water content is a key factor in the thermal behaviour of frozen soils. It plays a very important role in the process of water migration to the freezing zone and in improving the thermal contact between the soil matrix and the ice (Farouki, 1985).

The unfrozen water content w_u, has been related to the specific surface area S (m^2/g) and the temperature T (°C) as follows (Anderson et. al, 1973; Farouki, 2004):

[3-17] $$\log_e w_u = 0.2618 + 0.5519 \log_e S - 1.449 S^{-0.264} \log_e T$$

For the case of this study, the specific surface areas of the soil particles have not been determined. Published graphs available in the literature were used to determine the unfrozen water content function for the three materials in the model.

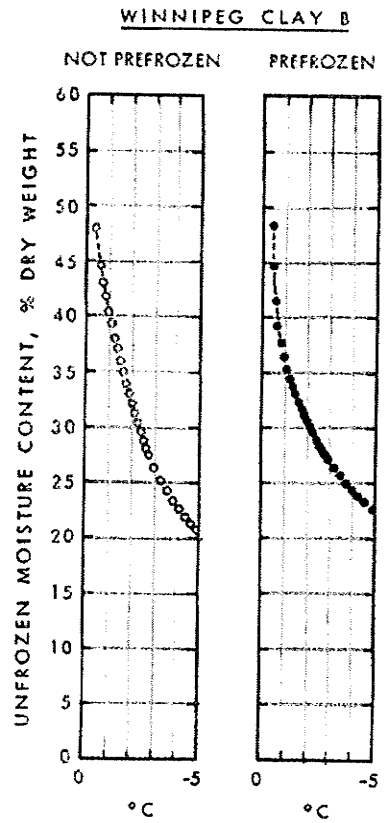

Figure 3-9: Unfrozen water content function for Winnipeg clay (After Johnston, 1981)

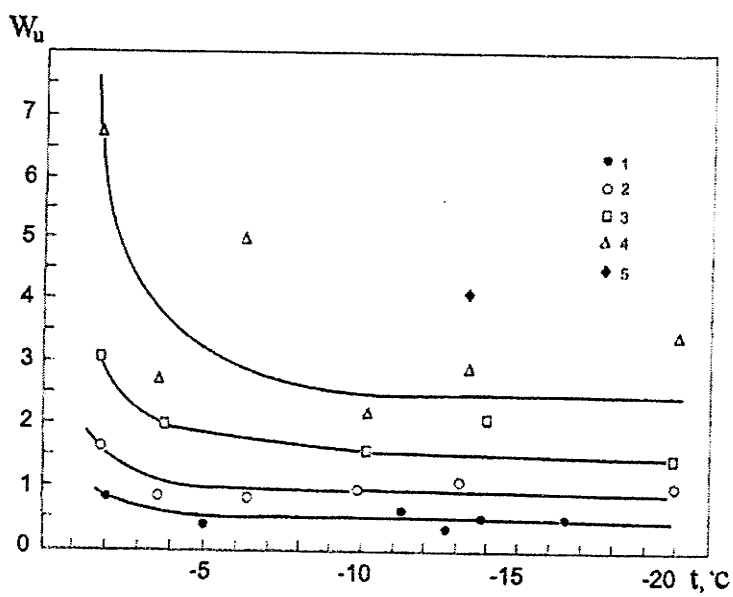

Figure 3-10: Unfrozen water content function for peat for different water contents. 1:2.20; 2:3.10; 3:6.40; 4:13.20; 5:21.40 (After Gavriliev, 2004)

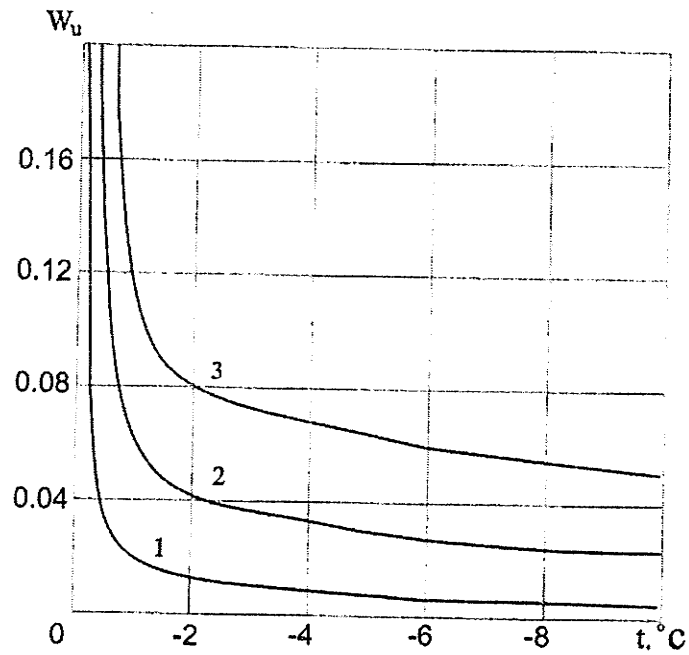

Figure 3-11: Unfrozen water content function. 1: Sand; 2: Sandy loam; 3: Clayey loam (After Gavriliev, 2004)

The values of unfrozen water content versus temperature were extracted from the graphs and the functions were constructed manually in TEMP/W. Figures 3-9, 3-10 and 3-11 are the graphs used to determine the unfrozen water content functions for clay, peat and coarse-grained soil respectively

3.7.3 THERMAL CONDUCTIVITIES

Thermal conductivity is a measure of the quantity of heat that will flow through a unit area of substance in unit time under a unit time gradient. For an unfrozen soil analysis the thermal conductivity can be considered constant. However, for frozen soil analysis this can not be neglected since the thermal

conductivity is function of the water content, which at the same time is a function of the temperature.

Since the thermal conductivity of ice can be four times the thermal conductivity of the water (Farouki, 1985), the thermal conductivity of the soil increases as the soil freezes and decreases as the soil thaws. Thermal conductivity can also be expressed in terms of its variation with respect to the change in temperature.

It is known that thermal conductivity is function of the saturation of the soil as well as the amount and type of minerals present in the soil. There are a different number of empirical methods to calculate the thermal conductivity of soils. Farouki (1985) reviews some the methodologies available to determine the thermal conductivity.

Johansen's method is the most common and reliable approach to compute the soil thermal conductivity. It is an interpolation technique between the dry and saturated values and does not take into account possible moisture migration at intermediate degrees of saturation (Andersland and Layandi, 2004). However, it is a method that requires knowing the amount of quartz fraction of the total solids content. Coarse-grained soils are dominated by quartz, which have a relative higher thermal conductivity compared to fine grained soils that are dominated by clay minerals, which are considered to have lower values of thermal conductivity.

Since the mineral composition of the soils is not known, the values of thermal conductivities were calculated based on graphs found in the literature that relate to the water content of the soil, its dry density and the percentage of saturation. The frozen and unfrozen values of thermal conductivity for the three soils were calculated and are summarized in Table 3-3. As in the case of the unfrozen water content the function thermal conductivity versus temperature was plot manually and input in TEMP/W in the format shown in the Figure 3-12.

Table 3-3: Frozen and unfrozen thermal conductivities

MATERIAL	Frozen thermal conductivity kJ/(day.m.°C)	Thawed thermal conductivity kJ/(day.m.°C)
Coarse-Grained soil	346	242
Peat	91	36
Clay	173	138

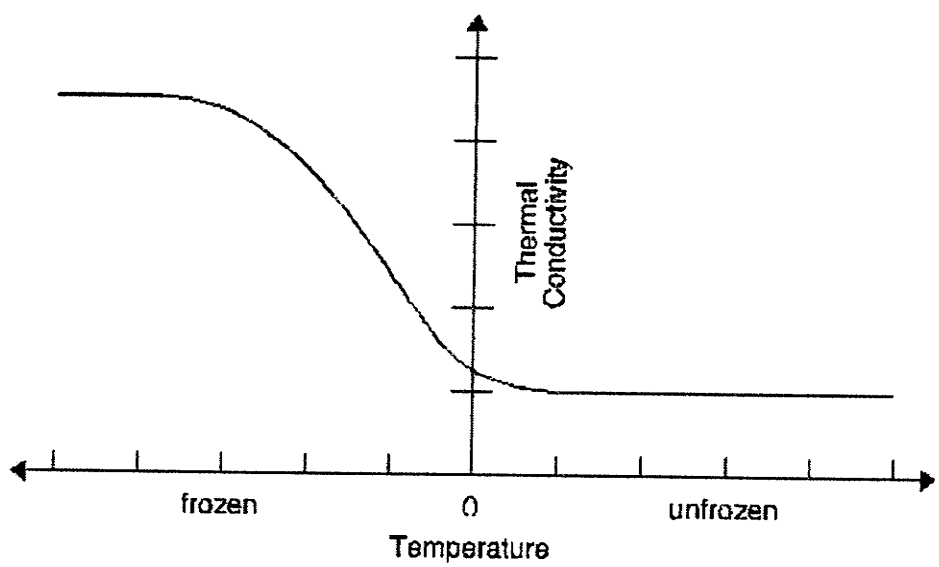

Figure 3-12: Thermal conductivity as function of temperature (After Geo-Slope International, 2004c)

Figures 3-13, 3-14 and 3-15 show the graphs used to calculate the frozen and unfrozen thermal conductivities for clay, sand, and peat, respectively.

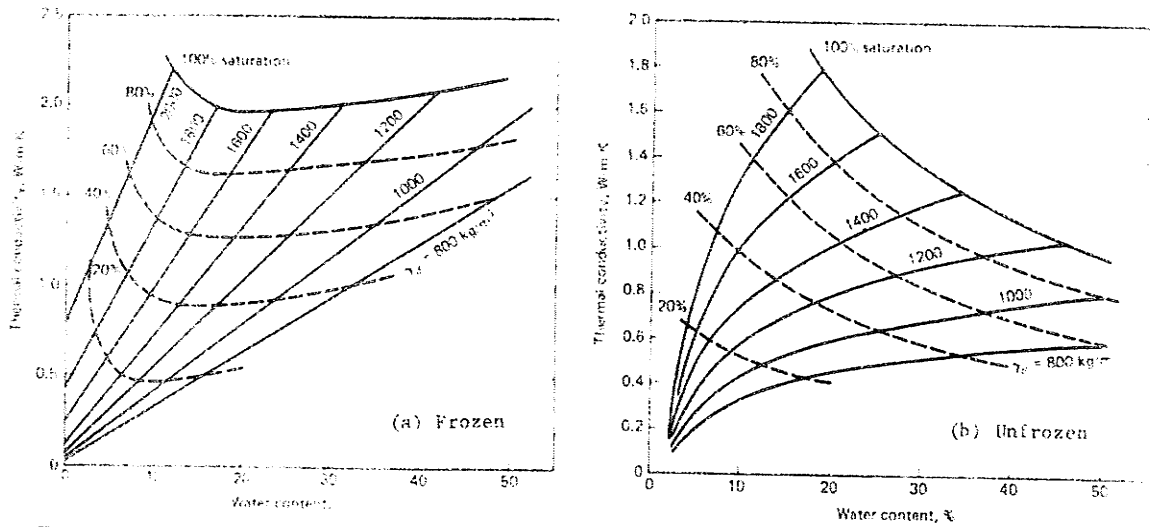

Figure 3-13: Average thermal conductivity function for clays: (a) frozen; (b) unfrozen (After Andersland and Ladanyi, 2004)

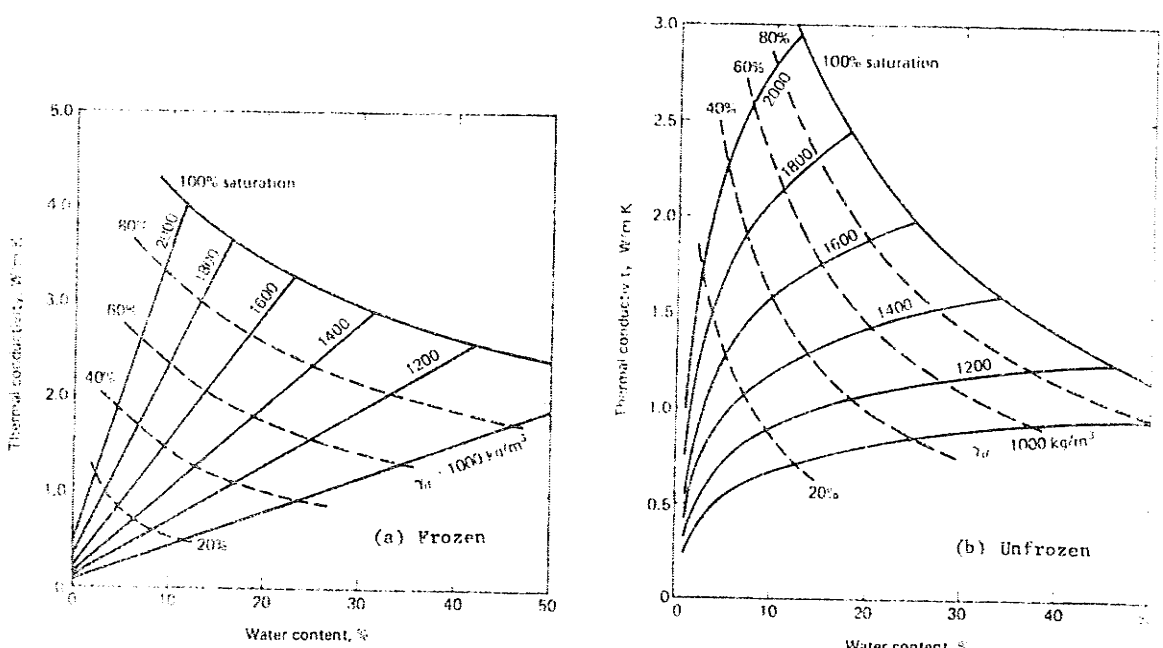

Figure 3-14: Average thermal conductivity for coarse-grained soils: (a) frozen; (b) unfrozen (After Andersland and Ladanyi, 2004)

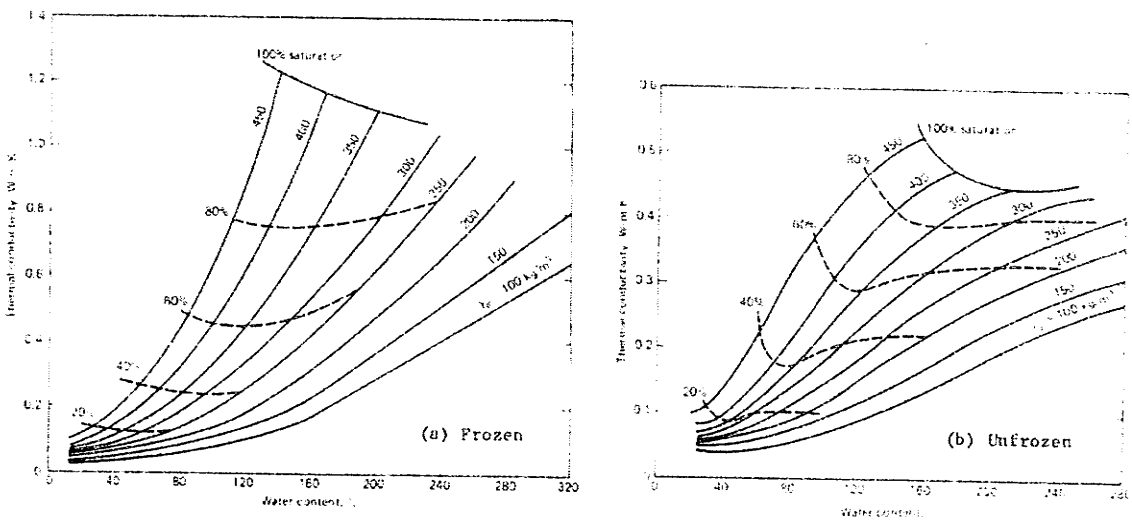

Figure 3-15: Average thermal conductivity for peat: (a) frozen; (b) unfrozen (After Andersland and Ladanyi, 2004)

3.7.4 HEAT CAPACITY

Heat capacity is the amount of heat required to raise the temperature of a unit mass of a substance by one degree. The heat capacity can be computed by adding the heat capacities of the soil constituents in a unit mass soil. Equation 3-18 is the basic formulation to calculate the heat capacity of a soil (Farouki, 2004):

[3-18] $$c(kJ/kg.\circ C) = x_s c_s + x_w c_w + x_a c_a$$

Where:

x_s = volume fraction of solids

c_s = heat capacity per unit volume of solids

x_w = volume fraction of water

c_w = heat capacity per unit volume of water

x_a = volume fraction of air

c_a = heat capacity per unit volume of air

For saturated soil the volumetric heat capacities in the unfrozen and frozen conditions are given by Equations (3-19) and (3-20) respectively:

[3-19]
$$c_u = \frac{\rho_d}{\rho_w}\left(0.18 + 1.0\frac{w}{100}\right)c_w$$

[3-20]
$$c_f = \frac{\rho_d}{\rho_w}\left(0.18 + 0.5\frac{w}{100}\right)c_w$$

Where:

ρ_d = soil dry density

ρ_w = water density

w = water or ice content

Table 3-4 summarizes the frozen and unfrozen heat capacities calculated for the three materials.

Table 3-4: Frozen and unfrozen heat capacities

MATERIAL	Frozen heat capacity kJ/(m^3.°C)	Thawed heat capacity kJ/(m^3.°C)
Coarse-Grained soil	3380	4118
Peat	2452	4328
Clay	2920	4174

3.8 SIMULATION STAGES

Once the material properties were determined, the model was run in two stages. First, a simulation between 1996 and 2004 was performed to compare the simulated values of ground temperature with the observed thermistor data in 1998. This simulation stage serves to fine tune and calibrate the model parameters.

A 50 year simulation was then performed beginning in 2005 to investigate the thermal behaviour of the soil foundation under a simulated climate change trend model from the Canadian Centre for Climate Modelling and Analysis (CCCMA).

TEMP/W includes a new adaptive time stepping scheme that inserts time steps when necessary. Extra time steps are inserted if the percentage change in temperature from one time step to the next exceeds the specified tolerance required by the user in the analysis. This prevents prediction of unrealistic values of temperature when there are abrupt changes of temperature during the day or

from one day to the next day. TEMP/W recommends maximum time steps of 1/8 or 1/4 of a day to prevent overshooting of the modelled values. A sensitivity analysis was performed to compare the results of using the time steps recommended by TEMP/W and the time step of 1 day. Very small differences were found in the results between time steps of 1 day and those recommended by TEMP/W. It was then decided to use time steps of 1 day because of the amount of data to simulate (50 years period). Furthermore, it was more important to represent an average future scenario rather than a very precise profile of temperatures.

3.8.1 SIMULATION

Manitoba Transportation and Government Services supplied data from 26 thermistor strings along PTH#290 from 1996 to 1998. The thermistors reported temperature until 9 metres depth from the ground surface. The boreholes reported by AMEC provide a good match between the finite element mesh and the field site investigation in terms of the soil layers and properties. It was decided that the model can be represented by average site conditions instead of specific site conditions to represent average conditions from the two sites (PTH#6 and PR 290).

The thermistor strings were installed at three points: at the toe in both the south and north slopes and at the centerline of the road embankment. Since the model was set up to be symmetrical around the centerline the influence of the

north or south slopes has not been considered. The south or north slopes would be important for the case of calculating the hours of sunshine the slope is exposed to. Since this analysis uses the empirical approach (as opposed to using analytical approach that considers the energy balance), it is not possible to determine the difference between the two slopes.

The periods in February, June, July, October and November were selected to fine tune and calibrate the model. These months are expected to represent most of the drastic changes in ground temperature due to the change in seasons. Figures 3-16 to 3-21 show the calibration results at the toe and at the centerline of the road embankment. The simulated values tend to show a change in the temperature profile at the edge of the peat layer because of the very low conductivity and high water content of the peat. Given that the soil profiles and material properties are not known at the same locations in which the thermistors are installed, the model is considered to have reflected the actual temperatures reasonably well.

It can be appreciated from Figures 3-16 to 3-21 that the simulated and observed data follow agreed and that the ground temperatures at the foundation soil level, which is most important for the analysis, match with reasonable accuracy with the observed data. The soil foundation is located 1 metre beneath the surface at the toe of the embankment and 2 metres at the centerline respectively.

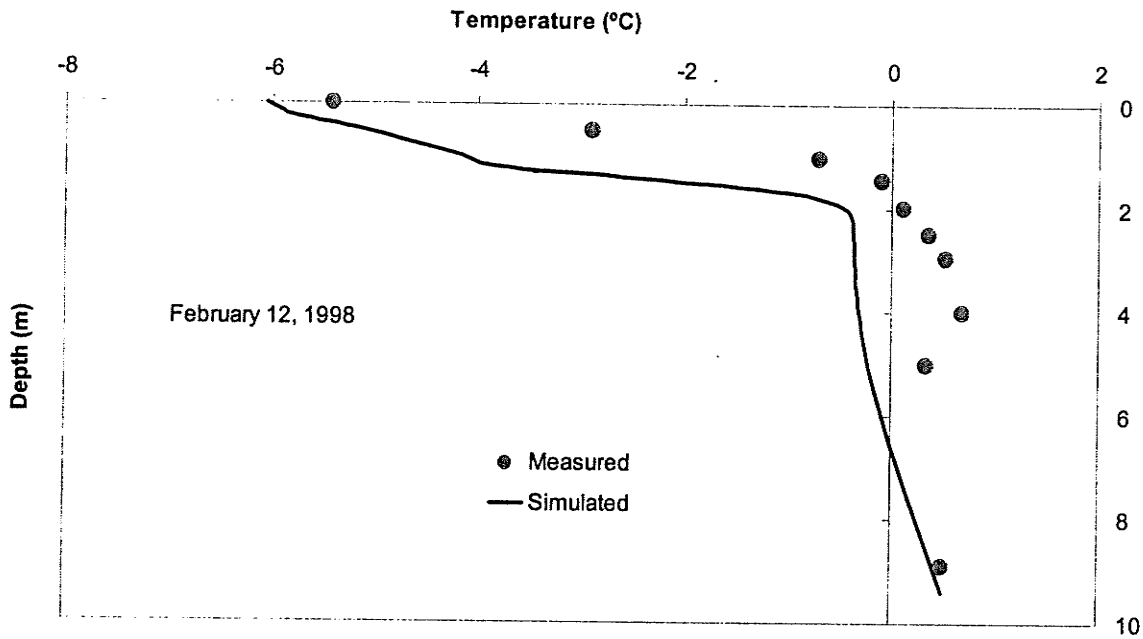

Figure 3-16: Thermal Modelling calibration analysis at the toe of the embankment for February 12, 1998

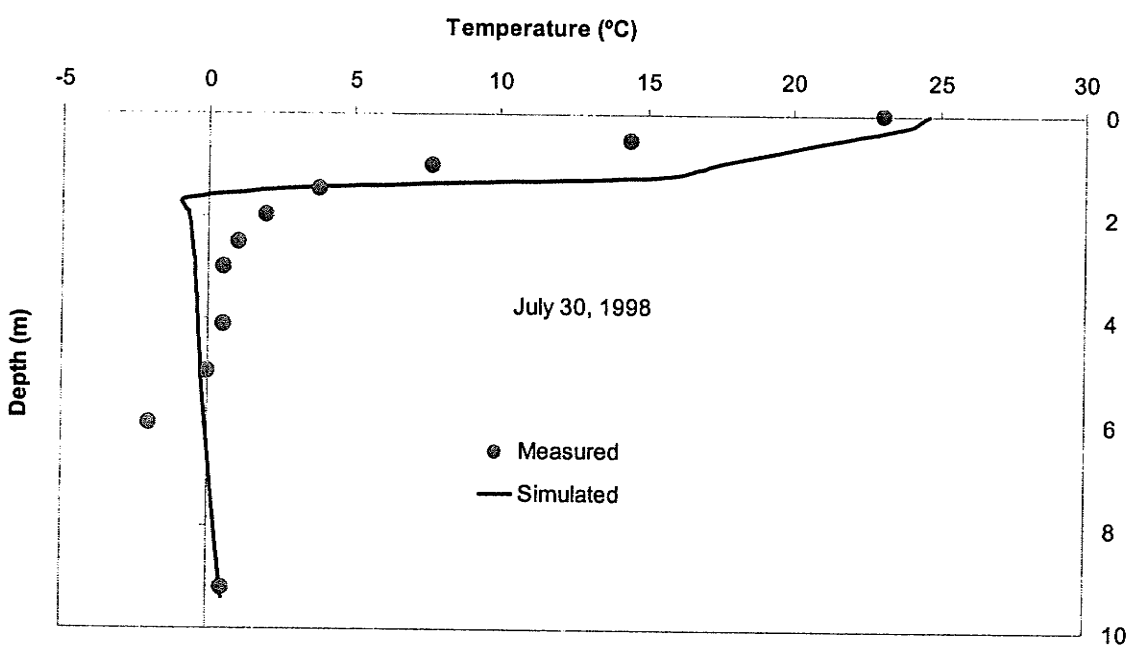

Figure 3-17: Thermal modelling calibration analysis at the toe of the embankment for July 30, 1998

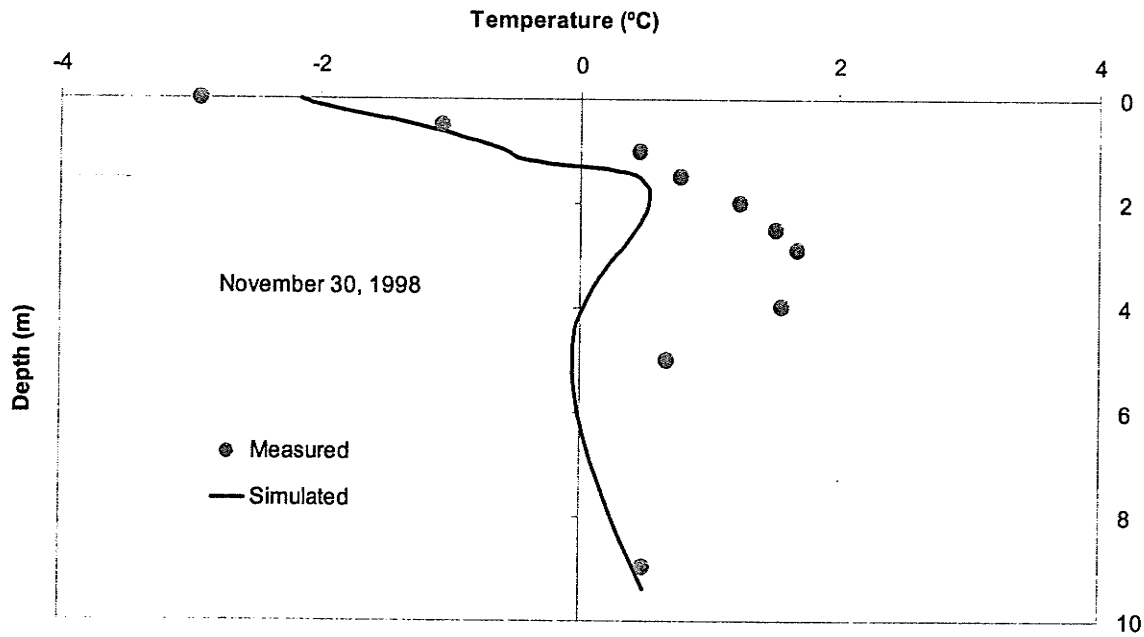

Figure 3-18: Thermal modelling calibration analysis at the toe of the embankment for November 30, 1998

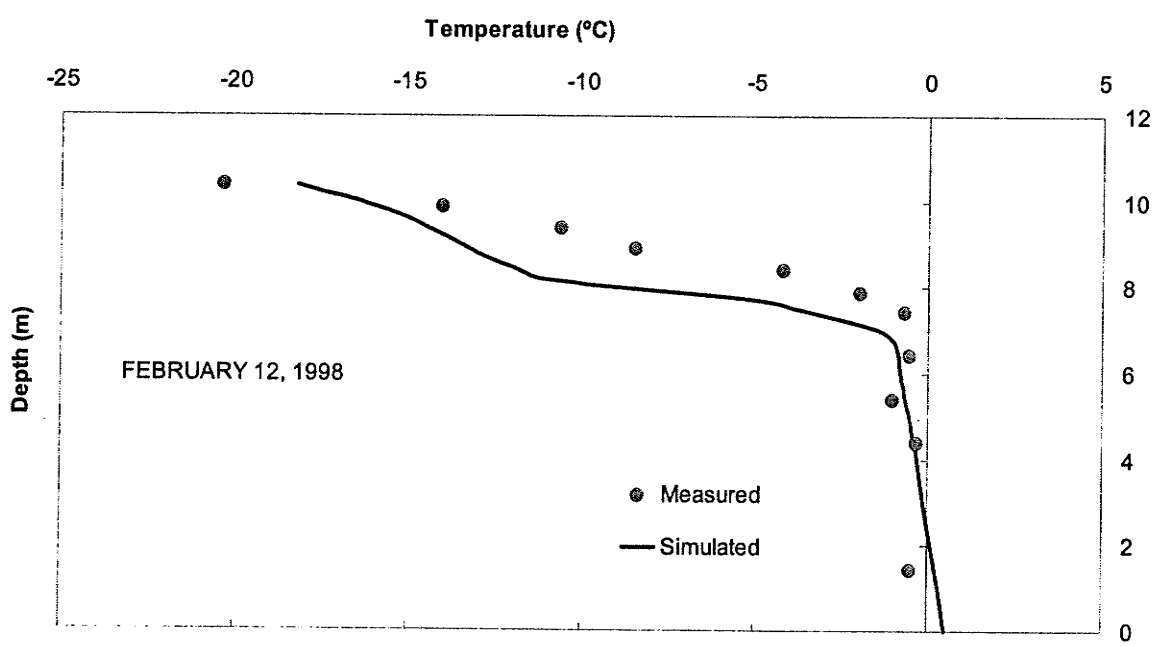

Figure 3-19: Thermal modelling calibration analysis at the centerline of the embankment for February 12, 1998

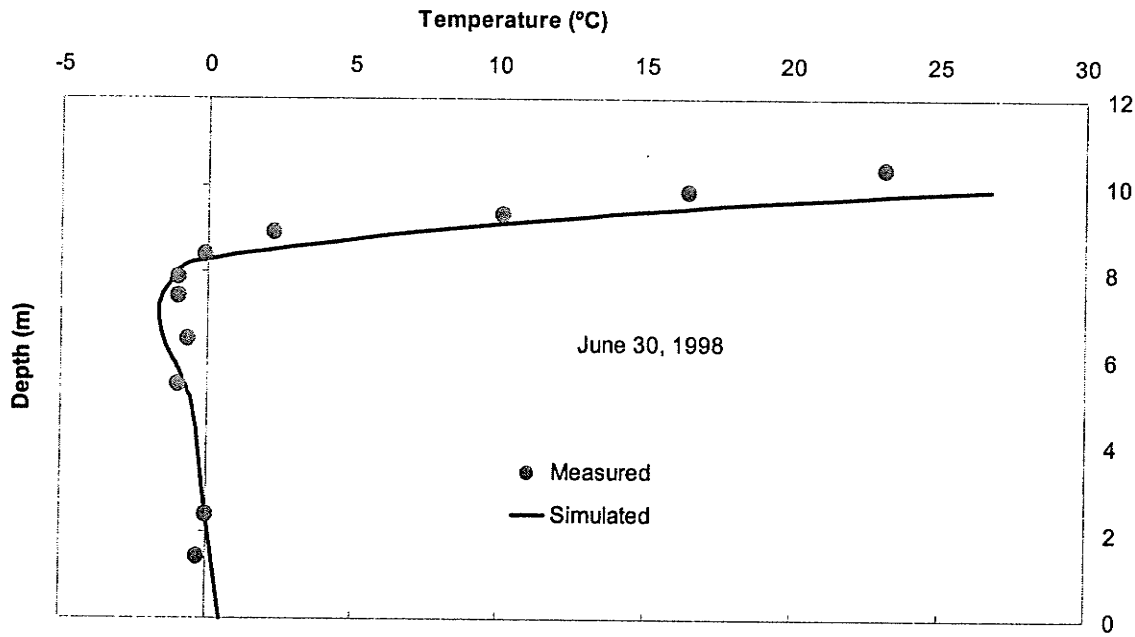

Figure 3-20: Thermal modelling calibration analysis at the centerline of the embankment for June 30, 1998

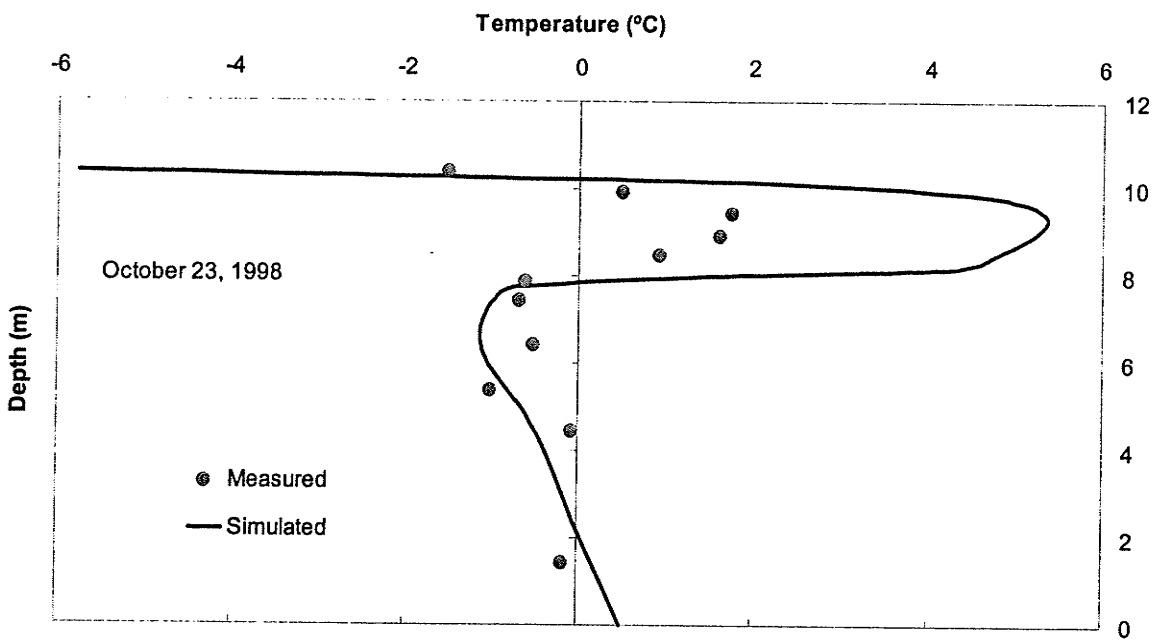

Figure 3-21: Thermal modelling calibration analysis at the centerline of the embankment for October 23, 1998

Figures 3-22 to 3-24 give the cross sections where the unfrozen and frozen zones within the foundation are obtained at various times of the year. It can be appreciated that the degradation of the permafrost begins at the toe of the embankment, which leads to loss of shoulder support causing longitudinal cracks and dips near the shoulder of the road.

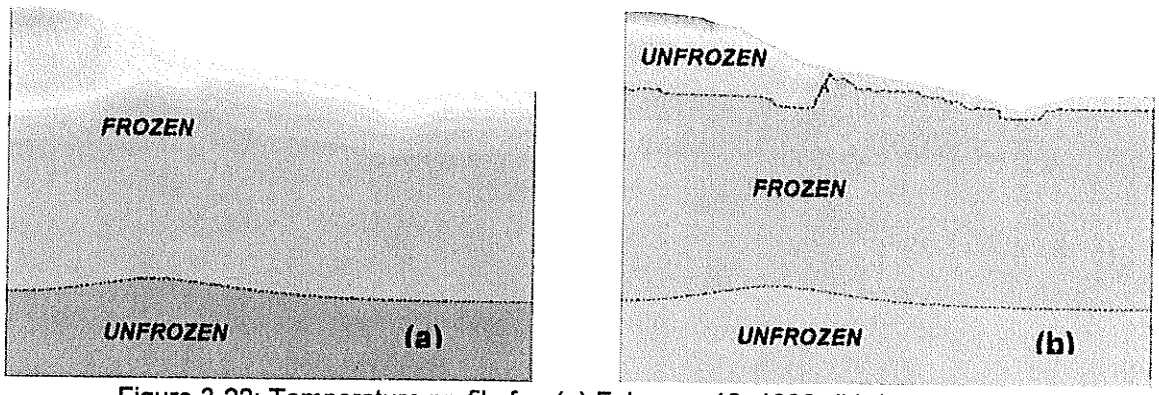

Figure 3-22: Temperature profile for: (a) February 12, 1998; (b) June 30, 1998

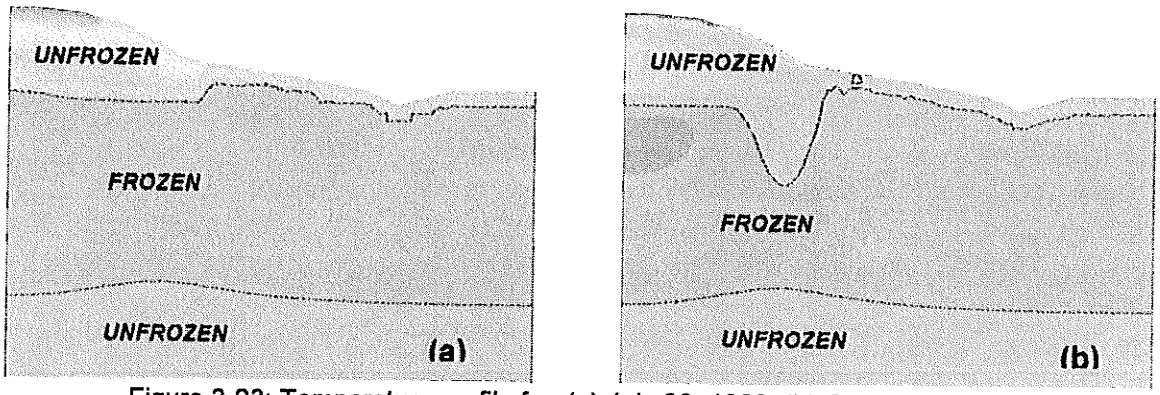

Figure 3-23: Temperature profile for: (a) July 30, 1998; (b) October 23, 1998

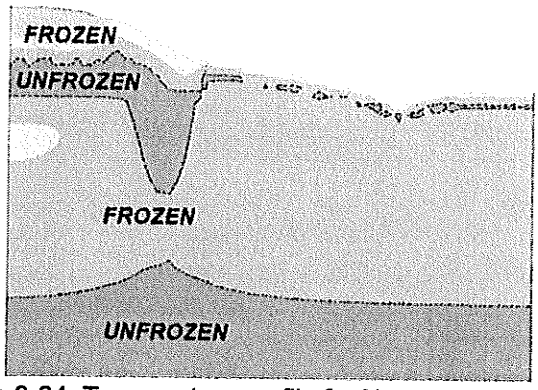

Figure 3-24: Temperature profile for November 30, 1998.

3.8.2 MODELLING CLIMATE CHANGE TREND

Climate models (CCCMA, 2005; MRI, 2005; The Hadley Centre, 2005) indicate that the Northern Regions are and will continue to be impacted heavily by climatic warming trend. Because permafrost is highly susceptible to a long-term warming, it is important to analyze the impacts and consequences of the warming trend and the construction of road embankments to ensure a safe and efficient system of transportation in the Northern Regions.

Climate scenarios are used to project how climate may change in the future. They represent future characteristics that might occur, however, they are not predictions of what will happen. The Canadian Center for Climate Modelling and Analysis (CCCMA) is a division of the Climate Research Branch of the Meteorological Service of Canada. This research center conducts coupled and atmospheric climate and sea-ice modelling, climate variability and predictability, carbon cycle and many other climate factors. CCCMA has developed a number

of climate simulations models for climate prediction, climate change and variability like the Atmospheric General Circulation Model (AGCM) versions 1, 2 and 3, and the Generation Coupled Global Climate Models (CGCM) versions 1, 2 and 3 (currently under construction) among others.

For the case of this research, the second version of the Generation Coupled Global Climate Analysis (CGCM2) was preferred because it couples the atmospheric component (AGCM2) with the ocean component and includes the effect of green house gases. At the same time the model displays different scenarios (Control, GHG+A, A2 and B1) for different combinations of green house and aerosol gases and initial conditions. Figure 3-25 shows the difference in temperature that different scenarios of the CGCM2 may produce.

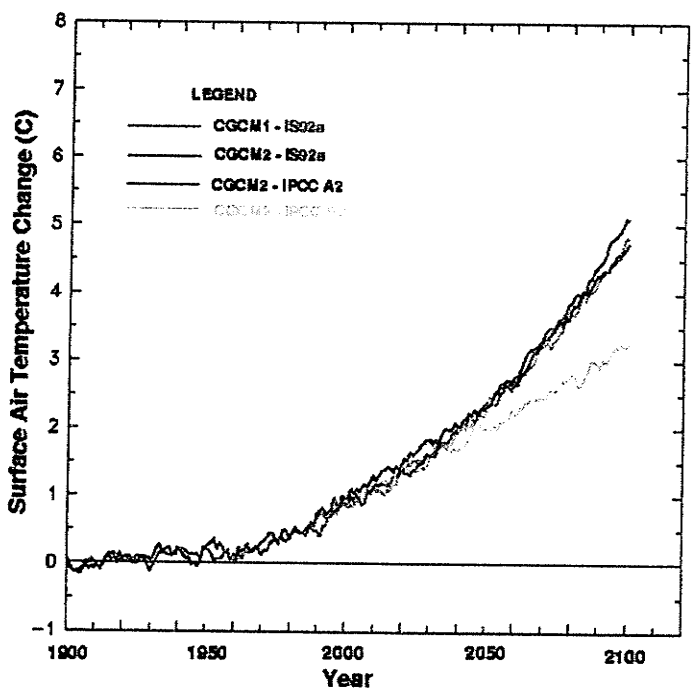

Figure 3-25: Global annual average surface temperature change, relative to 1990-1929 average as produced by CGCM1 and CGCM2 for various forcing scenarios (After CCCMA, 2005)

To be able to know what forcing scenario could better represent the site conditions, the known air temperature was compared to the modelled temperature generated by the forcing scenario. Scenario A2 reported to have the best match with the air temperature at the study site. This forcing scenario was used to run the 50 year simulation period to investigate the impacts of climate change. Figure 3-26 represents the final result obtained in the matching process.

Since scenario A2 proved to be suitable for the site being investigated, the temperature from January 2005 to December 31 2050 was downloaded and used in the simulation to further investigate the ground thermal regime below the road embankment. Figures 3-27 to 3-29 represent the chronological effects of road embankments and their foundations in the Northern Regions under global warming conditions. Basically, permafrost degradation starts at the toe of the embankment and advances laterally away towards the centerline of the model.

It can be seen that the thawing region increases dramatically after the first 10 years of modelling. Only the region below the peat and away from the embankment remains frozen because of the insulating effects of the peat layer and the fact that the construction of road embankments has limited effects in that region. Furthermore, the climatic warming trend seems to indicate that the permafrost below the embankment is not sustainable. Therefore, ground improvement techniques used in soft and compressible foundations can be used to improve the stability and reduce deformations associated with embankments in

degrading permafrost. They will be evaluated to find the most suitable strategies to make the road embankments to adapt to degrading permafrost caused by climatic warming.

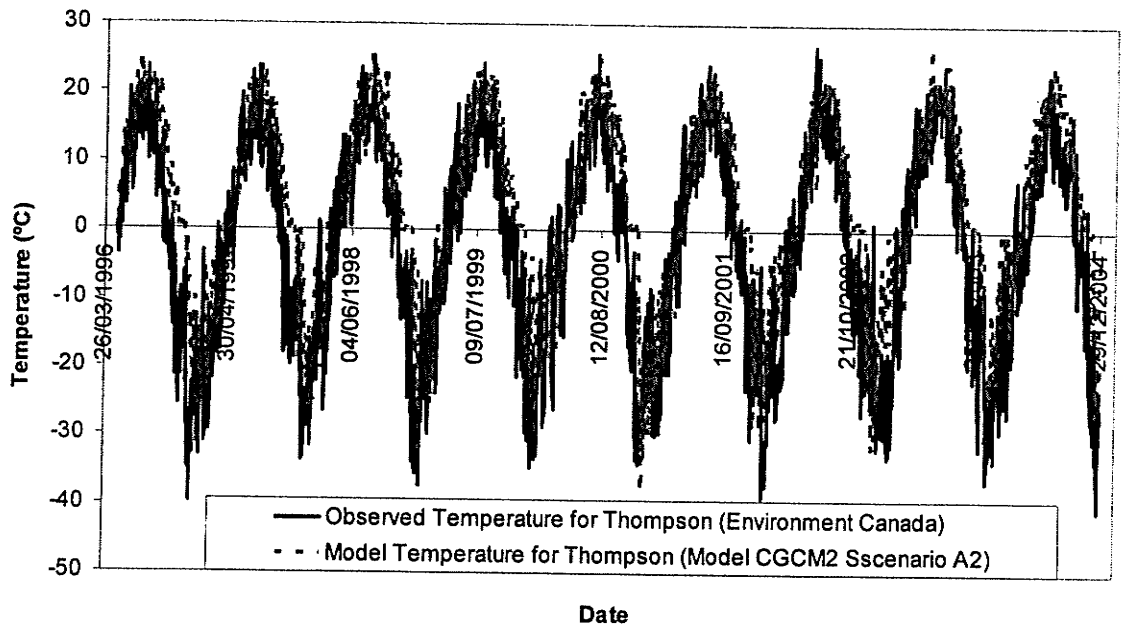

Figure 3-26: Comparison observed data to modelled data.

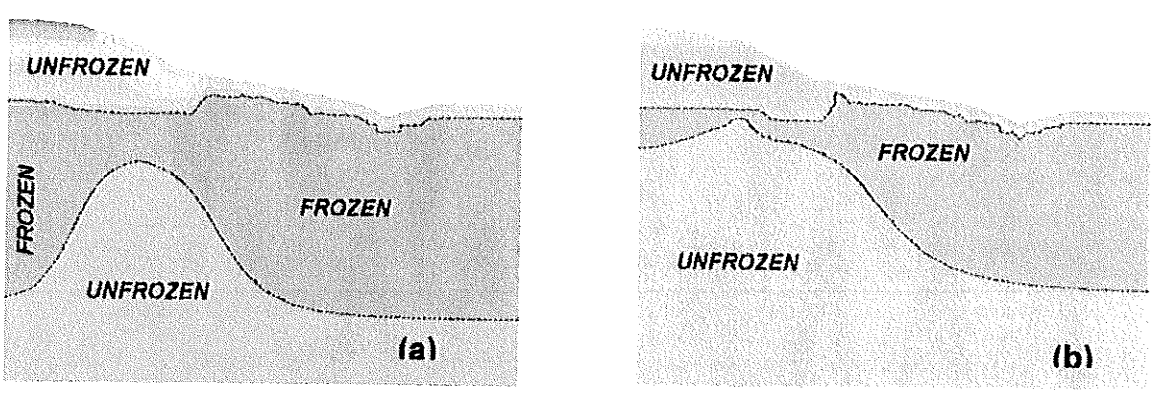

Figure 3-27: Temperature profile for: (a) July 24, 2010; (b) June 24, 2020.

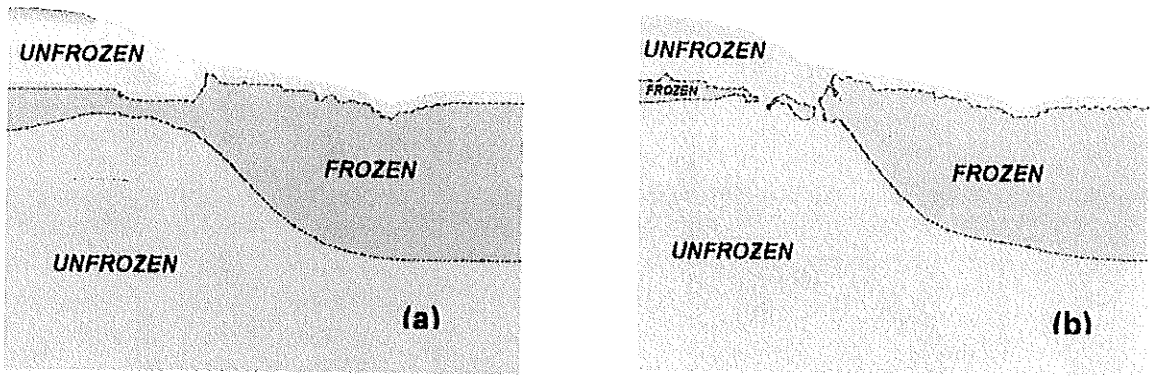

Figure 3-28: Temperature profile for: (a) July 4, 2030; (b) June 24, 2040.

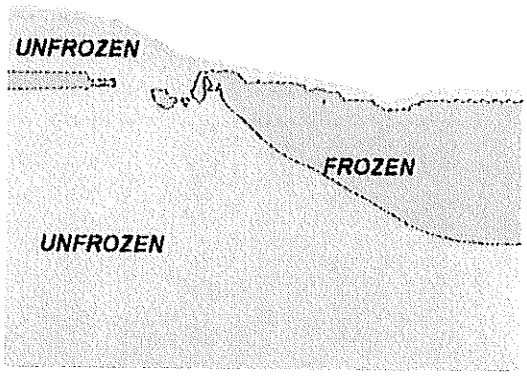

Figure 3-29: Temperature profile for June 24, 2050.

3.9 SUMMARY

The thermal model reflected the ground temperature trends reasonably well with the obtained from the thermistor data between 1996 and 1998. The simulated values tend to show a change in the temperature profile at the edge of the peat layer because of the very low thermal conductivity and high water content of the peat. Peat is considered to be a natural insulator that prevents the degradation of the permafrost.

The model indicated that the degradation of the permafrost is more severe in the lower slope areas where the thickness of the embankment is at minimum and the transmission of heat from the surface is more intense.

The climate warming trend model CGCM2, scenario A2, used in the 50 year simulation period proved to be suitable for the model since it matched the air temperature in the area reasonably well. This climate warming model has been used to study the ground thermal regime for the 50 years period of climate warming. With the climate warming trend the modelling results indicate that permafrost cannot be preserved or its presence is sporadic underneath the embankment. The degradation begins at the toe and moves outwards with degradation as well below the embankment. The permafrost is preserved in a zone far from the influence of the embankment and where the thickness of the peat layer is larger.

Peat plays a very important role in the performance of the embankments in the Northern Regions by preventing the rapid degradation of the permafrost due to the construction of road embankments coupled with climate warming trend. This combination modifies the thermal regime of the ground magnifying the degradation particularly in the discontinuous permafrost areas. Permafrost degradation increases dramatically after the first 10 years of modelling when the climate warming component is included in the analysis.

CHAPTER 4

STRESS-DEFORMATION ANALYSIS

4.1 INTRODUCTION

Evidence presented in the preceding chapter demonstrated that climatic changes will greatly affect earth surface processes that are related to permafrost conditions. The document "*Climate Change, Permafrost, and Impacts on Civil Infrastructure*" (U.S. Arctic Research Commission Permafrost Task Force Report, 2003) indicates that Alaska and Canada have population centers, pipelines and roads in areas of moderate and high hazard potential. Most of the concern is the effect of permafrost degradation in the transportation network, the communication with the Northern Regions, and the cost associated to mitigate these problems.

Building foundations, railways, road embankments among other geotechnical structures are going to be affected by the warming trend and the subsequently melting of the permafrost. The melting of the permafrost in spring and summer contributes to strength reduction of the foundation, which causes the bottom of the embankment to move outwards. In addition, the resultant generation and slow dissipation of excess pore pressures, including those from traffic loading, may lead to substantial loss of strength, decrease in bearing capacity and consequently stability problems (Instanes et al, 1998).

The results obtained in the thermal model showed that ground temperatures at the vicinity of embankment side slope are several degrees warmer than the ground temperatures at the centerline of the embankment. This occurs because only the paved surface is maintained free of snow in winter, which causes the side slopes to be insulated against refreezing in winter due to the additional snow plowed off from the surface. Therefore, 'talik' zones commonly develop and grow larger each year beneath the slopes, even if the paved or gravel covered road or runway surfaces remains well preserved (Esch, 1996).

The permafrost thawing beneath the side slopes is most severe in the lower slope areas where the thickness of the embankment material is at minimum and the transmission of heat from the surface is more intense. The effects associated with melting of permafrost beneath the slopes are that maximum settlements generally occur at that location, and exceed the settlement of the top surfaces of the embankment. This condition removes the lateral support from the sides of the pavement, and as a result, paved surfaces frequently develop longitudinal cracks and dip as the slopes settle and move outwards (Esch, 1996).

This chapter presents the results of a stress-deformation analysis. The objective is to be able to represent the response of the foundation soil when subjected to both the degradation of permafrost and the increase in pore water

pressures due to the embankment load. As will be discussed later the numerical analysis, this is only a qualitative evaluation of a physical process rather than a quantitative exercise.

The commercially-available computer software SIGMA/W (Geo-Slope International, 2004b) was used to determine the changes in stress in the soil. Results from the thermal model are used to delineate frozen and unfrozen zones and to visualize time-dependent degradation of the permafrost. The stress-deformation modelling results will be used to help evaluate the most suitable ground improvement techniques as forms of adaptation strategies to mitigate the adverse effects brought by the melting of permafrost. This will make the road embankments adaptive to the degradation of permafrost caused by climatic warming.

4.2 MODEL IDEALIZATION

It has been demonstrated that climate warming trend will melt most of the permafrost underneath the embankment. This time-dependent process is not easily analyzed because the degradation of permafrost is associated with the generation of excess pore water pressures in the clay. These excess pore water pressures are a function of the rate and magnitude of the permafrost thawing and the load applied to the clay. To be able to determine the change in magnitude of excess pore water pressures, it is necessary to simulate as closely as possible the rate at which the permafrost is degrading and the rate at which embankment

loading is applied to the foundation soil. The simulation is very complex due to the amount of time-dependent data required for the analysis and the potential to create an unstable solution associated with the incremental numerical analysis.

It was apparent that the available computer software is unable to perform changes in the size and properties of degraded permafrost while the simulation is running. To overcome this limitation, a number of separate, sequential sub-models that have different geometries of frozen and unfrozen zones were used to determine the changes in stress and deformations in the foundation soil during the thawing periods. The simulation of the thawing consolidation process is thoroughly explained in Appendix B. The analysis requires running each sub-model individually with the results of the preceding sub-model used as initial condition of the subsequent sub-model. This procedure requires removing the embankment loading at the end of the preceding sub-model because the subsequent sub-model needs 'switching-on' or application of the embankment loading to simulate the consolidation process. Application of the embankment loading is done in small increments with appropriate time duration to ensure stable solutions. Selection of this load and time increments was done through trial and error to obtain a stable solution.

To reproduce the stress history of the foundation soil in the subsequent sub-model, the stresses and deformations at the end of the preceding sub-model are imported to the subsequent sub-model. The unloading at the end of the

preceding sub-model with stresses and deformations imported to the subsequent sub-model has implications in simulating two distinct processes. First, the unloading will affect the subsequent sub-model in such a way that it will result to expansion of the foundation soil during the incremental loading. This should simulate the reduction of stresses in the soil associated with expansion of ice during freezing period. The idealization here comes from the fact that the reduction of stresses is also compensated with the increase associated with incremental loading to simulate consolidation in the thawing period (subsequent sub-model). Again, this idealization was made because of the lack of a computer software that permits full coupling of thermal and consolidation processes (fully coupled thermo-hydro-mechanical, or THM modelling).

The cross sections for the model were selected by determining the patterns of permafrost degradation as illustrated in Figures 4-1 to 4-3. The model in Figure 4-1 is used as the initial condition to calculate the stresses of the model in Figure 4-2. Then, the results from model in Figure 4-2 become the initial conditions for model in Figure 4-3 and on the subsequent models representing various thawing periods. The parameters used in the numerical simulation will be discussed in Section 4.5 of this Chapter.

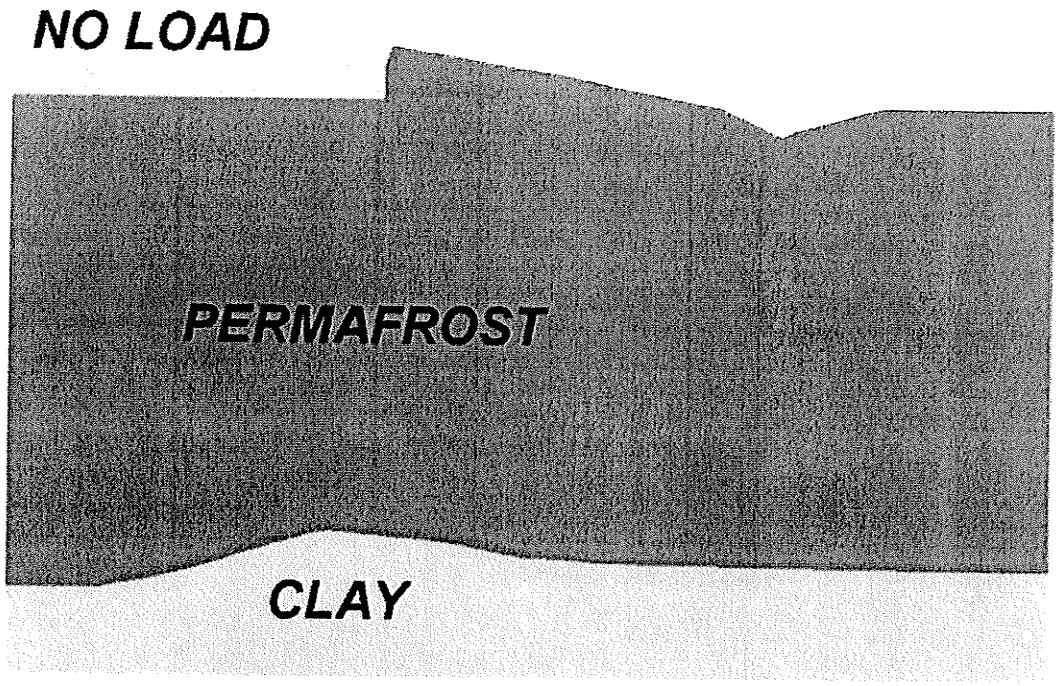

Figure 4-1: Permafrost distribution for model from January 15, 2006 to September 12-2006.

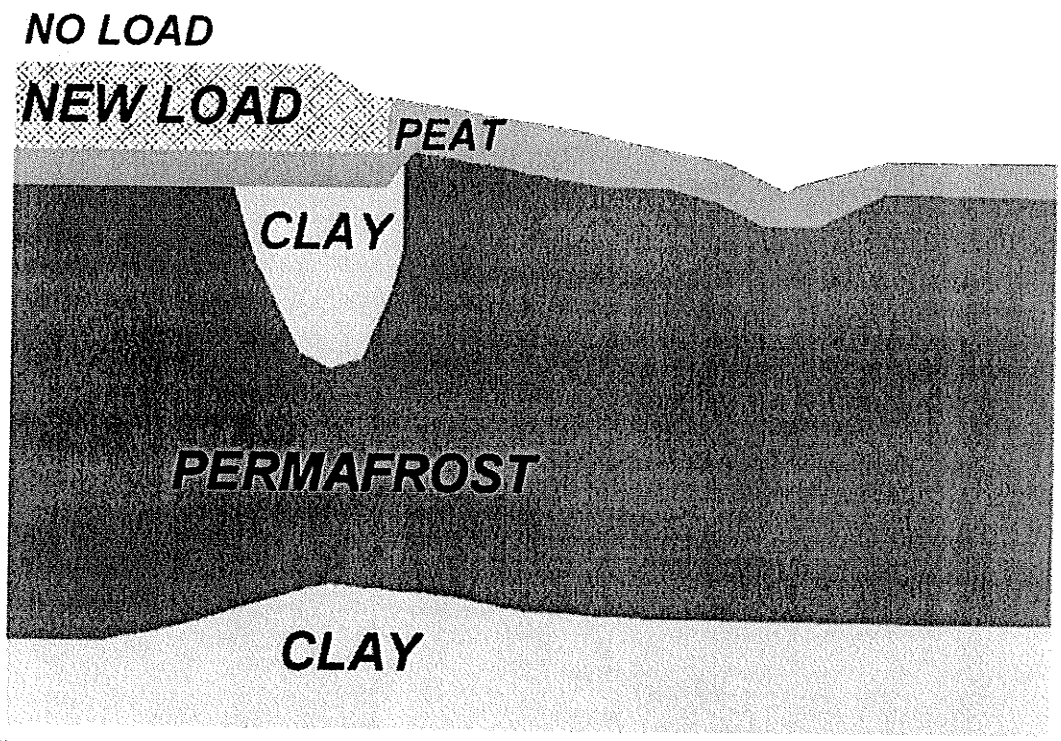

Figure 4-2: Permafrost distribution for model from September 12-2006 to January 31-2007.

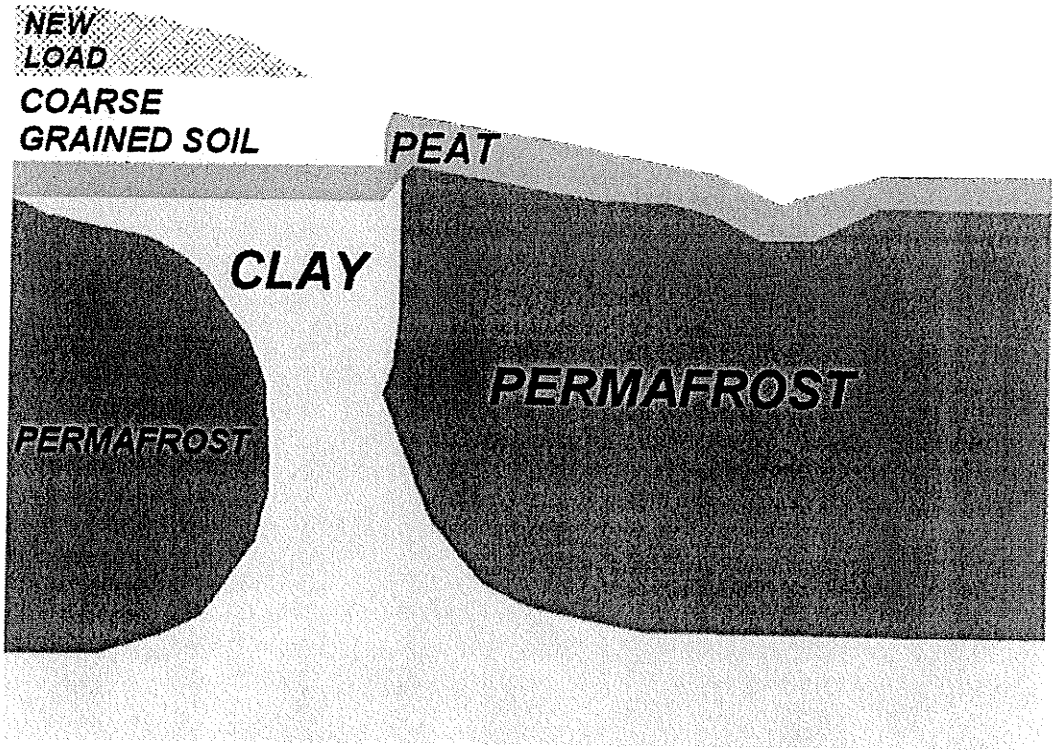

Figure 4-3: Permafrost distribution for model from January 31-2007 to April 10-2007.

4.3 GEOMETRY AND BOUNDARY CONDITIONS

The model cross-section used for this analysis is the same as used for the thermal model. Some changes have been made to incorporate the new conditions for the stress-deformation analysis and the evaluation of adaptation strategies. Symmetry is still assumed at the centerline of the embankment to maximize computing efficiency.

The changing conditions of permafrost with time make the model complex and time consuming. It needs to be divided into small models to reproduce the fluctuation of the permafrost table to get an analogous approach of the theory of

thawing soils described in Appendix B of this thesis. As explained in the model idealization (Section 2 of this Chapter) the cross sections for the model were selected by determining patterns of appreciable permafrost degradation as shown in Figures 4-1 to 4-3.

4.3.1 MODEL GEOMETRY

To represent the changing conditions of the permafrost geometry it is necessary to create a model that can be modified to represent the movement of the permafrost table, the loading and unloading of the surcharge (embankment load), and the incorporation of adaptation strategies. As previously shown in Figures 4-1 to 4-3, cross-sections of appreciable permafrost degradation were selected to determine the change in stresses in sequential models. Cross sections with similar thermal patterns (zones of frozen and unfrozen ground) were determined from the thermal modelling (Figures 4-4 to 4-7). Then, the frozen and unfrozen zones for every cross section were delineated, overlapped and grouped in templates. Figure 4-8 is the template number 1, which shows the overlapping of the frozen and unfrozen zones from Figures 4-4 to 4-7.

Several templates were generated by using the same technique as template number 1. The final mesh is then generated by overlapping the set of templates and drawing on top of the common regions. However, since the interest of this study is to propose some adaptation strategies, this needs to be included in the mesh. Vertical lines simulating rock columns and vertical drains

were also drawn on top of the resulting cross section of Figure 4-9. The permafrost has been almost completely degraded after October, 2011 is considered negligible for the stress deformation analysis and is the last cross-section used from the thermal model to delineate zones of frozen and unfrozen ground. From there, the model considers that the presence of permafrost underneath the embankment is insignificant and no more changes in foundation geometry and soil properties are needed.

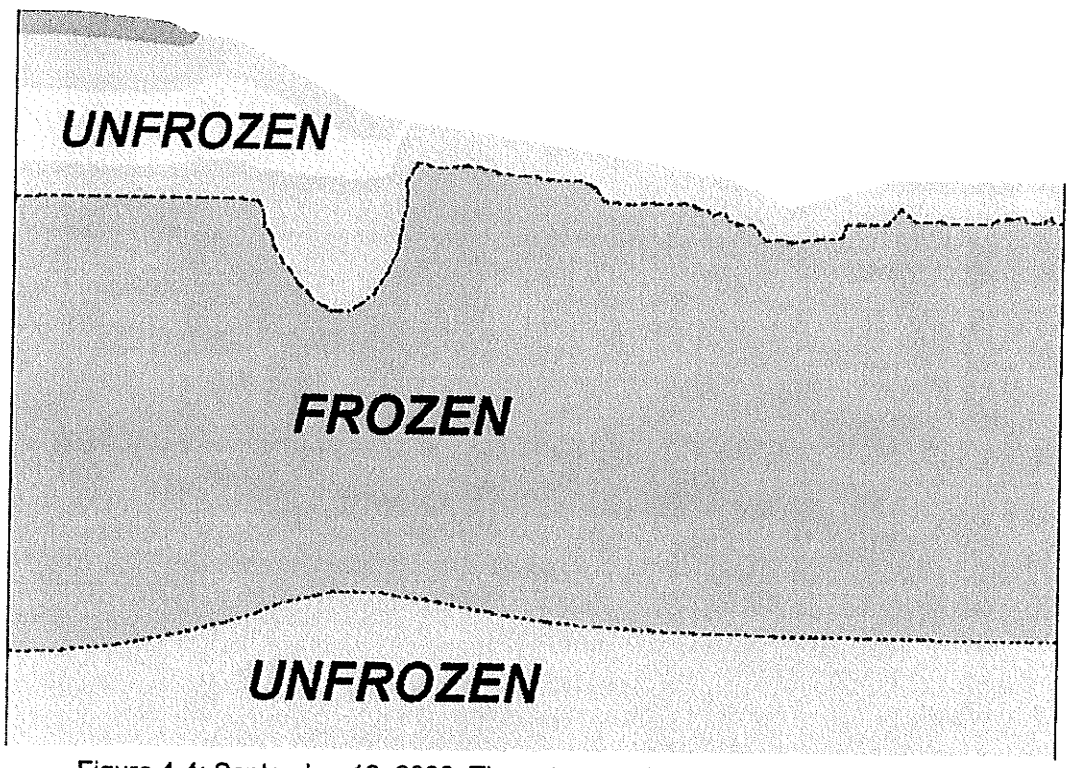

Figure 4-4: September 12, 2006. Time step number 620 from thermal model.

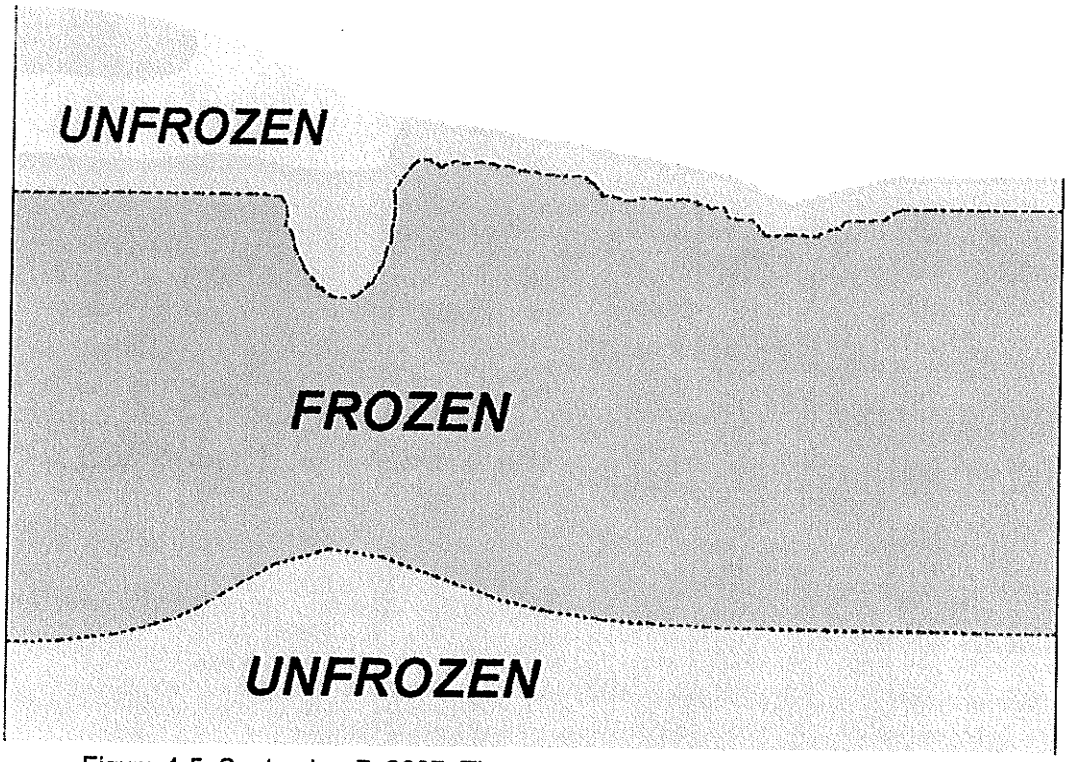

Figure 4-5: September 7, 2007. Time step number 980 from thermal model.

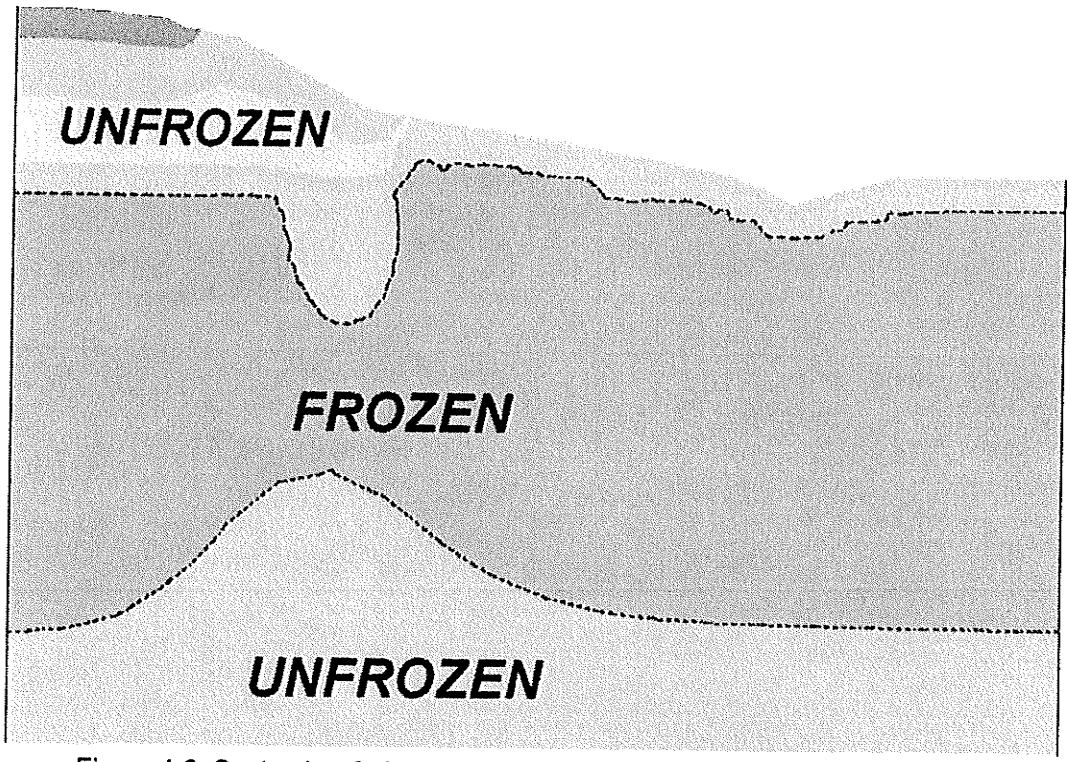

Figure 4-6: September 2, 2008. Time step number 1340 from thermal model.

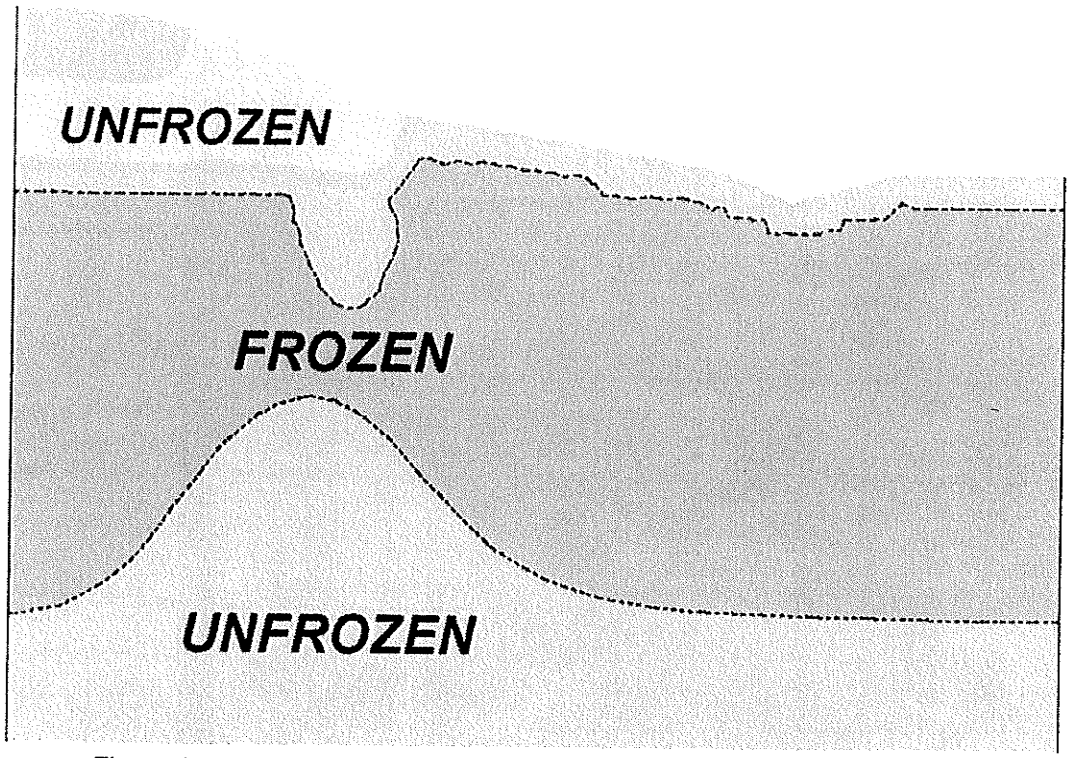

Figure 4-7: August 28, 2009. Time step number 1700 from thermal model.

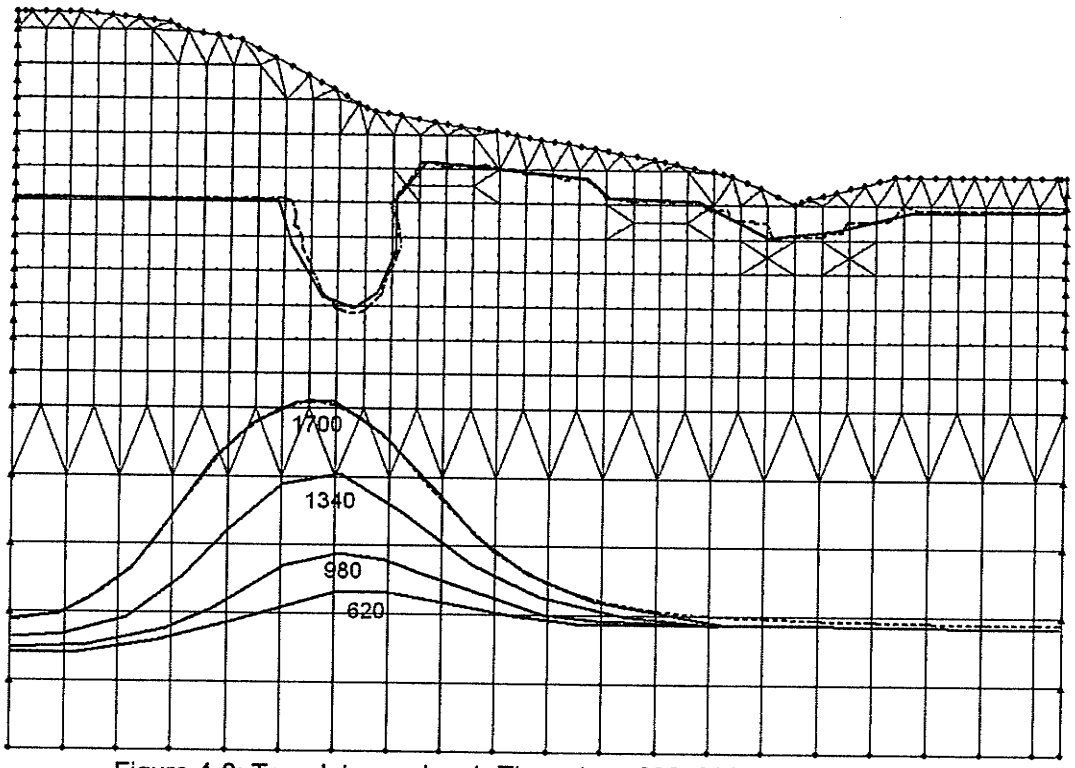

Figure 4-8: Template number 1. Time steps 620, 980, 1340, and 1700.

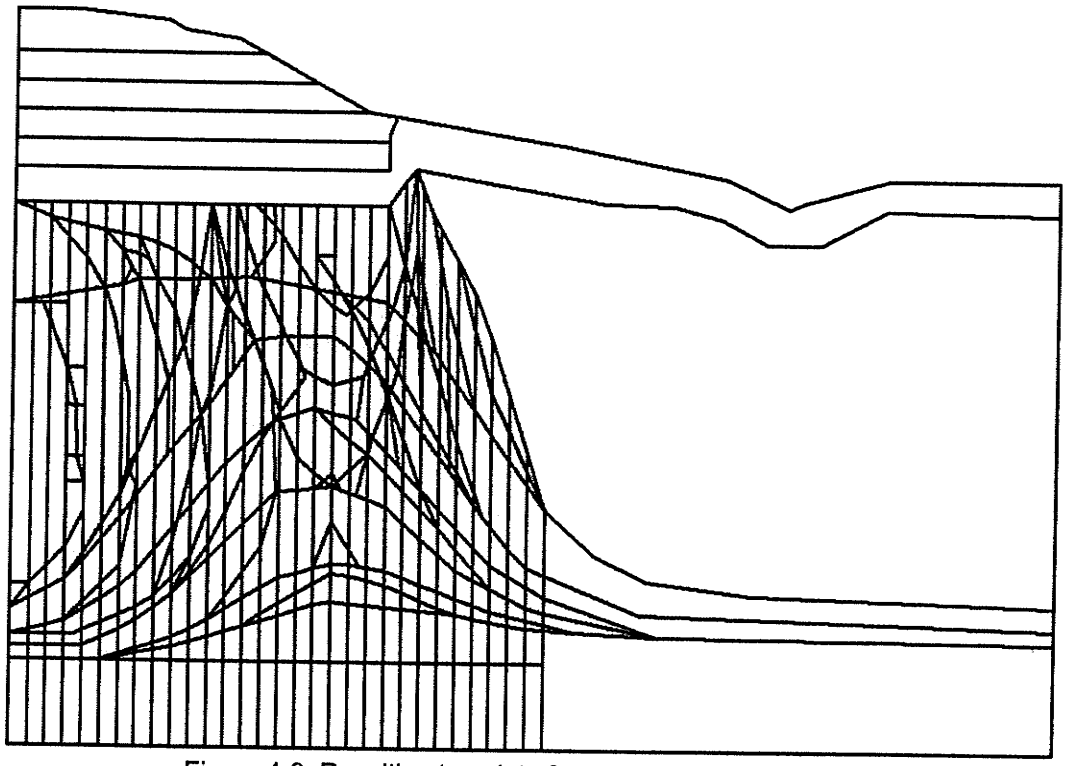

Figure 4-9: Resulting template for the finite element model

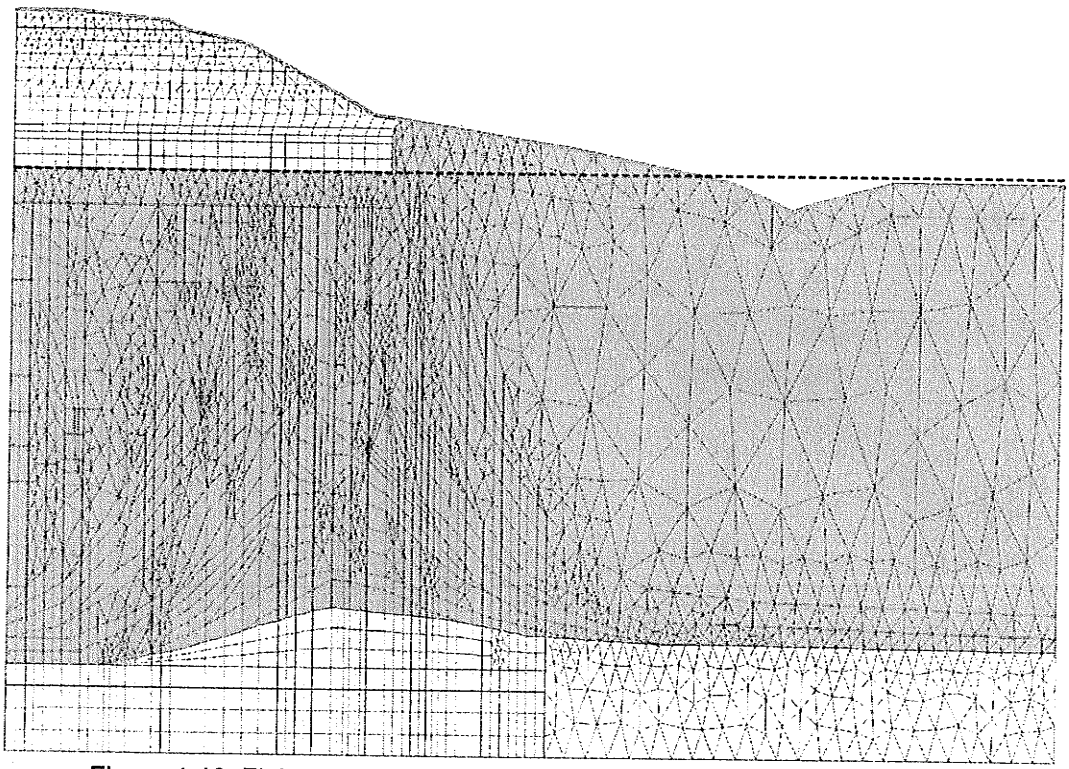

Figure 4-10: Finite element mesh used for the stress-deformation analysis.

The final cross-section of Figure 4-9 is used in SIGMA/W to build the finite element mesh that will allow modification of material properties to account for the stress-deformation analysis and the adaptation strategies to be analyzed in Chapter 5 of this thesis.

The new version of Geo-studio allows the user to determine the finite element mesh by superimposing preexisting figures. The resulting finite element mesh is shown in Figure 4-10. The mesh of the model is primarily composed of a combination of quadrilateral and triangular elements. The irregular shapes of elements are a compromise for building the same finite element mesh that satisfies all modelling exercises in this study.

4.3.2 BOUNDARY CONDITIONS

The type of approach used to determine the stresses and pore water pressures is a coupled consolidation analysis. This type of analysis requires solving groundwater conditions and deformations in the soil in a consecutive manner. SEEP/W is the module in Geo-Slope International (2004a) that is used to solve the problem of flow in porous media. On the other hand, SIGMA/W is the module used to solve the problem of stress and deformations in the soil. Two kinds of boundary conditions need to be determined: the flow boundary conditions and the deformation boundary conditions.

Flow boundary conditions

The flow model has two vertical zero-flux boundaries, an upper constant head boundary of 8.19 metres and a bottom constant head boundary of 8.09 metres. The zero-flux boundary condition at the left hand side is applicable because symmetry of the model forces the flux balance at the centerline to be zero (flux coming in and going out). At the right hand side the influence of this boundary is considered negligible. This was proven by performing a sensitivity analysis with different types of boundary conditions and realizing that the outcome in all cases leads to the same horizontal distribution of pore water pressures in the area of interest.

The value of the upper boundary condition of 8.19 metres is easily determined by the position of the water table at the surface of the soil foundation. Not much is known about the lower boundary condition. Drilling of boreholes did not report the presence of bedrock in any of the locations so it was assumed that a zero-flux boundary was deemed not applicable. An assumed value of hydraulic gradient of 10 cm seems to be adequate given the soil properties and topographic conditions. Based on that assumption, a value of constant head of 8.09 metres was specified for the bottom boundary.

The use of constant head values at the upper and bottom boundaries permit dissipation of the excess pore water pressures due to the load of the

embankment and prevent instability of the model and failure of the foundation soil.

Displacement boundary conditions

There are two ways to define boundary conditions in SIGMA/W: force boundary conditions and displacement boundary conditions. For the case of the analysis used here, only displacement boundary conditions are used. The model has restriction of movement in the horizontal direction at the centerline and restriction of movement in the vertical and horizontal directions at the bottom and right hand side of the model domain. The same boundary conditions have also been done on consolidation examples presented in the Geo-Slope International User's Manual (Geo-Slope International, 2004b).

The assumption of zero horizontal displacement at the centre line is valid because symmetry forces the summation of effects in that direction to be zero. Although at the bottom boundary there is no presence of bedrock, it is assumed that displacements are negligible at that depth given the fact that the soil layer at this level is relatively stiffer than the upper layers.

4.4 INITIAL CONDITIONS

The coupled consolidation approach used in the model requires knowing the initial stresses before placing any load on the soil foundation. The initial stresses

are calculated based on the cross-section of Figure 4-11, which corresponds to the permafrost distribution of January 1-2005, date when the model is initiated.

The type of analysis used is 'in-situ', which establishes the initial in-situ stress conditions using the submerged weight of the soil (soil unit weight minus the water unit weight). The initial pore water pressure conditions are calculated from the specified water table location. SIGMA/W calculates the submerged unit weight for the portion of the soil that is below the water table. With the submerged unit weight SIGMA/W applies the load that comes from the soil weight. The result gives the in-situ effective stresses below the water table and the total stresses above the water table.

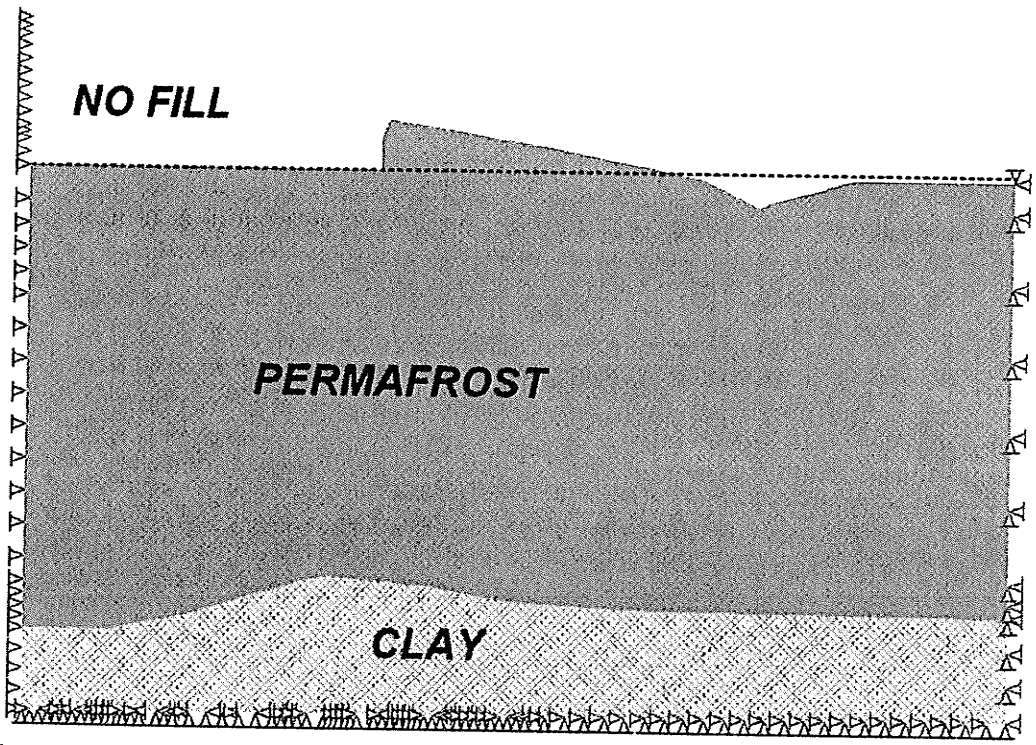

Figure 4-11: Cross section used to calculate the initial Stresses for the stress-deformation analysis.

4.4.1 SOIL MODELS USED IN THE INITIAL CONDITIONS

The constitutive soil models used in the determination of stresses for permafrost and clay layer shown in Figure 4-11 are linear elastic. It is assumed that permafrost is a very stiff soil with high strength properties and is linear elastic. In this type of model the stresses are directly related to the strains by Hooke's law:

[4-1]
$$\begin{Bmatrix} \sigma_1 \\ \sigma_2 \\ \sigma_3 \end{Bmatrix} = \frac{E}{(1+\nu)(1-2\nu)} \begin{bmatrix} 1-\nu & \nu & \nu \\ \nu & 1-\nu & \nu \\ \nu & \nu & 1-\nu \end{bmatrix} \begin{Bmatrix} \varepsilon_1 \\ \varepsilon_2 \\ \varepsilon_3 \end{Bmatrix}$$

Where:

E = elasticity modulus of the soil

ν = poisson's ratio

ε = strains in the soil

σ = stresses in the soil

The matrix in the right hand side is called the stiffness matrix. If the stresses and the parameters E and ν are known the strains can be calculated. From the strains the displacements can be calculated by using Equation [4-2].

[4-2]
$$\Delta z = \int \varepsilon_z dz$$

Because of the lack of information, the soil properties used in the analysis are taken from the literature. Atterberg's limits and borehole descriptions reported by AMEC (2002) are used as basis to determine the best set of soil properties for every soil layer. Table 4-1 contains the soil properties used to run the initial conditions (Budhu, 2000, Graham et al, 1983).

Table 4-1 Material properties for initial stresses using for linear elastic analysis

PROPERTY	Permafrost	Clay	Coarse-grained soil
Elasticity Modulus (kPa)	100000	1000	60000
Cohesion (kPa)	100	10	N.A.
Poisson's Ratio	0.3	0.35	0.3
Phi (Degrees)	N.A.	21.7	35
Hydraulic Conductivity (m/day)	8.6×10^{-6}	7.2×10^{-4}	8.64
Specific unit weight (kN/m^3)	10	18	18
K_o	0.48	0.64	N.A.

4.5 THAWING CONSOLIDATION PROCESS

The idealization of the thawing consolidation process has been discussed earlier in Section 4.2. The simulation starts on January 1, 2005 and is composed of the sequence of sub-models given in Appendix A. The in-situ stresses are determined according to the cross section shown in Figure 4-11. Once the initial stresses have been determined the model simulation can proceed and follows the loading and unloading process as described in Appendix A.

4.5.1 SOIL MODELS USED IN THE CONSOLIDATION PROCESS

Two types of soil models are used in this part of the simulation. For coarse-grained soils and permafrost (assuming it is a stiff material) the type of model is linear elastic and its formulation follows the one described in Section 4.4.1. Soil properties used for the linear-elastic model described in Table 4-1 are also used during the consolidation process.

For clay and peat the type of model selected is Modified Cam-Clay. This model is a critical state model as well as an elastic, hardening, plastic model (Geo-Slope International, 2004b). This is an effective stress model that couples effective stresses, stress paths, consolidation and shear strength of the soil. The fundamental concept is that all soils will fail on a unique failure surface.

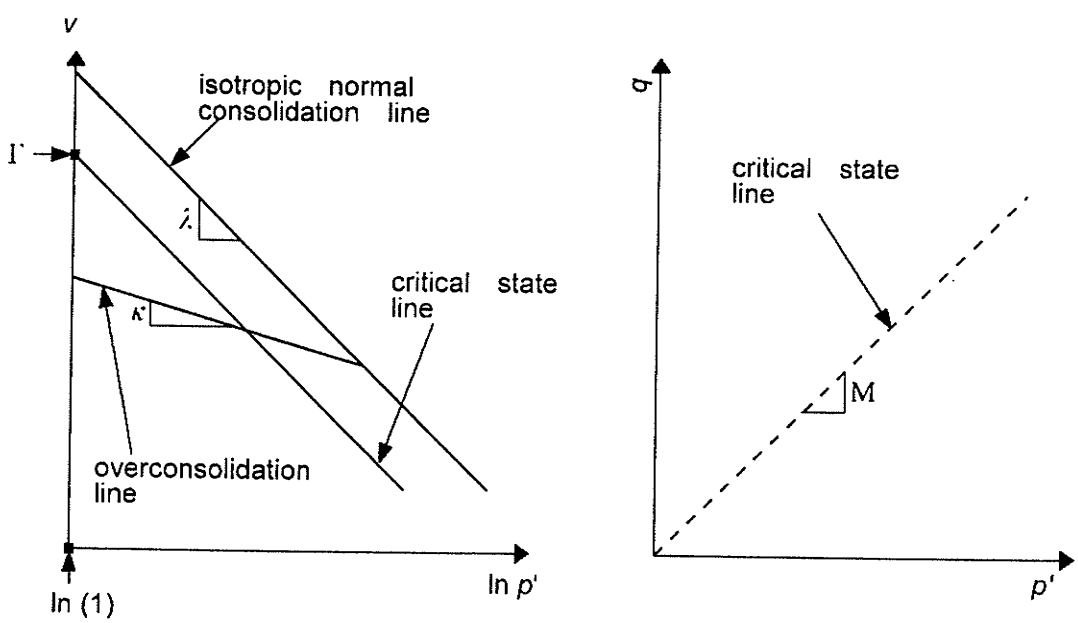

Figure 4-12: Definition of model parameters for Modified Cam-Clay model (After Geo-Slope International, 2004b)

The Modified Cam-Clay model uses the parameters: M, slope of the critical state line; λ, slope of the normal consolidation line; κ, slope of the unloading/reloading line; ν, specific volume; and Γ, specific volume at critical state when p' is 1. These parameters are described in Figure 4-12.

These parameters are calculated from results of triaxial, oedometer and/or one-dimensional consolidation tests. They are related to common soil parameters as ϕ', internal friction angle; C_c, compression index; C_r, recompression index; and, e, void ratio as follows:

[4-3] $$M = \frac{6\sin\phi'}{3-\sin\phi'}$$

[4-4] $$\lambda = \frac{C_c}{\ln(10)} = \frac{C_c}{2.3} = 0.434C_c$$

[4-5] $$\kappa = \frac{C_r}{\ln(10)} = \frac{C_r}{2.3} = 0.434C_r$$

[4-6] $$\nu = 1 + e$$

Another important parameter in the model is to determine the initial yield surface. The yield surface separates stress states that produce elastic responses from stress states that produce plastic responses. The yield surface is assumed to be an ellipse and its initial size or major axis represents the pre-consolidation

pressure, p'_c (the maximum vertical effective stress that a soil was subjected to in the past). The higher the pre-consolidation pressure, the larger the yield surface. The yield surface is also function of stresses, p', and is given by the following equation:

[4-7] $$q^2 = M^2 p' p'_c - M^2 p'^2$$

The soil parameters needed to calculate the critical stress parameters were taken from the literature and are contained in Table 4-2 (Graham, 1983; Hebib and Farrell, 2001; Rowe, 2001).

Table 4-2 Material properties for couple consolidation analysis

PROPERTY	Clay	Peat
Poisson's Ratio (ν)	0.35	0.15
Hydraulic Conductivity (m/day)	7.2×10^{-4}	7×10^{-2}
OCR	1	1
Lambda (λ)	0.305	0.3
Gamma (Γ)	3.933	9.07
M	0.668	1.07
Kappa (κ)	0.078	0.06

4.6 MODELLING RESULTS

When designing geotechnical structures two main factors need to be controlled: the stability and the serviceability of the structure (acceptable settlements/deformations) under the external loads. The change in pore water

pressure is a controlling factor in the consolidation process. The higher the dissipation of excess pore water pressures the higher the increase in effective stresses and the settlements of the structure.

It is important to know the change in stresses due to the change in permafrost conditions and the settlements associated with the consolidation process. The principal zone of analysis is at the toe of the embankment since it is the area where the thawing process starts. Results from pore water pressures, effective and shear stresses, and displacements of the area near the toe in Figure 4-13 are presented. Results from adjacent areas are also provided to verify how the stresses, pore water pressures and displacements change due to external loads.

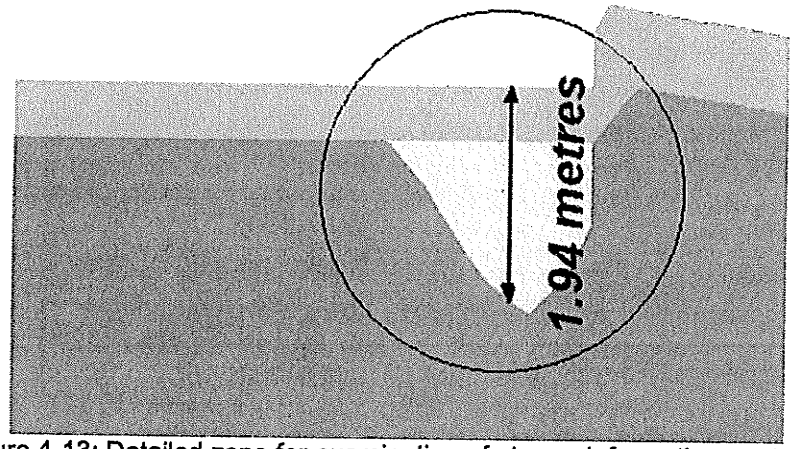

Figure 4-13: Detailed zone for examination of stress-deformation analysis.

The reason why only that area near the toe is considered in the analysis is because the stiffness of the permafrost is higher than the clay, which generates both higher effective stresses and lower displacements in the permafrost. These high stresses must be lowered when the permafrost melts, but this is something that can not be represented in the numerical model due to computer software limitations. In addition, the model is used to simulate thawing consolidation, which is only applicable in the thawing zone (refer to Appendix B of this Thesis).

4.6.1 PORE WATER PRESSURES

Excess pore water pressures are generated when the soil foundation is loaded. The excess in pore water pressures is the controlling factor in the consolidation process. Consolidation occurs when the excess pore water pressures are dissipated and the effective stresses of the soil are increased. Figure 4-14 shows the excess pore water pressures generated with depth for the area in analysis. The initial hydrostatic condition corresponds to January 1, 2005.

The freezing season from January 1, 2005 to November 26, 2005 is when no load is applied to the soil foundation. From November 26, 2005 to January 15, 2006, the foundation soil is loaded with the full weight of the embankment and excess pore water pressures are generated. This is represented by the results for December 6, 2005 as shown in the figures that are presented.

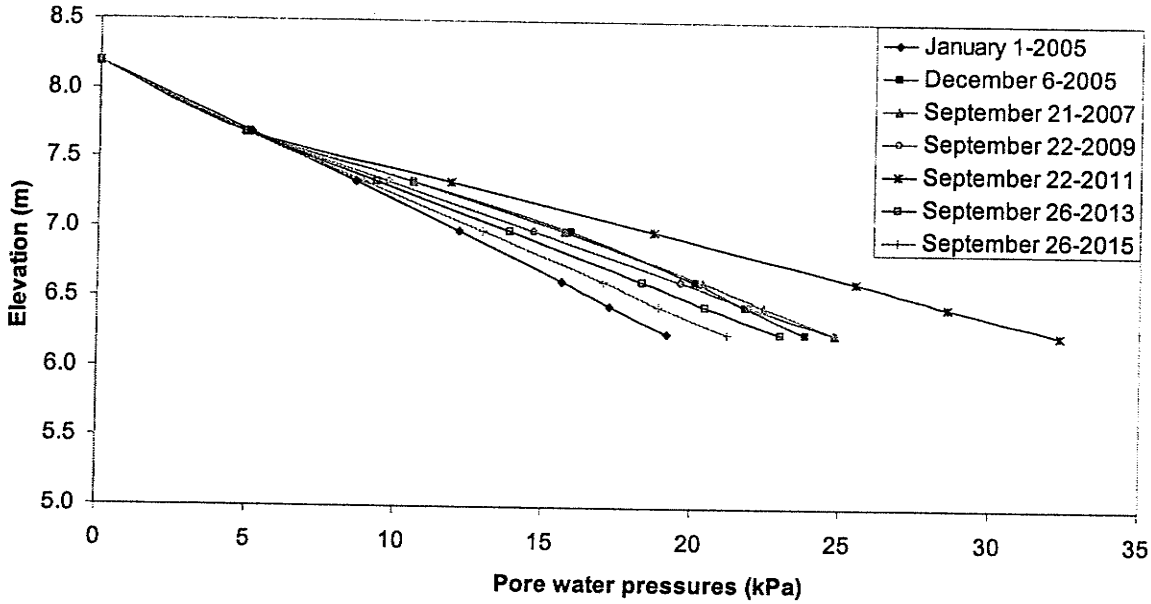

Figure 4-14: Pore water pressures versus depth.

Maximum pore water pressures are generated in 2011 when complete permafrost degradation is assumed. After loading and generation of excess pore water pressures, there is dissipation of those pressures and the values tend to revert to hydrostatic conditions.

Figure 4-15 shows the pore water pressures generated with depth for the time period of the model. High and low peaks can be observed that correspond for the periods when the embankment is loaded and unloaded. The pore water pressures are always increasing until complete degradation of permafrost is assumed and dissipation begins. Dissipation occurs because loading of the foundation soil is not longer necessary after 2011 when the permafrost is almost completely degraded.

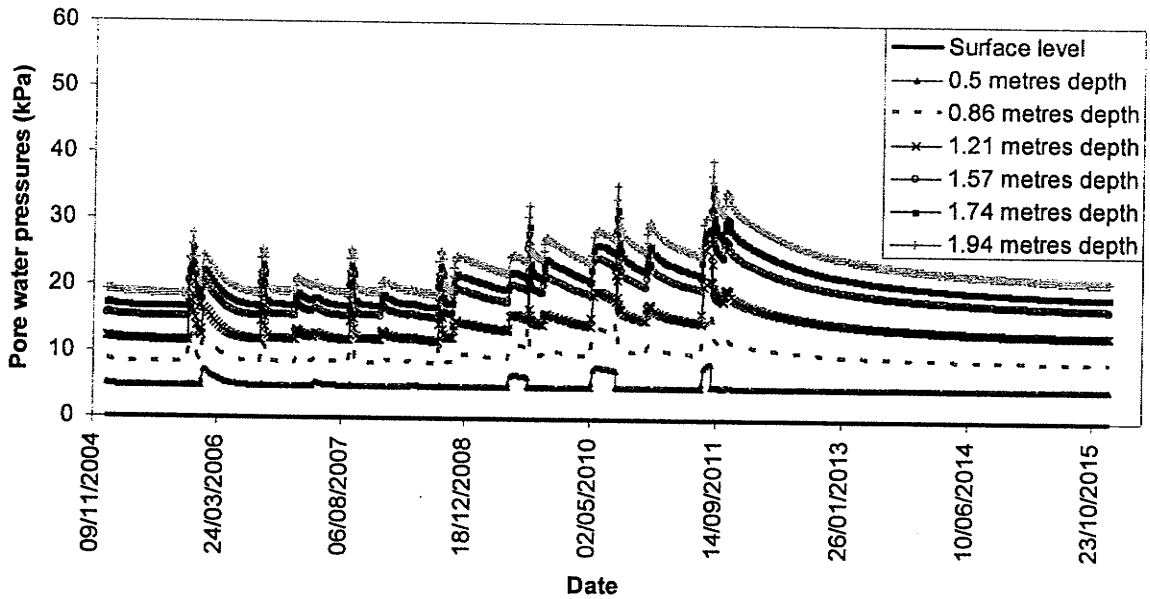

Figure 4-15: Pore water pressures versus time at different depths.

4.6.2 EFFECTIVE STRESSES

Effective stress is the stress carried by the soil particles. They are important because deformations and strength of soils depend on them. Pore water pressures cannot sustain shear forces then these forces must be supported by the soil. The changes in effective stress depend on the generation of excess pore water pressures. The higher the excess pore water pressures the lower the effective stresses. When the change in excess pore water pressures approaches to zero the change in effective stresses matches the change in total stresses and displacements would cease (Budhu, 2000). This effect is mainly controlled by the hydraulic conductivity of the soil and its capacity to dissipate the pore water pressures.

Figures 4-16 and 4-17 show the variation of vertical stresses with depth and time respectively. The variation of stresses with time and depth are considered reasonable but their magnitudes seemed to be unrealistically high. The high values of vertical stresses may be due to over simplification in the simulation of the highly coupled and complex thermal-hydraulic-mechanical processes. However, the simulation was done within the inherent limitations of the computer software used in this study.

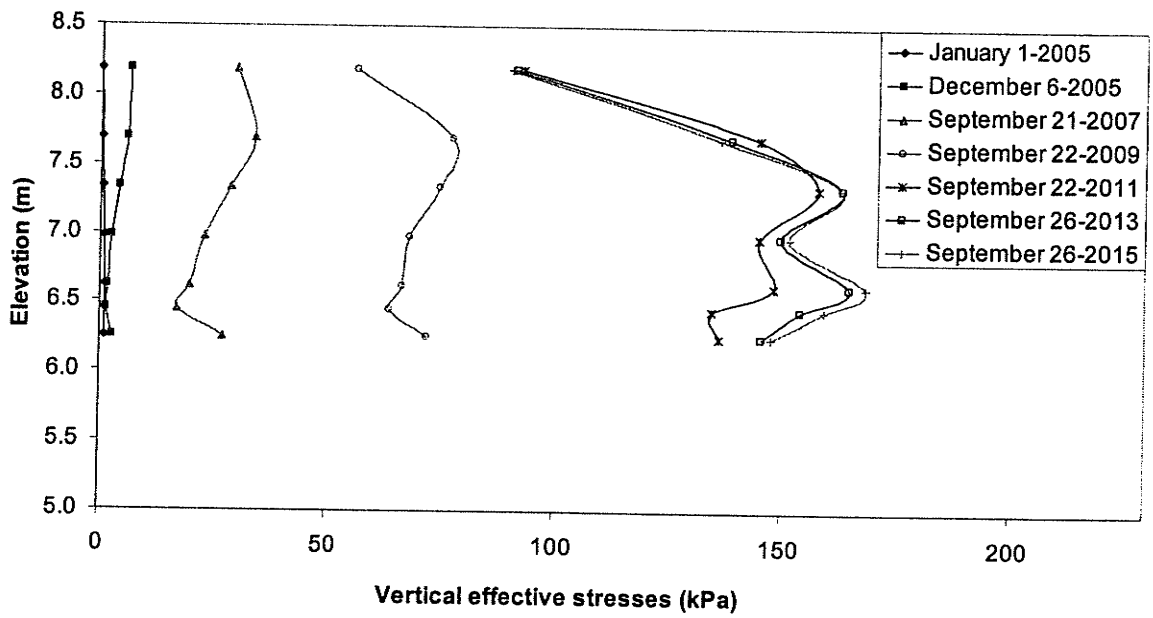

Figure 4-16: Vertical effective stresses versus depth.

As discussed earlier, the simulation was performed in a sequential manner from thermal modelling to stress-deformation and flow modelling in the foundation soil. It was decided during the course of this study to proceed in carrying out this over simplified simulation considering that the process is more

important that the prediction itself. Besides, the main objective of this study is to compare and evaluate various ground improvement techniques as forms of adaptation strategies for road embankments on permafrost affected by climate warming.

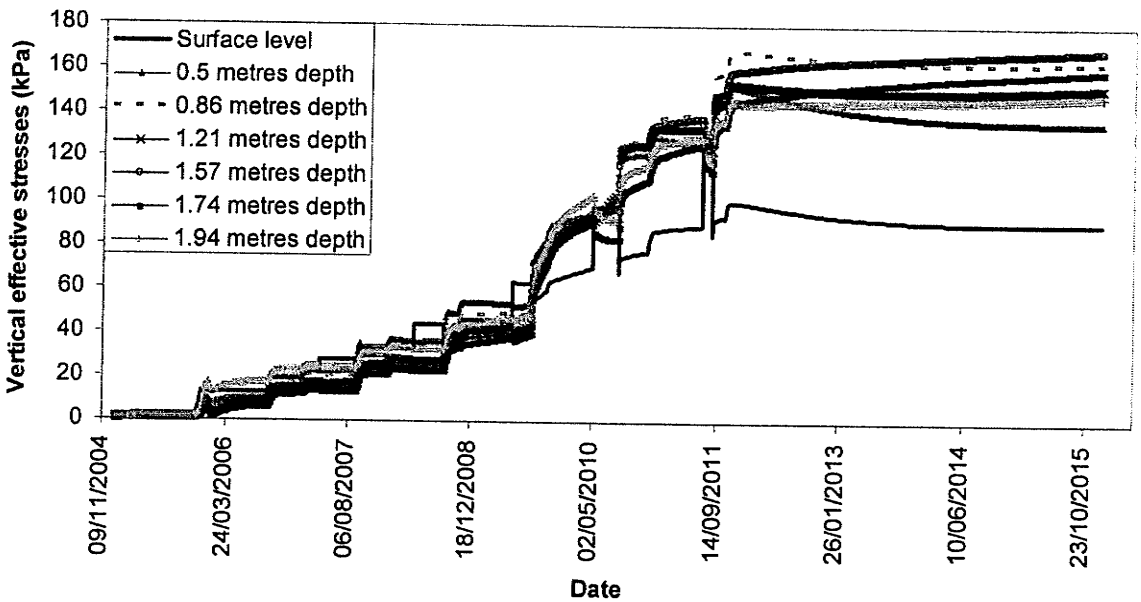

Figure 4-17: Vertical effective stresses versus time at different depths.

One of the limitations that has been found in the sequential modelling in this particular simulation exercise is the locking-in of stresses in the foundation soils during the freezing season as the embankment loading is removed. It should be noted that removal of the embankment was necessary for the subsequent simulation of consolidation process during the thawing season.

4.6.3 VERTICAL DISPLACEMENTS

Pore water pressures, effective stresses and displacements are closely related. When there is an external load (embankment loading) over the foundation soil, excess pore water pressures are generated. Dissipation of those pore water pressures increases the effective stresses, which leads to vertical displacements. Vertical displacements occur because there is a change in volume in the soil matrix due to the dissipation of excess pore water pressures.

Settlements of fine-grained soils do not occur linearly. Most of the settlements occur at the early periods when the excess pore water pressures are dissipated and their rates are much faster compared with later times. This phenomenon is called *primary consolidation* and it is basically the change in volume of the soil caused by the expulsion of water from the voids and the transfer of load from the excess pore water pressure to the soil particles. After the primary consolidation, the soil continues to settle at much lower rates because of the adjustment of the internal structure of the soil (fabric). This phenomenon is called *secondary compression* and sometimes is not very easily identified (Budhu, 2000).

It has been assumed in the analysis that consolidation and subsequently settlement at the top of the embankment has been compensated by adding fill material to the embankment. Asphalt is the material mostly used to maintain and fill dips and settlements produce on these roads in the Northern Regions as a

result of the melting permafrost. Field site investigations reported by AMEC, 2002, found fill thicknesses of up to 2.4 metres of asphalt as part of this maintenance process.

The consolidation process on degrading permafrost depends on the load applied to the soil foundation and the thawing of the permafrost. Figures 4-18, 4-19 and 4-20 show the vertical displacement occurred during the time period being analyzed. Figure 4-18 shows the settlements achieved for the three first years of modelling. Settlements are low at the beginning of the modelling due to the presence of the permafrost. However, the settlements increase with increasing in permafrost degradation.

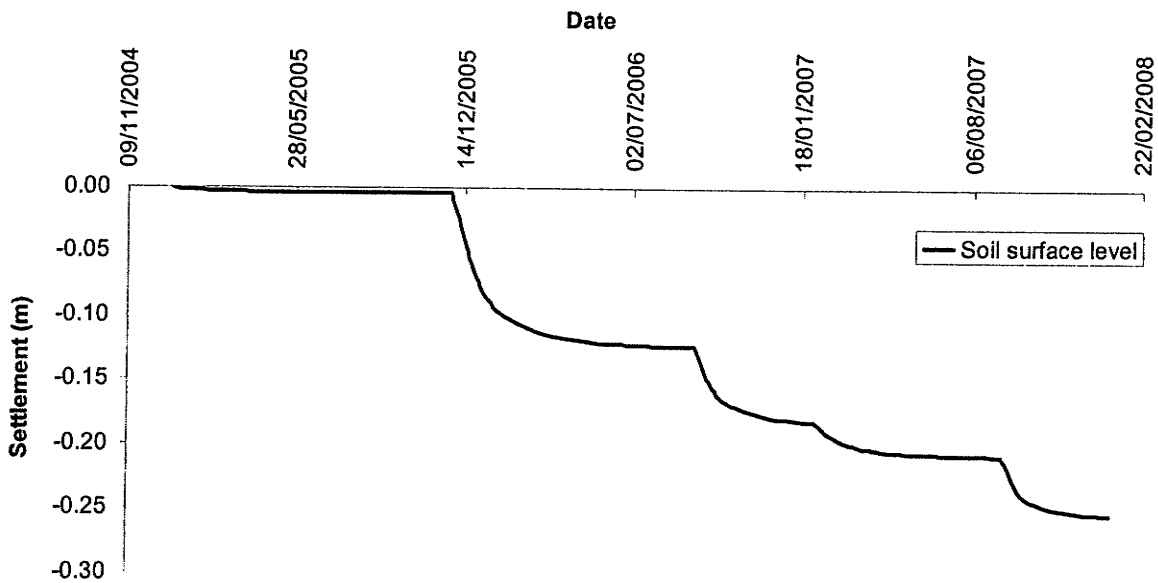

Figure 4-18: Total settlement reached for the models between January 2005 and January 2008.

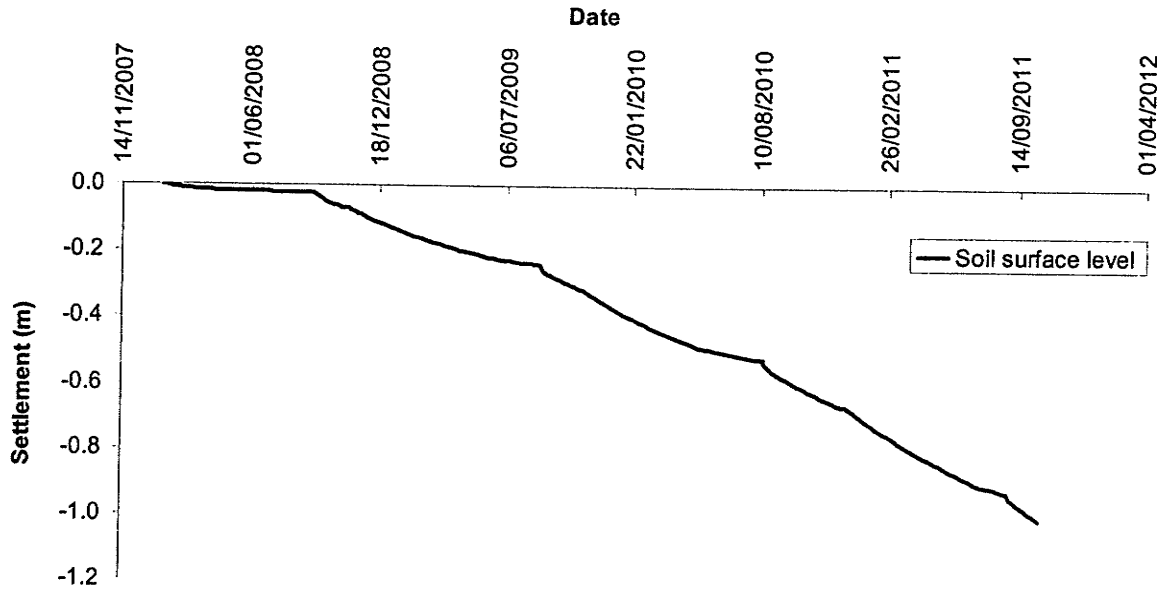

Figure 4-19: Total settlement reached for the models between January 2008 and October 2011.

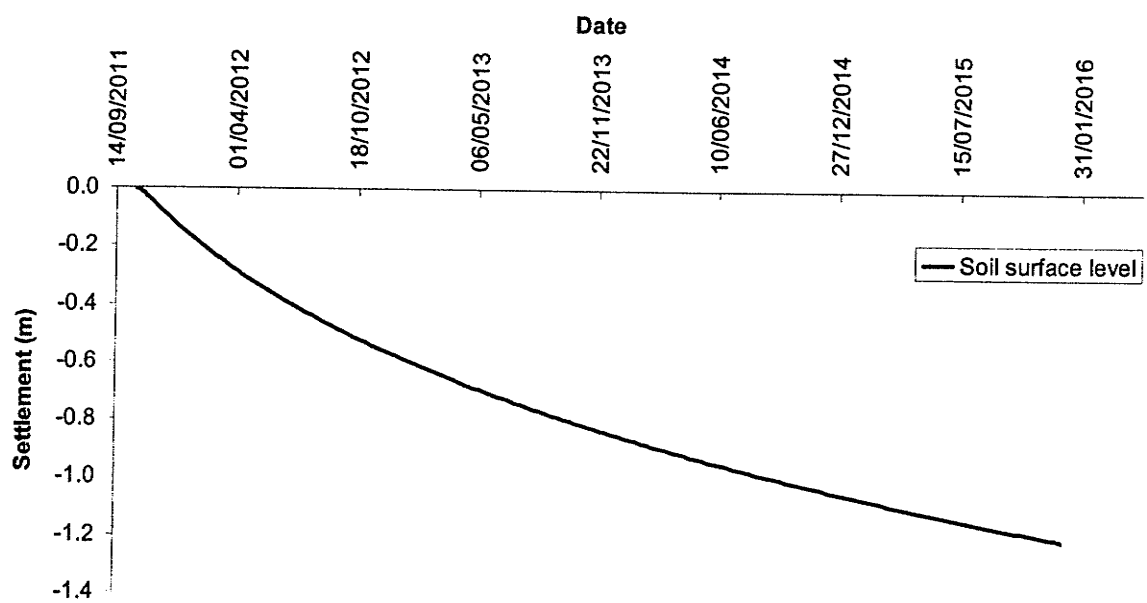

Figure 4-20: Total settlement reached for the model between October 2011 and December 2015.

Figure 4-19 shows the settlements achieved from January 2008 to October 2011, while Figure 4-20 are the results for the final five years of the

modelling. The values of settlement, even though high, match the values of 2.4 metres of asphalt used as filling material found in the field investigation of 2002 by AMEC.

4.6.4 HORIZONTAL DISPLACEMENTS

Degradation of the permafrost generally begins at the toe of the embankment due to the reduction in thickness of the fill material along the slope, which enhances the transmission of heat much faster. Further the degradation, slope movement begins being more evident at the toe. Figure 4-21 shows the horizontal displacements found at the toe of the embankment.

When degradation starts at the toe, the presence of permafrost underneath the embankment induces low displacements to the right. Later, when permafrost starts degrading underneath the embankment, the displacements increase in value and tend to move to the left since the movement to the right is contained by the permafrost outside the footprint of the embankment. The permafrost is not degraded in that area due to the presence of the peat layer that acts as natural insulator. This type of movement to the left produces deformations that induce failure mechanism like longitudinal cracking and shoulder rotation in the road embankment (Figure 4-22).

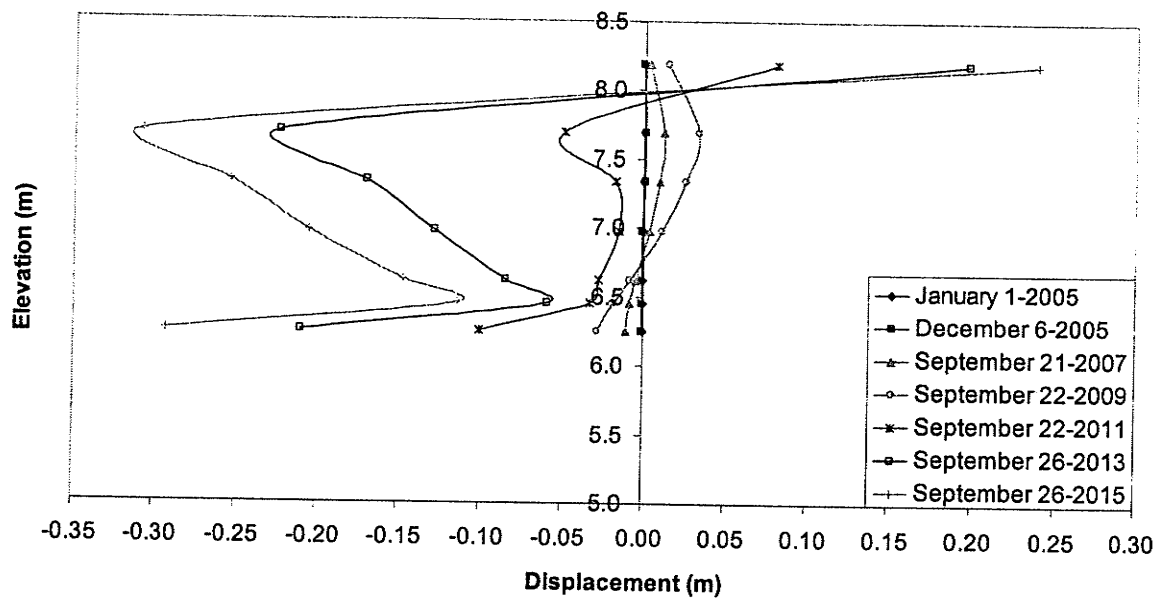

Figure 4-21: Maximum horizontal displacements at the toe of the embankment.

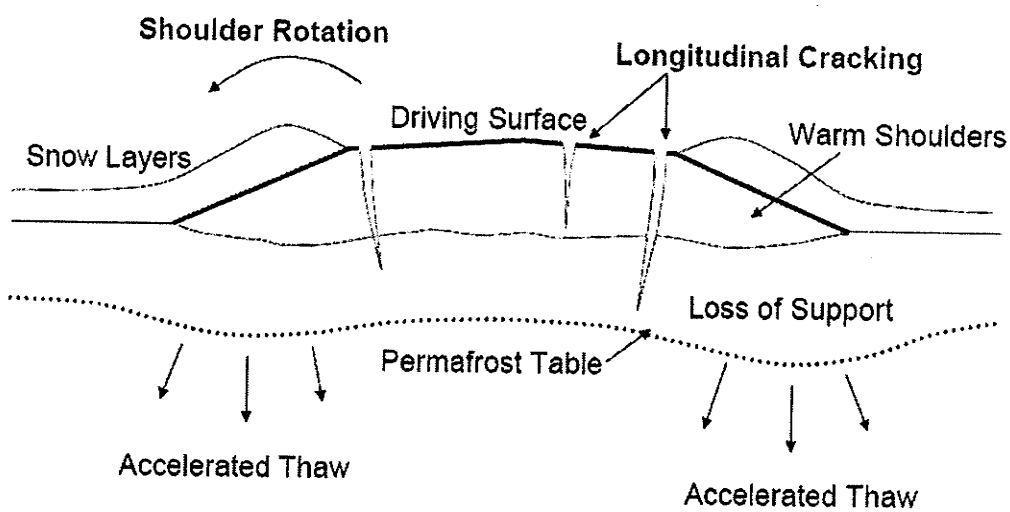

Figure 4-22: Failure mechanism induced by loss of support produced by permafrost melting (After Goering, 2005).

4.7 SUMMARY

The frozen and unfrozen regions predicted in the thermal model for the 50 year period with climate warming conditions were used to build a finite element mesh to predict the change in stresses in the foundation soil due to the construction of road embankments and the degradation of the permafrost. The main objective was to resemble the consolidation process in thawing soils. The ideal theory couples the thermal component and the classic theory of consolidation. Since the software used (Geo-Slope International, 2004b) does not couple the thermal properties with the mechanical properties the challenge was to find a way to incorporate those two components using the available computer software. The finite element mesh was built by overlapping regions of unfrozen and frozen ground. The model simulation was achieved by changing soil properties and controlling convergence of the results due to the high sensitivity of the model to the change in soil properties and geometries.

Even though the model neglects some changes in stresses because the exact transient conditions cannot be represented, the model idealization permits to account for the consolidation process in thawing soils due to the degradation of permafrost. The results reflect a process rather than prediction of the changes in stresses in the foundation soil.

Complete degradation of the permafrost was assumed in 2011 due to the combination of the construction of road embankments and the influence of

climate warming. This means that the influence of permafrost, even though present, is negligible for the analysis. The zone of permafrost at the toe of the embankment is not large compared to the beginning of the model and the change in stresses is assumed to be not appreciable.

The principal zone of analysis is at the toe of the embankment since it is the area where the thawing process starts. Adjacent areas are not considered in the analysis because the higher stiffness of the permafrost generates higher effective stresses and lower displacements. The stresses must be lowered and the displacements increased when the permafrost melts, but this is something that cannot be represented in the numerical model due to the way in which the model is set up. In addition, the model is used to simulate the theory of consolidation in thawing soils, which is only applicable in the thawing zone.

The model reflected the freeze and thawing seasons very well. During thawing excess pore water pressures are generated, high vertical effective stresses are developed, and large horizontal and vertical displacements are found. During freezing, the excess pore water pressures are lowered, the stresses remain constant, and the vertical and horizontal displacements diminished. Pore water pressures increase with degradation of the permafrost. Maximum values are reached when complete degradation of the permafrost is assumed. After this stage, excess pore water pressures start dissipating.

Similar behaviour is found with the effective vertical stresses, they increase with increase in permafrost degradation reaching constant values when complete thawing is assumed. The increase is higher at the surface; however, with time the effective stresses increase with depth.

Vertical displacements also increase with increase in the degradation of the permafrost. Thickness of asphalt to fill dips close to 2.4 metres was found in the site inspection of 2002 (AMEC, 2002).

The horizontal displacements at the toe of the embankment are restrained to the right by the permafrost; however, small displacements are found at the early stages of the modelling. With degradation of permafrost the displacements increase to the left of the toe where deformations are not restricted due to growing of thawing permafrost in that direction.

CHAPTER 5

EVALUATION OF GROUND IMPROVEMENT TECHNIQUES AS ADAPTATION STRATEGIES

5.1 INTRODUCTION

Many attempts have been made to develop efficient and cost saving strategies to mitigate and prevent the instability and deformation problems associated with the construction of embankments over degrading permafrost. Many of them are related with improving the insulating capabilities of the soils to preserve the permafrost, while others have focused in improving the mechanical properties of foundation soils.

Esch (1996) provides chronological information about some of the design and construction techniques that have been used to avoid or minimize problems, which may result form the construction and operation of roads embankments on degrading permafrost: reflective surfaces and paint coatings (1963); synthetic or polystyrene foam (geofoam) insulation (1969); peat underlays (1973); embankment lateral berms (1974); air cooling ducts (1974); pre-thawing before construction (1980); construction timing (1983); surface coverings (1984); thermosyphons (1985); geosynthetic reinforcement (1985); lightweight fill materials (1987); foundation bridge construction (1992); air convection

embankments (1995). The objectives of these techniques can be divided into two categories: 1) preserving permafrost by preventing heat penetration in embankments and extracting heat from embankments; and 2) adapting embankments to unstable degrading permafrost ground improvement (modification) techniques. The second category is the focus of this study following the premise that climatic warming can result in difficulties in maintaining freezing temperatures in the ground (*Esch and Osterkamp, 1990; Smith, 1990*). Engineers should be aware that the future presence and thermal stability of permafrost cannot be assured.

This chapter presents the results of numerical analyses developed to determine stresses and deformations of soils when ground improvement techniques are used to prevent and mitigate the effects of degrading discontinuous permafrost. Soil properties, cross section and mesh distribution are the same as those used in the previous chapter. The same sequence of models is applicable for all the proposed adaptation strategies to obtain reasonable comparison between ground improvement techniques.

The feasibility of using these techniques as adaptation strategies for road embankments affected by climate change is evaluated. The evaluation is restricted to the following techniques: lightweight fill materials, columnar inclusions such as stone columns (also known locally as rockfill columns), basal reinforcements, and sheet-piles. These techniques are selected because they

have proved to increase the bearing capacity of the soil (stone columns), reduce the stresses applied to it (lightweight fill materials), or restrain the movements that induce failure in the soil (basal reinforcement and sheet-piles).

5.2 LIGHTWEIGHT FILL MATERIALS

Placement of lightweight fill materials reduces the stresses applied to the foundation soil, as densities of these materials are significantly less than the compacted granular materials normally used as fill materials. Lighter fill materials result in smaller values of excess pore water pressures and smaller displacements.

The lightweight materials can be in the form of synthetic polystyrene or geofoam (Horvath, 1995) or the ones used in Japan which are made up of mixing a rock-like shale and concrete to obtain specific unit weights less than a conventional fill material (Miki, 2005). Those materials reduce the values of specific unit weights to almost half those of conventional soil fill materials.

5.2.1 MATERIAL PROPERTIES

Aside from their inherent lightweight, geofoam also have insulation capabilities that might prevent the degradation of the permafrost, while improving the bearing capacity of the soils; however, for the case of the analysis the influence of the insulating capabilities is neglected to represent a worse case

scenario. Table 5-1 shows the material properties of one of the products available in the market compared to conventional soil fill materials, which are coarse-grained soils. The model presented in chapter four is used as base to run the simulations for the adaptation strategies. The model allows changing soil properties to account for the ground improvement techniques analysed in this chapter.

Table 5-1 Comparison lightweight fill material to conventional fill material

MATERIAL	Dry loose density (kN/m^3)	Dry compacted density (kN/m^3)	Strength (Degrees)
Norlite Corporation	6.3	7.2	42 - 53
Coarse-Grained soils	14	23	36 - 42

For the case of lightweight fill materials, the specific unit weight of the embankment (coarse-grained soil) is replaced by the specific unit weight of the lightweight fill material. This will allow accounting for the reduction of forces in the foundation soil produced by the lighter embankment. Specific unit weight is the only property to modify. The remaining properties are constant to only account for the weight of the fill material over the foundation soil.

Even though strength and elasticity modulus of the lightweight fill material may be higher than the normal values of the conventional fill material, they are kept constant to omit reduction of forces from the embankment. Permafrost, clay

and peat properties remain constant too. Table 5-2 provides the lightweight fill material properties used to run the simulation.

Table 5-2 Material properties for lightweight fill material

PROPERTY	Lightweight fill material
Elasticity Modulus (kPa)	60000
Cohesion (kPa)	50
Poisson's Ratio	0.3
Phi (degrees)	35
Hydraulic Conductivity (m/day)	8.64
Specific Unit Weight (kN/m^3)	10
K_o	N.A.

5.2.2 PORE WATER PRESSURES

Pore water pressures generated from the placement of the lightweight fill material are shown in Figure 5-1. The analysis corresponds to the same points selected to visualize the pore water pressures in the previous modelling exercises performed in Chapter 4 (Figure 4-14). Lower values of excess pore water pressures are generated due to the lower specific unit weight of the lightweight fill materials relative to conventional coarse-grained soils.

Figure 5-2 shows the pore water pressures generated with time. As in the analysis from the previous Chapter (Figure 4-15), high and low peaks of pore water pressures are generated resulting from loading and unloading of the

foundation soil. However, the values are lower than for the case of coarse-grained soils.

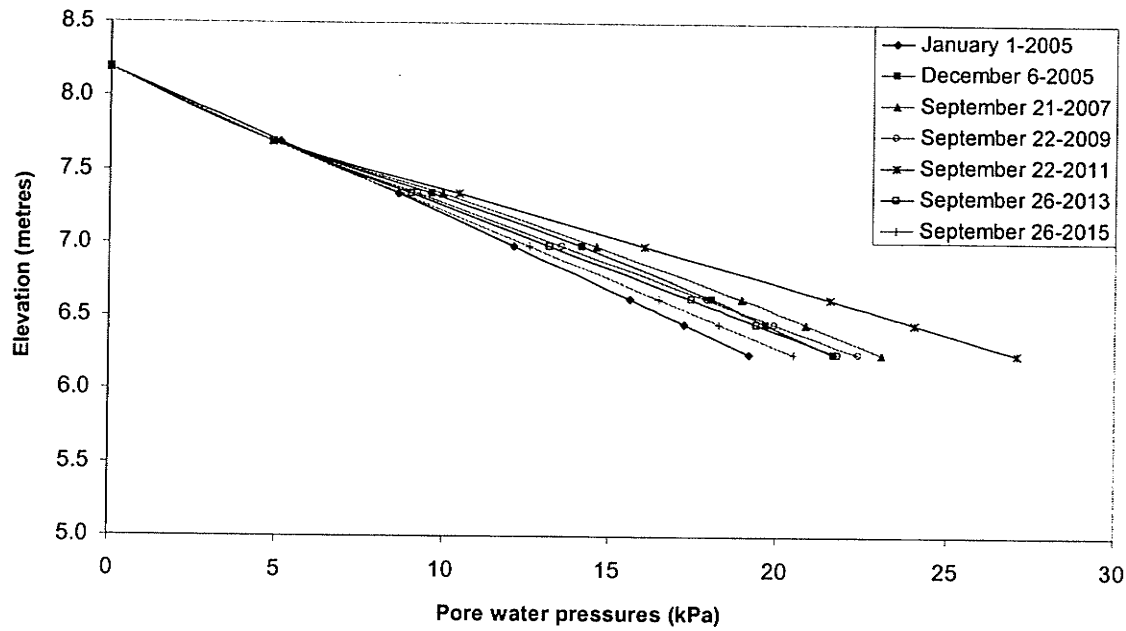

Figure 5-1: Pore water pressures versus depth generated by placement of lightweight fill materials

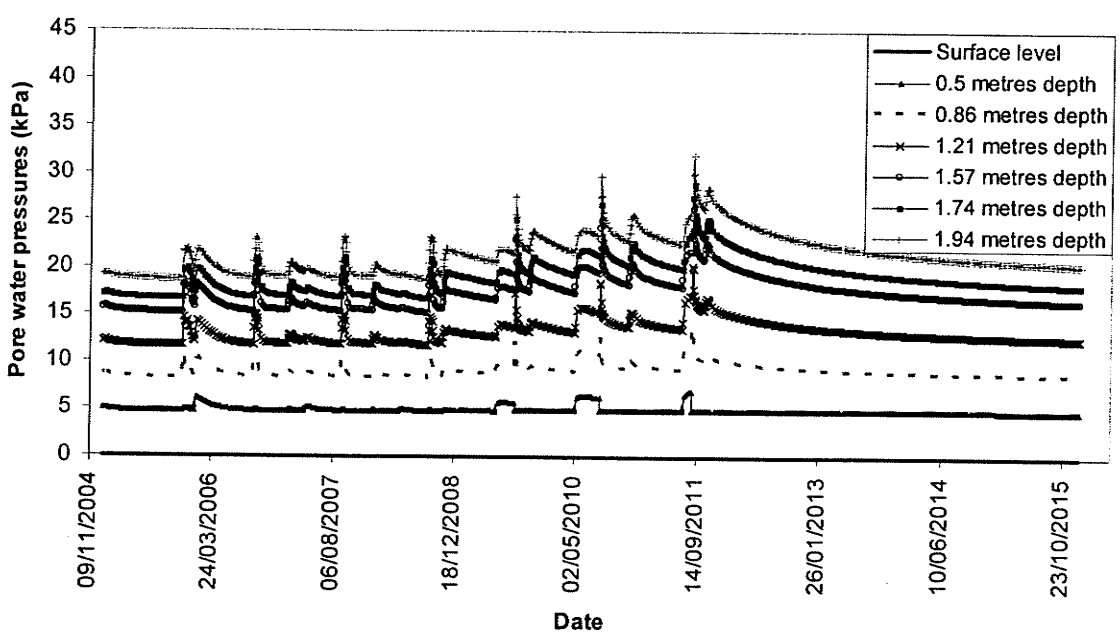

Figure 5-2: Pore water pressures versus time generated by placement of lightweight fill materials

5.2.3 EFFECTIVE STRESSES

Because of the dependence of effective stresses with the pore water pressures, these values are also reduced as it is shown in Figure 5-3. The behaviour is similar as for the coarse-grained soil fill materials (Figure 4-16) with the difference that the values are highly reduced by almost 50%. This would reduce settlements but decrease strength for foundation stability. However, the destabilizing forces are reduced due to lightweight fill and so stability is not affected significantly. Vertical effective stresses increase and reach maximum values when total degradation is achieved.

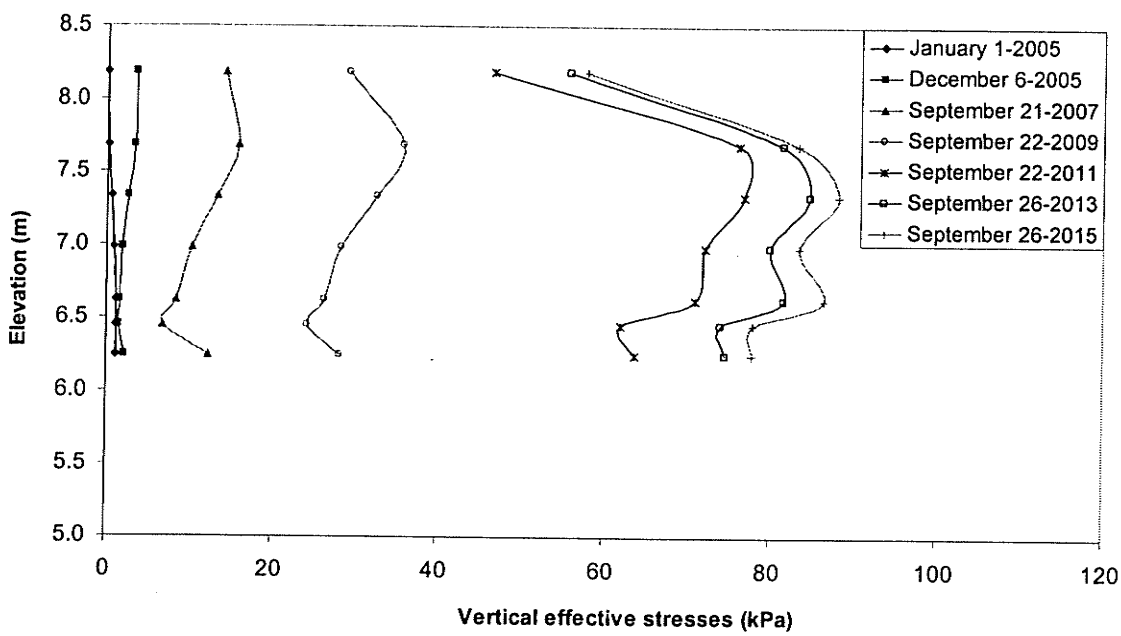

Figure 5-3: Vertical effective stresses versus depth for lightweight fill materials

Figure 5-4 shows the vertical effective stresses developed during the model period. Vertical effective stresses increase with time and become constant

when total permafrost degradation is achieved. The maximum values are reduced by 50% compared to the ones using coarse-grained soil fills (Figures 4-16 and 4-17).

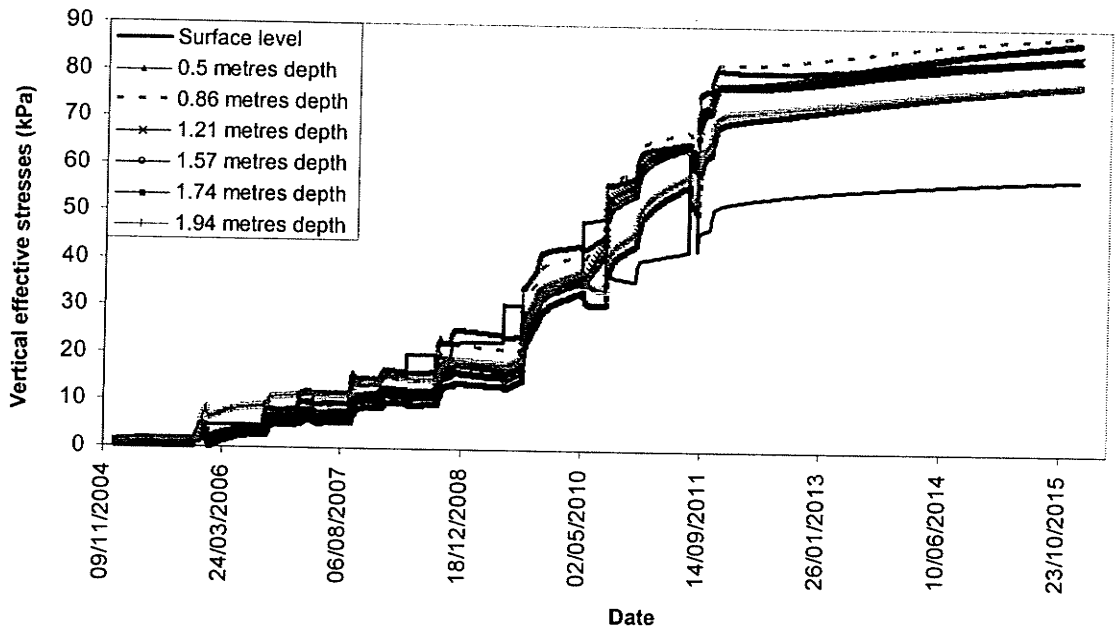

Figure 5-4: Vertical effective stresses versus time for lightweight fill materials

5.2.4 VERTICAL DISPLACEMENTS

Serviceability aspects may be the controlling factor in the design of any geotechnical structure. Designing structures that are in compliance with the specified allowable settlements is always a challenge for engineers. Lightweight fill materials are expected to reduce induced stresses and subsequently reduce the potential instabilities and excessive settlements in the foundation soil.

Figures 5-5, 5-6 and 5-7 show the maximum vertical displacements developed at the surface of the foundation soil when using typical lightweight fill materials such as geofoam. Figure 5-5 shows the conditions for the first three years of modelling until January of 2008. Figure 5-6 is for the results until October of 2011, while Figure 5-7 shows the results until December of 2015. Displacements are also reduced compared with the conventional earth fill materials (Figures 4-18, 4-19, and 4-20) although not by 50 % as for the case of the reduction in effective stresses.

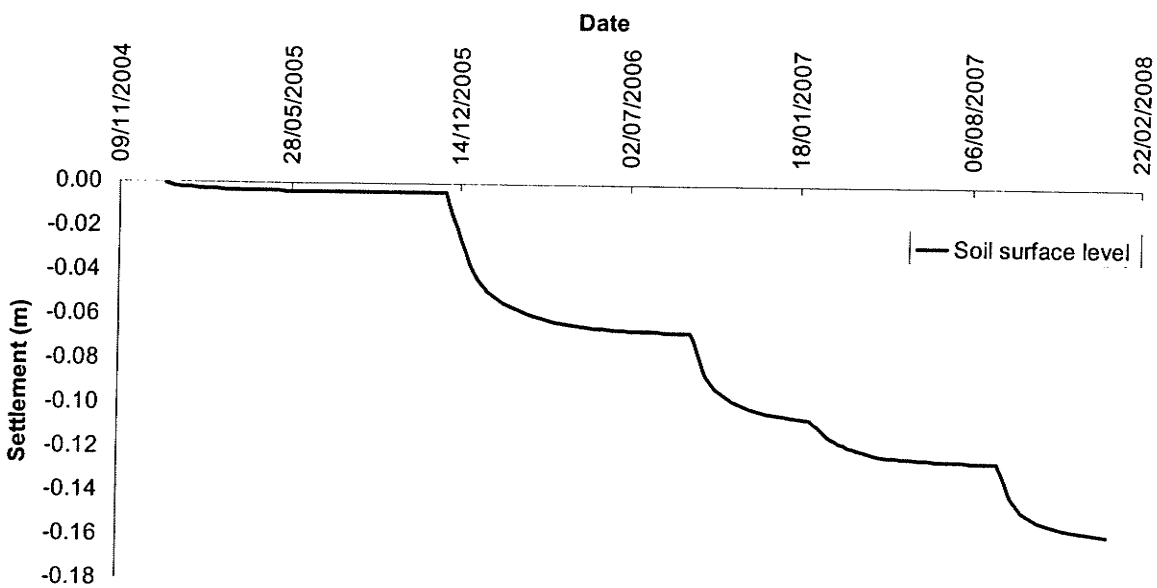

Figure 5-5: Total settlements for the models between January 2005 and January 2008 for lightweight fill materials.

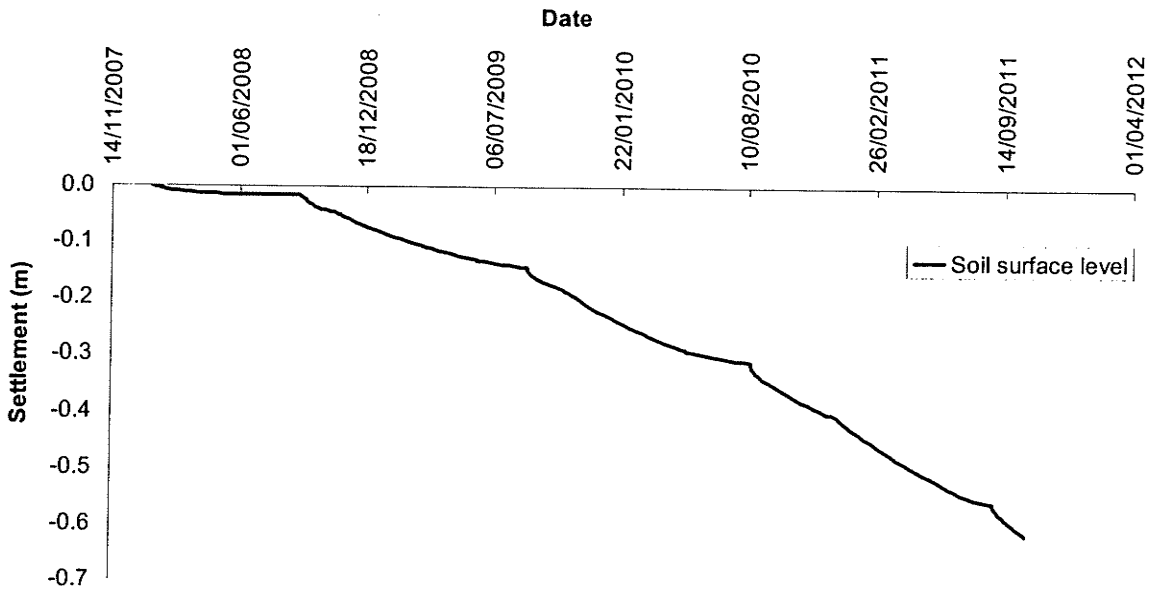

Figure 5-6: Total settlements for the models between January 2008 and October 2011 for lightweight fill materials.

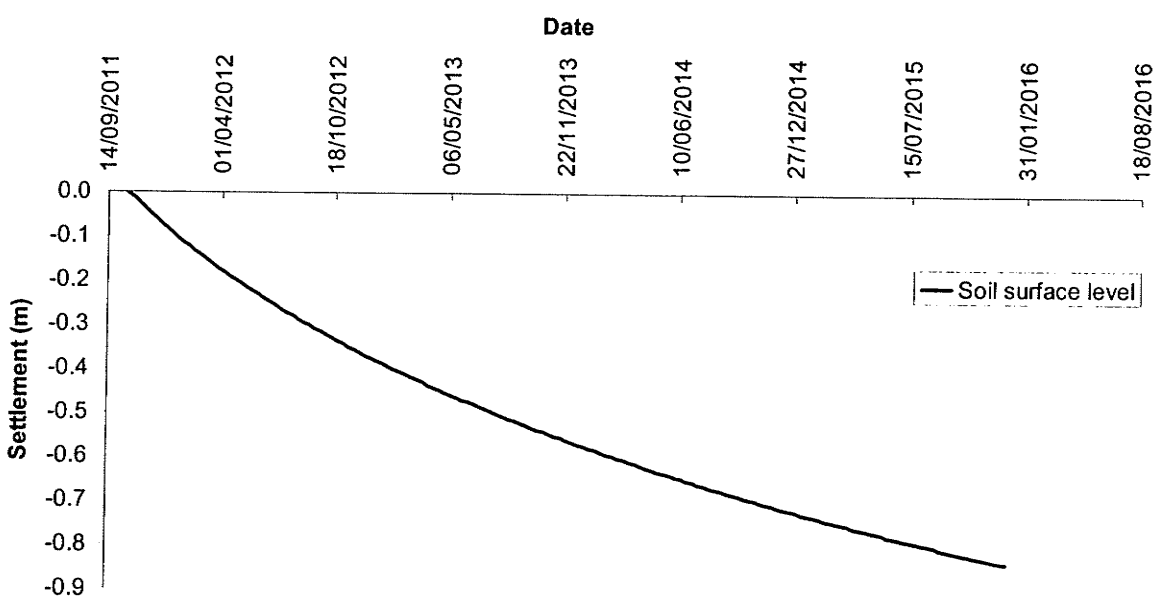

Figure 5-7: Total settlements for the model between October 2011 and December 2015 for lightweight fill materials.

5.2.5 HORIZONTAL DISPLACEMENTS

Horizontal displacements follow the same trend exhibited in the case where coarse-grained soil fill materials are used (refer to Figure 4-21). However, the horizontal displacements are reduced due to lower loads applied to the foundation soil. As for the case of coarse-grained soils, the horizontal displacements at the early periods are to the right when the permafrost is still present. When degradation starts and is more pronounced underneath the embankment, the soils loses strength and the deformations become higher and tend to the left. The maximum values of deformation are found when complete degradation of the permafrost is assumed.

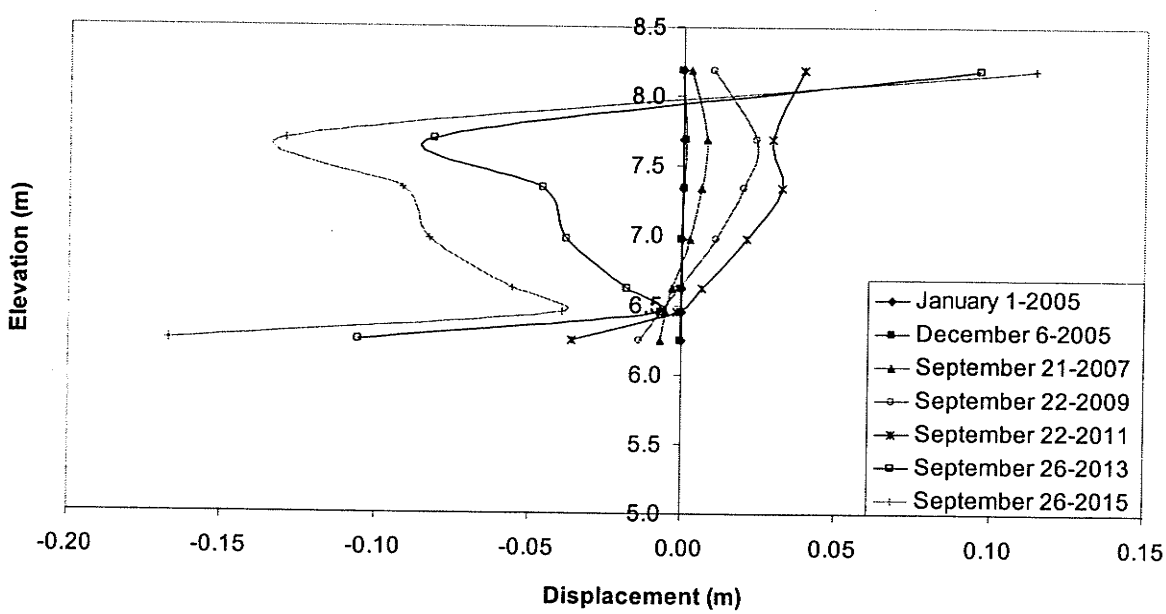

Figure 5-8: Maximum horizontal displacements at the toe of the embankment for lightweight fill materials.

5.3 ROCKFILL COLUMNS

Rockfill columns, otherwise known as stone columns, are vertical columns of compacted stones or rockfill materials that can be used to increase the stability and reduce settlements of embankments constructed over soft clay foundations. During the construction process between 15 to 35 percent of weak and compressible clay foundation can be replaced by the stones or rockfill materials. Since these columns are less compressible and have higher shear strength than the foundation clay soil they increase the overall performance of the new composite material (Partnership for Geotechnical Advancement, 2005).

When loaded, the column deforms by bulging into the soil strata and distributes the stresses at the upper part of the soil rather than transferring the stresses into the deeper layers, thus causing the soil to support it. As result, the strength and bearing capacity of the foundation soil are increased, while the compressibility is reduced. In addition, lesser stress concentration is developed in the columnar inclusions (Bergrado et al., 1994).

Stone or rockfill columns may also function as vertical drains when designed properly. This characteristic provides dissipation for pore water pressures, which increases the strength of the surrounding soil. It also provides a way to accelerate the consolidation and force undesirable settlements to occur before or during the construction of the road embankments. Modelling of rockfill columns was done by considering two different cases: (a) rockfill columns of 1

metre diameter and spacing of 1 metre on centres; and (b) rockfill columns of 2 metre diameter and 1 metre spacing on centres (this case simulates a continuous strip inclusion).

Since rock columns may function as drainage path, they can also increase the rate of degradation of the permafrost. This fact must be of special consideration when assessing the change in thermal properties of the frozen ground.

5.3.1 ROCKFILL COLUMNS 1 METRE IN DIAMETER

Figure 5-9 shows the cross section used to model the behaviour of rock columns of 1 metre diameter with spacing of 1 metre between them. The lengths of the rock columns into the soil is 7 metres and extend from the ground surface to lower boundary between the permafrost and the clay. The model used is the same as the one used in the modelling of coarse-grained soils and the lightweight fill materials. Since the model was thought to include the effect of rock columns it was only required to change the soil properties without the need to build a new mesh. Figure 5-10 is a zoom of the area to be used in the analysis of modelling results. The same thawing region was analyzed in the previous models discussed in Chapter 4 (refer to Figure 4-13).

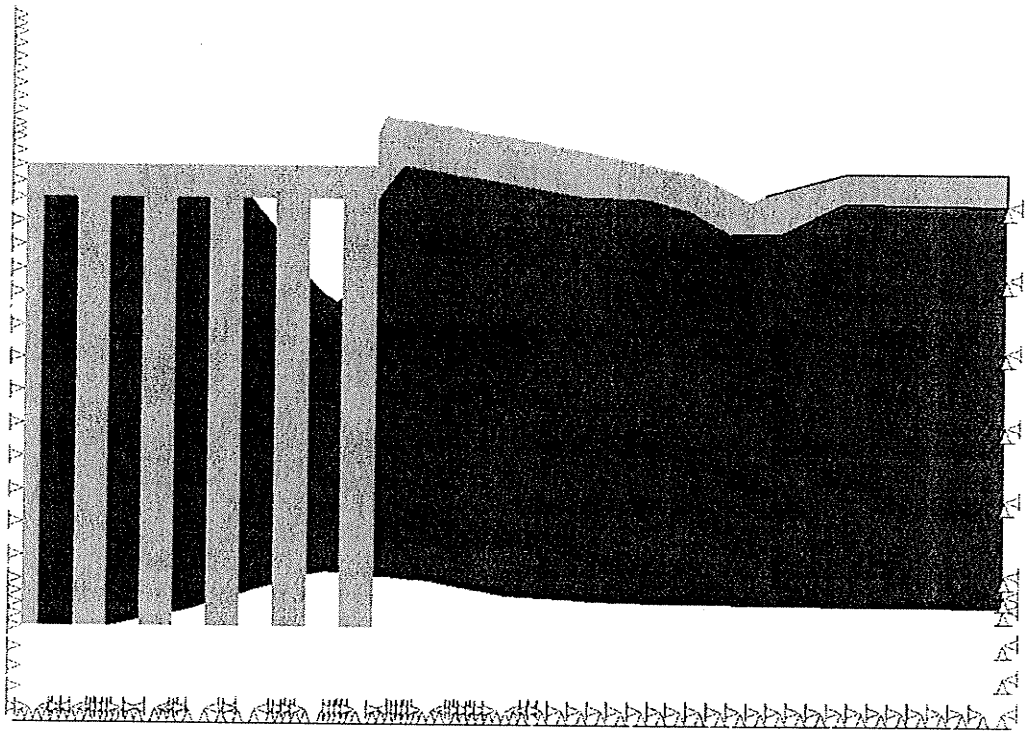

Figure 5-9: Cross section for rock columns 1 metre diameter.

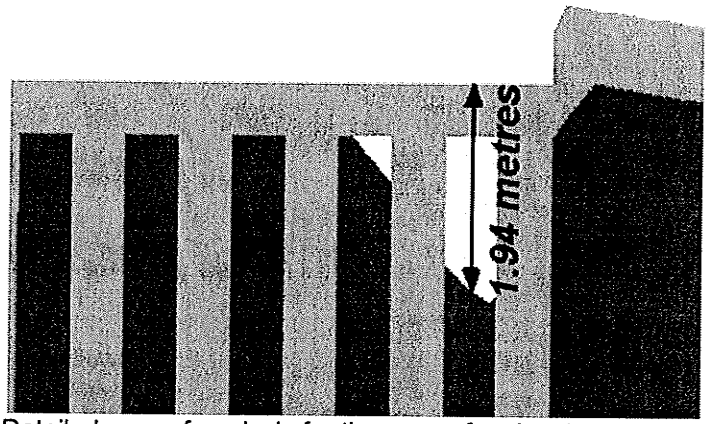

Figure 5-10: Detailed area of analysis for the case of rock columns 1 metre diameter.

Material Properties

Table 5-3 provides the material properties used to run the simulation. The rock columns were designed to have drainage capabilities to facilitate the dissipation of excess pore water pressures. These values were selected from the literature (Hudson and Harris, 1997) in the absence of laboratory or field test results of material properties.

Table 5-3 Material properties for rock columns

PROPERTY	Rock columns
Elasticity Modulus (kPa)	70000
Cohesion (kPa)	0
Poisson's Ratio	0.4
Hydraulic Conductivity (m/day)	8.64
Specific unit weight (kN/m^3)	18
Coefficient of earth pressure at rest (K_o)	0.4

Pore Water Pressures

Since rock columns have relatively good drainage capabilities, no excess pore water pressures are expected. Figure 5-11 shows the effect of drainage capabilities of the rock columns in controlling the increase in pore water pressures. The rapid dissipation of excess in pore water pressure accelerates the consolidation and increases the strength of the foundation soil. Figures 5-12 shows no generation of excess in pore water pressures takes place along the time period of the model.

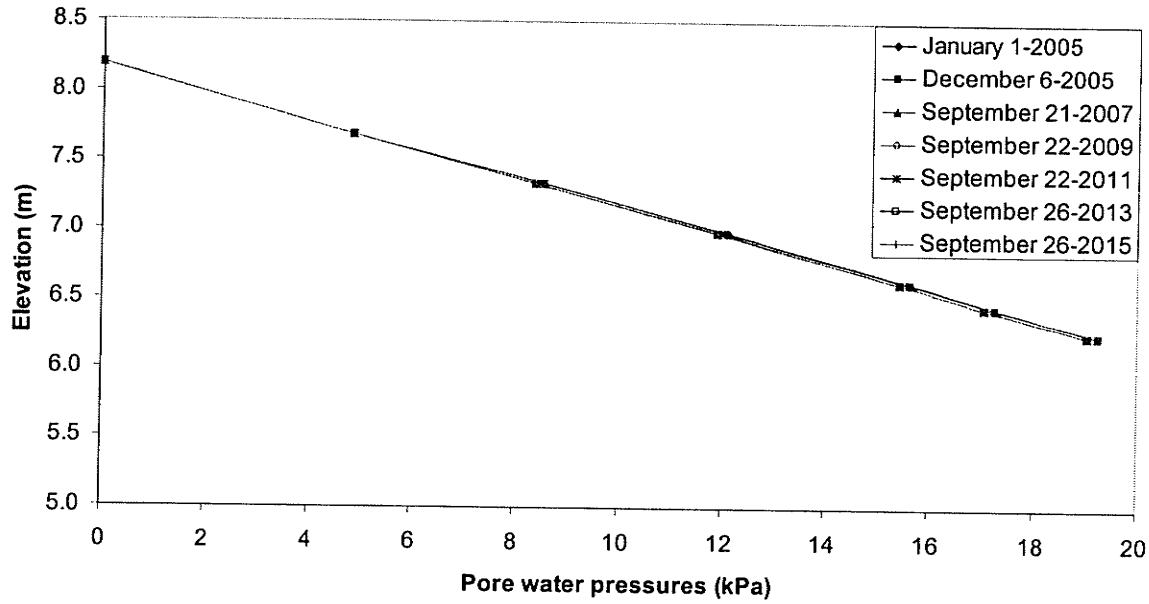

Figure 5-11: Pore water pressures versus depth for rock columns 1 metre diameter.

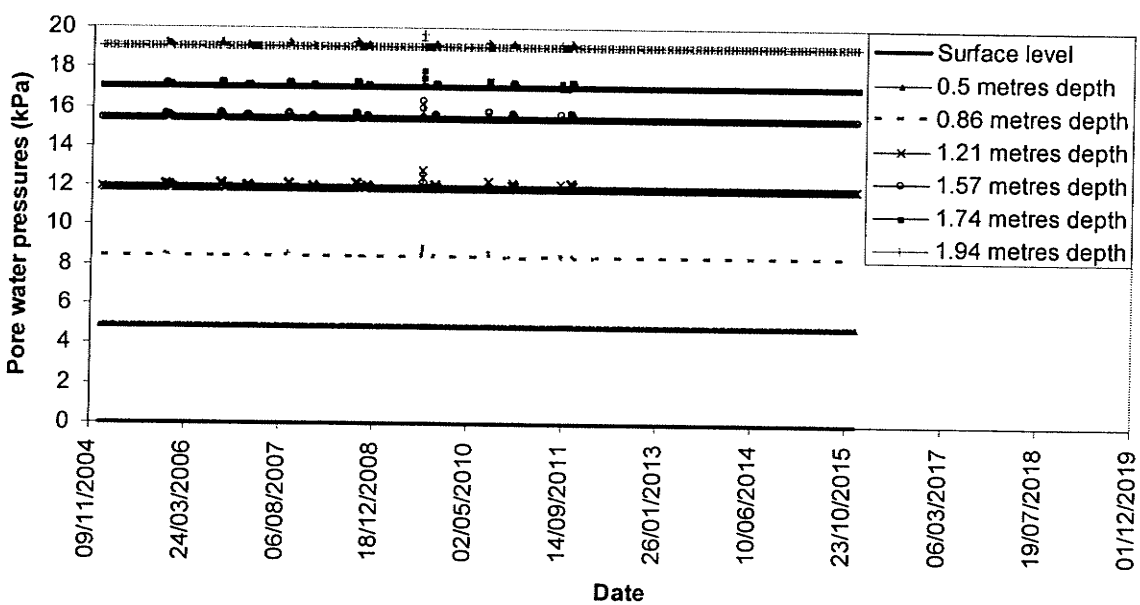

Figure 5-12: Pore water pressures versus time for rock columns 1 metre diameter.

Effective Stresses

Figure 5-13 shows the behaviour of the vertical effective stresses with depth at different time periods. Maximum vertical effective stresses are reached when degradation of the permafrost is complete and no more loading is applied to the foundation soil. The maximum values are quite a bit lower when compared to the coarse-grained soil and the lightweight fill material (refer to figures 4-16 and 5-3 respectively). This is because most of the load is concentrated in the rock columns and less load is transmitted to the native foundation soil.

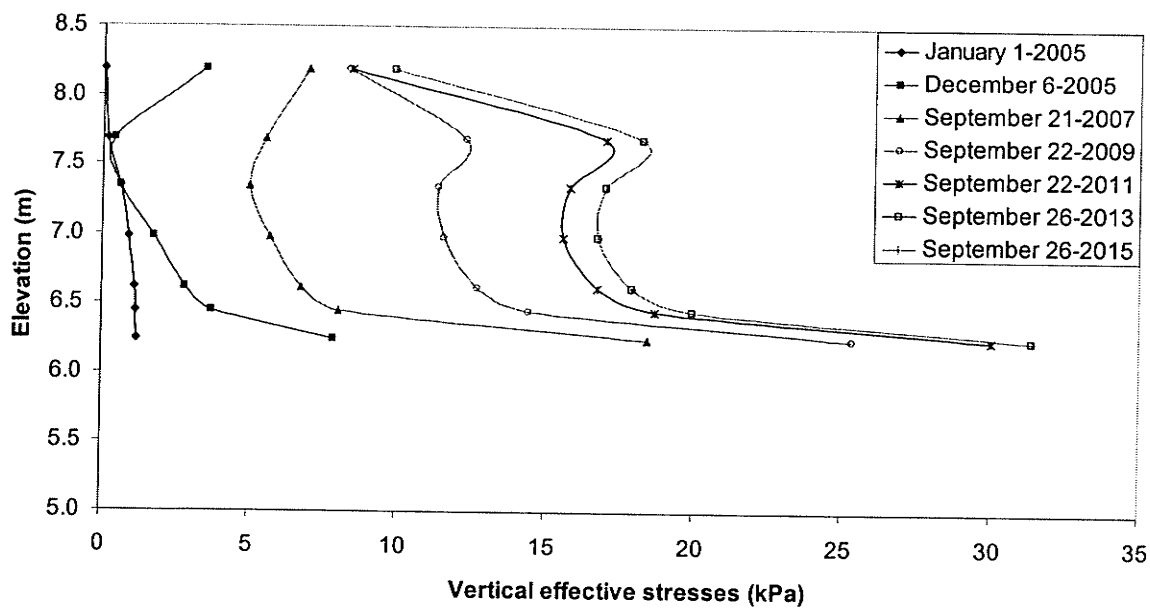

Figure 5-13: Vertical effective stresses versus depth for rock columns 1 metre diameter.

Figure 5-14 shows the change of vertical effective stresses with time. It can be seen that at the beginning when there is still permafrost, the values of stresses are very high at the contact between the permafrost and the clay. This is

due to the difference in stiffness between those two materials and a higher load applied at the early stages of the model. Once the degradation starts, the values at that depth follow a constant increasing trend until full degradation is achieved. The values are highly reduced compared to coarse-grained soils (Figure 4-16).

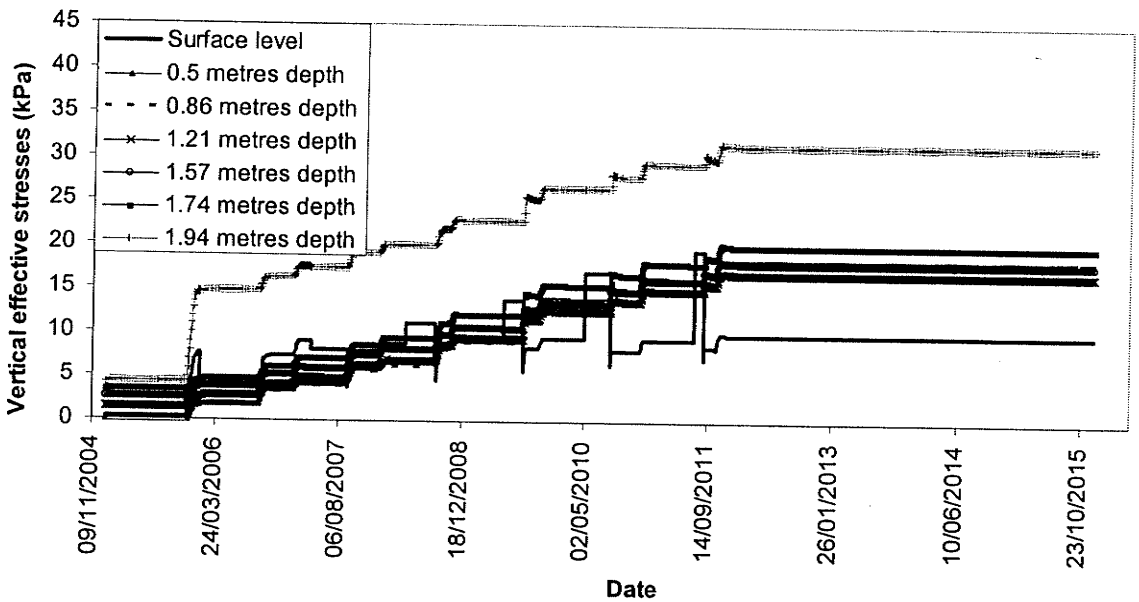

Figure 5-14: Vertical effective stresses versus time for rock columns 1 metre diameter.

Vertical Displacements

Since the highest concentration of stresses occurs in the rockfill columns, the vertical displacements in the clay are expected to be lower. A maximum value of 30 centimetres is reached at the end of the modelling period. This value is very small compared to the displacement achieved when using lightweight fill materials and coarse-grained soils (refer to figures 4-18 to 4-20 and 5-5 to 5-7). However, caution should be exercised given the fact that differential settlements

between the rockfill columns can lead to failures in the pavement. Figure 5-15 shows the displacements for the first three years of modelling. It can be seen that the consolidation is accelerated by the drainage capabilities of the rockfill columns.

It is evident in Figure 5-17 that the consolidation is accelerated due to the drainage capabilities of the rock columns. Very small displacements are reached after total degradation is assumed. This condition demonstrates the efficiency of the rock columns in accelerating the consolidation of the soil when they are properly designed to have drainage capabilities.

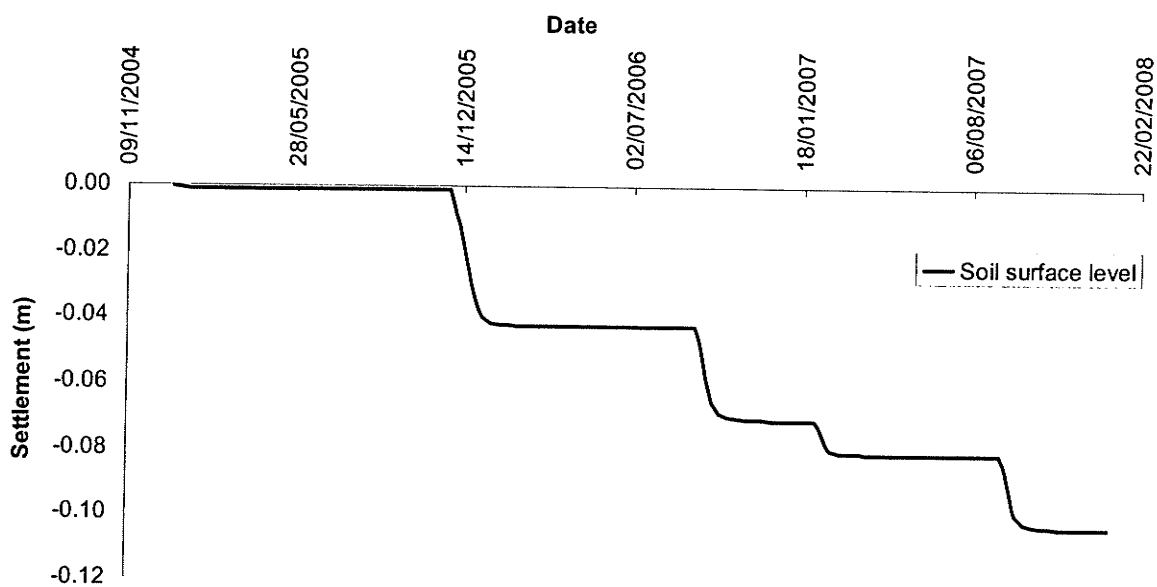

Figure 5-15: Total settlement for the models between January 2005 and January 2008 for rock columns 1 metre diameter.

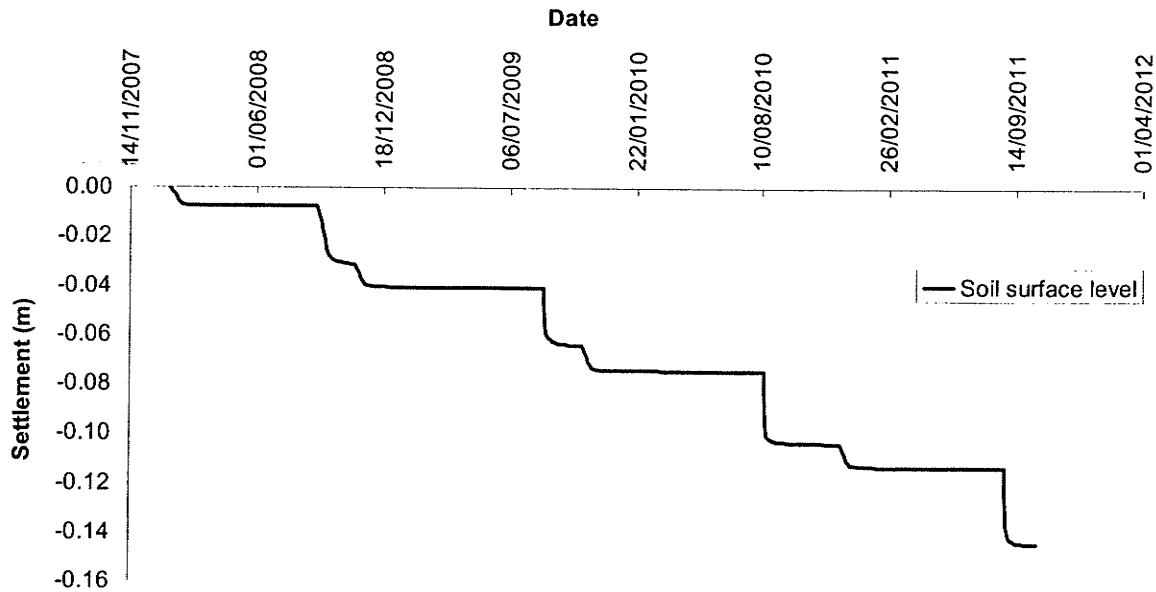

Figure 5-16: Total settlement for the models between January 2008 and October 2011 for rock columns 1 metre diameter.

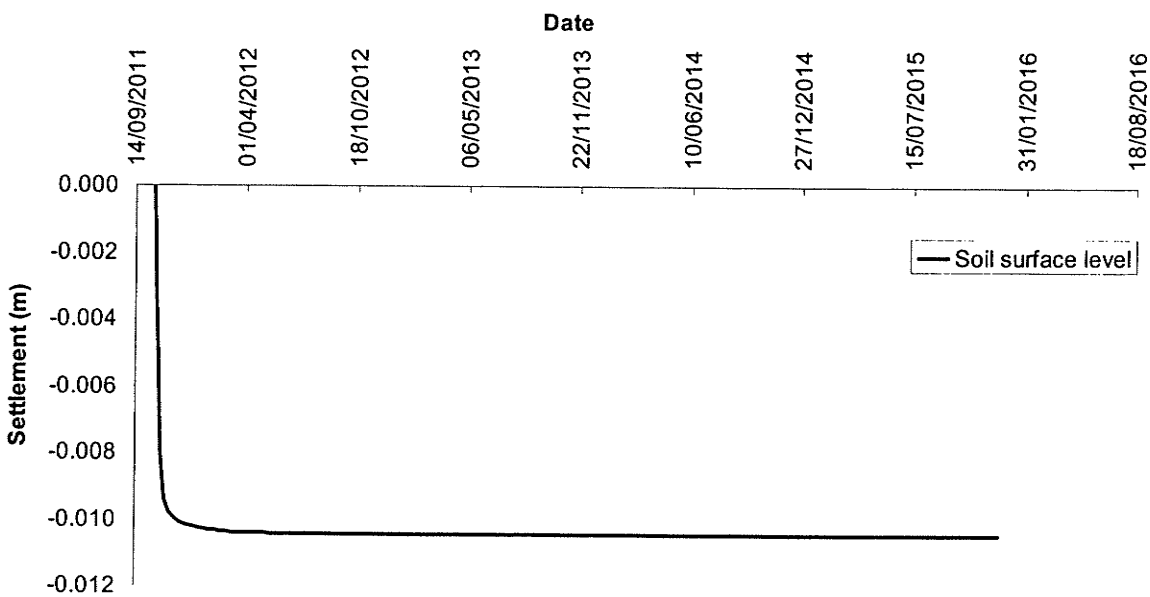

Figure 5-17: Total settlement for the models between October 2011 and December 2015 for rock columns 1 metre diameter.

5.3.2 ROCKFILL COLUMNS 2 METRES IN DIAMETER

As for the case of the rock columns of 1 metre diameter a new model with rock columns of 2 metres in diameter was built. In this case the spacing between columns is kept constant to analyze the influence of the diameter of the rock columns in the performance of the model. The material properties used in the model are the same as for the rock columns of 1 metre diameter. Figure 5-18 shows the resulting cross section for the analysis. The modelling involves the analysis of the same mechanical properties previously analyzed. Only changes in the distribution of stresses in the rock columns are expected.

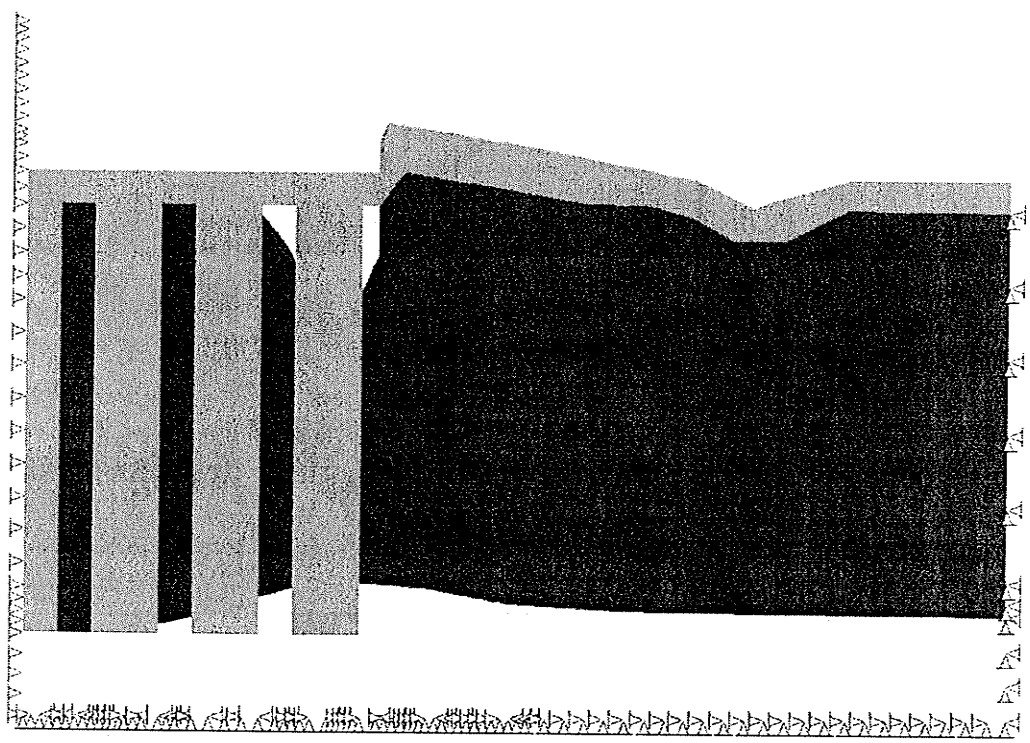

Figure 5-18: Cross section for rock columns 2 metres diameter

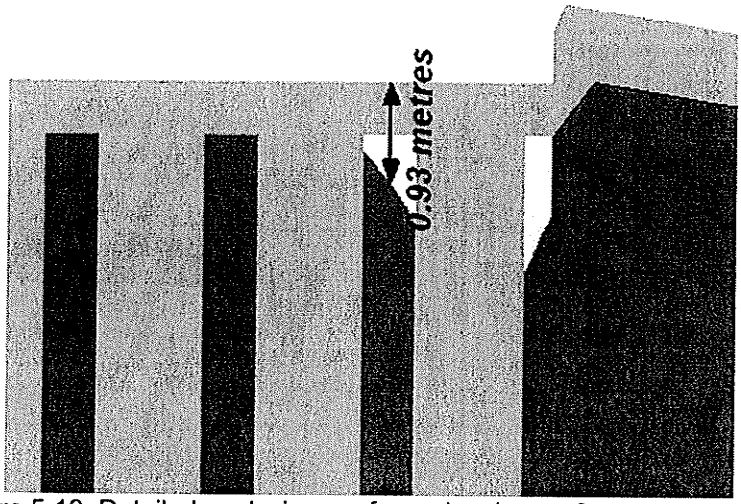

Figure 5-19: Detailed analysis area for rock columns 2 metres diameter

The area of analysis changes with respect to the previous models due to the distribution of the rock columns. Figure 5-19 shows the new area to be considered for the analysis compared to Figures 4-13 and 5-10.

Pore Water Pressures

This model is also designed to have drainage capabilities, which means that the excess pore water pressures should be easily dissipated and follow the trend exhibited for the model of rock columns 1 metre diameter. Figures 5-20 and 5-21 show the effects of the rockfill columns in dissipating the excess pore water pressures when the foundation soil is loaded every thawing season.

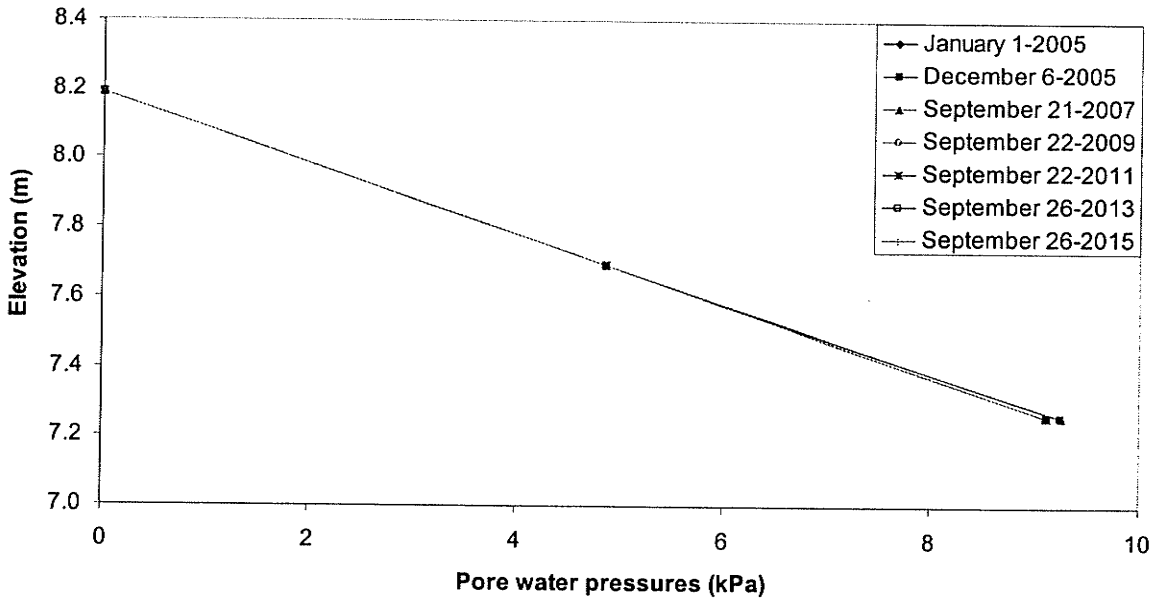

Figure 5-20: Pore water pressures versus depth for rock columns 2 metres diameter.

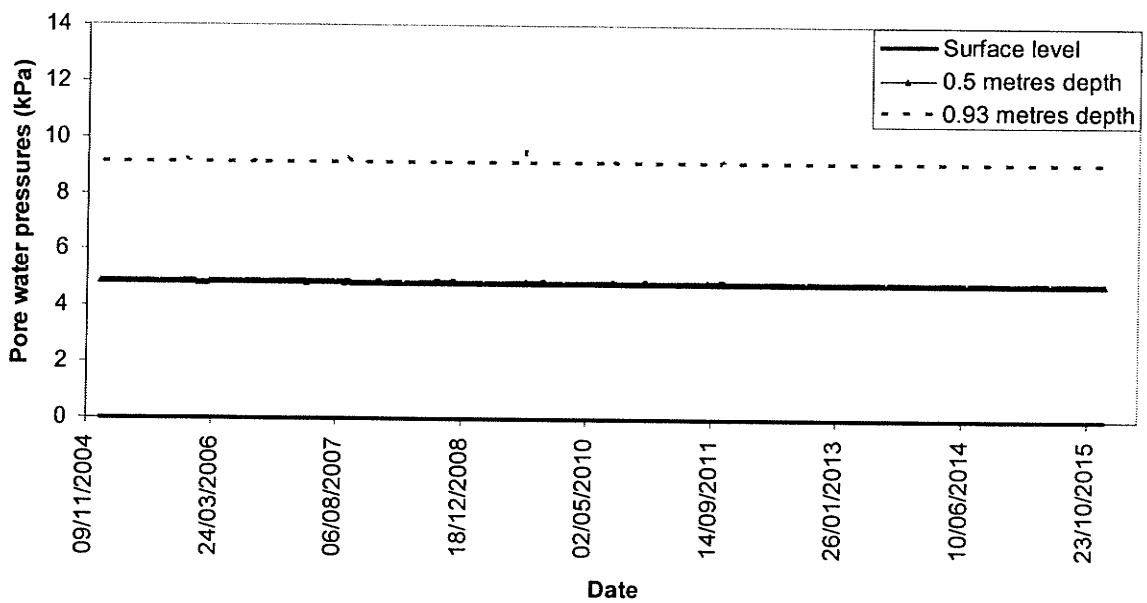

Figure 5-21: Pore water pressures versus time for the rock columns 2 metres diameter.

Effective Stresses

The main objective of modelling two different scenarios with rockfill columns is to analyze the distribution of stresses when the diameter of the rock column is increased. The effective stresses are reduced due to an increase in the area of the rock column, which leads to a better distribution of stresses. Figure 5-22 shows the maximum vertical effective stresses found. The effective stresses increase with increase in degradation of the permafrost. The increase is higher at the surface and at the initial boundary between the clay and the permafrost.

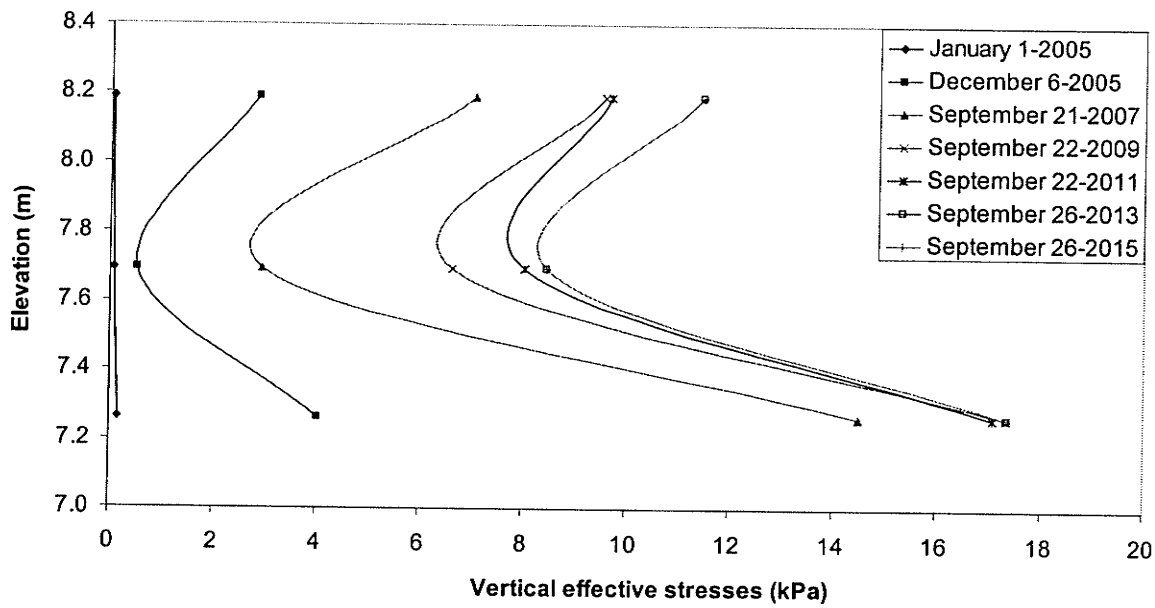

Figure 5-22: Vertical effective stresses versus depth for rock columns 2 metres diameter.

Figure 5-23 shows the maximum effective stresses generated during the time period.

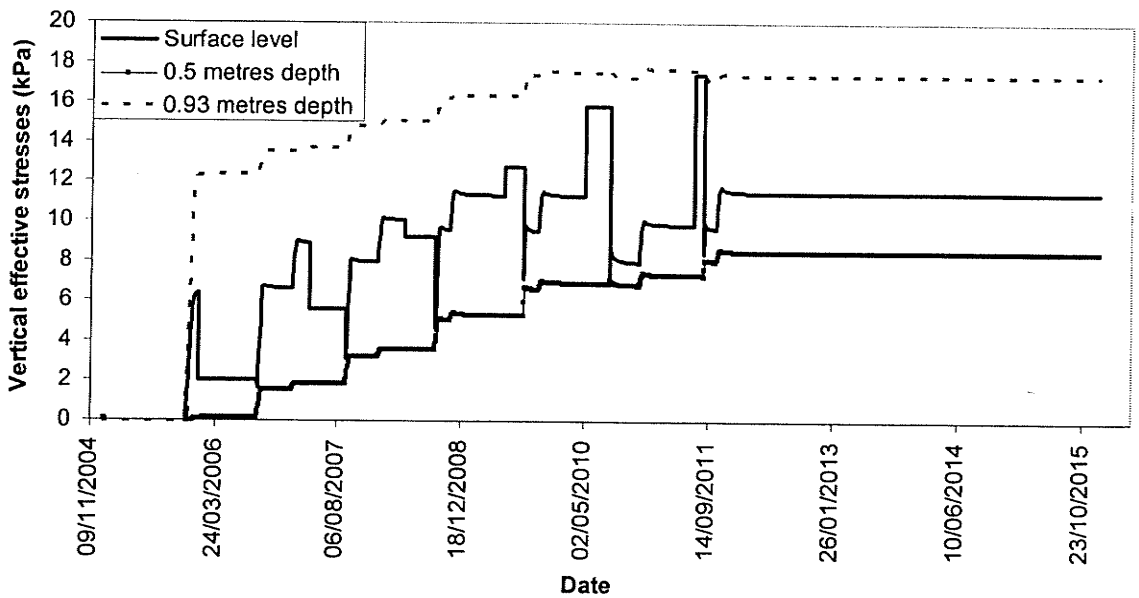

Figure 5-23: Vertical effective stresses versus time for rock columns 2 metres diameter.

Vertical Displacements

As for the case of rockfill columns 1 metre diameter, the vertical displacements are lower than for the previous models. A maximum settlement at the surface of the foundation soil was about 21 centimetres in this model. It is also shown in the figures how the consolidation is accelerated due to the drainage capabilities of the rock columns. Settlements exist because the rock columns also deform under the applied loads.

Settlements between the columns and the clay may be prevented if the rockfill columns were supported on a firm stratum of soil or bedrock. This condition, however, is not satisfied in the model since no evidence of bedrock was found in the field site investigation at a depth of more than 12 metres.

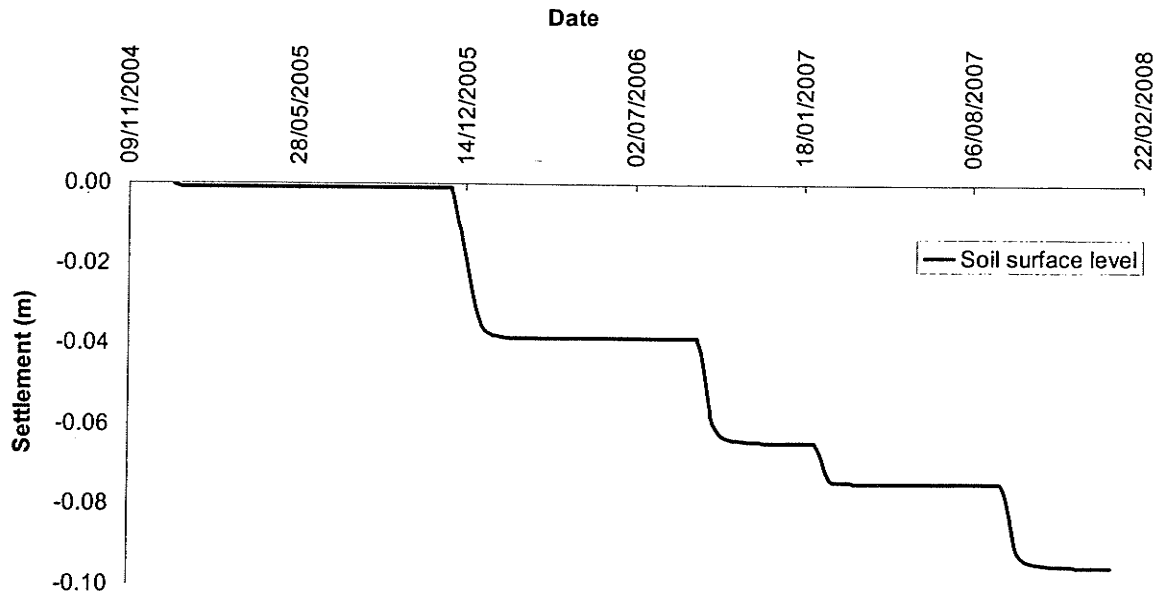

Figure 5-24: Total settlement for the models between January 2005 and January 2008 for rock columns 2 metres diameter

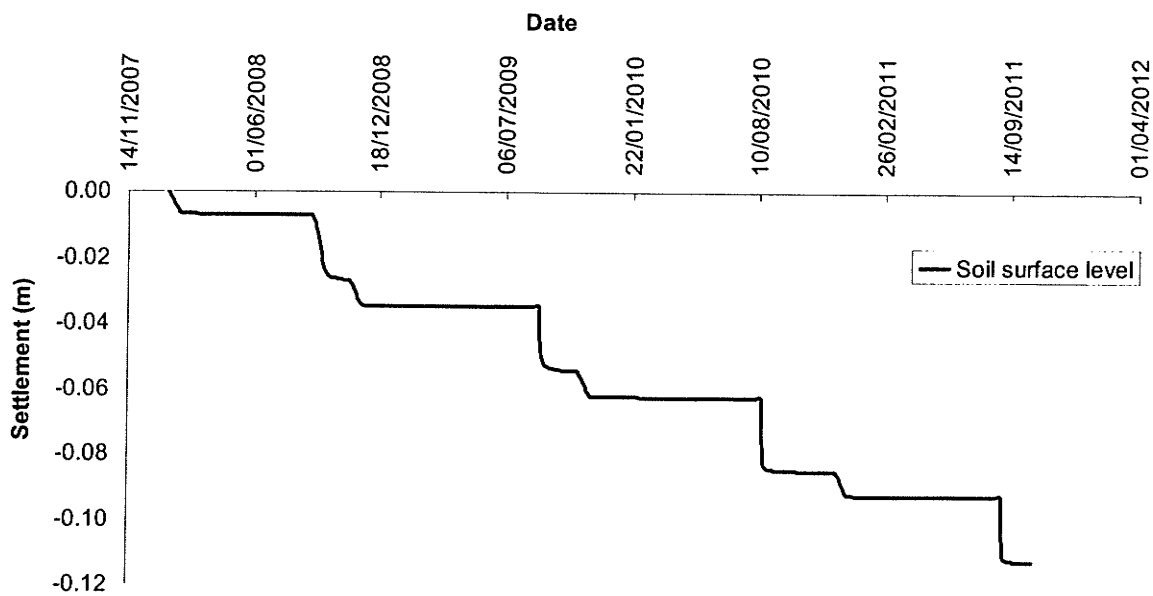

Figure 5-25: Total settlement for the models between January 2008 and October 2011 for rock columns 2 metres diameter

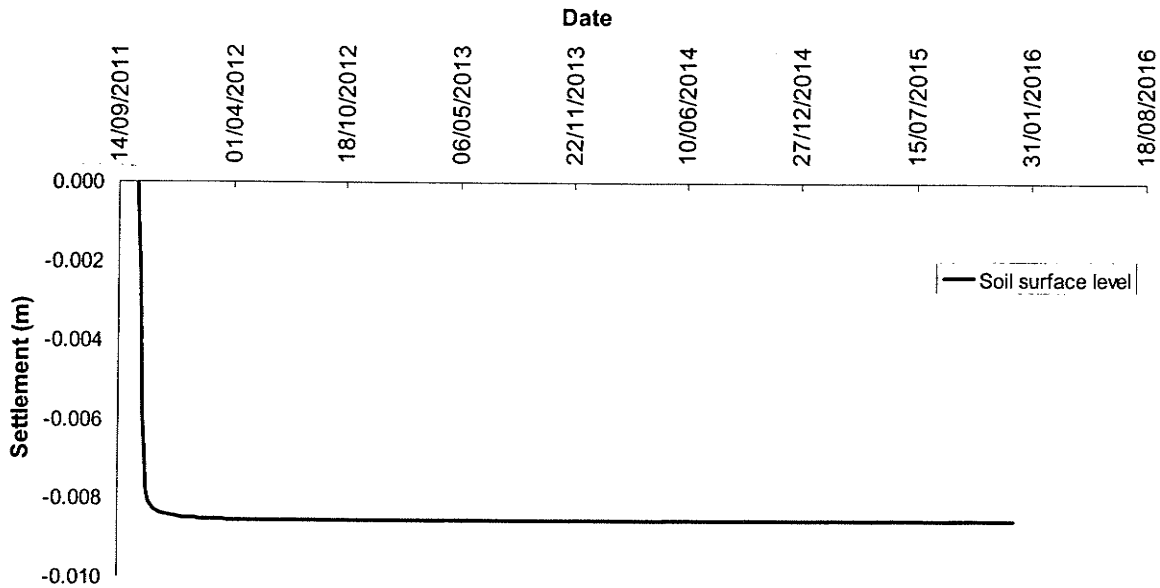

Figure 5-26: Total settlement for the models between October 2011 and December 2015 for rock columns 2 metres diameter

5.4 SHEET-PILES

Most of the problems associated with failure of road embankments in the Northern Regions are related to horizontal displacements at the toe of the embankment. As stated previously, degradation starts at the toe due to a reduction in thickness of the embankment at that location and higher transmission of heat from the surface. Thawing at the toe generates excess pore water pressures and thus foundation instability. Instability in the foundation can be reflected in the form of dips and cracks in the pavement. These dips and cracks are created by vertical and horizontal movements at the toe. If the movements are controlled, instabilities in the foundation of road embankments may be reduced.

Another technique that has been included in this research study is to analyze the benefits of using sheet-piles in minimizing the lateral displacements of the foundation soil. This technique consists of steel sheets driven into soil at the toe of the embankment to increase the shear resistance of the foundation soil. The advantage of this technique is that it might produce cost savings for the case of existing embankments. The thickness of these sheets can be as small as 5 cm, and can be installed more quickly than other techniques (Ochiai, et al., 1991a; Ochiai, et al., 1991b).

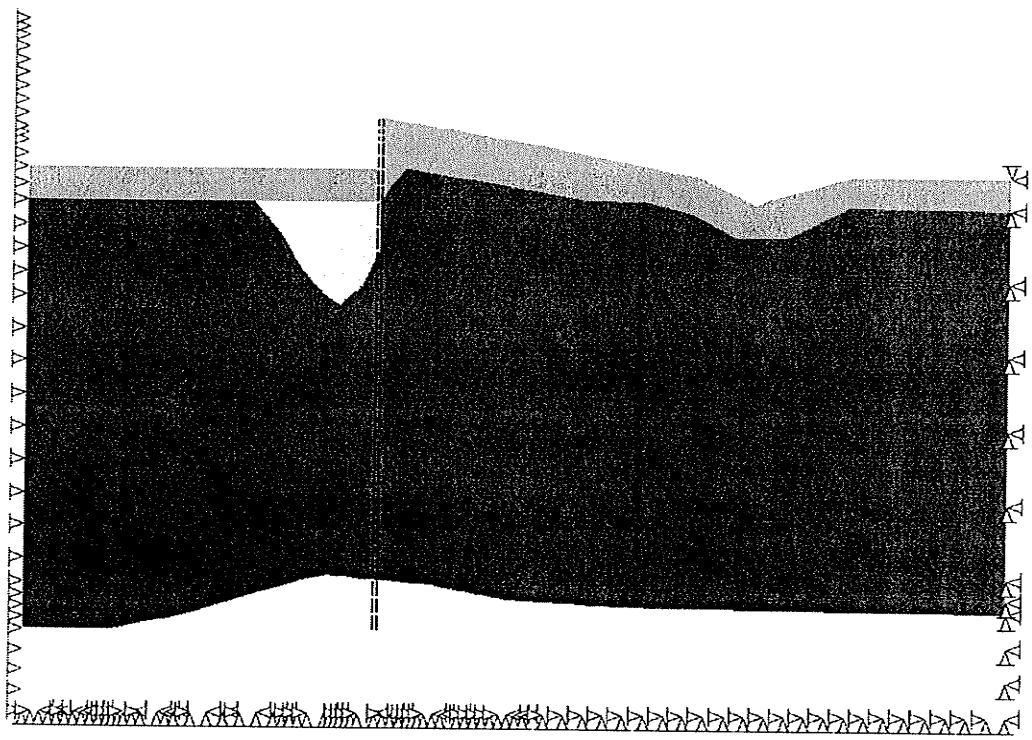

Figure 5-27: cross section for sheet-piles model

The cross section for the model is shown in Figure 5-27. The sheet-pile has a thickness of 5 cm, elasticity modulus of 200000 MPa taken from the

literature. Based on that information a moment of inertia of $1.04X10^{-5}$ m^4 was calculated for the simulation. Figure 5-28 presents a zoom of the area to be analyzed that compares to the same area analyzed for coarse-grained soils, lightweight fill materials and rockfill columns 1 metre diameter (refer to Figures 4-13 and 5-10).

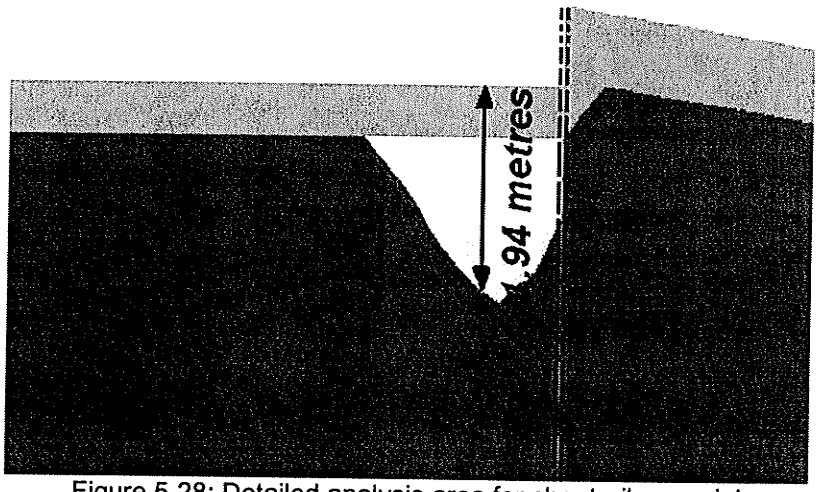

Figure 5-28: Detailed analysis area for sheet-piles model

5.4.1 PORE WATER PRESSURES

Figures 5-29 and 5-30 present the results for the pore water pressures. Excess in pore water pressures are generated during loading of the foundation soil.

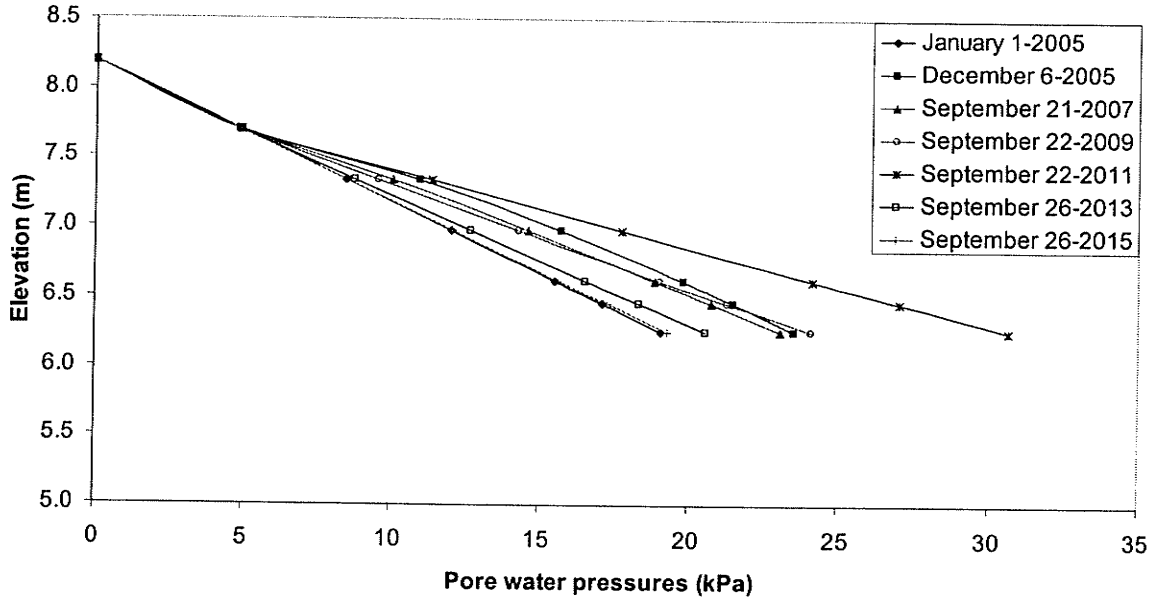

Figure 5-29: Pore water pressures versus depth for sheet-piles model

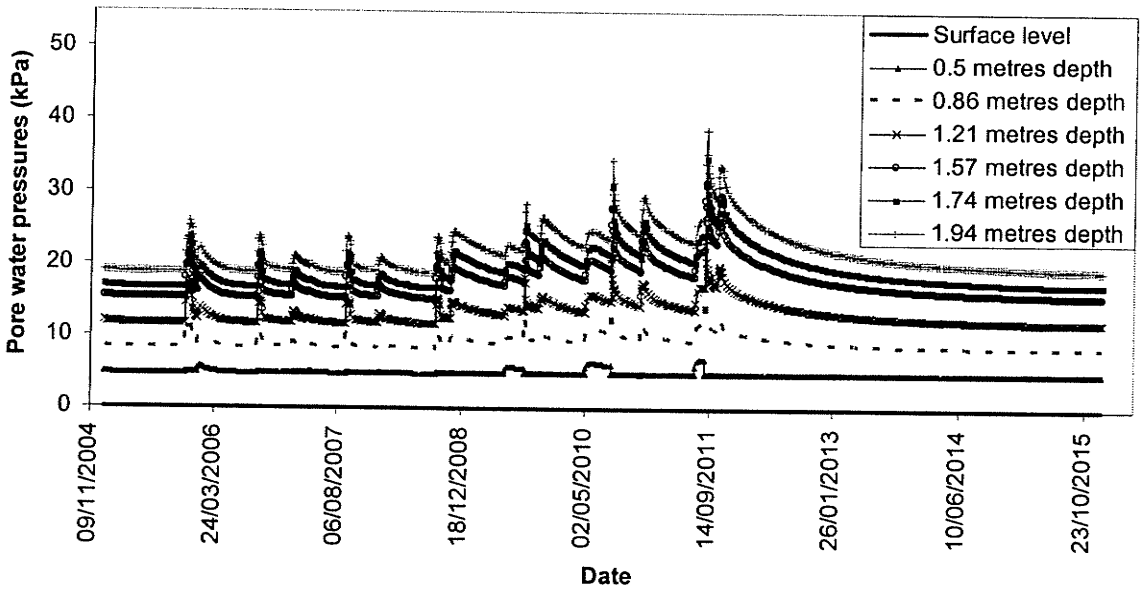

Figure 5-30: Pore water pressures versus time for sheet-piles model

The excess pore water pressures have the same values as the coarse-grained soil since no improvement is incorporated to dissipate the pore water pressures. Peaks of pore pressures during loading and unloading are also present as expected. These peaks are shown in Figure 5-30 and follow the same trend exhibited in the model for coarse-grained soil and lightweight embankment fill (refer to Figures 4-15 and 5-2). Constant building up of pore water pressures takes place for most of the model until degradation of the permafrost is achieved.

5.4.2 EFFECTIVE STRESSES

The vertical effective stresses are lower than for the case of coarse-grained soils. The reduction is in the order of 70 kPa at the end of the modelling period. However, the trend of increase in effective stresses is the same. They increase as the degradation of the permafrost increase.

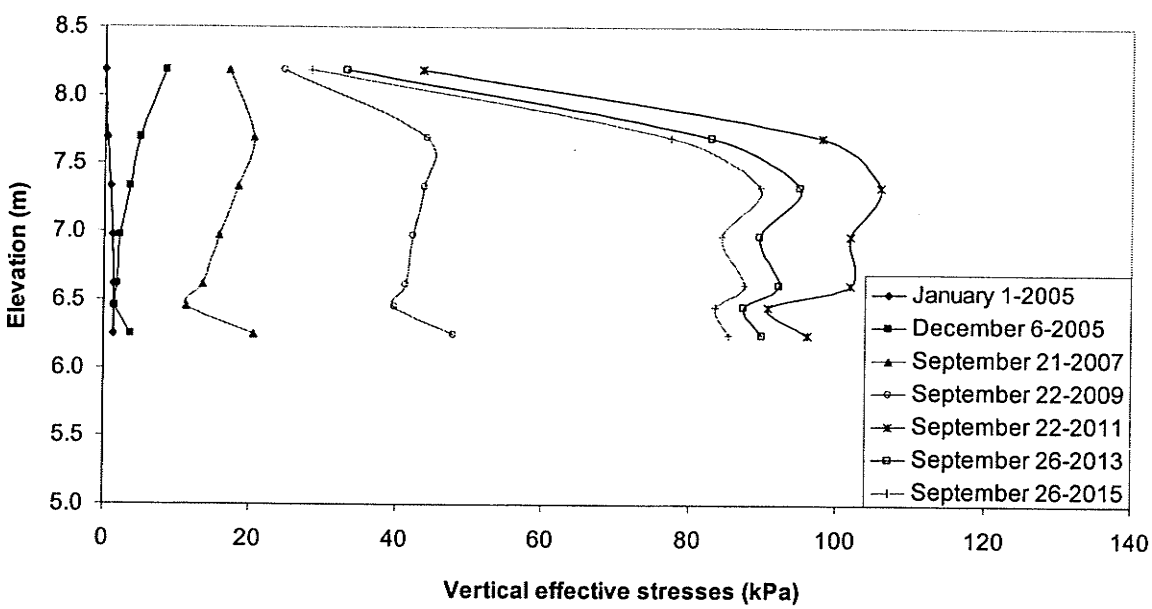

Figure 5-31: Vertical effective stresses versus depth for sheet-piles model.

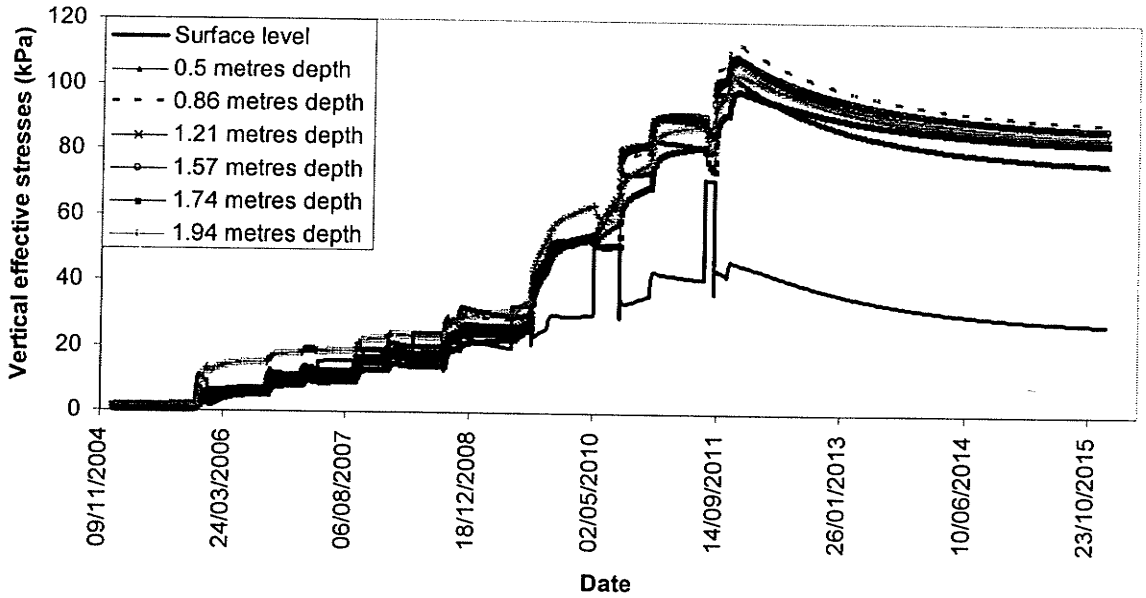

Figure 5-32: Vertical effective stresses versus time for sheet-piles model

5.4.3 VERTICAL DISPLACEMENTS

Since the effective stresses are reduced, vertical displacements are also reduced. As for the case of effective stresses most of improvement occurs at the end of the model period when complete degradation of the permafrost is reached. The reduction in settlement is close to 70 centimetres. However, the value of settlement is still high. The final settlements are very similar to the settlements obtained for lightweight fill materials (Figures 5-5 to 5-7). The displacements increase with increase in degradation of the permafrost and are higher at the end of the model period due to the poor drainage capabilities of the foundation soil, which does not allow a rapid dissipation of the excess pore water pressures.

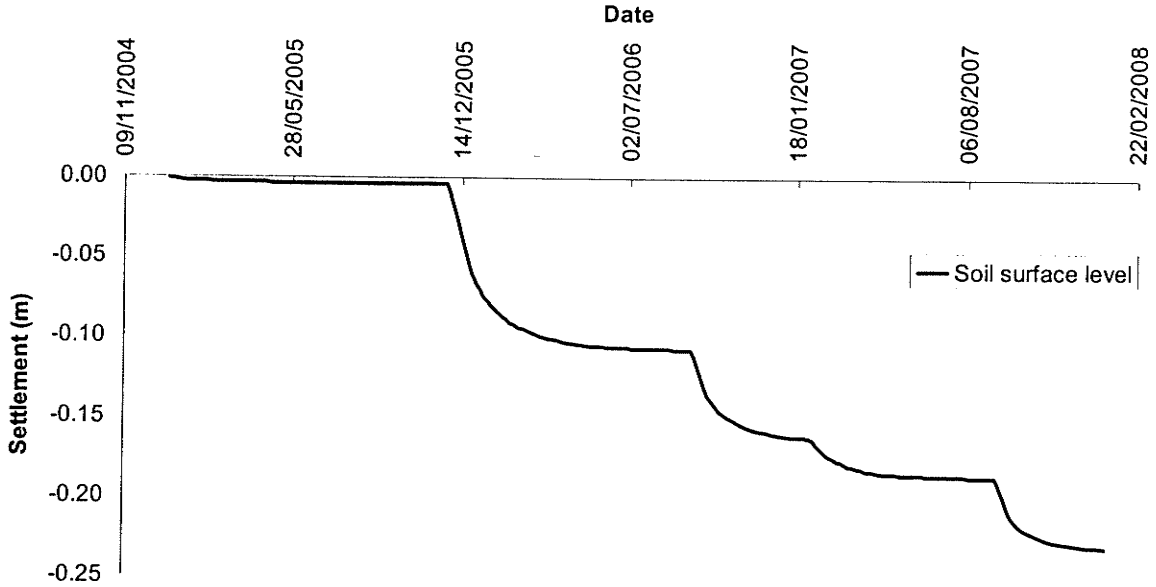

Figure 5-33: Total settlement reached for the models between January 2005 and January 2008 for sheet-piles model

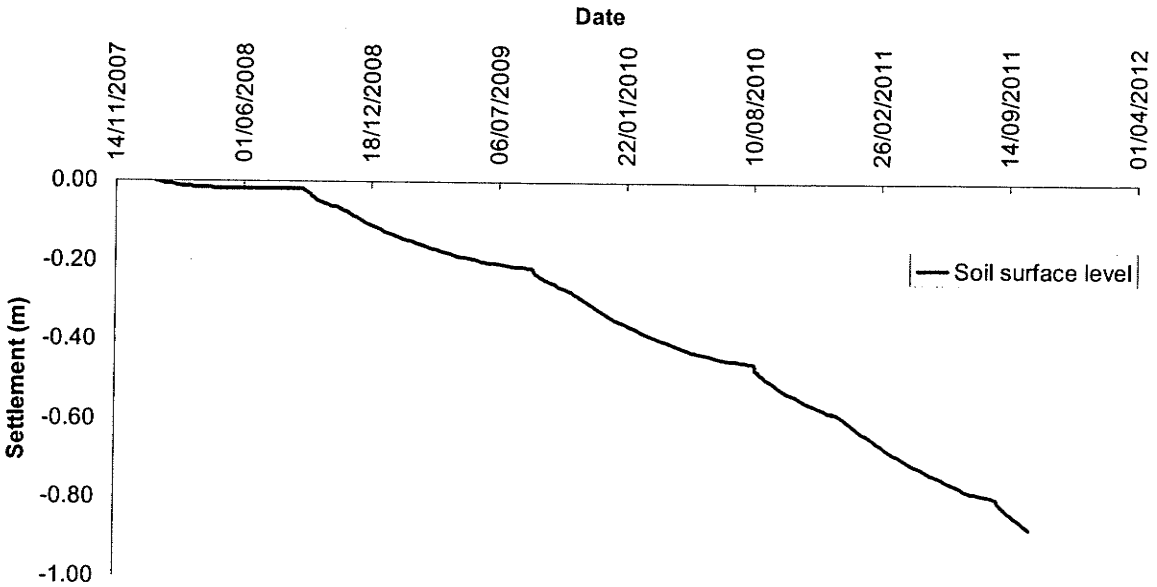

Figure 5-34: Total settlement reached for the models between January 2008 and October 2011 for sheet-piles model

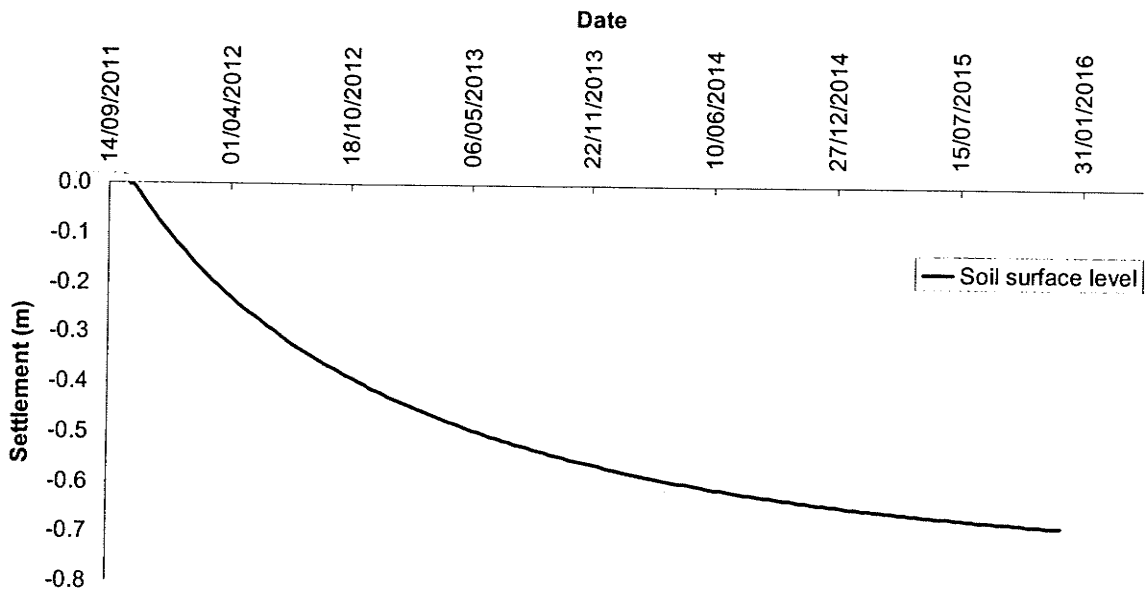

Figure 5-35: Total settlement reached for the model between October 2011 and December 2015 for sheet-piles model

5.4.4 HORIZONTAL DISPLACEMENTS

The main objective of the model is to reduce the horizontal displacements at the toe of the embankment to improve the stability of the road embankment. The sheet-piles model shows a high reduction in the horizontal displacements principally at the end of the model period. The reduction is in the order of 50% compared with that without remedial measures. This improvement in the reduction of horizontal displacements helps reduce the vertical settlements.

Figure 5-36 shows the maximum values of horizontal displacement found for different dates in the model.

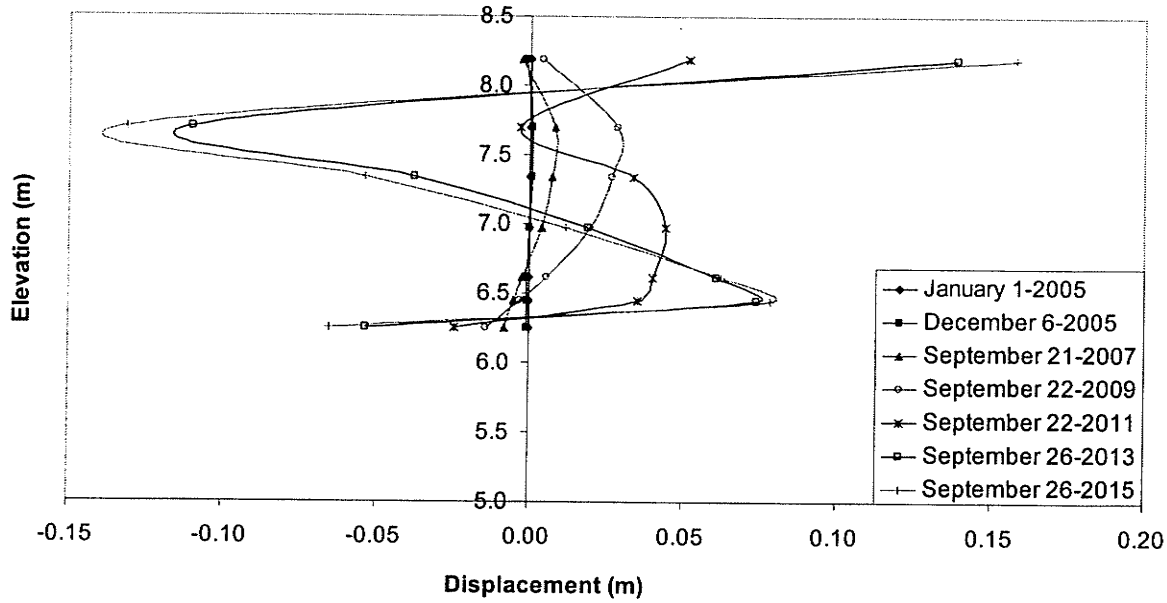

Figure 5-36: Maximum horizontal displacements at the toe of the embankment for sheet-piles model

5.5 SUMMARY OF RESULTS

A compilation of data is presented to compare the different models used to improve the ground properties. The results from pore water pressures, vertical effective stresses, and horizontal and vertical displacements are analyzed. Shear stresses are omitted since only cross sections with contour lines were provided for every case.

Figure 5-37 shows the results of pore water pressures versus elevation for September 26-2015, which corresponds to the date with maximum excess of pore water pressures for every adaptation strategy. Rockfill columns perform very well in controlling the excess pore water pressures generated during the loading of the embankment.

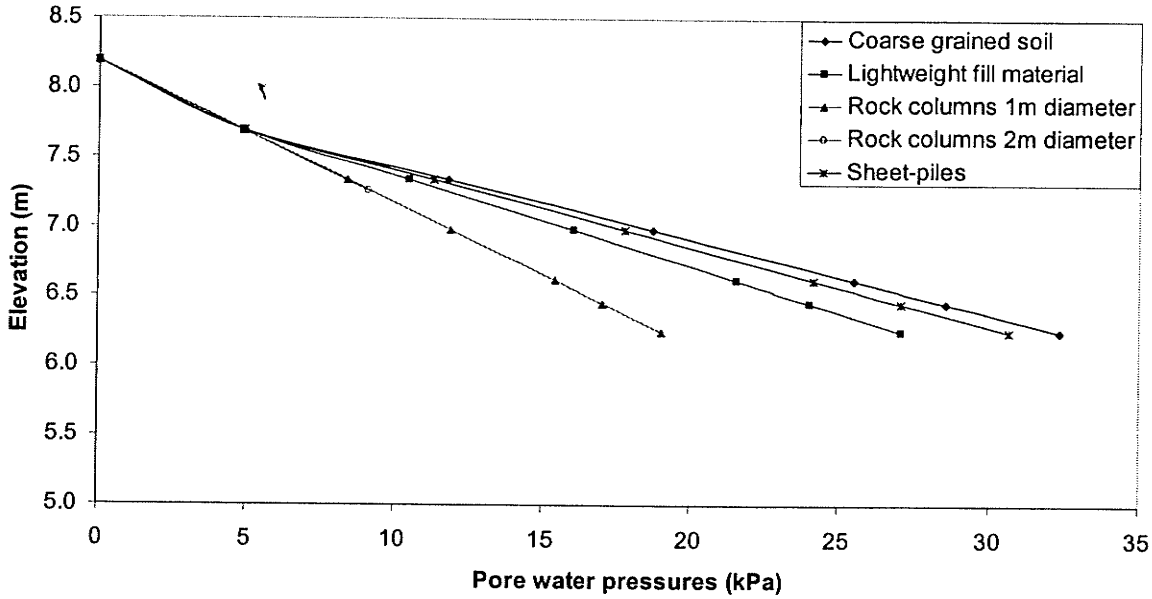

Figure 5-37: Pore water pressures versus depth for September 26-2015 for the different adaptation strategies

Lightweight fill materials and sheet-piles also reduced the pore water pressures generated. Their behaviour is very similar, however, the lower loads applied with the lightweight fill materials lead to lower pore water pressures during the model period as shown in Figure 5-38.

Loads in the degrading permafrost in terms of vertical effective stresses are highly reduced by using rockfill columns. The reduction in stresses is very significant as shown in Figure 5-39. Rockfill columns 2 metres diameter perform slightly better than those with 1 metre diameter. The reduction in stresses by using rockfill columns is close to 90 percent.

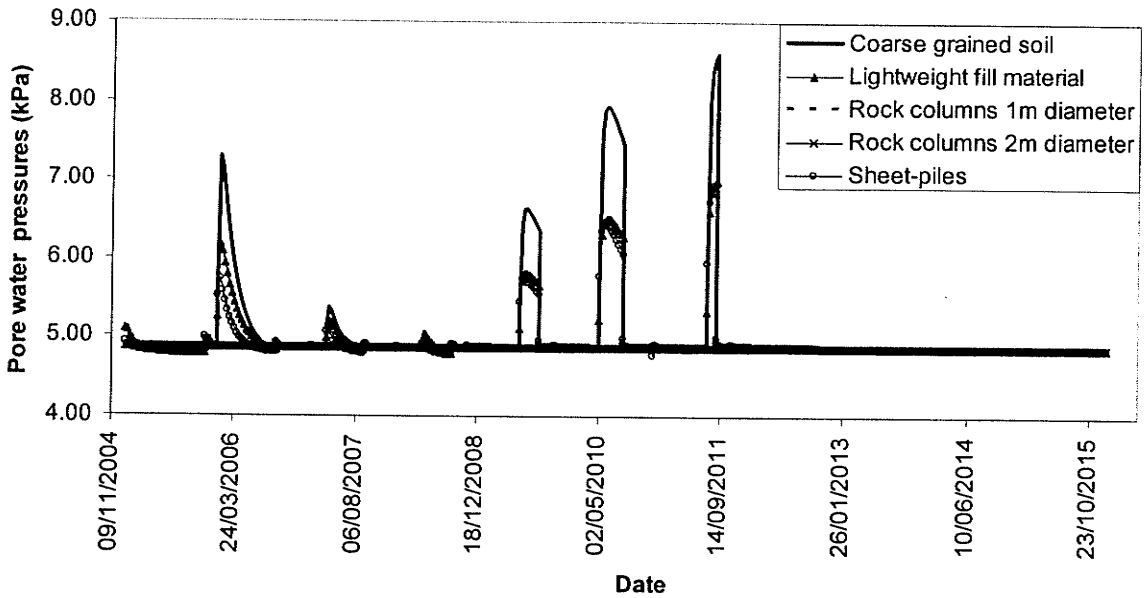

Figure 5-38: Pore water pressures versus time for point 0.5 metres from the surface for the different adaptation strategies

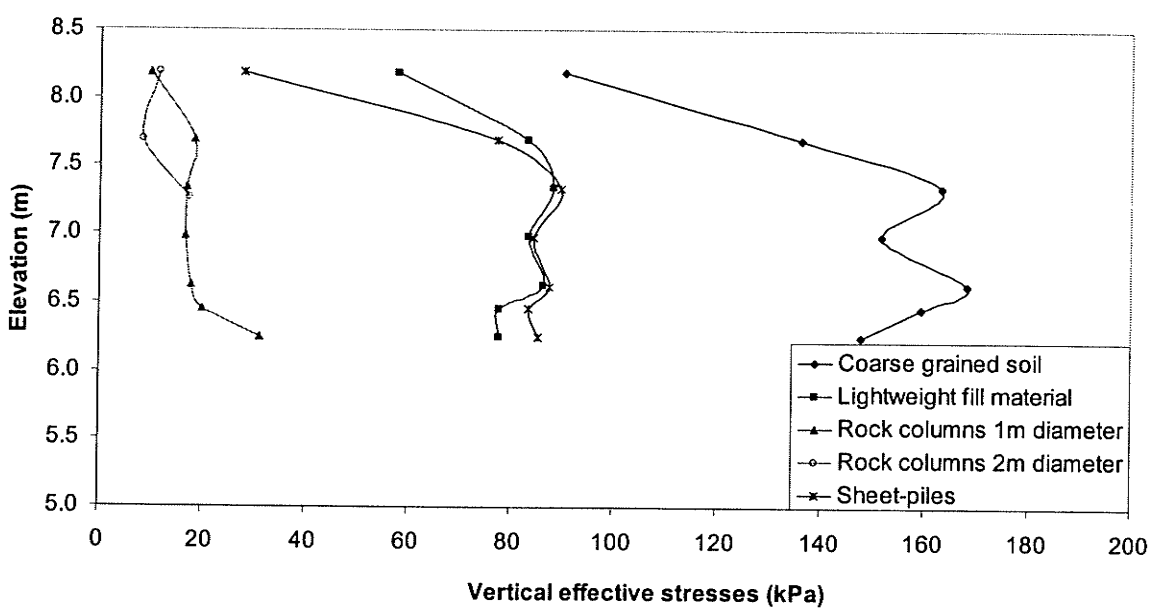

Figure 5-39: Vertical effective stresses versus depth for September 26-2015 for the different adaptation strategies

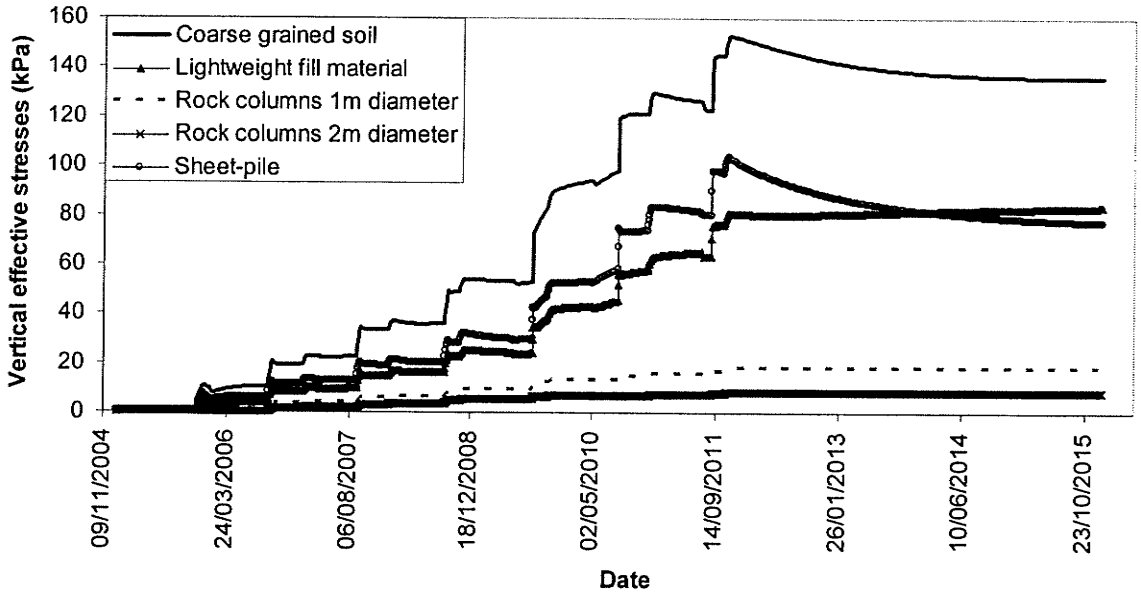

Figure 5-40: Vertical effective stresses versus time for a point 0.5 metres deep from the surface for the different adaptation strategies

Lightweight fill materials and sheet-piles also reduce the vertical effective stresses. Lightweight fill materials apply lower loads to the foundation soil while sheet-piles restrain the horizontal displacement at the toe of the embankment that induces failure in the foundation soil. The reduction in stresses for lightweight fill materials and sheet-piles ranges between 50 and 65 percent.

Figures 5-41, 5-42, and 5-43 show the settlements calculated for the different adaptation strategies for three periods of time for a point at the surface of the peat layer. The presence of permafrost at the early stages of the modeling help support the loads exerted from the embankment and low settlements are achieved (Figure 5-41). The lowest displacements are reached by using rockfill columns. Rockfill columns is the best technique that can be used to control the

vertical displacements. Lightweight fill materials perform better than sheet-piles to reduce the settlements of the foundation soil by exerting lower loads (Figures 5-41 and 5-42). Rockfill columns accelerate the consolidation due to the drainage capabilities and the displacements are almost negligible when total degradation of the permafrost is achieved (Figure 5-43).

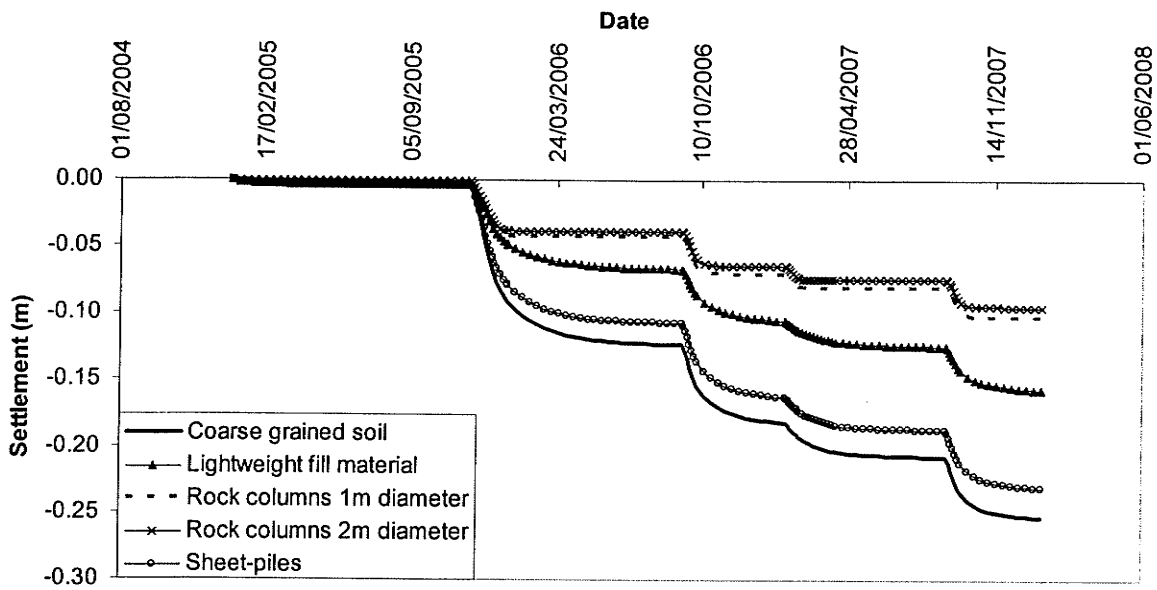

Figure 5-41: Total settlement reached for the models between January 2005 and January 2008 for the different adaptation strategies

Figure 5-44 shows the values of horizontal displacements for lightweight fill materials and sheet-piles compared to coarse-grained soils. Results from rockfill columns are not reported since they are negligible due to the higher stiffness provided by the rock columns and the small spacing between them.

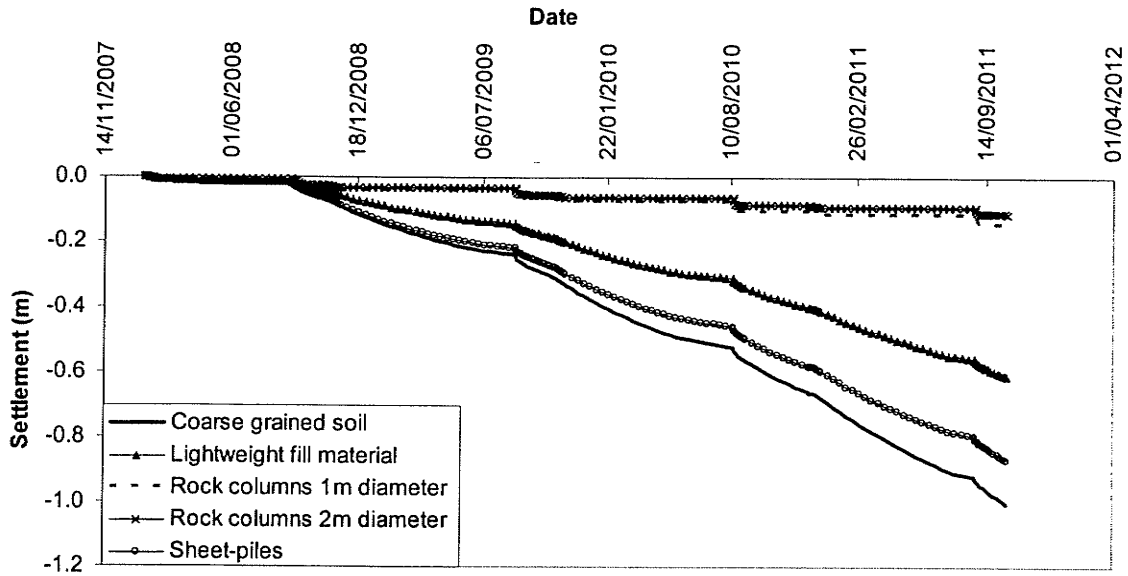

Figure 5-42: Total settlement reached for the models between January 2008 and October 2011 for the different adaptation strategies.

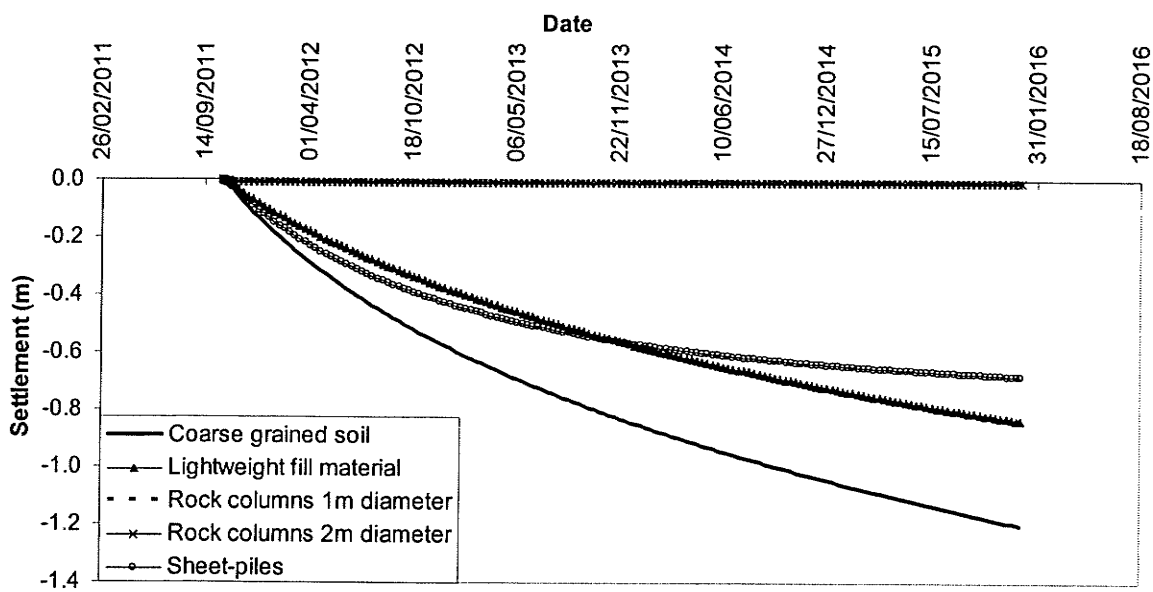

Figure 5-43: Total settlement reached for the model between October 2011 and December 2015 for the different adaptation strategies

Lightweight fill materials and sheet-piles reduce the horizontal displacements. However, the higher stiffness of the sheet-piles is more effective to reduce the horizontal displacements at the toe of the embankment.

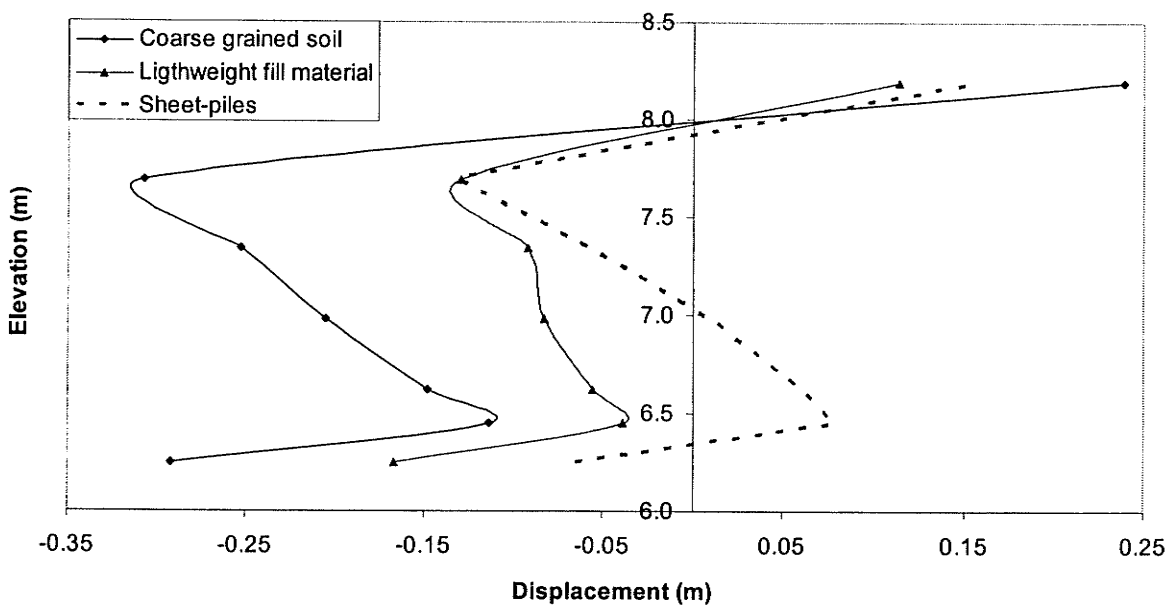

Figure 5-44: Maximum horizontal displacements at the toe of the embankment for lightweight and sheet-piles

The summary of results shows that rockfill columns are a suitable technique to be used in the Northern Regions when building road embankments. If they are designed to have drainage capabilities, the pore water pressures are dissipated, the effective stresses in the degrading permafrost are lower, and the settlements are reduced dramatically. Lightweight fill materials as well as sheet-piles also help reduce the vertical stresses; however, the settlements are still high and serviceability might be compromised.

The settlements with rockfill columns are close to 30 cm at the end of the modeling period. Geosynthetics were included in a new model to reduce the differential settlements between the rockfill columns and the surrounding degrading permafrost. The results were not satisfactory. The analysis showed that the rock columns also settled. Since the bedrock was not found in the field investigation, rockfill columns do not have support at the tip and the rockfill columns behave as a floating-type system resembling more the behaviour of a composite material that settles informingly. If the rockfill columns are founded on a stiffer material such as bedrock, they would settle less and the differential settlements may be prevented by using geosynthetics as bridge between caps of the rockfill columns.

Rockfill columns with drainage capabilities can be used in the construction of new road embankments if time is allowed to achieve consolidation before the operation of the highway. For existing embankments, rockfill columns can also be used; however, closing of the road is necessary during the construction activities.

Sheet-piles might be used to contain the lateral displacements at the toe of the embankment in existing embankments, but the model shows that some settlements may still occur. With sheet-piles, highways that require remedial measures may not necessarily be closed during their installations where work can be done at the toe of the embankments.

5.6 SUMMARY

The mesh used to carry out the stress-deformation analysis was also used to model the incorporation of adaptation strategies to mitigate the effects of thawing of the permafrost. This condition allowed comparison of results between the different models.

It was assumed that the incorporation of adaptation strategies will not modify the ground temperatures and consequently the permafrost conditions. For instance, lightweight fill materials have insulation capabilities, however, this conditions was neglected to reflect a worst case scenario. This implies that the same ground temperatures generated in the thermal model were used for every adaptation strategy analyzed.

Lightweight fill materials have lower specific unit weights, which transmit lower loads to the foundation soil. The effect is a reduction of the excess in pore water pressures, vertical effective stresses, shear stresses, and horizontal and vertical displacements. The trend of the values is similar to coarse-grained soils since only the specific unit weight changes in the model (it is reduced by 50%). The reduction in the values is close to 50% in every parameter being analyzed. The values of displacements are still high and the serviceability requirement might still be compromised.

Rockfill columns were designed to have drainage capabilities. The combination of high strength of the rock column and high hydraulic conductivity increase the bearing capacity of the foundation soil and the compressibility is reduced. In addition, provides a way to accelerate the consolidation and forces undesirable settlements to occur before or during the construction of the road embankment.

The design of the rockfill columns used in the modelling allowed a rapid dissipation of excess pore water pressures; decreased the vertical effective stresses by almost 90%; reduced the settlements by 75%; and provided a rapid consolidation time. However, the settlements are still high (close to 30 cm in total) since no stiff material was found to give support at the tip of rockfill columns (floating type). For that reason, the use of geosyntethics as bridges between column caps was found to be ineffective to reduce settlements since the rock columns and the clay behave as a composite material that settles uniformingly. In addition, the increase in rockfill columns diameter from 1 metre to 2 metres did not show major improvements.

Sheet-piles demonstrated to be effective in reducing the horizontal displacements at the toe of the embankment that produce failure in the foundation soil. The trend exhibited using this technique is very similar to the one exhibited by lightweight fill materials with improvement in values close to 50%. However, as for the case of lightweight fill materials the settlements are still high

and serviceability requirements may not be satisfied. Shear stresses increase along the sheet-piles because of the high stiffness of the steel sheet-piles.

The summary of results showed that rockfill columns are the most appropriate technique to be used to reduce stresses and displacements. However, rockfill columns must be that of an end bearing type. This technique is more appropriate for new embankments because the drainage capabilities create rapid consolidation of the foundation soil and these settlements can occur before the operation of road embankments. It also can be used in existing embankments, but traffic has to be interrupted to permit installation of these columns. For existing embankments, sheet-piles also reduce stresses and settlements, but the settlements are still high and serviceability requirements should be carefully assessed. The insulation properties of the lightweight fill materials might play an important role in preserving the permafrost underneath the embankment, which can lead to lower settlements in the foundation soil. This has not been taken into account in the evaluation of different adaptation strategies.

CHAPTER 6

CONCLUSIONS

6.1 THERMAL MODELLING

The thermal model reflected the ground temperature trends reasonably well with the results obtained from the thermistor data between 1996 and 1998. The simulated values tend to show a change in the temperature profile at the edge of the peat layer because of the very low thermal conductivity and high water content of the peat. Peat is considered to be a natural insulator that prevents the degradation of the permafrost. For a model that uses thermal properties mostly from the literature, it is considered sufficient to replicate the ground temperature and thus can be used for further analysis of the ground thermal regime.

The model indicates that degradation of the permafrost is more severe in the lower slope areas where the thickness of the embankment is at minimum and the transmission of heat from the surface is more intense. The climate warming trend model CGCM2, scenario A2, used in the 50 year simulation period proved to be suitable for the model since it matched the air temperature in the area quite reasonably well. This climate warming model has been used to study the ground thermal regime for 50 years period of climate warming modelling. With the

climate warming trend the modelling results indicate that permafrost cannot be preserved or its presence is sporadic underneath the embankment. The degradation begins at the toe and moves outwards with degradation as well below the embankment. The permafrost is preserved in a zone far from the influence of the embankment and where the thickness of the peat layer is larger.

Peat plays a very important role in the performance of the embankments in the northern regions by preventing the rapid degradation of the permafrost due to the construction of road embankments coupled with a climate warming trend. This combination modifies the thermal regime of the ground magnifying the degradation particularly in the discontinuous permafrost areas. Permafrost degradation increases dramatically after the first 10 years of modelling when the climate warming component is included in the analysis and the increase of warming trend is about 1°C.

The model results are suitable to delineate zones of frozen and unfrozen ground. This would then allow assigning of appropriate mechanical properties in the ground for stress-deformation analyses.

6.2 STRESS-DEFORMATION ANALYSIS

The frozen and unfrozen regions determined from the thermal model for the 50 year period incorporating climate warming conditions were used to build a new finite element mesh to predict the changes in stresses in the foundation soil

beneath road embankments as the permafrost degrades with time. The stress-deformation model allows the simulation of consolidation processes as the frozen soil is thawed. It should be noted that the actual thawing processes couple thermal and mechanical processes. Since the computer software used (Geo-Studio, 2004) does not couple the thermal and mechanical processes the challenge was to find a way to incorporate those two processes in one model.

The finite element mesh was built by overlapping regions of unfrozen and frozen ground. The model simulation was achieved by changing soil properties and controlling convergence of the results due to the high sensitivity of the model to the change in soil properties. It is noted that this procedure is highly idealized. However, it was assumed sufficient to help understand the processes of deformation and stress changes due to the progressive thawing of permafrost.

The focus of discussion in this study is at the toe of the embankment since it is the area where the thawing process starts. Adjacent areas are not considered in the analysis because the higher stiffness of the permafrost generates higher effective stresses and lower displacements. The stresses must be lowered during the freezing season when the permafrost melts. This is a process that was not captured well in the numerical model due to the way the model is set up and the restrictions with the computer software. Complete degradation of the permafrost was found to occur in 2011 when using the climate warming model described earlier. After 2011, the modelling proceeds as in

consolidation problems that used sequential modelling of stress-deformation and seepage analysis employed in the Geo-Studio computer program.

The model reflected the freeze and thawing seasons very well. During thawing excess pore water pressures are generated, high vertical effective stresses are developed, and large horizontal and vertical displacements are found. During freezing, the excess pore water pressures are lowered, the stresses remain constant, and the vertical and horizontal displacements diminished. These variables tend to reach constant values when complete degradation of permafrost is achieved.

Pore water pressures increase with degradation of the permafrost. Maximum values are reached when complete degradation of the permafrost is achieved; after degradation excess pore water pressures decrease as they dissipate.

Similar behaviour is found with the effective vertical stresses, they increase with increase in permafrost degradation reaching constant values when complete thawing is achieved. The increase is higher at the surface; however, with time the effective stresses increase with depth.

Vertical displacements also increase with increase in the degradation of the permafrost. The maximum values of settlement of 2.5 metres found in the

model. Thickness of 2.4 metres of asphalt used to fill dips was found as observed during the 2002 studies (AMEC, 2002).

The horizontal displacements at the toe of the embankment are restrained by the stiffer zones of yet frozen ground outside the embankment footprint. However, small displacements are found at the early stages of the modelling. With degradation of permafrost the displacements to go inwards beneath the embankment as the permafrost in this zone has been degraded. This will result to an overturning type of failure along the shoulder of the embankments which were also observed in road embankments on degrading permafrost.

6.3 ADAPTATION STRATEGIES

The finite element model used to carry out the stress-deformation analysis was also used to simulate the performance of road embankments incorporating various ground improvement techniques as adaptation strategies to mitigate the effects of thawing of the permafrost.

It was assumed in this study that the incorporation of adaptation strategies will not modify the ground temperatures and consequently the permafrost conditions. For instance, lightweight fill materials have insulation capabilities; however, this has not been included in the analysis to reflect a worst case scenario.

Lightweight fill materials have lower specific unit weights, which transmit lower loads to the foundation soil. The effect is a reduction of excess in pore water pressures, vertical effective stresses, and horizontal and vertical displacements. The trend of the values is similar to that of embankment fills using coarse-grained soils since only the specific unit weight changes in the model (it was reduced by 50%). The reduction in deformations, stresses, and pore pressures was close to 50%. The values of displacements can still be high and serviceability aspects may still be an issue.

Rockfill columns were designed to have drainage capabilities. The combination of high strength of the rock column and high hydraulic conductivity increase the bearing capacity of the foundation soil and reduce the compressibility. In addition, the drainage capabilities of rockfill columns help accelerate the consolidation process. However, the presence of water in the rock columns can accelerate permafrost degradation.

The design of the rock columns used in the modelling allowed a rapid dissipation of excess pore water pressures; decreased the vertical effective stresses in the foundation soil by almost 90%; reduced the settlements by 75%; and provided a rapid consolidation time. However, the vertical displacements in the foundation soil are still high (close to 30 cm in total). This was attributed to the fact that the columns were installed as floating columns. The vertical displacements may be further reduced by installing the rockfill columns as end

bearing piles and the provision of geosynthetic reinforcements to transfer the load of the fill in between of the rockfill columns.

Sheet-piles demonstrated to be effective to reduce the horizontal displacements at the toe of the embankment. The technique exhibited results that are very similar to the ones exhibited by lightweight fill materials with improvement in values close to 50%. However, as for the case of lightweight fill materials the settlements are still high.

In summary, the results of the study showed that rockfill columns are the most appropriate technique to be used to reduce stresses and displacements. However, rockfill columns must be properly supported at the tip to further reduce the settlements. This technique is more appropriate for new embankments because the drainage capabilities produce rapid consolidation of the soil and settlements can occur before or during the construction of road embankments. It also can be used in existing embankments, but traffic has to be interrupted during installations. For existing embankments, sheet-piles also reduce stresses and settlements, but the settlements are still high and serviceability aspects should be addressed.

CHAPTER 7

OPPORTUNITIES FOR FURTHER RESEARCH

7.1 THERMAL MODELLING

Investigate the thermal properties of the soils by conducting a complete and thorough site investigation and laboratory testing. Peat plays a very important role in preventing the degradation of the permafrost. Its thermal properties and its influence in the foundation soil must be considered relevant when designing embankments in the Northern Regions. Also, the effect of ponding water on the side slopes must be addressed to predict the changes in the rate of permafrost degradation.

The model CGCM2 was used to account for the climate warming trend. A comparison with other climate model is useful. A climate model is currently being developed at the University of Manitoba by Mr. Bhuiyan as part of his Master's thesis. The results of his studies will soon become available and the climate model being developed could potentially be used to compare the results from the CGCM2 model used in this study.

7.2 STRESS-DEFORMATION ANALYSIS

Since the mechanical properties for the stress-deformation analysis were taken from those available in the literature, it is very important to investigate the mechanical properties of the soils found in Northern Manitoba. A complete field and laboratory investigation will help in establishing boundary conditions and fine tuning of modelling results.

The model assumed conditions that led to the worst case scenario. The stresses are cumulative over the model period that may lead to over prediction of settlements, excess pore pressures and stresses. Since the change in stresses due to freeze-thawing cycles is not known, the model can be complemented using different conditions. This can be achieved by running a new model that can capture the release of stresses every freezing season when embankment unloading occurs.

7.3 ADAPTATION STRATEGIES

The incorporation of adaptation strategies may modify the rate and amount of degradation of permafrost. It is suggested to run thermal models for every adaptation strategy to analyze their influence such as either degrading or preserving the permafrost.

The adaptation strategies that have been studied followed the premise that the permafrost beneath the road embankments would eventually degrade when a climate warming trend is imposed. It might be valuable to study how these adaptation strategies (or other methods not necessarily included in this study) accelerate the degradation of permafrost. This will allow maintenance of roads before their operation and deal with ways to stabilize the foundation soil. Various ground improvements have already been used successfully in stabilizing soft and compressible ground.

The rockfill columns showed to be insensitive to the change in diameter of the columns when the spacing is constant. The analysis could be complemented by analyzing models with constant diameter and different spacing and variable diameter and variable spacing to find the most suitable condition. In addition, the length of the rockfill columns may be increased up to a stiffer stratum to further reduce the settlements.

Sheet-piles were analyzed at constant thickness. Increasing both the thickness and thus stiffness of steel might lead to better results.

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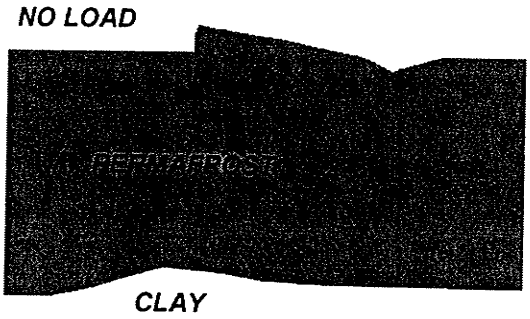
APPENDIX A

CONSECUTIVE SET OF MODELS FOR STRESS-DEFORMATION ANALYSIS



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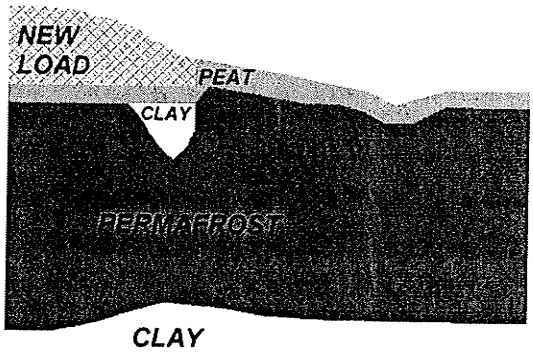

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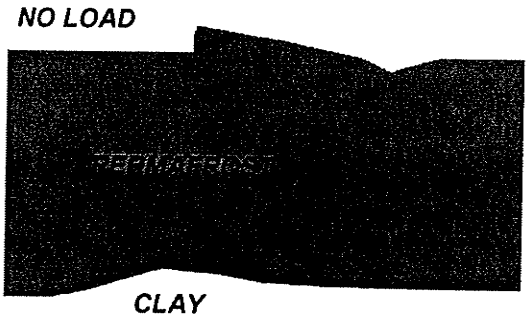

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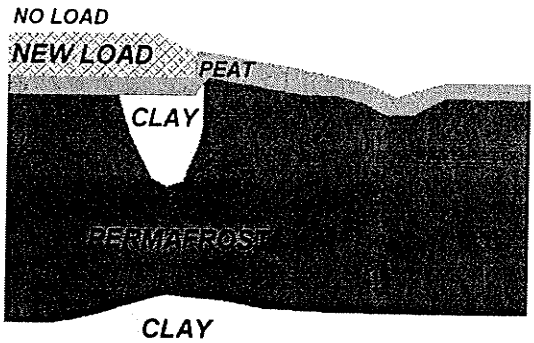

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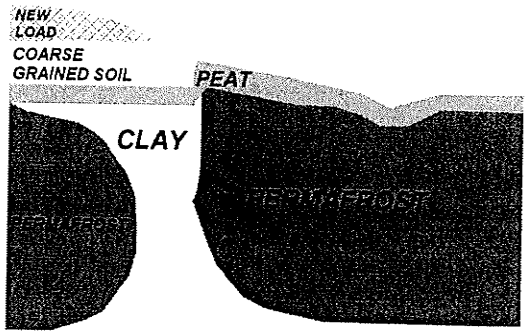

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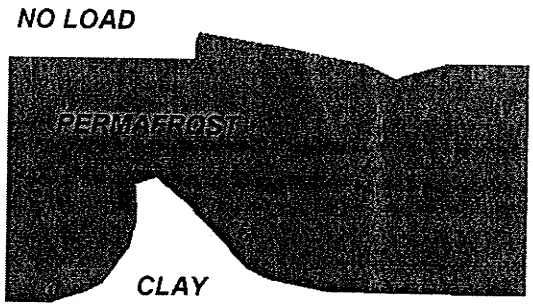

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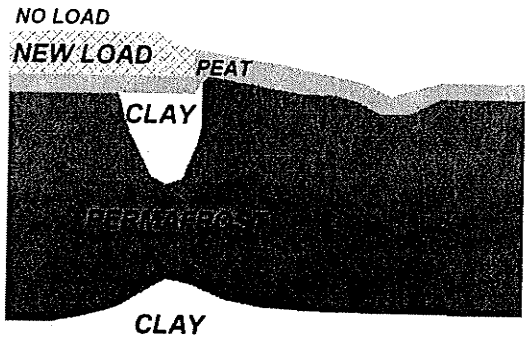

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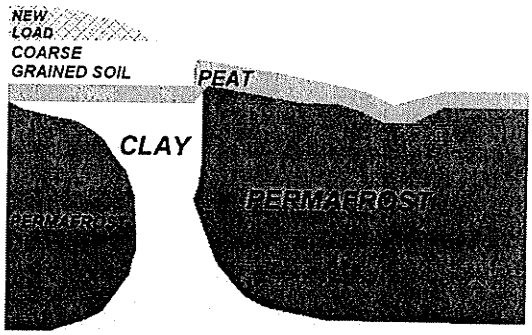

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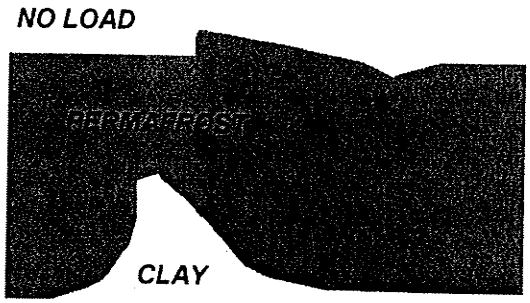

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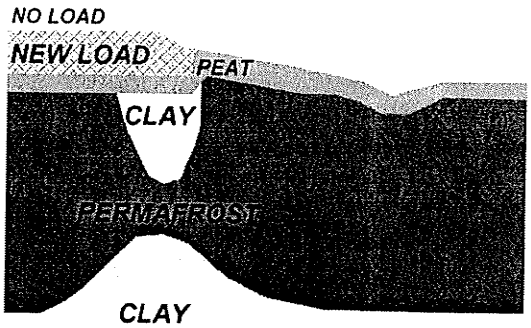

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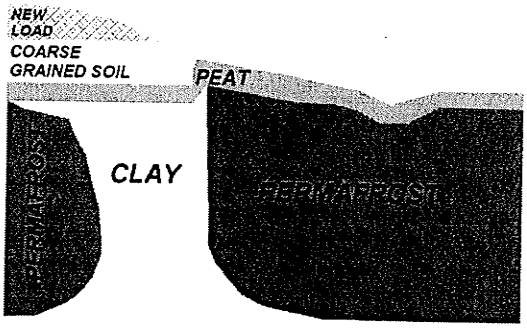

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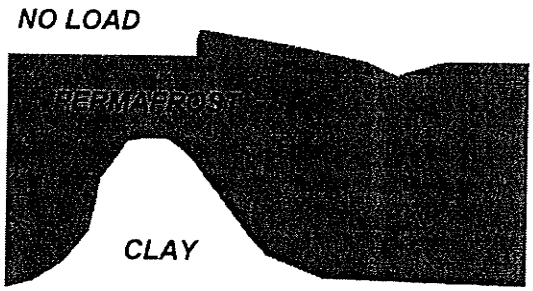

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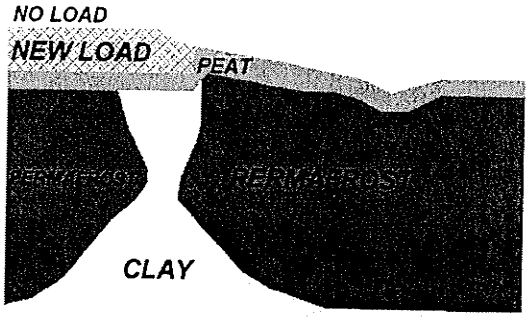

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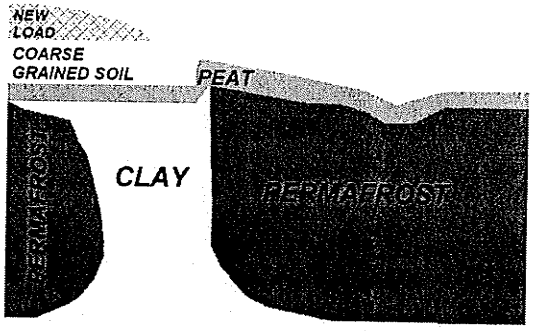

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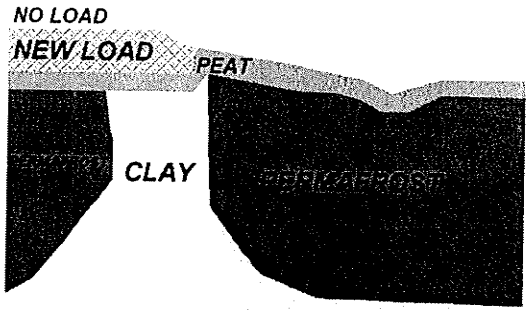

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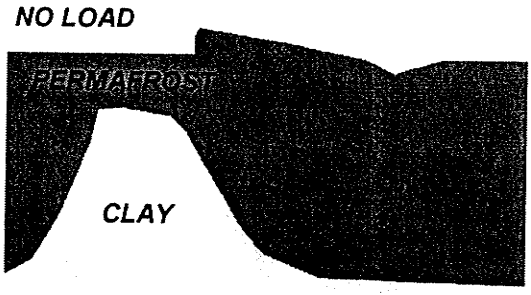

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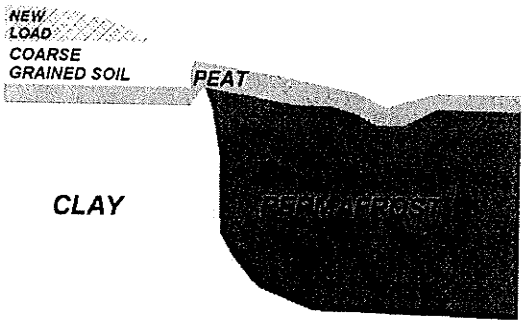
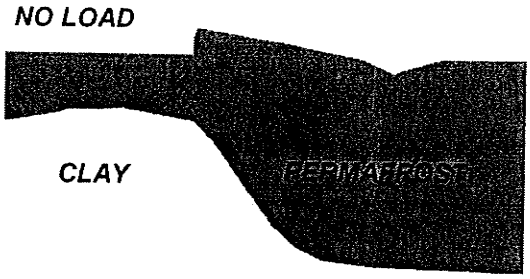
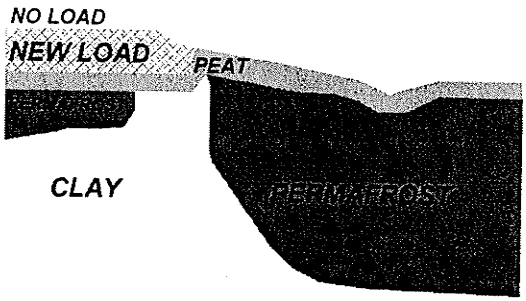


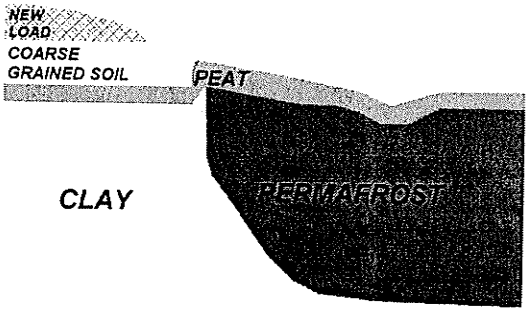
December 11-2010 to July 19-2011



July 19-2011 to August 28-2011



August 18-2011 to October 17-2011



October 17-2011 to December 31-2015

APPENDIX B

DERIVATION OF THEORY OF CONSOLIDATION IN THAWING SOILS

B.1 THEORY OF CONSOLIDATION IN THAWING SOILS

The physics of consolidation of a thawing soil results in a combination of the theories of heat conduction and of linear consolidation of a compressible soil (Morgenstern and Nixon, 1971). In the development of the theory, the permafrost table forms the lower boundary of the problem since the frozen soil does not transmit pore water pressures or deforms in any way. The permafrost table moves with time because the change in thermal conditions in the soil, then the consolidation is governed by a moving boundary. The thawed soil is considered saturated and the excess pore water pressures are generated by a combination of self-weight and external loads. This theory has not yet been implemented in the current Geo-Studio software package; such that there is still no communication between separate programs TEMP/W (thermal modelling), SIGMA/W (stress-deformation modelling), and SEEP/W (seepage or flow modelling). So far, only SIGMA/W and SEEP/W can be used sequentially as in the case of modelling the consolidation process. However, this theory is described here for completeness of information related to this study.

B.2 THEORY OF HEAT CONDUCTION

The case of melting or freezing involves a physical phenomenon in which one substance changes into another with emission or absorption of heat. This generates a moving surface that separates the frozen and unfrozen zones. The thermal properties of these two zones are different and the determination of their rate of change is difficult to predict.

If a region is frozen by removal of heat by a constant temperature $-T_{i1}$ the remaining unfrozen region has a temperature T_{i2}. When the permafrost table moves a distance dX a quantity of heat $L\rho_1 dX$ must be removed by conduction and the system satisfies Equation [B-1]:

[B-1]
$$K_1 \frac{\partial T_1}{\partial x} - K_2 \frac{\partial T_2}{\partial x_2} = L\rho_1 \frac{\partial X}{\partial t}$$

Where:

K = thermal conductivities for frozen and unfrozen conditions

ρ = density of the material

L = latent heat

X = location of the permafrost table

The solution of the problem of a material whose surface is kept at zero temperature under an initial temperature T_i is given by Equation [B-2], which depends on the thermal diffusivity, κ, of the material:

[B-2]
$$T = T_i erf\left\{\frac{x}{2\sqrt{(\kappa t)}}\right\}$$

Solving Equation [B-1] by using the same approach as Equation [B-2] gives Equations [B-3] and [B-4], where A and B are constants that result from the solving process:

[B-3]
$$T_1 = -T_{i1} + Aerf\frac{x}{2\sqrt{(\kappa_1 t)}}$$

[B-4]
$$T_2 = -T_{i2} + Berfc\frac{x}{2\sqrt{(\kappa_2 t)}}$$

The temperature at the boundary of the unfrozen zone, T_2, and frozen zone, T_1, can be assumed to be approximately zero at $x = X$, then Equations [B-3] and [B-4] reduce to Equations [B-5] and [B-6]:

[B-5]
$$Aerf\frac{X}{2\sqrt{(\kappa_1 t)}} = T_{i1}$$

[B-6]
$$Berfc\frac{X}{2\sqrt{(\kappa_2 t)}} = T_{i2}$$

Equations [B-5] and [B-6] have to satisfy all the values of t, so *X* must be proportional to $t^{\frac{1}{2}}$ and lead to Equation [B-7]

[B-7] $$X = \alpha t^{\frac{1}{2}}$$

Where α is a constant to be determined that depends on the type of material. Equations [B-3] and [B-4] can be replaced in Equation [B-1] to conduce to Equation [B-8]

[B-8] $$\frac{T_1 K_1 e^{\frac{-\alpha^2}{4\kappa_1}}}{\sqrt{(\pi\kappa_1) erf\left(\frac{\alpha}{2\sqrt{\kappa_1}}\right)}} - \frac{T_2 K_2 e^{\frac{-\alpha^2}{4\kappa_2}}}{\sqrt{(\pi\kappa_2) erfc\left(\frac{\alpha}{2\sqrt{\kappa_2}}\right)}} = \frac{L\rho_1\alpha}{2}$$

An approximated value of α can be achieved by replacing the error, complementary error and exponential functions by one or two terms of their power series to give a more manageable equation.

[B-9] $$\alpha^2 = \frac{2K_1T_1}{L\rho_1}$$

B.3 LINEAR CONSOLIDATION THEORY

When a load is applied to a saturated soil, all of the applied stress is supported initially by the pore water. In this primary stage the increase in

effective stresses, σ', are zero. If drainage of the excess pore water pressures is allowed, the initial pore water pressures decrease and soil settlement increases with time. In this stage, the change in effective stresses, Δσ', is given by the increase in total stresses, Δσ, minus the change in excess pore water pressures, Δu. With time, the change in volume and the change in excess pore water pressures approach to zero and the change in effective stresses, Δσ', is given by the change in total stresses, Δσ.

The derivation of the theory of consolidation is valid when the following assumptions are satisfied:

1. The soil is saturated, isotropic, and homogeneous
2. Darcy's law is valid
3. Flow only occurs vertically
4. The strains are small

The inflow of water in a quadrilateral element of area *dA* is $q_v dA$ and the outflow over the elemental thickness *dz* is $q_v + (\partial q_v / \partial z) dz dA$. Where q_v is the one dimensional flow of water, which is given by Darcy's law:

[B-10]
$$q_v = Ak_z i = Ak_z \frac{\partial h}{\partial z}$$

Where:

k_z = coefficient of permeability in the vertical direction

i = hydraulic gradient or change in head $\partial h / \partial z$ in the vertical direction

The change in flow is then $(\partial q_v / \partial z) dzdA$. The rate of change in volume of water expelled that corresponds to the rate of change of volume of the soil must equal the change in flow leading to Equation [B-11]:

[B-11]
$$\frac{\partial V}{\partial t} = \frac{\partial q_v}{\partial z} dzdA$$

The change in volume can also be expressed in terms of the volumetric strain of the soil, ε_p, by:

[B-12]
$$\partial V = \frac{\partial e}{1 + e_o} dzdA = m_v \partial \sigma' dzdA = m_v \partial u dzdA$$

Where:

m_v = modulus of volume compressibility

e = void ratio

Substitution of the Equation [B-12] into Equation [B-11] leads to:

[B-13]
$$\frac{\partial q_v}{\partial z} = \frac{\partial u}{\partial t} m_v$$

If Darcy's law is partially differentiated by *z* Equation [B-10] converts to:

[B-14]
$$\frac{\partial q_v}{\partial z} = k_z \frac{\partial^2 h}{\partial z^2}$$

The pore water pressure is proportional to the hydraulic head, *h*, the specific unit weight of water, γ_w, and is given by:

[B-15]
$$u = h\gamma_w$$

If Equation [B-15] is partially differentiated by *z* and replaced in Equation [B-14], Equation [B-16] is obtained.

[B-16]
$$\frac{\partial q_v}{\partial z} = \frac{k_z}{\gamma_w} \frac{\partial^2 u}{\partial z^2}$$

Equations [B-13] and [B-16] can be equated to get the solution for the problem of one-dimensional consolidation of soils.

[B-17]
$$\frac{\partial u}{\partial t} = \frac{k_z}{m_v \gamma_w} \frac{\partial^2 u}{\partial z^2}$$

Equation [B-17] can be expressed in terms of the coefficient of consolidation, c_v, which can replace the terms k_v, m_v and γ_w.

[B-18]
$$\frac{\partial u}{\partial t} = c_v \frac{\partial^2 u}{\partial z^2}$$

B.4 COUPLED CONSOLIDATION THEORY OF THAWING SOILS

As stated previously the permafrost table forms the lower boundary of the region of interest. Consolidation occurs because there is a change in pore water pressures and water is expelled out from the system. The excess pore water pressures are generated due to an external load or self-weight of the soil mass. The movement of the permafrost table or thawing plane was determined by Equation [B-7].

In the thawing region it is assumed that the soil follows the assumptions that are valid in deriving the theory of consolidation and hence Equation [B-18] is applicable. An expression for effective stress, σ', can be found by combining Equations [B-11] and [B-12] and express the results in terms of *X* given by Equation [B-7]:

[B-19]
$$\Delta\sigma' = \frac{c_v \dfrac{\partial u(X,t)}{\partial x}}{\dfrac{dX}{dt}}$$

From Figure B-1 is possible to determine equations for the total stresses and pore water pressures at plane x = *X*:

[B-20] $$\sigma(X,t) = P_o + \gamma X$$

[B-21] $$P_w(X,t) = u(X,t) + \gamma_w X$$

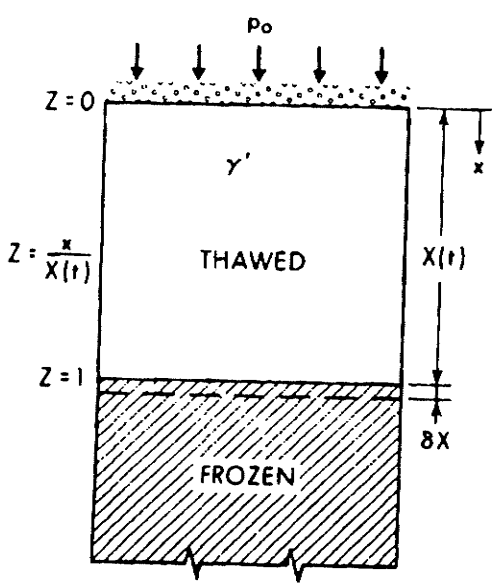
Where:

P_o = stress applied to the surface

γ = bulk density of the soil

Therefore the effective stress is given by:

[B-22] $$\sigma'(X,t) = P_o + \gamma' X - u(X,t)$$



One-dimensional thaw consolidation (after Andersland and Ladanyi, 2004).

Where γ' denotes the submerged density of the soil. $\Delta\sigma'$, the change in effective stress with respect to the initial stress, σ_o', can be assumed to be equal to the effective stress given in Equation [B-22] if the initial effective stress, σ_o', is considered to be small enough and taken equal to zero. In that case $\Delta\sigma'$ can be expressed according to Equation [B-23]:

[B-23]
$$\Delta\sigma' = P_o + \gamma' X - u(X,t)$$

Equating Equations [B-19] and [B-23] leads to an expression of the form:

[B-24]
$$P_o + \gamma' X - u(X,t) = \frac{c_v \frac{\partial u}{\partial x}(X,t)}{\frac{dX}{dt}}$$

Employing the same approach used to determine an analytical solution for the theory of heat conduction, an analytical solution for the theory of thawing consolidation might be given by:

[B-25]
$$u(X,t) = \frac{P_o}{erf(R) + \frac{e^{-R^2}}{\sqrt{\pi R}}} \times erf\left(\frac{x}{2\sqrt{c_v t}}\right) + \frac{\gamma' x}{1 + \frac{1}{2R^2}}$$

Where the term R is the thawing consolidation ration given by:

[B-26]
$$R = \frac{\alpha}{2\sqrt{c_v}}$$